Case Study: Retrofitting Large Bridge Piers on the Nakdong River, South Korea

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Abstract: The Gupo Bridge crosses the Nakdong River near the city of Busan, South Korea. During Typhoon Maemi in 2003, the old Gupo Bridge collapsed due to excessive pier scour. More recently, the highway construction on the left-bank floodplain required right-bank channel widening to restore the channel flood-carrying capacity. This 7 m deep floodplain excavation is expected to cause significant local scour around the 8–10 m wide and 3 m thick spread footings of Piers 11 and 12 of the Subway Bridge and Piers 15 and 16 of the Gupo Bridge. Three design options are examined for retrofitting floodplain bridge piers with massive spread footings. A solution with sheet piles and riprap was recommended in 2006 as the most appropriate design, but Plan III with a conical riprap structure around the footings was ultimately constructed in 2007 for economic reasons. Laboratory experiments also highlight the need to place gravel and synthetic filters under the designed riprap.

DOI: 10.1061/(ASCE)0733-9429(2008)134:11(1639)

CE Database subject headings: Bridges, piers; Scour; Bridge failure; Piles; Riprap; Rehabilitation; Bridge design; Case reports.

Introduction

The Nakdong River is located in the southeastern region of South Korea and flows 510 km from the Taebaek Mountains to the East Sea (Fig. 1). In 1987, the Nakdong River Estuary Barrage (NREB) was built near the river mouth to prevent saltwater intrusion in the lower 40 km of the river. The Old Gupo Bridge, the New Highway Gupo Bridge (Gupo Bridge), and the Subway Bridge are located 15 km upstream of the NREB on the Lower Nakdong River. These bridges connect the city of Busan to the southwestern part of South Korea. The Nakdong River has a drainage area of about 23,384 km^2 with frequent typhoons and floods from June to September. On September 12, 2003, Typhoon Maemi was the worst typhoon to hit South Korea in a decade. The resulting flood caused the Old Gupo Bridge to partially collapse after the loss of a bridge pier to excessive pier scour and high flow velocities (Ji and Julien 2005).

Pier scour has been extensively studied in many hydraulic laboratories for more than 100 years. In the United States, the Federal Highway Administration (FHwA) has identified more than 10,000 scour-critical bridges and almost 100,000 bridges with unknown foundations (Lagasse et al. 1997). Wardhana and Hadipriono (2003) collected and analyzed 503 cases of bridge failures from 1989 to 2000 and found that the leading cause of bridge failure relates to scour during floods. Several equations are available to estimate the depth of pier scour. Melville and Coleman (2000) reviewed and summarized some of the better-known and recent equations. Standard procedures can be found in Breusers and Raudkivi (1991), Hoffmans and Verheij (1997), USACE (1994), and the Federal Highway Administration [e.g., HEC-18 by Richardson and Davis (1995) and Richardson et al. (2001)]. Julien (2002) also reviews pier scour in the broader context of general scour, contraction scour, abutment scour, and pier scour. Recent contributions to the abundant literature on pier scour include Mia and Nago (2003), Chiew (2004), Sheppard et al. (2004), Chang et al. (2004), Ataie-Ashtiani and Beheshti (2006), and Sheppard and Miller (2006).

The rapid urban development of Busan City promoted the construction of the Dadae Harbor Highway on the left-bank floodplain of the Nakdong River. Accordingly, the excavation of the right-bank floodplain should ensure sufficient flood-conveyance capacity of the Nakdong River during typhoons. The piers on the floodplain were designed with massive footings and did not experience any local scour during Maemi. The proposed excavation, however, would expose these large pier footings and underlying piles. This case study presents unique conditions well beyond the scope of standard methods. The design of retrofitting countermeasures requires innovative thinking to seek ways to protect against pier scour around these massive spread footings. This paper examines three design options, one of which was selected and constructed.

Background

Site Description

Downstream of the confluence with the Milyang River near Samrangjin, the Lower Nakdong River has a very mild bed slope of approximately 10–20 cm/km. The design flood discharge with a

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Note. Discussion open until April 1, 2009. Separate discussions must be submitted for individual papers. The manuscript for this paper was submitted for review and possible publication on April 10, 2007; approved on April 21, 2008. This paper is part of the ***Journal of Hydraulic Engineering***, Vol. 134, No. 11, November 1, 2008. ©ASCE, ISSN 0733-9429/2008/11-1639–1650/$25.00.

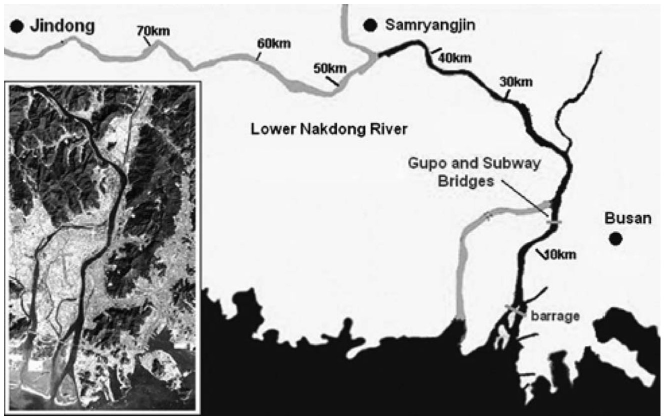


Fig. 1. Nakdong River Basin and Lower Nakdong River (Source: Korea Water Resources Corporation 2003)

return period of 200 years, estimated at 19,370 m^3/s, was used for the design of scour protection at Gupo Bridge. The corresponding design flow velocity 2.26 m/s and flow depth 6.62 m were recommended by the city of Busan (unpublished report, 2005) based on one-dimensional numerical modeling results. The median grain diameter of the bed material of the Nakdong River is 0.25 mm at the Gupo Bridge. The riverbed is mainly fine sand throughout the 40 km reach of the Lower Nakdong River downstream of Samryangjin (Ji 2006).

The mean annual precipitation of the Nakdong River basin is 1,186 mm and the mean annual temperature ranges from 12 to 16 °C. In South Korea, the average annual frequency of typhoons over a period of 30 years from 1971 to 2000 was 26.7 typhoons per year. Typhoon Maemi on September 12, 2003 was the worst typhoon to hit South Korea in more than a decade. It caused widespread devastation throughout the southeastern part of the Korean Peninsula and particularly hit the populated areas of the Nakdong River Basin and the port of Busan City. Typhoon Maemi caused extensive damage in the Lower Nakdong River from an extremely flashy hydrograph from over 400 mm of intense rainfall precipitation combined with a severe typhoon surge. The water level recorded at the Gupo Bridge reached a maximum elevation of 5.06 m on September 12, and the peak discharge reached 13,000 m^3/s on September 14. Unfortunately, as shown in Fig. 2, part of the 1.06 km long Old Gupo Bridge collapsed on September 14 after the loss of the 19th Bridge pier to local scour (Ji and Julien 2005). The pier collapse is attributed to high flow velocities and bridge pier scour in excess of 6 m as shown in Fig. 3. The results of calculations using the modified CSU equation (Richardson et al. 2001) indicated 4.9 m of scour depth around the reinforced steel piles and 6.7 m of scour around the original timber piles. The field measurements in Fig. 3 were in very good agreement with these calculations.

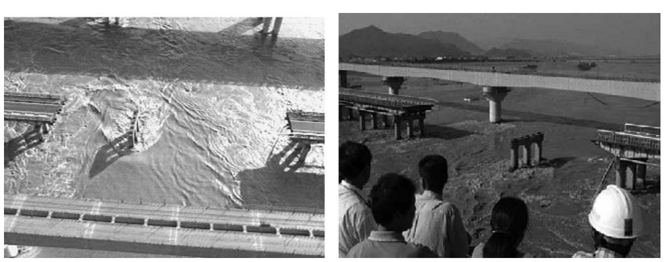


Fig. 2. Old Gupo Bridge failure after Typhoon Maemi (Source: *Yunhap News* 2003)

Geometry of Bridge Piers

Due to the ongoing Dadae Harbor Highway construction on the left bank of the Nakdong River in the vicinity of the Gupo and Subway Bridges shown in Fig. 4, excavation of the right-bank floodplain has been considered to ensure adequate flood conveyance. Accordingly, Piers 11 and 12 of the Subway Bridge and Piers 15 and 16 of the Gupo Bridge would be adversely affected by 7 m of excavation between the top of the concrete footing and the riverbed. The general layout of these four piers (Piers 11, 12, 15, and 16) is shown in Fig. 5. The widths of concrete spread footings range from 8 to 10.2 m with 2.5–3 m thickness. At this site, the proposed excavation would expose the piles under the footing and this is a major design concern.

Three Design Plans

Three design plans for retrofitting and protecting the Gupo and Subway Bridges' piers were proposed. Fig. 6 shows the general layout of the three option plans. Plan I would protect the bridge piers with vertical sheet piles and riprap. Plan II had a narrow wall caisson with grouting below the original foundation. Plan III had a gently sloping conical structure around the footing with riprap protection. This third plan first came to mind for its natural simplicity and possible reduced cost, but different plans were examined because of the reduced cross-sectional area and possible navigation problems between the piers. Plans I and II considerably increased the open cross-sectional area between the piers and decreased navigation concerns in the vicinity of the piers. Advantages and disadvantages of the three designs are listed in Table 1. Plan II was eliminated from further consideration because of possible stability problems during construction. Plan III provided adequate stability for the piers because the supporting piles are never exposed or disturbed during retrofitting. Although Plan III is considered as stable and economical, its design could possibly cause navigation problems and would decrease the flood-carrying capacity of the river at the bridge crossing. In view of the added cross-section opening and ease of navigation, Plan I was recommended in 2006.

Recommended Design

Scour Depth and Scour-Hole Geometry

The recommended Plan I for the Gupo and Subway Bridges' piers used sheet piles and riprap to protect the piers. The pier-scour depth (y_s) and scour-hole geometry (width w_s) of the retrofitted piers enclosed by sheet piles were calculated using different equations and the results are summarized in Table 2. Several equations selected by Melville and Coleman (2000) and FHwA's HEC-18 were used to calculate pier-scour depths for the Gupo and Subway Bridges' piers. The equations of Melville (1997), Ansari and Qadar (1994), and Neill (1973) resulted in relatively deeper scour depths than the equations from FHwA HEC-18 and Breusers and Raudkivi (1991). Richardson et al. (2001) indicate that existing equations, including the CSU equation, overestimate scour depth for wide piers in shallow flows. The Gupo Bridge case satisfies

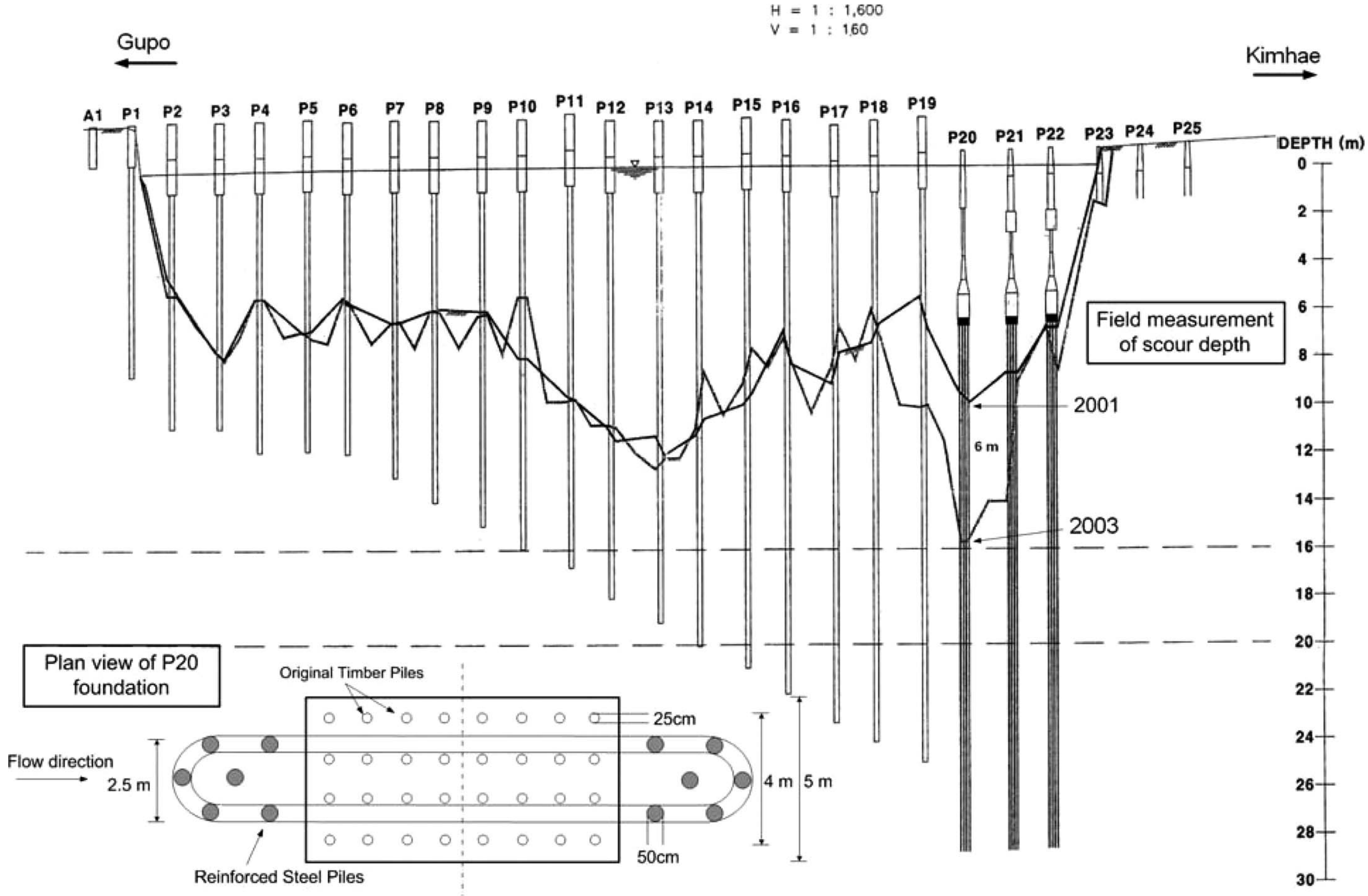


Fig. 3. Cross sections surveyed in 2001 and 2003 (after Typhoon Maemi) and plan view of P20 at the Old Gupo Bridge

the criteria for shallow flows: (1) ratio of flow depth y to pier width b less than 0.8; (2) ratio of pier width to the median diameter of the bed material d_{50} greater than 50; and (3) low Froude number corresponding to subcritical flow. The FHwA HEC-18 equation, however, contains a correction factor K_w for shallow flow condition. Therefore, the predicted scour depth resulting from the FHwA HEC-18 equation was selected for the Gupo and Subway Bridges' piers. The width of the pier-scour hole was then estimated from the calculated scour depth. Richardson et al. (2001) suggest 2.0 y_s for practical application, which is also used for the Gupo and Subway Bridges' cases. The results of the width of the scour hole are presented in Table 3 and range from 19 to 21 m.

Riprap Protection and Filter Design

Pier protection against local scour can be generally classified into two methods: (1) armoring methods such as riprap, tetrapods, tetrahedrons, grout-filled mats, gabions, mattresses, cable-tied blocks; and (2) flow changing methods such as sacrificial piles, Iowa vanes, and flow deflectors. The riprap protection method was selected for the pier retrofitting of the Gupo and Subway Bridges.

The riprap layer is the most widely used method to protect bridge piers against scour. Extensive studies include laboratory experiments and field measurements on the protection against pier scour. Also, many equations for sizing riprap and protecting bridge piers against scour have been proposed. Melville and Coleman (2000) compared the published equations and concluded that the Parola (1993, 1995), and Lauchlan (1999) equations lead to conservatively large riprap compared to other equations. For the pier protection design of the Gupo and Subway Bridges, the Parola (1993, 1995), Richardson and Davis (1995), and Lauchlan (1999) equations are considered. The calculated riprap sizes placed around sheet piled piers are listed in Table 4. The applied riprap size (50 cm) for the Gupo Bridge is slightly larger than the averaged value of the results calculated by the equations of Parola (1993, 1995), Richardson and Davis (1995), and Lauchlan (1999).

The U.S. Army Corps of Engineers method (USACE 1981) for riprap gradation is used and the results of the riprap gradation are shown in Fig. 7. The method defined the riprap layer thickness as a function of the median size of riprap stone d_{r50}, and the riprap size of which 100% of sample is finer d_{r100}. Accordingly, the riprap thickness should not be: (1) less than 12 in. or 30 cm; (2) less than the upper limit of d_{r100}; or (3) less than 1.5 times the diameter of the upper limit d_{r50}. The riprap thicknesses obtained from these three criteria are, respectively, 30, 100, and 75 cm. For safety considerations, a riprap thickness of 1.5 m has been recommended in this case.

In addition, filters are important to drain water between the riprap layer and bed layer without carrying out soil particles. Both stone and synthetic filters were considered for the retrofitting design of the Gupo and Subway Bridges' piers, and the calculation method of stone filters followed that for filter design of the riverbank riprap revetment (Julien 2002). The criteria for stone filters

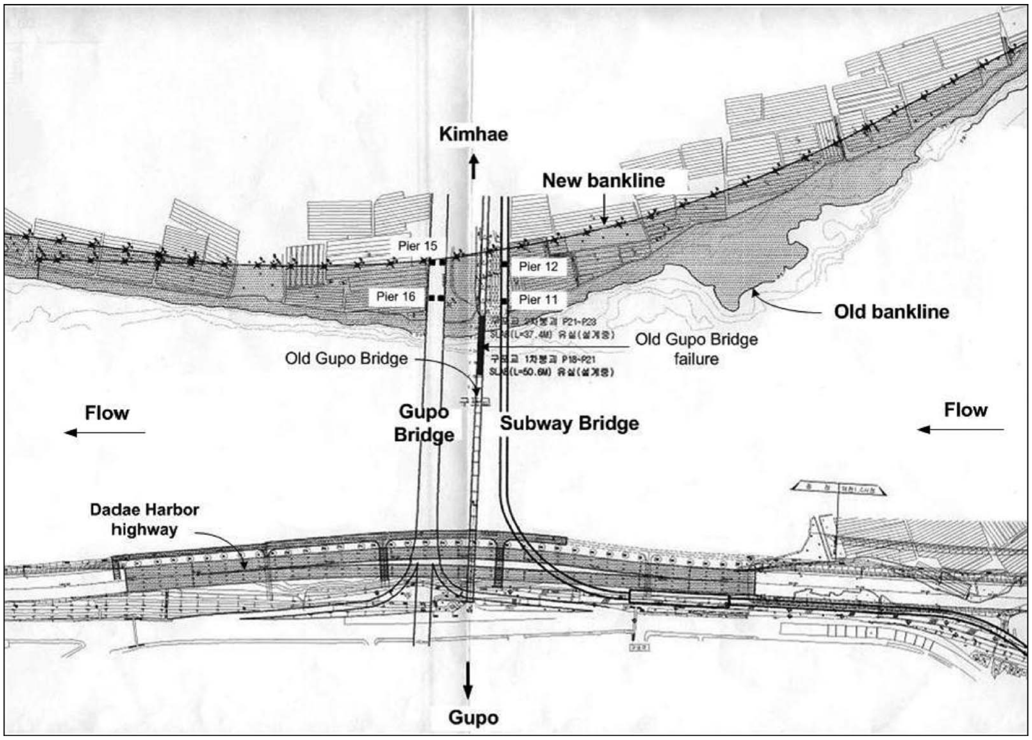


Fig. 4. Gupo and subway bridges on the Lower Nakdong River

were examined and the results are plotted in Fig. 7. As a result, the filter should include a double layer of stone filters for bed and riprap. The sizes for the two recommended stone filters were 4 mm for a filter adjacent to the bed and 40 mm for a filter adjacent to the riprap. Additionally, a synthetic filter was also recommended between the bed material layer and the bed filter layer to prevent pumping of soil particles, which can cause pier scour and scour hole development even in presence of riprap and filter layers. In the case of the Lower Nakdong River, the synthetic filter was required because the bed material is very fine compared to the riprap size. The need for a synthetic filter was questioned and so was the subject of the laboratory experiments. The double layout of stone filter layers and synthetic filter is shown in Fig. 8.

Melville and Coleman (2000) recommend a lateral extent of the riprap layer of $3b$ to $4b$. Therefore, the lateral extent from the edge of the pier footing was recommended to be at least 15 m in

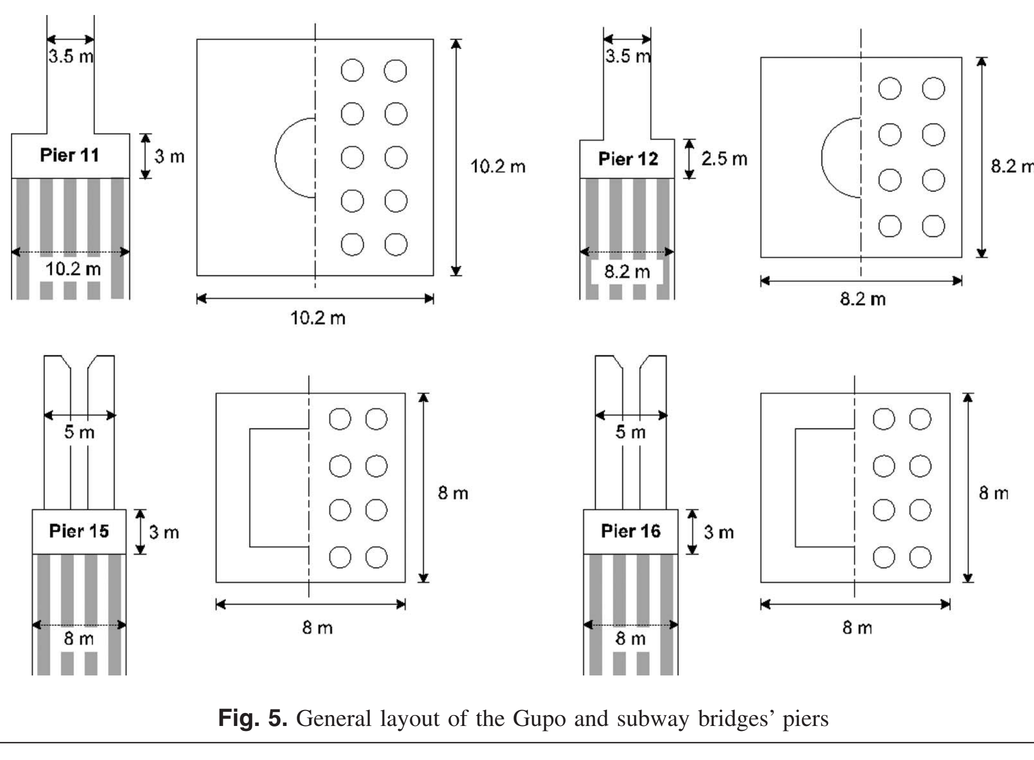


Fig. 5. General layout of the Gupo and subway bridges' piers

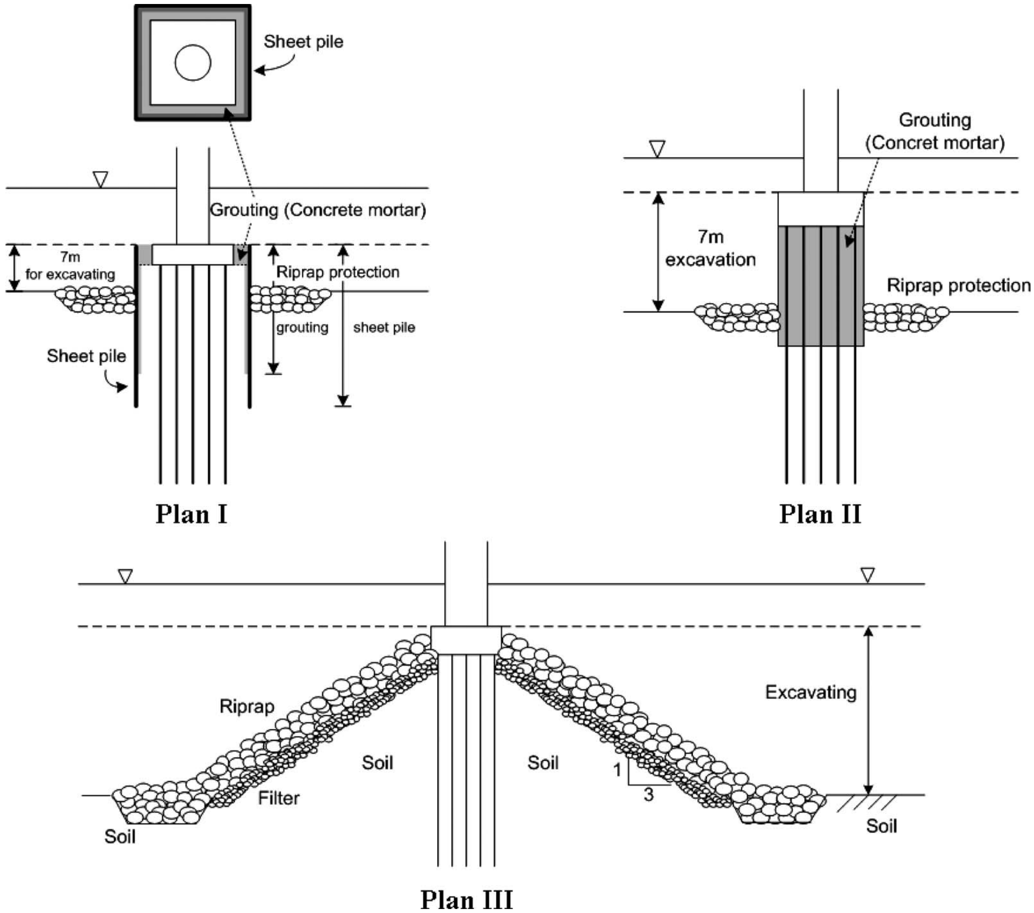


Fig. 6. Proposed plans to be considered

this case. Accordingly, only 10 m separates the filter layers of adjacent piers and this area would scour easily without riprap. It was hence strongly recommended to cover the entire area between the piers with riprap and filters, including a synthetic filter as shown in Fig. 8. For similar reasons, Piers 12 and 15 are close to the new riverbank and the same size and gradation for riprap and filters were suggested to protect the riverbank. Trenches were considered necessary near the edge of the riprap protection blanket toward the main channel in Fig. 8. This area between the natural river and the trenches allows erosion through natural processes until the erosion reaches and undercuts the supply of riprap. As the rock supply is undercut, it falls onto the eroding

Table 1. Strengths and Weakness of Three Design Options

Designs	Strengths	Weaknesses	Methods
Plan I	Very feasible for the Gupo and Subway Bridge's piers	Possibility of minor disturbance and subsidence	Sheet piles and riprap
	No exposing and disturbing the supporting piles under the foundation (strong stability)	Requires rather deep grouting that can be expensive	
	Convenient for navigation		
	Valuable gain in flood carrying capacity		
Plan II	Easy to grout the wall caisson	Highly concerned about the stability due to exposing and disturbing the supporting piles under the foundation	Wall caisson
	Does not require additional structures		
	Maximum gain in flood-carrying capacity	Additional load to the supporting piles under the foundation due to the grouted wall caisson	
	Most suitable for navigation		
Plan III	Easy construction	Possibility of particle erosion in floods, riprap slumping and sliding	Sloping structure with riprap
	Lower cost		
	No exposing and disturbing the supporting piles under the foundation (strong stability)	Reduction in cross-sectional area and minimal gain in flood-carrying capacity	
	Easy to repair in case of local damage and loss of riprap	Obstruction to navigation	

Table 2. Estimates of Pier-Scour Depth

Scour depth (m)	P11 square (sheet pile)	P12 square (sheet pile)	P15 rectangular (sheet pile)	P16 rectangular (sheet pile)
Pier width, b (m)	15.2	13.2	13	13
FHWA HEC-18 (Richardson et al. 2001: modified CSU)	10.4	9.7	9.6	9.6
Melville (1997)	22.1	20.6	20.4	20.4
Ansari and Qadar (1994)	19.9	18.8	18.6	18.6
Breusers and Raudkivi (1991)	8.7	8.1	7.9	7.9
CSU (Richardson et al. 1975)	14.5	13.2	13.1	13.1
Neill (1973)	22.8	19.8	19.5	19.5
Application	10.4	9.7	9.6	9.6

area, thus giving protection against further undercutting and halting further movement (Julien 2002). Trenches should be buried 4 m deep and 3 m wide.

Sheet Piles

Considering the machinery required for driving sheet piles around piers, sheet piles could be driven 2.5 m away from the edge of pier footings in a square shape around the Subway Bridge piers and in a rectangular shape around the Gupo Bridge piers. Fig. 9 illustrates the example case of the Subway Bridge piers. The recommendation to drive and construct sheet piles was the following: (1) piles should be driven 25 m deep; (2) grouting (concrete mortar) should fill the space between the cap concrete foundation and the sheet pile to the foundation depth; and (3) partial grouting 50 cm thick on the inner side of the sheet piles to a depth of 20 m from the top of the foundation.

The recommended depth of 25 m for the sheet piles is intended to protect piers and prevent sand motion in and out under the bed. For stability purposes, it would be better to drive the piles possibly even deeper. The partial grouting of the inner space at a depth of 20 m and thickness of 50 cm will increase stability, and provide additional support against momentum forces of flowing water and floating debris during floods. It would also prevent the fine material inside the piles from leaching out into the river. Paired Piers 15 and 16 would be best enclosed in a single sheet pile structure of rectangular shape, as shown in Fig. 8. It is easier to construct a more stable single structure at a lower cost rather than two separate square structures. Sheet piles must be driven before excavating the surrounding floodplain material. This excavated material can be temporarily used to extend protection near the main channel. After excavating and dewatering, filters and riprap can be dry placed and the trench construction can be completed.

Table 3. Estimates of Pier-Scour-Hole Geometry

Pier	Sheet pile	Width (m)	Scour depth y_s, (m)	Width of scour hole, w_s (m)
P11	Square	15.2	10.40	20.8
P12	Square	13.2	9.70	19.4
P15	Rectangular	13.0	9.60	19.2
P16	Rectangular	13.0	9.60	19.2

Laboratory Study of Plan I

An experimental study of Plan I was conducted in the Hydraulics Laboratory at Pusan National University, Busan, South Korea. The primary purpose of the experiments was to examine the need for a synthetic filter and it was also used for rough estimates of scour depth. A distorted Froude-similitude physical model was built at a horizontal scale of 1:400 and a vertical scale of 1:100. The main scaling ratios are given in Table 5. A distorted model was adopted as a feasible practical alternative given the width constraint from the available laboratory space. The pier width was scaled 1/400 of the horizontal scale and the water depth was scaled 1/100 of a vertical scale. The reach length of the model was 20 m with the widest stream width of 3.56 m. The 200 year flood discharge, 19,370 m^3/s, at the Gupo Bridge was scaled to a laboratory discharge equal to 0.048 m^3/s. The maximum water depth in the model was 14.5 cm and the approach velocity and flow depth upstream of Piers 11, 12, 15, and 16 were 22.6 cm/s and 6.62 cm, respectively. The particle sizes used in the model were 0.25 mm

Table 4. Riprap Size Calculations

Pier	Case	Sheet pile width (m)	Scour depth (m)	Flow depth (m)	Velocity (m/s)	Froude number	Specific gravity	Richardson and Davis (1995) (m)	Parola (1993, 1995) (m)	Lauchlan (1999) (m)	Average (m)	Application (m)	Weight (kg)
P11	Square	15.2	10.4	6.62	2.26	0.28	2.65	0.53	0.39	0.52	0.48	0.50	173.35
P12	Square	13.2	9.3	6.62	2.26	0.28	2.65	0.53	0.39	0.52	0.48	0.50	173.35
P15	Rectangular	13.0	9.2	6.62	2.26	0.28	2.65	0.53	0.39	0.52	0.48	0.50	173.35
P16	Rectangular	13.0	9.2	6.62	2.26	0.28	2.65	0.53	0.39	0.52	0.48	0.50	173.35

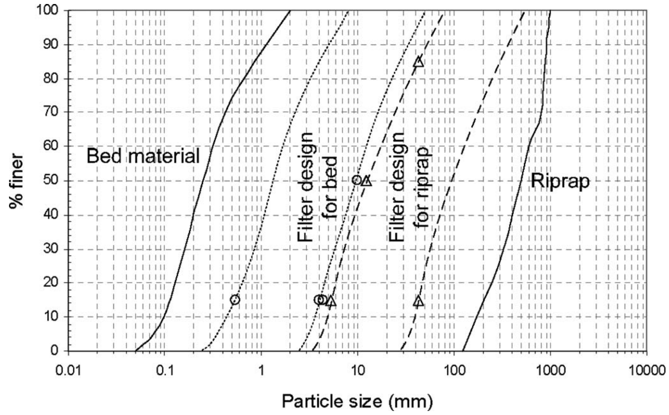


Fig. 7. Filter design

(specific gravity, 2.65) for the bed material, 2 mm for the stone filter, and 5 mm for riprap. The grain size of the bed material used in the model test was the same as the prototype. Therefore, the roughness factor was gradually adjusted in the model until the model results of discharge, velocity, and flow depth agreed with the prototype conditions. The model boundary roughness was, therefore, slightly higher than that required by the scale ratio and the consequence was that the model channel had a slightly higher boundary resistance than the prototype.

The clear-water condition was applied to the model and the scour rate was measured throughout the tests. It took about 1 h to establish the steady-state flow condition and then the equilibrium scour depths were reached after an additional 1.5 h. The laboratory test duration was decided based on the time to reach equilibrium scour depth, which was 1.5 h after the steady-state flow condition was settled. This corresponded to 4 days at peak discharge and exceeded the expected duration of most floods caused by typhoons. The experimental results of maximum scour depth are a little less than the estimates from the FHwA HEC-18 equation. For example, the experimental result of measured maximum local scour depth for P16 was 6.1 cm in the model (Fig. 10), which would be 6.1 m in the prototype. In addition, the HEC-18 equation was applied to the laboratory model condition to examine the effect of the distorted-scale model on the scour estimation. As a result, the predicted scour depth (5.84 cm) agreed well with the observation (6.1 cm) in the physical model for P16 as shown in Table 6. Because the laboratory condition did not satisfy one of the wide pier criteria $(y/b<0.8)$ due to the distorted scale in the model, K_5 factor of the HEC-18 equation was not applied to predict the scour depth in the model condition. The maximum scour-hole widths for P16 in the experiments ranged from 4.0 to 6.2 cm in the model. It corresponds to 16–24.8 m in the prototype, which is similar to calculation results as shown in Table 3. The results were considered acceptable given the built-in distortion of this physical model.

Physical model studies were also performed to specifically examine the necessity of using a synthetic filter, which was recommended in the filter design for Plan I. Fig. 11 shows the experimental result without the synthetic filter and Fig. 12 shows the results with the synthetic filter for comparison. Even though riprap and filter layers existed in the experimental case without synthetic filter, the bed material, filter, and riprap were rapidly mobilized and dispersed by the simulated flood. These physical model results convincingly demonstrated the need to include a synthetic filter in this design. The synthetic filter is required for this retrofitting design of the Lower Nakdong River because the bed material is very fine compared to the riprap size.

Implementation

After considering these three options, Plan I with sheet piles and riprap was recommended in 2006 as the most feasible and appropriate protection countermeasure. It was well received at all levels and seemed to be the best possible design. In conducting preliminary cost estimates for the options, however, the contractor concluded that the construction cost for Plan I doubled the cost of Plan III. Cost concerns prevailed and this extended the review process considerably.

After long deliberations, Plan III was finally selected and implemented for retrofitting the Gupo and Subway Bridge' piers by Busan City. As shown in Fig. 6, Plan III applied the method of riverbank protection using riprap on a 1:3 side-slope structure to protect the piers. Riprap and filters are placed to prevent surface erosion of a side slope. To estimate the riprap size and gradation, the CHANLPRO program of the U.S. Army Corps of Engineers (USACE) was used, which contained the USACE riprap design guidance (USACE 1994). Because the CHANLPRO program provided multiple results, the velocity method (Julien 2002) was also used for comparison and final recommendation on the riprap design. The proposed limit of the riprap size and weight in the diagram was that the median particle of a riprap stone should be bigger than 1.35 ft (0.41 m) and heavier than 180 lb (82 kg). Considering the results of the velocity method, the resulting riprap size for this option was 56 cm. Also, the filter calculation method previously used in Plan I was also applied to Plan III.

A modified version of Plan III was finally adopted by the contractor and the city of Busan for final implementation because of feasibility and practicality of construction. Fig. 13 shows the cross-sectional and plan views of the design plan for the modified Plan III. Among the modifications, the design includes a widening of the bridge pier footings with riprap. The side slope was increased to 1V:2H from the 1V:3H of the original proposed design. The retrofitting construction of P15 and P16 was completed and the construction of P11 and P12 was under final construction stage in June 2007 as shown in Fig. 14.

Conclusions

Three design options were proposed for retrofitting bridge piers with massive spread footings on the flood plain of the Lower Nakdong River. Of the three options for retrofitting the Gupo Bridge piers, Plan I using sheet piles and riprap was recommended in 2006 as the most feasible and appropriate for protection against pier scour. After estimating construction costs, a modified version of Plan III was adopted for the final design and construction.

The main conclusion of this paper is that it is possible to retrofit bridge piers with massive spread footings on flood plains. The final design required large riprap sizes to prevent scour depths up to 10 m. Detailed estimates of scour depth and scour-hole width around bridge piers with riprap and filter protections were obtained and compared with experimental laboratory studies for Plan I. Also, the physical model study demonstrated that both gravel and synthetic filters should be used in the case of the Lower Nakdong River because the bed material is very fine compared to the riprap size.

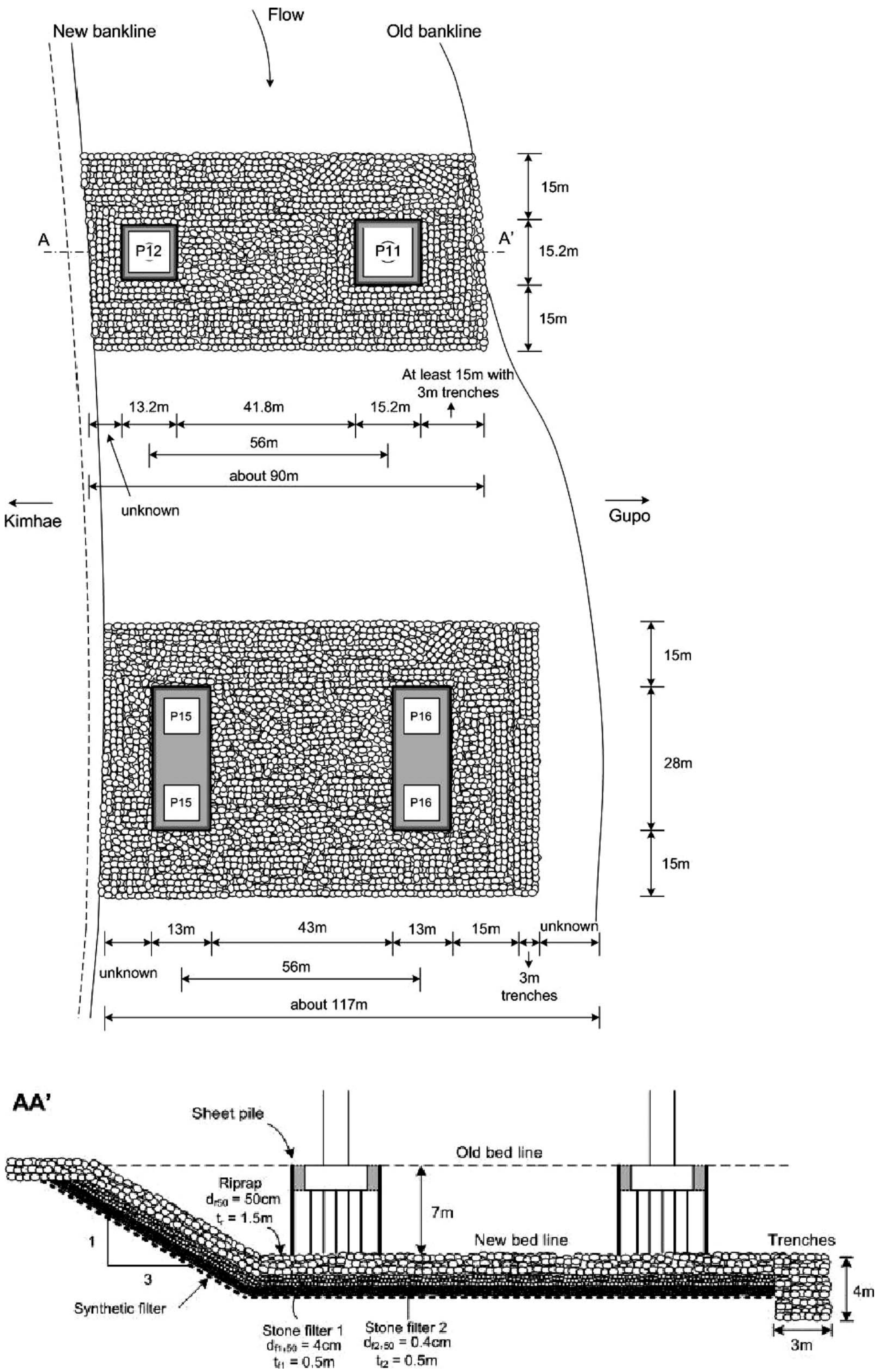


Fig. 8. Plan view and front view of Plan I

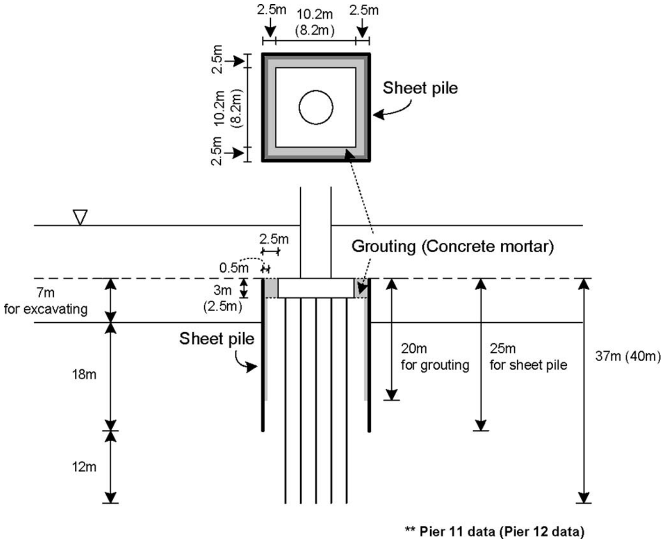


Fig. 9. Sheet pile layout for Piers 11 and 12

Table 5. Physical Model Scales

Parameter	Symbol	Scale	Reference
Horizontal length	x_r	1/400	
Vertical length	y_r	1/100	
Area	A_r	1/40,000	$x_r y_r = \frac{1}{400} \times \frac{1}{100}$
Discharge	Q_r	1/400,000	$x_r y_r^{3/2} = \frac{1}{400} \times \left(\frac{1}{100}\right)^{3/2}$
Velocity	V_r	1/10	$y_r^{1/2} = \left(\frac{1}{100}\right)^{1/2}$
Slope	S_r	4	$\frac{y_r}{x_r} = \left(\frac{1}{100}\right) / \frac{1}{400}$
Time (duration)	T_r	1/40	$x_r y_r^{-1/2} = \frac{1}{400} \times \left(\frac{1}{100}\right)^{-1/2}$
Manning's n	n_r	0.9283	$\frac{y_r^{2/3}}{x_r} = \left(\frac{1}{100}\right)^{2/3} / \left(\frac{1}{400}\right)^{1/2}$

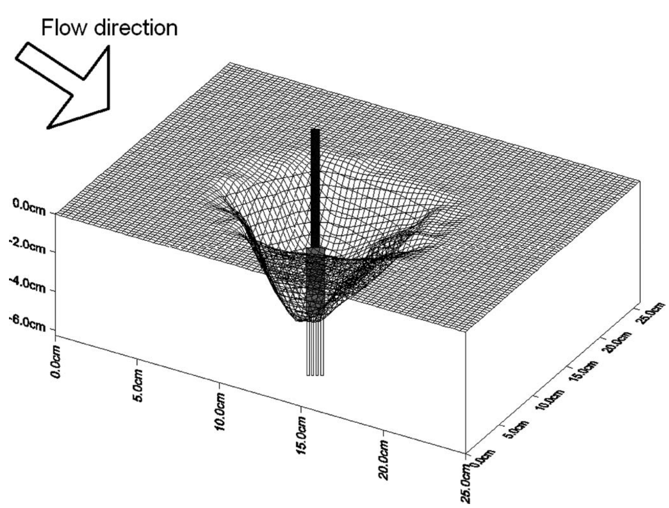


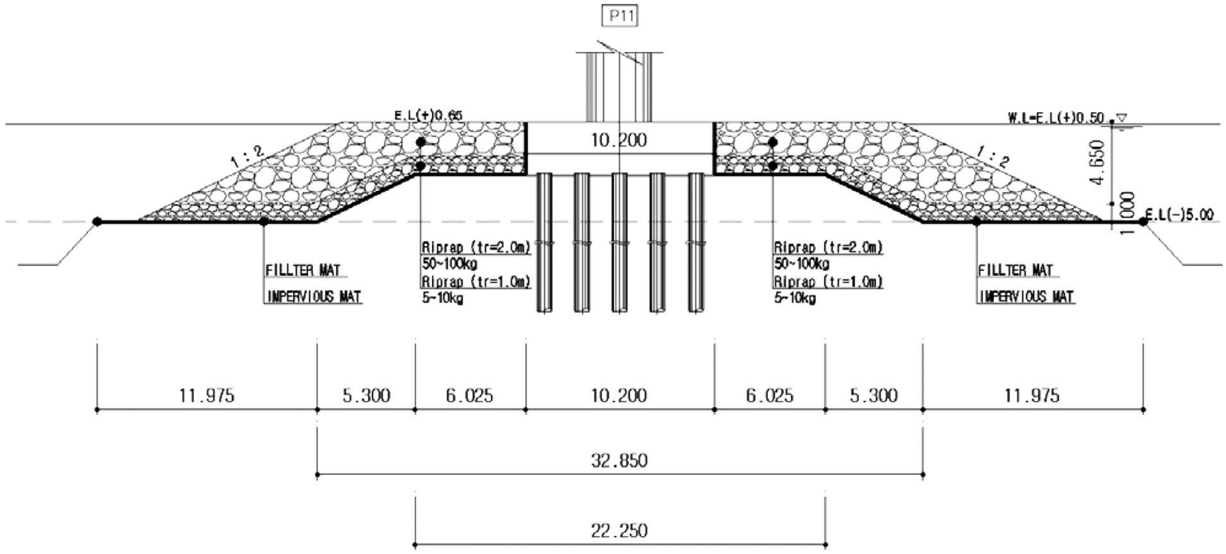
Fig. 10. Experimental results of P16 for a 200 year flood discharge

Table 6. Prototype and Model Conditions, and Scour Depth Estimations at P16

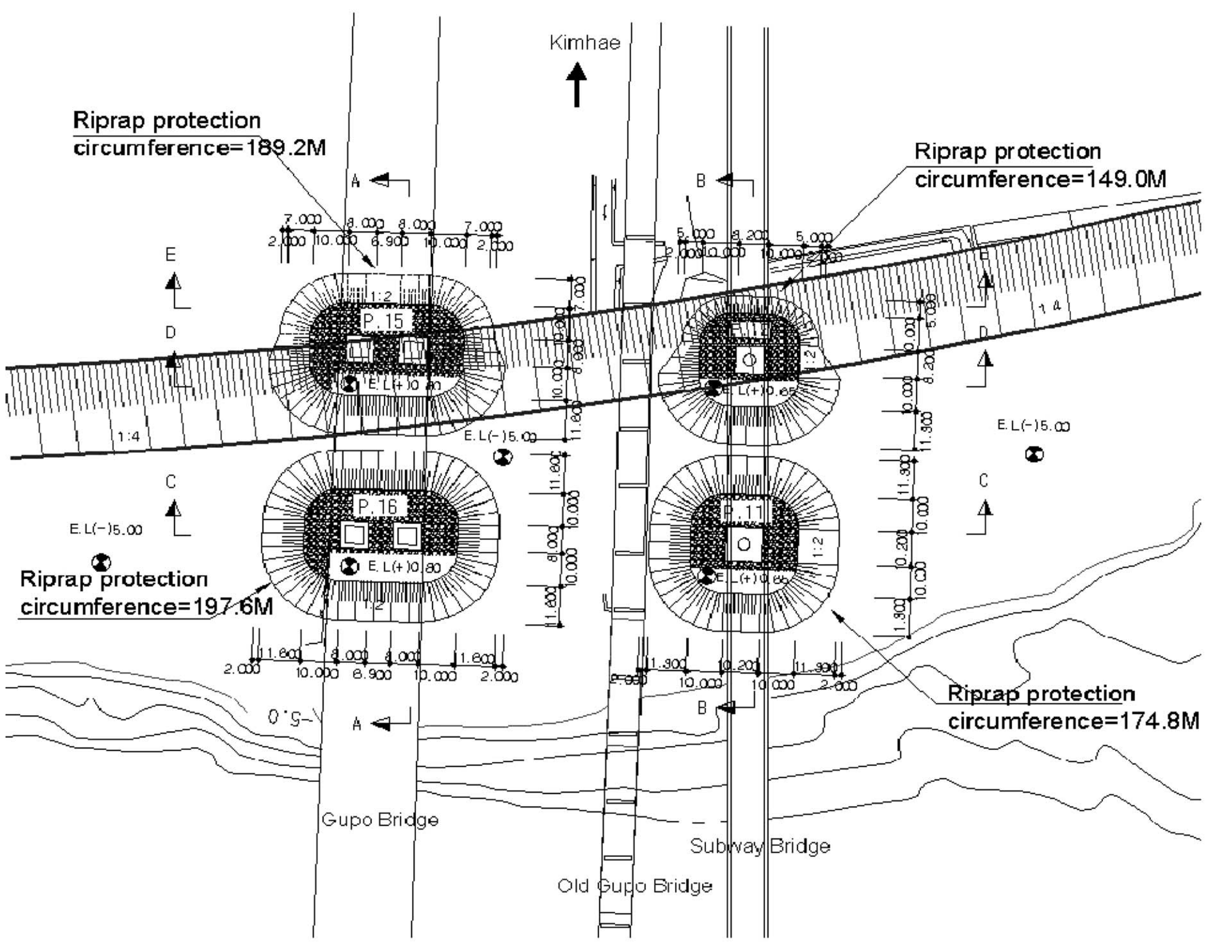
P16 (Plan I)	Prototype	Model	Reference
Discharge (Q)	19,370 cm s	0.048 cm s	1/400,000 scale
Flow depth (y)	6.62 m	6.62 cm	1/100 scale
Velocity (V)	2.26 m/s	22.6 cm/s	1/10 scale
Pier width (b)	13 m	3.25 cm	1/400 scale
Median particle size (d_{50})	0.25 mm	0.25 mm	
y/b	0.4355	2	<0.8
b/d_{50}	60,800	130	>50
Scour depth estimation	HEC-18: 9.6 m	HEC-18: 5.84 cm (Prototype: 5.84 m) Observation: 6.1 cm (Prototype: 6.1 m)	K_5 factor was not applied to predict the scour depth using HEC-18 and model parameters

Fig. 11. Experimental results without synthetic filters

Fig. 12. Experimental results with synthetic filters



(a) Cross-sectional view



(b) Plan view

Fig. 13. Implementation design drawing for modified Plan III (Source: Limkwang Engineering & Construction 2007)

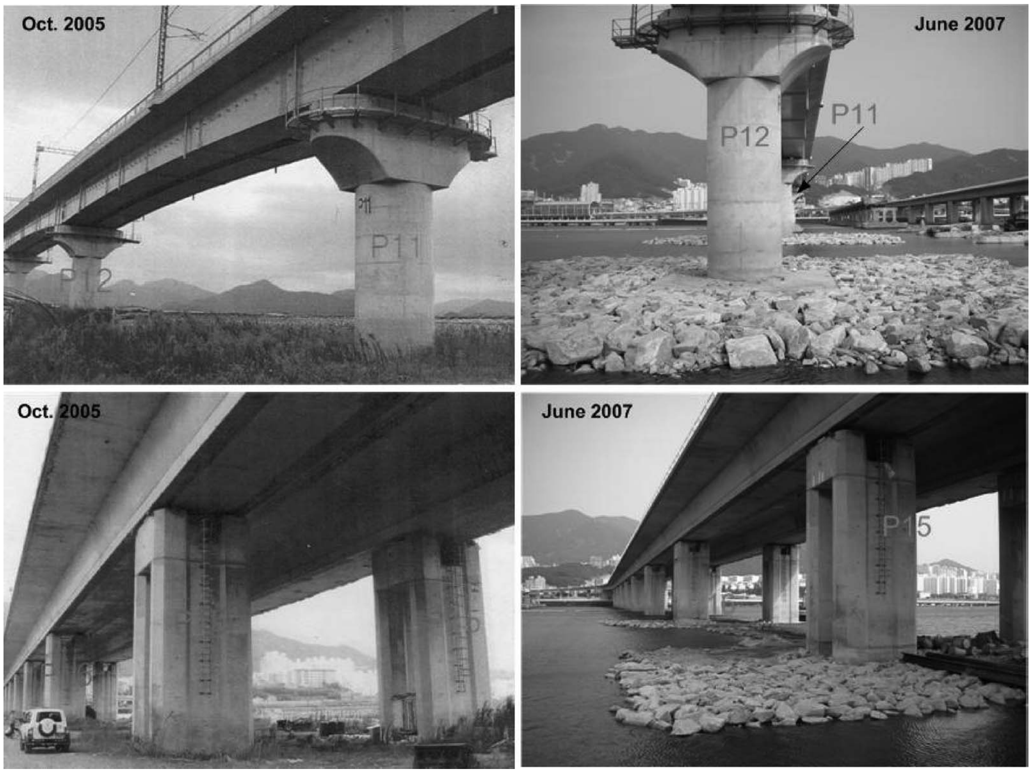


Fig. 14. Gupo and subway bridges' piers before and after retrofitting construction

Notation

The following symbols are used in this paper:

b = pier width;
d = particle size;
d_f = stone filter particle size;
d_r = riprap particle size;
K_w = correction factor of the shallow flow condition;
t_f = stone filter layer thickness;
t_r = riprap layer thickness;
w_s = width of the scour hole;
y = flow depth; and
y_s = scour depth.

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