APPENDIX-VI

Sabo/ Flood Control Structural Measures

THE STUDY ON SABO AND FLOOD CONTROL
FOR WESTERN RIVER BASINS OF MOUNT PINATUBO
IN THE REPUBLIC OF THE PHILIPPINES

FINAL REPORT

SUPPORTING REPORT

APPENDIX VI SABO/ FLOOD CONTROL STRUCTURAL MEASURES

Table of Contents

Page

List of Tables

List of Figures

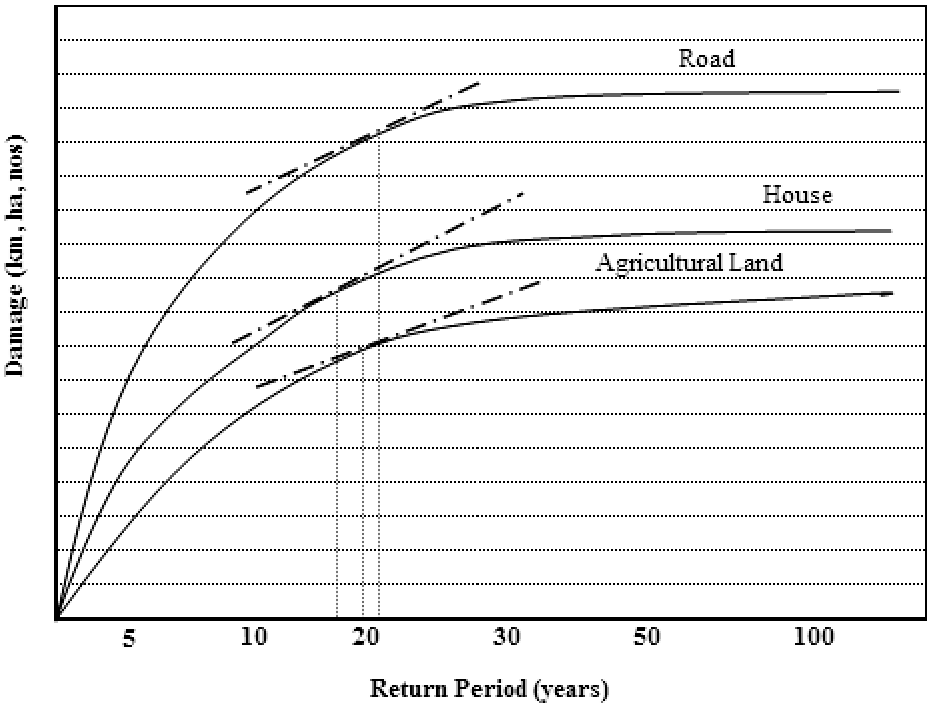
CHAPTER 1 MASTER PLAN OF STRUCTURAL MEASURES

1.1 Assessment of Planning Scale

For the purpose of this study, the benefit gained through the implementation of a project is defined as the reduction in the direct and indirect damage that would otherwise be caused by the flooding and/or mudflow.

The probable direct damage was evaluated under the without-project conditions. The damage under the with-project conditions was assumed to be zero under the design flood of a specified return period. Thus the project benefit constitutes the probable damage that would occur by the flooding and/or mudflow of the design scale.

The numbers of houses, roads and paddy fields in the three river basins were computed as described in "Section 2.2 in Appendix V", namely the Bucao River, the Maloma River and the Sto. Tomas River, respectively. The damaged assets curves based on damaged asset numbers due to the probable flood show almost same tendencies among the three river basins. The gradient of the curves in the schematic diagram below show that the incremental numbers of direct damage occurrences are not significant beyond the 20-year return period. Hence, the return period of 20 years is reasonable for the planning scale of the structures.



Schematic Diagram for the determination of Planning Scale

1.2 Possible Structural Measures

The list of possible structural measures for each river basin is given in Table 1.2.1. The design concept for each possible structural measure is described below.

1.2.1 Sediment Source Zone: to reduce the volume of sediment at the source

(1) Structure at Mount Pinatubo crater

The water in the crater of Mount Pinatubo filled up to the edge of the Maraunot Notch in September 2001, with an estimated storage volume of approximately 300 million m^3. Due to a heavy storm and a continuation of overflow from the crater lake, the Maraunot Notch collapsed in July 2002 and the water level of the crater lake was drawn-down by about 23m.

The discharged water volume was estimated about 46 million m^3 as mentioned in "Section 1.2 in Appendix IV". The water channel in the fractured rock was deeply incised due to the spilling water.

An overflow structure and protection works will be required to avoid further channel erosion if the results of the geological investigation show the possibility for further erosion and/or collapse.

(2) Re-vegetation

Seeding and planting of suitable vegetation on the sediment source is one of the effective methods, not only to reduce the sediment yield due to gully development, but also to contribute towards environmental conservation of the mountain slopes. The appropriate species for this purpose are recommended as robust perennials with vigorous rhizomes and extensive root systems.

(3) Small-scale sabo dam

A series of small-scale sabo dams at the downstream end of small tributaries would be effective in reducing the sediment yield from tributaries in the areas consisting of pyroclastic material.

(4) Large-scale sabo dam

A large-scale sabo dam crossing the major tributaries would be effective in storing the surplus sediment, stabilizing channel bed in the upstream from the dam, avoiding sudden riverbed degradation in the channels and lateral erosion of the riverbanks.

1.2.2 Sediment Deposition/Secondary Erosion Zone: to stabilize unstable sediment

(1) Consolidation Dam

A consolidation dam would be effective in stabilizing deposits in the channel and controlling sediment discharge to the downstream stretches.

(2) Sand Pocket Structures

A remarkable volume of sediment remains within the channel in the middle reaches of the Bucao River and the Sto. Tomas River. The lahar deposits will be remobilized by secondary erosion during every flood event.

To cope with the excessive sediment flowing down through the sediment transport zone in the rivers, a sand pocket structure is proposed to trap remobilized sediment deposits at the downstream end of the secondary erosion zone or at the upstream end of the sediment transport zone.

(3) Groundsills

A series of groundsills is one of the alternatives to regulate the secondary erosion of deposits and control the watercourse in the channel.

(4) Training Works

At Malomboy, 10 km upstream from the river mouth of the Bucao River, the river channel is frequently shifting after flood events and also the lower reaches of the Marella River is same as the vicinity of Malomboy in the Bucao River.

Therefore, it is recommended that river training works are one of effective structural measures to fix the course of river channel.

1.2.3 Sediment Conveyance Zone: for smooth transportation of sediment to the river mouth

(1) Dike System

A dike system is provided to protect landside areas from flooding and/or mudflow.

In the lower- middle reaches in the Bucao River, the existing dike height is lower than proposed design water level. The existing revetment on the existing dike is damaged in some portions due to annual flooding. Therefore existing dike in the Bucao River is also improved with reinforcement/ rehabilitation works.

In the middle stretch of the Sto. Tomas River, the height difference between the landside elevation and the water surface has reached several meters. The other hand, in the upper stretch of the Sto. Tomas River, the difference is created for maximum 12 m high with annual sediment deposits.

Therefore, reinforcement/ rehabilitation works on the existing dike system are proposed to protect the flood prone area in case a breach of the dike system occurs.

(2) Spur Dike

A spur dike is generally positioned at an outer curve of the watercourse to the riverbank to control the flow direction towards the center of the channel. A series of spur dikes is an effective means of protecting the dike system from local scouring in footings of the slope protection works.

(3) Channel Excavation

The purpose of channel excavation is aimed to maintain the flow capacity of the river channel, especially for the downstream stretch from the bridge to the river mouth.

Additionally, the measure is not proposed in the channel occurring sediment flow heavily, because the excavated channel may be buried and lose the function immediately due to sediment deposits swept away from secondary erosion zone and/or sediment source zone during one flood event.

1.3 Structural Measures recommended by USACE

The USACE prepared a comprehensive Recovery Action Plan (RAP) in 1994. The RAP evaluated the methods for controlling the sedimentation within the major eight river basins surrounding Mount Pinatubo, and the higher risk of flooding due to sediment–clogged drainage channels caused by the eruption of Mount Pinatubo.

In the western river basins, the following structural measures were recommended (Figures 1.3.1 to 1.3.3):

Bucao River Basin :

(1) Heightening of the existing dike system along the right bank in the lower stretch of the Bucao River.

(2) A sediment retention structure at Malomboy with a capacity of

1,000 million m3 to reduce downstream sedimentation.

Maloma River Basin : (1) Widening and straightening of the river channel with dike system in the downstream stretch.

(2) A sediment retention structure with a capacity storing a sediment volume of 12 million m3 in the upper reaches.

Sto.Tomas River Basin : (1) A dike system along both banks, 6 km along the right and 18 km along the left bank.

(2) Channel excavation work instead of a dike system, and

(3) A sediment retention structure with a capacity of 40 million m3 in the upper stretch of the Marella River.

The alignment of the existing dike system constructed by the DPWH principally coincides with the recommendation plan by RAP, however the sediment retention structures have not been constructed yet.

1.4 Hazard Scenarios by PHIVOLCS

The potential hazards were defined as 'Hazard Scenarios' by PHIVOLCS in 1998 for the Bucao and Sto. Tomas River basins. The synopsis is summarized below:

1.4.1 Bucao River Basin

(1) Deposited area/ lahar situation:

- Primary lahar deposits continue to occur mostly upstream from Malomboy.
- Flows will probably remain confined to the valley, especially in the upper and middle reaches of the river.
- Remobilization of pre-eruption and post-eruption lahar materials may also be expected, the phenomenon has been found in the Pasig-Potrero River of the eastern part of Mount Pinatubo.

(2) Hazard Scenarios:

- In the event of more rain, lahar deposits resulting from erosion and remobilization of lahar deposits, toward the lower reaches of the river may be expected.
- Dilute lahars and stream flows may undermine the northern part of the Bucao Bridge.
- Existing dikes are still prone to piping and erosion of basal support
- In the long-term, flows may encroach towards Botolan as a result of the perched condition of the lahar field.

(3) Comments and Recommendation by PHIVOLCS

- Repairs, maintenance or completion of three armoring of portions of the existing dike system should be done before the1998 rainy season to reduce the risk of breaching/avulsion due to lahars.
- Residents adjacent to the dike system should be aware that even with a properly constructed and aligned dike, there is always the potential for failure and as such, they should not be complacent and should always be on the alert especially during the rainy season.

1.4.2 Sto. Tomas River Basin

(1) Deposited area/ lahar situation:

- Primary lahar deposits continue to occur mostly upstream from the Marella River.
- Flows will probably remain confined to the valley, especially in the upper and middle reaches of the river.

(2) Hazard Scenarios:

- In the event of more rain, major lahars may still be expected to occur along the lower reaches of the channel downstream of Mount Bagang.
- Dilute lahars and stream flows may undermine the northern part of the Maculcuo Bridge and the southern part of the existing left banks
- Dilute lahars are expected to be more predominant especially during low flows periods, and may result in remobilization of pre-eruption and post-eruption lahar material along the upper and middle reaches of the river that would flow towards the lower reach of the river

(3) Comments and Recommendation by PHIVOLCS

- Major deposits along Mount Bagang stretch of the Sto. Tomas River may force lahars to pass through the back (north) side of Mount Bagang, which could threaten Santa Fe and Umaya.
- Should the flow continue to deliver sediment into Lake Mapanuepe, it may pose an imminent danger through clogging of the artificial outlet in the lake. The damming effect may then increase the level of the lake, and ultimately cause breaching of the lake with a tremendous volume of water and sediment overflowing from the lake area.
- Should lahars flow along the south side (Castillejos side) of the dike system, there is potential for breaching and/or overtopping at the Lawin-Vega Hill stretch, threatening barangays within this existing dike system.
- Where the river flow is adjacent to the dike, the dikes are prone to erosion of the basal support even by streams of normal volume.
- Due to the minimal freeboard at the Maculcol Bridge, there is the potential for overtopping, bridge damage, and road closure due to lahar.

1.5 Structural Measures in the Bucao River Basin

1.5.1 Reinforcement of the Maraunot Notch

(1) Damaged Maraunot Notch

The spilled water from the crater lake through the Maraunot Notch on 10 July 2002 triggered a flash flood through the Labao River, joining into the Baling-Baquero and Bucao Rivers. The volume of re-mobilized sediment was estimated at 46 million m^3 excluding the sediment outflow into the South China Sea.

(2) Proposed Structural Measures

Since the collapse in the Maraunot Notch, further collapse in the Notch has not been observed yet. To examine whether the notch is furthermore damaged caused by collapse in the future, the geological investigation is carried out with site inspection and geo-resistivity survey for revelation of geological structure underneath the notch.

According to the examinations of the investigation, it is judged that the risk due to the further collapse in the notch is kept getting lower than previous condition for a few decades.

Details of the results for the geology are described in “Section 2.4 in Appendix II” :

For reference, preliminary future plans are prepared for structures aimed to prevent further collapse of the lake outlet. The plans are (Figure 1.5.1):

(a) Training channel with gabion structure,
(b) Overflow weir and training channel (concrete structure), and
(c) Ttunnel outlet

Plans (a) and (b) are the most adequate methods of preventing further collapse, directly. However, the plans do not have the function to control the outflow.

By constructing the drainage tunnel as shown in plan (c), it is possible to maintain the water level at the required elevation. However, the major disadvantage of this plan is the remarkably high construction cost.

Plans (a) and (b) are recommended by stable geological conditions. However, plan (c) should be considered if the geological conditions are weak.

1.5.2 Sabo Dams in Sediment Source Zone

A series of sabo dams (sediment retention dams) can be provided at the downstream end of the sediment source area. A location map or the prospective sabo dam site is shown in Figure 1.5.2. However, since the depth of lahar deposits is grater than 20 m at the dam axis, only the floating type of dam is applicable and serious erosion at the downstream face of the dam would be expected. The typical layout of the sabo dam is shown in Figure 1.5.3.

The relationship between the sediment storage and the volume of dam body is given as follows:

Priority	Dam Site	Dam Height (m)	(C) Dam Volume (MCM)	(B) Sediment Storage (MCM)	(B)/(C) Index	Unstable Sediment underneath dam (MCM)
1	No.6	5.0	0.007	3.895	556.4	22.1
2	No.3	5.0	0.021	5.040	240.0	32.6
3	No.4	5.0	0.022	3.250	147.7	11.4
4	No.5	22.0	0.011	1.157	108.2	2.8
5	No.8	5.0	0.006	0.528	88.0	3.1
6	No.7	5.0	0.006	0.307	51.2	0.2

Note : MCM means million cubic meters.

In the preparation of the above table, the optimum development scale was determined for each dam site. A comparison was made between the volume of sediment storage and the volume of the dam body. If the optimum height was determined to be lower than 5.0 m, the type of dam was considered to be a consolidation dam.

Based on the results, Sabo dam No.6 (consolidation dam) was ranked as the highest priority. It should be noted, however, that the construction cost of sabo dams is significantly high due to the provision of steel sheet piling (underneath the dam foundations) required to cope with the deep lahar deposits. The implementation of sabo dam construction might be a good plan for the future.

1.5.3 Alternative Structural Measures in Sediment Conveyance Zone

(1) Proposed Alternatives

Results of mudflow hazard analysis show landside area in lower reaches in the Bucao River are prone to been damaged caused by lahar sediment flowing out if the existing dikes are breached due to the flood and/or mudflow. The hazard area is predicted for more than 13 km^2 under the 20-year probable flood; especially eastern part of downtown Botolan may be damaged seriously.

The following three alternatives are adopted for a comparative study. A schematic diagram showing the structural arrangement is given in Figure 1.5.4.

(a) Alternative-1: Heightening of the existing dike (**Figure 1.5.5**)

The existing dike is located along the right bank for the 6 km stretch from the Bucao Bridge. The possibility of a dike breach remains for this stretch as evaluated in "Section 1.3 in Appendix V". Heightening of the dike will be required to ensure safety against mudflow.

(b) Alternative-2: Heightening of the existing dike and Malomboy consolidation dam (Figures 1.5.6 and 1.5.7)

Proposed structural measures in this alternative are consisting of Malomboy consolidation dam and existing dike heightening.

The downstream end of the sediment deposits and secondary erosion zone is located at Malomboy. A spindle-shaped valley has formed for the upstream stretch from Malomboy to the confluence of the Bucao and Baling-Baquero Rivers. A huge volume of unstable lahar deposits remains in the valley. A consolidation dam at Malomboy would be effective in stabilizing the unstable lahar deposits. The location of the dam site is exactly the same as that of the sediment retention structure recommended by the USACE in 1994.

(c) Alternative-3: Heightening of the existing dike, Malomboy consolidation dam and sand pockets (Figures 1.5.8 and 1.5.9)

Proposed structural measures in this alternative are consisting of Malomboy consolidation dam, two rows of lateral dike, three sand pockets, separation dike and existing dike heightening, respectively.

Sand pockets can be provided in the downstream stretch of Malomboy consolidation dam. The area of the sand pockets is estimated at 16 km^2 between Malomboy consolidation dam and separation dike.

The purposes of the proposed structure are:

- To accelerate sediment deposits,
- To protect the confluence of the Baquilan River and the existing dike from re-mobilized mudflow (this generally flows straightly)
- To fix the river channel to the left bank, and
- To decrease the sediment load in the downstream stretch

The river course is to fix on the left side and the lahar deposited area on the right side is expected to recover as the agricultural land in future.

1.5.4 Riverbed Movement Analysis during the Flood

The individual effectiveness of the above alternatives was evaluated using the two-dimensional mudflow analysis. The results of two-dimensional mudflow analysis are described in "Section 3.2 in Appendix IV".

The following examinations for three alternatives are described based on the above-analyzed results:

(a) Alternative-1: Heightening of the Existing Dike (Figure 3.2.5 in Appendix IV)

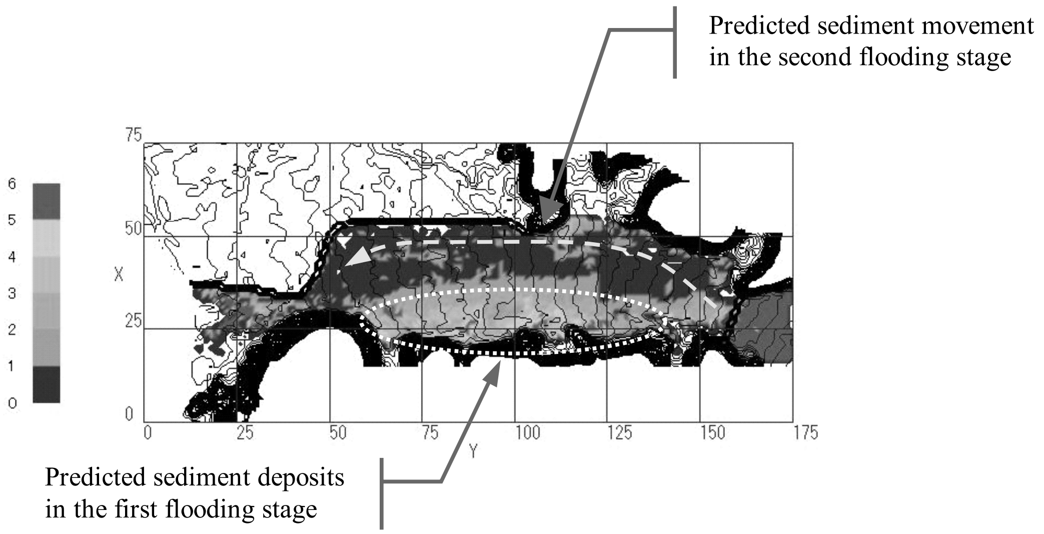
The simulation was made on condition, which is one lahar event with 20-year probable flood. The sediment deposits at middle- upper reaches were estimated at 3-6 m more in maximum, and at lower reaches, the significant sediment deposits were appeared in places. Such a localized sediment deposits is possible to occur anywhere in the river channel. Hence, proposed dike heightening is recommended to provide on the existing dikes.

(b) Alternative-2: Alternative-1 and Consolidation Dam (Figure 3.2.5 in Appendix IV)

The results of the simulation showed that the pattern of sediment deposits was tendency to move along left mountains side.

It is judged that this state is due to effectiveness of the controlled watercourse with consolidation dam in Malomboy. However, in the next flooding stage, sediment deposits from upper reaches may be swerved to the right existing dikes, because new watercourse may be created caused that the sediment deposits in the first flooding stage keep remaining along the foot of left mountains. Therefore, proposed dike heightening, which is almost same scale to those of Alternative-1, is recommended on the existing dikes to protect landside area from flood and/or mudflow damages if proposed dike heightening will be provided along existing dike.

The following figure shows the predictive movement of sediment deposits in the second flooding stage.



(c) Alternative-3: Alternative-2 and Sand Pocket (Figure 3.2.5 in Appendix IV)

The sand pocket structures, two rows of lateral dike and separation dike, are significantly effective in trapping the sediment with a depth of more than 6 m. The depth of sediment deposits along the existing dike is less than those in Alternative-1 and Alternative-2.

The openings of the lateral dikes and separation dike are provided adequately to fix the river flow along foot of mountains (left bank). As a result, the safety of right bank against the flooding and/or mudflow is increased.

Figure 1.5.10 shows the difference of presumed riverbed changes among three alternatives under the probable flood. As mentioned in the above, the height of dike heightening in Alternative-3 is the lowest between the alternatives, as the sediment retention capacity is the largest among the three alternatives. Taking into account the simulation results of the two-dimensional mudflow analysis, the required height of the dike (from the existing average river bed) is as follows:

Alternative	Averaged Sediment depth	Proposed Water depth	Freeboard	Proposed Averaged Dike Height from existing riverbed
Alternative-1	1.9 m	2.6 m	1.2 m	5.7 m
Alternative-2	1.9 m	2.2 m	1.2 m	5.3 m
Alternative-3	1.1 m	2.6 m	1.2 m	4.9 m

Note : Proposed water depth indicates average water depth with non-uniform flow analysis.

Above table shows the tendencies of riverbed movement with/without structural measures in Malomboy are not remarkable difference between Alternative-1 and –2 in terms of reduction of proposed dike height in the lower reaches.

Alternative-3 is providing the lowest dike height, the height is 0.8 m lower than Alternative-1. However, Alternative-3 compared with Alternative-1 and –2 requires more structural measures than other alternatives. Therefore, it is expected that the construction cost in Alternative-3 is highest than other alternatives based on relationship between predictive difference of dike height and required scale of structural measures.

Consequently, it is conceivable that adoptions of proposed consolidation dam, sand pockets and/or training channel in Alternative-2 or –3 are still not more feasible than those of Alternative-1 as the urgent scheme under present natural condition, because the estimated initial construction cost of Alternative-2 or –3 is even higher than those of Alternative-1. The respective difference is for 2.2 and 4.8 times based on Alternative-1.

1.6 Structural Measures in the Maloma River Basin

In the Maloma River basin, lahars were observed in 1991, however since 1992, no further lahars have occurred because the pyroclastic deposits in the sediment source zone were almost swept away to South China Sea due to heavy rain until 1992. In the master plan study, river channel improvement was proposed to be more important than mudflow control works.

The most serious concern from the viewpoint of flood control is insufficient flow capacity in the river channel at 3.5 km upstream from the Maloma Bridge. The river channel has a flow capacity of less than the 2-year flood discharge. Floods spill over the left riverbank, overflow the National Road No.7 and spread over the agricultural land between the National Road No.7 and the seashore.

Hence, the widening and straightening of the river channel with a scale of 20-year probable flood and the reconstruction of the Maloma Bridge are recommended to ensure enough flow capacity as shown in Figure 1.6.1. The longitudinal profile of the proposed river improvement works is shown in Figure 1.6.2.

In addition, rehabilitation of the dike will be required at the confluence of the Gorongorong/ Kakilingan River.

1.7 Structural Measures for the Sto. Tomas River Basin

1.7.1 Alternative Structural Measures

Results of mudflow hazard analysis show landside areas in lower- middle reaches in the Sto. Tomas River are prone to been damaged caused by lahar sediment flowing out if the existing dikes are breached due to the flood and/or mudflow. The hazard area is predicted for more than 56 km^2 under the 20-year probable flood. The scale of predicted hazard area is the largest among the three basins.

In the upstream stretch of the Marella River, there is a possible sabo dam site to store surplus lahar sediment. However, the ratio of proposed sediment storage to the concrete volume of dam body significantly lower than that in the Bucao River. Therefore, the following three alternatives are considered as shown in Figure 1.7.1.

(a) Alternative-1: Heightening/ Strengthening of The Existing Dike

During the flood on 8 July 2002 the landside area at the left bank for the stretch from the Maculcol Bridge to the river mouth was inundated with 1.0 m thick sediment due to the dike breaching. In the Sto. Tomas River basin, because the riverbed elevation is higher than the landside elevation, the mudflow would swerve to the landside area causing a considerable volume of sediment deposits if the dike breached. The general layout of dike system is shown in Figure 1.7.2 and the typical cross section of the dike is shown in Figure 1.7.3.

(b) Alternative-2: Heightening/ Strengthening of the Existing Dike, Consolidation Dam and Training Channel

Lahar deposits from the upstream reach of the Marella River created Lake Mapanuepe. The volume of water storage is estimated at 30 million m^3. The height difference between the normal water level and the surface elevation of the lahar deposits is about 8 m. At present, the lake functions are like a flood mitigation dam. The 20-year probable flood peak discharge can be reduced from 1,000 m^3/s to 250 m^3/s due to the effectiveness of flood mitigation in the lake.

For the Sto. Tomas River channel, the long-term forecast of sediment balance in “Section 3.3 in Appendix IV” shows the riverbed degradation of the Marella River from Mount Bagang to upstream stretch. If serious riverbed degradation occurs, the lake dikes will collapse due to erosion of natural levees created by lahar deposits.

To ensure the flood mitigation effect due to the lake continuing, erosion of the natural levees should be prevented or controlled. The construction of a consolidation dam at 4 km upstream from Mount Bagang and training channel will be effective in maintaining the existing riverbed elevation in the lower reaches of the Marella River. The training channel is designed to safely accommodate the 20-year design flood as shown in Figure 1.7.4. A typical cross section of the structures is shown in Figure 1.7.5.

(c) Alternative-3: Heightening/ Strengthening of The Existing Dike and Sand Pocket Structure

The purpose of this alternative is the same as Alternative-2. The sand pocket structure with three rows of lateral dikes of 3 m high and a ring dike of 8 m high is provided to avoid riverbed degradation and for trapping sediment as shown in Figures 1.7.6 and 1.7.7.

The lateral dikes have several openings. The locations of the openings are arranged alternately for each lateral dike in order to accelerate sediment deposits. The consolidation dam is provided at the downstream end of the sand pocket to stabilize the unstable lahar deposits underneath the sand pocket structures.

The advantages of the sand pocket structure are to protect Lake Mapanuepe from erosion as explained in Alternative-2 and because the construction costs are halve those of Alternative-2.

The disadvantage of this plan is not to be able to fix a river channel inside the sand pocket. It is possible that the river channel shift after every flood inside the sand pocket. Also the area of the sand pocket cannot be utilized as productive land.

1.7.2 Riverbed Movement Analysis during the Flood

The individual effectiveness of the above alternatives is evaluated using the two-dimensional mudflow analysis and one-dimensional sediment transport analysis. Details for the analyzed results are described in “Section 3.2 in Appendix IV”.

The examinations in viewpoint of structural measure effectiveness in the Marella River with results of both analyses are described as follows:

(1) Results of Two-Dimensional Mudflow Analysis

Results for three alternatives in Figure 3.2.6 (in Appendix IV) and 1.7.8 show almost same tendency to riverbed aggradation in downstream from Mount Bagang. The sediment deposits height is almost less than 1.0 m high in downstream from Mount Bagang.

This is conjectured that:

- In early flooding period, Sabo structures crossing the river in Alternative-2 and –3 may be buried caused by surging lahar sediment in the upstream from the Marella River and the structural functions are lost due to lahar flow conveying huge sediment deposits. Consequently, during middle to ultimate flooding stages, the states of downstream portion from the Marella River in Alternative-2 and –3 create same as the existing riverbed state. However, proposed sand pockets show the effectiveness trapping sediment deposits, comparatively.
- Above predicted process shows that huge sediment is still transported from sediment source zone in upstream from the Marella River and structural effectiveness (fixing watercourse, catching sediment deposits, etc) do not have adequate advantages under present condition.
- For lower-middle reaches of the Sto. Tomas River, the results exhibit the no significant difference among three alternatives.

The following table shows interval averages of maximum riverbed aggradation in each three alternatives:

River Stretch	Alt-1	Alt-2	Alt-3
River mouth to Paete Hill	1.1 m	1.1 m	1.1 m
Paete Hill to Vega Hill	2.0 m	2.1 m	2.1 m
Vega Hill to Mount Bagang	1.9 m	1.8 m	2.3 m

(2) Results of One-Dimensional Sediment Transport Analysis

Results for three alternatives in Figure 1.7.9 show the riverbed movement after 20 years are divided into two tendencies, one tendency is riverbed aggradation between river mouth and Vega Hill, the other is riverbed degradation between Vega Hill and the Marella River.

The following table shows averaged change of riverbed movements in each three alternatives:

River Stretch	Alt-1	Alt-2	Alt-3
River mouth to Paete Hill	0.7 m	0.5 m	0.5 m
Paete Hill to Vega Hill	0.6 m	0.7 m	0.7 m
Vega Hill to Mount Bagang	-0.5 m	-0.5 m	-0.4 m

It is conjectured that:

- Between the river mouth and Paete Hill, the results show almost same tendencies to riverbed aggradation for 20 years among three alternatives. However, in downstream from the Maculcol

Bridge, sediment transport in Alternative-1 is advanced little faster to river mouth than others. In terms of sediment transport ability in the channel, it is judged that Alternative-1 is a little better domination than other alternatives.

- Between Vega Hill and Mount Bagang, the results show tendency of almost riverbed degradation under the three alternatives. This tendency improves present state of risen bed river and it is expected to enhance safety of flooding and/or mudflow control ability to river dikes. However, the lower reaches of the Sto. Tomas River is affected by sediment transport from this section.
- In the lower reaches of the Marella River, the results show tendency of most riverbed degradation with proposed structures except for Alternative-2. The predicted degradation depth is range for 3 to 4m deep. This degradation may be effected to the natural levee of Lake Mapanuepe. However, the natural levee has an averaged height of 6 m between lake surface and natural levee crest at present. The natural levee having function, which divides in the Marella River and Lake Mapanuepe, may be still kept by remaining levee's height for about 2 m high within 20 years hence.

According to results of above two riverbed movement analyses, the tendencies of riverbed movement with/without structural measures in the Marella River are not remarkable difference among three alternatives during estimated 20 years in the lower- middle reaches of the Sto. Tomas River.

Consequently, it is conceivable that adoptions of proposed consolidation dam, sand pockets and/or training channel in Alternative-2 or –3 are still not more feasible than those of Alternative-1 as the urgent scheme under present natural condition.

However, if the sediment source zone in upstream from the Marella River will proceed to create further stable zone by the forces of nature in the future, the proposed structures of Alternative-2 or –3 will be applied to strengthen even further flood management and accelerate future agricultural development in the channel.

1.8 Recommended Structural Measures Proceeding to the Feasibility Study

The counter measures against flooding and/or mudflow for the three river basins are proposed with examinations of flood and mudflow control ability in above description.

To select the priority scheme among these proposed alternatives, the counter measures are also screened whether the implementation of individual proposed measure is feasible under the economic evaluation or not.

Additionally, the cost of reconstruction for the three bridges is included in the cost of structural measures for keeping traffic conditions on the National Highway No.7.

The results of the economic evaluation are summarized as follow:

River/ Alt –No.	Structural Measures	Project Cost (Million Peso)	EIRR
Bucao River			
Altternative-1	Dike Heightening/ Strengthening	981	15.2 %
Altternative-2	Alt-1 + Consolidation Dam	1,710	6.7 %
Altternative-3	Alt-2 + Sand Pockets	3,301	Negative quantity
Maloma River	Channel Widening	1,298	Negative quantity
Sto. Tomas River			
Altternative-1	Dike Heightening/ Strengthening	1,505	48.2 %
Alternative-2	Alt-1 + Training Channel in the Marella River	5,473	17.1 %
Altternative-3	Alt-1 + Consolidation Dam & Sand Pockets	3,556	25.5 %

Note: Detail breakdown of the project cost is described in Appendix-XII.

Above screened conclusions are described as follows:

(1) For the Bucao River

For the Bucao River, it is evaluated that Alternative-1, Dike Heightening is the most feasible in terms of the economic aspect.

The consolidation dam accelerates to keep the existing riverbed elevation and construction of sand pockets is to minimize lahar flow to the lower reaches of the Bucao River. However, these proposed structures is not be feasible under economic evaluation, because the huge estimated construction costs are why the river width at the location of proposed consolidation dam is more than 2 km wide and in the full length of the river width, deep sheet pile driving will be required to protect the structure against failure caused by local scouring at the downstream portion.

(2) For the Maloma River

For the Maloma River, it is evaluated that the proposed flood control measure is not feasible under economic aspect. Hereafter, it is expected that the adequate plan to current social aspect in the Maloma River basin will be reformulated based on this Master Plan Study.

(3) For the Sto. Tomas River

For the Sto. Tomas River, it is evaluated that Alternative-1, Dike Heightening is the most feasible in terms of the economic aspect.

All proposed alternatives are shown to be feasible from an economic viewpoint. The damage prone area reasonably wide and more than 6,000 households may sustain the damage caused by flooding and/or mudflow under the probable design flood.

However, it is difficult to find the significant difference to effectiveness for flood control ability among three alternatives under current river basin condition. Therefore, Alternative-1 is recommended as the most economical scheme among three alternatives.

Figure 1.8.1 and 1.8.2 are illustrated with bird-eye view sketch for the selected priority scheme, which is the heightening of the existing dikes in the Bucao River and strengthening of the existing dikes in the Sto. Tomas River, respectively.

CHAPTER 2 FEASIBILITY STUDY FOR THE BUCAO RIVER

2.1 General

Three alternatives of structural measures in the Bucao River have been formulated in the master plan study. In terms of economical assessment, Alternative-1 for "Heightening of Existing Dike" has been selected as the priority scheme to proceed to this feasibility study.

The main aim of this chapter is to propose an appropriate preliminary structural design for the Bucao River based on 1) structural recommendations in the master plan study and 2) results of relative studies obtained in the master plan study.

The structural measures for the preliminary design are itemized as follows:

1. New dike in the downstream from the Bucao Bridge
2. Heightening of the existing river dike and
3. Strengthening of the existing spur dike

Each preliminary design of structural measure is described with structural examinations as follows.

2.2 Design Condition in the Bucao River

(1) Design Discharge

Probable design discharge with a 20-year return period is applied to the Bucao River improvement works based on relation between required planning scale for determination of design discharge and probable direct damage occurrence.

Details of the provision of design discharge are described in "Section 3.2 in Appendix III".

The design discharge with planning scale of a 20-year return period is tabulated in relative reaches as follows:

River Sections	Design Discharge
From River mouth to Confluence of the Baquilan River (Sta.-2.4 km to Sta.+6.0 km)	3,800 m^3/s
From Baquilan River to Confluence of the Balin-Baquero River (Sta.+6.0 km to Sta.+11.2 km)	2,900 m^3/s
From the Balin-Baquero River (Upstream of Sta.+11.2 km)	1,300 m^3/s

(2) Proposed Dike Alignment

1) Downstream from the Bucao Bridge

This section from the river mouth to the Bucao Bridge does not have dike system yet in both banks at present. The purpose of proposed new dike is aimed to prevent flooding and/or mudflow coming into landside area in both banks.

Alignments of proposed new dike follow natural river terrace along the edge of the current watercourse.

The proposed lengths are for approximately 2.4 km long in right bank and 1.9 km in left bank.

2) From the Bucao Bridge to Confluence of the Baquilan River

This section follows the existing dike alignment, because the section has hydraulic required river width to flow design discharge at present and, the essential watercourse of the Bucao River is still not fixed yet caused by violent riverbed movement in the channel and it is intricate to expect future watercourse after 20 years.

Additionally, the current river width at vicinity of the Bucao Bridge is the narrowest in the Bucao River. The river width is about 300 m.

Commonly, the required river width to flow design discharge is shown by the engineering empirical standard (as following table) based on the existing river channel width investigations.

Design discharge (m^3/s)	**Required river width (m)**
300	40 to 60
500	60 to 80
1,000	90 to 120
2,000	160 to 220
5,000	350 to 450

Source: Technical Standards for River and Sabo Works, River Association of Japan

The above table shows that the river width at vicinity of the Bucao Bridge is sufficient to flow design discharge under the empirical standard

However, in the section of 1.7 km upstream from the existing dike end point (Sta.+4.8 km) new alignment is provided to protect the existing community road and existing irrigation channel from flooding and/or mudflow.

Figure 2.2.1 shows general plan of proposed river improvement in the Bucao River.

(3) Presumptive Riverbed Elevation after 20 years

The sediment deposits have still been deposited thick in the Bucao River channel since Mount Pinatubo eruption in 1991. The thickness has been range for a few meters to approx. 30 m in the upstream reaches. In rainy season, the sediment deposits are furiously swept and deposited or scoured caused by flooding and/or mudflow at present.

In order to provide proposed design water level, the riverbed movement after 20 years is computed with one-dimensional sediment transport analysis, and the results presume the design riverbed to compute proposed design water level. Figure 2.2.2 shows presumptive riverbed change after 20 years.

It is presumed that the riverbed is still in tendency of riverbed aggradation in the lower reaches of the Bucao River from Mount Pinatubo eruption, because there are huge amount of sediment deposits in middle- upper reaches of the Bucao River.

The future maximum riverbed aggradation may be about 4.0 m high from existing riverbed in 2002 between river mouth and the Baquilan River.

Details for the riverbed movement are described in “Section 3.3 in Appendix IV”.

(4) Design Water Level

In order to provide design dike crest elevation, design water level for the Bucao River is computed with non-uniform flow analysis based on the followings:

- Initial water level in river mouth corresponds with the maximum predicted tide occurring on

August 9 and 10, 2002. The water level is EL+1.43 m.

- Roughness coefficient is applied as n = 0.035.

The value of roughness coefficient in this feasibility study corresponds with the value adopted in " The Master Plan and Feasibility Study on Flood and Mudflow Control for the Sacobia-Bamban and Abacan Rivers draining from Mount Pinatubo" undertaken by JICA in May 1996, because above the project is neighborhood and similar project to this project and the project is contributing flood and/or mudflow management in the eastern Mount Pinatubo area.

Numerical proposed design water level is shown in Table 2.2.1.

Summary of the design water level at each significant point is tabulated as follows:

Station	Existing Riverbed in 2002	Presumptive Riverbed after 20 years	Design Water Level	Remarks
Sta.–2.40 km	EL+0.54 m	EL+1.40 m	EL+2.80 m	River Mouth
Sta.+0.00 km	EL+6.40 m	EL+10.16 m	EL+13.23 m	Bucao Bridge
Sta.+2.00 km	EL+13.64 m	EL+17.56 m	EL+18.90 m	
Sta.+4.00 km	EL+18.95 m	EL+21.92 m	EL+25.92 m	
Sta.+5.50 km	EL+26.80 m	EL+26.67 m	EL+30.92 m	Baquilan River (Right Bank)
Sta.+7.00 km	EL+34.01 m	EL+35.01 m	EL+35.92 m	
Sta.+10.00 km	EL+50.15 m	EL+51.69 m	EL+54.41 m	Malomboy (Right Bank)
Sta.+12.00 km	EL+63.80 m	EL+66.98 m	EL+69.00 m	Upper Bucao River

Note : Elevations of existing riverbed and presumptive riverbed indicate average values in the cross section.

Summary of presumed water depth between existing riverbed in 2002 and design water level, and presumed sediment deposit depth from existing riverbed are tabulated as follows:

River Stretch	Sediment Deposit Depth	Water Depth
River mouth to Bucao Bridge	Ave. 2.3 m	Ave. 4.4 m
Bucao Bridge to Baquilan River	Ave. 3.3 m	Ave. 6.5 m
Baquilan River to Malomboy	Ave. 1.0 m	Ave. 3.3 m
Malomboy to Upper Bucao River	Ave. 3.8 m	Ave. 6.1 m

Note : Base line is corresponding with existing riverbed in 2002.

(5) Freeboard

This is a margin against sudden overtopping wave in flooding. The required height of the freeboard is referred to the following table applied in the Philippines:

Item	Design discharge (m^3/s)	Required Freeboard
1	Less than 200	0.60 m
2	200 to less than 500	0.80 m
3	500 to less than 2,000	1.00 m
4	2,000 to less than 5,000	1.20 m
5	5,000 to less than 10,000	1.50 m
6	More than 10,000	2.00 m

Source: Design Guidelines Criteria and Standards for DPWH, Volume-II

The above table indicates that required freeboard is 1.20 m under the design discharge as Q = 3,800 m^3/s.

Figure 2.2.3 shows longitudinal profile of proposed river improvement in the Bucao River.

(6) Design Dike Crest Width

Provision of design dike width is to provide adequate dike width against seepage failure and the adequate width of the top embankment is also required to serve as a road for facilitating the transport of materials during the construction stage and maintenance operations.

According to as-built drawings for dike construction, obtained from DPWH Region-III, recent constructed dikes have dike crest width for 8.0 m wide in this study area.

To correspond with recent constructed dikes, proposed dike crest width is provided for 8.0 m wide.

For reference, the following table applied in the Philippines shows the required width:

Item	Design discharge (m^3/s)	Required Dike Crest Width
1	Less than 500	3.0 m
2	500 to less than 2,000	4.0 m
3	2,000 to less than 5,000	5.0 m
4	5,000 to less than 10,000	6.0 m
5	More than 10,000	7.0 m

Source: Design Guidelines Criteria and Standards for DPWH, Volume-II

(7) Required Dike Height

Above each design condition, required dike height in each section is summarized as following table:

River Stretch	Freeboard	Required Dike Height	Possible measure
River mouth to Bucao Bridge	1.2 m	Ave. 5.6 m	New Dike
Bucao Bridge to Baquilan River	1.2 m	Ave. 7.7 m	Dike Heightening

2.3 Preliminary Design for the Bucao River

2.3.1 New Dike in the Downstream Bucao Bridge

Proposed new dike is provided in the section, where there is no dike system at present, to protect flooding and/or mudflow coming into landside area.

The sections of proposed new dike are tabulated as follows:

Location	River Sections	Proposed Distance
Left Bank	River mouth to natural levee (Sta.-0.6 km)	1.91 km
	Vicinity of proposed Bucao Bridge	0.17 km
Right Bank	River mouth to Bucao Bridge (Sta.+0.0 km)	2.35 km

The slope protection is to protect lahar embankment from high flow velocity of flooding and/or mudflow caused by heavy rainfall in rainy season.

Proposed slope protections on the proposed new dike are grouted riprap in riverside and sodding in landside.

The grouted riprap for proposed slope protection is provided to existing dike protection along the Bucao

River, it is conceivable that boulder stone as main material of grouted riprap is available in the Bucao River neighborhood.

New dike is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	8 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike crest level
Side slope gradient	H : V= 2.0:1 (with revetment)
	Less than H : V = 3.0:1 (without revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Covering borrow soil and sodding
Provision of inspection road	Gravel pavement on the top of dike

(1) Proposed Riverside Slope Protection

Crest elevation of proposed slope protection is equivalent to design dike crest level. Stones provided in grouted riprap are recommended to expose to the watercourse, because the exposed stones are expected to reduce flow velocity of flooding and/or mudflow in vicinity of the slope protections.

Toe of slope protection is set for minimum 1.0 m deep from the existing riverbed to resist local scouring due to flooding and/or mudflow.

For embedment protection, an ancillary counter measure against local scouring in immediate toe of revetment recommends gabion mattress as foot protection. Gabion mattress has flexibility. If local scouring may proceed at the foot portion, gabion mattress can follow the some riverbed variations.

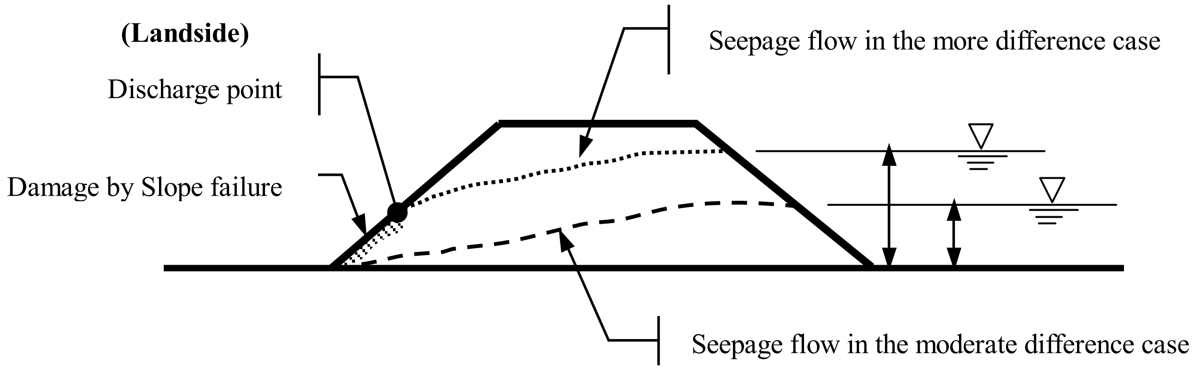
However, it is intricate to predict the local scouring depth with limited current survey and investigation results in this study. Hence, detail design stage is recommended to consider occurrence of local scouring and required embedment depth against local scouring in immediate toe of revetment with new required basic data (e.g. additional cross sectional survey, additional riverbed particle size investigation, additional hydraulic analysis, etc).

(2) Proposed Landside Slope Protection

According to the results of the riverbed movement analysis, in these sections, sediment deposits are presumed for 0.9 m to 3.8 m thick on the existing riverbed within 20 years. Because of above reason, difference between design water level and existing landside ground level will be appeared in the future flooding and the difference may be range for 2.3 m to 6.8 m high in the probable flooding.

Commonly, in the flooding stage, seepage flow caused by the difference is appeared in dike body. In case of the more difference, the seepage flow encouraged in dike body may appear on vicinity of landside slope (refer to the following figure).

The cohesion having dike embankment material under the phreatic surface of seepage flow is decreased in the more difference case.



Schematic Diagram for the Seepage Flow

If the dike size is smaller compared with the difference between water level and existing landside ground level, potential energy of seepage flow may encourage in the dike, seepage flow sweeps away embankment material at landside toe of the dike.

To prevent future occurrence of seepage failure in landside, sufficient seepage length against the seepage flow is recommended to provide with landside dike enlargement.

The landside dike enlargement is one of dominant measures against seepage failure. The landside dike enlargement is easy to provide embankment material with sediment deposits in riverside. Required landside slope gradient is provided with slope failure analysis.

The slope stability is assessed with stability analysis based on circular slip surface (circular arc analysis). This method is described in "Design Guidelines Criteria and Standards" for DPWH Volume-II.

The safety factor provision is recommended to describe as follow. The following equation refers to "Guideline of River Dike Design (June 2002) Ministry of Land, Infrastructure and Transport, Japan".

$Fs \geq 1.2$ (for riverside slope)

$Fs \geq 1.2 \times \alpha_1 \times \alpha_2$ (for landside slope)

where: Fs: safety factor for slope failure

α_1: coefficient of dike character

α_2: coefficient of foundation ground character

Coefficient of dike character (α_1)	**Correction**
In case of reinforced dike many times	$\alpha_1 = 1.2$
In case of reinforced dike several times	$\alpha_1 = 1.1$
In case of new dike	$\alpha_1 = 1.0$
Coefficient of foundation character (α_2)	
In case of critical damage occurrence before	$\alpha_2 = 1.1$
In case of no critical damage occurrence	$\alpha_2 = 1.0$

Based on results of field investigation for existing structures along the river, the safety factor of landside slope failure is recommended as FS = more than 1.32.

$Fs \geq 1.2 \times \alpha_1 \times \alpha_2 = 1.2 \times 1.1 \times 1.0 = 1.32$

For proposed embankment materials, geological investigation results (refer to "Section 2.2 in Appendix II") describe as follows:

The materials exhibit that soil particle size is almost uniform and the permeability coefficient is more than k = 1.0 x 10^{-3} cm/s around in accordance with fine sand. It is conceivable that the lahar material is comparatively more pervious material.

Figure 2.3.1 shows computed results for landside slope failure including estimated phreatic surface.

The computed results recommend that proposed landside slope gradient is less than H : V = 3.5 ~ 4.5:1 to satisfy required safety factor, because that different water depth is range for 2.3 m to 6.8 m deep.

Additionally, Figure 2.3.2 shows the manner of estimated phreatic surface in dike body. This manner is applied to the case that dike embankment is constructed with uniform soil particle size material This manner gives simply approximate height of phreatic surface in dike body from dike foundation ground.

However, it is intricate to predict phreatic surface in dike body, because of seepage flow relating flood duration time, characteristic of embankment material, condition of dike different locations, etc.

Therefore, it is recommended to analyze seepage flow in the dike with Finite Element Method (FEM) in detailed design stage, because the seepage flow movement is critical matter to determination of proposed landside slope gradient.

(3) Earthquake Case

In this study, it is conceivable that dike stability in earthquake case is not included based on following reasons:

- Proposed dike is basically constructed with lahar embankment material carried from vicinity of the dike location. If the dike may be damaged caused by earthquake, the dike will be easier to rehabilitate with same embankment material than concrete structures.
- There is little possibility of probable occurrence of flooding and earthquake in the same time.
- Result of water level investigation in 2002 shows that maximum water depth is about 0.3 m in rainy season excluding flooding and/or mudflow times.

Because of the above mention, it is conceivable that if proposed dike crest level cannot be preserved caused by earthquake, the damaged dike is still remain the dike height to prevent the river water from spreading in the landside area.

Additionally, investigation damaged river dikes caused by " The Great Hunshin- Awaji Earthquake" have reported the damaged dike height caused by earthquake is maintained for minimum 25 % height of constructed dike height in 1996 Japan.

Figure 2.3.3 shows typical cross section of proposed new dike.

2.3.2 Heightening of Existing River Dike

Proposed dike heightening is recommended to provide in this section, where the existing dike height is insufficient to protect flooding and/or mudflow coming into landside area.

The sections of proposed dike heightening are tabulated as follows:

Location	River Sections	Proposed Distance
Right Bank	Bucao Bridge (Sta.+0.0 km) to Sta.+5.0 km	5.80 km
Right Bank	Sta.+5.0 km upstream	1.65 km
Total		7.45 km

The slope protection works on existing dikes have been constructed along the Bucao River since Mount Pinatubo eruption. These are to protect lahar embankment from high flow velocity of flooding and/or mudflow caused by heavy rain in rainy season.

Proposed slope protections on the proposed dike are grouted riprap in riverside and sodding in landside.

Dike heightening is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	8 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike crest level
Side slope gradient	H : V = 2.0:1 (with revetment)
	Less than H : V = 3.0:1 (without revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Covering borrow soil and sodding
Provision of inspection road	Gravel pavement on the top of dike

(1) Proposed Riverside Slope Protection

Proposed riverside slope protections are corresponding with the protections for proposed new dike (refer to Section 2.3.1 in this Appendix).

(2) Proposed Landside Slope Protection

The results of the riverbed movement analysis show maximum sediment deposits may be presumed for about 4.0 m high from existing riverbed within 20 years in these sections. Consequently, the difference between design water level and existing landside ground is maximum 7.1 m high.

Because of above reason, the computed results for landside slope failure (refer to Figure 2.3.1) show that recommended landside slope gradient is less than H : V = 4.5 :1 to satisfy required safety factor, because that different water depth is maximum 7.6 m deep.

Figure 2.3.4 shows typical cross sections of proposed dike heightening.

(3) Earthquake Case

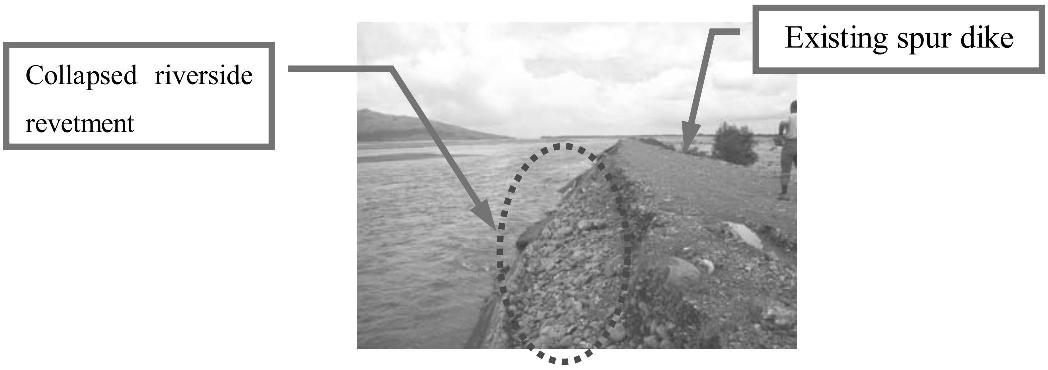
The consideration in earthquake case is corresponding with provisions for proposed new dike (refer to Section 2.3.1 in this Appendix).

2.3.3 Strengthening of Existing Spur Dike

(1) Existing Spur Dike

There is an existing spur dike in upper end portion of existing river dike. The existing spur dike length is about 200 m. The spur dike is accelerated to control the watercourses from the Baquilan River and upper reaches of the Bucao River. Consequently, the main watercourse in downstream from the spur dike is swerved effectively to foot of the northern mountains and a part of existing river dike is protected by effectiveness of that.

However, the existing riverside protections of spur dike are damaged due to annual flood in rainy season. Therefore, at present, the collapsed riverside revetments and dike embankment exposed to watercourse are still observed. The following photo (taken on July 29, 2002) shows present damages of the existing spur dike:



Proposed spur dike strengthening is recommended to maintain the function of the spur dike with rehabilitation of riverside revetments.

(2) Proposed Spur Dike Strengthening

Spur dike strengthening is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	6 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike water level
Side slope gradient	H : V = 2.0:1 (with revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Grouted riprap or equivalent
Provision of inspection road	Grouted riprap or equivalent

Proposed riverside slope protections are corresponding with the protections for proposed new dike (refer to Section 2.3.1 in this Appendix).

The design dike crest level corresponds to proposed design water level because the spur dike is commonly placed in the channel to control the direction of watercourse.

For landside slope protections, to avoid local scouring in the landside due to spilled water during flood, the protection is recommended to correspond with the riverside slope protections.

Figure 2.3.5 shows typical cross section of proposed spur dike strengthening.

CHAPTER 3 FEASIBILITY STUDY FOR THE STO. TOMAS RIVER

3.1 General

Three alternatives of structural measures in the Sto.Tomas River have been formulated in the master plan study. In terms of economical assessment, Alternative-1 for "Heightening/ Strengthening of Existing Dike" has been selected as the priority scheme to proceed to this feasibility study.

The main aim of this chapter is to propose an appropriate preliminary structural design for the Sto.Tomas River based on 1) structural recommendations in the Master Plan Study and 2) results of relative studies obtained in the master plan study.

The structural measures for the preliminary design are itemized as follows:

1. New dike in the downstream from the Maculcol Bridge
2. Heightening of the existing river dike
3. Strengthening of the existing river dike and
4. Diversion channel for the Gabor River

Each preliminary design of proposed structural measure is described with structural examinations as follows.

3.2 Design Condition in the Sto.Tomas River

(1) Design Discharge

Probable design discharge with a 20-year return period is applied to the Sto. Tomas River improvement works based on relation between planning scale for determination of design discharge and probable direct damage occurrence.

Details for the provision of design discharge are described in "Section 3.2 in Appendix III".

The design discharge with planning scale of a 20-year return period is tabulated in relative reaches as follows:

River Sections	Design Discharge
From River mouth to Confluence of the Santa Fe River (Sta.-1.5 km to Sta.+11.5 km)	1,200 m^3/s
From the Santa Fe River to Confluence of Lake Mapanupe (Sta.+11.5 km to Sta.+21.0 km)	860 m^3/s
the Marella River (Upstream of Sta.+21.0 km)	680 m^3/s

(2) Proposed Dike Alignment

1) Downstream from the Maculcol Bridge

This section from river mouth to the Maculcol Bridge does not have dike system yet in right bank at present. The purpose of proposed new dike is aimed to prevent flooding and/or mudflow coming into landside area in right bank.

Alignment of proposed new dike in right bank follows natural river terrace along the edge of current watercourse. The length is approximately 1.9 km.

In left bank, there is existing dike constructed by DPWH with lahar sediment deposits along the watercourse.

Alignment of proposed dike strengthening in left bank is following existing dike alignment.

2) the Maculcol Bridge to Vega Hill

For the vicinity of Maculcol Bridge, proposed dike alignment recommended to widen for about 30 m in both banks, based on reconstruction plan of the Maculcol Bridge.

The reconstruction plan of the Maculcol Bridge has been planned undertaken by DPWH Region-III. According to the detailed drawings of the New Maculcol Bridge, the proposed bridge length is longer than the existing bridge for about 60 m long. The new bridge length is 430.8 m.

This new bridge expansion shows that it is difficult to adopt a drastic improvement of right existing dike alignment in immediately upstream of the bridge, but existing river width widening for about 30 m in both banks can be adopted to control the watercourse as well as possible.

For other section, current river width has hydraulic required river width to flow design discharge and it is not necessary to widen existing river width. Alignments of proposed dike heightening and dike strengthening are following existing dike alignment.

3) Vega Hill to Lawin (End point of existing dike)

In this section, existing dike has already been constructed since Mount Pinatubo eruption in left bank. Current river width in this section has larger flow section than lower reaches. Thus alignment of dike heightening is following the existing dike alignment.

Figure 3.2.1 shows general plan of proposed river improvement in the Sto. Tomas River.

(3) Presumptive Riverbed Elevation after 20 years

The sediment deposits have still been deposited thick in the Sto. Tomas River and the Marella River channels since Mount Pinatubo eruption in 1991. The thickness has been range for a few meters to approx. 30 m in the upstream reach. In rainy season, the sediment deposits are furiously swept and deposited or scoured caused by flooding and/or mudflow at present.

In order to provide design water level, riverbed movement after 20 years is computed with one-dimensional sediment transport analysis, and the results presume the design riverbed to compute proposed design water level. However, the presumed design riverbed is not always corresponding with predicted riverbed after 20 years.

As the results, Figure 3.2.2 shows presumptive riverbed change after 20 years.

It is presumed that the riverbed is still in rising tendency between river mouth and the Santa Fe River. The maximum riverbed aggradation may be about 1.5 m high on the existing riverbed in 2002.

The other hand, the section between the Santa Fe River and outlet of Lake Mapanuepe shows that the riverbed movement is aggradation and/or degradation, repeatedly. The maximum riverbed aggradation is about 0.7 m upward, and the maximum riverbed degradation is about 2.0 m downward from existing riverbed in 2002, respectively.

Details for the riverbed movement are described in “Section 3.3 in Appendix IV”.

(4) Design Water Level

In order to provide design dike crest level, design water level of the Sto. Tomas River is computed with non-uniform flow analysis in accordance with the design condition for the Bucao River.

Numerical proposed design water level is shown in Table 3.2.1. Summary of the design water level at each significant point is tabulated as follows:

Station	**Existing Riverbed in 2002**	**Presumptive Riverbed after 20 years**	**Design Water Level**	**Remarks**
Sta.–1.50 km	EL+1.98 m	EL+1.98 m	EL+3.95 m	River Mouth
Sta.+0.00 km	EL+6.42 m	EL+7.25 m	EL+8.95 m	Maculcol Bridge
Sta.+7.25 km	EL+27.03 m	EL+28.26 m	EL+29.68 m	Paete Hill (Right Bank)
Sta.+10.50 km	EL+41.46 m	EL+40.89 m	EL+42.48 m	Vega Hill (Left Bank)
Sta.+11.50 km	EL+46.65 m	EL+46.12 m	EL+46.83 m	Santa Fe River (Right Bank)
Sta.+18.00 km	EL+90.22 m	EL+90.04 m	EL+90.66 m	Lawin (Left Bank)

Note : Elevations of existing riverbed and presumptive riverbed indicate average values in cross section.

Summary of presumed water depth, between existing riverbed in 2002 and design water level, and presumed sediment deposit depth from existing riverbed is tabulated as follows:

River Stretch	**Sediment Deposit Depth**	**Water Depth**
River mouth to Maculcol Bridge	Ave. 0.3 m	Ave. 2.1 m
Maculcol Bridge to Paete Hill	Ave. 0.8 m	Ave. 2.5 m
Paete Hill to Vega Hill	Ave. 0.5 m	Ave. 2.0 m
Vega Hill to Lawin	Ave. -0.4 m	Ave. 0.6 m

Note : Base line is corresponding with existing riverbed in 2002.

(5) Freeboard

The table applied in the Philippines (refer to Section 2.2 in this Appendix) indicates that required freeboard is 1.00 m under probable design flood of Q_{20} = 860 and 1,200 m^3/s.

Figure 3.2.3 shows longitudinal profile of proposed river improvement in the Sto. Tomas River.

(6) Design Dike Crest Width

The design dike crest width is applied for 8.0 m wide (refer to Section 2.2 in this Appendix).

(7) Required Dike Height

Above each design condition, required dike height in each section is summarized as following table:

River Stretch	**Freeboard**	**Required Dike Height**	**Possible measure**
River mouth to Maculcol Bridge	1.0 m	Ave. 3.1 m	New Dike/ Dike Strengthening
Maculcol Bridge to Paete Hill	1.0 m	Ave. 3.5 m	Dike Heightening/ Dike Strengthening
Paete Hill to Vega Hill	1.0 m	Ave. 3.0 m	Dike Heightening
Vega Hill to Lawin	1.0 m	Existing Dike Height	Dike Strengthening

3.3 Preliminary Design of the Sto. Tomas River

3.3.1 New Dike in Downstream from the Maculcol Bridge

Proposed new dike is provided in the section, where there is no dike system at present, to protect flooding and/or mudflow coming into landside area.

The section of proposed new dike is tabulated as follows:

Location	River Section	Proposed Distance
Right Bank	River mouth to Maculcul Bridge	1.95 km

The slope protection is to protect lahar embankment from high flow velocity of flooding and/or mudflow caused by heavy rainfall in rainy season.

Proposed slope protections on the proposed new dike are grouted riprap in riverside and sodding in landside.

New dike is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	8 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike crest level
Side slope gradient	H : V = 2.0:1 (with revetment)
	Less than H : V = 3.0:1 (without revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Covering borrow soil and sodding
Provision of inspection road	Gravel pavement on the top of dike

(1) Proposed Riverside Slope Protection

Proposed riverside slope protections are corresponding with slope protections for proposed new dike in the Bucao River (refer to Section 2.3.1 in this Appendix).

(2) Proposed Landside Slope Protection

The results of the riverbed movement analysis show maximum sediment deposits is presumed for about 0.8 m upward from existing riverbed within 20 years in these sections. Consequently, difference between design water level and existing landside ground is maximum 2.5 m high.

Because of above reason, the computed results for landside slope failure (refer to Figure 2.3.1) show that proposed landside slope gradient is less than V: H = 1: 3.5 to satisfy required safety factor, because that different water depth is maximum 2.5 m deep.

Figure 3.3.1 shows typical cross section of proposed new dike.

3.3.2 Heightening of Existing River Dike

Proposed dike heightening is provided in the section, where the existing dike height is insufficient to protect flooding and/or mudflow coming into landside area. The sections of proposed dike heightening are tabulated as follows:

Location	River Sections	Proposed Distance
Left Bank	Sta+1.5 km to Vega Hill (Sta.+10.5 km)	9.00 km
Right Bank	Sta.+3.0 km to Paete Hill (Sta.+7.3 km)	4.30 km
Total		13.30 km

The slope protection works on existing dikes have been constructed along the Sto. Tomas River since Mount Pinatubo eruption. These are to protect lahar embankment from high flow velocity of flooding and/or mudflow caused by heavy rain in rainy season.

Proposed slope protections are grouted riprap in riverside and sodding in landside.

Dike heightening is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	8 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike crest level
Side slope gradient	H : V = 2.0:1 (with revetment)
	Less than H : V = 3.0:1 (without revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Covering borrow soil and sodding
Provision of inspection road	Gravel pavement on the top of dike

(1) Proposed Riverside Slope Protections

Proposed riverside slope protections are corresponding with slope protections for proposed new dike in the Bucao River (refer to Section 2.3.1 in this Appendix).

(2) Proposed Landside Slope Protections

The results of the riverbed movement analysis show maximum sediment deposits may be presumed for about 1.5 m high on the existing riverbed within 20 years in these sections. Consequently, difference between design water level and existing landside ground is maximum 3.2 m high.

According to computed results for landside slope failure (refer to Figure 2.3.1), it is recommended that proposed landside slope gradient is range for V: H = 1: 4.0 to 4.5 to satisfy required safety factor, because that different water depth is range for 4.0 m to 6.5 m deep.

Figure 3.3.2 shows typical cross section of proposed dike heightening.

3.3.3 Strengthening of Existing River Dike

Proposed dike strengthening is provided in the section, where the existing dike height is sufficient to protect flooding and/or mudflow.

However, it can be observed that:

- Some portions without riverside revetments are damaged caused by annual flooding and/or mudflow in lower reaches of the river.
- Landside slopes are damaged caused by seepage flow in dike body along existing dike in upper reaches of the river.

Thus strengthening of existing river dike is recommended to be rehabilitation of existing dikes against flow velocity of flooding and/or mudflow and landside slope failure caused by seepage flow.

The sections of proposed dike strengthening are tabulated as follows:

Location	River Sections	Proposed Distance
Left Bank	River mouth to Sta. +1.5 km	2.80 km
	Vega Hill to Lawin	8.00 km
Right Bank	Maculcol Bridge to Sta. +3.0 km	3.10 km
Total		13.90 km

In these sections, there are two kinds of existing revetment, concrete facing and gabion mattress, respectively.

Dike strengthening is proposed in accordance with following design dike dimensions:

Structural Item	Description
Top width of dike	8 m (in accordance with existing dike crest width)
Proposed dike height	In accordance with design dike crest level and/or existing dike crest
Side slope gradient	H : V = 2.0:1 (with revetment)
	Less than H : V = 3.0:1 (without revetment)
Embankment material	Lahar sediment deposits (principally)
Slope protection (Riverside)	Grouted riprap or equivalent
Slope protection (Landside)	Covering borrow soil and sodding
Provision of inspection road	Gravel pavement on the top of dike

(1) Between Maculcol Bridge to Sta.+3.0 km in Right Bank

1) Proposed Riverside Slope Protections

The existing gabion mattress facing have been constructed in 2001 between the Maculcol Bridge and Sta.+3.0 km in right bank.

The revetments are newest in the Sto. Tomas River. However, design velocity under probable design flood is a 2.0 m/s around. It is possible that abrasion caused by sweeping sediment deposits breaks main steel wire of existing gabion mattress, further, the gabion mattress may be damaged.

Thus existing gabion mattress protection is recommended to replace with grouted riprap, which is more durability than gabion mattress.

Proposed riverside slope protections are corresponding with slope protections for proposed new dike in the Bucao River (refer to Section 2.3.1 in this Appendix).

2) Proposed Landside Slope Protections

The results of the riverbed movement analysis show that maximum sediment deposits is presumed for about 1.5 m high on the existing riverbed within 20 years. Consequently, difference between design water level and existing landside ground is maximum 3.5 m high.

According to computed results for landside slope failure (refer to Figure 2.3.1), it is recommended that proposed landside slope gradient is V: H = 1: 4.0 to satisfy required safety factor, because that the different water depth is maximum 3.5 m deep.

(2) Between Vega Hill to Lawin in Left Bank

1) Riverside Slope Protections

The existing concrete facing have been constructed since 1997 to 1998 between Vega Hill to Lawin in left bank.

It is conceivable that the revetment is able to resist flow velocity caused by flooding and/or mudflow and attrition caused by sweeping sediment, because the revetment is new constructed comparatively and the damage portions are not observed at present.

The existing riverside revetments in these sections are preserved as the riverside slope protections.

2) Proposed Landside Slope Protection

The results of the riverbed movement analysis show that maximum sediment deposits is presumed for less than 1.0 m high on the existing riverbed within 20 years.

However, it is observed that difference between existing riverbed and existing landside is higher than the other sections in the Sto. Tomas River. The different height is range for 5.0 to 11.0 m high at present.

Figure 3.3.3 shows site inspection results of different height between existing riverbed and existing landside ground.

Because of above mention, there is some possibility of huge seepage failure occurrence.

To prevent the occurrence of seepage failure in landside, landside dike enlargement against seepage flow is proposed to provide.

According to computed results for landside slope failure (refer to Figure 2.3.1), it is recommended that proposed landside slope gradient is range for V: H = 1: 4.0 to 5.0 to satisfy required safety factor, because that the different water depth is range for 7.0 m to 12.0 m deep.

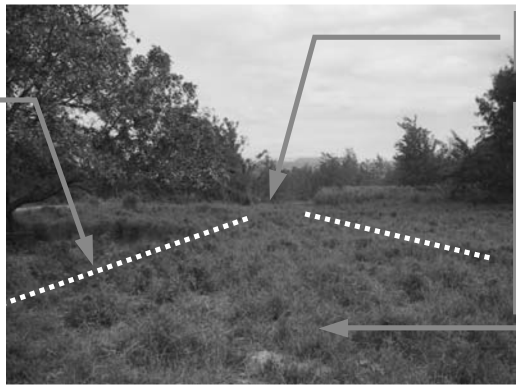
Figure 3.3.4 shows typical cross sections of proposed dike heightening in above-mentioned sections.

3.3.4 Diversion Channel of the Gabor River

(1) Existing Condition

The Gabor River is one of tributaries of the Sto. Tomas River in right downstream from Maculcol Bridge. The confluence point of the Gabor River has been quite buried by sediment deposits for about 1.5 m deep because of repeated riverbed movement in the Sto. Tomas River after Mount Pinatubo eruption. The clogging at the river mouth due to sediment deposits occurs inundation in vicinity of downstream from the Gabor River due to overflowing from the river in rainy season.

The following photo (taken on February 12, 2003) shows present state of river mouth clogging in the Gabor River:



Furthermore, according to the results of riverbed movement analysis in the Sto. Tomas River, in this reach, sediment deposits are predicted for about 0.3 m high from existing riverbed. Total sedimentation depth may be about 1.8 m.

(2) Countermeasure

As an above result, it is difficult to preserve the Gabor River function as acceleration of discharge into the Sto. Tomas River in the original confluence point because of the riverbed aggradation in the Sto. Tomas River.

Hence, the diversion channel system forward to seashore along proposed dike is recommended instead of the confluence river system. The shape of proposed diversion channel is trapezoidal open channel.

Additionally, the Gabor River channel in downstream of Gabor Bridge is not affected by clogging at the river mouth based on field investigation. Therefore, proposed improvement stretch is about 1.7 km long from the seashore.

Figure 3.3.5 shows general plan of proposed diversion channel of the Gabor River.

(3) Design Discharge

For design discharge, the standard for DPWH shows that kinds of drainage channel divide into three classifications as follows:

Kinds of drainage channel	Required design scale	Remarks
Open waterways	For a 50-year return period	
Roadside drainage channel	For a 10-year return period	
Others	For a 10-year return period	

Source : Design Guidelines Criteria and Standards for DPWH, Volume-II, Section 3.121.

Above table shows that design scale of proposed diversion channel is applied as a 10-year return period.

The probable mean daily rainfall in the Sto. Tomas River basin is tabulated as follows:

Basin	Return Period (mm/day)						
	2 yr	5 yr	10 yr	20 yr	30 yr	50 yr	100 yr
Sto. Tomas	202	305	395	500	568	665	814

Design discharge of proposed diversion channel is estimated at 112.1 m^3/s with the following equation:

$$Q_{10} = \frac{1}{3.6} C \times R \times A = 112.1 \ m^3/s$$

where:

Q_{10} : design discharge (m^3/s)

C : runoff coefficient (= 0.75 as paddy field)

R : design rainfall intensity (= 16.5 mm/hr: 10-year return period)

A : catchment area (= 32.6 km^2)

(4) Preliminary Design

Proposed design water level and channel section scales are provided with non-uniform flow calculation under the condition that computed flow velocity in the each section is less than 3.0 m/s in accordance with the design criteria for DPWH.

Additionally, the roughness coefficient applied to the flow calculation is n=0.035 except proposed box culvert section.

For freeboard of diversion channel, because the design discharge is less than 200 m^3/s, the freeboard of open channel section is adopted for 0.6 m high. The height of barrel in proposed box culvert is designed that the design flood water level is set at 80% height of barrel to provide suitable freeboard.

On the diversion channel bank, proposed bank protection of grouted riprap is provided because that proposed flow velocity of the diversion channel is a 2.20 m/s around and the proposed revetment avoids bank erosion caused by flow velocity.

Figure 3.3.6, 3.3.7 and Table 3.3.1 show longitudinal profile of proposed diversion channel, typical cross sections of proposed diversion channel and numerical design water level, respectively.

Tables

Table 1.2.1 Possible Structural Measures in Western River Basins of Mount Pinatubo

ZONE	STRUCTURAL MEASURE	PURPOSE	BUCAO RIVER BASIN		MALOMA RIVER BASIN		SANTO TOMAS RIVER BASIN	
			Dimension/Component	Evaluation	Dimension/Component	Evaluation	Dimension/Component	Evaluation
Sediment Source Zone	Strengthening of Notch	To protect from the further erosion at overflow section of the Maraunot Notch.	Three Alternatives for outlet work: (1) Gabion mattress, (2) Concrete weir, and (3) Discharge tunnel	The plans (1) and (2) are recommendable if the geological condition is rigid/stable.	N.A.	The river originates at the lower part of slope of Mt.Pinatubo.	N.A.	There is no collapse of crater at the uppermost stretch of the Marella River.
	Re-vegetation	To prevent gully erosion, To accelerate catchment conservation	N.A.	91% mountain slope has already become stable. Thus, not applicable.	N.A.	There is no unstable mountain slope in the upper catchment area.	N.A.	95% mountain slope has already become stable.
	Small-scale Sabo Dam	To trap sediment from small-scale tributaries.	N.A.	Sediment control effect is small.	N.A.	Sediment control effect is small.	N.A.	Sediment control effect is small.
	Large-scale Sabo Dam	To trap sediment from main tributaries, To stabilize unstable lahar deposits	Six large-scale sabo dam sites were identified. The priority for development was evaluated.	The construction cost for foundation underneath dam is remarkably high because of thick lahar deposits at dam site.	N.A.	The current problem is flood inundation in the lower stretch, rather than sedimentation.	N.A.	A sabo dam site was identified at the Marella River, which was recommended by the RAP in 1994. However, it is not economical.
Sediment Deposition/ Secondary Erosion Zone	Consolidation Dam	To stabilize in-channel deposition	Consolidation dam at the Malumboy is proposed to stabilize the unstable sediment.	The dam is able to stabilize the unstable sediment of more than 300 million m^3.	N.A.	Same as above.	Consolidation dam is proposed to stabilize the unstable lahar deposits in the Marella River.	It is important to stabilize the unstable lahar deposits and to fix a river channel.
	Sand Pocket	To trap remobilized lahar deposits	Sand pocket at down-stream of the Malumboy is effective in trapping re-mobilized sediment.	The sand pocket can trap the remarkable volume of remobilized sediment.	N.A.	Same as above.	Sand pocket is proposed in the vicinity of Mt.Bagang.	To avoid the collapse of the Mapanuepe Lake, this has large flood control effect.
	Groundsill	To regulate secondary erosion of in-channel deposition To fix riverbed elevation	Lateral dikes as part of sand pocket are provided to fix the river channel.	Lateral dike functions as groundsill.	N.A.	Same as above.	A series of groundsills are provided at training channel mentioned below.	To maintain the riverbed elevation of training channel. To avoid shifting a channel
	Channel Training Works	To fix river channel To reduce in-channel sediment deposition	Openings of lateral dike of sand pocket are provided at left bank.	The river channel should be fixed at left bank along mountain side to protect right bank.	River channel improve-ment works are provided for lower stretch.	Widening/Straightening of river channel is required for ensure enough flow capacity.	Training channel is proposed in the vicinity of Mt.Bagang to fix river channel.	To avoid the collapse of the Mapanuepe Lake, this has large flood control effect.
Sediment Transport Zone	Channel Excavation	To maintain flow capacity of river channel	Maintenance excavation is required, if necessary, until the new bridge is constructed.	No clogging of river channel is identified at river mouth.	N.A.	No clogging is identified at river mouth.	Maintenance excavation is required until the new bridge is constructed.	No clogging of river channel is identified at river mouth.
	Dike	To protect inland from flood/ mudflow.	Raising/Strengthening of existing dike	Strengthening of existing dike is required to avoid the breach of the dike.	Dike is provided as part of river channel improvement.	Widening/Straightening of river channel is required for ensure enough flow capacity.	Raising/Strengthening of existing dike	Strengthening of existing dike is required to avoid the breach of the dike.
	Spur Dike	To control flow direction To protect from local scouring.	Spur dikes were provided to fix a river channel at left bank.	The location of spur dike should be determined based on the monitoring of flood flow condition	Spur dikes were provided to protect the dike from local scouring.	The location of spur dike should be determined based on the monitoring of flood flow condition	Spur dikes were provided to fix a river channel apart from the left bank.	The location of spur dike should be determined based on the monitoring of flood flow condition

Table 2.2.1 Numerical Proposed Design Water Level in the Bucao River

the Bucao River
Q=3,800m3/s (20yr Probable Flood Discharge)

Sta. (km)	Accumulated Distance (m)	Existing Riverbed in 2002 (EL-m)	Existing Dike Crest (EL-m)	Presumed Riverbed after 20 yrs (EL-m)	Design Discharge (m^3/s)	Design Water Level (EL-m)	Design Slope Grade of D.W.L	Design Dike Crest (EL-m)	Remarks
-3.00		-10.00							
-2.40	0	0.54	3.03	1.40	3,800	**2.80**	1/ 230	4.00	River Mouth (Sta. -2.4km)
-2.20	200	1.00	1.50	2.33	3,800	3.67		4.87	
-2.00	400	1.49	3.00	3.25	3,800	4.54		5.74	
-1.75	650	2.20	5.10	4.13	3,800	5.63		6.83	
-1.50	900	2.91	5.10	5.01	3,800	6.71		7.91	
-1.25	1,150	3.55	5.70	5.81	3,800	7.80		9.00	
-1.00	1,400	4.25	6.20	6.61	3,800	8.89		10.09	
-0.85	1,550	4.60	6.20	7.20	3,800	9.54		10.74	
-0.70	1,700	5.05	6.38	7.78	3,800	10.19		11.39	
-0.55	1,850	5.30	6.57	8.18	3,800	10.84		12.04	
-0.40	2,000	5.58	6.63	8.57	3,800	11.50		12.70	
-0.35	2,050	5.73	6.69	8.90	3,800	11.71		12.91	
-0.30	2,100	5.85	6.75	9.23	3,800	11.93		13.13	
-0.25	2,150	5.98	6.82	9.53	3,800	12.15		13.35	
-0.20	2,200	6.11	6.88	9.55	3,800	12.37		13.57	
-0.15	2,250	6.18	6.94	9.75	3,800	12.58		13.78	
-0.10	2,300	6.25	10.66	9.95	3,800	12.80		14.00	
-0.05	2,350	6.33	10.66	10.15	3,800	13.02		14.22	
0.00	2,400	6.40	10.66	10.16	3,800	13.23		14.43	Bucao Bridge (Sta. 0.0km)
0.05	2,450	6.63	10.62	10.42	3,800	13.45		14.65	
0.10	2,500	6.86	10.58	10.68	3,800	13.67		14.87	
0.20	2,600	7.09	10.49	10.94	3,800	**14.10**	1/ 375	15.30	
0.30	2,700	7.35	10.41	11.26	3,800	14.37		15.57	
0.40	2,800	7.59	10.33	11.57	3,800	14.64		15.84	
0.60	3,000	8.09	10.16	11.47	3,800	15.17		16.37	
1.00	3,400	9.80	13.07	13.00	3,800	16.24		17.44	
1.50	3,900	11.32	15.72	15.09	3,800	17.57		18.77	
2.00	4,400	13.64	18.89	17.56	3,800	**18.90**	1/ 285	20.10	
3.00	5,400	16.65	21.36	20.71	3,800	22.41		23.61	
4.00	6,400	18.95	24.20	21.92	3,800	**25.92**	1/ 300	27.12	
5.00	7,400	24.80	28.14	25.40	3,800	29.26		30.46	
5.50	7,900	26.80	31.80	26.67	3,800	30.92		32.12	Baquilan River (Sta. 5.5km)
6.00	8,400	28.75		27.94	2,900	32.59			
6.50	8,900	30.89		31.54	2,900	34.26			
7.00	9,400	34.01		35.01	2,900	**35.92**	1/ 175		
7.50	9,900	37.06		37.44	2,900	38.78			
8.00	10,400	39.65		40.14	2,900	41.64			
8.25	10,650	40.57		41.42	2,900	43.06			
8.50	10,900	41.79		42.66	2,900	44.49			
8.75	11,150	43.31		44.13	2,900	45.92			
9.00	11,400	44.05		45.19	2,900	47.35			
9.25	11,650	44.97		46.87	2,900	48.78			
9.50	11,900	46.19		48.72	2,900	50.21			
9.75	12,150	47.71		49.28	2,900	**51.64**	1/ 90		
10.00	12,400	50.15		51.69	2,900	54.41			Malomboy (Sta. 10.0km)
10.25	12,650	51.67		54.31	2,900	57.19			
10.50	12,900	53.20		56.67	2,900	59.97			
10.75	13,150	54.72		58.53	2,900	**62.75**	1/ 200		
11.00	13,400	57.33		61.70	2,900	64.00			
11.50	13,900	60.38		65.06	2,900	66.50			
12.00	14,400	63.80		66.98	2,900	69.00			Upper Bucao River (Sta. 12.0km)
13.00	15,400	71.02		73.12	1,500	**74.00**	1/ 110		
14.00	16,400	80.11		81.73	1,500	83.09			

Table 3.2.1 Numerical Proposed Design Water Level in the Sto. Tomas River

the Sto. Tomas River
Q=1,200m3/s (20yr Probable Flood Discharge)

Sta. (km)	Accumulated Distance (m)	Existing Riverbed in 2002 (EL-m)	Existing Dike Left Bank (EL-m)	Existing Dike Right Bank (EL-m)	Presumed Riverbed after 20 yrs (EL-m)	Design Discharge (m^3/s)	Design Water Level (EL-m)	Design Slope Grade of D.W.L	Design Dike Crest (EL-m)	Remarks
-2.00		-10.00								
-1.50	0	1.98	2.23		1.98	1,200	**3.95**	1/ 300	4.95	River Mouth
-1.25	250	3.01	10.54		3.13	1,200	4.78		5.78	
-1.00	500	3.76	10.08		4.00	1,200	5.62		6.62	
-0.85	650	4.26	12.35		4.57	1,200	6.12		7.12	
-0.60	900	5.09	11.51		5.34	1,200	6.95		7.95	
-0.50	1,000	5.93	8.93		5.79	1,200	7.28		8.28	
-0.30	1,200	5.86	9.46	8.05	6.30	1,200	7.95		8.95	
0.00	1,500	6.42	10.82	7.54	7.25	1,200	8.95		9.95	Maculcol Bridge
0.13	1,630	6.60	9.79	10.87	7.69	1,200	9.38		10.38	
0.33	1,830	7.11	12.97	11.24	8.37	1,200	10.05		11.05	
0.50	2,000	7.43	13.67	11.90	8.90	1,200	**10.62**	1/ 420	11.62	
1.00	2,500	9.02	13.70	13.22	10.05	1,200	11.81		12.81	
1.50	3,000	10.38	13.53	14.91	11.38	1,200	13.00		14.00	
2.00	3,500	12.49	15.17	16.46	12.76	1,200	14.19		15.19	
2.50	4,000	13.43	17.47	18.27	13.77	1,200	15.38		16.38	
3.00	4,500	14.88	17.39	17.77	15.15	1,200	**16.57**	1/ 320	17.57	
3.50	5,000	15.40	18.37	18.71	16.82	1,200	18.13		19.13	
4.00	5,500	17.65	19.84	19.93	18.62	1,200	19.69		20.69	
4.50	6,000	19.02	20.74	20.42	19.98	1,200	21.26		22.26	
5.00	6,500	20.17	23.41	22.61	21.17	1,200	22.82		23.82	
5.25	6,750	21.50	23.68	24.91	21.88	1,200	23.60		24.60	
5.50	7,000	21.91	24.32	23.95	22.36	1,200	**24.38**	1/ 330	25.38	
5.75	7,250	23.16	24.65	24.65	23.00	1,200	25.14		26.14	
6.00	7,500	22.99	25.63	25.23	23.58	1,200	25.90		26.90	
6.25	7,750	24.11	26.20	25.90	24.56	1,200	26.65		27.65	
6.50	8,000	24.70	26.51	26.34	25.49	1,200	27.41		28.41	
6.80	8,300	25.24	27.58	27.78	26.36	1,200	28.32		29.32	
7.00	8,500	26.36	26.87	27.50	27.19	1,200	28.93		29.93	
7.25	8,750	27.03	28.16	27.94	28.26	1,200	29.68		30.68	Paete Hill
7.50	9,000	28.12	28.31		29.40	1,200	**30.44**	1/ 260	31.44	
7.70	9,200	28.48	31.19		29.93	1,200	31.21		32.21	
8.00	9,500	30.29	33.79		31.37	1,200	32.37		33.37	
8.50	10,000	32.53	35.92		33.25	1,200	34.29		35.29	
9.00	10,500	35.03	37.03		35.34	1,200	36.21		37.21	
9.50	11,000	36.91	38.78		36.71	1,200	**38.13**	1/ 230	39.13	
10.00	11,500	38.20	39.50		38.57	1,200	40.31		41.31	
10.50	12,000	41.46	47.11		40.89	1,200	42.48		43.48	Vega Hill
11.00	12,500	44.02	47.08		43.54	1,200	44.66		45.66	
11.50	13,000	46.65	49.80		46.12	1,200	46.83		47.83	Santa Fe River
12.00	13,500	48.92	52.35		48.01	860	**49.00**	1/ 170	50.00	
12.50	14,000	51.53	55.28		50.82	860	51.95		52.95	
13.00	14,500	54.65	58.37		53.75	860	54.89		55.89	
13.50	15,000	57.34	61.05		56.07	860	57.83		58.83	
14.00	15,500	60.06	65.28		59.37	860	60.77		61.77	
14.50	16,000	63.62	68.95		63.19	860	**63.71**	1/ 140	64.71	
15.00	16,500	67.25	71.89		66.49	860	67.28		68.28	
15.50	17,000	69.30	77.00		70.00	860	70.85		71.85	
16.00	17,500	73.19	80.56		73.91	860	74.42		75.42	
16.50	18,000	76.92	84.06		77.28	860	78.00		79.00	
17.00	18,500	81.26	87.55		80.55	860	**81.57**	1/ 110	82.57	
17.50	19,000	84.83	92.32		85.10	860	86.11		87.11	
18.00	19,500	90.22	93.29		90.04	860	**90.66**	1/ 130	91.66	
18.50	20,000	95.28			93.25	860	94.50			
19.00	20,500	97.78			97.11	860	**98.35**	1/ 120		
19.50	21,000	103.33			101.78	860	102.52			
20.00	21,500	106.48			106.13	860	106.68			
20.50	22,000	108.48			109.40	860	110.85			
21.00	22,500	114.95			114.53	860	**115.02**	1/ 40		Outlet of Mapanuepe Lake
21.50	23,000	127.95			126.94	680	127.52			Mt. Bagang

Table 3.3.1 Numerical Proposed Design Water Level in the Diversion Channel of the Cabor River

the Proposed Diversion Channel in the Cabor River

$Q=112.1m^3/s$ (10-yr Probable Flood Discharge)

Sta. (m)	Accumulated Distance (m)	Design Discharge (m3/s)	Design Riverbed (EL-m)	Design Water Level (EL-m)	Design Slope Gradient of D.W.L	Design Dike Crest (EL-m)	Remarks
CA0+000	0.0	112.10	-1.50	1.43	1/ 770	2.03	River Mouth
+100	100.0	112.10	-1.21	1.56		2.16	
+200	200.0	112.10	-0.93	1.69	1/ 430	2.29	
+300	300.0	112.10	-0.64	1.92		2.52	
+400	400.0	112.10	-0.36	2.15		2.75	
+500	500.0	112.10	-0.07	2.39		2.99	
+600	600.0	112.10	0.21	2.62		3.22	
+700	700.0	112.10	0.50	2.85	1/ 350	3.45	
+800	800.0	112.10	0.79	3.14		3.74	
+900	900.0	112.10	1.07	3.42		4.02	
CA1+000	1,000.0	112.10	1.36	3.71		4.31	Proposed Improvement Stretch
+100	1,100.0	112.10	1.64	4.00		4.60	
+200	1,200.0	112.10	1.93	4.28		4.88	
+300	1,300.0	112.10	2.21	4.57		5.17	
+400	1,400.0	112.10	2.50	4.85		5.45	
+500	1,500.0	112.10	2.79	5.14		5.74	
+600	1,600.0	112.10	3.07	5.42		6.02	
+700	1,700.0	112.10	3.36	5.71		6.31	
+800	1,800.0	112.10	3.64	6.00		6.60	
+900	1,900.0	112.10	3.93	6.28		6.88	
CA2+000	2,000.0	112.10	4.21	6.57		7.17	
+100	2,100.0	112.10	4.50	6.85		7.45	Existing Cabor Bridge

Figures

THE GOVERNMENT OF THE PHILIPPINES
THE DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS

The Study on Sabo and Flood Control for Western River Basins of Mount Pinatubo in the Republic of the Philippines

JAPAN INTERNATIONAL COOPERATION AGENCY

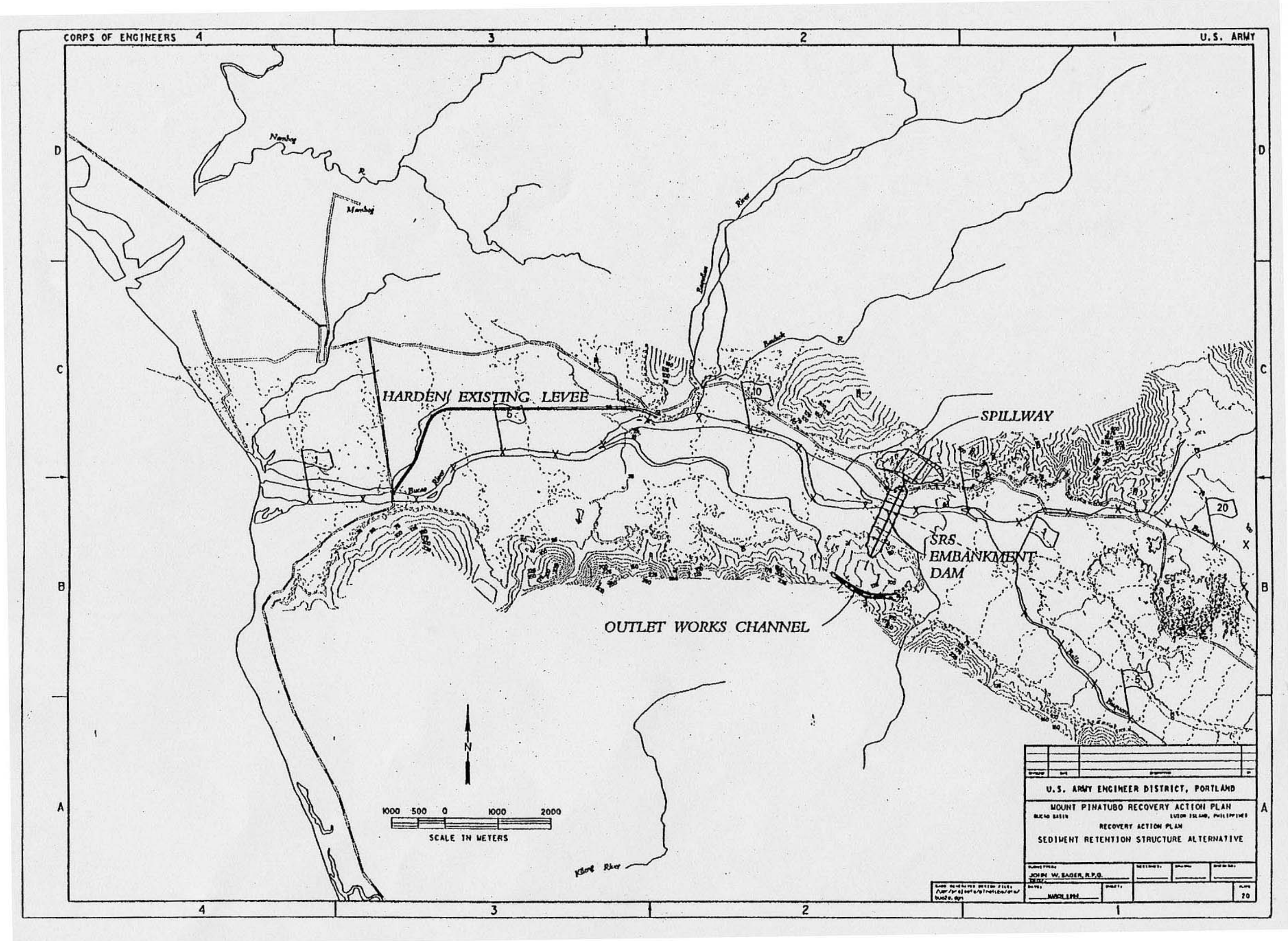


Figure 1.3.1
Proposed Structural Measures in Recovery Action Plan (1994) for the Bucao River

The Study on Sabo and Flood Control for Western River Basins of Mount Pinatubo in the Republic of the Philippines

JAPAN INTERNATIONAL COOPERATION AGENCY

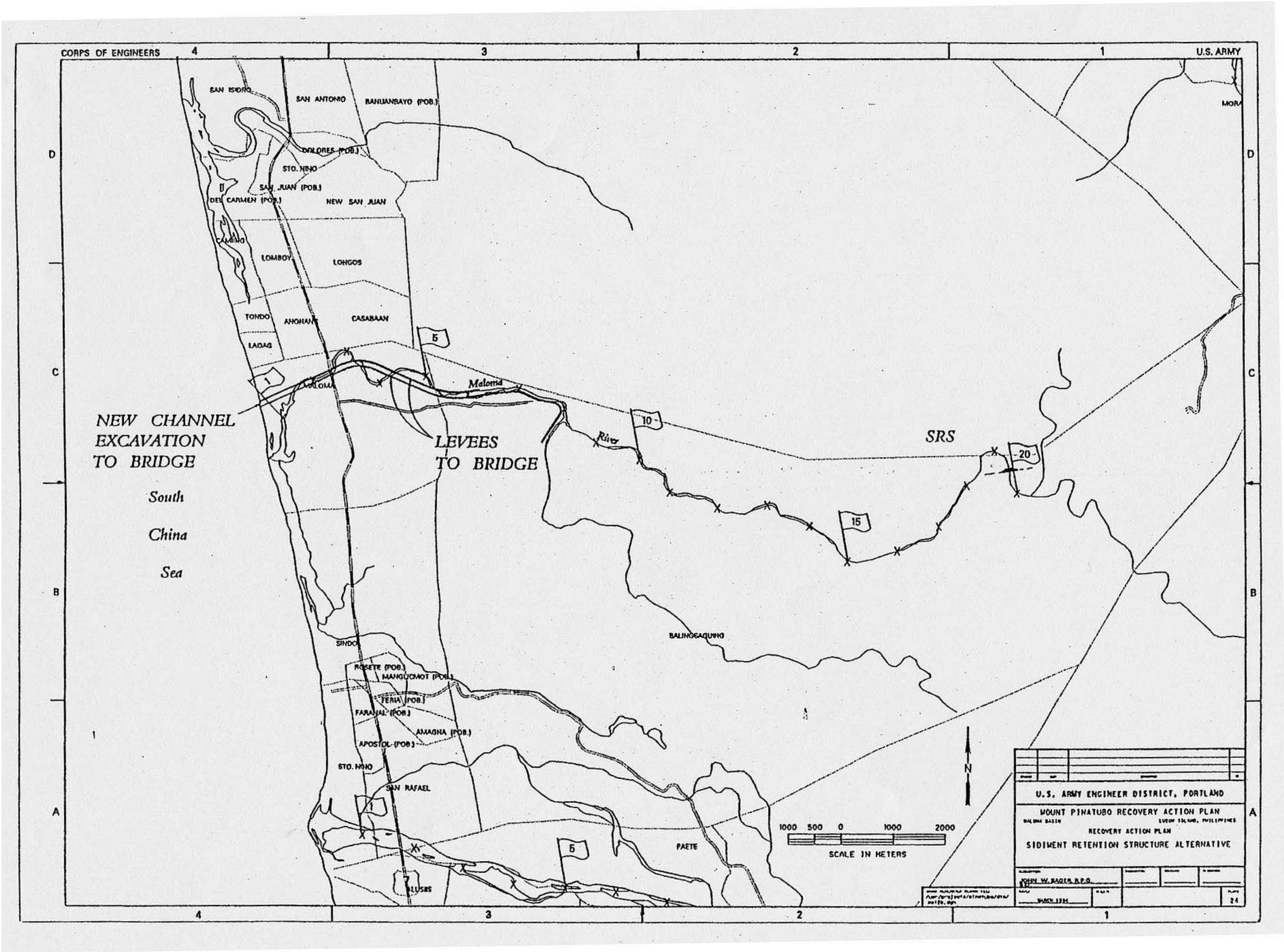


Figure 1.3.2

Proposed Structural Measures in Recovery Action Plan (1994) for the Maloma River