REPUBLIC OF KIRIBATI
MINISTRY OF PUBLIC WORKS & UTILITIES

THE PREPARATORY SURVEY ON THE PROJECT FOR RECONSTRUCTION ON NIPPON CAUSEWAY ON TARAWA TO ADAPT CLIMATE CHANGE IN REPUBLIC OF KIRIBATI

FINAL REPORT

MAY 2016

JAPAN INTERNATIONAL COOPERATION AGENCY

CTI ENGINEERING INTERNATIONAL CO., LTD.
IDES INC.

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PREFACE

Japan International Cooperation Agency (JICA) decided to conduct the preparatory surveyed on the project for reconstruction on Nippon Causeway on Tarawa in the Republic of Kiribati and organized a survey team headed by Dr. Shingo Gose of CTI Engineering International Co., LTD. between 2015 to 2016.

The survey team held a series of discussions with the officials concerned of the Republic of Kiribati, and conducted field investigations. As a result of further studies in Japan, the present report was finalized.

I hope that this report will contribute to the promotion of the project and to the enhancement of friendly relations between our two countries.

Finally, I wish to express my sincere appreciation to the officials concerned of the Republic of Kiribati for their close cooperation extended to the survey team.

May, 2016

Akira NAKAMURA
Director General
Infrastructure and Peacebuilding Department
Japan International Cooperation Agency

Summary

(1) Situation of Republic of Kiribati (hereafter referred to as Kiribati)

- Population is 100,300 people and the area is 730 km^2 consisting of 33 atolls with very large exclusive economic zone which is 3,550,000 km^2 of the third place of the world.
- The causeway with a length of 3.2 km and a width of 11m which is only the road to connect Betio island where the international port exists and Bairiki island where the head quarters of administrative agencies and residential area exist is indispensable to sustain the life of citizens and economical activities
- The causeway mentioned above has been eroded and corrupted due to the aging and natural disasters like high tide water. Therefore, the repair and strengthening works for the entire section is the urgent issue to be addressed.

(2) Development Plan and Background

- Rehabilitation of the aged road is described in Kiribati Development Plan（2012-2015） as a priority item of establishment of infrastructures.
- In order to improve such situations, the government of Kiribati (GOK) requested the Government of Japan (GOJ) for a grant aid to undertake the “The Project for Reconstruction of Nippon Causeway on Tawara to adapt climate change”.

(3) Outline and Results of the Survey

JICA dispatched the study team for the project to Kiribati as shown in table-1. In the 1st and 2nd site survey, determination of project scope, methodology for strengthening of revetment and bridge, pavement type, relocation of utilities, traffic volume survey, load axle survey, geological survey, topological survey were implemented based on discussion with GOK. And outline design for strengthening of road, revetment and bridge were implemented based on the results of them. In the 3rd site survey, contents of outline design and undertaking of Kiribati side were explained by JICA expert team, and GOK was agreed.

In addition, the construction supervision for the emergency restoration works of the causeway was implemented based on countermeasure which was considered and proposed by JICA study team in 1st and 2nd site survey.

Table-1 Site Survey and Construction Supervision Schedule

Site Survey / Construction Supervision	Period
1st Site Survey	May 26th 2015 ~ July 6th 2015
2nd Site Survey	August 18th 2015 ~ September 21st 2015
1st Construction Supervision	January 5th 2015 ~ February 3rd 2016
2nd Construciton Supervision	February 16th 2016 ~ March 12th 2016
3rd Site Survey	February 23rd 2016 ~ March 4th 2016
3rd Constrcution Supervision	April 5th 2016 ~ May 9th 2016

Points in the project are as follows;

- The tide level which is applied in this design is set in consideration of the impact El Niño and future climate changes such as the future sea level rise.
- The revetment structures are selected for strengthening accordingly to each section considering damages of the existing revetment.
- As for the pavement type, the asphalt pavement is selected as the most economical option as a result of comparison among asphalt pavement, concrete pavement and DBST considering the life cycle cost.
- As a result of the visual inspection, crack measurements and simple strength test of concrete, it was not confirmed that the existing bridge structure had critical damages or deformations. Therefore, the minor repair is planned, and widening of the bridge are designed in order to ensure the width and the continuity of the embankment.
- The utility box is designed to be set along the side of the road separated from the Causeway structure in order to improve maintenance.

The outline of the Project to be proposed is as follows:

Table- 2　Outline of the Project (Embankment Section)

Road Length			3,220m
Road Width	Normal Section	Carriageway	W=6.0m（3.0m×2）
		Shoulder/Walkway	W=5.0m（2.5m×2）
		Total	W=11.0m
	Utility Box Installed Section	Carriageway	W=6.0m（3.0m×2）
		Shoulder/Walkway	W=3.5m（1.75m×2）
		Utility Box	W=1.5m
		Total	W=11.0m
Pavement		Type	Asphalt Pavement
		Surface	50mm
		Upper Basecourse	150mm
		Lower Basecourse	200mm
Revetment		Option-1	L=400m
		Option-2	L=2,127.35m
		Option-3	L=1,700m

Table-3　Outline of the Project (Bridge Section)

Length		10m
Type		Box Culvert
Width	Carriageway	W=6.0m（3.0m×2）
	Shoulder/Walkway	W=5.0m（2.5m×2）
	Total	W=11.0m
Pavement	Type	Asphalt Pavement
	Surface	70mm

(4) Project Implement Schedule and Project Cost

This part is closed due to the confidentiality

(5) Project Evaluation

1) Relevance

Implementation of the Project under Japanese Grant Aid has been determined to be valid for the following reasons:

- As described above, the causeway is only the road to connect Betio Island where the international port exists and Bairiki Island where the headquarters of administrative agencies and residential area are located. Therefore, it is expected that the Project will benefit a considerable number of people in Kiribati.
- The effect of the Project is a secure, safe and a smooth travel along Nippon causeway. These would consequently contribute to the improvement of people's living condition.
- By the full scale repair and strengthening of Nippon causeway, the maintenance cost of revetment for Nippon causeway will be remarkably reduced.
- Negative environmental and social impacts of the Project are relatively small.
- Operation and maintenance after the Project can be implemented by the Kiribati side under its own budget and staff without the need for very advance skill and technology.

2) Effectiveness

Quantitative Effects

The expected quantitative effects of the Project are 1) the reduction of annual road traffic control day due to natural disaster, 2) the reduction of the revetment collapse, 3) the improvement of travel speed and 4) the reduction of maintenance/repair cost for revetment.

Table-4 Qualitative Effects

Indicators	Reference Value (Actual Value in 2015)	Target Value (2022) (After 3 years in service)
Number of day for road traffic control due to natural disaster	28 days (revetment repair work by king tide)	0 day
No. of revetment collapse	6 times	0 time
Average Travel Speed	20 km/h	40 km/h
Maintenance/repair cost for revetment	381,408 AUD	28,599 AUD

<u>Target Value of Average Travel Speed</u>

Though the alignment of Nippon causeway are almost straight and flat and the design speed is 60 km/h, the free speed is assumed to be 50 km/h due to many trucks traffic. As it is also considered that the average toll payment time and waiting time at toll gate is 35 seconds and stopped time at roundabout near the end point is 15 seconds, the average travel speed is calculated as 40 km/h.

Travel time along Nippon causeway = 3.2 km／50 [km/h]×3600+35 sec.+15 sec.

= 28.0 sec. ＝0.078 hour

Travel speed along Nippon causeway＝3.2 km／0.078＝41≒40 [km/h]

<u>Target Value of Maintenance/Repair Cost</u>

Currently, when the revetment collapse occurred, repair work was implemented. In order to utilize the infrastructure for a longer life time, it is necessary to confirm the periodical inspection and minor maintenance annual cost. The annual maintenance cost for road revetment is estimated to be about 19,297 AUD and that for bridge section (l=l0m) is 9,302 AUD as described in “2.5.2 Operation and Maintenance Cost”. So, the total annual maintenance cost is estimated to be 28,599AUD

<u>Qualitative Effects</u>

The qualitative effects of the Project are as follows:

- Stable lifeline will be secured all year around
- The safety of pedestrians and vehicles will be improved
- Logistics and confluence between Betio and Bairiki will be secured all year around

TABLE OF CONTENTS

Preface
Summary
Contents
Location Map/Perspective
List of Figures & Tables
Abbreviations

1 Background of the Project 1-1
1.1 Outline of Republic of Kiribati 1-1
1.1.1 Situation of Republic of Kiribati 1-1
1.1.2 Development Plan and Background 1-1
1.1.3 Precedent surveys and other donors' activities 1-1
1.1.4 Scope of the Request 1-1
1.1.5 Objectives of the Survey 1-1
1.1.6 Goal and Achievement of the project 1-1
1.2 Natural Environmental Condition 1-2
1.2.1 Temperature 1-2
1.2.2 Rainfall 1-2
1.2.3 Wind Direction and Velocity 1-3
1.2.4 Tide 1-5
1.2.5 Climate Change 1-7
1.2.5.1 El Niño and La Niña Events 1-7
1.2.5.2 Mean Sea Level Rise due to Emission Scenario 1-10
1.2.6 Wave 1-11
1.2.6.1 Lagoon Side 1-11
1.2.6.2 Ocean Side 1-11
1.2.7 Cyclone Path 1-13
1.2.8 Coastal Stream (Current) 1-15
1.2.8.1 Original Design 1-15
1.2.8.2 JICA/SOPAC Study Report (1995) 1-15
1.2.8.3 T & TI Study Report (1995) 1-15
1.2.9 Coast 1-16
1.2.10 Geological Survey 1-17
1.2.10.1 General 1-17

1.2.10.2 General Topography and Geology 1-17
1.2.11 Traffic Count Survey Results 1-22
1.2.12 Axle Load Survey 1-23
1.3 Environment and social considerations 1-25
1.3.1 Status of environmental license acquisition 1-25
1.3.2 Existing environment around the Causeway 1-25
1.3.2.1 Pollution 1-25
1.3.2.2 Natural environment 1-25
1.3.2.3 Social environment 1-26
1.3.3 Potential environmental impacts and mitigation measures 1-26
1.3.3.1 Construction phase 1-26
1.3.3.2 Post-construction phase 1-27
1.3.3.3 Environmental management plan and monitoring plan 1-27
1.3.3.4 Stakeholder meeting 1-27
1.3.3.5 Conclusion and recommendation 1-27
2 Contents of the Project 2-1
2.1 Basic Concept of the Project 2-1
2.1.1 Background 2-1
2.1.1.1 Situation of Republic of Kiribati 2-1
2.1.1.2 Development Plan and Background 2-1
2.1.1.3 Precedent Surveys and Other Donors' Activities 2-1
2.1.2 Scope of the Request 2-1
2.1.3 Objectives of the Survey 2-2
2.1.4 Goal and Achievement of the project 2-2
2.1.5 Environment and social considerations 2-2
2.2 Outline Design of the Japanese Assistance 2-6
2.2.1 Design Policy 2-6
2.2.1.1 Site Condition 2-6
2.2.1.2 Concept of Improvement for the Project 2-16
2.2.1.3 Road Design Policy 2-20
2.2.1.4 Revetment Design Policy 2-25
2.2.1.5 Policy of Bridge Design 2-39
2.2.1.6 Policy for Relocation of Utilities 2-43
2.2.1.7 Emergency Restoration Works Policy 2-44
2.2.1.8 Points of the Project 2-51
2.2.1.9 Technical Assistance for Emergency Countermeasures 2-51
2.2.2 Basic Plan 2-56
2.2.2.1 Applicable Standards 2-56
2.2.2.2 Road Design 2-56

2.2.2.3 Revetment Design 2-64
2.2.2.4 Bridge Design 2-84
2.2.2.5 Emergency Restoration Works Plan 2-91
2.2.3 Outline Design Drawings 2-96
2.2.4 Implementation Plan 2-96
2.2.4.1 Implementation Policy 2-96
2.2.4.2 Implementation Condition 2-97
2.2.4.3 Scope of Works 2-98
2.2.4.4 Construction Supervision Plan 2-98
2.2.4.5 Quality Control Plan 2-99
2.2.4.6 Procurement Plan 2-100
2.2.4.7 Soft Component (Technical Assistance) Plan 2-102
2.2.4.8 Implementation Schedule 2-109
2.3 Obligations of Recipient Country 2-110
2.3.1 General Obligations under Japan's Grant Aid Scheme 2-110
2.3.2 Specific Obligations under the Project 2-110
2.3.2.1 Obtaining Permits for the Implementation of the Project 2-110
2.3.2.2 Relocation of Obstacles (Buried Items such as Telephone Cables and Electric Cables) and Relocation of Street Lighting 2-110
2.3.2.3 Temporary Yard 2-110
2.3.3 Requests to the Recipient Country 2-110
2.3.3.1 Public Meeting to Explain the Project to Residents along the Nippon Causeway Sections 2-111
2.3.3.2 Traffic Safety 2-111
2.3.3.3 Notification of Inconvenience during the Road Work 2-111
2.4 Project Operation Plan 2-112
2.4.1 Operation and Maintenance Setup 2-112
2.4.2 Maintenance Work following Project Implementation 2-112
2.4.3 Routine Maintenance 2-112
2.4.4 Periodic Maintenance 2-112
2.5 Project Cost Estimation 2-113
2.5.1 Initial Cost Estimation 2-113
2.5.1.1 Japan's Contribution 2-113
2.5.1.2 Kiribati's Contribution 2-113
2.5.1.3 Cost Estimation Condition 2-113
2.5.2 Operation and Maintenance Cost 2-114
3 Project Evaluation 3-1
3.1 Precondition 3-1
3.2 Necessary Input by Recipient Country 3-1

3.3 Important Assumption 3-1
3.4 Project Evaluation 3-1
3.4.1 Relevance 3-1
3.4.2 Effectiveness 3-2

Appendix-1 : Member List of Study Team
Appendix-2 : Study Schedule
Appendix-3 : List of Parties Concerned in the Recipient Country
Appendix-4 : Minutes of Discussions (1st)
Appendix-5 : Minutes of Discussions (2nd)
Appendix-6 : Minutes of Discussions (3rd)
Appendix-7 : Technical Notes
Appendix-8 : Soft Component Plan
Appendix-9 : Cost Estimate of Pavement Type
Appendix-10: Geotechnical Survey Result
Appendix-11: Outline Design Drawings

LOCATION MAP

Perspective

LIST OF FIGURE & TABLES

FIGURES

FIGURE 1.2.2-1 ANNUAL RAINFALL 1-3
FIGURE 1.2.3-1 ANNUAL WIND ROSES AT THE ORIGINAL DESIGN 1-3
FIGURE 1.2.3-2 ANNUAL WIND ROSES FOR BETIO ISLAND 1-3
FIGURE 1.2.3-3 EFFECT ON SOI FOR WIND ROSES 1-4
FIGURE 1.2.3-4 MAX. INSTANTANEOUS WIND SPEED BY DIRECTIONS (LA NIÑA PERIOD) 1-4
FIGURE 1.2.3-5 MAX. INSTANTANEOUS WIND SPEED BY DIRECTIONS (EL NIÑO PERIOD) 1-4
FIGURE 1.2.4-1 COMPARISON OF PAST TIDE OBSERVATIONS AND DATUMS 1-5
FIGURE 1.2.4-2 ANNUALLY AVERAGED TIDAL LEVEL AND FREQUENCY OF KING TIDE 1-7
FIGURE 1.2.5-1 HISTORICAL CHANGE OF MEAN SEA LEVEL (TOP) AND HIGHEST HIGH WATER LEVEL (BOTTOM) AT BETIO PORT 1-9
FIGURE 1.2.5-2 DIFFERENCE BETWEEN ANNUAL MAXIMUM AND AVERAGE/ BETWEEN PREDICTION AND OBSERVATION OF KING TIDE 1-10
FIGURE 1.2.5-3 PREDICTION OF GLOBAL MEAN SEA LEVEL RISE (IPCC AR5) 1-10
FIGURE 1.2.7-1 PAST TYPHOON AND CYCLONE PATHS 1-14
FIGURE 1.2.7-2 STREAMLINE ANALYSIS AROUND KIRIBATI ON 11TH OF MARCH 2015 1-14
FIGURE 1.2.7-3 PATH MAP OF CYCLONE PAM 1-15
FIGURE 1.2.9-1 SEDIMENTATION OF BAIRIKI SIDE 1-16
FIGURE 1.2.9-2 SEDIMENTATION OF BETIO SIDE 1-16
FIGURE 1.2.10.2-1 ARIAL VIEW OF THE CAUSE WAY FROM WEST (PHOTO TAKEN IN 1943:LEFT, 2015:RIGHT) 1-18
FIGURE 1.2.10.2-2 MAP OF THE SURVEY POINTS 1-20
FIGURE 1.2.10.2-3 GEOLOGICAL CROSS SECTION IN LONGITUDE DIRECTION 1-20
FIGURE 1.2.11-1 HOURLY TRAFFIC VOLUME DISTRIBUTION OF HOLIDAY (TO BAIRIKI) 1-22
FIGURE 1.2.11-2 HOURLY TRAFFIC VOLUME DISTRIBUTION OF HOLIDAY (TO BETIO) 1-23
FIGURE 1.2.11-3 HOURLY TRAFFIC VOLUME DISTRIBUTION OF WEEKDAY (TO BAIRIKI) 1-23
FIGURE 1.2.11-4 HOURLY TRAFFIC VOLUME DISTRIBUTION OF WEEKDAY (TO BETIO) 1-23
FIGURE 2.2.1-1 REVIEW OF THE ROAD ELEVATION 2-22
FIGURE 2.2.1-2 FEATURES OF THE CAUSEWAY DAMAGE 2-25
FIGURE 2.2.1-3 WAVE HEIGHTS USED IN THE ORIGINAL DESIGN 2-29
FIGURE 2.2.1-4 CHARACTERISTICS RELATED TO WAVE TRANSFORMATION ON REEF 2-30
FIGURE 2.2.1-5 DIAGRAM FOR ESTIMATING OVERTOPPING RATE (GODA) 2-30
FIGURE 2.2.1-6 EXPLANATORY SKETCH OF TERMS USED AT REVETMENT 2-34
FIGURE 2.2.1-7 OVERTOPPING CONDITION IN CADMAS (ORIGINAL DESIGN) 2-37
FIGURE 2.2.1-8 OVERTOPPING CONDITION IN CADMAS (ORIGINAL DESIGN/ KING TIDE) 2-38

FIGURE 2.2.1-9 DISPLAY OF COASTAL CALCULATOR (ORIGINAL DESIGN/ KING TIDE) 2-38
FIGURE 2.2.1-10 CROSS SECTION OF THE BRIDGE 2-41
FIGURE 2.2.1-11 NAVIGATIONAL CLEARANCE UNDER CAUSEWAY BRIDGE 2-41
FIGURE 2.2.1-12 TYPICAL CROSS SECTION OF UTILITIES 2-44
FIGURE 2.2.1-13 RELOCATED SECTION 2-44
FIGURE 2.2.1-14 CRACK REPAIR MANUAL 2-47
FIGURE 2.2.1-15 MORTAL SANDBAG MANUAL 2-50
FIGURE 2.2.2 -1 SIZE AND INSTALLATION INTERVAL OF THE TRANSVERSE DRAINAGE 2-61
FIGURE 2.2.2-2 ROAD DRAINAGE SHAPE 2-62
FIGURE 2.2.2-3 DETAIL OF UTILITY BOX 2-63
FIGURE 2.2.2-4 APPLICATION MAP FOR REVETMENT STRENGHENING 2-69
FIGURE 2.2.2-5 WAVE HEIGHT AT SLIDING OF EXISTING REVETMENT 2-71
FIGURE 2.2.2-6 PARAPET SHAPE OF OCEAN SIDE (BAIRIKI SIDE) 2-72
FIGURE 2.2.2-7 PARAPET SHAPE OF OCEAN SIDE (BETIO SIDE) 2-72
FIGURE 2.2.2-8 PARAPET SHAPE OF LAGOON SIDE 2-73
FIGURE 2.2.2-9 CALCULATION MODEL OF REVETMENT SLOPE 2-75
FIGURE 2.2.2-10 REFRACTION AT BAIRIKI OCEAN SIDE 2-76
FIGURE 2.2.2-11 WATER LEVEL DIFFERENCE OF SLOPE 2-79
FIGURE 2.2.2-12 UPLIFT OF FABRIMAT MAT 2-80
FIGURE 2.2.2-13 RESIDUAL WATER LEVEL AND CALCULATION MODEL FOR FABRIMAT MAT WITH SHEET PILE 2-81
FIGURE 2.2.2-14 SLOPE TOE PROTECTION 2-82
FIGURE 2.2.2-15 CALCULATION MODEL OF STEEL SHEET PILE 2-83
FIGURE 2.2.2-16 CALCULATION MODEL 2-89
FIGURE 2.2.2-17 TYPICAL CROSS SECTION OF BRIDGE WIDENING 2-90
FIGURE 2.5.2-1 ORGANIZATIONAL CHART OF CIVIL ENGINEERING SECTION 2-114

TABLES

TABLE 1.2.1-1 MONTHLY AVERAGED TEMPERATURE, ... 1-2
TABLE 1.2.4-1 COMPARISON OF TIDAL LEVELS 1-6
TABLE 1.2.4-2 TIDAL ANALYSIS ON THE BASIS OF OBSERVATION RECORD AT BETIO PORT 1-7
TABLE 1.2.5-1 PHASE OF EL NIÑO/ LA NIÑA EVENTS 1-8
TABLE 1.2.5-2 HISTORICAL IMPACT OF EL NIÑO TO PIC 1-8
TABLE 1.2.6-1 EXTREME OFFSHORE WAVE HINDCASTED IN THE ORIGINAL DESIGN 1-12
TABLE 1.2.6-2 EXTREME OFFSHORE WAVE BY T&TI 1-12
TABLE 1.2.6-3 FREQUENCY OF OFFSHORE WAVE HEIGHT AND PERIOD AT THE ADJACENT AREA 1-13
TABLE 1.2.10-1 SUMMARY OF GEOLOGICAL SURVEY 1-17
TABLE 1.2.10-2 STRATIGRAPHIC FORMATIONS 1-20
TABLE 1.2.11-1 TRAFFIC COUNT SURVEY CONDITION 1-22
TABLE 1.2.11-2 TRAFFIC COUNT SURVEY RESULTS 1-22
TABLE 1.2.12-1 SURVEY CONDITION 1-24
TABLE 1.2.12-2 AXLE LOAD 1-24
TABLE 2.2.1-1 CLASSIFICATION OF SEAWALL DAMAGE 2-6
TABLE 2.2.1-2 LENGTHS OF SEAWALL DAMAGE DEGREE 2-6
TABLE 2.2.1-3 CLASSIFICATION OF SEAWALL DAMAGE DEGREE 2-7
TABLE 2.2.1-4 METHODOLOGY AND MAJOR RESULT OF THE EVALUATION OF BRIDGE SOUNDNESS 2-12
TABLE 2.2.1-5 PHOTO RECORD OF EXISTING BRIDGE (1/3) 2-13
TABLE 2.2.1-6 PHOTO RECORD OF EXISTING BRIDGE (2/3) 2-14
TABLE 2.2.1-7 PHOTO RECORD OF EXISTING BRIDGE (3/3) 2-15
TABLE 2.2.1-8 ALTERNATIVE OF IMPROVEMENT MEASURES (1) 2-17
TABLE 2.2.1-9 ALTERNATIVE OF IMPROVEMENT MEASURES (2) 2-18
TABLE 2.2.1-10 ALTERNATIVE OF IMPROVEMENT MEASURES (3) 2-19
TABLE 2.2.1-11 ROAD WIDTH FOR THE PROJECT 2-20
TABLE 2.2.1-12 CROSS SECTION ALTERNATIVES (SECTION WITH UTILITIES SEPARATED FROM CAUSEWAY STRUCTURE) 2-21
TABLE 2.2.1-13 OUTLINE OF HORIZONTAL ALIGNMENT 2-23
TABLE 2.2.1-14 OUTLINE OF PROFILE 2-23
TABLE 2.2.1-15 COMPARISON TABLE FOR PAVEMENT TYPE 2-24
TABLE 2.2.1-16 DESIGN CONDITION FOR ROAD DRAINAGE 2-25
TABLE 2.2.1-17 BASIC CONCEPT FOR THE MEASURES OF THE CAUSEWAY 2-26
TABLE 2.2.1-18 REVISION OF TIDAL CONDITIONS 2-27
TABLE 2.2.1-19 ORIGINAL AND REVISED TIDAL CONDITIONS 2-28
TABLE 2.2.1-20 DESIGN WAVE HEIGHT 2-29
TABLE 2.2.1-21 OVERTOPPING RATE UNDER REVISED CONDITIONS (OCEAN SIDE - BASIC CASES) 2-31
TABLE 2.2.1-22 OVERTOPPING RATE UNDER REVISED CONDITIONS (OCEAN SIDE - SENSITIVITY ANALYSIS) 2-32

TABLE 2.2.1-23 PRESUMED OVERTOPPING RATE AT DAMAGE ... 2-32
TABLE 2.2.1-24 OVERTOPPING RATE AND LIMITS (OCEAN SIDE)... 2-33
TABLE 2.2.1-25 TRANSFORMING OF OFFHORE WAVE FROM LAGOON OPENING TO REEF EDGE ... 2-34
TABLE 2.2.1-26 OVERTOPPING RATE AND LIMITS (LAGOON SIDE) ... 2-35
TABLE 2.2.1-27 COMPARISON OF RESULTS BY METHOD OF ANALYSIS (OCEAN SIDE) ... 2-36
TABLE 2.2.1-28 COMPARISON OF RESULTS BY METHOD OF ANALYSIS (LAGOON SIDE) ... 2-37
TABLE 2.2.1-29 ALTERNATIVE FOR BRIDGE STRENGTHENING ... 2-40
TABLE 2.2.1-30 NAVIGATIONAL CLEARANCE IN THE SAME WAY OF ORIGINAL DESIGN ... 2-42
TABLE 2.2.1-31 NAVIGATIONAL CLEARANCE CONSIDERING BOAT TRAFFIC AND WATER DEPTH ... 2-43
TABLE 2.2.2-1 APPLICABLE STANDARDS ... 2-56
TABLE 2.2.2-2 TRANSITION OF TRAFFIC VOLUME IN KIRIBATI ... 2-56
TABLE 2.2.2-3 TRAFFIC CLASS AND FATIGUE FRACTURE WHEEL LOAD ... 2-57
TABLE 2.2.2-4 RELATIONSHIP OF TRAFFIC CLASS AND DESIGN CBR ... 2-58
TABLE 2.2.2-5 MINIMUM THICKNESS OF EACH LAYER ... 2-58
TABLE 2.2.2-6 COMPARISON TABLE OF PAVEMENT STRUCTURE ... 2-59
TABLE 2.2.2-7 AXLE LOAD... 2-60
TABLE 2.2.2-8 DESIGN CONDITION... 2-60
TABLE 2.2.2-9 RESULT OF PAVEMENT THICKNESS DESIGNED BY AASHTO ... 2-60
TABLE 2.2.2-10 REOCCURRENCE PERIOD OF RAINFALL... 2-61
TABLE 2.2.2-11 RUN-OFF FACTOR ... 2-62
TABLE 2.2.2-12 CALCULATION CONDITION FOR INTENSITY OF RAINFALL ... 2-62
TABLE 2.2.2-13 CALCULATION RESULT ... 2-62
TABLE 2.2.2-14 STRUCTURAL CALCULATION FOR BECHICLE VOLLISION ... 2-66
TABLE 2.2.2-15 REVETMENT ALTERNATIVES ... 2-66
TABLE 2.2.2-16 APPLIED WAVE HEIGHT AND PRESSURE ... 2-71
TABLE 2.2.2-17 RESULT OF SAFETY COMPUTATION FOR OCEAN SIDE (BAIRIKI SIDE) ... 2-72
TABLE 2.2.2-18 RESULT OF SAFETY COMPUTATION FOR OCEAN SIDE (BETIO SIDE)... 2-73
TABLE 2.2.2-19 RESULT OF SAFETY COMPUTATION FOR LAGOON SIDE ... 2-73
TABLE 2.2.2-20 REVIEW OF EXISTING FABRIMAT MAT ... 2-76
TABLE 2.2.2-21 VERIFICATION OF PLANNED FABRIMAT MAT THICKNESS (WAVE ACTION)... 2-78
TABLE 2.2.2-22 VERIFICATION OF PLANNED FABRIMAT MAT THICKNESS (RESIDUAL WATER LEVEL)... 2-81
TABLE 2.2.2-23 RESULT OF CALCULATION ... 2-83
TABLE 2.2.2-24 DESIGN CONDITION OF EXISTING BRIDGE... 2-84
TABLE 2.2.2-25 DESIGN CONDITION OF ANALYSIS FOR CURRENT STATUS OF THE BRIDGE... 2-84
TABLE 2.2.2-26 SOIL CONSTANTS FOR BEARING CAPACITY CHECK OF EXISTING BRIDGE ... 2-85
TABLE 2.2.2-27 CONDITION OF NAVIGATION ... 2-85
TABLE 2.2.2-28 COUNTERMEASURE AND ISSUES ON THE BRIDGE IMPROVEMENTS ... 2-86
TABLE 2.2.2-29 QUANTITY OF BRIDGE REPAIR... 2-87
TABLE 2.2.2-30 ANALYSIS RESULT OF BOX CULVERT ... 2-88

TABLE 2.2.2-31 ANALYSIS RESULT OF WING PART 2-88
TABLE 2.2.2-32 COMPARISON CHART OF EMERGENCY COUNTERMEASURES (1) 2-92
TABLE 2.2.2-33 COMPARISON CHART OF EMERGENCY COUNTERMEASURES (2) 2-93
TABLE 2.2.2-34 COMPARISON CHART OF EMERGENCY COUNTERMEASURES (3) 2-94
TABLE 2.2.2-35 IMPLEMENTATION SCHEDULE 2-95
TABLE 2.2.3-1 TABLE OF CONTENTS FOR OUTLINE DESING DRAWING 2-96
TABLE 2.2.4-1 BURDEN CLASSIFICATION OF THE BOTH COUNTRIES GOVERNMENT. 2-98
TABLE 2.2.4-2 CONCRETE QUALITY CONTROL PLAN 2-99
TABLE 2.2.4-3 QUALITY MANAGEMENT PLAN FOR EARTHWORK AND PAVEMENT WORK 2-100
TABLE 2.2.4-4 PROCUREMENT OF MAJOR CONSTRUCTION MATERIALS 2-100
TABLE 2.2.4-5 MAJOR CONSTRUCTION EQUIPMENT TO BE PROCURED 2-102
TABLE 2.2.4-6 PRESENT MAINTENANCE LEVEL AND TARGET LEVEL FOR MPWU 2-104
TABLE 2.2.4-7 ACTIVITY SCHEDULE FOR SOFT COMPONENT 2-106
TABLE 2.2.4-8 TABLE IMPLEMENTATION SCHEDULE 2-108
TABLE 2.2.4-9 IMPLEMENTATION SCHEDULE 2-109
TABLE 2.5.1-1 APPROXIMATE COST ESTIMATE OF JAPANESE CONTRIBUTION 2-113
TABLE 2.5.1-2 APPROXIMATE COST ESTIMATION OF KIRIBATI CONTRIBUTION 2-113
TABLE 2.5.2-1 MAINTENANCE ITEMS AND ANNUAL COST OF EXISTING BRIDGE 2-115
TABLE 2.5.2-2 MAINTENANCE ITEMS AND ANNUAL COST OF EMBANKMENT SECTION 2-115

Photo

PHOTO 2.2.1-1 TYPICAL CRACK DUE TO BENDING MOMENT 2-8
PHOTO 2.2.1-2 TYPICAL SHEAR CRACK 2-8
PHOTO 2.2.1-3 COLLAPSE FROM CRACK EXPANSION 2-9
PHOTO 2.2.1-4 COLLAPSE TYPES OF REVETMENT 2-10
PHOTO 2.2.2-1 SITE CONDITION OF CRACKS ON REVETMENT SLOPE (BAIRIKI OCEAN SIDE) 2-74

ABBREVIATIONS

ADB	Asian Development Bank
AASHTO	American Association of State Highway and Transportation Officials
BEIA	Basic Environmental Impact Assessment
C/P	Counter Part
CADMAS	Computer Aided Design of Maritime Structure
CBR	California Bearing Ratio
CEIA	Comprehensive Environmental Impact Assessment
CES	Civil Engineering Section
DBST	Double Bituminous Surface Treatment
E/N	Exchange of Notes
EIA	Environmental Impact Assessment
ESAL	Equivalent Single Axle Load
G/A	Grant Agreement
HWL	High Water Level
IDA	International Development Association
IPCC	Intergovernmental Panel on Climate Change
IUCN	International Union for Conservation of Nature and Natural Resources
JICA	Japan International Cooperation Agency
KRRP	Kiribati Road Rehabilitation Project
MELAD	Ministry of Environmental, Lands & Agriculture Development
MHWS	Mean High Water Spring
MLWS	Mean Low Water Spring
MPWU	Ministry of Public Works & Utilities
NIWA	National Institute of Water and Atmosphere Research
T&TI	Tonkin & Taylor International Ltd.

Chapter 1 Background of the Project

1.1 Outline of Republic of Kiribati

1.1.1 Situation of Republic of Kiribati

- Population is 103,000 people, the area is 730 km^2 consisting of 33 atolls with very large exclusive economic zone which is 3,550,000 km^2 of the third place of the world.
- The causeway with a length of 3.2 km and a width of 11m is only the road to connect Betio island where the international port exists and Bairiki island where the headquarters of administrative agencies and residential area exist. It is indispensable in sustaining the life of citizens and economic activities.
- The causeway mentioned above has been eroded and corrupted due to aging and caused by natural disasters like high tide water. Therefore, the repair and strengthening works for the entire section is an urgent issue to be addressed.

1.1.2 Development Plan and Background

- ➢ The rehabilitation of the aged road is described in the Kiribati Development Plan (2012-2015) as a priority item in the establishment of infrastructures.
- ➢ In order to improve such situations, the Government of Kiribati (GOK) requested the Government of Japan (GOJ) for a grant aid to undertake the "The Project for Reconstruction of Nippon Causeway on Tarawa to adapt climate change".

1.1.3 Precedent surveys and other donors' activities

- ➢ The Project for the Improvement of Fisheries Transportation in South Tarawa in the Republic of Kiribati was implemented in 2007 by JICA. The South Tarawa road was partially rehabilitated at Betio, Bairiki and Bikenibeu, which was completed in 2008.
- ➢ In 2012-2016, the Kiribati Road Rehabilitation Project amounting to 38 million US dollars funded by ADB (Asian Development Bank, IDA (International Development Agency) and Government of Australia was implemented.

1.1.4 Scope of the Request

- ➢ Reconstruction of Nippon causeway structure between Betio and Bairiki (approx.3.2 km)
- ➢ Widening of Bridge Section

1.1.5 Objectives of the Survey

Objectives of the Survey are to:

- ➢ Understand the background, purpose and scope of the grant aid project,
- ➢ Study the feasibility of the project in terms of effectiveness, human, technology and economic justification,
- ➢ Conduct the outline design for the minimum but optimal scope and size of the project required in achieving the outcomes of the cooperation,
- ➢ Estimate the project cost, and
- ➢ Propose the contents, implementation and maintenance plan as well as critical points to be undertaken by the GOK in order to achieve the outcome and targets set for the project.

1.1.6 Goal and Achievement of the project

- ➢ Goal: To secure smooth and stable traffic condition between Betio and Bairiki.
- ➢ Achievement：To secure smooth and safe traffic condition of the Nippon Causeway.

1.2 Natural Environmental Condition

1.2.1 Temperature

The averaged temperature at Tarawa atoll is almost constant through the year, and there is an extent of 5-6 degree differences within a day. The monthly and annual averaged temperature are indicated in Table 1.2.1-1 for the time period of 31 years until the original design (1978), in the year 1997, 19 years after, and in the year 2014 the further 17 years after. Although the temperature have nearly been the same for the long periods, it can seem to rise a little recently. The sea level and sea surface temperature, which indicate small increase, are shown in the same Table for reference.

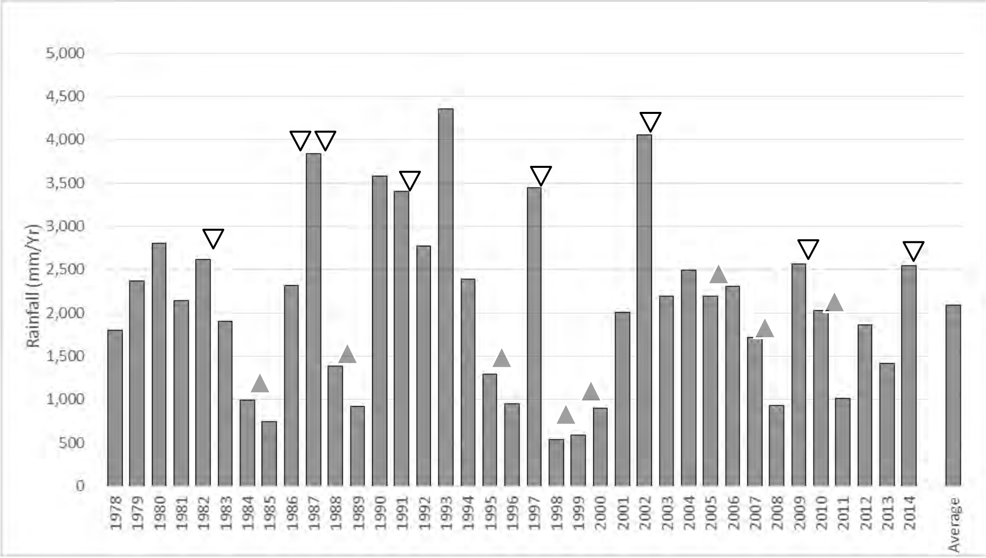
Table 1.2.1-1 Monthly Averaged Temperature, Sea Level and Sea Surface Temperature at Tarawa

Month	1947-78	1997			2014		
	Temperature (degC)	Sea Level (m)	Sea Surface Temp. (degC)	Temperature (degC)	Sea Level (m)	Sea Surface Temp. (degC)	Temperature (degC)
1	28.2	1.76	29.8	28.5	1.72	29.2	29.3
2	28.1	1.71	29.4	28.0	1.81	30.4	29.4
3	28.0	1.82	29.9	28.4	1.82	29.7	29.3
4	28.2	1.74	29.6	28.1	1.77	29.9	28.8
5	28.5	1.68	29.7	28.3	1.70	30.7	29.6
6	28.3	1.66	29.3	27.9	1.67	30.2	29.2
7	28.2	1.66	30.2	28.3	1.74	29.9	29.1
8	28.3	1.69	29.0	27.8	1.76	30.6	29.3
9	28.4	1.65	29.3	28.1	1.75	30.8	29.4
10	28.6	1.64	28.8	27.8	1.78	30.6	28.9
11	28.4	1.54	29.9	28.1	1.74	30.9	29.1
12	28.3	1.50	29.5	28.3	1.73	30.3	28.6
Annual Average	28.3	1.67	29.5	28.1	1.75	30.3	29.2

Source: The Study Team on the basis of Kiribati Meteorological Services(MET) and Basic Design of Betio Bairiki Causeway and Fishery Channel Project (1985)

1.2.2 Rainfall

The annual rainfall at Tarawa recorded from 1978 through 2014 is indicated in Figure 1.2.2-1. The averaged annual rainfall is 2,091 mm/yr during this period. It is obviously seen in Figure 1.2.2-1 that the rainfall increases at the phase of El Niño, and decreases at the phase of La Niña. There is no clear difference in the rainfall from the time of the construction of the causeway other than the reason of El Niño and La Niña events.

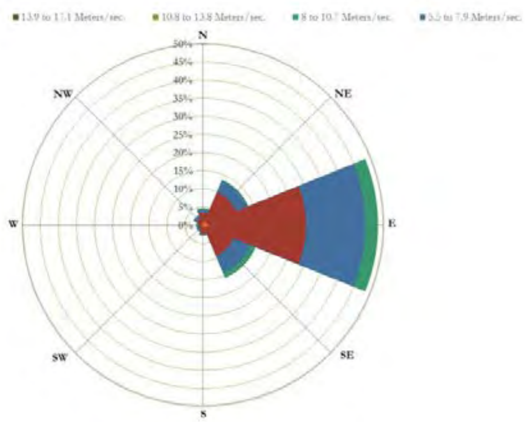


Source: The Study Team on the basis of Kiribati Meteorological Service
Phase of El Niño (▽) and La Niña（▲） with reference to *Table 1.2.1-1*

Figure 1.2.2-1 Annual Rainfall

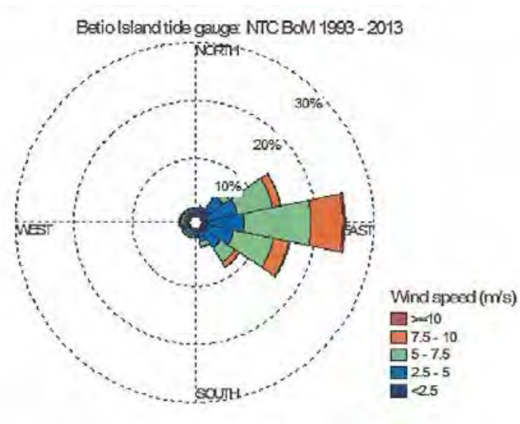
1.2.3 Wind Direction and Velocity

The wind rose in Figure 1.2.3-1 for the period of the years 1970 through 1981 (no record in ’75 and ’77) was generated with the table containing wind frequency by direction and velocity provided in the design report of the original design. The East (E) direction reaches 50%, and the Eastern directions including E, Northeast (NE) and Northwest (NW) cover over 80%. The original design report described the strong wind from the direction of Southwest (SW) and Northwest (NW) under a rear occurrence. The wind rose during 1993 through 2013 is shown in Figure 1.2.3-2 referring to the report by the New Zealand consulting firm (Tonkin & Taylor International Ltd.: T& TI). It indicates that the Eastern trade wind directions including E, NE and SE are covering the half and reach to about 70% together with both side directions although it is not a direct comparison with the wind rose at the original design due to different allocation of wind directions. The overall tendency of prevailing trade wind has not changed from the time of the original design.



Source: The Study Team on the basis of Kiribati Meteorological Services(MET） and Basic Design of Betio Bairiki Causeway and Fishery Channel Project (1985)

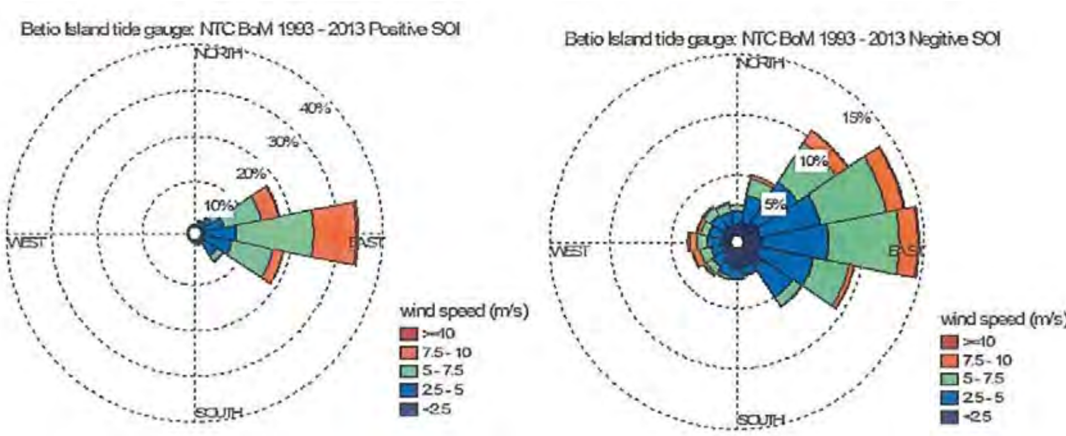
Figure 1.2.3-1 Annual Wind Roses at the Original Design



Source: Tonkin & Taylor International Ltd, "Preparation of Remedial Design for Dai Nippon Causeway Site Investigation and Concept Design"

Figure 1.2.3-2 Annual Wind Roses for Betio Island

Figure 1.2.3-3 in the T&TI report gives the wind roses by positive and negative SOI (South Oscillation Index). SOI expresses an extent of the propagation of El Niño and La Niña, and is determined with the differences in air pressure between Tahiti and Darwin. SOI over +8 means La Niña event and below -8 means El Niño event.

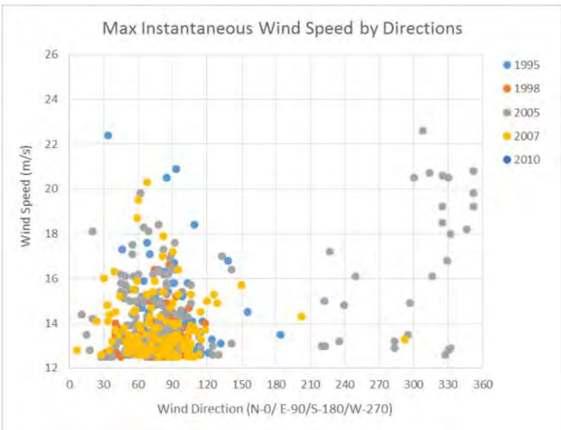


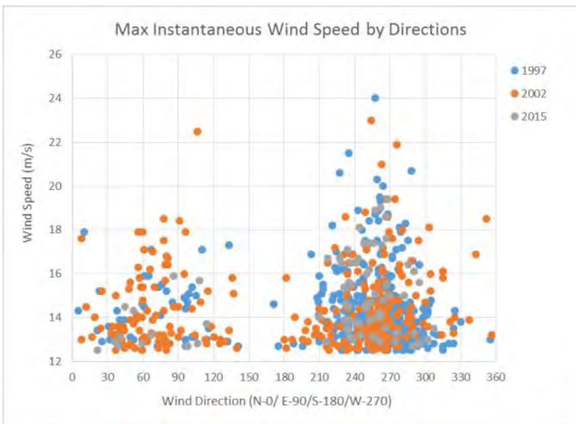
Source: Tonkin & Taylor International Ltd,"Preparation of Remedial Design for Dai Nippon Causeway Site Investigation and Concept Design"

Figure 1.2.3-3 Effect on SOI for Wind Roses

It seems clear difference that the East wind particularly dominates under tendency of La Niña (Left Figure), on the other hand the wind spreads among the Eastern directions, frequency and velocity become increase and strong, and strong west wind is generated under tendency of El Niño (Right Figure).

This phenominon is also seen in Figure 1.2.3-4 and Figure 1.2.3-5, which indicate the maximum instantaneous wind speed over 12.5m/s by directions for the years of La Niña /El Niño on the basis of hourly wind observation data during 1994-20. '90' and '270' in the figures give the East and the West wind directions, respectively. The Eastern (around 90) wind in La Niña (Left Figure) and the Western (around 270) strong wind in El Niño (Right Figure) are dominant. The maximum indtantenous wind speed does not exceed 24 m/s.





Source : The Study Team on the basis of Kiribati Meteorological Services(MET)

Figure 1.2.3-4 Max. Instantaneous Wind Speed by Directions (La Niña Period)

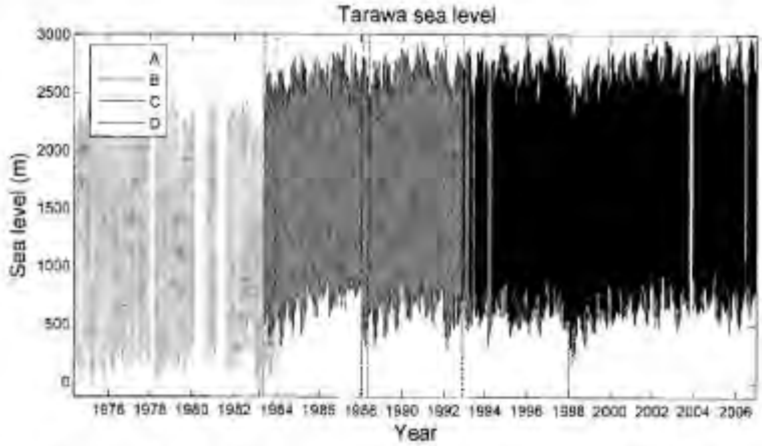
Figure 1.2.3-5 Max. Instantaneous Wind Speed by Directions (El Niño Period)

1.2.4 Tide

The tidal observation started with the tide gauge installed at the Betio Port during the period of 1974 through 1983, and the original design adopted the results of tide analysis for the periods of 1974 through 1978 done by Hawaii University. The datum in that periods was University of Hawaii Tide Gauge Zero (UoH). It had been followed by the observation at Bairiki in the period of 1983 through 1988, and at Betio in the period of 1988 through 1997, and the datum of UoH was adopted. In 1993, the new tide gauge was installed under South Pacific Sea Level and Climate Monitoring Project (SPSLCMP), and the observation has been continued to the present. The datum had changed to SEAFRAME Tide Gauge Zero (SEAFRAME). The difference of the datum between UoH and SEAFRAME had been analyzed by NIWA (National Institute of Water and Atmosphere Research), and concluded as the difference of 0.419m. There was no significant difference between UoH after the year 1983 and SEAFRAME. Figure 1.2.4-1 indicates the analysis the history of the datum provided by NIWA.

Sea level records available for Tarawa. The tabulated datum shift for each record based on consideration of the overlapping components of each record, is shown in the last column.

Name	Location	Start date	Finish date	Datum shift (mm)
Tarawa A	Betio	31 May 1974	31 Dec 1983	+419
Tarawa B	Bairiki	17 May 1983	10 May 1988	+23
Tarawa C	Betio	20 Jan 1988	31 Dec 1997	+23
SEAFRAME (D)	Betio	27 Mar 1993	Ongoing	0



Plot of measured Tarawa sea levels between May 1974 to the present from the four available sea-level records (Table 1).

Source: NIWA, "Kiribati Adaption Programme. Phase II: Information for Climate Risk Management" (2010)

Figure 1.2.4-1 Comparison of Past Tide Observations and Datums

The national datum for survey in Kiribati was reviewed in 2011, and UoH was adopted as there was no difference between UoH and SEAFRAME. The past JICA Study "The Study for Port Development Planning in Kiribati" (1995) also reviewed the difference of the datum, and concluded the difference of 0.74m between the datum of SPSLCMP and the datum used in the previous construction project.

Recognizing the difference between the current UoH (i.e.; SEAFRAME or SPSLCMP) and the beginning UoH, the Study Team compared the original design and current elevation of the causeway road by the topographic survey. The results show the elevation of +4.06 m (average in Betio side) and +4.08m (average in Bairiki side) according to the current UoH or SEAFRAME, while the designed elevation is +3.3m (to the beginning UoH). Therefore the difference of the datum becomes 0.76m and 0.78m if there have been no settlement caused after construction, and it can be concluded that the difference of 0.74m is reasonable. However, another difference in the analysis by NIWA and the history of the datum have not been clarified by the information collection through relevant parties.

The tidal information and datum considered in the original design, “The Study for Port Development Planning in Kiribati” (1995), NIWA, and “The Study of the Project for Expansion of Betio Port” are summarized in Table 1.2.4-1. The mean sea level (MSL) among those sources coincide when the datum difference of 0.74m is considered.

Table 1.2.4-1 Comparison of Tidal Levels

	Original Design	The Study for Port Development Planning in Kiribati	KAP (Kiribati Adaptation Project) by NIWA	The project for Expansion of Betio Port
	HHWL +2.45		SEAFRAME +3.00	HHWL +2.98
				HWL +2.79
			MHPWS +2.24 (king tide: every few months) / MHPWS +2.66	
	MHWS +1.80	HWL +1.84	MHWS +2.12 (every 14 days) / MHWS +2.54	
	MHWS(SEAFRAME)+2.54	HWL(SEAFRAME)+2.58		
			SEAFRAME +2.00	
	MSL(1974-1978) +0.94	MSL(1995) +0.95	MSL(2007) +1.22 / MSL(2007) +1.64 MSL(1980-1999) +1.20 / MSL(1980-1999) +1.62 MSL(1974-1977) +1.19 / MSL(1974-1977) +1.61	MSL +1.63
	MSL(SEAFRAME)+1.68	MSL(SEAFRAME)+1.69		
			SEAFRAME +1.00	
	MLWS +0.09	LWL +0.06		
	Datum University of Hawaii Gauge Zero (UoH) 0.0			
		0.74 m	Datum University of Hawaii Gauge Zero (UoH) 0.0	
			0.419 m	
			Datum SEAFRAME Gauge Zero 0.0	LWL +0.17
Source	JICA, "Basic Design of Betio Bairiki Causeway and Fishery Channel Project "(1985)	JICA, “The Study for Port Development Planning in Kiribati” (1995)	NIWA, “Kiribati Adaptation Project Phase II: Climate Information for Risk Management Coastal calculator operational handbook” (2010)	JICA, “The project for Expansion of Betio Port” (2010)

HHWL: Highest High Water Level/ HWL: Mean monthly-highest Water Level/ MHWS: Mean High Water Spring/ MHPWS: Mean High Water Perigean Spring
MSL: Mean Sea Level
LWL: Mean monthly-lowest Water Level/ MLWS: Mean Low Water Spring/

The observed tidal levels by SPSLCMP during 2003 through 2006, and the latest 5 years were analyzed and compared with ones adopted in “The project for Expansion of Betio Port”. The results in Table 1.2.4-2 show that the rise of the mean sea level (MSL) during 2003 and 2006 is not identified, however, the high water level (HWL) seems a small rise.

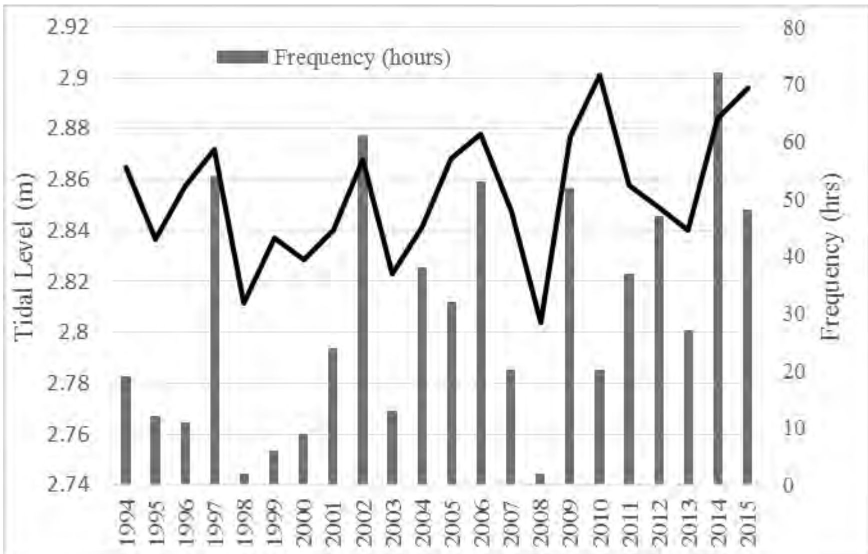
Table 1.2.4-2 Tidal Analysis on the basis of Observation Record at Betio Port

Year	2003	2004	2005	2006	Average
H.W.L.	2.78m	2.83m	2.81m	2.85m	2.82m
M.S.L.	1.65m	1.71m	1.66m	1.69m	1.68m
L.W.L.	0.61m	0.66m	0.59m	0.63m	0.62m
Record Missing	Feb, June, Oct-Dec	July		June	

Year	2010	2011	2012	2013	2014	Average
H.W.L.	2.77m	2.85m	2.87m	2.84m	2.91m	2.85m
M.S.L.	1.61m	1.67m	1.70m	1.68m	1.75m	1.68m
L.W.L.	0.56m	0.61m	0.63m	0.61m	0.68m	0.62m
Record Missing	Aug.	Oct.	Aug.		Oct.	

Source: The Study Team on the basis of Kiribati Meteorological Services(MET）

Although there is a description of king tide+2.66m in the analyses by NIWA, the tide larger than +2.80m is defined as the king tide in this study, and the frequency and annually averaged level of the king tide for the period from 1994 through 2015, and summarized in Figure 1.2.4-2. The average level of the king tide is +2.85m for that period, and is the same as HWL in Table 1.2.4-2. The maximum king tide during that period is +3.12m on 19th of February, 2015.



Source: The Study Team on the basis of Kiribati Meteorological Services(MET）

Figure 1.2.4-2 Annually Averaged Tidal Level and Frequency of King Tide

1.2.5 Climate Change

1.2.5.1 El Niño and La Niña Events

Japan Metrological Agency has published the list of phase period subject to El Niño/ La Niña events as indicated in Table 1.2.5-1. An overview of El Niño historical impacts to the Pacific Island Countries are presented in Table 1.2.5-2, and Kiribati is the country likely to face higher sea level and increased rainfall during El Niño.

Table 1.2.5-1 Phase of El Niño/ La Niña events

Phase of El Niño	Phase of La Niña
	Summer 1949 - 1950 Summer
Spring 1951 - 1951/52 Winter	
Spring 1953 - 1953 Autumn	Spring 1954 - 1955/56 Winter
Spring 1957 - 1958 Spring	
Summer 1963 - 1963/64 Winter	Spring 1964 - 1964/65 Winter
Spring 1965 - 1965/66 Winter	Autumn 1967 - 1968 Spring
Autumn 1968 - 1969/70 Winter	Spring 1970 - 1971/72 Winter
Spring 1972 - 1973 Spring	Summer 1973 - 1974 Spring
	Spring 1975- 1975 Spring
Summer 1976 - 1977 Spring	
Spring 1982 - 1983 Summer	Summer 1984 - 1985 Autumn
Autumn 1986 - 1987/88 Winter	Spring 1988 - 1989 Spring
Spring 1991 - 1992 Summer	Summer 1995 - 1995/96 Winter
Spring 1997 /1998 Spring	Summer 1998 - 2000 Spring
Summer 2002 - 2002/03 Winter	Autumn 2005 - 2006 Spring
	Spring 2007 - 2008 Spring
Summer 2009 - 2010 Spring	Summer 2010 - 2011 Spring
Summer 2014 -	

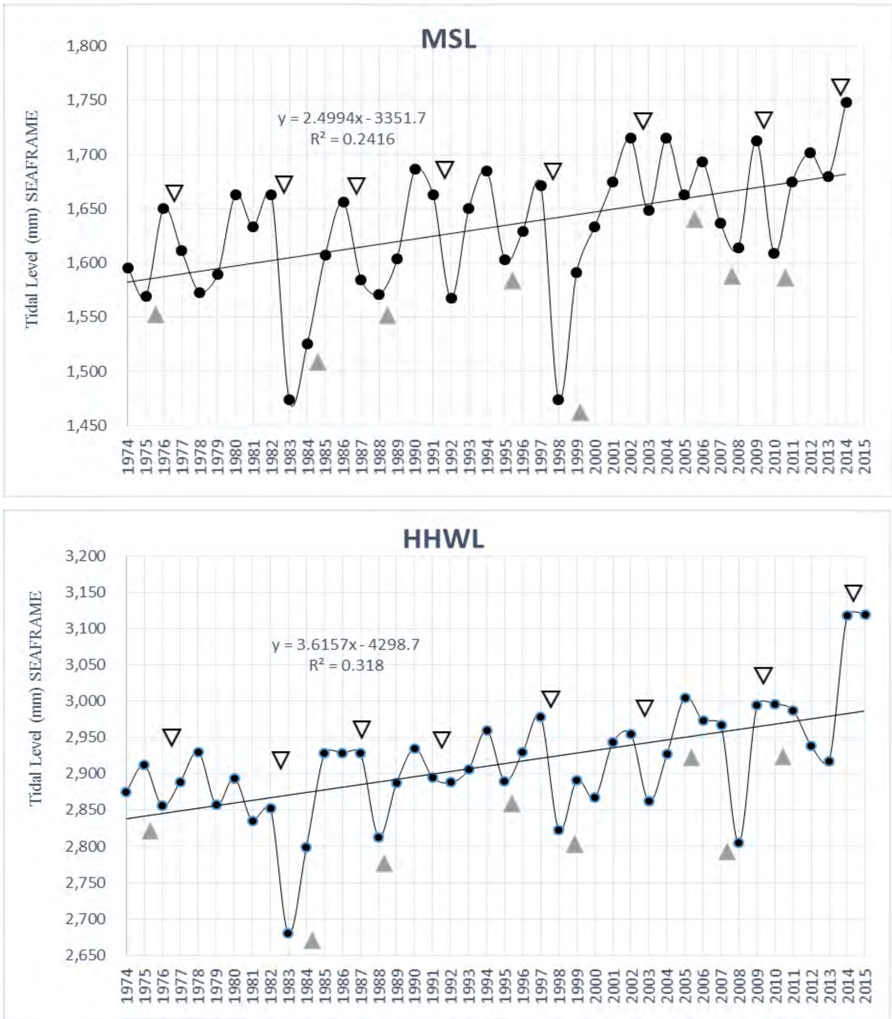
Source: Japan Meteorological Agency

Table 1.2.5-2 Historical Impact of El Niño to PIC

Country	Rainfall	Tropical cyclones	Sea level
Northern islands			
Federated States of Micronesia	Decreased	-	Lower
Marshall Islands	Decreased	(more intense)	Lower
Palau	Decreased	No impact	-
Central islands			
Kiribati	Increased	No impact	Higher
Nauru	Increased	-	Higher
Papua New Guinea	Decreased	Less frequent	Lower
Solomon Islands	Decreased	More frequent	Lower
Timor-Leste	Decreased	No impact	(no data)
Tuvalu	Decreased	More frequent	No impact
Southern islands			
Cook Islands	Decreased	More frequent	Higher
Fiji	Decreased	More frequent	Lower
Niue	Decreased	More frequent	(no data)
Samoa	Decreased	More frequent	Lower
Tonga	Decreased	More frequent	Lower
Vanuatu	Decreased	No impact	No impact

Source: Compiled from Australian Bureau of Meteorology & CSIRO, 2011

The variation of Mean Sea Level (MSL) and Highest High Water Level (HHWL) are summarized in Figure 1.2.5-1 for the year at the start of the tidal observation in Tarawa ; 1974 through the year of 2014. The phase of El Niño (▽) and La Niña （▲） are deeply related to the tidal vatiations . The both tidal levels of MSL and HHWL become higher during the phase of El Niño, and lower in the La Niña. MSL and HHWL tended to rise in the long term, and the magnitude of rise was larger in HHWL than that in MSL. Prior to the damage of the Nippon Causeway in 2014 and 2015, the fabriform mat of the revetment in the Betio Port were damaged in the end of the year 2002 during the phase of El Niño. HHWL in 2014 and 2015 was exceptionally high compared with that in the previous years, therefore it is understood that one of the major reason of the damages is the rise of maximum tidal level.



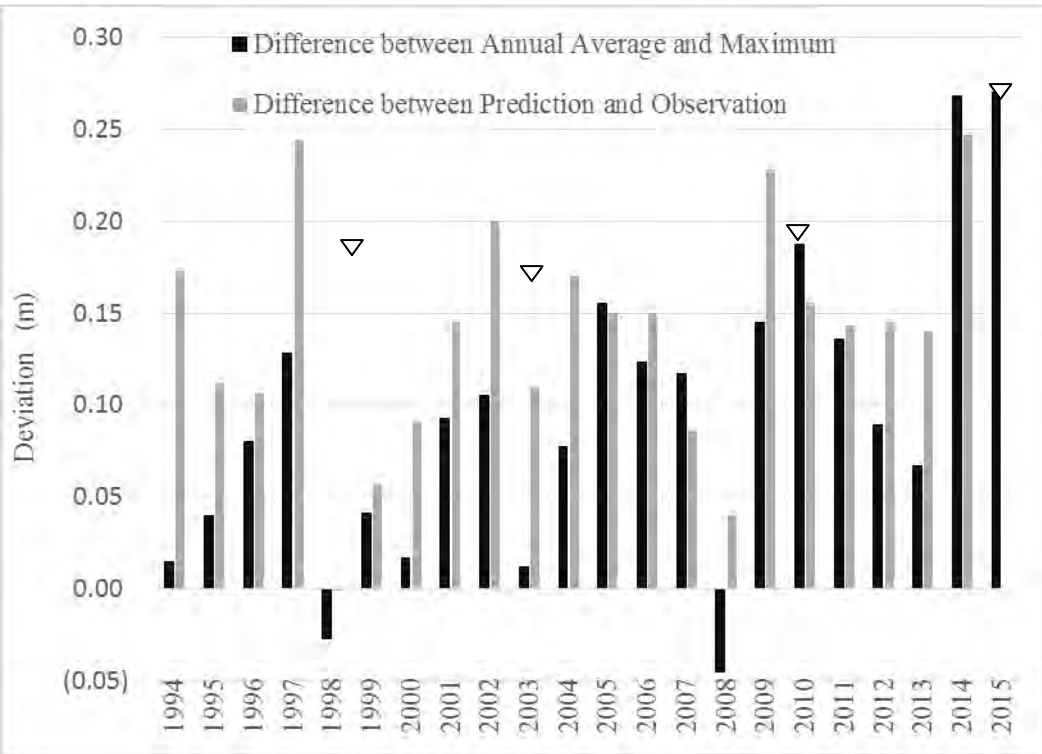
Source: The Study Team on the basis of Kiribati Meteorological Services(MET)

Phase of El Niño (▽) and La Niña（▲） with reference to *Table 1.2.1-1*.

The datum from 1974 through 1982 is adjusted by +0.419m in accordance with the analysis by NIWA..

Figure 1.2.5-1 Historical Change of Mean Sea Level (Top) and Highest High Water Level (Bottom) at Betio Port

As previously described, the averaged level of the king tide observed during 1995 and 2015 is +2.85m. The deviation of the maximum tidal level from the average, and the deviation of observation from the prediction are shown in Figure 1.2.5-2. The deviation range of 20cm to 25cm is seen particularly in the year of El Nino.

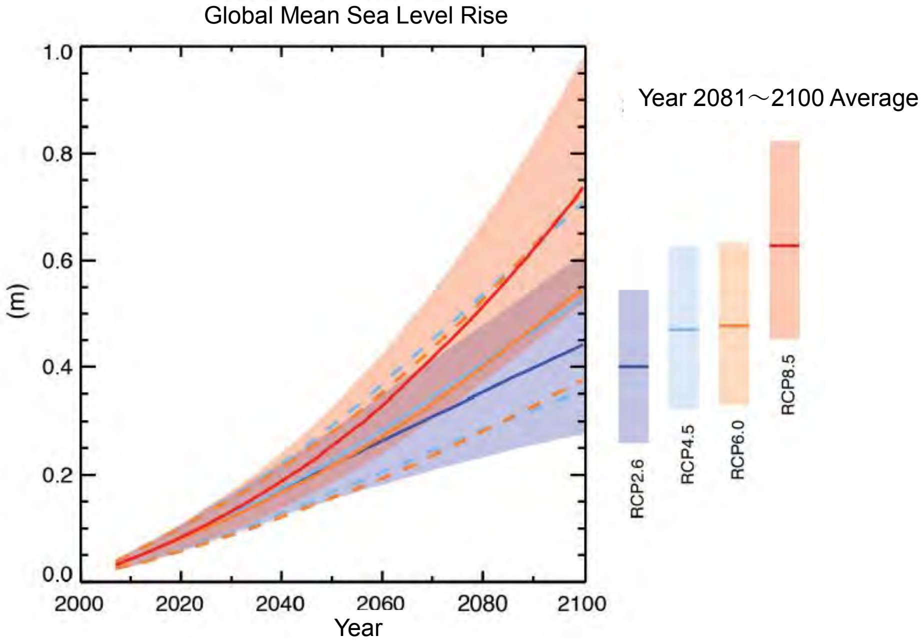


Phase of El Niño (▽) with reference to *Table 1.2.1-1*

Source: The Study Team on the basis of Kiribati Meteorological Services(MET)

Figure 1.2.5-2 Difference between Annual Maximum and Average/ between Prediction and Observation of King Tide

1.2.5.2 Mean Sea Level Rise due to Emission Scenario



Source: Translation of Summary by Japan Meteorological Agency (2015) "Climate Change 2013 – The Physical Science Basis"

Figure 1.2.5-3 Prediction of Global Mean Sea Level Rise (IPCC AR5)

The mean sea level rise of 10cm in 2025 and 20 cm in 2045 are predicted in the severest scenario (RCP8.5) used in IPCC AR5 shown in Figure 1.2.5-3.

1.2.6 Wave

No wave observation has been carried out. Wave is generated by the prevailing and constant eastern trade wind year around, and propagated as swell to reach at the atoll. There also exists strong west wind in spite of low occurrence generating relatively large wind wave inside the atoll, and west offshore wave approaches from the west opening of the atoll as well.

On the basis of the wave observation carried out for 1 month (December 1976) at the original design, the wave height of 0.1m – 0.45m and the wave period of 1 – 17 sec on the east reef side (offshore side), and the wave height of 0.15m – 0.77m and the wave period of 2 6 sec at Betio port (inside of lagoon) were reported.

Since there was no record of wave observation, an extreme offshore wave height was estimated at the original design by means of the following methods;

1.2.6.1 Lagoon Side

A probabilistic wind velocity by direction was estimated using the information on the annual maximum wind velocity by direction (the period of 1948-1984) and assumed probabilistic distribution (Gumbel). With 50 years occurrence wind velocity (15.1 m/s) and the longest effective fetch of North direction, the wave height (H=1.14m) inside the lagoon was hindcasted by means of Bretschneider.

1.2.6.2 Ocean Side

The maximum wave height (H=5m) and the wave period (T=9sec) were obtained from the offshore information at the adjacent area published by Japan Coast Guard, and the maximum wind velocity (20.56 m/s) was assumed. An effective fetch and wind duration were estimated using SMB diagram to meet the obtained conditions. As the Method A, the effective fetch (250 km) and the wave period (8.6 sec) and the wind duration (13 hrs) were obtained with the maximum wave height and wind velocity. As the Method B, the effective fetch (350 km) and the wave height (5.5m) and the wind duration (16 hrs) were obtained with the maximum wave period and wind velocity. After considering attenuation distance, they adopted the effective fetch of 250km and the wind duration of 14 hrs for the hindcasting with the 50 years probability wind velocity, and obtained the offshore wave height by directions (6.1m and 9.3 sec for SW) as indicated in Table 1.2.6-1.

Table 1.2.6-1 Extreme Offshore Wave hindcasted in the Original Design

			Wave on Ocean Side			Lagoon Side
Wind Direction			SW	S	SE	NW
Wind Speed		(m/sec)	23.5	15.4	15.6	23.3
Estimation based on Effective Fetch	Wave Height	$H_{1/3}$ (m)	6.2	3.5	3.5	6.2
	Period	T (sec)	9.5	7.5	7.5	9.3
Estimation based on Wind Duration	Wave Height	$H_{1/3}$ (m)	6.1	3.3	3.3	6.1
	Period	T (sec)	9.3	7.1	7.1	9.3
Used Values	Wave Height	$H_{1/3}$ (m)	6.1	3.3	3.3	6.1
	Period	T (sec)	9.3	7.1	7.1	9.3

Source: Basic Design of Betio Bairiki Causeway and Fishery Channel Project (1985)

Tonkin & Talor International estimated the probabilistic offshore wave height on the basis of 10 years (1997-2007) wave information in WAVEWACH III provided by NOAA. According to Table 1.2.6-2, the 50 years probability offshore height (2% AEP) and its period were 3.26m and 7.12 sec for the Ocean side and 1.54m and 2.97 sec for the Lagoon side, respectively.

Table 1.2.6-2 Extreme Offshore Wave by T&TI

Event	Lagoon side			Ocean side		
	H_s (m)	T_m (s)	T_p (s)	H_s (m)	T_m (s)	T_p (s)
10% AEP	1.34	2.77	3.38	2.82	6.62	8.08
2% AEP	1.54	2.97	3.62	3.26	7.12	8.69
1% AEP	1.64	3.06	3.73	3.48	7.36	8.97

Source: Tonkin & Taylor International Ltd,"Preparation of Remedial Design for Dai Nippon Causeway Site Investigation and Concept Design"

In the Study of Rehabilitation for Betio Port, the frequency of the offshore wave height and period at the adjacent area were presented in Table 1.2.6-3 with reference to Global Wave Statistics by British observation on board and hindcast. An occurrence of wave height larger than 6m was very rare, but observed as 0.1%. However, there existed no further record for a larger wave height, the wave height of 6.1m used for the original design is considered reasonable.

Table 1.2.6-3 Frequency of Offshore Wave Height and Period at the Adjacent Area

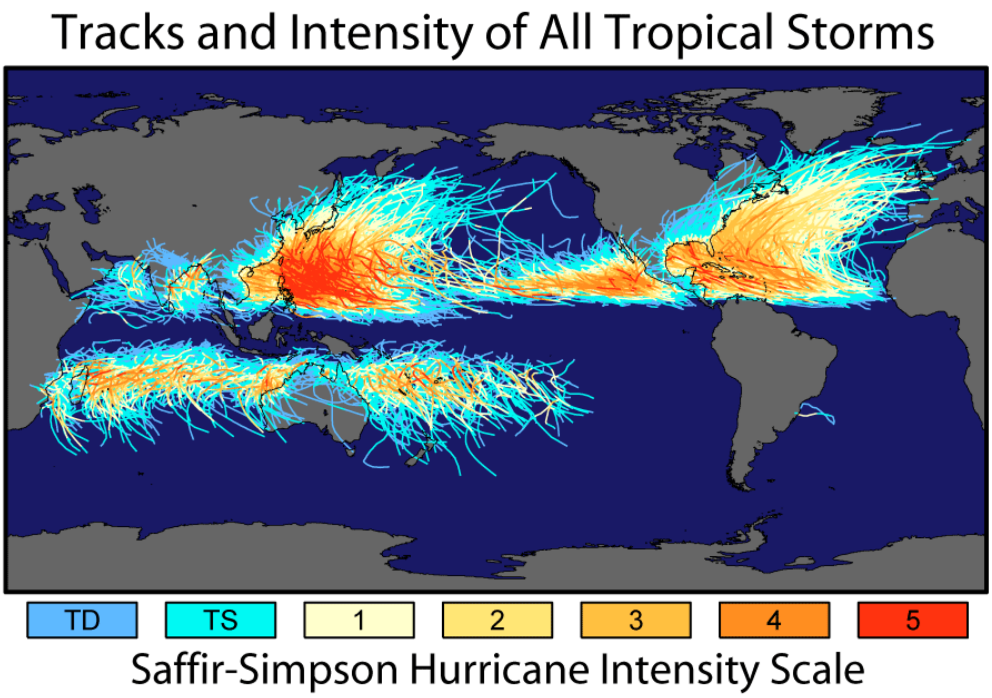
SIGNIFICANT WAVE HEIGHT (M)	SIGNIFICANT WAVE PERIOD (SEC)											Total
	<4	4>5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	>13	
>14	-	-	-	-	-	-	-	-	-	-	-	-
13-14	-	-	-	-	-	-	-	-	-	-	-	-
12-13	-	-	-	-	-	-	-	-	-	-	-	-
11-12	-	-	-	-	-	-	-	-	-	-	-	-
10-11	-	-	-	-	-	-	-	-	-	-	-	-
9-10	-	-	-	-	-	-	-	-	-	-	-	-
8-9	-	-	-	-	-	-	-	-	-	-	-	-
7-8	-	-	-	-	-	-	-	-	-	-	-	-
6-7	-	-	-	-	-	-	-	-	-	-	-	0%
5-6	-	-	-	0%	0%	-	-	-	-	-	-	0%
4-5	-	-	0%	0%	0%	0%	0%	-	-	-	-	1%
3-4	-	0%	1%	2%	1%	1%	0%	0%	-	-	-	5%
2-3	0%	2%	5%	6%	4%	2%	1%	0%	-	-	-	19%
1-2	1%	7%	16%	13%	6%	2%	0%	0%	-	-	-	45%
0-1	3%	10%	10%	5%	1%	0%	-	-	-	-	-	30%
Total	4%	19%	32%	26%	13%	5%	1%	0%	0%	0%	0%	100%

Source: The Study of Rehabilitation for Betio Port (2007)

1.2.7 Cyclone Path

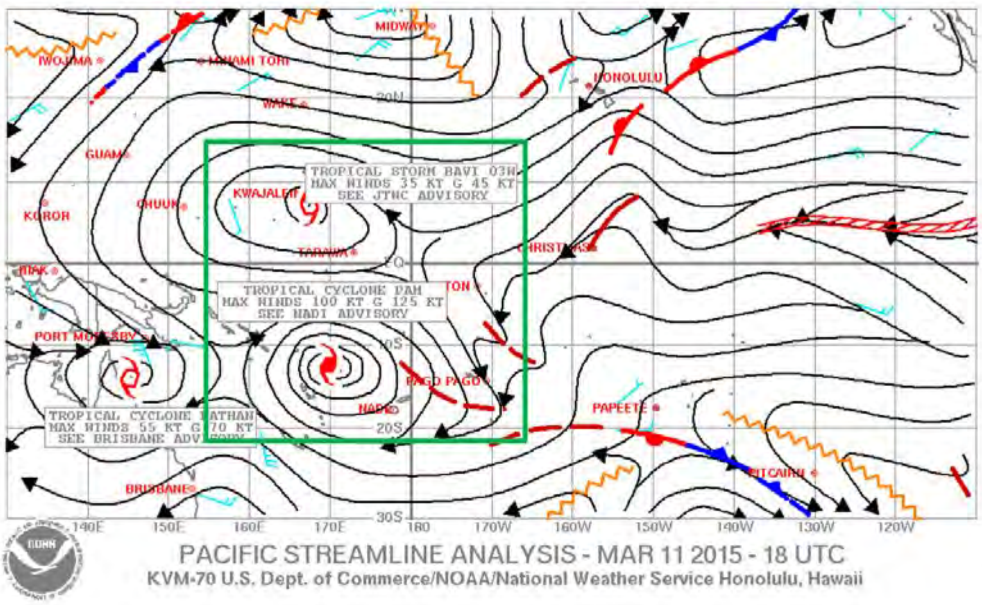
With reference to the cyclone path of the past 150 years (Figure 1.2.7-1), very few of the cyclone passing through Kiribati was recorded. It is because that the air stream does not get the needed spin to grow a cyclone along the Equator around where Coriolis force is very weak. However, we should know a potential of a cyclone affecting seriously to Kiribati, considering the fact that the cyclone Pam caused significant damage to Kiribati including the causeway on March of 2015.

According to the report by Kiribati Meteorological Service, the damage by the cyclone Pam expanded due to the tropical storm which was generated simultaneously. Figure 1.2.7-2 indicates the location of the cyclone Pam and the tropical storm at the both side along the Equator on 11th of March 2015. This situation was likely to intensify the concentration of western wind and wave which approached to Tarawa. Since the cyclone Pam caused the damage in spite of away path (Figure 1.2.7-3), it is also understood the reason that the impact by swell and sea level rise became strong subject to the development of the cyclone Pam, and the south coast of Tarawa was facing swell approaching direction.



Source: NASA Earth Observatory

Figure 1.2.7-1 Past Typhoon and Cyclone Paths



Source: Kiribati Meteorological Service

Figure 1.2.7-2 Streamline Analysis around Kiribati on 11th of March 2015

Source: Map of World
(http://www.mapsofworld.com/hurricane/cyclone-pam-in-south-pacific-ocean.html)

Figure 1.2.7-3 Path Map of Cyclone Pam

1.2.8 Coastal Stream (Current)

1.2.8.1 Original Design

In the original design report, the flow volume and current speed were calculated on the basis of the assumption on flow sections on the reef after and before the construction of the causeway. The results showed that the flow volume reduced to 1/30, and the current speed increased to 3 m/s (the maximum) at the fisheries channel. It was concluded that the construction of the causeway did not affect significantly to the current condition at the atoll, since the causeway (with the length of 3.4km, the averaged water depth of 1.3m, and the area of 4,420m^2) blocked small area of 4% (4,420/100,800) assuming the water depth against the west opening of the atoll (with the length of 24km, the averaged water depth of 4.2m, and the area of 100,800 m^2) which was prevailing current exchange between inside and outside of the atoll. It was verified with the further current analysis that the change of the current condition was small after and before construction of the causeway.

For additional transverse channel for water exchange, if newly planned with the size of the existing one (area of 40 m^2), the above review suggests that the new channel will not able to significantly contribute water exchange of the atoll since that area becomes less than 0.1% of the opening.

1.2.8.2 JICA/SOPAC Study Report (1995)[1]

There was the study report of erosion at Tarawa atoll conducted by JICA/ SOPAC. The report described that “There has been substantial accretion along the Nippon Causeway since construction was finished in 1987. As of February 1988 (18 months after the start of construction and 7 months after completion), a total sediment volume > 108,000 m^3 had accumulated along the causeway.” There was also concern on the water quality and reef productivity from the viewpoints of tidal exchange which the report commented the reduction by 95% to 97% or more.

1.2.8.3 T & TI Study Report (1995)[2]

This report qualitatively described that prior to the construction of the causeway the lagoon currents adjacent the south Tarawa islands were much weaker with water entering the lagoon

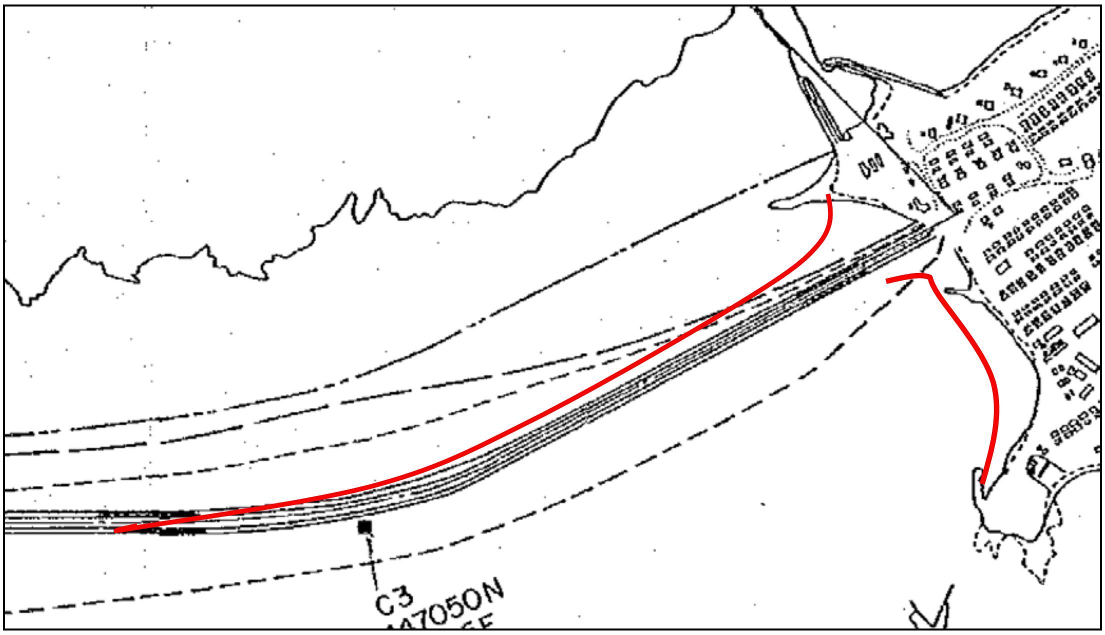
[1] Forbes & Hosoi, “Coastal Erosion in South Tarawa, Kiribati” SOPAC Technical Report 225

[2] Tonkin & Taylor International Ltd, ”Preparation of Remedial Design for Dai Nippon Causeway Site Investigation and Concept Design”

across the reef from the south during flood tides and leaving during ebb tides. The report also pointed out focusing on deposit that sedimentation was occurring as sandbars forming on both the lagoon, and particularly on the ocean side approximately 150 m from the causeway bridge as cross-shore velocities reduce away from the bridge while the high velocities under the bridge, essentially creating an artificial ebb- and flood-tide deltas. It was predicted that residence time would increase from between 1and 5 days before the causeways to up to 70 days after. There was a comment about a flow toward the lagoon on ebb tide and toward the ocean on flood tide, opposite to that predicted.

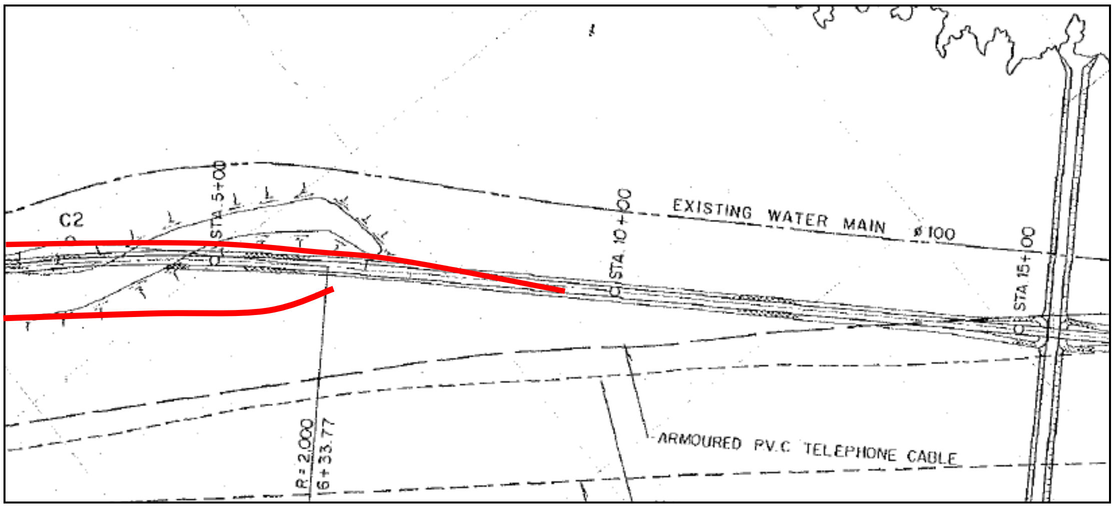
1.2.9 Coast

As formerly mentioned, JICA/SOPAC Study reported that a total sediment volume more than 100,000 m3 had accumulated along the causeway. (18 months after the start of construction and 7 months after completion). Figure 1.2.9-1 and Figure 1.2.9-2 illustrate a comparison with the original coastal line before the construction of the causeway and the present, showing the accumulation.



The present coastal line shown in the red line.
Source: The Study Team

Figure 1.2.9-1 Sedimentation of Bairiki Side



The present coastal line shown in the red line.
Source: The Study Team

Figure 1.2.9-2 Sedimentation of Betio Side

1.2.10 Geological Survey

1.2.10.1 General

The purpose of the Survey is to acquire geotechnical data to carry out the detailed design of the project for reconstruction of Nippon causeway on Tarawa. The Geological Survey of this project is consisted with two phases, phase I is microtremor survey survey in July and phase II is drillings and laboratory tests in September. The summary of the phase I survey is shown in Table 1.2.10-1 .

Table 1.2.10-1 Summary of Geological Survey

Particlars	Number	Chainage	Position	Easting			Northing			Elevation	Handy Penetration Test		
		Final		°	'	"	°	'	"	m	Ocean	Lagoon	Embankment
Microtremor Array Survey	M-1	64+23	R	1	19	50.56	172	58	26.60	3.97	*		
	M-2	57+46	R	1	19	52.91	172	58	16.24	4.12	*	*	
	M-3	49+32	R	1	19	57.62	172	58	04.04	4.06	*	*	
	M-4	43+33	L	1	20	03.36	172	57	56.12	1.00	*	*	*
	M-5	37+0	L	1	20	10.10	172	57	47.75	1.00	*	*	*
	M-6	28+33	O	1	20	20.19	172	57	38.71	0.50	*	*	*
	M-7	25+18	O	1	20	23.15	172	57	34.09	1.52	*	*	*
	M-8	19+4	R	1	20	30.55	172	57	27.03	4.02	*	*	
	M-9	11+5	R	1	20	39.50	172	57	17.72	4.02	*	*	
	M-10	0+10	O	1	20	50.06	172	57	03.36	4.16	*	*	
	M-G			1	19	44.61	172	58	36.67	4.00			
Bore hole	M-3	19+4	R	1	19	57.62	172	58	04.04	4.06			
	M-6	28+33	R	1	20	21.19	172	57	38.71	5.51			
	M-8	42+33	R	1	20	30.55	172	57	30.55	4.02			

1.2.10.2 General Topography and Geology

(1) Topography

Nippon causeway was constructed in the shallow ocean between Betio island and Bairiki island. This area is located in the atoll named Tarawa, the road was set in the line connecting shallower shore and islets by coral sediments.

The causeway separates the topography into the ocean side and the lagoon side, both of them are at the shallow lagoon (called "moat"). The depth of sea water along the road changes as its potion and the tidal range. The land (not submerged) part distributes near the islands especially at the lagoon side.

(2) Geology

The geology of the subsurface in Tarawa island is consisted by the following formations from top to bottom (Marshall1985).

1-Cemented reef top sediment (cay rock)

2-Unconsolidated sediment (sand and gravel)

3-Corals

4-Leached limestone

Most part of the causeway base ground is consisted by unconsolidated sediments, which tends to increase its thickness and fine grain proportion toward lagoon center (eastward).

Figure 1.2.10-1 Arial view of the cause way from west (photo taken in 1943:left, 2015:right)

https://en.wikipedia.org/wiki/Battle_of_Tarawa : Aerial view of Betio Island, Tarawa Atoll before invasion of the island by U.S. Marines, 18 September 1943. The image was shot by an aircraft from Composite Squadron (VC) 24.

(3) Result

The every survey pointd are shown in Figure 1.2.10-2.

1） Drilling survey

The drilling survey was conducted at 3 points where are the lagoon side of the road near the top of the embankment slope. The drilled depth is 15m in every borehole.
The classification of the drilled layers correlated with the prior study and the result of the microtermor array survey is shown in Figure 1.2.10-3.
Road embankment is consisted by the unconsolidated sediment(sand and gravel)
Cemented reef top sediment(cay rock) is a slightly consolidated reef distributes in the unconsolidated sediments.
Unconsolidated sediments is the most typical layer of the coaseway basement ground consisted by sand and gravels. The gravel size is less than 5cm in average.
The ground water usually distributed at the depth of GH-2.0~3.0m.

2） Microtremor array survey (MAS)

Microtremor array survey(MAS) was conducted at 11 sites shown in Figure 1.2.10-2 and was penetrated the depth of 15m to 70m in the subsurface ground.
The Figure 1.2.10-3 shows the soil structure at the Nippon causeway correlated the S wave velocity structure analyzed by the MAS. Mean Converted N value is calculated by the formula -(1). And shows the s-wave velocity structure in the causeway area.

$Vs=89.8N^{0.341}$ （$N=\log_{0.341}$（Vs/89.8)） ---(1) Imai's formula

The distribution of formations are shown in Table 1.2.10-2.

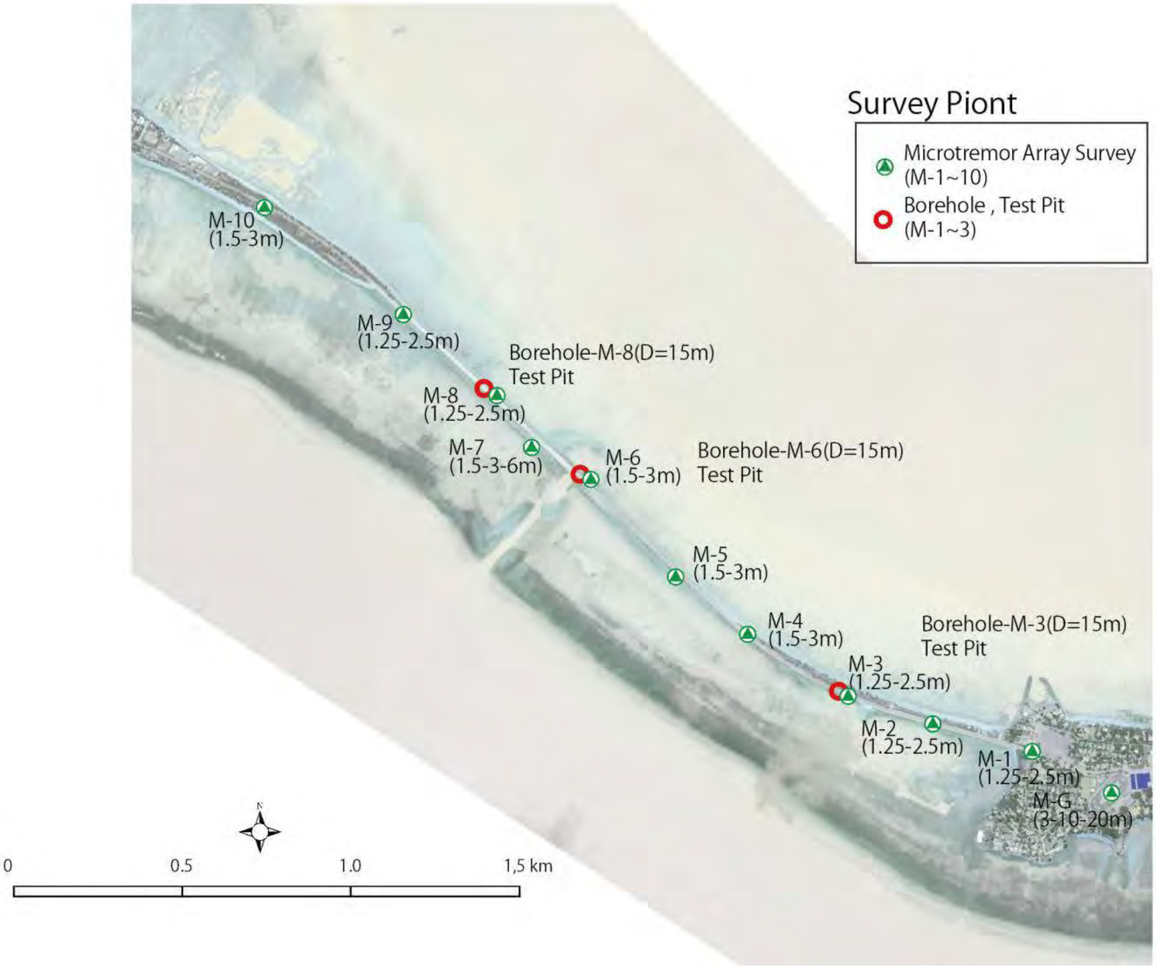
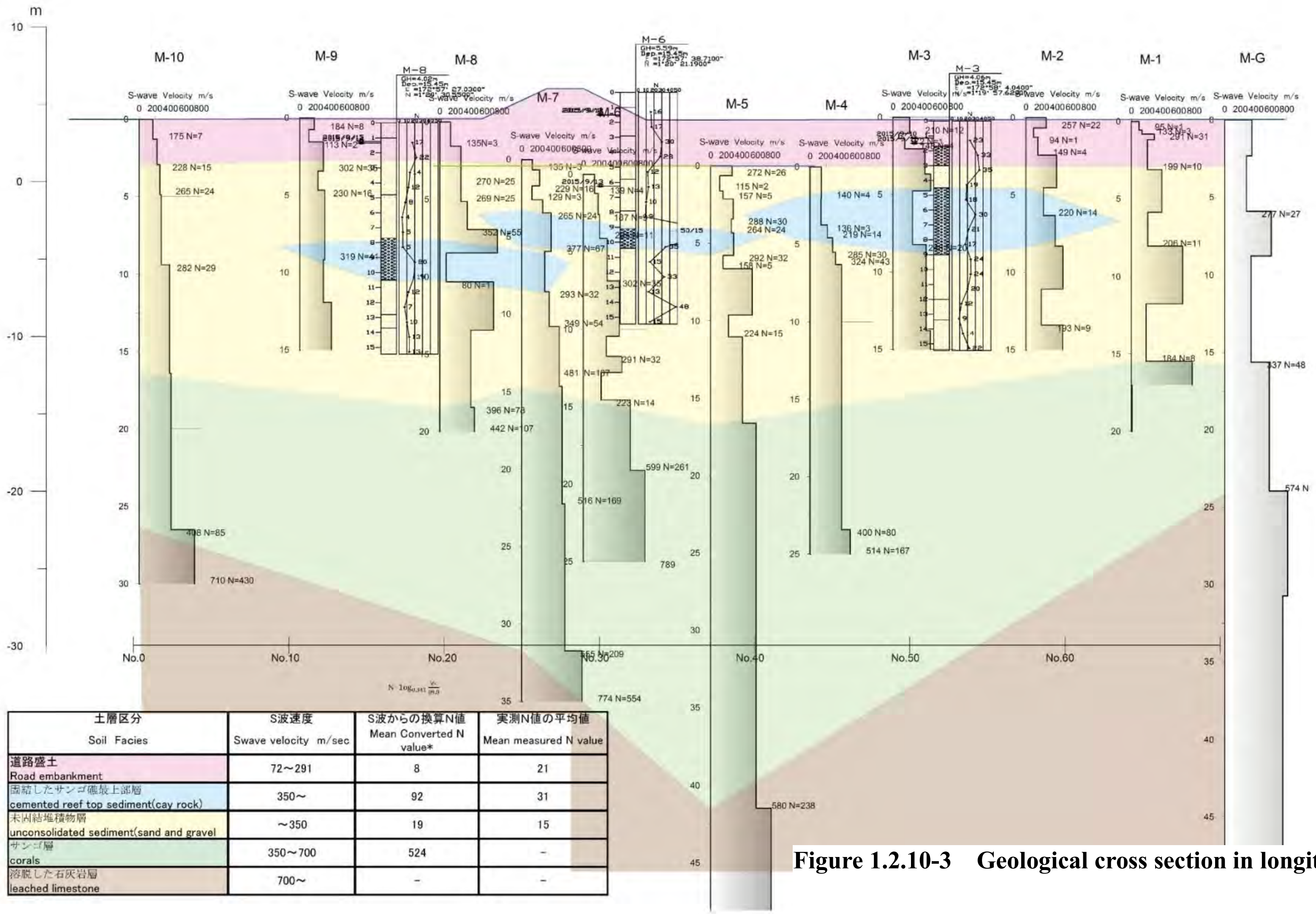


Figure 1.2.10-2 Map of the survey points

Table 1.2.10-2 Stratigraphic formations

Soil Facies	Swave velocity m/sec	Mean Converted N value*	Mean measured N value*
Road embankment	72～291	8	21
cemented reef top sediment(cay rock)	～500	19	31
unconsolidated sediment(sand and gravel	～500	19	15
Corals	500～600	50~	-
leached limestone	600～	-	-



土層区分 Soil Facies	S波速度 Swave velocity m/sec	S波からの換算N値 Mean Converted N value*	実測N値の平均値 Mean measured N value
道路盛土 Road embankment	72～291	8	21
固結したサンゴ礁最上部層 cemented reef top sediment(cay rock)	350～	92	31
未固結堆積物層 unconsolidated sediment(sand and gravel	～350	19	15
サンゴ層 corals	350～700	524	-
溶脱した石灰岩層 leached limestone	700～	-	-

Figure 1.2.10-3 Geological cross section in longitude direction

1.2.11 Traffic Count Survey Results

The traffic count survey of Nippon Causeway is conducted to understand the traffic condition of Nippon causeway and to summarise necessary based data for the pavement structure design and the traffic future forecast. The traffic count survey condition and results are shown in Table 1.2.11-1 and Table 1.2.11-2.

Table 1.2.11-1 Traffic Count Survey Condition

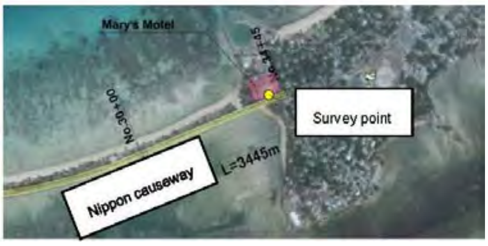
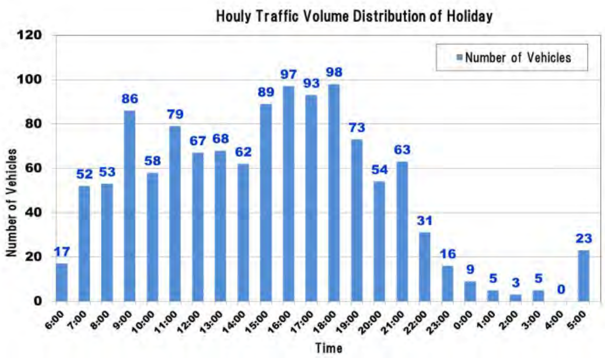
Items	Contents	
Date	Date 1 : 21th June 2015 (Sun) 6:00 a.m ~ 22th June 2015 (Mon) 6:00 a.m Date 2 : 24th June 2015 (Wed) 6:00 a.m ~ 25th June 2015 (Thu) 6:00 a.m	
Output	- 24hours Traffic Volume (Weekday) - 24hours Traffic Volume (Holiday)	
Target	(1) Padistrian (3) Motorcycle (5) Small Bus (7) 2 Axle Trailer	(2) Bicycle (4) Car or Taxi (6) Large Bus (8) 3 Axle or More Trailer
Site Location	- Nippon Causeway (Bairiki Side) 	

Table 1.2.11-2 Traffic Count Survey Results

Survey Date	Traffic Volume※1 (car / 24 hour)	Heavy Traffic Volume※2 (car / 24 hour)	PCU※3 (Passenger Car Unit)
June 21th (Sun)	2110	308	2485
June 24th (Wed)	3894	835	5015

※1 : Sum of Motorcycle, Car or Taxi, Small Bus, Large Bus, 2 Axle Trailer and 3 Axle or More Trailer.
※2 : Sum of Large Bus, 2 Axle Trailer and 3 Axle or More Trailer.
※3 : Correction factor for the PCU conversion is shown below.
Motorcycle: 0.5, Car or Taxi: 1.0, Small Bus: 1.5, Large Bus:2.0, 2 Axle Trailer: 2.0, 3 Axle of More Trailer 3.0



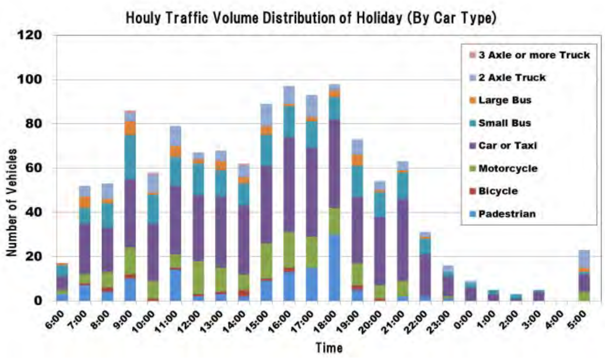


Figure 1.2.11-1 Hourly Traffic Volume Distribution of Holiday (To Bairiki)

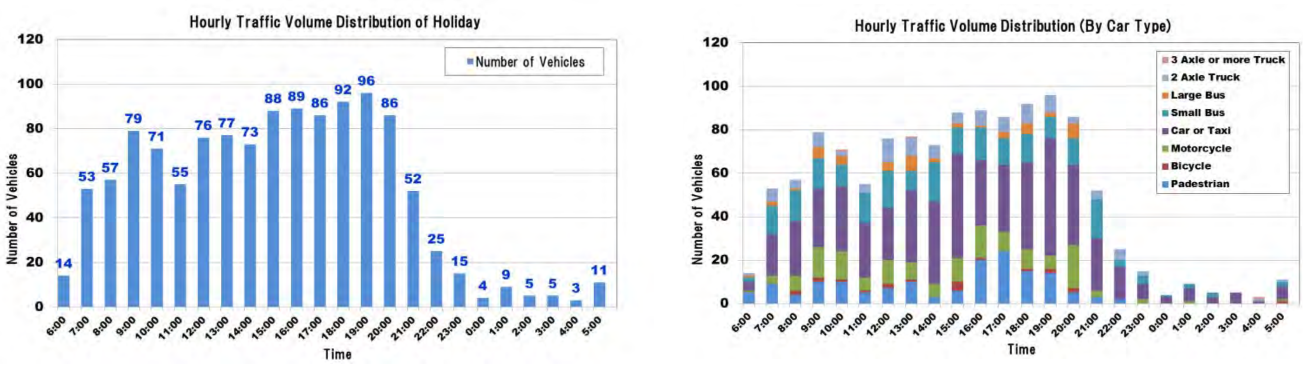


Figure 1.2.11-2 Hourly Traffic Volume Distribution of Holiday (To Betio)

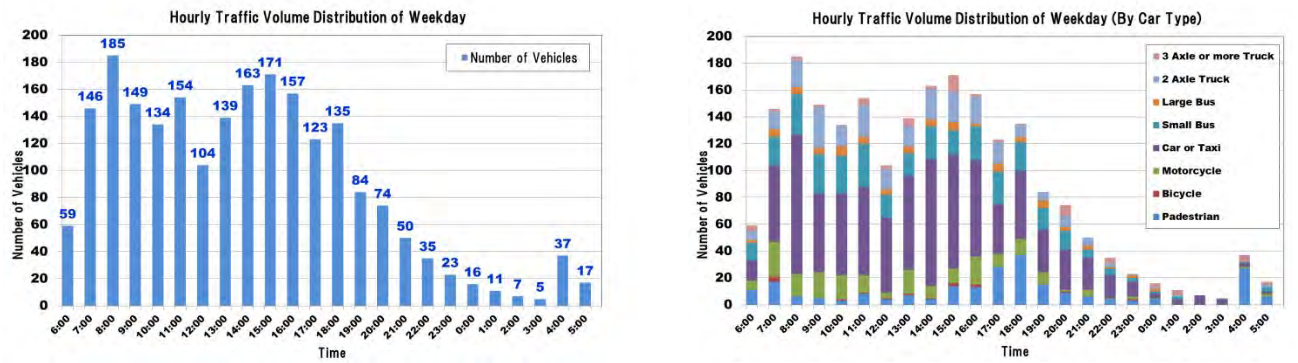


Figure 1.2.11-3 Hourly Traffic Volume Distribution of Weekday (To Bairiki)

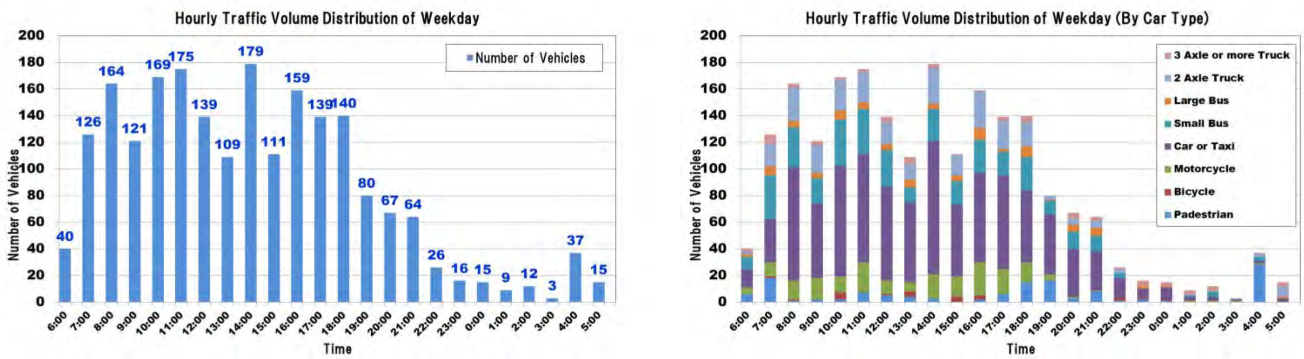


Figure 1.2.11-4 Hourly Traffic Volume Distribution of Weekday (To Betio)

1.2.12 Axle Load Survey

The Axle load survey will be carried out to collect the actual loading data of heavy vehicle and for the basic information of pavement design. The survey equipment is manual weight scale. In results of the axle load survey, more than 3 axle trucks are about 12% of the total. Average ESAL is used as the condition of the pavement structure design (AASHTO).

Table 1.2.12-1 Survey Condition

Items	Contents		
Date	14 September, 2015 Monday 9:00 a.m～17:00 p.m.		
Target	(1)Bus	(2) 2 AXLE Trailer	(3) 3 AXLE Trailer
			
	(4) 4 AXLE Trailer	(5) 5 ~6 AXLE Trailer	
			
Location	・NIPPON CAUSEWAY (BETIO side)		
			

Table 1.2.12-2 Axle Load

Design Condition		Car	Small Bus	Large Bus	2-Axle Truck	3- or 4-Axle Truck	5- or 6-Axle Truck
Traffic Volume		1,946	705	163	535	116	21
Average Axle Load (kip)		1.00	1.00	6.51	6.51	22.20	40.20
Load Equivalency Factor	1st Axle	0.0002	0.0002	0.0031	0.0031	0.0610	0.7624
	2nd Axle	0.0002	0.0002	0.0017	0.0017	0.1678	0.2774
	3rd Axle	-	-	-	-	0.0723	0.4114
	Total	0.0004	0.0004	0.0048	0.0048	0.3010	1.4512
Design ESAL		8,592	3,113	8,550	28,064	385,425	336,405
Total ESAL		770,149					

1.3 Environment and social considerations

1.3.1 Status of environmental license acquisition

In accordance to the Environment (Amendment) Act 2007, any project that potentially involves environmentally-significant activities must obtain an "Environmental License" from the Ministry of Environment Lands and Agricultural Development (MELAD). The type of activities that require "Environmental License" are prescribed under the Environment (Amendment) Act 2007, where causeway is included under the public works sector. Upon reviewing the application document for the Environment License, MELAD duly requested MPWU to submit a Basic Environmental Impact Assessment (BEIA) report for approval.

The JICA expert team prepared the draft BEIA document and submitted to MPWU in December 2015 (the draft BEIA is attached to this report as Appendix -1). MPWU then submitted the final BEIA to MELAD in January 2016. The Environmental License is expected to be obtained by the end of April 2016.

1.3.2 Existing environment around the Causeway

1.3.2.1 Pollution

The JICA expert team conducted water and sediment quality surveys around the Causeway. Although no significant water pollution was identified, the lagoon side had higher levels of turbidity, Total Nitrogen and Total Phosphorus compared to the ocean side. This is understandable as there is less water exchange in the lagoon side. No signs of sediment pollution were detected.

Due to the bad road condition, dust and noise emissions from the passing vehicles are an ongoing issue. The JICA expert team conducted air quality survey (PM10, NO_2, SO_2) in February 2016 to understand the level of pollution around the Causeway. The results showed that the level of air pollution around the Causeway was still within acceptable levels compared to international standards. The environmental condition around the Causeway is expected to improve through the Project by resurfacing the roadway.

1.3.2.2 Natural environment

The JICA expert team have conducted marine and terrestrial ecosystem surveys around the Causeway. No sensitive marine habitats (e.g. corals) or endangered marine species were found in the vicinity of the Causeway. Although two IUCN threatened species (one coral species and one fish species) were found in the reef slope area, these species are likely to be unaffected by the Project as the reef slope area is located far from the Causeway (around 400-500 m).

A total of 17 terrestrial vegetation species were recorded consisting of trees, shrubs, herbaceous plant and vines. None of the identified species are classified as threatened under the IUCN Red List.

1.3.2.3 Social environment

The Causeway and the sandy beach that was naturally formed after the Causeway construction are owned by the state. No new land acquisition will be required under this project as reconstruction works will be done within the boundary of the existing Causeway.

The lands adjacent to the Causeway landing area in Bairiki and Betio are designated as a commercial and an open space area respectively. The reconstruction works will not cause any alteration to the current land use plan.

The shallow reef flat area adjacent to the Causeway is used by the locals for fishing and bathing. A boat channel also exists at the bridge section. Although, there will be temporary water use restrictions around the construction area, impacts of such restrictions are likely to be minor as the restrictions will be limited in area and duration.

1.3.3 Potential environmental impacts and mitigation measures

1.3.3.1 Construction phase

The Causeway reconstruction works are expected not to cause any significant negative environmental impacts, primarily because the project does not entail any loss of natural environment, change in land use and resettlement.

One of the main environmental concerns during the construction phase is the potential pollution that may arise from the operation of the asphalt and concrete plants and waste generation. The following are the main mitigation measures planned for the asphalt and concrete plants and waste generation.

- The main concern for the asphalt plant is the dust generated from the aggregate drying process. Dust emission from this process is planned to be reduced significantly by installing primary and secondary dust collection units.
- The main concern for the concrete plant is the concrete wash water generated from agitator washout and charging areas, slumping station and so on. The wash water will include concrete materials (e.g. cement, sand, aggregates) and will be highly alkaline. The plant will be designed so that all wash water (including contaminated storm water) are retained on site by collecting and diverting the wash water to an impermeable settling pond, and reusing the captured wash water. Discharge of wash water will only be allowed provided that pH and suspended solid levels are within the World Bank discharge standard (pH: 6-9, suspended solids: < 50 mg/l).
- Waste management is a key issue, especially since South Tarawa has limited landfill capacity and has no facility to receive hazardous wastes. Waste volume will be minimized by promoting 3R (reduce, reuse and recycle). Wastes that cannot be appropriately reused/recycle or disposed in South Tarawa is planned to be transported and disposed overseas.

While impacts from the asphalt and concrete plants are planned to be avoided or minimized by implementing strict pollution control measures, it is also important that these facilities are to be located as far as possible from sensitive areas (e.g. residential areas).

1.3.3.2 Post-construction phase

In accordance to the scoping process, no negative impacts were identified for the post-construction phase. In fact, due to the better road condition, the local environment is expected to improve significantly as there will be less dust and noise emitted from the passing vehicles.

1.3.3.3 Environmental management plan and monitoring plan

Based on the environmental assessment, an Environmental Management Plan (EMP) and monitoring plan were prepared to ensure that reconstruction works are implemented with minimal environmental impacts. The EMP summarizes the planned mitigation measures against the anticipated environmental impacts, the responsibility for its implementation and supervision, and estimated cost. The mitigation and monitoring costs will be included in the project budget to ensure its implementation.

However, since the construction plan and methods, including the location, layout and specifications of the asphalt and concrete plants, are to be determined at a later stage by the construction contractor, the EMP and monitoring plan should be considered as a tentative document and be revised/finalized at later appropriate stage through consultation with MELAD and local stakeholders, and obtain approval from MELAD accordingly.

MPWU is required to regularly submit to JICA the results of the environmental monitoring using for example the attached Environmental Monitoring Form.

1.3.3.4 Stakeholder meeting

A public consultation meeting was held on September 11th, 2015 at KNYC Maneaaba. The purpose of the meeting was to inform the stakeholders and public about the planned reconstruction works of the Causeway and the scoping of environmental impacts. The stakeholders and public were invited by sending invitation letters and through public radio announcement.

Around 20 people participated in the meeting including local residents, relevant government agencies and Australian and New Zealand Commission. None of the participants raised any objections towards the project’s plan and design as their concerns were answered. The minutes of the meeting is attached to the draft BEIA report.

1.3.3.5 Conclusion and recommendation

As mentioned previously, no significant negative environmental impacts are expected to occur through the Causeway reconstruction works, provided that the EMP and monitoring plan are appropriately implemented. The following are recommendations or actions necessary for the ensuring project stages:

- The asphalt and concrete plants should be located as far as possible from the sensitive areas (e.g. residential areas).
- The EMP and monitoring plan should be revised/finalized in the process of finalizing the construction plan through consultation with local stakeholders and obtain approval from MELAD.
- To ensure that the EMP and monitoring is effectively implemented, a qualified and experienced environmental officer should be assigned to the contractors team and supervising consultant.

Chapter 2 Contents of the Project

2.1 Basic Concept of the Project

2.1.1 Background

2.1.1.1 Situation of Republic of Kiribati

- Population is 103,000 people, the area is 730 km^2 consisting of 33 atolls with very large exclusive economic zone which is 3,550,000 km^2 of the third place of the world.
- The causeway with a length of 3.2 km and a width of 11m is only the road to connect Betio island where the international port exists and Bairiki island where the headquarters of administrative agencies and residential area exist. It is indispensable in sustaining the life of citizens and economic activities.
- The causeway mentioned above has been eroded and corrupted due to aging and caused by natural disasters like high tide water. Therefore, the repair and strengthening works for the entire section is an urgent issue to be addressed.

2.1.1.2 Development Plan and Background

- The rehabilitation of the aged road is described in the Kiribati Development Plan (2012-2015) as a priority item in the establishment of infrastructures.
- In order to improve such situations, the Government of Kiribati (GOK) requested the Government of Japan (GOJ) for a grant aid to undertake the "The Project for Reconstruction of Nippon Causeway on Tarawa to adapt climate change".

2.1.1.3 Precedent Surveys and Other Donors' Activities

- The Project for the Improvement of Fisheries Transportation in South Tarawa in the Republic of Kiribati was implemented in 2007 by JICA. The South Tarawa road was partially rehabilitated at Betio, Bairiki and Bikenibeu, which was completed in 2008.
- In 2012-2016, the Kiribati Road Rehabilitation Project amounting to 38 million US dollars funded by ADB (Asian Development Bank, IDA (International Development Agency) and Government of Australia was implemented.

2.1.2 Scope of the Request

- Reconstruction of Nippon causeway structure between Betio and Bairiki (approx.3.2 km)
- Widening of Bridge Section

2.1.3 Objectives of the Survey

Objectives of the Survey are to:

- Understand the background, purpose and scope of the grant aid project,
- Study the feasibility of the project in terms of effectiveness, human, technology and economic justification,
- Conduct the outline design for the minimum but optimal scope and size of the project required in achieving the outcomes of the cooperation,
- Estimate the project cost, and
- Propose the contents, implementation and maintenance plan as well as critical points to be undertaken by the GOK in order to achieve the outcome and targets set for the project.

2.1.4 Goal and Achievement of the project

- Goal: To secure smooth and stable traffic condition between Betio and Bairiki.
- Achievement：To secure smooth and safe traffic condition of the Nippon Causeway.

2.1.5 Environment and social considerations

(1) Status of environmental license acquisition

In accordance to the Environment (Amendment) Act 2007, any project that potentially involves environmentally-significant activities must obtain an “Environmental License” from the Ministry of Environment Lands and Agricultural Development (MELAD). The type of activities that require “Environmental License” are prescribed under the Environment (Amendment) Act 2007, where causeway is included under the public works sector. Upon reviewing the application document for the Environment License, MELAD duly requested MPWU to submit a Basic Environmental Impact Assessment (BEIA) report for approval.

The JICA expert team prepared the draft BEIA document and submitted to MPWU in December 2015. MPWU then submitted the final BEIA to MELAD in January 2016. The Environmental License is expected to be obtained by the end of March 2016.

(2) Existing environment around the Causeway

1） Pollution

The JICA expert team conducted a water and sediment quality surveys around the Causeway. Although no significant water pollution was identified, the lagoon side had a higher levels of turbidity, T-N and T-P compared to the ocean side. This is understandable as there is less water exchange in the lagoon side. No signs of sediment pollution were detected.

Due to the bad road condition, the dust and noise emissions from the passing vehicles are an ongoing issue. The environmental condition around the Causeway is expected to improve through the Project by resurfacing the roadway.

2) Natural environment

The JICA expert team conducted marine and terrestrial ecosystem surveys around the Causeway. No sensitive marine habitats (e.g. corals) or endangered marine species were found in the vicinity of the Causeway. Although two IUCN threatened species (one coral specie and one fish specie) were found in the reef slope area, these species are likely to be unaffected by the Project as the reef slope area is located far from the Causeway (around 400-500 m).

A total of 17 terrestrial vegetation species were recorded consisting of trees, shrubs, herbaceous plant and vines. None of the identified species are classified as threatened under the IUCN Red List.

3) Social environment

The Causeway and the sandy beach that was naturally formed after the Causeway construction are owned by the state. No new land acquisition will be required under this project as reconstruction works will be done within the boundary of the existing Causeway.

The lands adjacent to the Causeway landing area in Bairiki and Betio are designated as a commercial and an open space area respectively. The reconstruction works will not cause any alteration to the current land use plan.

The shallow reef flat area adjacent to the Causeway is used by the locals for fishing and bathing. A boat channel also exists at the bridge section. Although, there will be temporary water use restrictions around the construction area, impacts of such restrictions are likely to be minor as the restrictions will be limited in area and duration.

(3) Potential environmental impacts and mitigation measures

1) Construction phase

The Causeway reconstruction works are expected not to cause any significant negative environmental impacts, primarily because the project does not entail any loss of natural environment, change in land use and resettlement.

One of the main environmental concerns during the construction phase is the potential pollution that may arise from the operation of the asphalt and concrete plants and waste generation. The following are the main mitigation measures planned for the asphalt and concrete plants and waste generation.

- The main concern for the asphalt plant is the dust generated from the aggregate drying process. Dust emission from this process is planned to be reduced significantly by installing primary and secondary dust collection units.

- The main concern for the concrete plant is the concrete wash water generated from agitator washout and charging areas, slumping station and so on. The wash water will include concrete materials (e.g. cement, sand, aggregates) and will be highly alkaline. The plant will be designed so that all wash water (including contaminated storm water) are retained on site by collecting and diverting the wash water to an impermeable settling pond, and reusing the captured wash water. Discharge of wash water will only be allowed provided that pH and suspended solid levels are within the World Bank discharge standard (pH: 6-9, suspended solids: < 50 mg/l).
- Waste management is a key issue, especially since South Tarawa has limited landfill capacity and has no facility to receive hazardous wastes. Waste volume will be minimized by promoting 3R (reduce, reuse and recycle).

While impacts from the asphalt and concrete plants are planned to be avoided or minimized by implementing strict pollution control measures, it is also important that these facilities are to be located as far as possible from sensitive areas (e.g. residential areas).

2） Post-construction phase

In accordance to the scoping process, no negative impacts were identified for the post-construction phase. In fact, due to the better road condition, the local environment is expected to improve significantly as there will be less dust and noise emitted from the passing vehicles.

3） Environmental management plan and monitoring plan

Based on the environmental assessment, an Environmental Management Plan (EMP) and monitoring plan were prepared to ensure that reconstruction works are implemented with minimal environmental impacts. The EMP summarizes the planned mitigation measures against the anticipated environmental impacts, the responsibility for its implementation and supervision, and estimated cost. The mitigation and monitoring costs will be included in the project budget to ensure its implementation.

However, since the construction plan and methods, including the location, layout and specifications of the asphalt and concrete plants, are to be determined at a later stage by the construction contractor. The EMP and monitoring plan should be considered as a tentative document and be revised/finalized at later appropriate stage through consultation with MELAD and local stakeholders, and obtain approval from MELAD accordingly.

4） Stakeholder meeting

A public consultation meeting was held on September 11th, 2015 at KNYC Maneaaba. The purpose of the meeting was to inform the stakeholders and public about the planned reconstruction works of the Causeway and the scoping of environmental impacts. The stakeholders and public were invited by sending invitation letters and through public radio announcement.

Around 20 people participated in the meeting including local residents, relevant government agencies and Australian and New Zealand Commission. None of the participants raised any objections towards the project's plan and design as their concerns were answered. The minutes of the meeting is attached to the draft BEIA report.

5) Conclusion and recommendation

As mentioned previously, no significant negative environmental impacts are expected to occur through the Causeway reconstruction works, provided that the EMP and monitoring plan are appropriately implemented. The following are recommendations or actions necessary for the ensuring project stages:

- The asphalt and concrete plants should be located as far as possible from the sensitive areas (e.g. residential areas).
- The EMP and monitoring plan should be revised/finalized in the process of finalizing the construction plan through consultation with local stakeholders and obtain approval from MELAD.
- To ensure that the EMP and monitoring is effectively implemented, a qualified and experienced environmental officer should be assigned to the contractors team and supervising consultant.

2.2 Outline Design of the Japanese Assistance

2.2.1 Design Policy

2.2.1.1 Site Condition

(1) Embankment Section

1） Causeway Damage Condition

In order to identify the damage of the causeway (Embankment section), inventory survey was conducted at 20m internal. The degree of the causeway's damage is classified as either large, medium or small. (see Table 2.2.1-1) The inventory result of total length for damage degree is shown in Table 2.2.1-2 and Table 2.2.1-3.

It concludes that the seawall at the ocean side had serious damages, especially the section between bridge and Bairiki.

Table 2.2.1-1 Classification of Seawall Damage

	Large	Medium	Small
Photo			
Damage Degree	➢ There are repeated repair marks and serious damages are seen in the whole section. ➢ There are broken fabrimats at the foundation. ➢ There are many cracks for the whole section and a big cross-section deformation. ➢ A big cavity is also seen at the foundation	➢ Though there are some cracks, it can be repairable and small cross-section deformation. ➢ There are partially-repair marks in the fabrimat. ➢ There are small cross-section deformation.	➢ Though there are partial cracks, seawall is almost the same condition as the beginning. ➢ There are non-repair marks in the fabrimat. ➢ There are little cross-section deformation.

Table 2.2.1-2 Lengths of Seawall Damage Degree

Unit：(m)

	Degree of damage	Large	Medium	Small	Almost no damage or beach	Total
Seawall (Left) (Lagoon side)	Betio ～ Bridge	86	380	320	618	1,404
	Bridge ～ Bairiki	0	160	613	1,053	1,826
	Lagoon side Total	**86**	**540**	**933**	**1,671**	**3,230**
Seawall (Right) (Ocean Side)	Betio ～ Bridge	220	459	0	479	1,158
	Bridge ～ Bairiki	869	863	340	0	2,072
	Ocean Side Total	**1,089**	**1,322**	**340**	**479**	**3,230**

Table 2.2.1-3 Classification of Seawall Damage Degree

(Section 1 – station columns)

Temporary Station No.	Station No.	Accumulated Distance (m)
319-16	BP	0
319	BP+16	16
319+9	BP+25	25
318	BP+26	26
317	BP+36	35
316	BP+46	45
315	1+6	56
314	1+16	66
313	1+26	76
312	1+36	86
311	1+46	96
310	2+6	106
309	2+16	116
308	2+26	126
307	2+36	136
306	2+46	146
305	3+6	156
304	3+16	166
303	3+26	176
302	3+36	186
301	3+46	196
300	4+6	206
299	4+16	216
298	4+26	226
297	4+36	236
296	4+46	246
295	5+6	256
294	5+16	266
293	5+26	276
292	5+36	286
291	5+46	296
290	6+6	306
289	6+16	316
288	6+26	326
287	6+36	336
286	6+46	346
285	7+6	356
284	7+16	366
283	7+26	376
282	7+36	386
281	7+46	396
280	8+6	406
279	8+16	416
278	8+26	426
278+2	8+28	428
277	8+36	436
276	8+46	446
275	9+6	456
274	9+16	466
273	9+26	476
272	9+36	486
271	9+46	496
270	10+6	506
269	10+16	516
268	10+26	526
267	10+36	536
266	10+46	546
265	11+6	556
264	11+16	566
263	11+26	576
262	11+36	586
261	11+45	595
260	12+5	605
259	12+15	615
258	12+25	625
257	12+35	635
256	12+45	645
255	13+5	655
254	13+15	665
253	13+25	675
252	13+35	685
251	13+45	695
250	14+5	705
249	14+15	715
248	14+25	725
247	14+35	735
246	14+45	745
245	15+5	755
244	15+15	765
243	15+25	775
242	15+35	785
241	15+45	795
240	16+5	805
239	16+15	715
238	16+25	825
237	16+35	835
236	16+45	845
235	17+5	855
234	17+15	865
233	17+25	875
232	17+35	885
231	17+45	895
230	18+5	905
229	18+15	915
228	18+25	925
227	18+35	935
226	18+45	945
225	19+5	955
224	19+15	965
223	19+25	975
222	19+35	985
221	19+45	995
220	20+5	1005
219	20+15	1015
218	20+25	1025
217	20+35	1035
216	20+45	1045
215	21+5	1055
214	21+15	1065
213	21+25	1075
212	21+35	1085
211	21+45	1095
210	22+5	1105
209	22+15	1115

(Section 1 – classification cells, left to right)

Revetment (Lagoon Side) Riprap Damage	Revetment (Lagoon Side) Parapet Wall Condition	Road Surface Cross Drainage	Road Surface Water Compaction	Road Surface Damage	Road Surface Condition	Road Surface Water Compaction	Road Surface Cross Drainage	Revetment (Ocean Side) Parapet Wall Condition	Revetment (Ocean Side) Riprap Damage
G	E			B	8			–	G
G				G				–	G
G				G				–	G
G				G				–	G
G				G				–	G
G				G				E	G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				G					G
G				B	There are a lot of exfoliations and cracks as a whole.				G
G				B					F
G		B		B					F
G		B		B					F
G		B		B					P
G		B		B					P
G		B		B					P
G		B		B					P
F				B					F
F				B					F
F				B					B
F				B					P
F				B					P
F				B					P
F				B					P
F				B					P
F				B					P
F				B					P
F				B					F
F				B					F
F				B					F
F				B					P
F				B					P
F				B					P
P				B					F
P				B					F
P				B					B
P				B					B
P				B					P
P				B					P
P				B					P
P				B					B
P				B					B
P				B					B

(Section 2 – station columns)

Temporary Station No.	Station No.	Accumulated Distance (m)
209	22+15	1115
208	22+25	1125
207	22+35	1135
206	22+45	1145
205	23+5	1155
204	23+15	1165
203	23+25	1175
202	23+35	1185
201	23+45	1195
200	24+5	1205
199	24+15	1215
198	24+25	1225
197	24+35	1235
196	24+45	1245
195	25+5	1255
194	25+15	1265
193	25+25	1275
192	25+35	1285
191	25+45	1295
190	26+5	1305
189	26+15	1315
188	26+25	1325
187	26+35	1335
186	26+45	1345
185	27+5	1355
184	27+15	1365
183	27+25	1375
183+5	27+30	1381
182	27+35	1385
181	27+45	1395
181+9	28+4	1404
180	28+5	1405
179	28+15	1415
178	28+25	1425
177	28+35	1435
176	28+45	1445
175	28+55	1455
174	29+15	1465
173	29+25	1475
172	29+35	1485
171	29+45	1495
170	29+55	1505
169	30+15	1515
168	30+25	1526
167	30+36	1536
166	30+46	1546
165	30+56	1556
164	31+16	1566
163	31+26	1576
162	31+36	1586
161	31+46	1596
160	31+56	1606
159	32+16	1616
158	32+26	1626
157	32+36	1636
156	32+46	1646
155	32+56	1656
154	33+16	1666
153	33+26	1676
152	33+36	1686
151	33+46	1696
150	33+56	1706
149	34+16	1716
148	34+26	1726
147	34+36	1736
146	34+46	1746
145	34+56	1756
144	35+16	1766
143	35+26	1776
142	35+36	1786
141	35+46	1796
140	35+56	1806
139	36+16	1816
138	36+26	1826
137	36+36	1836
136	36+46	1846
135	36+56	1856
134	37+16	1866
133	37+26	1876
132	37+36	1886
131	37+46	1896
130	37+57	1907
129	38+17	1917
128	38+27	1927
127	38+37	1937
126	38+47	1947
125	38+57	1957
124	39+17	1967
123	39+27	1977
122	39+37	1987
121	39+47	1997
120	40+7	2007
119	40+17	2017
118	40+27	2027
117	40+37	2037
116	40+47	2047
115	41+7	2057
114	41+17	2067
113	41+27	2077
112	41+37	2087
111	41+47	2097
110	42+7	2107
109	42+17	2117
108	42+27	2127
107	42+37	2137
106	42+47	2147
105	43+7	2157
104	43+17	2167
103	43+27	2177
102	43+37	2187
101	43+47	2197
100	44+7	2207
99	44+17	2217

(Section 2 – classification cells, left to right)

Revetment (Lagoon Side) Riprap Damage	Revetment (Lagoon Side) Parapet Wall Condition	Road Surface Cross Drainage	Road Surface Water Compaction	Road Surface Damage	Road Surface Condition	Road Surface Water Compaction	Road Surface Cross Drainage	Revetment (Ocean Side) Parapet Wall Condition	Revetment (Ocean Side) Riprap Damage
P				B	There are a lot of exfoliations and cracks as a whole.				B
B				B					B
B				B					B
B				B					B
P				B					B
P				B					P
P				B					P
P				B					P
P				B					P
P				B					P
P				B					P
P				B					B
B				B					B
Bridge				B	3	Bridge			
F				B	5				P
F				B	10				P
F				B	12				P
F				B	14				P
F				B	13				P
F				B	7				P
F				B	15	R	B	R	F
F				B	11				F
F				B	12				F
F				B	17				F
P				B	10				F
P				B	10				F
P				B	16				F
P				B	13			R	P
P				B	14			R	P
P				B	13				P
P				B	15				P
F				B	19				P
F				B	21			R	P
F				B	11			R	B
F				B	9				P
F				B	16				P
F				B	14				P
F				B	8				P
F				B	16				P
F				B	8				B
F				B	15				B
F				B	22	R	B	R	B
F				B	19	R	B	R	B
P				B	15	R	B	R	B
F				B	17	R	B	R	B
F				B	15	R	B	R	B
F				B	13	R	B	R	B
F				B	12	R	B	R	B
F				B	13	R	B	R	B
F				B	16				P
F				B	14				P
F				B	15				P
F				B	17				P
G				B	19				P
G		B		B	18				P

(Section 3 – station columns)

Temporary Station No.	Station No.	Accumulated Distance (m)
99	44+17	2217
98	44+27	2227
97	44+37	2237
96	44+47	2247
95	45+7	2257
94	45+17	2267
93	45+28	2278
92	45+38	2288
91	45+48	2298
90	46+8	2308
89	46+18	2318
88	46+28	2328
87	46+38	2338
86	46+48	2348
85	47+8	2358
84	47+18	2368
83	47+28	2378
82	47+38	2388
81	47+48	2398
80	48+8	2408
79	48+18	2418
78	48+28	2428
77	48+38	2438
76	48+48	2448
75	49+8	2458
74	49+18	2468
73	49+28	2478
72	49+38	2488
71	49+48	2498
70	50+8	2508
69	50+18	2518
68	50+28	2528
67	50+38	2538
66	50+48	2548
65	51+8	2558
64	51+18	2568
63	51+28	2578
62	51+38	2588
61	51+48	2598
60	52+8	2608
59	52+18	2618
58	52+28	2628
57	52+38	2638
56	52+49	2649
55	53+9	2659
54	53+19	2669
53	53+29	2679
52	53+39	2689
51	53+49	2699
50	54+9	2709
49	54+19	2719
48	54+29	2729
47	54+39	2739
46	54+49	2749
45	55+9	2759
44	55+19	2769
43	55+29	2779
42	55+39	2789
41	55+49	2799
40	56+9	2809
39	56+19	2819
38	56+29	2829
37	56+39	2839
36	56+49	2849
35	57+9	2859
34	57+19	2869
33	57+29	2879
32	57+39	2889
31	57+49	2899
30	58+9	2909
29	58+19	2919
28	58+29	2929
27	58+39	2939
26	58+49	2949
25	59+9	2959
24	59+19	2969
23	59+29	2979
22	59+39	2989
21	59+49	2999
20	60+9	3009
19	60+20	3020
18	60+30	3030
17	60+40	3040
16	60+50	3050
15	61+10	3060
14	61+20	3070
13	61+30	3080
12	61+40	3090
11	61+50	3100
10	62+10	3110
9	62+20	3120
8	62+30	3130
7	62+40	3140
6	62+50	3150
5	63+10	3160
4	63+20	3170
3	63+30	3180
2	63	3190
1	64	3200
EP	EP	3230

(Section 3 – classification cells, left to right)

Revetment (Lagoon Side) Riprap Damage	Revetment (Lagoon Side) Parapet Wall Condition	Road Surface Cross Drainage	Road Surface Water Compaction	Road Surface Damage	Road Surface Condition	Road Surface Water Compaction	Road Surface Cross Drainage	Revetment (Ocean Side) Parapet Wall Condition	Revetment (Ocean Side) Riprap Damage
G		B		B	22				P
G		B		B	22			R	B
G		B		B	12			R	B
G		B		B	11			R	B
G		B		B	13				B
G		B		B	15				B
G		B		B	19	R		R	B
G		B		P	7	R		R	B
G		B		P	4	R		R	B
G		B		P	7	R		R	B
G		B		P	16				P
G		B		F	17	R			P
G		B		F	17				P
G		B		F	19	R		R	P
G		B		F	17			R	B
G		B		F	18				B
G		B		F	17				B
G		B		P	17				B
G		B		P	12				B
G		B		P	12				B
G		B		P	21				B
G		B		P	18				B
G		B		P	12				P
G				P	17				P
G				P	9				P
G				P	12				P
G				P	19				B
G				P	20				B
G				P	27	R			B
G				P	13				B
G				P	12				B
G				P	14		B	R	B
G				P	17	R	B	R	B
G				B	13	R	B	R	B
G				B	16	R	B	R	B
G				P	7	R	B	R	B
G				P	5	R			B
G				P	6	R		R	B
G				P	5			R	B
G				B	7				P
G				B	15				P
G				B	21				P
G				B	12				P
G				B	7				P
G				B	12				P
G				B	11				P
G				P	6				P
G				P	6				P
G				P	4				P
G				P	8				P

Legend

Riprap – Damage

G	Good (Beach)
F	Fair
P	Poor
B	Bad

Parapet Wall

E	Wall edge
R	To be repaired
–	It is not installed

Cross Drainage

B	Drainage is clogged

Water Compaction

R	To be acted

Surface-Damage/Condition

G	Good
F	Fair
P	Poor
B	Bad
Figures	Number of potholes
Note	Conditions

2) Possible Damage Mechanism and Damage Types of the Revetment

Identification of the possible damage mechanism including damage causes is essential in alternative study on countermeasures for the revetment improvement.

a) Trigger of Damage Development

Cracks on the fabriform mats are damaged commonly and widely observed on the revetment facing both lagoon and ocean sides. These cracks are supposed to have triggered the proceeding damages. These damages have developed being affected according to located sections and/or whether facing lagoon side or ocean side.

These cracks were at first caused by bending moment acting on the top of the slope due to dead weights of the shoulder part after the embankment consisting of sand was softened or settled by impacts from passing vehicles and waves (Photo 2.2.1-1). Then cracks at the maximum bending moment location, which is the top of sandbags inside the embankment, occurred due to vibrations by waves and deformation of the fabriform mats toward the inside of the embankment.

The settlement of the fabriform mats along the slope is different in the locations, which cased shear forces and developed shear cracks (Photo 2.2.1-2).

The first cracks including bending and shear cracks occurred due to the softness and/or small voids of the sand embankment. This phenomenon is commonly observed along the entire causeway not related to locations. This means that differences of damage scales actually observed along the revetment are to be largely dependent on those of the external forces acting on the revetment.

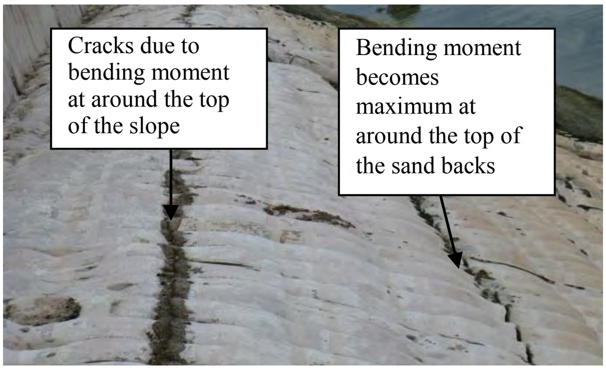


Photo 2.2.1-1 Typical Crack due to Bending Moment

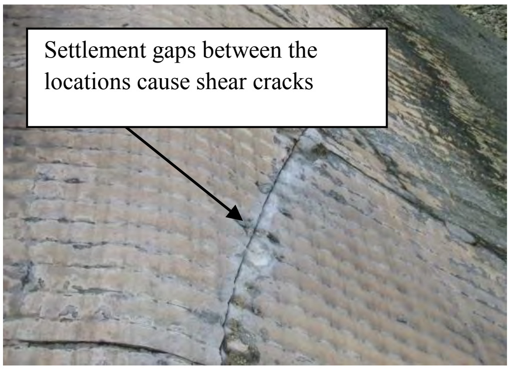
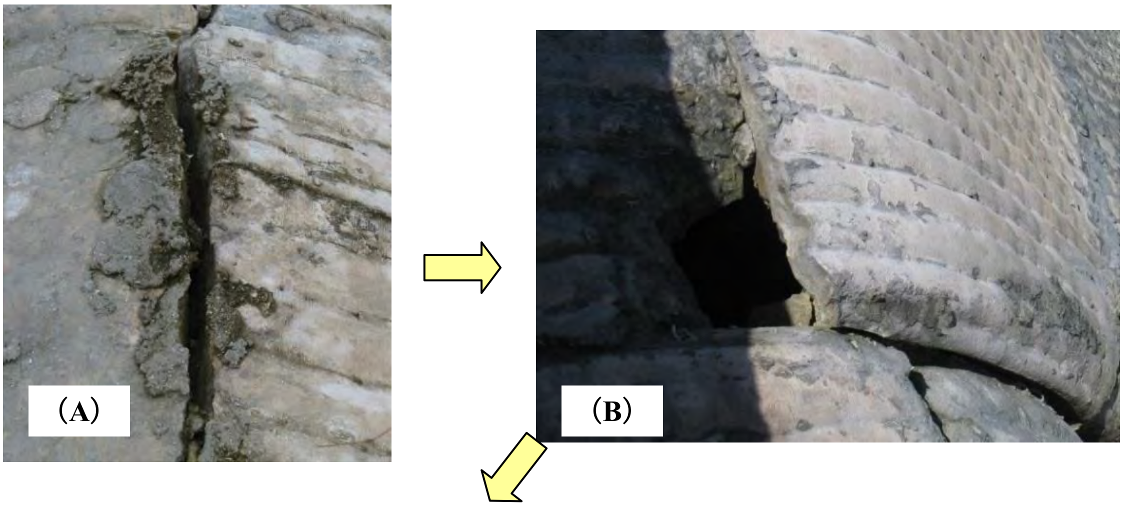


Photo 2.2.1-2 Typical Shear Crack

b) Causes of Developing and Expanding Cracks and Collapse

The occurrence of cracks is commonly observed along the entire revetment as mentioned above. Along the section facing the ocean side, however, between the existing bridge and Bairiki area, repeated attacks of large waves such as tiger tides and billows sucked sands inside the bank out and expanded those cracks, and finally parts of the revetment collapsed. Leaking of water from the pipe under the ground is considered to be one of reasons of sands coming out. The influence of wave motions on the lagoon side is to be much smaller than that of the ocean side because crack development and expansion on the lagoon side are much smaller than those on the ocean side.

It is commonly observed that damages with a hole shown in Photo 2.2.1-3 (B) developed and expanded quickly due to repeated attacks of large waves. Coming out of sands constructing the revetment is to be the cause of damage after development of cracks.



(A) : Expansion of Cracks (Expansion of cracks made by repeated attacks of large waves and washing away or coming out of sand. Leaking water is not directly related to the expansion of cracks.)

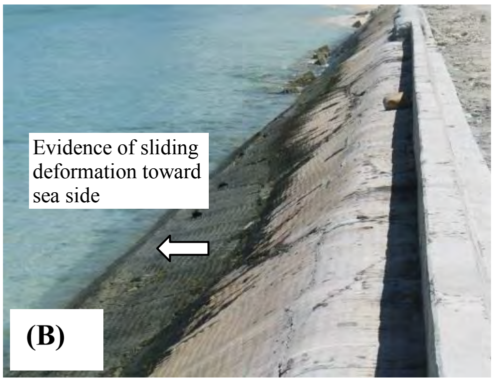
(B) : Development of holes from cracks (the development of holes is not only due to repeated attacks of large waves but also due to leaking water from the water supply pipe.)

(C) : Collapse (Collapse by slope sliding failure of the revetment due to further crack expansion and damage development.)

Photo 2.2.1-3 Collapse from Crack Expansion

c) Collapse Types of Revetment

Collapse types of the revetment are to be classified into two types, which are collapse due to crack expansion and collapse due to lack of bearing capacity of foundation, as shown in Photo 2.2.1-4. Collapse due to crack expansion is caused by coming out and washed away of sand inside of the bank, on the other hand collapse due to lack of bearing capacity of foundation is caused by the revetment foundation not firmly being placed in the proper ground.



(A)：Collapse due to Crack Expansion , **(B)**：Collapse due to Lack of Bearing Capacity of Foundation

Photo 2.2.1-4 Collapse Types of Revetment

d) Lessons for Alternative Study on Revetment Improvement Measures

The following four (4) points are to be lessons learned from the first site survey for alternative study on the revetment improvement measures.

(a) To protect sand from being washed away from the embankment,

(b) To place revetment foundation into the firm ground in order to secure the proper foundation capacity,

(c) To upgrade fabriform mats (increasing the thickness of the fabriform mat or strengthening the fabriform mat concrete), and

(d) To accommodate utilities such as water supply pipe, telecommunication and electric power cables on the outside of the causeway.

3) Bridge Section

The visual inspection, dimensional measurement survey, crack measurement, concrete strength test and hearing investigation are conducted to identify the damage of existing bridge. The major damage and deterioration of existing bridge is shown in Table 2.2.1-4.

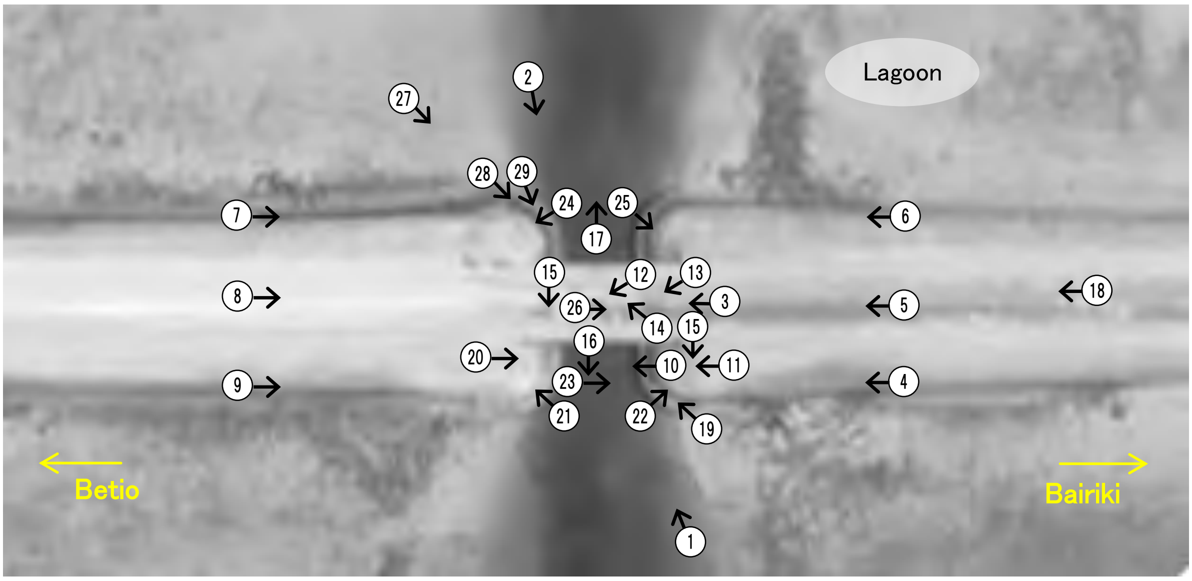
Table 2.2.1-4 ～Table 2.2.1-5 (1/3)-(3/3). As a result of the visual inspection of the existing bridge, the fatal damage such as the cause of corruption was not found meanwhile some minor damage or deterioration such as the deterioration on the road surface was found. Therefore, the repair work or the partial retrofitting of the existing bridge is recommendable. The design water level including the vertical navigational clearance will be studied as a design condition.

Table 2.2.1-4 Methodology and Major Result of the Evaluation of Bridge Soundness

	Item	Inspection Method	Inspection Location	Rating of Damages	Major Result
Structural Soundness	Pavement	- Visual Inspection	Road Surface on the Bridge and on the Road	Severe	- The removal of the existing pavement was remarkably confirmed at the surface of the bridge and the road. - The reinforced soil by cement on the road shoulder was replaced during installation of utilities. The some portion was depressed due to washing out of sand at the embankment.
	Load Capacity	- Visual Inspection (Crack, Free Line, rust fluid)	Soffit of Slab	None	- The major crack was not confirmed at the soffit of slab. The concrete slab has enough capacity for the live load.
	Concrete	- Measurement of shape - Visual Inspection - Measurement of Crack - Concrete Strength Test by Concrete Hammer	Structural Concrete	Minor	- The crack and rust fluid were confirmed at the concrete wall and slab edge. - The internal reinforcements seems rusted. - The compressive strength of the existing concrete was estimated to be around 30 to 40 kN/mm^2 and maintain the design compressive strength of 20.5 kN/mm^2.
	Foundation/ Stability of Bridge and Embankment	- Visual Inspection - Measurement of the Deformation	Steel Straight Sheet Pile Foundation/ Retaining Wall/ Riverbed	Minor	- The deformation of the bridge foundation is not confirmed. The Steel Straight Sheet Pile of bridge foundation is not rusted and deformed. However the covering concrete of the top of sheet piles are totally replaced by the wave. - The retaining wall (Fabric Mattress) at Lagoon side was damaged due to the vortex flow by the water head difference. - The riverbed protection still remains and works properly. The partial scouring is not confirmed.
	Ancillary Items and etc.	- Visual Inspection	Hand rail/ Utilities/ Waterway and etc.	Minor	- The depression at road is not confirmed. - The sedimentation at waterway is affecting the navigational operation. - Bridge newel post at Betio-Ocean side is corrupted. - The lighting on the bridge is not operational. Only the electric pale and foundation remained. - Concrete handrail on the bridge is sound and properly working. - There is no expansion joint and bearing used for the box culvert.
Functional Soundness	Vehicle Operation (Cross Section)	- Measurement of shape - Visual Inspection	Road Surface	Minor	- The width of carriageway is the same as road section of 3.0 m x 2. - The width of shoulder at the bridge section is reduced to 1.6 m instead of 2.5 m of road section. 1.1 m of 1.6 m of shoulder width is used as mount up pedestrian way. The utility cables were installed into the mount up pedestrian way. - The reduction of shoulder and mount up pedestrian way may cause bottleneck of the main traffic (Actual shoulder width: 0.5 m).
	Navigational Operation (Waterway)	- Visual Inspection - Interview Survey	Waterway	None (Under Survey on Navigational Clearance)	- A few vessels passed under the bridge (several vessels per hour). The types of vessels operated in this area are mostly small fisher boat. - The vertical clearance is properly maintained and the vessel could be operated smoothly in the ordinary conditions. However, the design conditions for the navigational clearance shall be studied with consideration of the rising of the sea level. - The sedimentation at waterway may affect the navigation operation during low tide.
	Socio-environmental Conditions	- Visual Inspection - Interview Survey	Lagoon	None	- The significant change of environmental condition was not confirmed after construction of the causeway.

Table 2.2.1-5 Photo Record of Existing Bridge (1/3)

Plan View and Layout of the Photo No.



		
[Photo-1] View of bridge from ocean side	[Photo-2] View of the bridge from lagoon side	[Photo-3] View of bridge surface (Mount up pedestrian way and reduced shoulder)
		
[Photo-4] Ocean side shoulder from Bairiki side	[Photo-5] Road center from Bairiki side	[Photo-6] Lagoon side shoulder from Bairiki side
		
[Photo-7] Lagoon side shoulder from Betio side	[Photo-8] Road center from Betio side	[Photo-9] Ocean side shoulder from Betio side

Table 2.2.1-6 Photo Record of Existing Bridge (2/3)

<table>
<tr><th colspan="3">Layout of the Photo No.</th></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>[Photo-10] Attached water pipe at ocean side of bridge (There is no water leaking and the condition is sounded)</td><td>[Photo-11] The water bulb at Bairiki side.</td><td>[Photo-12] The lighting pole not operated.</td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>[Photo-13] Handrail at ocean side (There is no damage)</td><td>[Photo-14] Handrail at lagoon side (There is no damage)</td><td>[Photo-15] The boundary of box culvert and road (There is no expansion joint)</td></tr>
<tr><td colspan="3"></td></tr>
<tr><td colspan="3">[Photo-16] Waterway at ocean side
(The opening is protected by concrete blocks. The navigational operation is difficult during low tide)</td></tr>
<tr><td colspan="3"></td></tr>
<tr><td colspan="3">[Photo-17] Waterway at ocean side
(The opening of only Bairiki side is protected by concrete blocks. The navigational operation is difficult during low tide)</td></tr>
</table>

Table 2.2.1-7 Photo Record of Existing Bridge (3/3)

Layout of the Photo No.		
		
[Photo-18] Road surface condition (The pavement had washed out)	[Photo-19] Bridge surface condition (The pavement had washed out)	[Photo-20] Bridge newel post at Betio-Ocean side is corrupted.
		
[Photo-21] Damage of concrete wall with rust fluid and spalling (Betio-ocean side)	[Photo-22] Damage of concrete wall with rust fluid and spalling (Bairiki-ocean side)	[Photo-23] Damage of concrete slab with rust fluid and spalling (Ocean side)
		
[Photo-24] Damage of concrete wall with rust fluid and spalling (Betio-lagoon side)	[Photo-25] Damage of concrete wall with rust fluid and spalling (Bairiki-lagoon side)	[Photo-26] Damage and deterioration on the bridge slab
		
[Photo-27] Damage of road retaining wall (Fabric mattress) and repair work (Betio-lagoon side)	[Photo-28] Damage of Steel Straight Sheet Piles (The concrete cover has washed off)	[Photo-29] Steel Sheet Pile (There is no damage and deterioration)

2.2.1.2 Concept of Improvement for the Project

The Comparison of reinforcement of existing revetment (Alternative-1) and construction of the bridge (Alternative-2 and Alternative-3) are compared as viewpoint of drivability, durability, workability, economy, maintenance and social environmental consideration. In the alternative-2 and alternative-3, the bridge length is about 3.0km. The results of comparison are shown in Table 2.2.1-8Table 2.2.1-8-Table 2.2.1-10. As a result of the comparison, Alternative-1 is recommended. Alternative-1 has more advantages in terms of workability, economy and social environmental consideration.

Alternative-1 ： Strengthening of the Existing Revetment (Embankment Structure)
Alternative-2 ： Construction of the Bridge (PC Bridge)
Alternative-3 ： Construction of the Bridge (Slab Bridge on Pile Bent)

Table 2.2.1-8 Alternative of Improvement Measures (1)

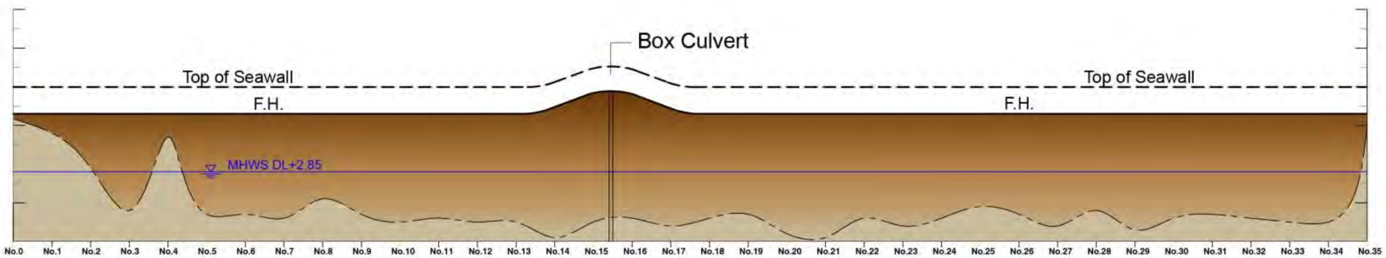
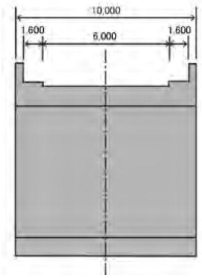
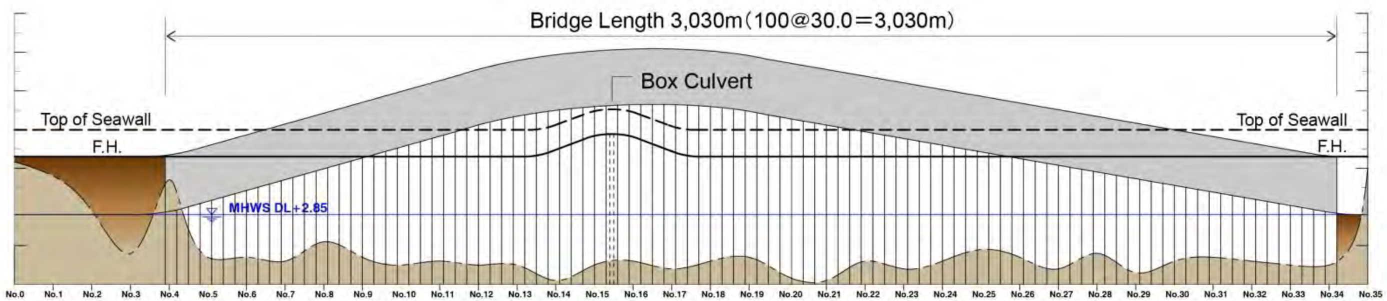
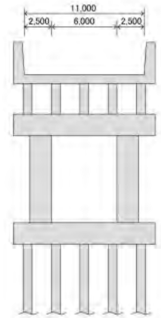
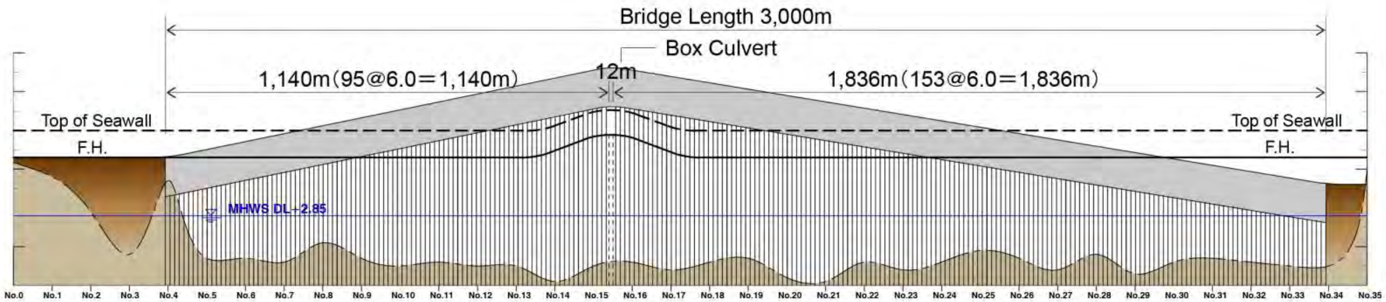
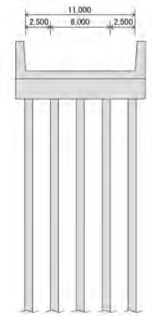
Alternatives	[Alternative-1] Strengthening of Existing Revetment		[Alternative-2] Construction of the Bridge (PC Bridge)		[Alternative-3] Construction of the Bridge (Slab Bridge on Pile Bent)	
Abstract	Existing revetments are strengthened in response to the damage condition.		PC bridge of about 3.0km is constructed at the lagoon side parallel to the Nippon Causeway.		Slab bridge on pile bent of about 3.0km is constructed to lagoon side in parallel to Nippon Causeway.	
Drivability	Drivability is good. ➢ Horizontal Alignment: R=1500 ~ ∞(≧150) ➢ Profile: I = Level ~ 2.0% (≦5.0%)	○	Drivability is good. ➢ Horizontal Alignment: R=300 ~ ∞(≧150) ➢ Profile: I = Level ~ 0.5% (≦5.0%)	○	Drivability is good. ➢ Horizontal Alignment: R=300 ~ ∞(≧150) ➢ Profile: I = Level ~ 0.5% (≦5.0%)	○
Navigation	➢ Sands have been deposited around the channel easily, and periodic maintenance for the channel such as the dredging is required. ➢ Vertical clearance of the bridge cannot be ensured due to reviewing of the design tide level.	△	➢ The majority of the Causeway are changed to bridge structure, deposition of sands around the channel are improved. ➢ Vertical clearance of the bridge can be ensured.	○	➢ The majority of the Causeway are changed to bridge structure, deposition of sand around the channel are improved. ➢ Vertical clearance of the bridge can be ensured.	○
Durability	Design Period: more than 30 years* (* Depend on strengthening specification of the revetment)	△	Design Period: more than 50 years*	○	Design Period: more than 50 years*	○
Strength for the Wave	➢ Strength of the seawall is required to withstand the wave force.	○	➢ Road profile is higher than alternative-1, and effect of ocean waves is lesser.	◎	➢ Road profile is higher than the alternative-1, and effect of ocean waves is lesser.	◎
Workability	➢ Temporary traffic regulation of one lane is required. ➢ Restrictions during construction are lesser than the other alternatives.	○	➢ Temporary traffic regulation of one lane is required. ➢ In the construction of the connecting section between the existing road and bridge, temporary bypass is required.	△	➢ Temporary traffic regulation of one lane is required. ➢ In the construction of the connecting section between the existing road and bridge, temporary bypass is required.	△
Economy	➢ Concrete volume is smaller than the other alternatives. ➢ Existing revetments are strengthened in response to the damage conditions, it is possible to reasonable measures compared with other alternatives.	○	➢ Concrete volume is bigger than alternative-1, and the construction cost is very high. (The construction of alternative-2 is 9.0 times the alternative-1.)	×	➢ Concrete volume is bigger than alternative-1, and the construction cost is very high. (The construction of alternative-2 is nine 6.5 times the alternative-1.)	×
Ratio	1.0 (base)		9.0		6.5	
Maintenance	➢ Maintenance of the revetment and the road pavement and dredging of the channel are required. ➢ These maintenances can be implemented by local technology.	○	➢ Frequency of maintenance for the structures is lesser than alternative-1, but bridge maintenance and inspection cannot be implemented by local technology.	△	➢ Frequency of maintenance for the structures is lesser than alternative-1, but bridge maintenance and inspection cannot be implemented by local technology.	△
Social Environmental Consideration	➢ Natural environments are not change by this construction. ➢ Land acquisition is not required.	◎	➢ As the most of causeway section become bridge structure, the flow of sea water will change and the sand deposition along Nippon Causeway will disappear. ➢ Land acquisition is required to construct a new bridge.	△	➢ As the most of causeway section become bridge structure, the flow of sea water will change and the sand deposition along Nippon Causeway will disappear. ➢ Land acquisition is required to construct a new bridge.	△
Total Evaluation	◎ **Workability, economy and social environment are better than the other alternatives.**		△		△	

Legend : ◎ Excellent ○ Better △Good × Undesirable

Table 2.2.1-9 Alternative of Improvement Measures (2)

<table>
<tr><th>Alternatives</th><th>Plan</th></tr>
<tr><td>[Alternative-1]
Strengthening of Existing Revetment</td><td>
</td></tr>
<tr><td>[Alternative-2]
Construction of the Bridge
(PC Bridge)</td><td rowspan="2">
</td></tr>
<tr><td>[Alternative-3]
Construction of the Bridge
(Slab Bridge on Pile Bent)</td></tr>
</table>

Table 2.2.1-10 Alternative of Improvement Measures (3)

Alternatives	Profile	Cross Section
[Alternative-1] Strengthening of Existing Revetment		
[Alternative-2] Construction of the Bridge (PC Bridge)		
[Alternative-3] Construction of the Bridge (Slab Bridge on Pile Bent)		

2.2.1.3 Road Design Policy

(1) Road Cross Section

1） Road Width

Road width is planned based on the following concept;

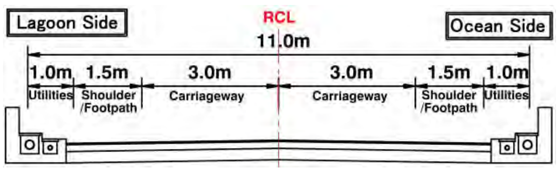
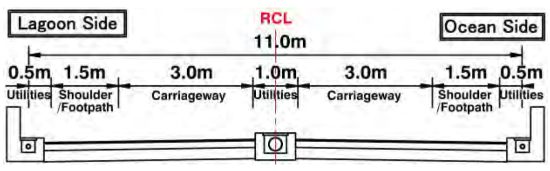
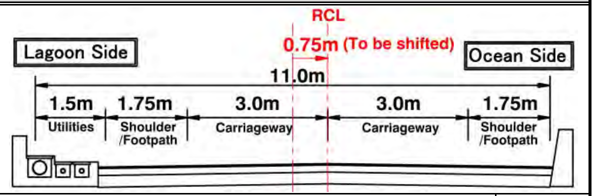
- Carriageway width shall be decided in consideration with road service level of causeway and continuity of Kiribati Road Rehabilitation Project (hereafter KRRP) which has been implemented through Australian aid,
- The Causeway is an important highway leading to the only international port (Betio Port) of Kiribati,
- Therefore, carriageway width is set at 3.0m (Carriageway width of KRRP is also 3.0m),
- And Shoulder/Footpath shall be decided in consideration with the relocation of utilities.

Road width for the project is shown in Table 2.2.1-11. And comparison table for road width and relocation of utilities is shown in Table 2.2.1-12.

Table 2.2.1-11 Road Width for the Project

Items	Road Width		
	Original Design	KRRP	The Project
Carriageway	3.00m	3.00m	3.00m
Shoulder/Footpath	2.50m	1.00m/1.50m	1.75m

Table 2.2.1-12 Cross Section Alternatives (Section with utilities Separated from Causeway Structure)

Alternatives	Alternative-1		Alternative-2		Alternative-3	
Cross Section						
Road Width/ Safety	➢ Carriageway is 3.0m and shoulder/footpath is 1.5m. ➢ Utilities box part for small utilities such as telecommunication cable and electric power cable can be utilized as footpath.	△	➢ Carriageway is 3.0m and shoulder/footpath is 1.5m. ➢ Utilities box part for small utilities such as telecommunication cable and electric power cable can be utilized as footpath. ➢ Utility box installed at the road center can be utilized as the center median, so road safety is higher than that of the other alternatives.	○	➢ Carriageway is 3.0m and shoulder/footpath is 1.75m. (Road width is the widest of all.) ➢ Utilities box part for small utilities such as telecommunication cable and electric power cable can be utilized as footpath. ➢ Shift of road center line is needed.	◎
Maintenance of Utilities	➢ Maintenance of utilities is performed without the shoulder excavation for the utility's maintenance. ➢ Utilities space is 1.0m.	○	➢ Maintenance of utilities is performed without the shoulder excavation for the utility's maintenance. ➢ Utilities space of road edge is 0.5m ➢ Utilities space of road center is 1.0m ➢ Maintenance of center pipe (water pipe) is less efficient than other utilities installed at parapet.	○	➢ Maintenance of utilities is performed without the shoulder excavation for the utility's maintenance. ➢ All utilities are placed to lagoon side, so wave impact from ocean side can be avoided.	◎
Economy (Ratio)	1.10	○	1.12	△	1.00	◎
Evaluation	**Not preferable** **: This alternative is less attractive than the other alternatives in terms of advantages.**	△	**Preferable** **: Maintenance work will be easier than the other alternatives, because three types of utilities are installed separately.**	○	**Most preferable** **: Road width is wider than the other alternatives because space for utilities can be made most compact among all alternatives.**	◎

2） Superelevation

Superelevation is set as 2.0% normal crown.

3） Review of Road Elevation

Review of the road elevation associated with review of the current tide level data shall be conducted. Review of the tide level will be described in “Section 2.1.4.2”.

Review of the road elevation is considered based on the following concept;

- Result of the tide level review, HWL is D.L+2.85m,
- And the road elevation rises to a position (D.L.+2.85m +1.5m = D.L.+4.35m) where the road groundwater level (HWL) is not affected to road base course.

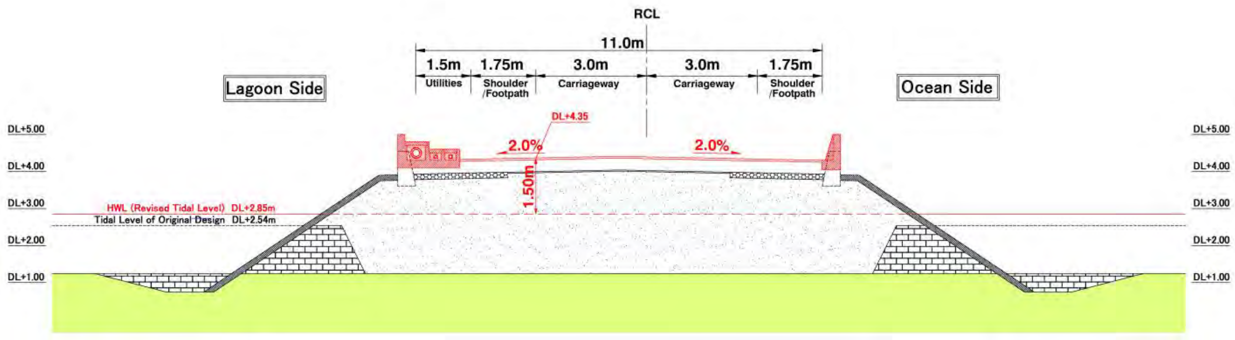


Figure 2.2.1-1 Review of the Road Elevation

(2) Design Speed and Geometrical Structure

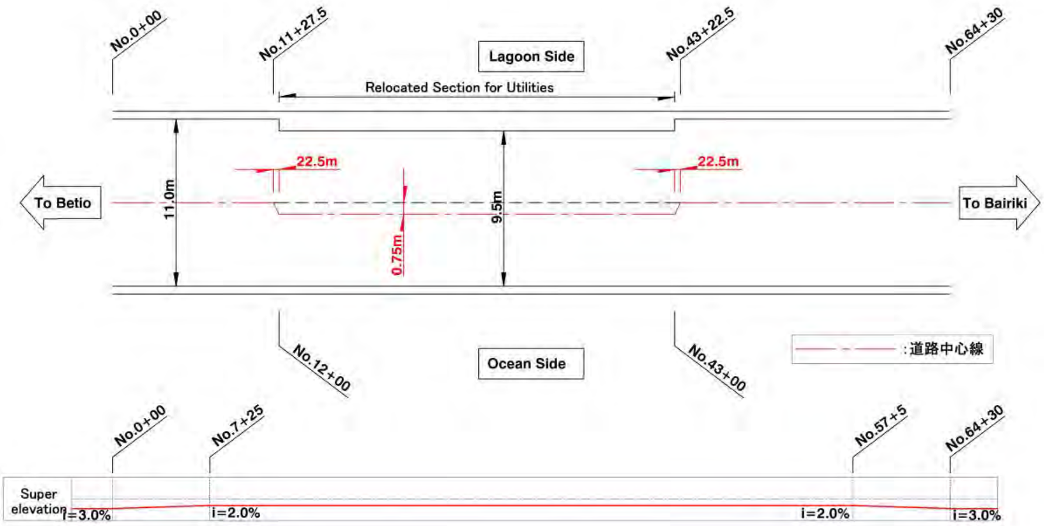
1） Design Speed

Design Speed is set as 60km/h.

2） Horizontal Alignment

The concept of the project is reconstruction of the existing causeway, therefore the horizontal alignment is not changed. However, in relocated section of utilities, road center line shift is needed due to installation of utilities box. Outline of horizontal alignment is shown in Table 2.2.1-13.

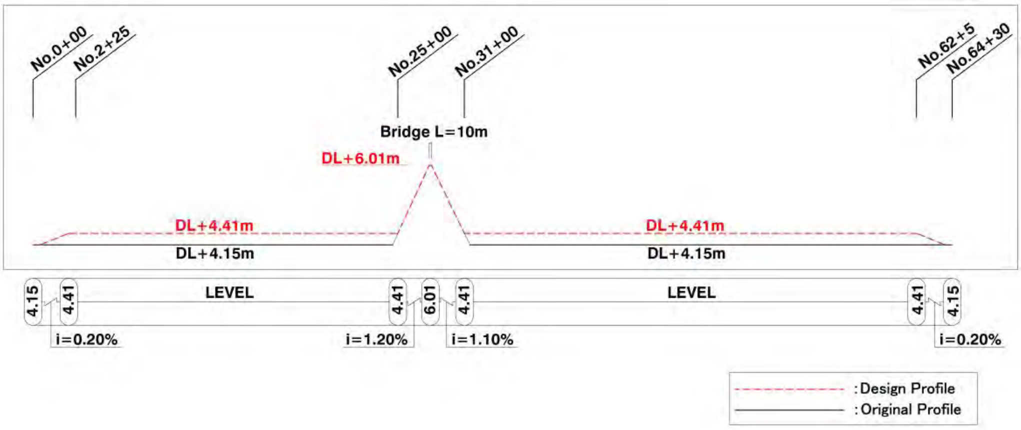
Table 2.2.1-13 Outline of Horizontal Alignment

Minimum Curve Radius	R=1500m (≧150m)
Outline	 

3) Profile

Profile is designed in consideration with the review of road elevation. Outline of profile is shown in Table 2.2.1-14.

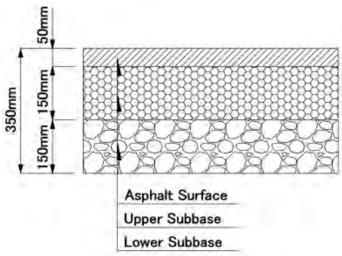
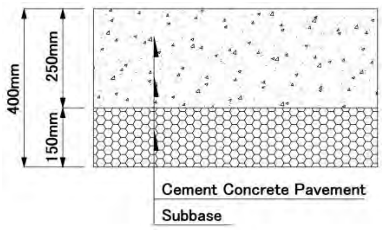
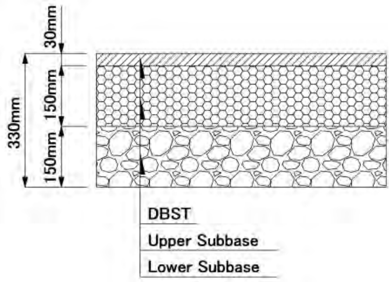
Table 2.2.1-14 Outline of Profile

Maximum Gradient	1.1% (≦5%)
Outline	

(3) Pavement Type

Most preferable pavement type shall be selected in consideration with maintenance and cost (initial cost and maintenance cost). In this project, pavement type is considered compared with asphalt concrete, cement concrete and DBST. Comparison table of pavement type is shown in Table 2.2.1-15.

Table 2.2.1-15 Comparison table for Pavement Type

Type	Asphalt Concrete	Cement Concrete	DBST (Double Bituminous Surface Treatment)
Structure			
Design Period	10 years	20 years	3～5 years
Maintenance	➢ The pavement is maintained by patching and sealing as routine maintenance. ➢ Overlay of the pavement is required every 10 years.	➢ The pavement is maintained by patching and sealing as routine maintenance. ➢ Resurfacing of the pavement is required every 20 years.	➢ The pavement is maintained by patching and sealing as routine maintenance. ➢ Overlay or Resurfacing of the pavement are required every 3 to 5 years.
Initial Cost (Ratio)*	1.18	1.47	1.00
Initial Cost + LCC (Ratio)*	1.00	1.13	1.19
Total Evaluation	○ Initial cost is higher than DBST, but total cost (initial cost + LCC) is the cheapest.	△ Initial cost is the highest, but total cost is higher than asphalt concrete.	× Initial cost is the cheapest, but total cost is the highest.

* Cost estimation tables are attached in Appendix-9

(4) Road Drainage

Road drainage is designed based on the following concept;

- ➢ Transverse drainage shall be installed at the bottom of the parapet to prevent the flow of water inside of the causeway. (same as existing drainage structure)
- ➢ Road profile is level except around the bridge section, therefore drainage slope for road drainage shall be planned to prevent the retention of water.
- ➢ Transverse drainage and installation interval shall be decided in consideration with intensity of rainfall and overtopping.

Design condition for road drainage is shown in Table 2.2.1-16.

Table 2.2.1-16 Design Condition for Road Drainage

No.	Item	Figure	Remarks
1	Overtopping Volume	$0.02m^3/m/sec$	
2	Design Traffic Volume	5718	10 years later
		7685	20 years later
3	Reoccurrence period of rainfall	3 years	
4	Road Area	35.53	km2
5	Amount of Rainfall	150	mm/d(2014/12/31)
6	Intensity of Rainfall	37.5	In=Rn*βn=Rn*a'/(t+b)

2.2.1.4 Revetment Design Policy

(1) Concept for the Measure of Revetment

Schematic drawing of the causeway which is divided into four sections is shown in Figure 2.2.1-2. Existing revetments of lagoon side and ocean side are same structure. But disaster situation by the waves in the past differs in each section. In the results of inventory survey of the revetments, the revetment damage of ocean side is very serious, and revetment damage of lagoon side is small. In particular, the damage of the revetments of ocean side and Bairiki side are very serious.

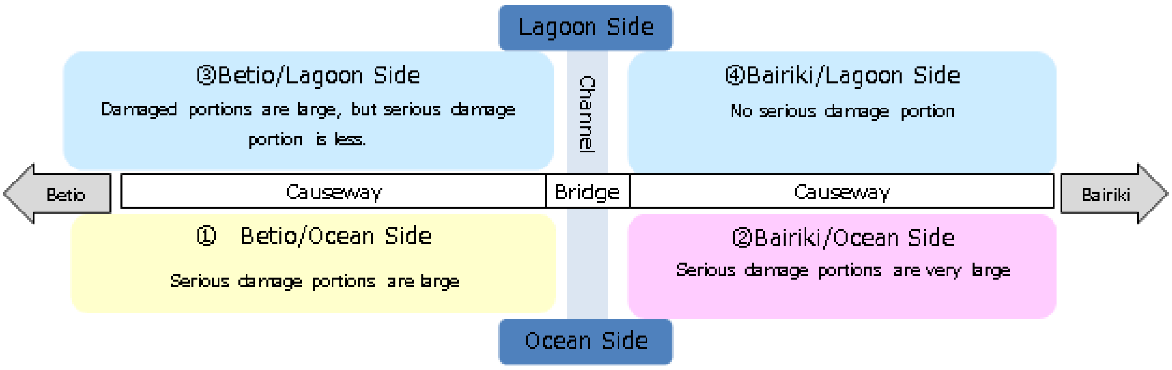


Figure 2.2.1-2 Features of the Causeway Damage

From the perspective of secure expression of the project effect and cost reduction, it is not reasonable to take the same measures to all sections against all of four sections which have different features of the damage. Therefore, as shown in Table 2.2.1-17, the improving concept for the revetment is compared among several alternatives to adopt the reasonable measures in response to the damage of the revetment.

Table 2.2.1-17 Basic Concept for the Measures of the Causeway

Alternatives		Alternative-1	Alternative-2	Alternative-3
Abstract		Same measures are taken to all sections	Measures are taken separately in lagoon side and ocean side	Measures are taken separately in four sections. (refer to figure 3.1-1)
Measures	1)Betio/Ocean Side	Massive Measure (Reinforcing)	Massive Measure (Reinforcing)	Medium-scale Measure (Reinforcing)
	2)Bairiki/Ocean Side			Massive Measure (Reinforcing)
	3)Betio/Lagoon Side		Small-scale Measure (Reinforcing or Repair)	Small-scale Measure (Reinforcing or Repair)
	4)Bairiki/Lagoon Side			
Evaluation		➢ Massive measure is applied also to the area of minor damage. ➢ From the perspective of cost-effectiveness, this is not reasonable.	➢ Although it is possible to apply the measures in consideration with the different natural conditions and characteristics of the ocean side and lagoon side, it does not match the reality of the damage situation.	➢ It is possible to apply the measures in consideration with the different natural conditions characteristic of ocean side and lagoon side, and it matches the reality of the damage situation.
		×	△	◎

Legend) ◎ : Most Preferable、○ : Preferable、△ : Fair、× : Undesirable

(2) Design Wave Height / Tidal Level / Overtopping

1) Tidal Level

The design tidal levels are determined by the following three kinds of idea:

1) Tidal level in the same manner of the original design (HWL)
2) Tidal level in a counter measure for storm surge stipulated in "Technical Standards and Commentaries for Port and Harbour Facilities in Japan 2009" (TSPHS)
3) Tidal level with climate change as an addition to the above levels

Please refer to the chapter of Tide in Natural Environmental Condition in the Appendix for the further elaboration about the tide used in the original design, tidal datum, tide observation and effect of the climate change.

The tidal level 1) is used for the road design such as surface road elevation. In the revetment design the structural dimensions such as the height of the parapet is determined under the critical conditions with combinations of the tidal levels 1) - 3) and wave conditions.

The design tidal level in the original design was +2.54m of MHWS (Mean High Water Spring) which was +1.80m related to the datum in the original design. The HWL (High Water Level) becomes +2.85m, obtained from the observation data for the latest 5 years from 2010 to 2014. This is determined as the design tidal level 1) in the same manner as the original design, and considered as the case of the king tide. There is difference in the method - MHWS is obtained from the harmonic analysis and HWL is from the observed record. However, both tidal levels are almost the same. Thus, the king tide used here is defined as the tidal level larger than +2.80m.

In accordance with TSPHS, the tidal level in a countermeasure for storm surge is determined with the following four methods: [1] HHWL (Highest High Water Level) in the past records, [2] HWL (mean monthly-average Highest Water Level) plus the maximum tide deviation in the past records, [3] By using a probabilistic distribution of abnormal high tide in the past records, and [4] To economically determine by using a probabilistic distribution of extreme high tide and amount of damage. It becomes clear through the survey of the natural conditions that the highest tidal level in the past records was related closely with the El Niño event. Since the probabilistic distribution of the extreme high tide caused by El Niño event is not well defined, the methods of [1] and [2] are adopted for the determination of the design tidal levels. In accordance with the method [1], HHWL is +3.12m in the past records from 1974 to 2015. The maximum deviation between prediction and observation of the king tide (25 cm) is added to the HWL of +2.85m, and +3.10m is obtained to the method [2]. In the event that this nearly equals to the tidal level of +3.12m by the use of method [1], then the result by the use of method [2] is applied in this study.

The tidal level 3) under consideration of the climate change is determined with the addition of the mean global water level rise of 20 cm referring to the AR5 by IPCC.

The revised tidal conditions are summarized in Table 2.2.1-18 and Table 2.2.1-19. The same tidal conditions are adopted for both the ocean and the lagoon sides, although there exist time lags of tide between those.

Table 2.2.1-18 Revision of Tidal Conditions

Levels related to Tide	Design Tide at the original design	Revised Tidal Level		
		High Tide (King Tide)	King Tide (Under El Niño Phase)	King Tide (under El Niño and Sea Level Rise)
Tidal Level (m)	MHWS +2.48	HWL +2.85	+2.85	+2.85
Level Rise due to El Niño event	—	—	+25cm	+25cm
Level Rise due to Climate Change	—	—	—	+20cm
Design Tide (m)	+2.48	+2.85	+3.10	+3.30
Road Design	----	○	----	----
Revetment Design*		○	○	○
Remark	*: In the revetment design, the external forces and overtopping volume are estimated and confirmed within the allowable criteria under the critical conditions in the combination of the tidal levels 1) to 3) and waves at simultaneous event occurrence.			

Datum: SEAFRAME、MHWS (Mean High Water Spring) ≒HWL (High Water Level)

Source: The Study Team

The tidal level at the Cyclone Pam becomes +2.79m on 9th of March 2015.

Table 2.2.1-19 Original and Revised Tidal Conditions

Original Design	Revised Tidal Levels
	Extrema High Tide under El Niño and Sea Level Rise **+3.30**
HHWL +2.45 HHWL(SEAFRAME)+3.19	Extrema High Tide under El Niño Phase **+3.10**
	High Water Level (HWL) **+2.85**
MHWS +1.80 MHWS(SEAFRAME)**+2.54**	
MSL(1974-1978) +0.94 MSL(SEAFRAME)+1.68	Mean Sea Level (MSL) +1.68 (2010-2014)
MLWS +0.09 MLWS(SEAFRAME) +0.68	Lowest Low Water Level (LWL) +0.62
Datum University of Hawaii Gauge Zero (UoH) 0.0	
0.74m	Datum SEAFRAME Gauge Zero 0.0
Source Basic Design of Betio Bairiki Causeway and Fishery Channel Project (1985)	The Study Team on the basis of Kiribati Meteorological Services(MET)

2) Design Wave Height and Overtopping

a) Design Wave Height

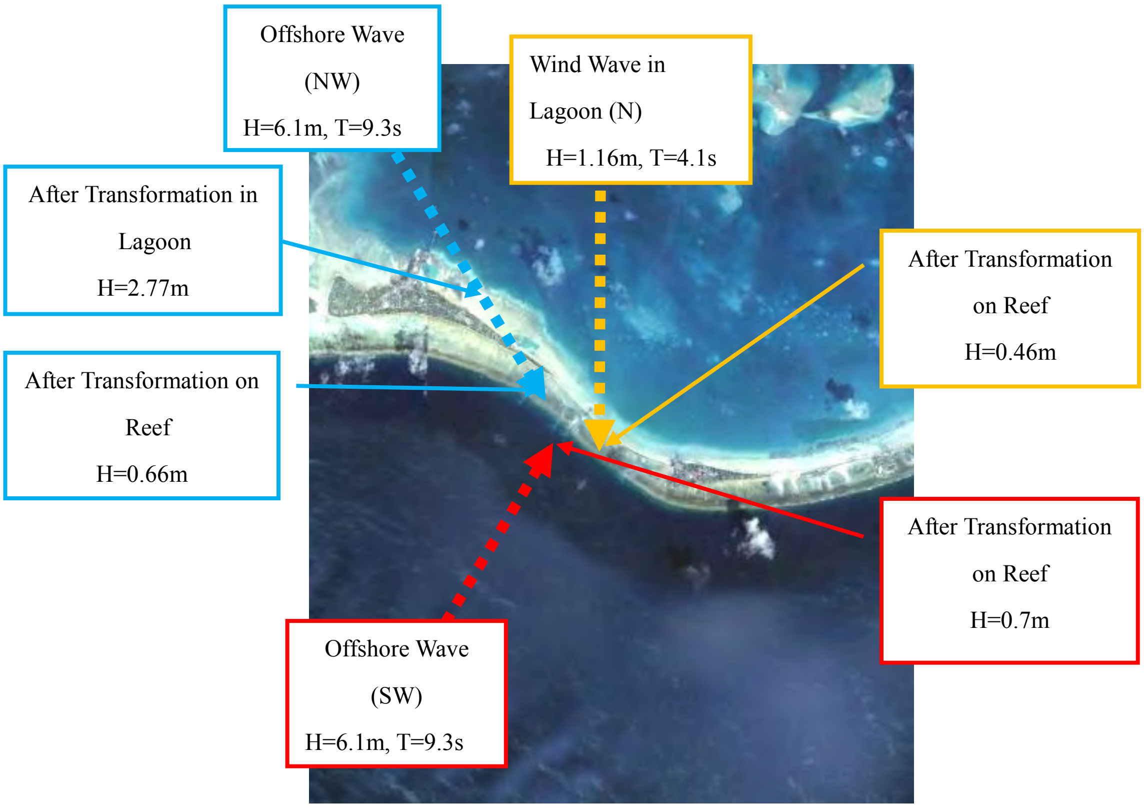
Since there is no record of the wave observation, the wave height from hindcast of the original design (50 years probability, $H_{1/3}$=6.1m, T=9.3s[3]) is applied for this study because of the conservative choice in comparison with another data source of the wave height by T&TI (Tonkin & Talor International, 50 years probability, Hs=3.26m, Tm=7.1s). With reference to the chapter of Wave in Natural Environmental Condition in the Appendix, it is considered that the adoption of the original offshore wave height is reasonable by the following reasons:

1) The offshore wave reaches at the revetment after a transformation on the reef shown in Figure 2.2.1-3. As indicated later in the results of the analysis, the offshore wave height does not affect much to the one at the revetment, because any high offshore wave becomes an extent of 1m after attenuation on the reef.
2) On the other hand the wave set up is sensitive by the offshore wave height. However, there is no records of the larger wave height than the applied offshore wave height.

[3] $H_{1/3}$: significant wave height, T: significant wave period, Hs: significant wave height, Tm: mean wave period

As indicated later in the results of the analysis, the offshore wave height does not affect much to the one at the revetment, because any high offshore wave becomes an extent of 1m after attenuation on the reef. On the other hand the wave set up is sensitive by the offshore wave height. However, there was no records of the larger wave height than the applied offshore wave height, therefore it is considered that the adoption of the original offshore wave height is reasonable.

For the lagoon side, the original design considered both cases of the same offshore wave incident from the west opening and wind wave generated inside the lagoon. The same wave height as in the original design is correspondingly applied for the lagoon side. The wave height used in the original design and the revised design wave height are indicated in Figure 2.2.1-3 and Table 2.2.1-20, respectively. The wave at the revetment is newly calculated under the condition of wave deformation on the reef.



Source: The Study Team on the basis of Kiribati Meteorological Services(MET)

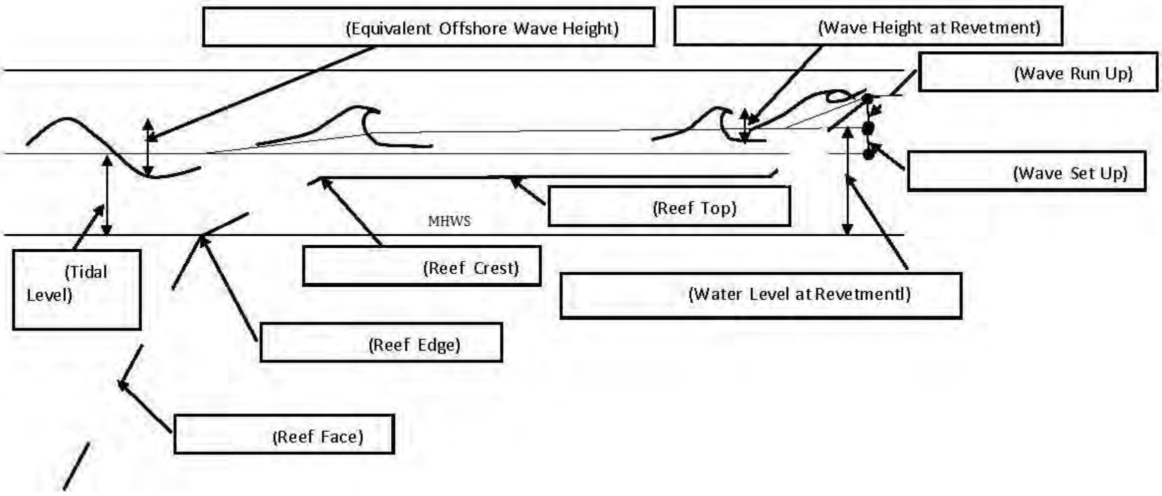
Figure 2.2.1-3 Wave Heights used in the Original Design

Table 2.2.1-20 Design Wave Height

Item	Adopted the same wave conditions as the original design		
	Ocean side	Lagoon side	
		Incident wave from ocean	Wind wave in Lagoon
Offshore Wave Height (m)	6.1	6.1	1.14
Offshore Wave Period (s)	9.3	9.3	4.1

b) Overtopping (Ocean Side)

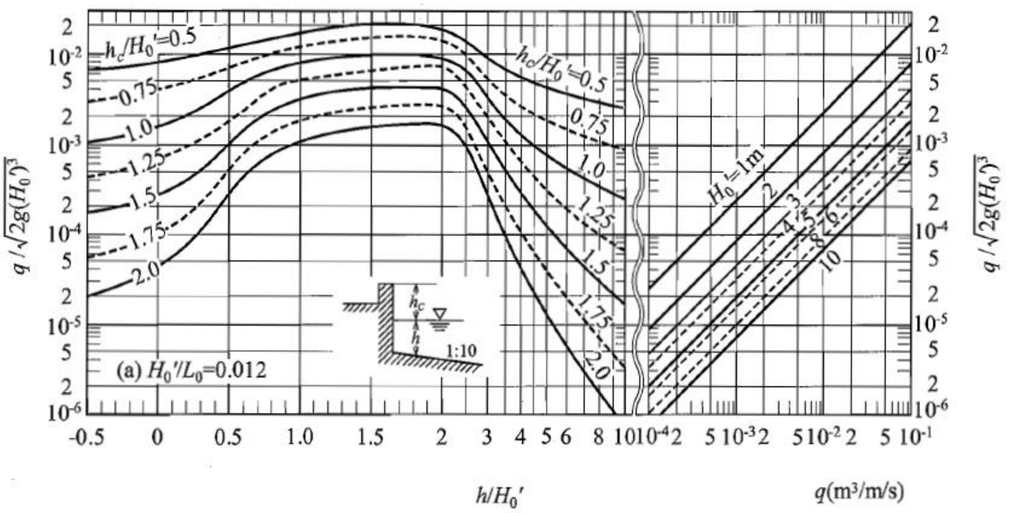
With revised conditions of wave and tidal levels, transformed wave characteristics on the reef as indicated in Figure 2.2.1-4, the wave height at revetment, and the overtopping rate are obtained.



Source: The Study Team

Figure 2.2.1-4 Characteristics related to Wave Transformation on Reef

The wave height and the water level at the revetment were calculated using Takayama (1977) which is described in TSPHS and in the same manner as in the original design. The top elevation of the original revetment were determined using wave run up from the water level on the middle height of the revetment slope. Because the water run up is not appropriate to the revised water levels which reach to the shoulder of the revetment slope, the revised cases are compared with an overtopping rates. The overtopping rate can be obtained from the diagram by GODA (Figure 2.2.1-5), although the applied parameters differ a little from the specified. The applicability of the diagram was confirmed through the comparison with the results by other methods of analysis.



Source: Technical Standards and Commentaries for Port and Harbour Facilities in Japan (2009)

Figure 2.2.1-5 Diagram for Estimating Overtopping Rate (GODA)

Although a reduction either to the design wave height or the revised tidal levels can be possible in case probability of exceedance is considered as applied in Coastal Calculator[4], the simultaneous occurrence of the wave height and tidal levels considered in the revised conditions is adopted to this study for the severest case. The high tide (king tide) may be encountered at an average rate of about 20 times a year. The king tide under El Niño phase presents the maximum tidal level in the past, and the king tide under El Niño and sea level rise becomes the higher limit of tidal level rise after 30 years. The wave height and water level at the revetment, and overtopping rate are obtained and summarized in Table 2.2.1-21 using the above-mentioned method and the combination of the design wave height and the revise tidal levels as the basic cases

Table 2.2.1-21 Overtopping Rate under Revised Conditions (Ocean Side - Basic Cases)

Items	Original Design MHWS	Revised Conditions (Basic Cases)		
		HWL High Tide (King Tide)	King Tide (Under El Niño Phase)	King Tide (under El Niño and Sea Level Rise)
Offshore Wave Height (m)	6.1	6.1	6.1	6.1
Offshore Wave Period (s)	9.3	9.3	9.3	9.3
Design Tidal Level (m)	+2.54	+2.85	+3.10	+3.30
Wave Set Up (m)	0.70	0.67	0.62	0.61
Water Level at Revetment (m)	+3.18	+3.52	+3.72	+3.91
Wave Height at Revetment (m)	0.70	0.82	0.89	0.95
Wave Run Up (m)	1.3	—	—	—
Parapet Height (m)	+4.54	+5.0	+5.0	+5.0
Overtopping Rate (m^3/m/s) (Overtopping Rate for (+4.54)existing height)	0.0016	0.0020 (0.011)	0.0067 (0.016)	0.012 (0.028)

Datum: SEAFRAME
Source: The Study Team

Since the cyclone Pam caused the damage in spite of away path, it is understood the reason that the impact by swell and sea level rise became strong subject to the development of the cyclone Pam, and the south coast of Tarawa was facing swell approaching direction. Therefore, the sensitivity analysis in both cases of the selected long periods related to the damage by swell, and magnitude of the offshore wave height were carried out. The offshore wave height with the wave period of 15 seconds was taken into account to acquire the effect of wave period corresponding to the actual damage by swell, although the long period of 15 seconds was not recorded in Global Wave Statistics. In case of the water level rise due to climate change, the extreme design wave height and such a long period is not presumed to generate at the extreme tidal level, a moderate wave height of 3.5m was selected. In case of the tidal level under El Niño, the design wave period of 9.3 sec is chosen as a realistic case with reference to Global Wave Statistics. The combination of the king tide and a daily wave height is included for a possible condition as well. The results of the sensitivity analysis and comparative case of the cyclone Pam are summarized in Table 2.2.1-22.

[4] Coastal Calculator is a tool developed by NIWA and provided to Kiribati under Kiribati Adaptation Program. It can estimate probabilistic tidal levels, wave run up, or overtopping at particular location in Tarawa atoll taking into account of the climate change.

Table 2.2.1-22 Overtopping Rate under Revised Conditions (Ocean Side - Sensitivity Analysis)

Items	Original Design	Current Condition Cyclone Pam Observed Tide	Cases of Analysis (Sensitivity)			
			Long Period (King Tide)	Long Period Medium Offshore Wave Height (King Tide) Under El Niño	Medium Wave Offshore Height (King Tide)	Daily Offshore Wave Height (King Tide)
Offshore Wave Height (m)	6.1	6.1	6.1	3.5	3.5	2.0
Offshore Wave Period (s)	9.3	9.3	15.0	15.0	9.3	9.3
Design Tidal Level (m)	+2.54	+2.79	+2.85	+3.10	+2.85	+2.85
Wave Set Up (m)	0.70	0.66	0.92	0.53	0.38	0.29
Water Level at Revetment (m)	+3.24	+3.35	+3.93	+3.90	+3.56	+3.13
Wave Height at Revetment (m)	0.70	0.79	0.92	0.86	0.74	0.63
Wave Run Up (m)	1.3	-	-	-	-	-
Parapet Height (m)	+4.54	+4.54	+5.0	+5.0	+5.0	+5.0
Overtopping Rate (m3/m/s)	0.0016	0.0078	0.012	0.0053	0.0009	<0.00001

Datum: SEAFRAME
Source: The Study Team

To verify the above calculation and its appropriateness, the overtopping rate at the time of the damage is assumed on the basis of the site photo, and compared with the result of the calculation. Before and after 12th of March 2015 the cyclone Pam caused damage to the causeway, the overtopping rate of 0.0016～0.080(m3/m/wave) is presumed at the time on the site photo shown in Table 2.2.1-23. Referring to the calculation in Table 2.2.1-22, the overtopping rate of 0.0078(m^3/m/s) or 0.073(m3/m/wave) using the wave period of 9.3s is obtained under the present parapet height (+4.54m), and therefore the result of the calculation seems reasonable comparing the above assumed rate at the damage.

Table 2.2.1-23　Presumed Overtopping Rate at Damage

Source: KRRP

Appearance of Overtopping	Overtopping Rate in one Wave ($2\pi Q/(HoL_A)$)
Splash (water mass)	$< 10^{-4}$
Splash, Jumping water (large water mass) Part of wave body over	10^{-4} - $5x10^{-3}$
Wave body continuously over	$5x10^{-3}$ - 10^{-2}
Entire wave over	$>10^{-2}$

Source: Akira TAKADA "Wave Run Up and Overtopping"

Photo at the damage on 12th of March 2015

The overtopping rate of 10^{-4}～$5x10^{-3}$ ($2\pi Q/(HoL_A)$) is obtained from the above Table, and Q=0.0016～0.080(m^3/m/wave) is presumed using the wave height of 0.8m and the period of 9 sec.

c) Planned Parapet Height (Ocean Side)

The parapet height is determined so that the maximum overtopping rate in the related cases including the sensitivity analysis does not exceed the permissible rate. The permissible overtopping rates are specified in several standards and guidelines as follows:

0.2 (m3/m/s) as a threshold limit of damage prevention for seawall paved behind

0.02 (m3/m/s) as a permissible limit for the important hinterland

The above limits are described in "Technical Standards and Commentaries for Port and Harbour Facilities in Japan (2009)",

- 1×10−4 - 1×10−6 (m^3/m/s) as standard allowance for revetment road stipulated in "Road Design Guideline (2015)" of Hokkaido Regional Development Bureau, and
- 0.01-0.05(m^3/m/s) of mean discharge as a limit for vehicles driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed, introduced in EurOtop (2007)

Under consideration of improved drainage capacity on road after the rehabilitation and traffic restriction which can be required only in the short duration for the period of the analyzed cases, the limits used in this Study are set as follows:

- 0.02 (m^3/m/s) as the limit for vehicles under traffic control, but prevention of road damage
- 1×10^{-4} (m^3/m/s) as the limit for the ordinary traffic

The parapet height of +5.0 m is determined for the ocean side, and it is confirmed in Table 2.2.1-24 that the obtained overtopping rates are within the permissible limit. The maximum overtopping rate is 0.012(m^3/m/s) at the king tide under El Niño and Sea Level Rise (+3.30m) in case of the design wave height.

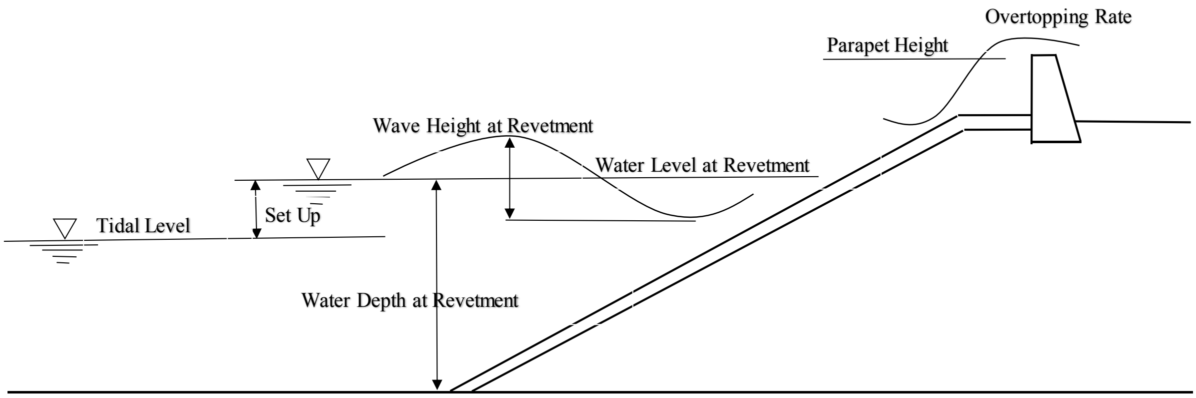
Table 2.2.1-24 Overtopping Rate and Limits (Ocean Side)

Items	Original Design	Case of Analysis						
		Design Tidal Level (Base)			Sensitive Analysis			
		High Tide (King Tide)	King Tide (Under El Niño Phase)	King Tide (under El Niño and Sea Level Rise)	Long Period (King Tide)	Long Period Medium Offshore Wave Height (King Tide) Under El Niño	Medium Wave Offshore Height (King Tide)	Daily Offshore Wave Height (King Tide)
Offshore Wave Height (m)	6.1	6.1	6.1	6.1	6.1	3.5	3.5	2.0
Offshore Wave Period (s)	9.3	9.3	9.3	9.3	15.0	15.0	9.3	9.3
Design Tidal Level (m)	+2.54	+2.85	+3.10	+3.30	+2.85	+3.30	+3.10	+2.85
Wave Height at Revetment (m)	0.70	0.82	0.89	0.95	0.92	0.86	0.74	0.63
Parapet Height (m)	+4.54	+5.0	+5.0	+5.0	+5.0	+5.0	+5.0	+5.0
Overtopping Rate (m^3/m/s)	0.0016	0.0020 (0.098: Existing Parapet Height)	0.0067 (0.019: Existing Parapet Height)	0.0120 (0.033: Existing Parapet Height)	0.0120	0.0053	0.0009	<0.00001
Permissible Overtopping Rate (m^3/m/s)	–	0.0200						1×10^{-4}
Applied Conditions	–	traffic control necessary, but prevention of road damage						traffic available by immediate drainage

Datum: SEAFRAME

Source: The Study Team

The wave height and water level at the revetment, the parapet height, and overtopping rate are illustrated in Figure 2.2.1-6.



Source: The Study Team

Figure 2.2.1-6　Explanatory Sketch of Terms used at Revetment

d）　Overtopping Rate and Planned Parapet Height (Lagoon Side)

The offshore wave height in the original design (50 years probability, H1/3=6.1m, T=9.3s) is applied likewise.　The case of the wind wave generated in the lagoon (50 years probability, H1/3=1.14m, T=4.19s) was separately considered in the original design. In this study, it is only confirmed that the wind wave in the lagoon does not cause a critical overtopping because of the lower wave height at the revetment than that by the incident wave even at the severer revised tidal levels.

The incident offshore wave from the west opening of the lagoon is propagated with transforming to the reef edge of the causeway like on the reef. Table 2.2.1-25 gives the conditions of the transforming in the original design and at the revised tidal levels.

Table 2.2.1-25　Transforming of Offhore Wave from Lagoon Opening to Reef Edge

Items	Original Design MHWS	Revised Conditions (Basic Cases)		
		HWL High Tide (King Tide)	King Tide (Under El Niño Phase)	King Tide (under El Niño and Sea Level Rise)
Offshore Wave Height (m)	6.1	6.1	6.1	6.1
Offshore Wave Period (s)	9.3	9.3	9.3	9.3
Tidal Level (m)	+2.54	+2.85	+3.10	+3.30
Wave Set Up(m)	0.41	0.31	0.32	0.32
Water Level at Reef Edge (m)	+2.95	+3.16	+3.42	+3.62
Wave Height at Reef Edge (m)	2.77	2.84	2.93	3.00

Datum: SEAFRAME

Source: The Study Team

The same calculation as for the ocean side is carried out with the wave height at the reef edge in Table 2.2.1-25.

The same parapet height of +5.0 m at the lagoon side is determined, and the overtopping rates are indicated in Table 2.2.1-26. Although the incident angle of wave may reduce the overtopping rate, it is not considered in that estimates as conservative side. The incident angle of wave is not considered in that estimates as conservative side. The maximum overtopping rate is 0.022(m^3/m/s) at the king tide under El Niño and Sea Level Rise (+3.30m) in case of the design wave height, and it slightly exceeds the permissible limit of 0.02. However, it can be judged that it should become within the limit, because the parapet height of +5.0m could be reduced[5] to cover the limits if the incident angle from the lagoon opening is taken into account.

Table 2.2.1-26 Overtopping Rate and Limits (Lagoon Side)

Items	Original Design	Case of Analysis					
		Design Tidal Level (Basic Cases)			Sensitive Analysis		
		High Tide (King Tide)	King Tide (Under El Niño Phase)	King Tide (under El Niño and Sea Level Rise)	Long Period (King Tide)	Daily Offshore Wave Height (King Tide)
Offshore Wave Height (m)	6.1	6.1	6.1	6.1	6.1	2.0
Offshore Wave Period (s)	9.3	9.3	9.3	9.3	15.0	9.3
Design Tidal Level (m)	+2.54	+2.85	+3.10	+3.30	+2.85	+2.85
Water Level at Reef Edge (m)	+2.95	+3.16	+3.42	+3.62	+3.37	+2.81
Wave Height at Reef Edge (m)	2.77	2.84	2.93	3.00	2.91	2.73
Wave Height at Revetment (m)	0.66	0.98	1.08	1.15	1.11	0.86
Wave Set Up (m)	0.28	0.21	0.22	0.21	0.34	0.22
Parapet Height (m)	+4.54	+5.00	+5.00	+5.00	+5.00	+5.00
Overtopping Rate (m^3/m/s)	0.001	0.007	0.010	0.022(note)	0.016	0.007
Permissible Overtopping Rate (m^3/m/s)	－	0.020				
Applied Conditions	－	traffic control necessary, but prevention of road damage				

Datum: SEAFRAME

Note: This should become within the limit, because the parapet height of +5.0m could be reduced to cover the limits if the incident angle from the lagoon opening is taken into account.

Source: The Study Team

[5] Takayama et al, "Hydraulic Model Test for wave overtopping characteristics of sea walls against diagonal random incident waves", Proceeding of Coastal Eng. JSCE, Vol 31, shows the equivalent wall height, which defines the ratio of height giving the same overtopping rate, and is reducing pro rata to sinθ with the incident angle: θ. The equivalent wall height ratio becomes 0.6 compared with the perpendicular incident in the similar conditions to the analyzed case of this study.

e) Comparison with Analysis by Other Methods

The results using Takayama and Goda are compared with the ones using the further elaborate analysis: CADMAS-SURF/2D[6], and with Coastal Calculator and estimates done by T&TI for reference. Table 2.2.1-27 and Table 2.2.1-28 summarize the comparison. The overtopping rates using CADMAS are smaller in case of the King Tide, and about the same in case of the Sea Level Rise, but fairly larger in case of long wave period, than those using Takayama and Goda. This combination of long wave period and extreme wave height is adopted for the sensitivity purpose, but out of real range. Therefore, we can judge that the comparison may support the applicability of Takayama and Goda for the realistic range of the wave height and period. Coastal Calculator shows a little larger or the same figures for the case of the similar tidal level and top width of 1m. The estimation by T&TI, which does not differ very much, is also included in the table. It may also assist the conclusion of the availability of Takayama and Goda.

The some outputs of overtopping in CADMAS are illustrated in Figure 2-2-1-7 and Figure 2-2-1-8. The figures show some overtopping under the original design condition, and the severer overtopping cased in the revised tidal level which represents the present situation. A sample display of Coastal Calculator is shown in Figure 2-2-1-9. Necessary input in the left area gives overtopping or other required parameters by frequency of tide and wave in the right area. This figure shows the overtopping of 22.58ℓ/m/s (=0.023 m^3/m/s) in case of 1% AEP (Annual Exceedance Probability) at the existing parapet height as indicated in Table 2-2-24.

Table 2.2.1-27 Comparison of Results by Method of Analysis (Ocean Side)

Compared Items	Original Design (+2.54m)			Current Condition High Tide (King Tide:+2.85m) Existing Parapet Height			Long Wave Period High Tide (King Tide:+2.85m) Planned Parapet Height		Revised Tidal Condition / Planned Parapet Height King Tide under El Niño and Sea Level Rise: +3.30m			
	Takayama & Goda	CADMAS	Coastal Calculator	Takayama & Goda	CADMAS	Coastal Calculator	Takayama & Goda	CADMAS	Takayama & Goda	CADMAS	Coastal Calculator	T&TI Report
Offshore Wave Height (m)	6.1	6.1	3.26	6.1	6.1	3.26	6.1	6.1	6.1	6.2	3.26	3.48
Offshore Wave Period (s)	9.3	9.3	—	9.3	9.3	—	15.0	15.0	9.3	9.3	—	7.36
Tidal Level (m)	+2.54	+2.54	+2.60 Note 1	+2.85	+2.85	+2.86 Note 1	+2.85	+2.85	+3.30	+3.30	+2.86 Note 1	-+2.76 Note 1
Water Level at Revetment (m)	+3.22	+3.09-	+3.16	+3.52	+3.43	+3.30	+3.77	+3.85	+3.91	+3.83	+3.30	+3.45
Wave Height at Revetment (m)	0.70	0.67 Note 3	0.63	0.82	0.73 Note 3	0.68	0.92	1.23 Note 3	0.95	0.79 Note 3	0.68	0.80
Parapet	+4.54	+4.54	+4.54	+4.54	+4.54	+4.54	+5.00	+5.00	+5.00	+5.00	+5.00	+4.70

6 CADMAS (Super Roller Flume for Computer Aided Design of Maritime Structure), Coastal Development Institute of Technology

Height (m)												Note 2
Overtopping Rate ($m^3/m/s$)	0.002	0.001	0.013	0.01	0.005	0.023	0.012	0.045	0.012	0.014	0.014	0.008

Datum: SEAFRAME

Note 1 : Tidal condition in Coastal Calculator-IPCC AR4 A1F1(2012-2039), T&TI report- A1B(2050-2059)

Note 2 : The existing parapet height in T&TI report is +4.7m. Note 3 : verified by passing wave *Source: The Study Team*

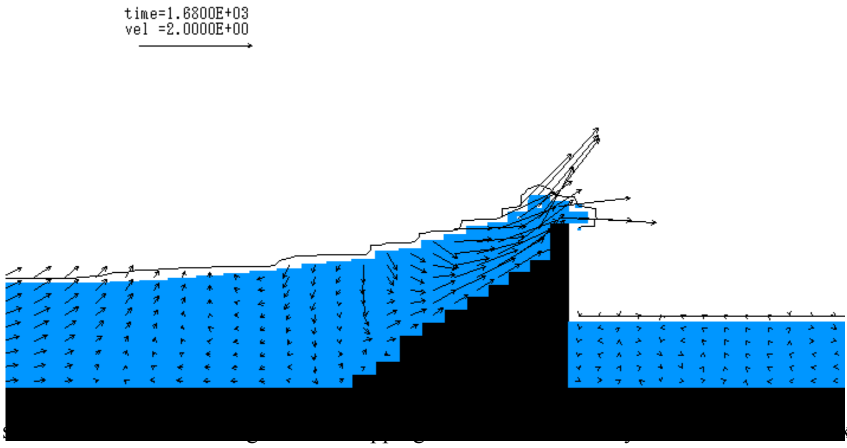
Table 2.2.1-28　Comparison of Results by Method of Analysis (Lagoon Side)

Compared Items	Revised Tidal Condition / Planned Parapet Height King Tide under El Niño and Sea Level Rise: +3.30m			
	Takayama & Goda	CADMAS	Coastal Calculator	T&TI Report
Offshore Wave Height (m)	3.00	3.00	1.54 (Note 1)	1.64 (Note 1)
Offshore Wave Period (s)	9.3	9.1	-	3.06
Tidal Level (m)	+3.62	+3.62	+2.86 (Note 2)	+3.03 (Note 2)
Water Level at Revetment (m)	+3.83	+3.72	+3.18	+3.29
Wave Height at Revetment (m)	1.15	0.96 (Note 4)	1.13	1.25
Parapet Height (m)	+5.00	+5.00	+5.00	+4.70 (Note 3)
Overtopping Rate ($m^3/m/s$)	0.022	0.019	0.003	0.047 (Note 3)

Datum: SEAFRAME

Note 1 : Coastal Calculator and T&TI adopt the wind wave generated inside lagoon
Note 2 : Tidal condition in Coastal Calculator-IPCC AR4 A1F1(2012-2039), T&TI report- A1B(2050-2059)
Note 3 : The existing parapet height in T&TI report is +4.7m. 0.02 is presumed in case of +5.0m.
Note 4 : verified by passing wave

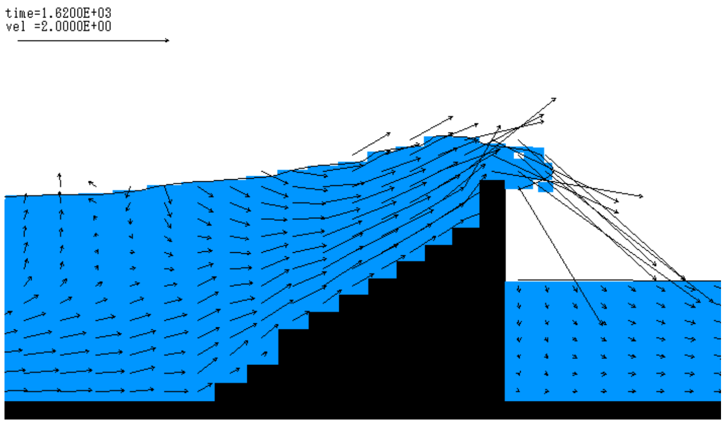
Source: The Study Team



Note: The road s ... xperimental flume.

Source: The Study Team

Figure 2.2.1-7　Overtopping Condition in CADMAS (Original Design)



Note: The road side is formed as a drainage for overtopping flows to enable analysis in the numerical experimental flume.

Source: The Study Team

Figure 2.2.1-8 Overtopping Condition in CADMAS (Original Design/ King Tide)

Source: Coastal Calculator property of MPWU

Figure 2.2.1-9 Display of Coastal Calculator (Original Design/ King Tide)

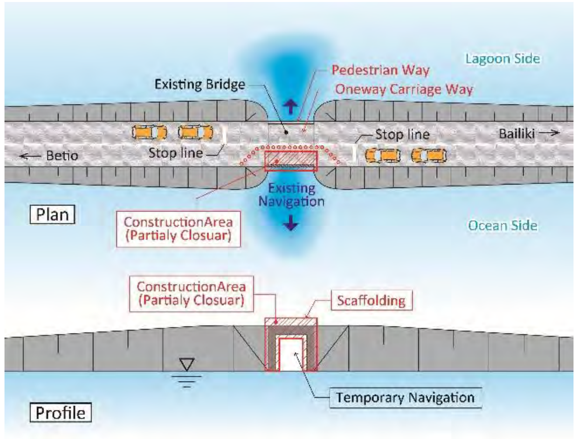
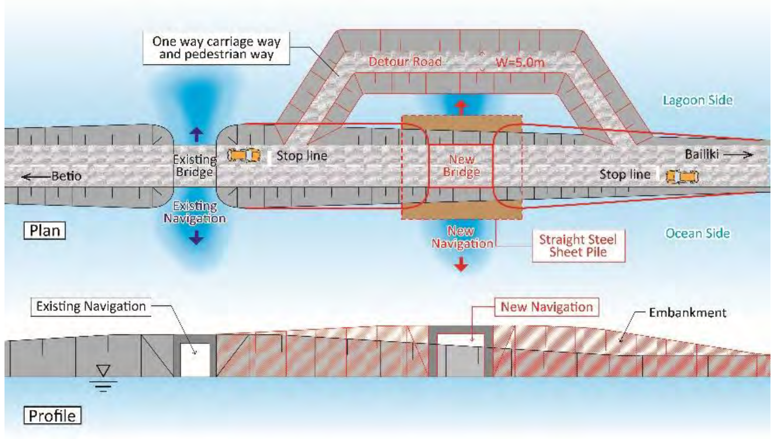
2.2.1.5 Policy of Bridge Design

(1) Concept for the Bridge Strengthening

In the result of visual inspection for the existing bridge, no fatal damage was found. However, repair work or the partial retrofitting of the existing bridge is recommended.

In this part, repair of the existing bridge (Alternative-1) and construction of new bridge (Alternative-2) which is one of the bridge strengthening option, are compared in consideration with workability, economy and social environment. In the result of comparison, alternative-1 is recommended. The comparison of bridge strengthening is shown in Table 2.2.1-29.

Table 2.2.1-29 Alternative for Bridge Strengthening

Alternatives	[Alternative-1]: Repair of Existing Bridge		[Alternative-2]: Construction of New Bridge	
Plan				
Overview	Repair of concrete wall and slab of existing bridge is conducted. Regulation of the traffic on the bridge (1 lane) is needed to repair the bridge. Although size of the ship is regulated, passing of the ship under the bridge during the repair work is allowable.		The new bridge is constructed away about 50m from the existing bridge, and existing bridge is repaired. Regulation of the traffic on the bridge (1 lane) is needed to repair the bridge. Passing of the ship under the bridge during the repair work is allowable.	
Workability	Workability is simple. Regulation of the traffic (1 lane) during the construction is needed. Construction period is shorter than alternative-2.	○	The steel sheet pile is driven to keep the construction yard for new bridge. Temporary road is needed for construction of new bridge. Regulation of the traffic (1 lane) during the construction is needed. Construction period is longer than alternative-1.	△
Navigation Clearance	Temporary regulation of passing the ship under the bridge is needed.	○	Existing navigation clearance is available during the construction. Construction of the channel for the new bridge is needed. (It shall be constructed by MPWU)	△
Economy	Cost is cheaper than alternative-2.	◎	Cost is higher than alternative-1.	×
	Cost Ratio：1.0		Cost Ratio：4.0（Cost of the channel for the new bridge is excluded）	
Maintenance	Maintenance is good.	○	Maintenance is good.	○
Social Environmental Consideration	Social environment is not affected.	○	Social environment is affected due to massive excavation for the new channel.	×
Evaluation	◎		△	

凡例） ◎: Most Preferable ○: Preferable △: Fair ×: Undesirable

(2) Cross Section of the Bridge

Cross section of the bridge is same as the road and widening of the bridge is needed as shown in Figure 2.2.1-10.

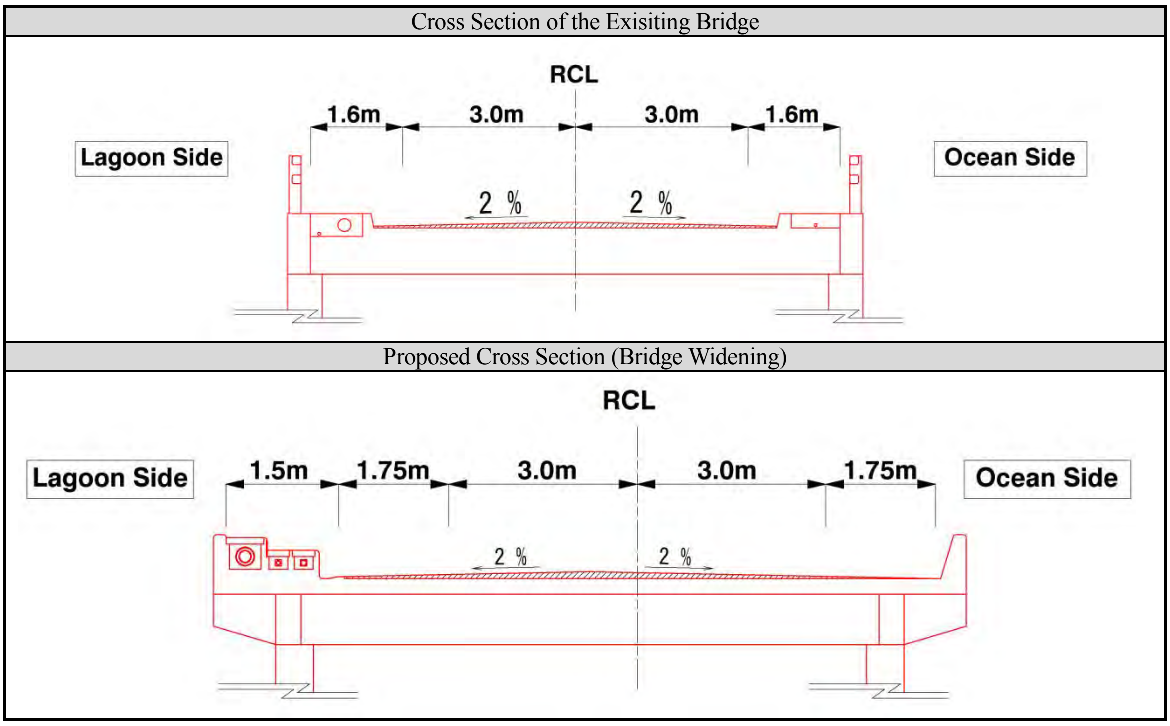


Figure 2.2.1-10 Cross Section of the Bridge

(3) Navigational Clearance

The bottom surface elevation of the causeway bridge is +5.14m (+4.40m related to the original design datum) as indicated in Figure 2.2.1-11, and 2.6m of the navigational clearance was secured above the design tidal level of +2.54m in the original design. This clearance at the revised tidal level was reviewed if any difficulty is prevalent. In addition to the review in the same way of the original design, the water depth of the fishery channel and wave conditions was also considered and reviewed.

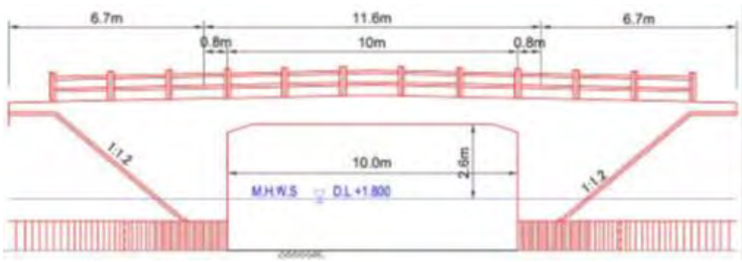


Figure 2.2.1-11 Navigational Clearance under Causeway Bridge

1) Review in the same way of the original design

The navigational clearance in case of the design tidal level (MHWS) in the original design was determined by using the wave height and wave setup after breaking of the offshore wave (50 years probability) on the reef, the draft of the fishing boat, and seated height as indicated in Table 2.2.1-30. The required navigational clearance for the revised tidal level (HWL) is obtained from the same way of the original design by using the same offshore wave height and conditions of the boat. It becomes 34 cm higher than the current elevation of +5.14. However, it is acceptable way of thinking that a fishing boat may not intend to pass the channel under an extreme condition such as high tide and 50 years wave.

Table 2.2.1-30 Navigational Clearance in the same way of Original Design

Item		Original Design (MHWS)	Revised Tidal Level (HWL)
Design Tidal Level (m)	a	+2.54	+2.85
Wave Setup after breaking of the offshore wave (50 yrs.) on the reef (m)	b	0.7	0.67
Wave Height on the reef /2 (m)	c	0.35	0.41
Draft + Trim + Seated Height + Others (m)	d	1.55	1.55
Required Elevation （a+b+c+d）		+5.14	+5.48
Bottom Surface Elevation (m)		+5.14	

Source: The Study Team

2) Review applying the depth of the channel and the wave conditions

The basis of the above review is different from the site phenomenon in the viewpoints of adoption of wave deformation on shallow reef to the actually deeper channel, further to the combination of the high tide and the extreme wave. Accordingly the clearance is reviewed applying the depth of the channel and the wave conditions for traffic of the fishery boat separately applied in the original design.

The water depth of the fishery channel (-2.26m at the access and -0.96m at the main part) was determined in the original design under the non-breaking wave conditions to enable the traffic of the fishery boat; i.e., the annual maximum offshore wave of 1.8m was not broken at the MSL, and the offshore wave of 0.8m at some occurrences in a year was not broken at LWL. The clearance is reviewed under the same boat traffic conditions as in the original design thought to no change at the present. Although the same offshore wave height given in the original design is adopted, the offshore wave at HWL reaches the main part of the channel without attenuation because of the revised higher tide. The wave height in the channel at MSL and LWL remains the same, since the revised tidal levels of MSL and LWL does not change significantly

Table 2.2.1-31 Navigational Clearance considering Boat Traffic and Water Depth

Item		Original Design			Revised Tidal Level		
		High Tide (MHWS)	Mean Tide (MSL)	Low Tide (MLWS)	High Tide (HWL)	Mean Tide (MSL)	Low Tide (LWL)
Tidal Level (m)	a	+2.54	+1.68	+0.83	+2.85	+1.68	+0.62
Offshore Wave height (m)		1.8	1.8	0.8	1.8	1.8	0.8
Wave Height at Main Part (m)		1.75	1.67	1.07	1.8	1.67	1.07
Wave Height /2	b	0.875	0.835	0.535	0.9	0.835	0.535
Draft + Trim + Seated Height + Others (m)	c	1.55	1.55	1.55	1.55	1.55	1.55
Required Elevation (m)	a+b+c	+4.97	+4.07	+2.92	+5.30	+4.07	+3.24
Bottom Surface Elevation (m)		+5.14					

Source: The Study Team

With reference to Table 2.2.1-31, it is clarified that the current bottom elevation of +5.14m satisfies the required navigational clearance of +4.07m for the fishing boat traffic under the annual maximum wave height at MSL, and it does not changed from the original design. At the revised HWL, it becomes short at 16 cm to the required clearance. However, it can be allowed for a limited occurrences and duration of HWL.

3） Wave crest

To confirm whether the crest of the presumed incident wave passes under the bottom of the bridge:

Sea Bed Depth of the channel: -0.96m (-1.70m related to the original design datum)
Tidal Level under Climate Change: +3.30m
Water Depth: h= 3.3+0.96=4.26m
Wave Height limited by the water depth: 0.78h= 0.78 x 4.26 = 3.32m
Height of Wave Crest: +3.30 + 0.5 x 3.32 = +3.30 + 1.66m = +4.96 m < +5.14m

The wave crest of the marginal wave height restricted by the water depth can pass the bottom of the bridge in the case of the maximum tidal level and sea level rise.

2.2.1.6 Policy for Relocation of Utilities

Concept of relocation of utilities is shown in the following items;

- Water pipes, communication cables and electric cables are buried under the shoulder.
- So far, decrease in the strength of soil cement associated with excavation of the shoulder for utilities maintenance and water leak of water pipe have affected on the structure of Nippon Causeway. (refer to Figure 2.2.1-12)
- To prevent them, utilities shall be installed in separate utility boxes.
- Relocated section is shown in Figure 2.2.1-13.

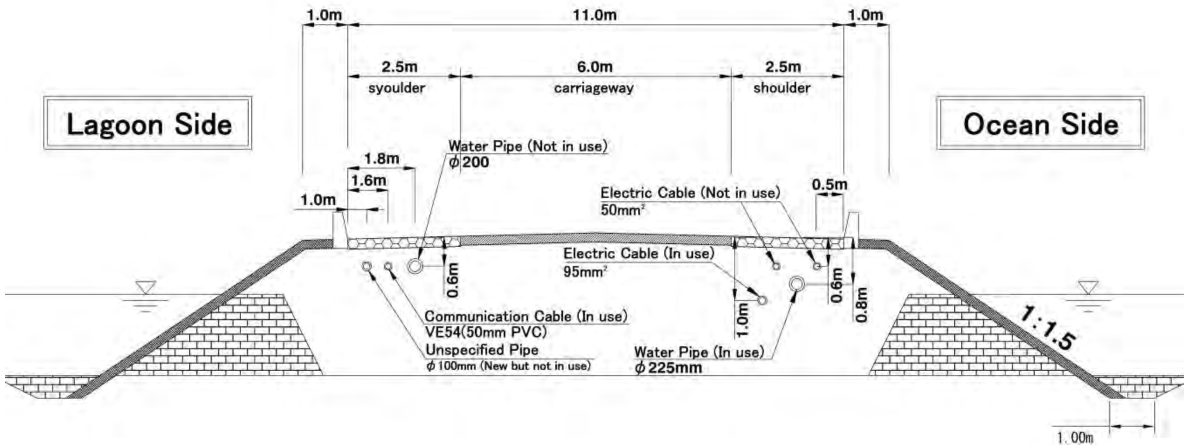


Figure 2.2.1-12 Typical Cross Section of Utilities

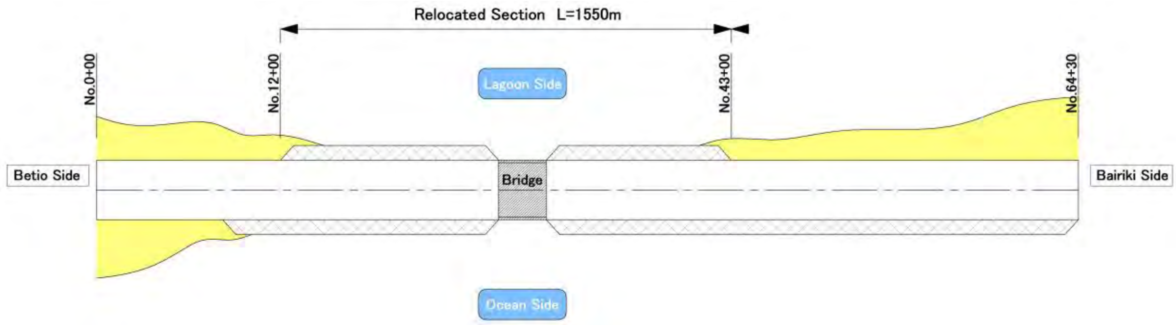


Figure 2.2.1-13 Relocated Section

2.2.1.7 Emergency Restoration Works Policy

(1) Content and evaluation of the emergency restoration work implemented by the local government

The restoration work is done by 10 workers in 1 group from 9 o'clock to 16 o'clock from Monday to Friday. The main works are as follows.

（1） Filling of pot holes on the surface pavement

Filling while walking with a dump truck

Filling pot holes by workers

Scraping and filling by motor grader

Natural compaction by vehicles

The procedure of filling is through spreading of sand in pot holes, level by foot, and then compaction by a vehicle. It shall be effective for a while, but gradually come off and wash away by rainfalls. Hence, it is merely a short term countermeasure.

In Betio side from the bridge, the MPWU's mortar grader is directly scraping the asphalt pavement for leveling and filling pot holes due to poor pavement condition.

(2) Slope crack repair by using mortar

Mortar mixture

Mortar application

Peeling after application of mortar due to poor workmanship

Mortar was made from cement and sand mixture with seawater and applied to a crack surface manually. Mixture ratio was not good and cement content was low. Neither V-cut at the crack nor pushing mortar inside of the crack was applied.

There were many places that applied mortals were peeling off due to improper work and lack of quality control concept.

A technical transfer was conducted at the first field survey by providing simple manuals for mortal mixture procedure and crack repair (see Figure 2.2.1-14).

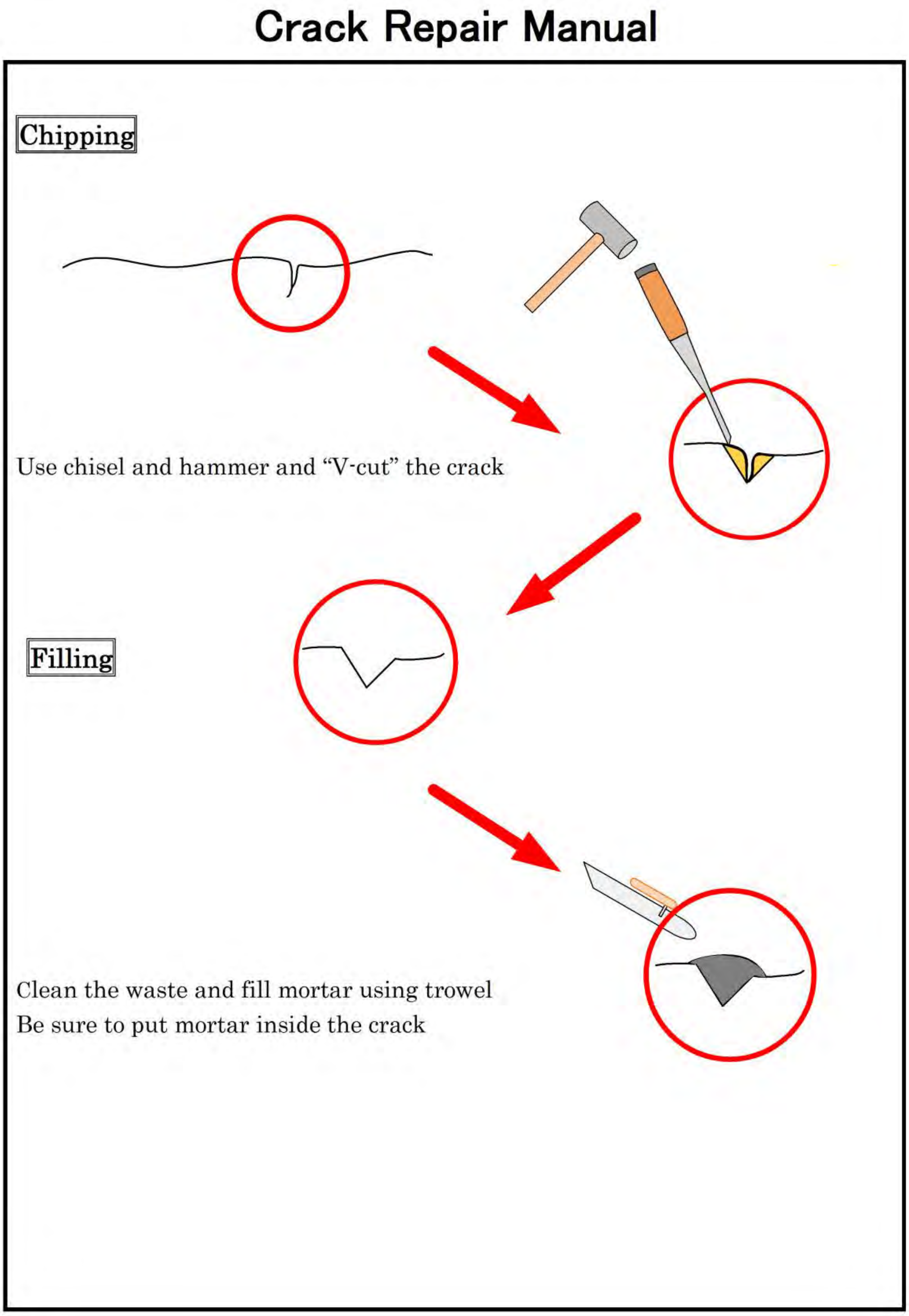


Figure 2.2.1-14　Crack Repair Manual

(3) Cavity filling behind slopes

Trial excavation

Large cavity under seawall

Hydraulic filling

Outflow of filling materials from cracks

A trial excavation was conducted at the shoulder bump and filling was conducted where sand was washed off and already depressed.

7Kw small generator and 2” water pump were utilized for hydraulic filling. The filling was properly conducted, but the cracks caused for cavity were not repaired. Hence, sand may be wash off again from the cracks.

It was instructed that hydraulic filling should be conducted during high tide to evade using water pump but in case the filling sand is washed off from the cracks, mark the location and repair the cracks during the next low tide.

(4) Repair of shoulders, slopes and seawalls by mortar sandbags

Sandbags are piled in one column (not alternately piled)

Repair of slopes

Sandbag piling for seawall was rather messy. Usually sandbags are piled alternately to get the strength, but it was observed that there were some locations that sandbags are piled in just one column.

Many sandbags piled for slope protection are torn apart or worn due to UV and waves.

However, they were made of mortar and relatively robust and no sandbags with cracks were observed during low tide inspection.

Sandbags were already piled in all necessary locations at the first field survey.

With regard to piling procedures, a manual was made by the consultant and technical transfer was conducted (see Figure 2.2.1-15).

Mortar Sandbag Manual

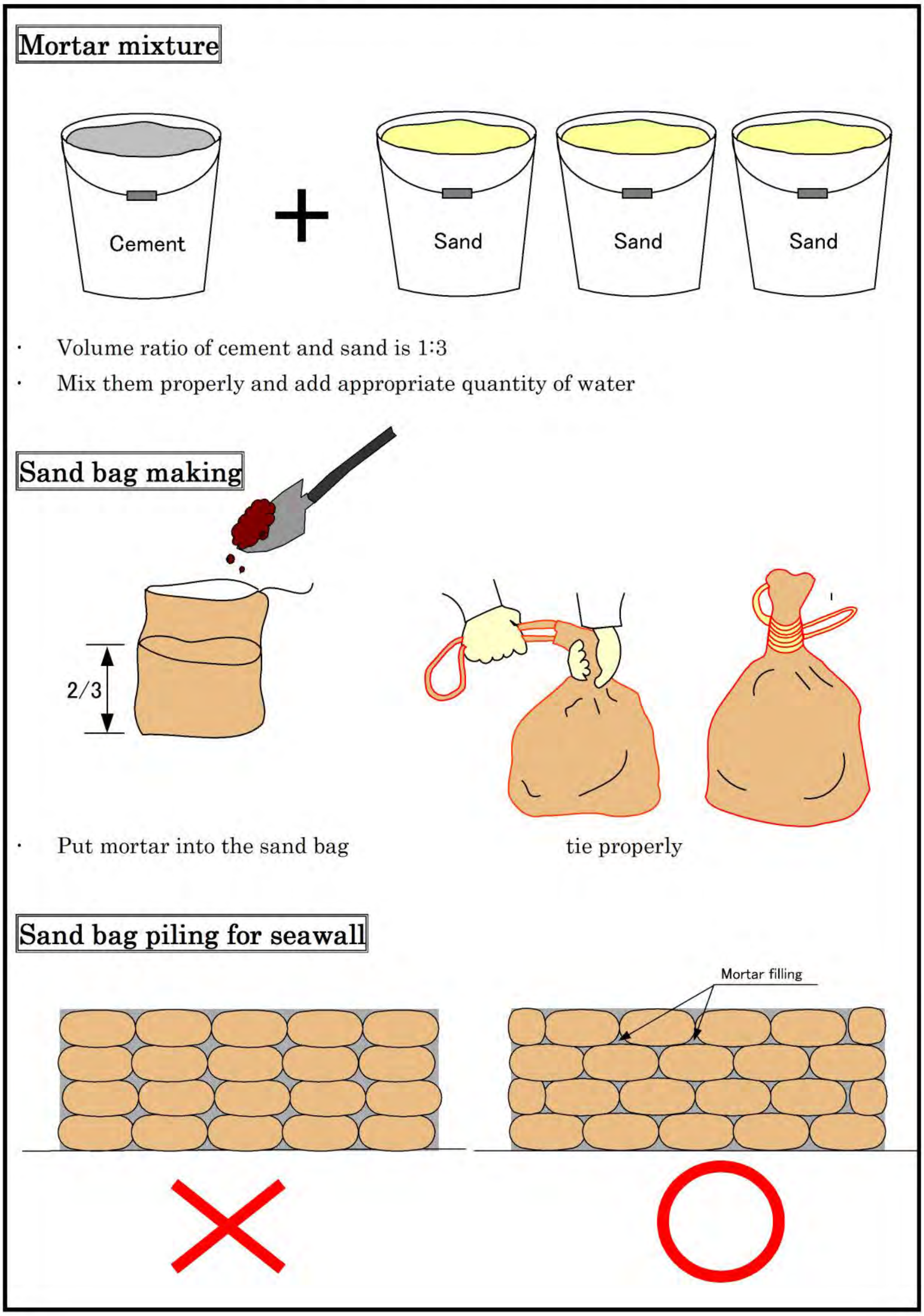


Figure 2.2.1-15 Mortal Sandbag Manual

(5) Corresponding policies for the waves during rough water

The damage of causeway was brought about by the overtopping seawater which overflows on the shoulder and penetrates the soil from the loose place and flows along with backfilled sand. Then the slope collapsed and the seawall is also destroyed by the repeating wave force.

Photos provided by Mr. Patrick

Necessary countermeasure to be required is that it can be applied until the start of causeway reconstruction begins and that it will not affect the reconstruction work. Moreover, it is important that it can be restored locally without using any special materials, technologies nor machines.

As a specific method, consider both proposals of reinforcement of seawalls and slope protection.

2.2.1.8 Points of the Project

The damage of the causeway such as revetment collapse has occurred due to King Tides and waves. However, rehabilitation of disaster places and the maintenance of the causeway such as pavement, revetment and parapet was not sufficiently conducted.

In the near future, it is considered that the soundness of the causeway will changed significantly before the detailed design or the construction of it starts. Therefore, the necessity of additional site survey such as topological survey or geological survey should be considered depending on the site situation at the time of the detailed design and the construction of the causeway.

In addition, the emergency measure of the revetment which is installation of the sandbags on the revetment slope to prevent the collapse of it has been performed since January 2016.

2.2.1.9 Technical Assistance for Emergency Countermeasures

The purpose of this technical assistance is to maintain the slope of the causeway and avoid collapse for road traffic.

5,500 pcs of large sandbags and 6,000m2 of geotextile sheets were provided by Japan. Labors and sand were provided by Kiribati.

Dispatch terms of Japanese supervisor are as follows;

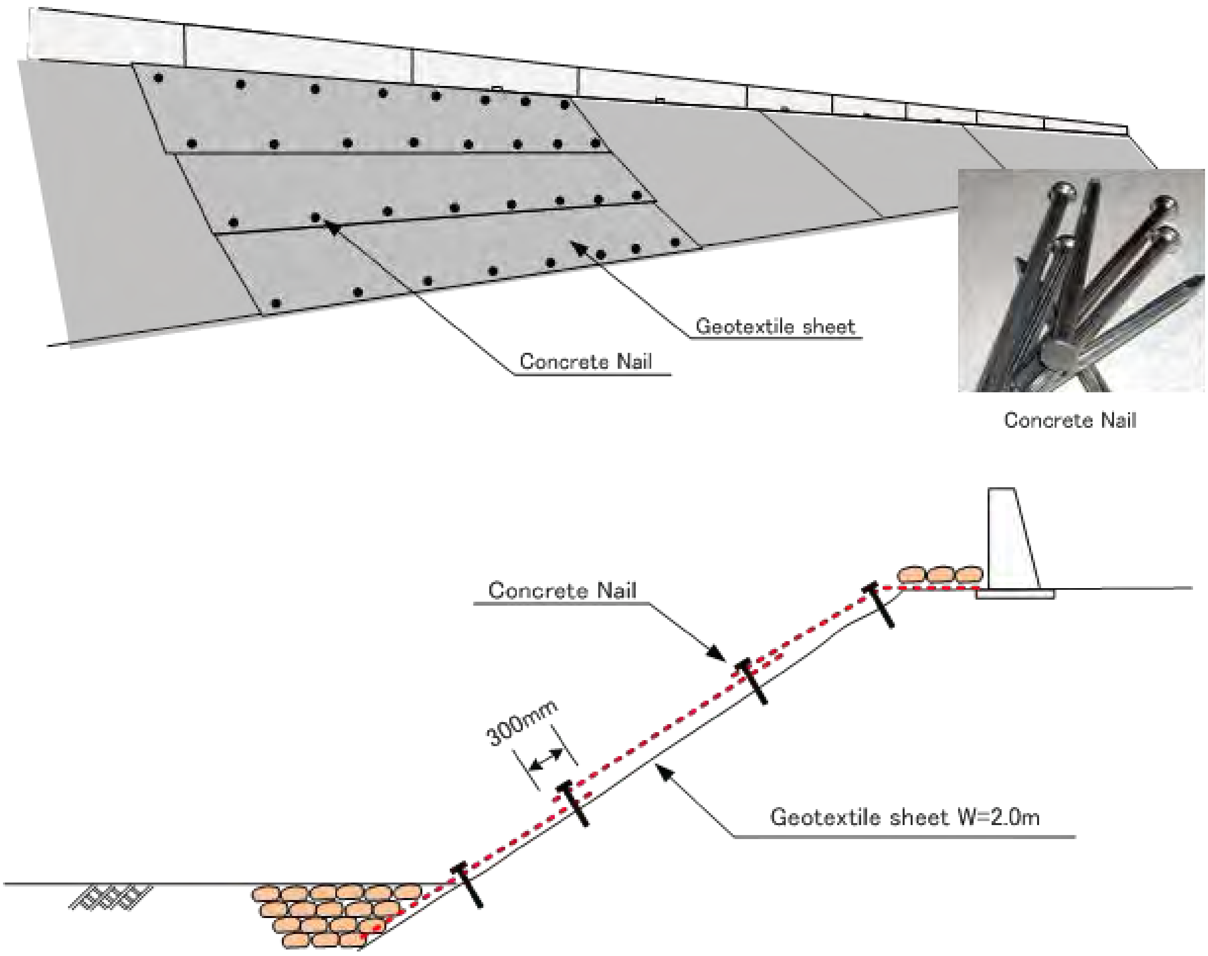
*First dispatch: 5 January to 3 February, 2016

*Second dispatch: 16 February to 11 March, 2016

*Third dispatch: 5 April to 9 May, 2016

(1) Construction method

1） Installation of Geotextile Sheet

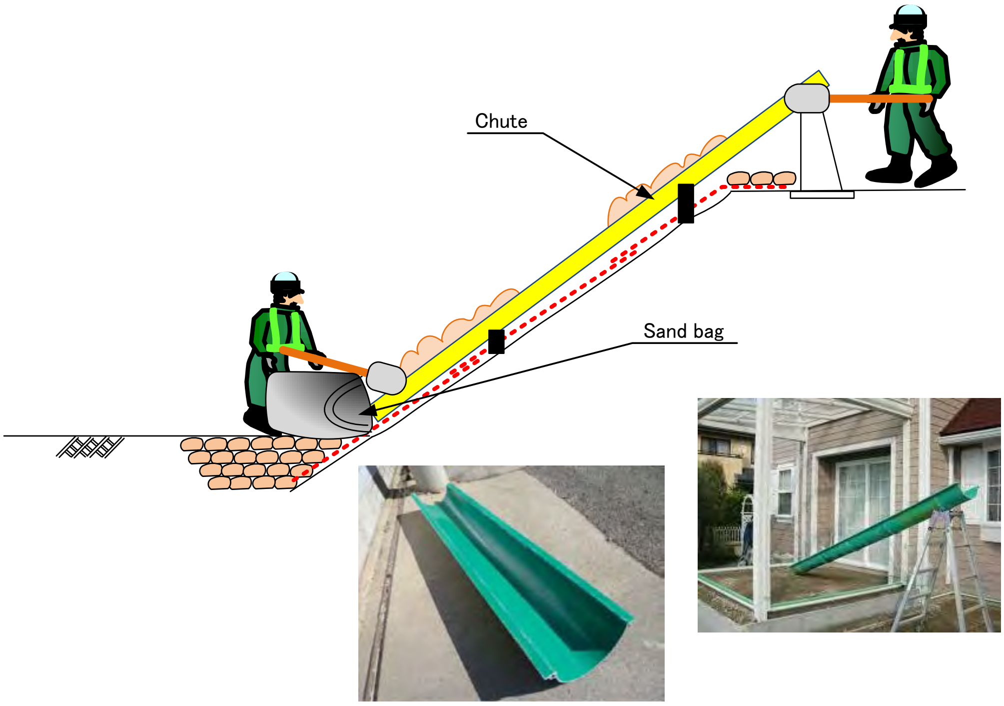


Installation of Geotextile sheet

The geotextile sheets shall be spread out manually and hold by concrete nails. The width of the sheet is 2 meters, hence it shall be overlapped.

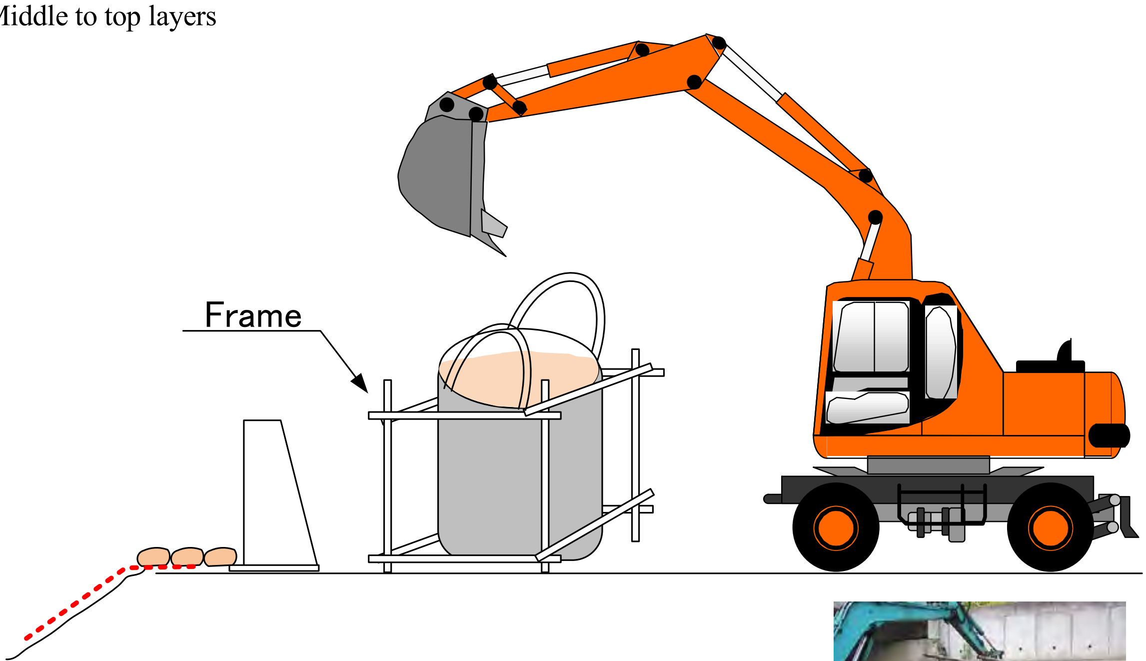
2） Installation of Sandbags

*Low to middle layers

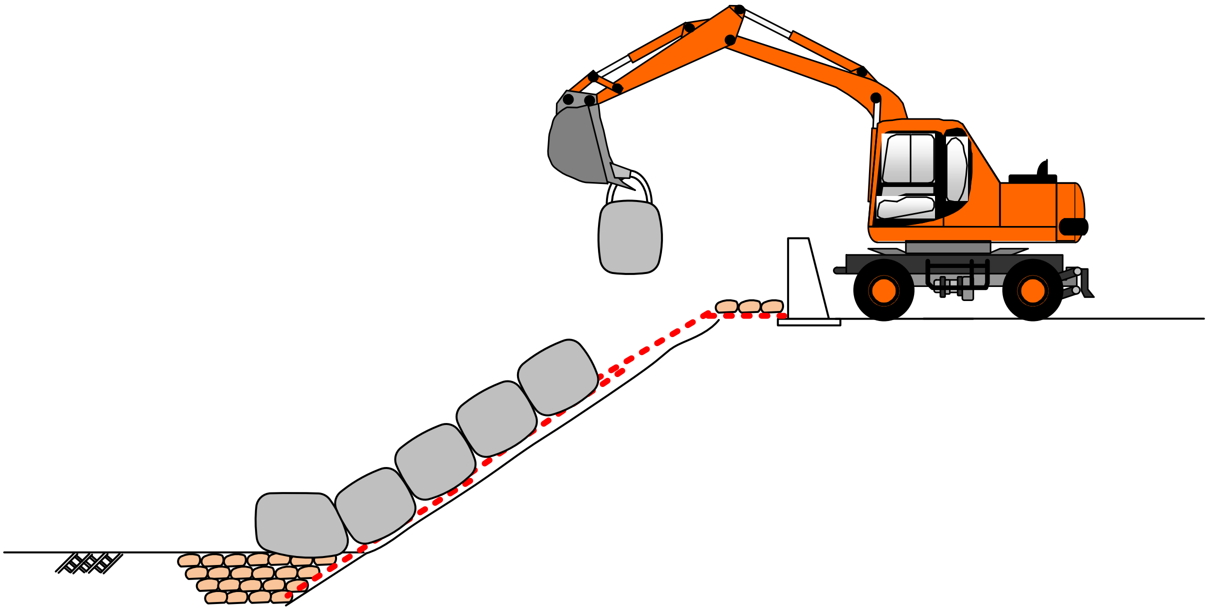


Fabrication and Installation of Sandbags

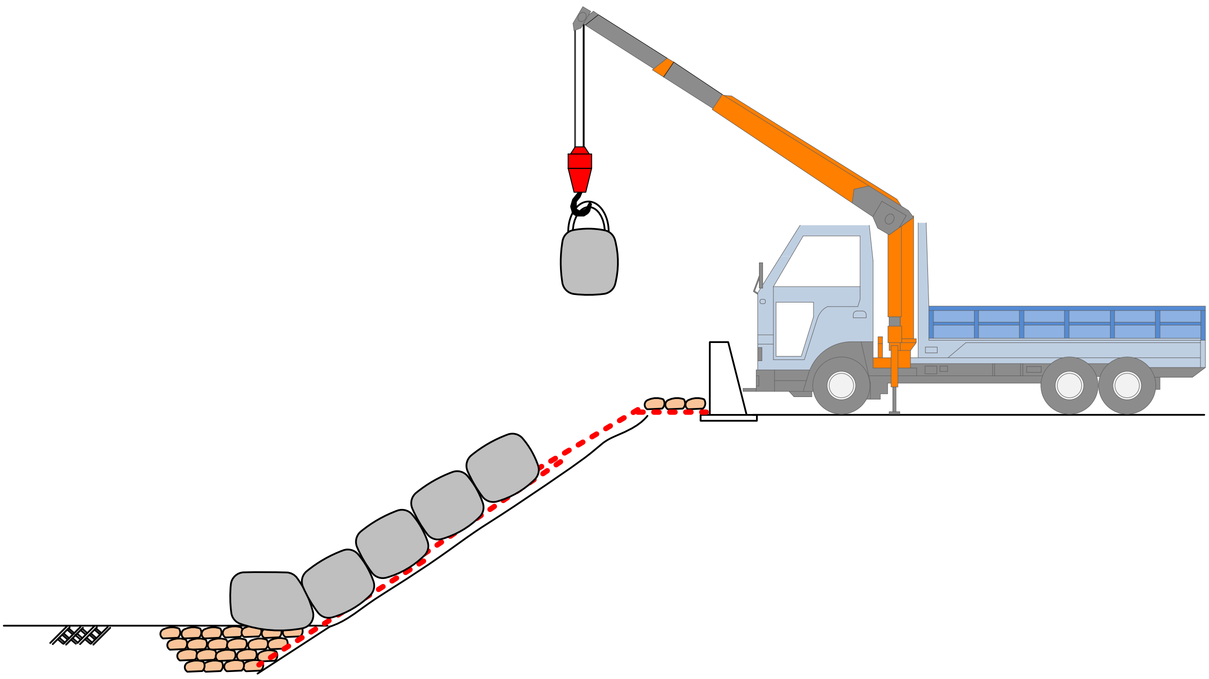
*Middle to top layers



* Installation by back hoe



*Installation by truck with crane



Fabrication and Installation of Sandbags

(2) Pictures


Materials


Installation of geotextile sheets


Installation of sandbags

2.2.2 Basic Plan

2.2.2.1 Applicable Standards

The standards which apply in the project are shown in Table 2.2.1-1.

Table 2.2.1-1 Applicable Standards

Category	Standards	Issue	Institution
Road Design	1) Commentary on Road Structure Ordinance	2015	Japan Road Association
	2) A Policy on Geometric Design of Highway and Streets (AASHTO)	2011	AASHTO
	3) Manual for Pavement Design	2006	Japan Road Association
	4) AASHTO for Design of Pavement Structures	2011	AASHTO
	5) Manual for Road Earthworks	2014	Japan Road Association
Revetment Design	1) Technical Standards and Commentaries for Port and Harbor in Japan	2009	The Overseas Coastal Area Development Institute of Japan
Bridge Design	1) Commentary on Road Structure Ordinance	2015	Japan Road Association
	2) Specification for Highway Bridges I ~ IV	2012	Japan Road Association
	3) Manual for Concrete Bridge Design	1994	Japan Road Association
	4) Manual for Concrete Bridge Construction	1998	Japan Road Association
	5) Guideline for Box Culverts	2010	Japan Road Association

2.2.2.2 Road Design

(1) Pavement Structure Design

1) Design Policy

Pavement structure design in the project is conducted based on Japan design standard (TA method) and by the use of AASHTO to check the validity of the result designed by Japan standard.

2) Design Traffic Volume

The design traffic volume is the estimated traffic volume in 2038, 20years after the completion of reconstruction of the causeway. The transition of traffic volume (All vehicles and large vehicles) is shown in Table 2.2.2-2. Based on the large vehicle volume in 2038, the design traffic volume was calculated as 227 vehicles/day.

Table 2.2.2-2 Transition of Traffic Volume in Kiribati

Year	Growth Rate	Traffic Volume (Vehicle/day)	Large Vehicle Volume (Vehicle/day)	Remarks
2015		3,894	300	
2016	0.03	4,011	309	
2017	0.03	4,131	318	
2018	0.03	4,255	328	
2019	0.03	4,383	338	1 year
2020	0.03	4,514	348	2 years
2021	0.03	4,650	358	3 years
2022	0.03	4,789	369	4 years
2023	0.03	4,933	380	5 years

Year	Growth Rate	Traffic Volume (Vehicle/day)	Large Vehicle Volume (Vehicle/day)	Remarks
2024	0.03	5,081	391	6 years
2025	0.03	5,233	403	7 years
2026	0.03	5,390	415	8 years
2027	0.03	5,552	428	9 years
2028	0.03	5,718	441	10 years
2029	0.03	5,890	454	11 years
2030	0.03	6,067	467	12 years
2031	0.03	6,249	481	13 years
2032	0.03	6,436	496	14 years
2033	0.03	6,629	511	15 years
2034	0.03	6,828	526	16 years
2035	0.03	7,033	542	17 years
2036	0.03	7,244	558	18 years
2037	0.03	7,461	575	19 years
2038	0.03	7,685	592	20 years

Resource: Study Team

3) Design Period

Design period is set as 20 years.

4) Design CBR

Based on the CBR test at the causeway's subgrade, CBR was 30.3-35.3% and average value was 32.5%, Therefore design CBR is applied as 20% (maximum CBR values for pavement design) for the pavement design structure.

5) Fatigue Fracture Wheel Load

Fatigue fracture wheel load is defined depending on the traffic class as shown in Table 2.2.2-3. Since the design traffic volume is 227 vehicles/day, traffic class is "N4" and the fatigue fracture wheel load is 150,000 times/10years.

Table 2.2.2-3 Traffic Class and Fatigue Fracture Wheel Load

Traffic Class	Design Traffic Volume (Vehicle/day*direction)	Fatigue Fracture Wheel Load (times/10years)
N7	Over 3,000	35,000,000
N6	1,000 ~ 3,000	7,000,000
N5	250 ~ 1,000	1,000,000
N4	100 ~ 250	150,000
N3	40 ~ 100	30,000
N2	15 ~ 40	7,000
N1	Less than 15	1,500

Resource: Japan Pavement Design Standard, P.30

6) TA Value

TA value is calculated by formula 3-1, and relationship of traffic class, design CBR and TA value based on formula 3-1 is shown in Table 2.2.2-4. From Table 2.2.2-4, TA value is 14.0.

$$T_A = \frac{3.84N^{0.16}}{CBR^{0.3}} \quad (3\text{-}1)$$

T_A : Necessary Equivalent Conversion Thickness

N : Fatigue Failure Wheel Times

CBR : Design CBR of Subgrade

Resource: Japan Pavement Design Standard, P.76

Table 2.2.2-4 Relationship of Traffic Class and Design CBR

Traffic Class	Design CBR					
	3	4	6	8	12	20
N7	50	46	41	38	33	29
N6	39	36	32	29	26	22
N5	29	26	23	21	19	16
N4	21	20	17	16	14	12
N3	17	15	14	12	11	10
N2	13	12	11	10	9	8
N1	10	10	9	8	7	6

Resource: Study Team

7) Design Condition of Pavement Thickness

Minimum thickness of each layer is shown in Table 2.2.2-5.

Table 2.2.2-5 Minimum Thickness of Each Layer

Layer	Minimum Thickness	Remarks
Asphalt (Surface)	50mm	
Bituminous Stabilization	50mm	2 times of maximum dimension of aggregate and over 5cm
Other Subbase Material	100mm	3 times of maximum dimension of aggregate and over 10cm

Resource: Japan Pavement Design Standard, P.77 ~ 78

8) Comparison of Pavement Structure

Bitumen, cement and aggregate for asphalt concrete or concrete slab are not available in Kiribati. These materials have to be imported from Fiji, and the pavement cost significantly affects the project cost. Therefore, the pavement cost of each case is compared as shown in Table 2.2.2-6, and the cheapest case is adopted as pavement structure in the project.

The comparison table of pavement structure is shown in Table 2.2.2-6. As a result of comparison, case-6 is adopted as the pavement structure in the project.

Table 2.2.2-6 Comparison Table of Pavement Structure

Material		Load Equivalency Factor	Unit Cost (JPY/m^3)	Pavement Structure (mm)				
				Case-1	Case-2	Case-3	Case-4	Case-5
Surface	Asphalt Concrete	1.00	300,000	50	50	50	50	50
Upper Subbase	Bituminous Stabilization	0.80	112,000	50	100			
	Stabilization with Cement	0.55	90,000			50	100	
	Mechanical Stabilized Base*	0.20	28,000					150
Lower Subbase	Crusher-run*	0.20	28,000	150	100	250	100	200
T_A Value (≧14)				12.00	15.00	12.75	12.50	12.00
Total Thickness (mm)				250	250	350	250	400
Cost (JPY/m)				27,600	29,000	29,600	32,700	24,800
Evaluation				△	△	△	△	◎

*Upper subbase and lower subbase plan to be used as the coral rock. Load equivalency factor is set as more than 20 to less than 30. (CBR of the coral rock was confirmed as more than 20 by CBR test of the causeway in the project.)

9) Pavement Structure by AASHTO

In this part, the validity of pavement structure by TA method is verified by AASHTO. The basic formula of AASHTO for pavement design is shown in 3-2, and the design condition and result are shown in Table 2.2.2-7 ~ Table 2.2.2-9.

As a result, it was verified that the pavement structure designed by the TA method has met the designed pavement thickness by AASHTO.

$$\log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

$$\log_{10}(W_{18}) = Z_R \times S_0 + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624\times10^{7}}{(D+1)^{8.46}}} + (4.22 - 0.32 \times P_t) \times \log_{10}\left[\frac{S'_c \times C_d \times (D^{0.75}-1.132)}{215.63 \times J\left[D^{0.75}-\frac{18.42}{\left(\frac{E_c}{k}\right)^{0.25}}\right]}\right] \quad (3\text{-}2)$$

W_{18} : Predicted number of 18-kip equivalent single axle load application

Z_R : Standard normal deviation

S_0 : Combined standard error of traffic prediction and performance prediction

ΔPSI : Difference between the initial design serviceability index, P0, and the design terminal serviceability index, Pt

M_R : Resilient coefficient (psi)

SN : Structural Number

Resource: AASHTO Guide for Design of Pavement Structure p.I-5

Table 2.2.2-7 Axle Load

Design Condition		Car	Small Bus	Large Bus	2-Axle Truck	3- or 4-Axle Truck	5- or 6-Axle Truck
Traffic Volume		1,946	705	163	535	116	21
Average Axle Load (kip)		1.00	1.00	6.51	6.51	22.20	40.20
Load Equivalency Factor	1st Axle	0.0002	0.0002	0.0031	0.0031	0.0610	0.7624
	2nd Axle	0.0002	0.0002	0.0017	0.0017	0.1678	0.2774
	3rd Axle	-	-	-	-	0.0723	0.4114
	Total	0.0004	0.0004	0.0048	0.0048	0.3010	1.4512
Design ESAL		8,592	3,113	8,550	28,064	385,425	336,405
Total ESAL		770,149					

Table 2.2.2-8 Design Condition

Parameter	Value	Remarks
W_{18}	433,744	
Design Period	20 years	
Z_R	-1.282	AASHTO Guide for Design of Pavement Structure p.I-62
S_0	0.45	AASHTO Guide for Design of Pavement Structure p.I-62
⊿PSI	1.7	AASHTO Guide for Design of Pavement Structure p.II-10
M_R	18,000	AASHTO Guide for Design of Pavement Structure p.I-14
SN	2.376	

Table 2.2.2-9 Result of Pavement Thickness designed by AASHTO

Material	Layer Coefficient (a)	Thickness (cm) (D)	Drainage Coefficient (m)	Structural Number SN = a*D*m
Asphalt (Surface)	0.400	5	―	0.787
Upper Subbase	0.150	15	1.0	0.886
Lower Subbase	0.090	20	1.0	0.709
Total Thickness	-	40	-	2.382 (≧2.376)

(2) Road Drainage Design

1) Design Policy

Concept for the drainage design is shown in the following items;

- Transverse drainage is installed at the bottom of the revetment and road drainage is installed edge of shoulder/footpath are designed,
- The road drainage size is designed based on rational runoff formula (Japan standard),
- And transverse drainage is designed wider than the existing drainage interval (5m) to prevent backflow of seawater by wave.

2) Design for Transverse Drainage

Transverse drainage size is H=30cm W=50cm, installation interval is 50m.

Schematic view of the transverse drainage is shown in Figure 2.2.2-1.

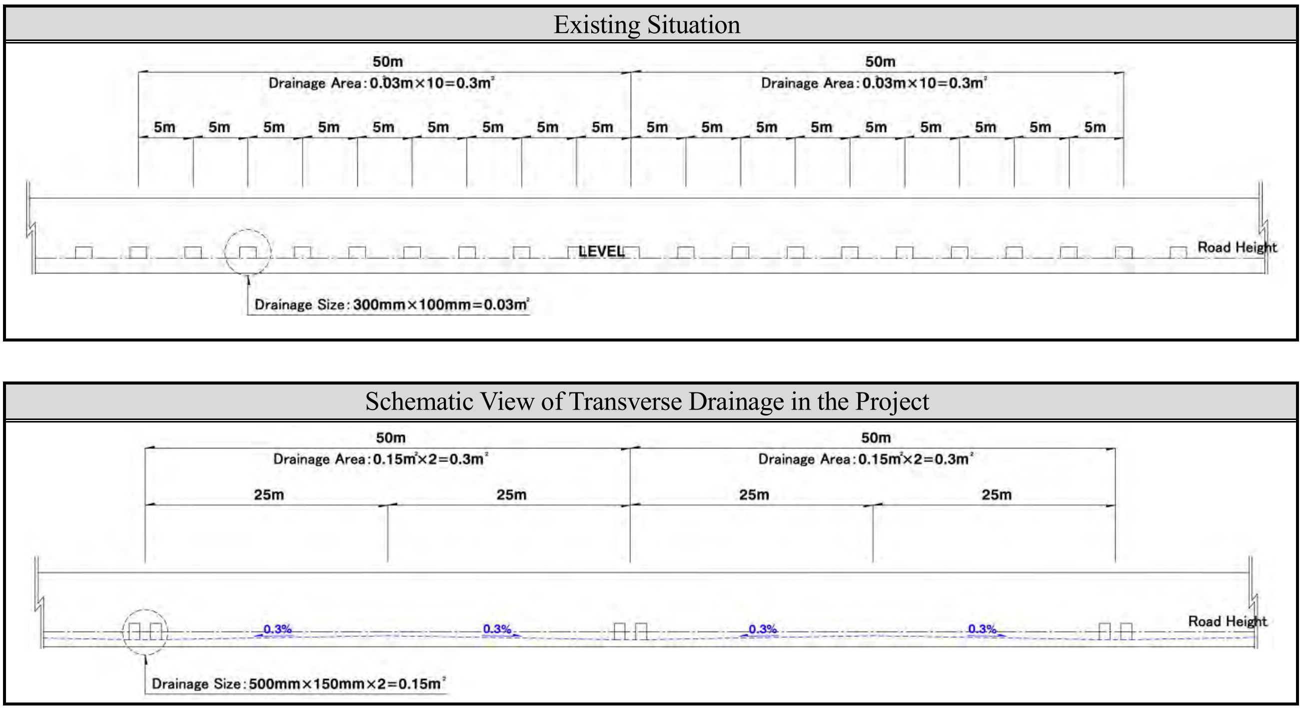


Figure 2.2.2-1 Size and Installation Interval of the Transverse Drainage

3) Design for Road Drainage

Road drainage size is designed so that rainfall can be drained. Run-off is calculated by using formula 3-2, and calculation conditions are shown in Table 2.2.2-10~ Table 2.2.2-12.

$$Q = \frac{1}{3.6} \cdot C \cdot I \cdot A \qquad (3-2)$$

Q : Run-off (m^3/s)
C : Run-off Factor
I : Intensity of Rainfall (mm/h)
A : Catchment Area (km^2)
Resource: Manual for Road Earthworks P.135

Table 2.2.2-10 Reoccurrence Period of Rainfall

Category	Drainage Capacity	Reoccurrence Period of Rainfall	
		(*1)	(*2)
A	High		Over 10 years
B	Middle	3 years	7 years
C	Low		5 years

*1: It is applied as general road drainage such as road surface or short slope.
*2: It is applied to the urban area where is difficult to drain the water and important facilities.
Resource: Manual for Road Earthworks, P.112

Table 2.2.2-11　Run-off Factor

Ground Surface		Run-off Factor
Road	Paved	0.70~0.95
	Unpaved	0.30~0.70

Resource: Manual for Pavement Design , P.134

Table 2.2.2-12　Calculation Condition for Intensity of Rainfall

Parameter		Remarks
Catchment Area	$275m^2$	50m×5.5m
Amount of Rainfall	150mm/day	3 years : 2014/12/31
Intensity of Rainfall (I_n)	37.5mm/h	$I_n=R_n*\beta_n$
60 minutes Intensity of Rainfall (R_n)	6.25	
Characterization Factor of Reoccurrence Period (β_n)	6.0	

The result of run-off calculation is shown in Table 2.2.2-13, and the road drainage shape is shown in Figure 2.2.2-2.

Table 2.2.2-13　Calculation Result

Parameter		Remarks
Coefficient of Roughness	0.015	Manual for Road Earthworks P.137
Gradient	0.3%	
Sectional Area	$0.011m^2$	Refer to figure 3.1-7
Hydraulic Radius	0.34m	Refer to figure 3.1-7
Velocity of Flow	1.78m/s	
Allowable Flow Volume	$0.016m^3/s$	($\geqq 0.0013m^3/s$)

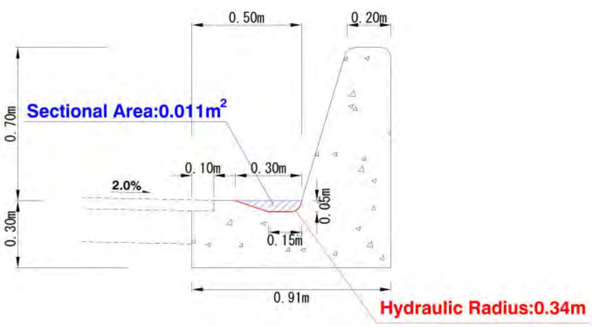


Figure 2.2.2-2　Road Drainage Shape

(3)　Utilities Relocation Plan

Relocated utilities includes a high-voltage (11KV) electrical power line. Based on the Japan electrical equipment technical standard, arrangement of utilities is planned that the space of high-voltage electrical power line, water pipeline and communication cable is over 30cm secured. Between high-voltage electrical power line and water pipeline or communication cable has to install concrete wall.

Detail of utility box is shown in Figure 2.2.2-3, and the result of structural calculation for vehicle collision is shown in Table 2.2.2-14.

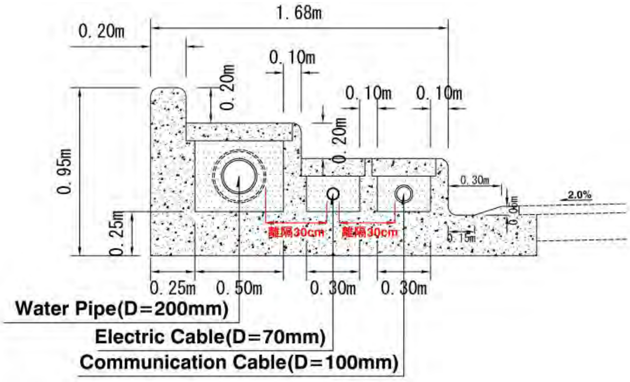
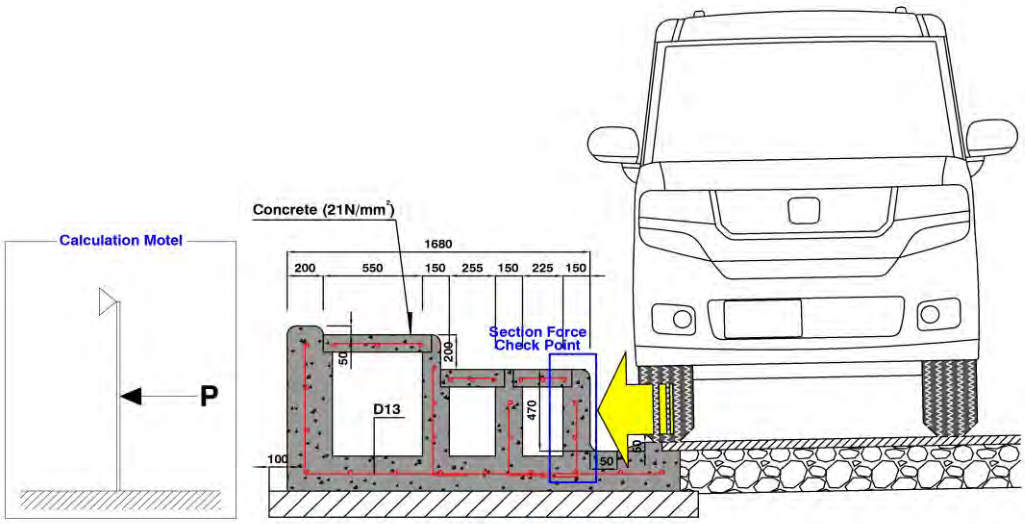


Figure 2.2.2-3　Detail of Utility Box

Table 2.2.2-14　Structural Calculation for Vehicle Collision

Items		Remarks
Collision Load	43kN	Specification for Vehicle Gurde Fence in Japan, P115
Effective Depth (d)	120mm	
Cross Sectional Width of Beam (b)	1000mm	
Cross Sectional Area of Tension Reinforcement (As)	506.8mm^2	D13@250
Reinfocement Ratio (P)	0.00442	
Design Strength of Concrete (f'ck)	21N/mm^2	(≧0.0013m^3/s)
Allowable Tensile Stress due to Bending (σ ca)	7 N/mm^2	
Tensile Stress of Reinforcement (σ sa)	157 N/mm^2	SD295
Moment at the time of Collision	3.16 kN・m	
Compression Resisting Momont (Mrc)	13.53 kN・m	≧3.16 kN・m
Tension Resisting Moment (Mrs)	8.60 kN・m	≧3.16 kN・m



2.2.2.3 Revetment Design

(1) Selection of the revetment strengthening Measures

In this project, the strengthening measures for revetment used by vanishing wave block and covered stone are excluded based on the following reasons;

- ➢ The design wave height do not require the wave dissipating block for revetment.
- ➢ The armor stone is commonly used for wave dissipation in case of relatively small design wave height. However, armor stone should be imported due to none local availability, and also a large stone size is not easily procured even in Fiji of an economical import country.
- ➢ A filter layer of rock should be required under wave dissipating blocks or armor stone. It should be imported as well which leads to another cost increase factor.

Instead of wave dissipating block and armor stone, a sand bag (mat) and a fabrimat (or equivalent), which have several local experiences and advantage of ease of maintenance, are considered as alternatives. As filling materials, the sand bag should contain mortar, and the fabrimat should contain concrete.

Alternatives potentially applicable to the project are shown in Table 2.2.2-15 (1) - (3). The following alternatives are selected as applicable measures.

【Option-1 (Alternative-1)】: Present Slope Maintained
【Option-2 (Alternative-3)】: Overlaid with Fabrimat
【Option-3 (Alternative-5)】: Foot Protection Steel Sheet Pile

Option1～Option3 are used depending on the damage level of the causeway. Applicable condition of each option is shown below.

【Option-1: Present Slope Maintained】

- ➢ Scour and subsidence of slope foundation are not confirmed.
- ➢ Shear of the revetment is not confirmed.
- ➢ Cracks width and area of the revetment are small.

【Option-2: Overlaid with Fabrimat】

- ➢ Scour and subsidence of slope foundation are small or not confirmed.
- ➢ Shear of the revetment is small or not confirmed.
- ➢ Cracks width and area of the revetment are large relatively. (Overlaid thickness is selected 20cm or 25cm depending on crack condition.)

【Option-3: Foot Protection Steel Sheet Pile】

- ➢ Scour and subsidence of slope foundation are large.
- ➢ It has been affected by disaster in the past.
- ➢ Shear of the revetment is large.

- Cracks width and area of the revetment are large.

Table 2.2.2-15　REVETMENT ALTERNATIVES (1)

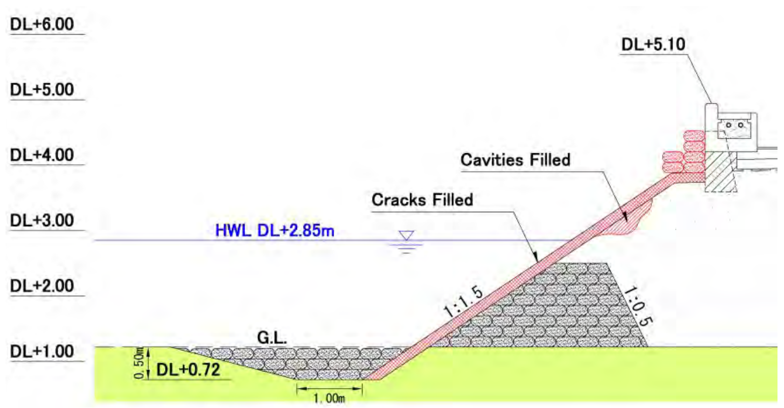
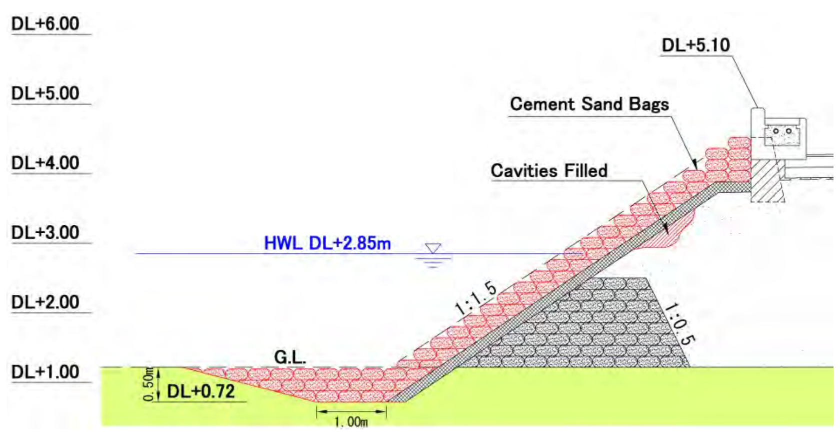
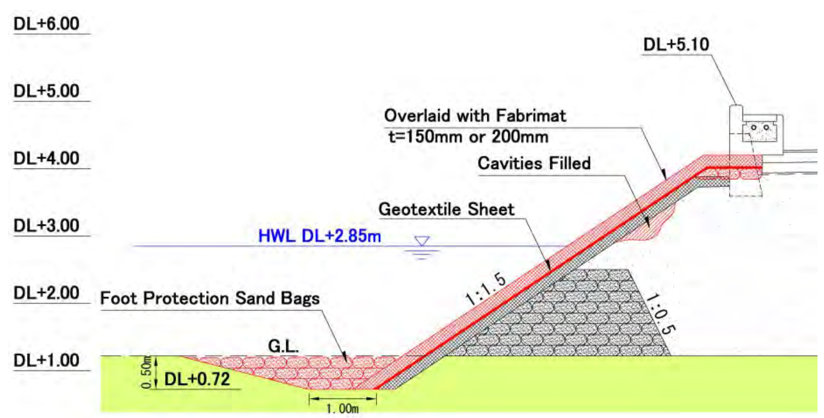
Alternative	【ALT①】: Present Slope Maintained	【ALT②】: Overlaid with Sand Bags	【ALT③】: Overlaid with Fabrimat
Conceptual Sketch			
Abstract	The present slope is maintained. The identified cracks and cavities under the slope should be filled. A repair of the slope covered with the accumulated sand is not required. New parapet wall is installed. The height will be determined with overtopping rate. The height of the road should be raised.	The sand mats are overlaid on the present fabrimat slope. The identified cracks and cavities under the slope should be filled. The existing foot protection will be removed, and covered with sand mats. New parapet wall is installed. The height will be determined with overtopping rate. The height of the road should be raised.	New fabrimat is overlaid on the present fabrimat slope. The thickness of the new mat (15 cm or 20 cm) will be determined through comprehensive review. The identified cracks and cavities under the slope should be filled. The existing foot protection will be removed, and covered with sand mats. New parapet wall is installed. The height will be determined with overtopping rate. The height of the road should be raised.
Cost	The lowest cost as rehabilitation of the present slope. All works can be carried out only with local materials in case of cavity filling by mortar.	The cost of installation of the sand mats and the foot protection added to ALT①. Although a large quantities of sand mats is required, all works can be carried out only with local materials in case of cavity filling by mortar.	The cost of installation of the fabrimat and the foot protection added to ALT①.
Durability	The remained life period becomes unknown as the present slope being kept, the durability is lessor than that of new slope. Since the potential risk of crack remained same as the present slope, the damage to the road cannot completely be prevented. A maintenance should be essential to keep the durability.	The durability is enhanced with coverage of new sand mats. The long durability of the bag is not taken into account, the durability should be secured by the strength of the mat. The damage to the road can be prevented by the present mat as a protection layer against sand suction even if the crack is generated on the new sand mat. A maintenance of the new sand mat should be required.	The durability is enhanced with coverage of new fabrimat. The higher durability is secured with thicker fabrimat. The damage to the road can be prevented by the present mat as a protection layer against sand suction even if the crack is generated on the new fabrimat. A maintenance of the new fabrimat should be required.
Workability	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability except for cavity filling as the experienced works in Kiribati. Difficult determination of quantities and identification of stoppage for the cavity filling.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability except for cavity filling as the experienced works in Kiribati, but it takes time if the mat is placed by man-power. Difficult determination of quantities and identification of stoppage for the cavity filling. The productivity of the foot protection work becomes lower in case of underwater.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability except for cavity filling as the experienced works in Kiribati. Difficult determination of quantities and identification of stoppage for the cavity filling. The productivity of the foot protection work becomes lower in case of underwater.
Sustainability of Maintenance	Continuous maintenance should be managed as the present slope remained. Systematic process and organization should be secured to keep reliable maintenance.	Ease maintenance because of locally experienced structure and new slope. A continuous maintenance should be organized and managed against cavity and deterioration of the sand mats.	Ease maintenance because of new slope. A continuous maintenance should be organized and managed against cracks.
Environmental Social Considerations	No issues as far as the planned section	No issues as far as the planned section	No issues as far as the planned section
Others	Shortest construction period	Relatively short construction period. Applicable for the medium term rehabilitation only with local technology in case of no cavity.	Relatively short construction period
Application	Applicable for the sections of no damaged and no potential risk for damage	Applicable for the damaged sections and small numbers of cracks and cavities	Applicable for the damaged sections and small numbers of cracks and cavities

Table 2.2.2-15 REVETMENT ALTERNATIVES (2)

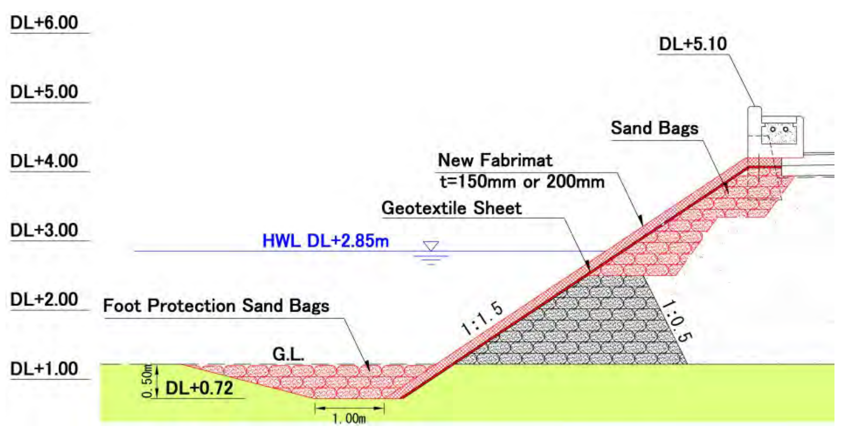
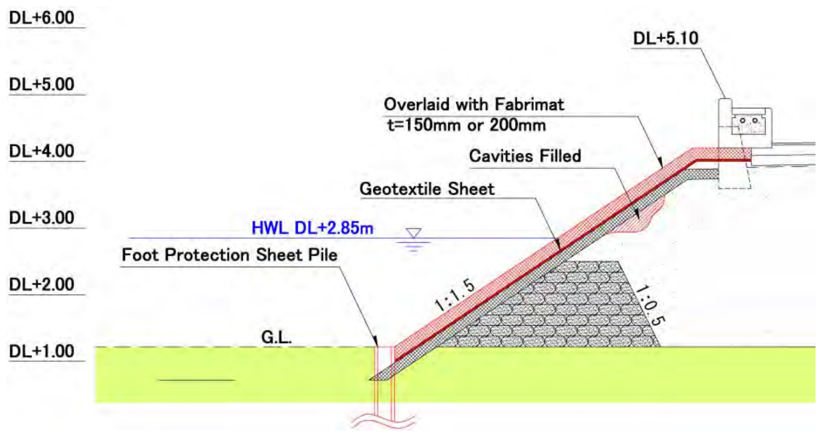
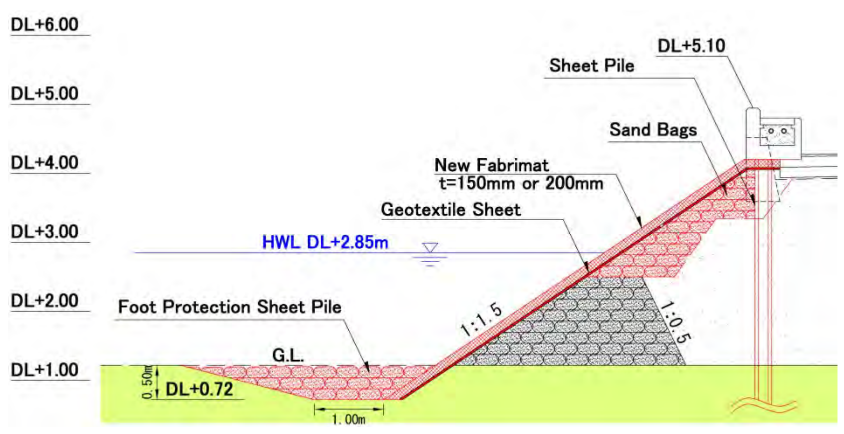
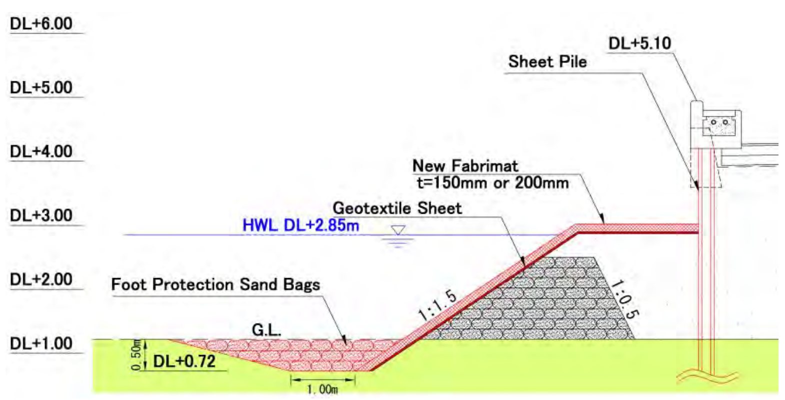
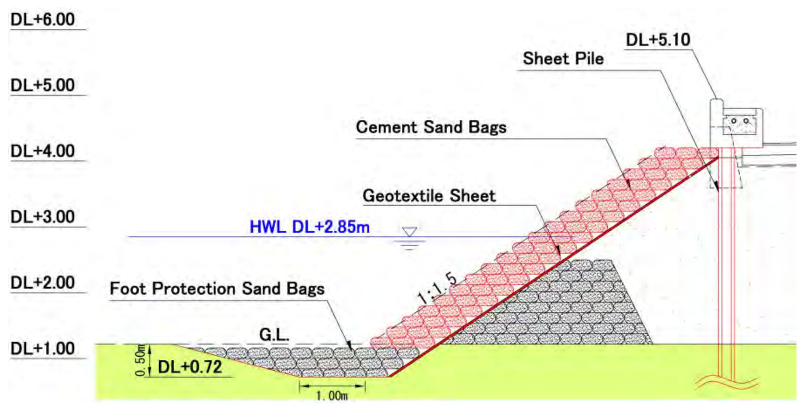
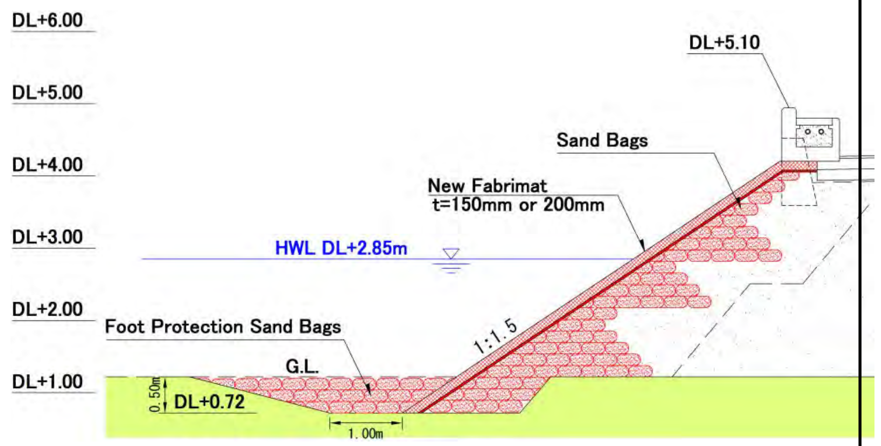
Alternative	【ALT④】: New Fabrimat	【ALT⑤】: Foot Protection Sheet Pile	【ALT⑥】: Sheet Piled Separation Wall（New Slope） or （Present Slope Maintained）
Conceptual Sketch			
Abstract	New fabrimat is laid after removal of the present fabrimat. The thickness of the new mat (15 cm or 20 cm) will be determined through comprehensive review. The existing slope should be excavated for new fabrimat slope. Geotextile sheet should be installed under the fabrimat to prevent sand suction. The existing foot protection will be removed, and covered with sand mats. New parapet wall is installed. The height will be determined with overtopping rate. The height of the road should be raised.	The sheet pile is driven at the slope toe to prevent scoring and sand suction. The slope should be maintained or overlaid by the fabrimat, if required. Possible as the alternative of the foot protection in ALT③ Overlaid with Fabrimat.	The sheet pile is driven at the slope shoulder to prevent sand suction. The heavily damaged upper slope should be removed and re-build with new fabrimat. The present slope can be maintained where the upper slope is covered with accumulated sand (as ALT⑥'). The existing sand bags and filling will be re-used as much as possible, since the slope angle remains unchanged to apply the fabrimat with 15cm thickness for protection of the sheet pile.
Cost	The cost of removal and installation of the fabrimats and the foot protection added to ALT①, although no repair cost on the present slope is required.	Additional resources are required for driving the sheet pile. In case the present slope maintained, the cost becomes as the same level of ALT③, in case of the overlaid slope, the same level of ALT④.	The cost of removal and installation of the fabrimats and the foot protection added to ALT①, although no repair cost on the present slope is required. (Only driving cost of the sheet pile added in case of ALT⑥') Additional resources are required for driving the sheet pile.
Durability	The durability is enhanced with coverage of new fabrimat. The higher durability is secured with thicker fabrimat. The damage to the road can be prevented by the geotextile sheet against sand suction even if the crack is generated on the new fabrimat. A maintenance of the new fabrimat should be required.	In case the present slope maintained, the durability becomes as the same level of ALT ①, in case of the overlaid slope, the same level of ALT③. The durability of the foot protection becomes higher with the sheet pile. A maintenance of the new fabrimat should be required. The sheet pile requires a corrosion protection, but corrosion allowance may be sufficient because of underwater.	The durability is enhanced with coverage of new fabrimat. The damage to the road can be almost completely prevented by the sheet pile wall, and geotextile sheet functions against sand suction even if the crack is generated on the new fabrimat. A normal maintenance of the new fabrimat should be required. The sheet pile requires a corrosion protection, but corrosion allowance may be sufficient because of underwater.
Workability	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability as the experienced works in Kiribati. The productivity of the foot protection work becomes lower in case of underwater.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The driving of the sheet pile into coral ground is possible, but not good productivity.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability as the experienced works in Kiribati. The productivity of the foot protection work becomes lower in case of underwater. The driving of the sheet pile into coral ground is possible, but not good productivity.
Sustainability of Maintenance	Ease maintenance because of new slope. Continuous maintenance should be organized and managed against cracks.	In case the present slope maintained, the necessity of the continuous maintenance is the same as required for ALT①. In case of the overlaid slope, ease maintenance the same level of ALT③. A continuous maintenance should be organized and managed against cracks.	Ease maintenance because of new slope. A continuous maintenance should be organized and managed against cracks, but necessity and urgency become lower than those in case without the sheet pile.
Environmental Social Considerations	No issues as far as the planned section	No issues as far as the planned section	No issues as far as the planned section
Others	Relatively long construction period as removal and newly installation. Re-use or recycling should be considered for removed materials in large quantity.	Long construction period due to sheet pile driving included.	Long construction period as removal and newly installation of the slope and driving of the sheet pile. Re-use or recycling should be considered for removed materials in large quantity.
Application	Applicable for the damaged sections and large numbers of cracks	Applicable for the sections at bridge side where the existing sheet pile driven. Applicable as an alternative for ALT③	Applicable for the sections of mostly damaged and high risk of damage

Table 2.2.2-15 REVEMENT ALTERNATIVES (3)

Alternative	【ALT⑦】: Sheet Pile Wall（Partly Slope Removal）	【ALT⑧】: Sheet Pile Wall （Slope Removal Covered with Sand Bags）	【ALT⑨】: New Fabrimat（Replacement at Slope Toe）
Conceptual Sketch			
Abstract	The sheet pile (type IV) is driven at the slope shoulder. The present slope above the existing sand bags should be removed and covered with new fabrimat on the top surface. The existing foot protection will be removed, and covered with sand mats. New parapet wall both as superstructure is installed. The height will be determined with overtopping rate. The height of the road should be raised.	The sheet pile (type VL) is driven at the slope shoulder. The present slope above the reef should be removed, but remained subject to the condition of accumulated sand. The existing foot protection will be removed, and covered with sand mats. New parapet wall both as superstructure is installed. The height will be determined with overtopping rate. The height of the road should be raised.	The present fabrimat, foot protection, loose soil in the slope and the reef should be removed. After replacement of the loose soil, new slope will be rebuilt with sand bags and filling, and new fabrimat will be installed. The thickness of the new mat (15 cm or 20 cm) will be determined through comprehensive review. Geotextile sheet should be installed under the fabrimat to prevent sand suction. The foot protection will be covered with sand mats. New parapet wall is installed. The height will be determined with overtopping rate. The height of the road should be raised.
Cost	The cost of upper slope rebuilding is reduced from the cost of ALT③, however increased by the weight of heavier sheet pile. Additional resources are required for driving the sheet pile.	The cost of slope rebuilding is not required comparing the cost of ALT③, however increased by the cost of new sand bags, removal, and heavier sheet pile. Additional resources are required for driving the sheet pile.	The cost of removal and installation of the fabrimats and the foot protection added to ALT①, although no repair cost on the present slope is required.
Durability	The durability is enhanced with coverage of new fabrimat. The damage to the road can be almost completely prevented by the sheet pile wall. Normal maintenance of the new fabrimat should be required. Because the sheet pile requires a corrosion protection, covering or coating method should be adopted.	The damage to the road can be almost completely prevented by the sheet pile wall. Maintenance of the new sand bags should be required.	The durability is enhanced with coverage of new fabrimat. The higher durability is secured with thicker fabrimat. The damage to the road could be prevented by the geotextile sheet against sand suction, should the crack be generated on the new fabrimat. A maintenance of the new fabrimat should be required.
Workability	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability as the experienced works in Kiribati. The productivity of the foot protection work becomes lower in case of underwater. The driving of the sheet pile into coral ground is possible, but not good productivity. Protection of coating/ covering to the sheet pile is required during the excavation in front.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The driving of the sheet pile into coral ground is possible, but not good productivity.	The rehabilitation works of the slope will be able to proceed irrespective of the road works. The good workability as the experienced works in Kiribati. The productivity of the foot protection work becomes lower in case of underwater.
Sustainability of Maintenance	Ease maintenance because of new slope. Continuous maintenance should be organized and managed against cracks, but necessity and urgency become lower than those in case without the sheet pile. Corrosion inspection of the exposed surface of the sheet pile is required.	Maintenance is required only for sand bags.	Ease maintenance because of new slope. A continuous maintenance should be organized and managed against cracks.
Environmental Social Considerations	No issues as far as the planned section	No issues as far as the planned section	No issues as far as the planned section
Others	Long construction period because of removal and newly installation of the slope, and driving of the sheet pile. Re-use or recycling should be considered for removed materials in large quantity. Overtopping rate is reduced compared with the all slope.	Long construction period as removal of the slope, and driving of the sheet pile. Re-use or recycling should be considered for removed materials in large quantity. Overtopping rate is minimized among alternatives.	Relatively long as removal and newly installation. Re-use or recycling should be considered for removed materials in large quantity.
Application	Applicable for the sections of mostly damaged and high risk of damage	Applicable for the sections of mostly damaged and high risk of damage	Applicable for the sections at loose ground and caused deformation.

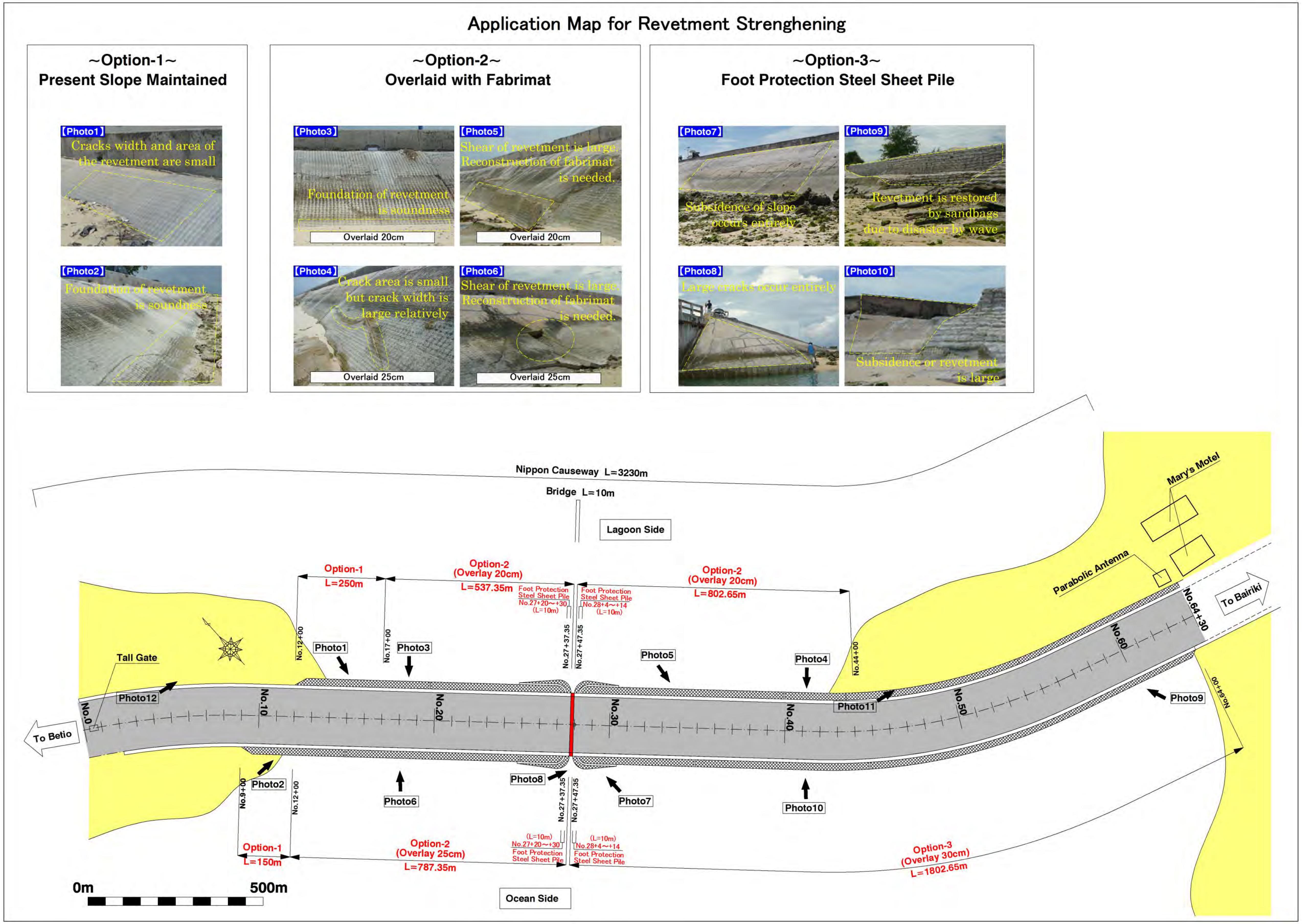


Figure 2.2.2-4 Application Map for Revetment Strenghening

(2) Revetment Design

1) Parapet

a) Conditions

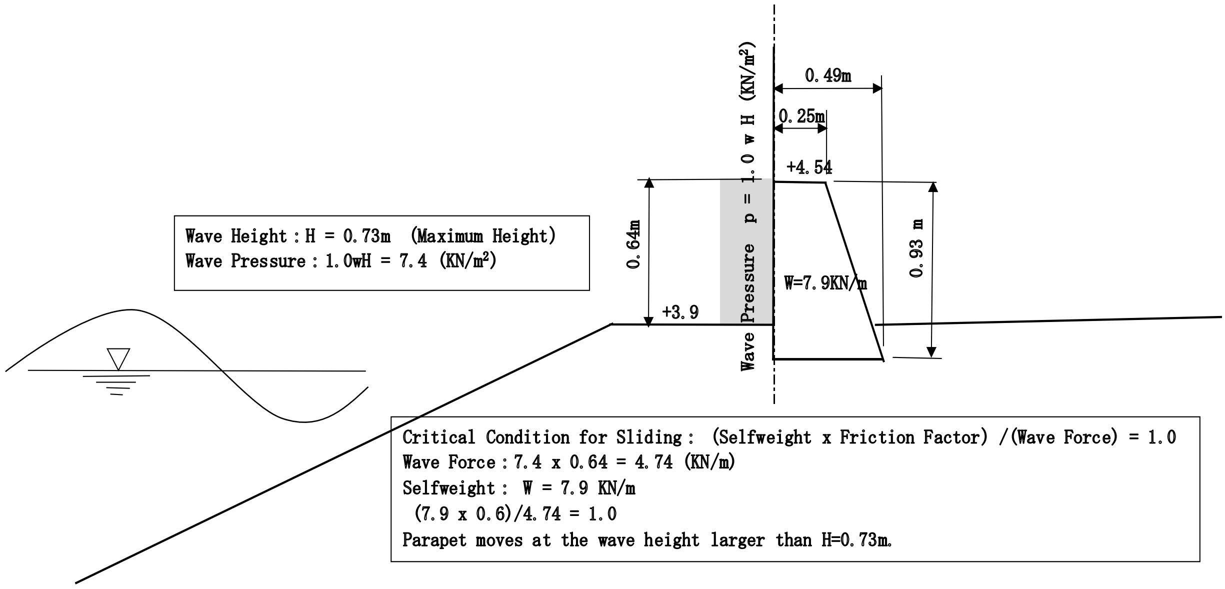
The existing parapet had moved by wave actions to cause increase of the overtopping and expansion of the damage to the road. Therefore, the parapet is revised from the view points of the following considerations:

- To be stable structure against wave actions
- Applied wave height and wave pressure are determined according to the actual sliding conditions of the existing parapet.
- Water pressure of the maximum wave height is taken into account.
- The angle of incident is taken into account for lagoon side, as the western wave from the opening of the lagoon is prevailing.
- Difference in the actual damage of revetment between Bairiki and Betio ocean sides is taken into account.
- The embedded type structure is adopted to enhance sliding resistance, and passive earth pressure of road side is taken into account.

The wave height: H=0.73m (the maximum wave height) at the revetment was obtained from Figure 2.2.2-5 as the wave height causing the sliding of the parapet with a safety factor of 1.0. A wave pressure: p=1.09woH at the water level was estimated under the conditions of the revetment with the Goda formula which can be applied for the vertical wall. Since the wave pressure at the parapet on the slope shoulder become smaller than that at the water level, the wave pressure of P=1.0woH along the total height of the parapet was presumed. The dimensions of the existing parapet were assumed as the top and bottom width of the parapet were not clarified in the original design.

For the parapet located at the Betio ocean side and the entire lagoon side, the wave larger than H=0.73m was not experienced as seen the fact that almost all parapet did not move. The significant wave height becomes H1/3=0.73/1.8=0.4m that is smaller than the wave height in the original design: 0.7m (ocean side) and 0.68m(lagoon side). However, it seems likely because there are a shallow reef spreading at the Betio ocean side, and the long distance from the entrance of the atoll to the causeway.

On the other hand, a plenty numbers of the parapet at the Bairiki ocean side had moved or falled down. The wave height larger than H=0.73m should be experienced, but the wave height could not be defined.



Source: The Study Team

Figure 2.2.2-5 Wave Height at Sliding of Existing Revetment

In the light of the above considerations, the attenuation factors of α =0.4/0.7=0.57 and α =0.4/0.68=0.59 are adopted for the Betio ocean side and the entire lagoon side respectively. The maximum wave height, which is 1.8 times of the significant wave height, is adopted to estimate wave pressure, and summarized in Table 2.2.2-16.

Table 2.2.2-16 Applied Wave Height and Pressure

Location	Ocean Side		Lagoon Side
	Bairiki side	Betio side	
Case of Analysis	Revised Tidal Condition King Tide under El Niño and Sea Level Rise: +3.30m Design Wave Height (H=6.1m)		
Water Level at Revetment (m)	+3.91		+3.83
Significant Wave Height at Revetment $H_{1/3}$ (m)	0.95		1.15
Attenuation Factor: α	1.0	0.57	0.59
Wave Height: H= α $H_{1/3}$ (m)	0.95	0.54	0.68
Maximum Wave Height: H_{max} =1.8 H (m)	1.71	0.97	1.22
Wave Incident Angle β (deg.)	90		45
Wave Pressure: $p=1.0w_oH_{max}\cdot\cos\beta$ (KN/m²)	17.3	9.8	8.7

Source: The Study Team

b) Stable Computation

Stable computation of the parapet is conducted based on the conditions as shown in Table 2.2.2-17. And Safety factor is set as 1.20.

c) Ocean Side (Bairiki Side)

The result of safety computation for ocean side (Bairiki side) revetment is shown in Table 2.2.2-18 and Figure 2.2.2-6.

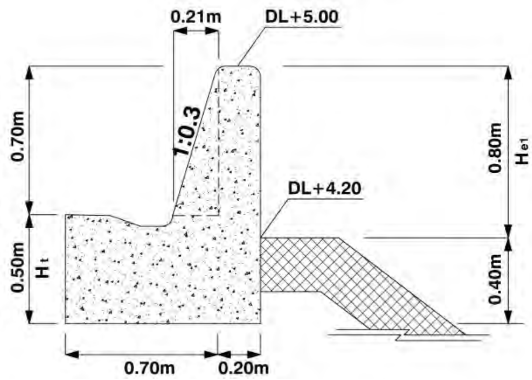


Figure 2.2.2-6 Parapet Shape of Ocean Side (Bairiki Side)

Table 2.2.2-17 Result of Safety Computation for Ocean Side (Bairiki Side)

External Force	Wave Pressure (Wp)			$1.77 tf/m^2$
	Wave Force (WF)		$=W_p \times H_{e1}$	1.42tf/m
Counterforce	Earth Pressure	Angle of Internal Friction (Ø)		35°
		Unit Weight (γ_s)		$1.90 tf/m^3$
		Coefficient of Passive Earth Pressure (K_p)	$=Tan^2(\pi/4+Ø/2)$	3.7
		Passive Earth Pressure (P_p)	$=1/2 \times K_p \times \gamma_s^2 \times Ht^2$	0.87tf/m
	Concrete	Unit Weight (γ_c)		$2.0 tf/m^3$
		Sectional Area (A_c)		$0.77 m^2$
		Parapet Weight (W_c)	$=\gamma_c \times A_c$	1.54tf
		Coefficient of Friction (μ)		0.6
	Total Counterforce (RF)		$=P_p+W_c \times \mu$	1.80
Safety Factor (SF)			=RF/WF (≧1.20)	1.27

d) Ocean Side (Betio Side)

The result of safety computation for ocean side (Betio side) revetment is shown in Table 2.2.2-17 and Figure 2.2.2-7.

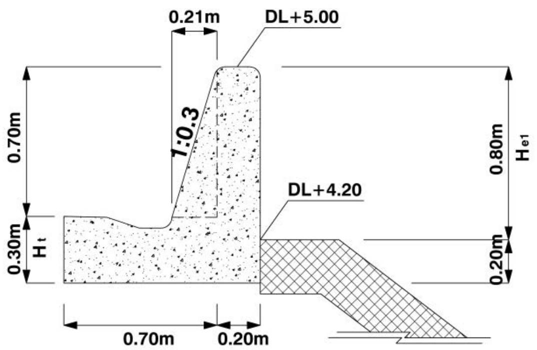


Figure 2.2.2-7 Parapet Shape of Ocean Side (Betio Side)

Table 2.2.2-18 Result of Safety Computation for Ocean Side (Betio Side)

External Force	Wave Pressure (Wp)			$1.01 tf/m^2$
	Wave Force (WF)		$=W_p \times H_{e1}$	0.80tf/m
Counterforce	Earth Pressure	Angle of Internal Friction (Ø)		35°
		Unit Weight (γ_s)		$1.90 tf/m^3$
		Coefficient of Passive Earth Pressure (K_p)	$=Tan^2(\pi/4+Ø/2)$	3.7
		Passive Earth Pressure (P_p)	$=1/2 \times K_p \times \gamma_s{}^2 \times Ht^2$	0.31tf/m
	Concrete	Unit Weight (γ_c)		$2.0 tf/m^3$
		Sectional Area (A_c)		$0.55 m^2$
		Parapet Weight (W_c)	$=\gamma_c \times A_c$	1.09tf
		Coefficient of Friction (μ)		0.6
	Total Counterforce (RF)		$=P_p+W_c \times \mu$	0.97
Safety Factor (SF)			=RF/WF (≧1.20)	1.21

e） Lagoon Side

The result of safety computation for lagoon side revetment is shown in Table 2.2.2-18 and Figure 2.2.2-8

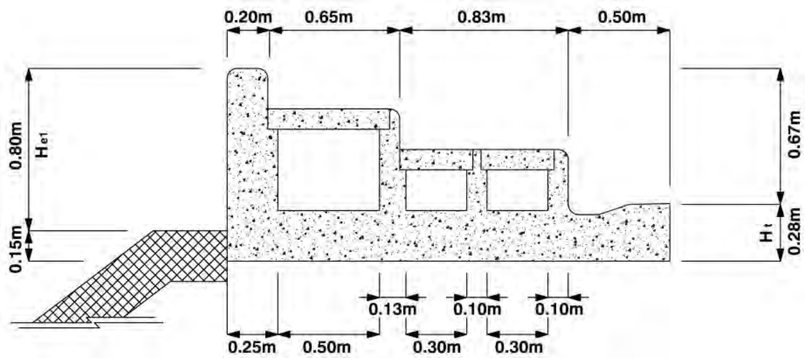


Figure 2.2.2-8 Parapet Shape of Lagoon Side

Table 2.2.2-19 Result of Safety Computation for Lagoon Side

External Force	Wave Pressure (Wp)			$0.90 tf/m^2$
	Wave Force (WF)		$=W_p \times H_{e1}$	0.72tf/m
Counterforce	Earth Pressure	Angle of Internal Friction (Ø)		35°
		Unit Weight (γ_s)		$1.90 tf/m^3$
		Coefficient of Passive Earth Pressure (K_p)	$=Tan^2(\pi/4+Ø/2)$	3.7
		Passive Earth Pressure (P_p)	$=1/2 \times K_p \times \gamma_s{}^2 \times Ht^2$	0.27tf/m
	Concrete	Unit Weight (γ_c)		$2.0 tf/m^3$
		Sectional Area (A_c)		$0.94 m^2$
		Parapet Weight (W_c)	$=\gamma_c \times A_c$	1.88tf
		Coefficient of Friction (μ)		0.6
	Total Counterforce (RF)		$=P_p+W_c \times \mu$	1.40
Safety Factor (SF)			=RF/WF (≧1.20)	1.96

2） Calculation of Strength

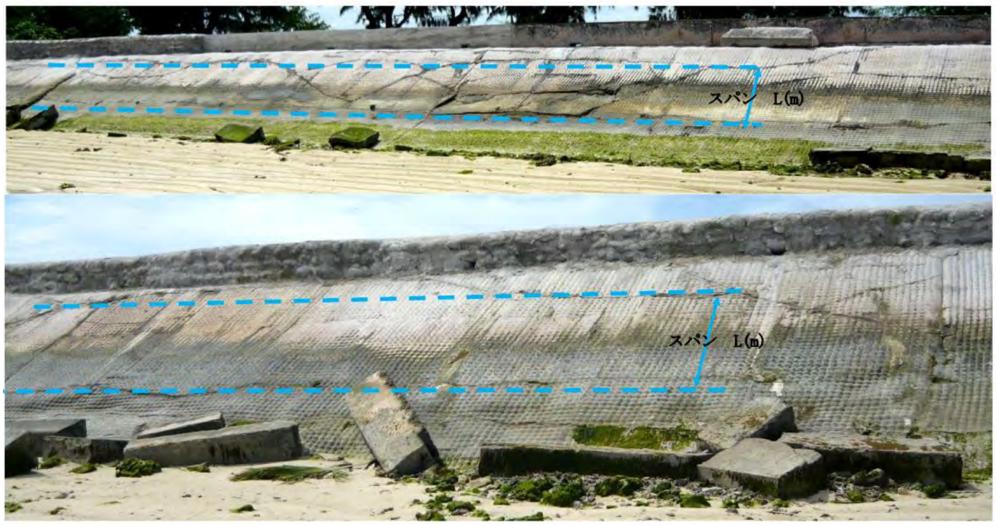
a） Thickness of Fabrimat Mat (Review for Wave Action)

A method of selecting thickness of the fabrimat mat on the basis of wave force described in the technical document by the manufacturer of the fabrimat mat is as follows:

'The function of the fabrimat mat can be achieved in one body with the reclaimed sand. Assuming the minimum fracture size generated by the bending moment under action of wave and selfweight in case some void develops due to settlement of the reclaimed sand etc., a weight of fabrimat mat fraction under can be estimated, and confirmed its weight larger than the required weight for armoring function using the Hudson formula.'

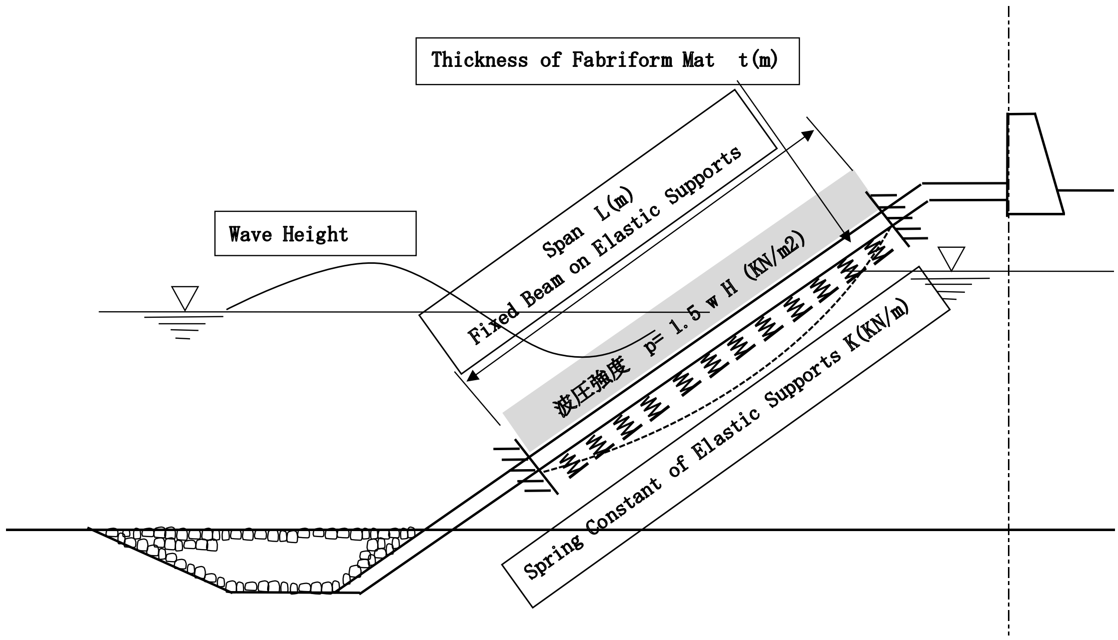
There is an assumption in the above method that the function of armor is maintained after generation of fracture of the fabrimat mat. However, the fraction of the existing fabrimat mats could not prevent outpouring of the reclaimed sand after fracture. It is planned in this study that geotexitile sheet under the fabrimat mat can prevent the reclaimed sand from outporing even after generation of the fracture. In addition, the thickness of the fabrimat mat should be increased as possible with the calculation model corresponding to the actual conditions of the fracture and referred to the technical document by the manufacturer so that the thickness of the fabrimat mat may prevent the fracture.

The water proof characteristics of the fabrimat mat should cause a repeated alternate forces acting to the mat surface due to the difference in ground water level, wave height and tidal level on the both faces. Since the upper and lower slope of the fabrimat mat are fixed, cracks around at the fixed parts are likely to be generated due to the bending moment caused by external forces when the fabrimat mat fluctuates. In fact the cracks develop at the upper and lower parts on the many slope surfaces as shown in Photo 2.2.2-1.



Source: The Study Team

Photo 2.2.2-1 Site Condition of Cracks on Revetment Slope (Bairiki Ocean Side)



Source: The Study Team

Figure 2.2.2-9 Calculation Model of Revetment Slope

The actual crack generation was compared with the calculation applying the presumed model in Figure 2.2.2-9 that the crack at the fixed support developed due to excess tensile stress of the bending moment over the strength caused by the wave action at high tide. The existing mat thichness of 15cm and 20cm were reviewed under the following conditions and assumptions:

- The slope of the fabrimat mat is considered as a fixed beam on elastic supports. The spring constant: Kv is assumed with the formula stipulated in 'The Specification for Highway Bridge'. $Kv=(1/0.3) \cdot \alpha \cdot Eo \cdot (Bv/0.3)^{-3/4}$

where, α =1,Eo=2800N(KN/m2), Bv=(1m x Spen)1/2, N=3 to 5 is assumed as void of loosen ground caused cracks,

therefore, Kv=10,000 (KN/m2)

- The length of the exposed existing slope is about 4.7m, and the spacing of the generated crack lines (upper and lower) along longitudinal direction range 2.5m to 3.5m. By this observation the span of L=3.5m is used.
- The wave pressure of p=1.5wH is considered on the slope with reference to Tamai[7] el al (1975) as Surging Wave. This wave pressure is the same as mentioned in the technical document by the manufacturer of the fabrimat mat. The self-weight of 22.5KN/m3is also considered.

[7] Tamai and Kobayashi 'Studies on the Wave Pressure Acting on the Slope Wall' JSIDRE Apr. 1975

- The significant wave height is taken into account since the crack was generated by the repeated wave actions. For Bairiki ocean side, where plenty numbers of damage and cracks have been occurred and concentration of wave due to overlapping was observed at the time of king tide during the study period, the wave height is increased by 1.2 with refraction coefficient of 0.7 taking into account of wave overlapping as shown in Figure 2.2.2-10.

Source: The Study Team on the basis of Google Earth

Figure 2.2.2-10 Refraction at Bairiki Ocean Side

- The wave height at the revetment in the original design: 0.7m (wave period 9.3 sec.), w: unit weight of sea water (10.1 KN/m^3).
- The tensile strength is used as the ultimate states, for the reason of the cracks already generated. The tensile strength:1.6(N/mm^2 of the existing fabrimat mat containing mortar is presumed in accordance with the relational expression: f_{tk}=0.23・f_{ck}2/3 described in 'The Standard Specifications for Concrete Structures' in case of the concrete strength: f_{ck}=18N/mm^2.
- The section modulus of the fabrimat mat is obtained from Z=bt3/6. For t=15cm, Z=0.00375m^3/m, and for t=20cm, Z=0.00667m^3/m.

The results of calculation are presented in Table 2.2.2-20. The tensile stress of the mat thickness: 15cm exceeds the tensile strength much. In case of the thickness: 20cm, the tensile stress reaches at the tensile strength and crack may occurs where weak strength. By comparison of the results and site observation, this calculation model seems reasonable.

Table 2.2.2-20 Review of Existing Fabrimat Mat

Item	Existing Fabrimat Mat	
Thickness of Mat: t (cm)	15	20
Wave Height: H (m)	0.84	0.84
Wave Pressure: p (KN/m^2/m)	12.7	
Self-Weight: w x cosθ (KN/m^2/m)	2.8	3.7
Load: p+w (KN/m)	15.5	16.4

Item	Existing Fabrimat Mat	
Bending Moment: M (KNm/m)	15.1	14.0
Section Modulus: Z (m^3/m)	0.00375	0.00667
Tensile Stress: σ=M/Z (N/mm^2)	4.0	2.1
Tensile Strength (N/mm^2)	1.6	
Site Observation on Condition of Cracks	Cracks found on not only upper and lower locations, but also other area at the entire slope	Cracks found on upper and lower locations at a limited slope

Source: The Study Team

With this calculation model, the thickness of the fabrimat mat is determined to prevent the cracks as found on site for the case of the maximum wave height at the revetment in the presume combination of the revised tidal level and wave height. The similar conditions and assumptions are taken into account as follows:

- The cases of the overlaid new mat and the replaced new mat are analyzed.
- The planned thickness is 25/30cm for Bairiki ocean side, 20/25cm for Betio ocean side, and 15/20cm for the entire lagoon side. (The figures present the thickness for Overlaid/ Replaced)
- The spring constant of Kv=140,000 (KN/m^2) is selected for the overlaid mat considering modulus of the covered existing mat to the extent of N=60. For the replaced mat, Kv=80,000 (KN/m^2) is applied under the condition that the base slope should be prepared to the extent of N=35.
- As the same way of the review of the existing slope, the significant wave is applied, and the wave height for Bairiki ocean side is increased by 1.2 with refraction coefficient of 0.7 taking into account of wave overlapping.
- The wave height for the lagoon side is reduced by using the angle of 45 degrees to the slope taking account of the incident wave angle of about 60 degrees from the west.
- The adopted load consist of the self-weight and wave pressure. The both load factors are 1.1 and 1.2 respectively in accordance with TSPHF. The factors of structure and analysis are both the same of 1.0. The member factor takes 1.1.
- The wave height at the revetment :0.95m (wave period 9.3sec.) for the ocean side, and 1.15m (wave period 9.3sec.) for the lagoon side
- The tensile strength: 1.8 (N/mm^2 of the new fabrimat mat containing concrete is estimated in accordance with the previously described expression. The material factor is 1.3.
- The section modulus of the fabrimat mat is obtained from Z=bt3/6. For t=25cm, 0.0104m^3/m , and for t=30cm, Z=0.015m^3/m

Under the above conditions and the applied span of L=3.5m, which is close to the space of the cracks observed on site, the moment resistance of the fabrimat mat with the planned thickness is confirmed to cover the bending moment caused by the wave action.

Table 2.2.2-21 Verification of Planned Fabrimat Mat Thickness (Wave Action)

Item	Plan					
	Replaced Mat			Overlaid Mat		
Location	Betio Ocean	Bairiki Ocean	Lagoon Side	Betio Ocean	Bairiki Ocean	Lagoon Side
Tensile Strength(N/mm^2)	1.8			1.8		
Applied Span: L (m)	3.5			3.5		
Mat Thickness: t (cm)	25	30	20	20	25	15
Wave Height: H (m)	0.95	1.14	1.15	0.95	1.14	1.15
Wave Pressure: P (KN/m^2)	14.4	17.3	12.3	14.4	17. 3	12.3
Self Weight: D=w x cosθ (KN/m^2)	4.7	5.6	3.7	3.7	4.7	2.8
Load: 1.1D+1.2P (KN/m)	22.5	26.9	18.8	21.4	25.9	17.8
Bending Moment: M (KNm)	13.1	18.9	8.1	6.9	11.8	3.6
Section Modulus: Z (m^3)	0.0104	0.015	0.00667	0.00667	0.0104	0.00375
Tensile Strength: ftk' (N/mm^2) γm=1.3	1.4			1.4		
Moment Resistance: Rd Rd=Z・ftk'/γb(KNm) γb=1.1	13.2	19.1	8.5	8.5	13.2	4.8
M/Rd γi=1.0/γa=1.0	1.0	1.0	0.9	0.8	0.9	0.7
Verification	OK	OK	OK	OK	OK	OK

Source: The Study Team

b) Thickness of Fabrimat Mat (Review for Residual Water Level)

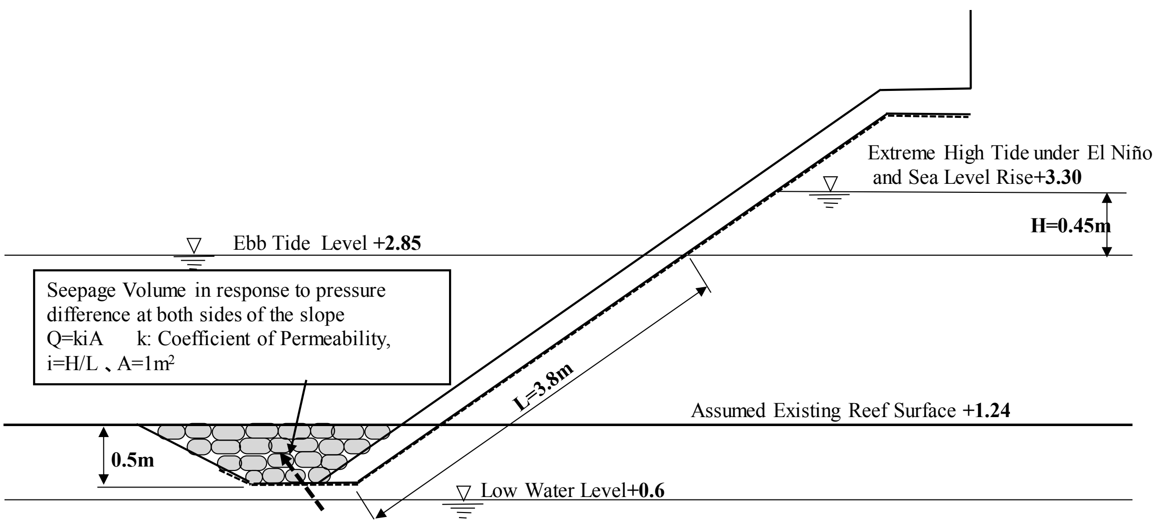
A time lag between outer tidal change and corresponding internal water level variation will be generated due to the water stop characteristics of the fabrimat form, which covers the entire slope of the causeway. The residual water level caused by the time lag may fall according to seepage through the slope toe from the inside of the causeway. The required time duration depends on a coefficient of permeability on the existing ground and the fill material of the causeway. By way of example, in the Land improvement business planning criteria issued by Agricultural Structure Improvement Bureau of Ministry of Agriculture, Forestry and Fisheries, there is a description that the water level inside of fill dam falls according to the water level fall of the reservoir if the coefficient of permeability is larger than 1x10-3(cm/s). With reference to this description, in the design of the Terre Armee wall that has a similar structure to the fabrimat mat, the residual water level is not considered where the coefficient of permeability exceeds 1×103(cm/s).

To know the extent of permeability on the causeway fill, the estimated time required for the water level fall by 0.45m (tidal level change during 1 hour) was reviewed, while the tidal level falls from the +3.30m (under sea level rise) to the LWL of +0.6m during assumed 6 hours. The conditions and results of the review are as follows:

- The coefficient of permeability: 0.05(cm/s) was taken from the specified range of 0.02 - 0.4 (cm/s) corresponding to the particle size (fine to coarse) of the existing fill. The grain size analysis for the specimens at the depth above 4m resulted in the D20 range of 0.13 - 0.63 (mm).

The averaged coefficient of permeability estimated from D20 using Creager became 0.07 (cm/s) which gave the close value.

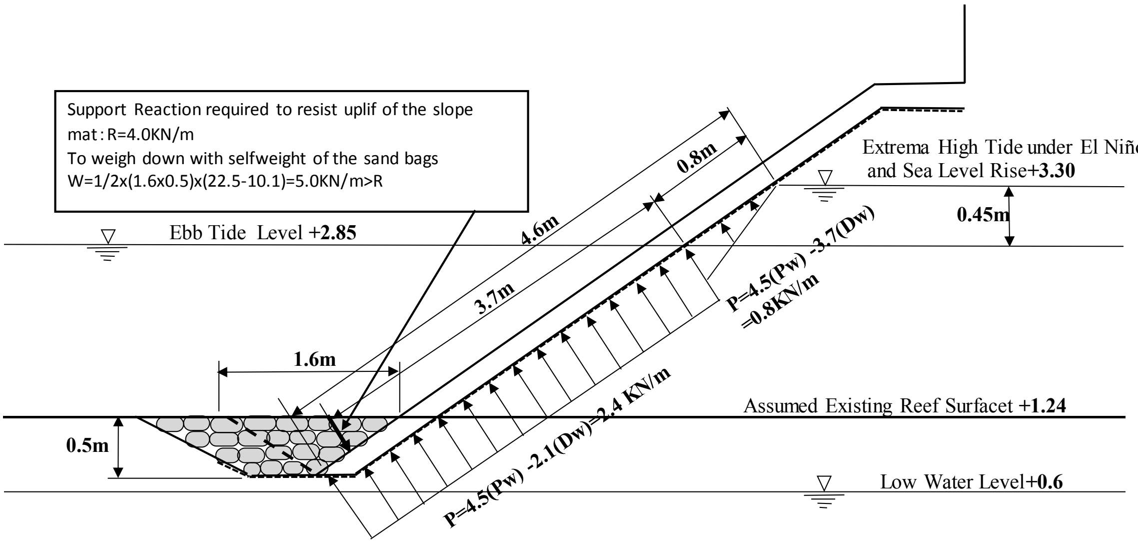
- A unit sectional area of $A=1m^2$ was taken into account.
- The seepage volume was estimated with Q=kiA from Darcy.
- The water level was presumed to fall at maintaining the same water difference.
- The hydraulic gradient become i= (0.45/3.8)=0.12, in the case of the seepage distance along the slope: 3.8m as indicated in Figure 2.2.2-11.
- The moisture content:0.5 of the fill was presumed, then the seepage volume through the unit section corresponding to the water difference of 0.45m become Q=0.5x45cm・m^2, and the required time for the fall of 45cm was estimated with t=Q/(ki)=1.06 hours (3,800 sec.)
- This review suggested that water difference larger than 0.45cm could not be generated under this condition since the discharge from the fill almost equaled to the required seepage volume for the water level fall of 45cm during the tidal change of 1 hour. Therefore, the residual water level was not considered to the cause way slope.



Source: The Study Team

Figure 2.2.2-11 Water Level Difference of Slope

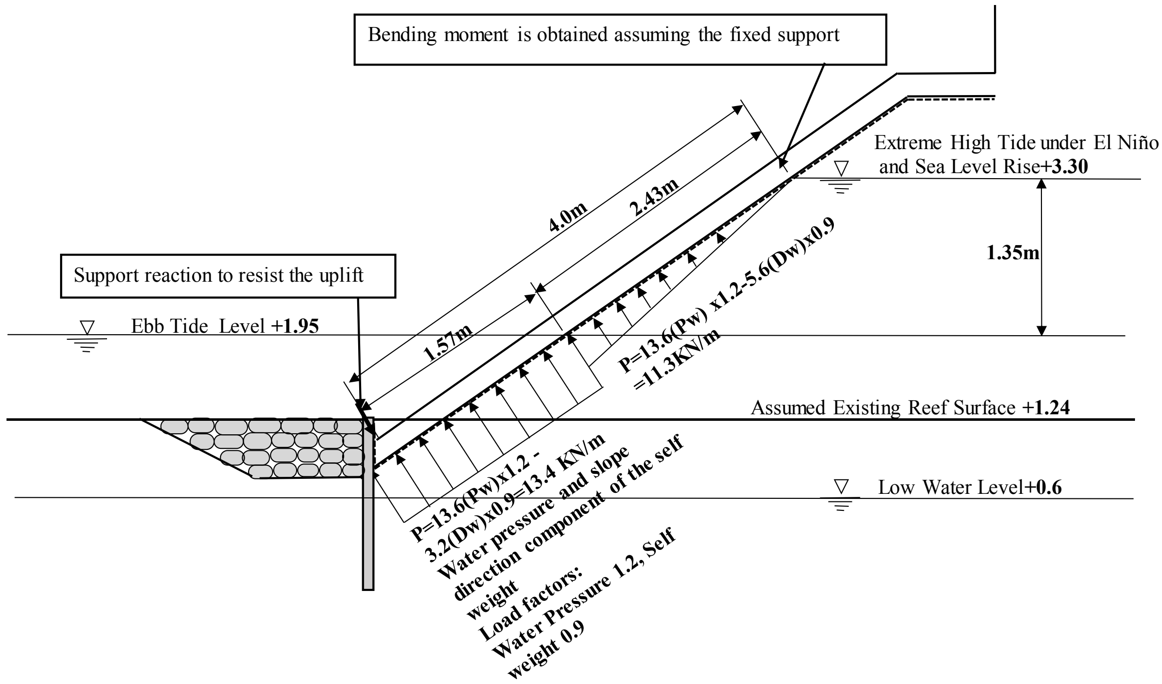
About the thickness (0.5m) of the sand bags installed at the slope toe protection, the submerged weight of 6KN/m2 can weigh down the water pressure of smaller than 4.5 KN/m2 at the residual water of smaller than 0.45m. The uplift of the fabrimat mat of 20cm thickness can also be resisted with the weight of the sand bags as examined in Figure 2.2.2-12.



Source: The Study Team

Figure 2.2.2-12 Uplift of Fabrimat Mat

Where the sheet pile (L=4m) are provided at the slope toe, the hydraulic gradient becomes i=0.45/11.8=0.04 under the same conditions as formerly described. Since this leads to 3 times longer duration to reach the same water level fall, a certain extent of residual water level is anticipated. Taking into account of seepage discharge at the lagoon side, the residual water level to 1/2 of the tidal difference is considered in the calculation model for the slope with the sheet pile in Figure 2.2.2-13. The estimated bending moment in the Fabrimat mat is confirmed within the resistance moment as indicated in Table 2.2.2-22. There causes a horizontal reaction force at the top of the sheet pile as the result of the resistance against the uplift, this is considered in the calculation of the sheet pile as described hereinafter.



Source: The Study Team

Figure 2.2.2-13 Residual Water Level and Calculation Model for Fabrimat Mat with Sheet Pile

Table 2.2.2-22 Verification of Planned Fabrimat Mat Thickness (Residual Water Level)

Location	Bairiki Ocean
Tensile Strength(N/mm^2)	1.8
Applied Span: L (m)	4.0
Mat Thickness: t (cm)	30
Wave Pressure: Pw (KN/m^2)	13.6
Self Weight: Dw = w x cosθ (KN/m^2)	3.2/5.6
Load: 1.2Pw – 0.9 Dw (KN/m)	13.4/11.3
Bending Moment: M (KNm)	17.0
Section Modulus: Z (m^3)	0.015
Tensile Strength: ftk' (N/mm^2) γm=1.3	1.4
Moment Resistance: Rd Rd=Z・ftk'/γb(KNm) γb=1.1	19.1
M/Rd (OK in case not exceeds 1.0) γi=1.0, γa=1.0	0.9
Assessment	OK
Support Reaction: R (KN/m)	18.2

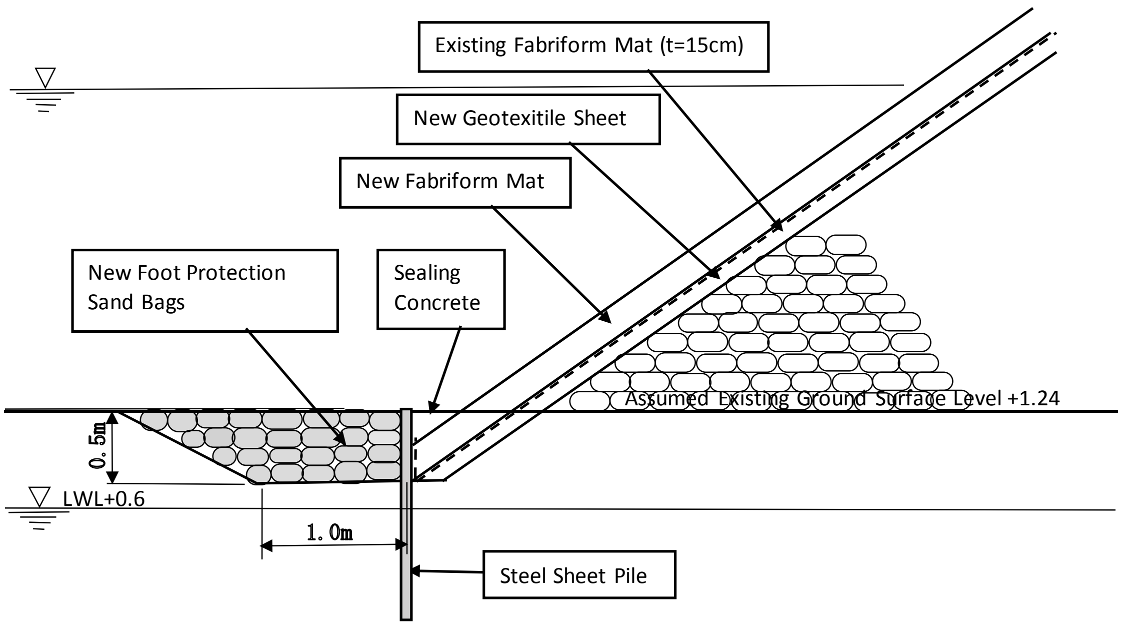
Source: The Study Team

c) Slope Toe Protection

Sheet piled toe protection is adopted for the area where the frequent damages were encountered and high risk of the further damage is anticipated. The structure of the toe protection is shown in Figure 2.2.2-14.

A new fabrimat mat should be embedded into the existing ground and its front should be protected with the sheet pile. The surface layer of the existing ground is found to be covered with loose coral sand according to the soil investigation. To avoid excess lateral displacement of the sheet pile, layers of sand bags should be installed in front of the sheet pile.

For the area where the damage due to the wave action was not frequently occurred, the foot protection without the sheet pile is adopted.



Source: The Study Team

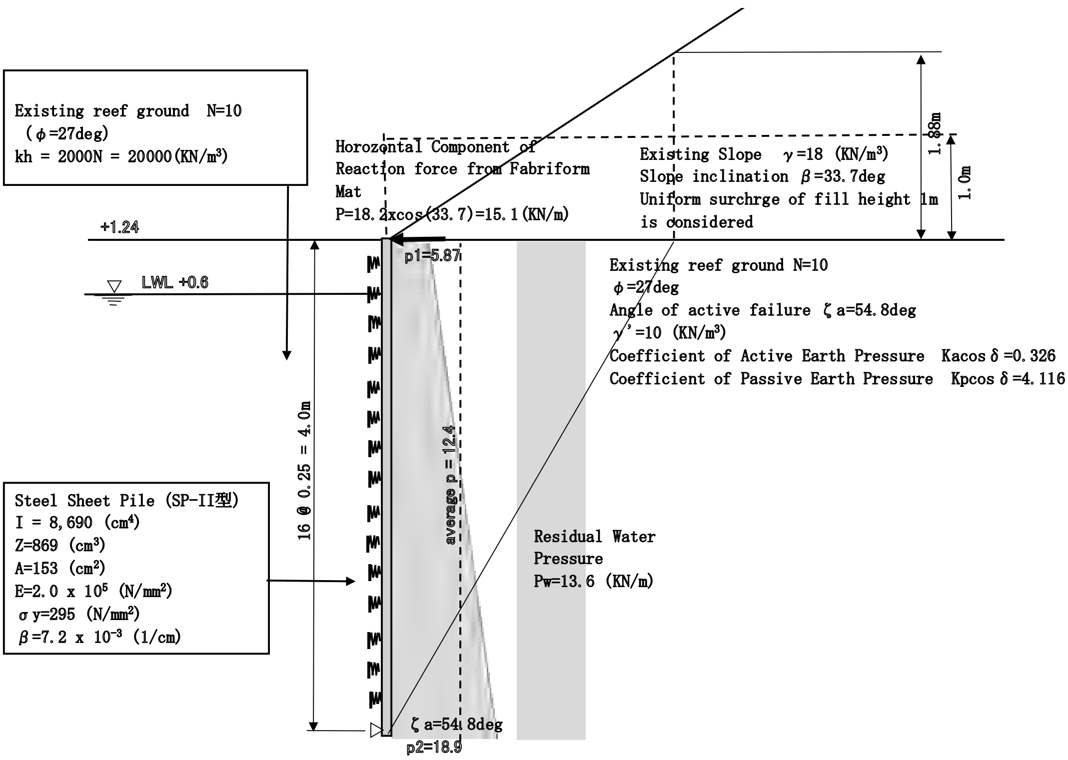
Figure 2.2.2-14 Slope Toe Protection

The minimum size of the steel sheet pile should be Type II, taking into account of the hard driving at the coral partly encountered. The bending stress of the sheet pile is calculated with the models illustrated in Figure 2.2.2-15 considering the applied loads of the active earth pressure including slope effect and the residual water pressure.

The conditions and results are as follows:

- The sheet pile is analyzed with a beam model with the width of 1m, and the passive earth pressure is evaluated as spring supports. The spring constant is obtained from the coefficient of lateral subgrade reaction. A hinged support is adopted at the toe of the sheet pile.

- The existing ground surface elevation of 0.6m, the surface N value of 10, and the internal friction angle of 27 degrees are assumed.
- With the coefficient of lateral subgrade reaction given by 2000N (KN/m^3), the characteristics value of β =7.2x10-3(1/cm) is obtained. The required length of the sheet pile should be at least the length considered as semi-infinite, thus taking 2.5/β, the length of 4m is determined.
- Uniform surcharge of 1m height is considered, since the average height over the effective area of the slope above the active failure plain drawn from the toe of the sheet pile becomes 0.94m. This surcharge earth pressure is conservative side compared with the earth pressure taking the inclination of the slope angle. The uniform load by average of the active earth pressure along the sheet pile length is used in the calculation.
- The case of LWL is considered, as the surcharge load becomes the maximum.
- The residual water pressure and the support reaction resisting uplift of the fabrimat mat are considered according to the reviews previously mentioned.



Source: The Study Team

Figure 2.2.2-15 Calculation Model of Steel Sheet Pile

Table 2.2.2-23 Result of Calculation

Maximum Bending Moment (KNm)	3.8
Maximum Bending Stress (N/mm^2)	4.3> 180
Head Lateral Displacement (cm)	0.0006 (m)

2.2.2.4 Bridge Design

(1) Design Condition

1） Bridge Strengthening

Design condition of the existing bridge is shown in Table 2.2.2-24 to Table 2.2.2-26.

Table 2.2.2-24 Design Condition of Existing Bridge

Pavement	Asphalt Pavement	: 110mm
Geometric Structure	Profile	: LEVEL
	Superelevation	: 2% (Normal Crown)
Design Load	Dead load for utilities	: Electric Cable (9kg/m), Communication Cable (2kg/m), Water Pipe (47kg/m)
	Live Load	: TL-20 (Concrete slab is checked by B-live load)
	Pedestrian Load	: 350kg/m^2
	Wind Load	: Design Wind Speed 23.2m/s(84km/h), Recurrent Interval 50 years, Basic Wind Load 244km/m2, Basic Wind Speed 160.9km/h
	Seismic Load	: Horizontal Seismic Load (he=0.05W)
	Temperature Change	: Standard Temperature ±10 degree
	Wave Force	: P=1.5w･H (P: Breaking Wave Pressure (t/m^2), w: Bulk Density of Seawater (t/m^3), H: Wave Height(m))
Materials	Reinforced Concrete	: 210kg/cm^2
	Levelling Concrete	: 180kg/cm^2
	Reinforcing Bar	: SD30

Table 2.2.2-25 Design Condition of Analysis for Current Status of the Bridge

Bulk Density of Materials	Pavement			kN/m^3	22.50
	Embankment (Wet)			kN/m^3	18.00
	Embankment (Saturated)			kN/m^3	18.80
	Reinforced Concrete			kN/m^3	24.50
	Water			kN/m^3	9.80
Earth Pressure Coefficient	Vertical Earth Pressure			-	1.00
	Horizontal Earth Pressure			-	0.50
Temperature Change	Upper			degree	15.0
	Lower			degree	-15.0
Allowable Stress of Materials	Concrete	Design Strength		N/mm^2	21.0
		Allowable Bending Stress	Normal	N/mm^2	7.0
			Haunch	N/mm^2	7.0
			No Haunch	N/mm^2	5.25
		Allowable Bearing Stress		N/mm^2	6.30
		Allowable Shear Stress (1)		N/mm^2	0.360
		Allowable Shear Stress (2)		N/mm^2	1.600
		Allowable Punching Shear Stress		N/mm^2	0.850
		Allowable Bond Stress	Normal	N/mm^2	1.40
			Corner of Intersection	N/mm^2	1.40
		Modulus of Elasticity		N/mm^2	2.35×10^4
	Reinforcing Bar	Quality of Material		-	SD295
		Allowable Tensile Stress		N/mm^2	160.0
		Allowable Tensile Stress (Top Slab)		N/mm^2	180.0
		Allowable Compressive Stress		N/mm^2	180.0
Deepness of Cover for Reinforcing Bar				cm	10.0
Live Load				kN	25.0

Table 2.2.2-26 Soil Constants for Bearing Capacity Check of Existing Bridge

Type of Soil	Thickness (m)	Average N Value	Bulk Density (kN/m^3)	Angle of Internal Friction (Degree)
S1 (Embankment)	2.90	16	20	30
S2 (Cemented reef top sediment)	2.07	24	20	34
	0.83		11	34
S3 (Unconsolidated Sediment)	3.30	10	10	27
G (Corals)	1.30	Over 50	11	42
S4 (Leached limestone)	5.05	25	10	34

Note: Soil investigation result by JICA team.

2） Navigation

Condition of Navigation is shown in Table 2.2.2-27.

Table 2.2.2-27 Condition of Navigation

Target	Fish boat with outboard engine (Length: 6.4m, Width: 2.0m, Maximum Draft: 0.78m)	
Width of Channel (Horizontal Limit of Channel)	10m (Horizontal Channel Limit = Maximum Boat Length×1.5=6.4×1.5=9.6m)	
Deepness of Channel	Mean Low Water Springs: MLWS -1.780m/	
Vertical Limit of Channel	Wave Setup after breaking of the offshore wave (50 yrs.) on the reef (m)	0.70m
	Wave Height on the reef /2 (m)	0.35m
	Draft + Trim + Seated Height + Others (m)	1.55m
	Total	2.60m
Slope Gradient of Channel	1:3	
Design Standard	Specification for Highway Bridge (Japan Road Association)	

(2) Design for Bridge Strengthening

1） Bridge Repair

As a result of Table 2.2.2-28, the repair works and partial retrofitting of the existing causeway was selected for recommendation. For the bridge section, the repair works and partial retrofitting is also recommended. The recommendable countermeasure for the improvement of the existing bridge and some issues/recommendations are shown in Table 2.2.2-24.

Moreover, in consideration with the navigational clearance with the surplus of design water level, the replacement of the existing bridge will be studied on the second site investigation.

Table 2.2.2-28 Countermeasure and Issues on the Bridge Improvements

Item	Location	Countermeasure and Issues
Pavement	Surface of Bridge	[Countermeasure] Re-pavement of the bridge section as well as the road section. [Issues] The crack on top of the slab shall be confirmed and repaired, if necessary.
	Road	[Countermeasure] The condition of existing pavement on the road is not good. The all pavement will be replace and reconstructed. [Issues] The embankment shall be properly filled in by sand. The sub-base and base coarse shall be reconstructed.
Concrete Wall	Wall	[Countermeasure] If the existing reinforcement had rusted, all of spalling concrete shall be removed and the reinforcement will be newly installed and the covering concrete will be reconstructed. [Issues] The condition of the existing reinforcement shall be confirmed after demolishing the spalling concrete. The joint with existing reinforcement and newly installed reinforcement shall be studied.
	Slab	[Countermeasure] If the existing reinforcement had rusted, all of spalling concrete shall be removed and the reinforcement will be newly installed and the covering concrete will be reconstructed. [Issues] The condition of existing reinforcement shall be confirmed after demolishing the spalling concrete. The joint with existing reinforcement and newly installed reinforcement shall be studied. The attached water pipeline at the ocean side shall be tentatively replaced.
Foundation	Steel Straight Sheet Pile	[Countermeasure] There is no damage, deterioration and deformation on the existing steel straight sheet piles, so the repair work is not required. However, the reconstruction of top concrete is needed because the existing top concrete are totally washed out. The foundation area constructed by steel straight sheet pile is not enough and the foundation area will be expanded by 10 m. [Issues] The steel straight sheet piles have to fit the improvement of slope protection. The steel structure shall be weathering materials.
	Slope Protection/ Groin Works	[Countermeasure] The slope shall be repaired and/or retrofitted. The foundation of steel straight sheet piles shall be expanded with 10 m along longitudinal direction. The waterway shall be protected by the concrete blocks to avoid the damage due to vortex flow. [Issues] The steel straight sheet piles have to fit the improvement of slope protection.
	Riverbed Protection	[Countermeasure] Some pert of river protection was washed away. Riprap with concrete block will be installed along the waterway embankment. [Issues] The minimum size or weight of riprap shall be studied so that the concrete block will not be washed out.
Ancillary Works	Pedestrian Way/ Utilities	[Countermeasure] The mount up pedestrian way will be renovated as flat type pedestrian way to avoid bottleneck of vehicles. The buried utilities under the pedestrian way will be relocated at the side of the bridge or the independent structure. [Issues] The layout plan including chamber of the utilities shall be studied.
	Bridge newel post/ handrail	[Countermeasure] The collapsed bridge newel post shall be reconstructed. The existing handrail could be maintained. [Issues] None
	Lighting	[Countermeasure] The lighting system with solar energy generation is newly installed along causeway including the bridge section. The existing lighting facilities will be removed. [Issues] The acceptance by the Government of Kiribati is required to remove the existing lighting facilities.
	Waterway	[Countermeasure] The sediment soil shall be dredged to maintain the navigation. [Issues] It shall be confirmed that the dredging of waterway is under the responsibility of the Government of Kiribati.

Table 2.2.2-29 Quantity of Bridge Repair

Section	Part		Chipping of Cocrete (t=150mm)			Cover of Concrete (t=150mm)			Scouring of Reinforcing Bar/ Rust-proof Treatment						Injection Volume of Epoxy Resin				
			Longitudinal	Traverse	Volume	Longitudinal	Traverse	Volume	Number					Length	No	Length	Width	Deepness	Volume
			(m)	(m)	(m^3)	(m)	(m)	(m^3)	D10	D13	D19	D22	D25	(m)		(m)	(m)	(m)	(L)
Oecan Side	Front Face	①	2.0	2.0	**0.60**	2.0	2.0	**0.60**	--	--	**8**	--	**16**	**48.0**	IB-1	1.1	0.0007	0.1	**0.08**
		②	0.7	0.3	**0.03**	0.7	0.3	**0.03**	--	**2**	--	--	**6**	**3.2**	IB-2	1.5	0.0008	0.1	**0.12**
		③	0.5	1.2	**0.09**	0.5	1.2	**0.09**	**5**	**2**	--	--	--	**4.9**	IE-1	2.6	0.0015	0.1	**0.39**
		④	0.5	0.5	**0.04**	0.5	0.5	**0.04**	**2**	**2**	--	--	--	**2.0**	IE-3	1.0	0.0010	0.1	**0.10**
		⑤	0.5	0.5	**0.04**	0.5	0.5	**0.04**	**2**	**2**	--	--	--	**2.0**	IE-4	1.0	0.0050	0.1	**0.50**
		⑥	0.4	4.0	**0.24**	0.4	4.0	**0.24**	**16**	**2**	--	--	--	**14.4**					
		⑦	0.5	0.3	**0.02**	0.5	0.3	**0.02**	--	**2**	--	--	**4**	**2.2**					
		⑧	1.5	2.5	**0.56**	1.5	2.5	**0.56**	--	--	**10**	--	**12**	**45.0**					
Subtotal			--	--	**1.62**	--	--	**1.62**	**25**	**12**	**18**	**0**	**38**	**121.7**		--	--	--	**1.19**
Lagoon Side	Front Face	①	1.0	0.6	**0.09**	1.0	0.6	**0.09**	**3**	**4**	--	--		**5.4**	IB-1	3.4	0.0040	0.1	**1.36**
		②	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	**2**	--	--	**4**	**3.0**	IB-2	3.4	0.0014	0.1	**0.48**
		③	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	**2**	--	--	**4**	**3.0**					
		④	1.0	0.5	**0.08**	1.0	0.5	**0.08**	--	**2**	--	--	**8**	**6.0**					
		⑤	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	--	**2**	--	**4**	**3.0**					
Subtotal			--	--	**0.13**	--	--	**0.13**	**3**	**10**	**2**	**0**	**20**	**8.4**		--	--	--	**1.84**
BP Side	Side Wall	①	1.6	1.6	**0.38**	1.6	1.6	**0.38**	--	--	**6**		**13**	**31.0**	I-4	2.0	0.0050	0.1	**1.00**
		②	1.6	1.6	**0.38**	1.6	1.6	**0.38**	--	--	**7**		**13**	**32.0**	I-5	2.0	0.0005	0.1	**0.10**
		③	1.2	1.8	**0.32**	1.2	1.8	**0.32**	--	--	**8**		**10**	**27.6**	I-6	1.0	0.0005	0.1	**0.05**
		④	1.0	0.3	**0.05**	1.0	0.3	**0.05**	--	--	**2**		**8**	**4.4**					
Subtotal			--	--	**1.14**	--	--	**1.14**	**0**	**0**	**23**	**0**	**44**	**95.0**		--	--	--	**1.15**
EP Side	Side Wall	①	1.8	1.3	**0.35**	1.8	1.3	**0.35**	--	--	**6**		**15**	**30.3**	I-1	1.2	0.0010	0.1	**0.12**
		②	1.0	1.4	**0.21**	1.0	1.4	**0.21**	--	--	**6**		**8**	**17.2**	I-2	1.5	0.0005	0.1	**0.08**
		③	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	--	**2**		**4**	**3.0**	I-3	1.0	0.0005	0.1	**0.05**
		④	2.1	2.3	**0.72**	2.1	2.3	**0.72**	--	--	**10**		**17**	**60.1**	I-4	0.4	0.0005	0.1	**0.02**
Subtotal			--	--	**1.32**	--	--	**1.32**	**0**	**0**	**24**	**0**	**44**	**110.6**		--	--	--	**0.27**
Top Slab		①	0.8	7.0	**0.84**	0.8	7.0	**0.84**	--	--	**28**	**7**	--	**71.4**	I-1	7.0	0.0005	0.1	**0.35**
		②	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	--	**2**	**4**	--	**3.0**					--
		③	0.5	0.5	**0.04**	0.5	0.5	**0.04**	--	--	**2**	**4**	--	**3.0**					--
Subtotal			--	--	**0.92**	--	--	**0.92**	**0**	**0**	**32**	**15**	**0**	**77.4**		--	--	--	**0.35**
Total			--	--	**5.12**	--	--	**5.12**	**28.0**	**22.0**	**99.4**	**15.0**	**146.0**	**413.1**		--	--	--	**4.79**

2） Bridge Strengthening

a） Analysis for Current Status of the Bridge

Box Culvert

Analysis result is shown in Table 2.2.2-30.

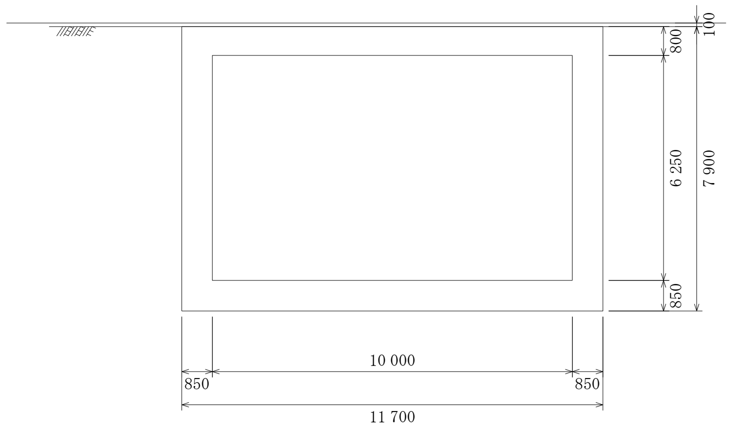
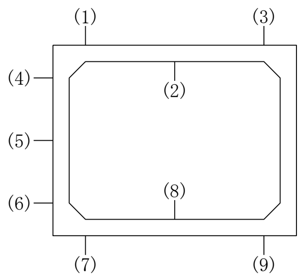


Table 2.2.2-30 Analysis Result of Box Culvert

		Unit	Top Slab			Side Wall		
			(1)	(2)	(3)	(4)	(5)	(6)
Sectional Force	M	kN.m	-163.4	175.9	-163.4	-182.0	-66.2	-374.2
	N	kN	55.6	55.6	55.6	131.6	203.4	281.0
	S	kN	126.3	-98.9	-126.3	-70.1	118.0	202.6
Reinforcing Bar		mm	D22@250	D22@125	D22@250	D22@250	D25@250	D25@125
			D19@250	-	D19@250	D19@250	D19@250	-
Depth of Cover for Reinforcing Bar		mm	100.0	100.0	100.0	100.0	100.0	100.0
			100.0	-	100.0	100.0	100.0	-
Stress	σ_c	N/mm^2	2.32	2.43	2.32	2.51	0.86	4.54
	σ_s	N/mm^2	86.02	81.74	86.02	76.40	-10.32	106.69
	τ_m	N/mm^2	0.180	0.141	0.180	0.093	0.157	0.270
Allowable Stress	σ_{ca}	N/mm^2	5.25	7.00	5.25	5.25	7.00	5.25
	σ_{sa}	N/mm^2	180.00	180.00	180.00	160.00	-180.00	160.00
	τ_a	N/mm^2	0.478	0.567	0.478	0.480	0.609	0.556

		Unit	Sole Slab		
			(1)	(2)	(3)
Sectional Force	M	kN.m	-360.7	334.1	-360.7
	N	kN	218.0	199.6	218.0
	S	kN	-269.9	197.8	264.8
Reinforcing Bar		mm	D25@125	D25@125	D25@125
Depth of Cover for Reinforcing Bar		mm	100.0	100.0	100.0
Stress	σ_c	N/mm^2	3.87	3.87	3.87
	σ_s	N/mm^2	107.62	100.47	107.62
	τ_m	N/mm^2	0.359	0.264	0.353
Allowable Stress	σ_{ca}	N/mm^2	5.25	7.00	5.25
	σ_{sa}	N/mm^2	160.00	160.00	160.00
	τ_a	N/mm^2	0.544	0.728	0.544



Wing

Analysis result of the wing part is shown in Table 2.2.2-30.

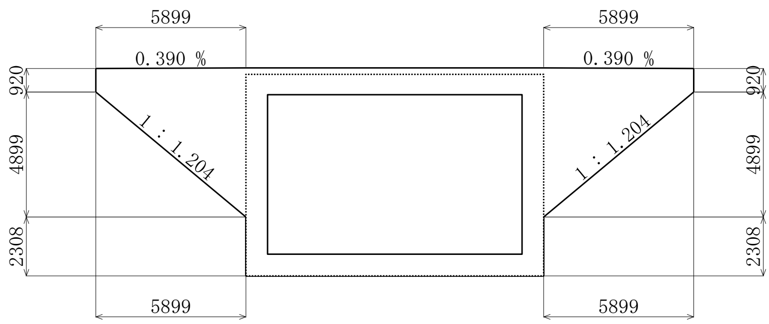


Table 2.2.2-31 Analysis Result of Wing Part

		Unit	Left Side		Right Wall		Retaining Wall
			付根	Reinforcing Bar	付根	Reinforcing Bar	
Sectional Force	M	kN.m	144.0	144.0	144.0	144.0	0.2
	N	kN	0.0	0.0	0.0	0.0	-
	S	kN	78.2	-	78.2	-	1.5
Reinforcing Bar		mm	D25@125	D25@250	D25@125	D25@125	D16@125
Stress	σ_c	N/mm^2	3.42	2.04	3.42	1.68	0.01
	σ_s	N/mm^2	81.57	77.88	81.57	46.49	0.42
	τ_m	N/mm^2	0.156	-	0.156	-	0.005
Allowable Stress	σ_{ca}	N/mm^2	7.00	7.00	7.00	7.00	7.00
	σ_{sa}	N/mm^2	160.00	160.00	160.00	160.00	160.00
	τ_a	N/mm^2	0.392	-	0.392	-	0.375

Wing thickness is 600mm

3) Bearing Capacity of the Existing Bridge

a) Calculation Method

Allowable Bearing Capacity of the foundation ground is set as formula 1 based on the Specification for Highway Bridge. Calculation model is shown in Figure 2.2.2-16.

$$Q_u = A_e \times \left(k \times c \times N_c \times S_c + k \times q \times N_q \times S_q + \frac{1}{2} \times \gamma_1 \times \beta \times B_e \times N_\gamma \times S_\gamma\right) \cdot \cdot$$

c	: Cohesion of Foundation Ground (kN/m^2)
q	: Loading (kN/m^2)
A_e	: Effective Loading Area (m^2)
γ_1	: Bulk Density of Foundation Ground (kN/m^2)
B_e	: Effective Loading Width of Foundation in consideration with Eccentricity of Load (m)
α, β	: Coefficient of Shape
k	: Rate of Increase
N_c, N_q, N_γ	: Coefficient of Bearing Capacity
S_c, S_q, S_γ	: Revision Coefficient for Size Effect of Bearing Capacity

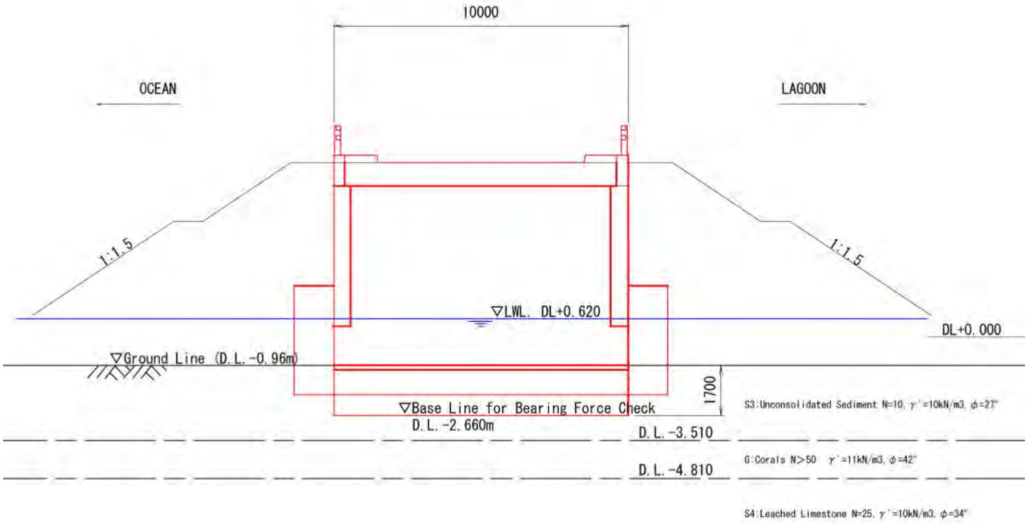


Figure 2.2.2-16 Calculation Model

b) Calculation Result

The result of calculation is shown in Table 2.2.2-32.

Table 2.2.2-32 Calculation of Bearing Capacity of Bridge

c	: Cohesion of Foundation Ground (kN/m^2)	0
q	: Loading (kN/m^2)	17.0
A_e	: Effective Loading Area (m^2)	117.0
γ_1	: Bulk Density of Foundation Ground (kN/m^2)	10.0
B_e	: Effective Loading Width of Foundation in consideration with Eccentricity of Load (m)	10.0
α	: Coefficient of Shape	1.26
β	: Coefficient of Shape	0.66
k	: Rate of Increase	1.05
N_q	: Coefficient of Bearing Capacity	32
N_c	: Coefficient of Bearing Capacity	48
N_γ	: Coefficient of Bearing Capacity	35

S_c	: Revision Coefficient for Size Effect of Bearing Capacity	1.000
S_q	: Revision Coefficient for Size Effect of Bearing Capacity	0.838
S_γ	: Revision Coefficient for Size Effect of Bearing Capacity	0.464
Q_u	: Ultimate Bearing Capacity (kN)	118,600
Q_a	: Vertical Load (kN)	9,800
F_s	: Safety Factor (≧3.0)	12.10

4) Design for Bridge Widening

Cross section of the bridge is adapted in same as the road cross section. Therefore, widening of the existing bridge is needed. The structure of widening by overhanging beam of concrete and the increase in bridge slab (25cm) is designed based on the following items;

- The walkway of the existing bridge is mount-up 25cm from road surface and is the reinforcing structure,
- If the widening of bridge is performed based on the present road surface, the mount-up part of the existing bridge walkway must be cut,
- In case of the above, it is difficult to understand the effect of cutting of mount-up part on the existing bridge structure,
- Therefore, the concrete slab is increased 25cm to avoid the cutting of the mount-up part.

Typical cross section of the bridge widening is shown in Figure 2.2.2-17.

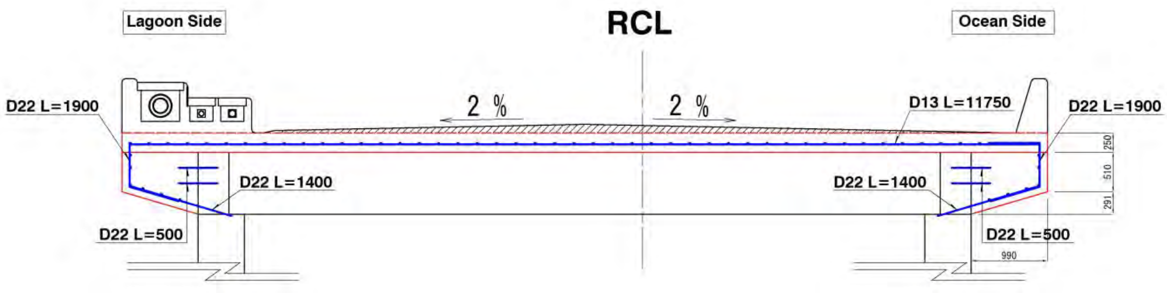


Figure 2.2.2-17 Typical Cross Section of Bridge Widening

2.2.2.5 Emergency Restoration Works Plan

(1) Condition

1） Application policy

Countermeasures shall be applied to ocean side only and shall not be applied to the lagoon side. Reinforcement of seawalls shall be applied to all the existing seawalls except the place where there is sand during high tide.

Slope protection plan shall be applied to where there are many cracks in fabric mat. Offshore breakwater plan shall be applied to all lines except both ends of causeway and channel considering direction of waves by wind.

2） Cost sharing

Japanese side shall be burdened with difficulty to procure the supply of materials in either Kiribati or Fiji and the construction supervisor. There is a 30t-rough terrain crane owned by KPA, but the rental fee is A$230 per hour and may be difficult to use in actual. Also, there is no crawler crane for lease in Kiribati. Therefore it is necessary to think a construction method that can be conducted even without crane.

(2) Selection of Measures

1） Method comparison

Consider the proposed measures in order to preserve the function of the causeway until the construction starts.

There are damage of the causeway such as destruction of slopes and seawalls.

These are because during stormy weather or by storm surge, seawater, either coming in from drainage holes or overtopping, penetrate through loose shoulder and go out with backfilled sand, then the slope and the seawalls collapse or seawall destroy directly by wave power

For slope protection, slope covering plan and offshore breakwater plan shall be examined. With regard to the reinforcement of seawalls, installation of sand bags behind the seawall for stabilization shall be examined.

The comparison chart is shown in Table 2.2.2-33.

Table 2.2.2-33 Comparison chart of emergency countermeasures (1)

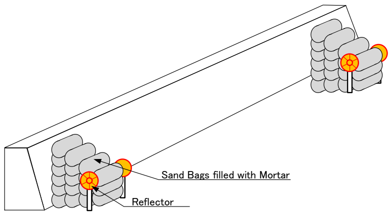
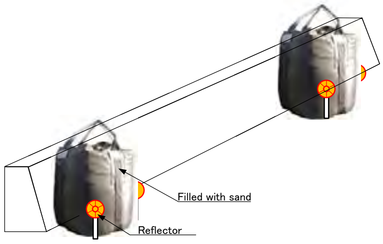
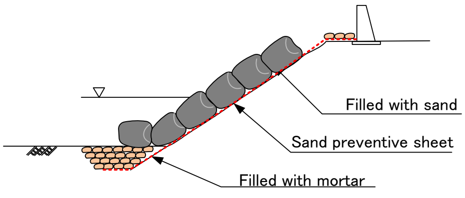
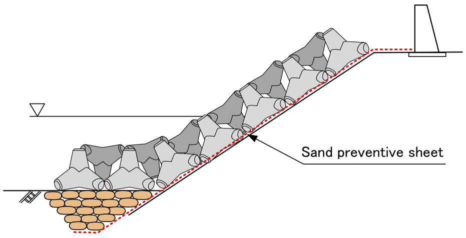
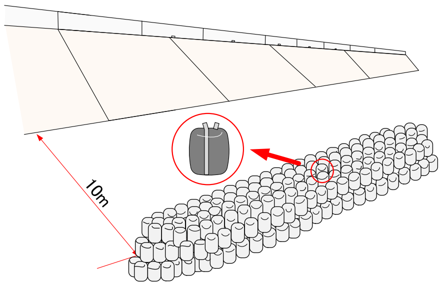
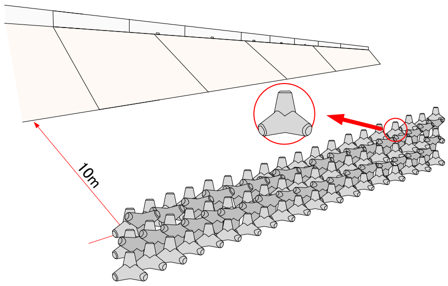
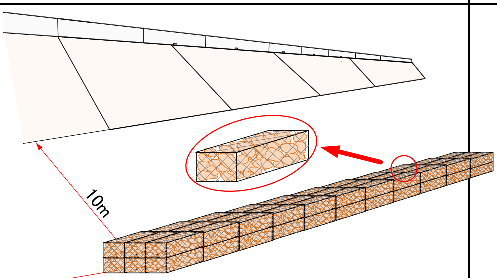
Seawall Protection	Seawall protection		
	Method	Mortar sandbags (P-1)	Large sandbags(P-2)
	Conceptual diagram		
	Outline	Install mortar sandbags behind the seawall by manpower	Install weather resistant large sandbags behind the seawall by using backhoe
Slope Protection	Slope protection		
	Method	Large sandbags(SL-1)	Wave-dissipating blocks (SL-2)
	Conceptual diagram		
	Outline	Install geotextile sheet on the slope, then install weather resistant large sandbags on a slope for slope protection	Install geotextile sheet on the slope, then install wave dissipating block on a slope for slope protection and wave dissipation.
	Offshore breakwater		
	Method	Large sandbags (SL-3)	Wave-dissipating blocks(SL-4)
	Conceptual diagram		
	Outline	Construct offshore breakwater around 10m from the slope toe by weather resistant large sandbags	Construct offshore breakwater around 10m from the slope toe by blocks
	Method	Gabions(SL-5)	Soldier pile and lagging method (SL-6)
	Conceptual diagram		
	Outline	Install gabions around 10m from the slope to bay backhoe	Apply soldier pile and lagging method. Drive steel sheet piles at the foot of the slope toe and install timbers

Table 2.2.2-33 Comparison chart of emergency countermeasures (2)

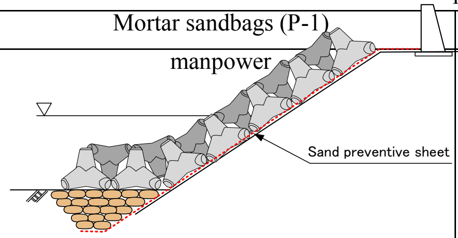
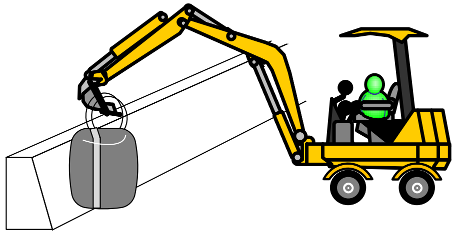
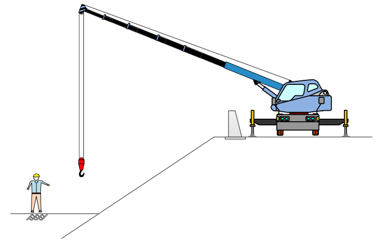
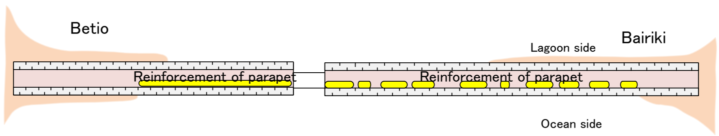
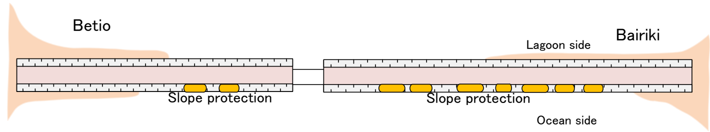
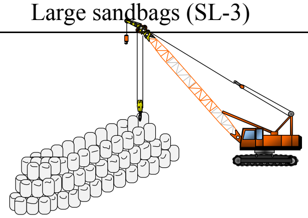
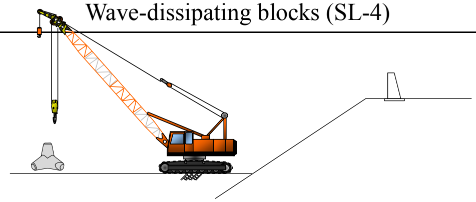
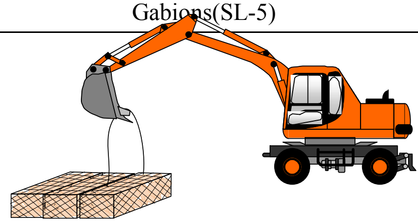
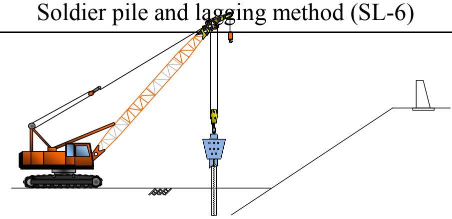
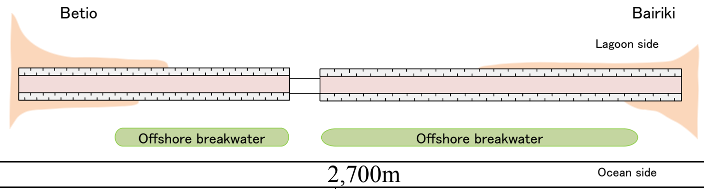
	Seawall protection						Slope protection					
Method	Mortar sandbags (P-1)			Large sandbags (P-2)			Large sandbags (SL-1)			Wave-dissipating blocks (SL-2)		
Conceptual diagram												
Outline	Install mortar sandbags behind the seawall by manpower			Install weather resistant large sandbags behind the seawall by using backhoe			Install geotextile sheet on the slope, then install weather resistant large sandbags on a slope for slope protection			Install geotextile sheet on the slope, then install wave dissipating block on a slope for slope protection and wave dissipation.		
Main equipment	✓ Dump truck 4t (owned by MPWU)			✓ Back hoe (owned by MPWU) ✓ Dump truck 4t (owned by MPWU)			✓ Back hoe (owned by MPWU) ✓ Dump truck 4t (owned by MPWU) ✓ Truck crane			✓ Truck crane 2 nos. ✓ Concrete mixer		
Budget allocation (plan)		MPWU	JICA		MPWU	JICA		MPWU	JICA		MPWU	JICA
	Sandbags Sand Labor Supervision	○ ○ ○ ×	× × × ○	Large sandbags sand Dump truck/backhoe Labor Supervision	× ○ ○ ○ ×	○ × × × ○	L- sandbags/geotextile sheet Sand Dump truck/backhoe Truck crane Labor Supervision	× ○ ○ × ○ ×	○ × × × × ○	Cement Aggregates Geotextile sheet Dump truck/backhoe Truck crane Concrete mixer Labor Supervision	× × ○ × × ○ × ×	○ ○ × ○ × × ○ ○
	Assumed burden charge for 2,520m	A$ 41,000	¥	Assumed burden charge for 2,520m	A$ 60,000	¥	Assumed burden charge for 1,000m	A$ 0.33M	¥	Assumed burden charge for 1,000m	A$ 5.2M	¥
Workability /construction Period	Easy			Easy			Easy execution. Installation work shall be done by a crane set on the causeway.			Necessary for large fabrication and storage yard. Needs long period for fabrication. Installation work shall be done by a crane set on the causeway.		
Applicable policies	Where concrete seawall exists.						Construction shall be conducted where large damages are observed.					
Location plan												
Const. length	2,520m						1,000m					
Characteristic	It will not work if the slopes collapse.			It will not work if the slopes collapse. Sand shall be filled in the large sandbags.			Contents of sandbags shall be coral sand			Diversion use shall be possible.		
Comments	Study team: Better to conduct with slope protection	MPWU: Better to conduct with slope protection		Study team: Better to conduct with slope protection	MPWU: Better to conduct with slope protection		Study team: Easy execution and repair.	MPWU: The best, but no availability of crane		Study team:: Needs long period High cost	MPWU:	

Table 2.2.2-33 Comparison chart of emergency countermeasures (3)

	Offshore breakwater											
Method	Large sandbags (SL-3)			Wave-dissipating blocks (SL-4)			Gabions(SL-5)			Soldier pile and lagging method (SL-6)		
Conceptual diagram												
Outline	Construct offshore breakwater around 10m from the slope toe by weather resistant large sandbags			Construct offshore breakwater around 10m from the slope toe by blocks			Install gabions around 10m from the slope to bay backhoe			Apply soldier pile and lagging method. Drive steel sheet piles at the foot of the slope toe and install timbers		
Main equipment	✓ Back hoe (owned by MPWU) ✓ Dump truck 4t (owned by MPWU) ✓ Crawler crane ✓ Concrete mixer			✓ Truck crane (for fabrication) ✓ Crawler crane (for installation) ✓ Concrete mixer			✓ Back hoe (owned by MPWU) ✓ Dump truck 4t (owned by MPWU) ✓ Concrete mixer			✓ Crawler crane ✓ Vibratory hammer		
Budget allocation (plan)		MPWU	JICA		MPWU	JICA		MPWU	JICA		MPWU	JICA
	Large sandbags cement Aggregates Dump truck/backhoe Concrete mixer Crawler crane Labor Supervision	× ○ ○ ○ ○ × × ×	○ × × × × × ○ ○	Cement Aggregates Dump truck/backhoe Concrete mixer Crawler crane Truck crane Labor Supervision	○ ○ ○ ○ × × ○ ×	× × × × × × × ○	Gabions Rubble stones Sump truck/backhoe Labor Supervision	× × ○ ○ ×	○ ○ × × ○	H beams Crawler crane Vibratory hammer Labor Supervision	× × × × × ×	○ ○ × × ○ ○
	Assumed burden charge for 2,700m	A$ 2.2M	¥	Assumed burden charge for 2,700m	A$ 6.9M	¥	Assumed burden charge for 2,700m	A$ 1.3M	¥	Assumed burden charge for 2,700m2.	A$	¥
Workability /construction Period	Poor. Installation can be done during low tide only. Workable hours shall be limited to more or less 60hours per month for installation works			Poor. Necessary for large fabrication and storage yard. Needs long period for fabrication. Installation work shall be done during low tide only. Workable hours shall be limited to more or less 60hours per month for installation works.			Poor. Necessary for large storage yard. Installation work shall be done during low tide only. Workable hours shall be limited to more or less 60hours per month for installation works.			Poor. Necessary for large storage yard. Installation work shall be done during low tide only. Workable hours shall be limited to more or less 60hours per month for installation works.		
Applicable policies	Construction shall be conducted except in sandy shore of ocean side, sandbag slopes are constructed and channelled.											
Location plan												
Const. length	2,700m											
Characteristic	Mortar shall be filled in the large sandbags.			Diversion use shall be possible			Necessary to import rubble stones.			Diversion use for materials shall not be expected for main construction works.		
Comments	Study team: Needs long period	MPWU:		Study team Needs long period High cost	MPWU:		Study team Needs long period	MPWU:		Study team Needs long period	MPWU:	

2) Method comparison

The collapse of the seawall is often associated with the collapse of the seawalls in front. However, it makes no sense to reinforce only the front seawalls in case slope revetment collapse. Therefore, it will be effective by performing a complex with other proposals.

It is impossible to conduct offshore breakwater plan because the working radius is too big when a crane is on the causeway. Likewise, it is important to use a barge due to low water depth. Therefore, work shall be conducted during low tide in setting up a crane at the toe of the slope. Since the crane can enter only from the both edge of the causeway, the mobilization will take time and the construction period shall become very long.

The slope covering plan is to install a geotextile sheet and put counterweights on top. Direct effect shall be expected. In case tetrapod is used for counterweight, a large construction area and stock yard shall be required and it takes time for fabrication.

On the other hand, if large sandbag is used for counterweight, filling material is sand and fabrication and installation is easy. Moreover, if a crane cannot be provided, fabrication /installation can be done by manpower. Therefore by teaching the construction method in the beginning, it is possible to perform additional installation or recovery by MPWU alone.

Based on the above, the recommended plan for slope covering by the consultant as well as the MPWU after an internal meeting is by large sandbag.

(3) Implementation schedule (draft)

The implementation schedule is shown in Table 2.2.2-34.

Table 2.2.2-34 Implementation schedule

	2015				2016															
	Nov.		Dec.		Jan.		Feb.		Mar.		Apr.		May		June		July		Aug.	
	15	30	15	31	15	31	15	28	15	31	15	30	15	31	15	30	15	31	15	31
Procurement																				
Mobilization																				
Preparation																				
Repair works																				
Cleaning																				
Supervision																				

2.2.3 Outline Design Drawings

Outline design drawings are attached inAppendix-2, and the table of contents is shown in Table 2.2.3-1.

Table 2.2.3-1 Table of Contents for Outline Design Drawing

No.	Drawing Title	Sheet No.	DRG No.
1	Location Map	1	LM-1
2	Horizontal Alignment	2~10	HA-1~9
3	Plan	11~19	PL-1~9
4	Profile	20~30	PF-1~11
5	Typical Cross Section	31~34	TCS-1~4
6	Cross Section	35~71	CS-1~37
7	Typical Drawing of Revetment Strengthening	72~73	TRS-1~2
8	Typical Drawing of Steel Sheet Pile	74	TSP-1
9	Fabrimat Details	75~77	FD-1~3
10	Parapet Details	78~79	PD-1~2
11	Transverse Drainage Details	80~81	TDD-1~2
12	Utilities Connection Part Details	82~83	UCP-1~2
13	Typical Drawing of Bridge Strengthening	84	TBS-1
14	Typical Drawing of Structures	85~86	TDS-1~2

2.2.4 Implementation Plan

2.2.4.1 Implementation Policy

The basic points for implementation of the project are as follows:

- This project will be implemented under the Grant Aid Scheme of the Government of Japan (GOJ) in accordance with the Grant Agreement (G/A) and the Exchange of Notes (E/N) by the Republic of Kiribati and the GOJ.
- The executing agency for the implementation of the project is the Ministry of Public Works & Utilities (MPWU) of the Republic of Kiribati.
- The consulting services including detailed design, tender-related works and construction supervision services, will be provided by a Japanese consulting firm in accordance with the consultancy contract that shall be executed with the Republic of Kiribati.
- The construction of road will be executed by a Japanese construction firm that shall be selected through pre-qualification and bidding, in accordance with the construction work contract that shall be executed between the said construction firm and the Republic of Kiribati.

The basic policies for the construction/procurement of this project are as follows:

- The equipment, materials and labor for construction shall be, as much as possible, procured locally. In cases where local procurement is not possible, they shall be procured either from a third country or from Japan where it is most economical insofar as the required quality and supply are secured.

- Construction method and the construction process shall be consistent with the local climate, topography, geology and natural conditions including the river characteristics.
- Plan the general and easy construction method which does not need the special possible equipment or technology.
- The contractor's site organization shall be planned to satisfy the established construction specifications and construction management standards set for this project. Likewise, the consultant's organization shall be based on such project management standards.
- To ensure safety during construction, appropriate traffic management plan including placement of construction and deployment of traffic personnel shall be considered.
- In order to reduce the influence of the environment on Kiribati, strive for preservation of environment, such as selecting the temporary place and garbage dump which were specified from the Republic of Kiribati.
- Since the enterprise for cooperation is a road of the beach, a great deal of damage is easily suffered. Condition of completing at an early stage as much as possible is considered.

2.2.4.2 Implementation Condition

(1) Considerations on the Natural Conditions

This construction site is directly affected by the influence of ebb and flow or a billow. During construction, these influences decreased as much as possible, and it can be constructed while passing an established road.

(2) Social and Environmental Consideration

It is a narrow island with a coral reef, and the temporary planned site also must be distributed and planned. In order to lessen the effect on the environment, in a temporary lot, implement the measure against noise, the number of construction-related vehicles must be lessened, or to shorten mileage.

(3) Reservation of a causeway user's traffic.

The causeway is the only land transport way to which the central part and the harbor of the capital are connected, and cannot intercept traffic for a long period of time. The two present lanes (single-sided 1 lane) are considered as mutual passing of single-sided 1 lane in a construction zone, and it is considered as a plan to arrange a traffic advisory.

(4) Exploitation of local materials and human resources.

Although there are few materials, skilled laborers, etc. who are needed there, it is considered to utilize the resources of Kiribati, and human resources as much as possible.

2.2.4.3 Scope of Works

The matter which the both-countries government should share is as in Table 2.2.4-1.

Table 2.2.4-1 Burden classification of the both countries government.

Item	Content	Responsible		Remarks
		Japan	Kiribati	
Land acquisition			○	
Procurement	Procurement of Materials and equipment	○		
	Custom clearance of materials and equipment	○	○	
Preparation	Reservation of a lot required for construction		○	Project office, equipment storage yard, workshop, etc.
	Other than the above	○		
Move of a construction obstacle	Move of an obstacle		○	A water pipe, a power line, a communication line
Main Construction	Causeway construction and bridge reinforcement work	○		Shore protection and pavement construction , repair, etc.

2.2.4.4 Construction Supervision Plan

Basically, the Japanese Consultant will enter into an agreement with the Republic of Kiribati to undertake the detailed design and construction supervision of the project.

(1) Major Works to be Undertaken

The major works to be carried out by the detailed design consultant are as follows:

- Undertake consultations with concerned authorities of South Sudan; field surveys,
- Detailed design and drawings preparation
- Project cost estimate

The duration to carry out the detailed design work is about 3.5 months.

(2) Bidding Activities

The major tasks to be undertaken from bid announcement to construction agreement include:

- Preparation of bid documents (in parallel with the detailed design).
- Bid announcement
- Pre-qualification of bidders
- Bidding
- Evaluation of bid documents
- Preparation of Contract Agreement

The duration of the bid-related activities is about 6.5 months.

(3) Construction Supervision

The Consultant will supervise the Contractor's planning and implementation of the construction contract. The major tasks under this stage include:

- Verification/Approval of related surveys and quantities
- Review/Approval construction plans
- Quality Control
- Process Control
- Work Output Control
- Safety Management
- Turnover Inspection and Acceptance

The duration of construction supervision is approximately 27 months.

The construction supervision team shall consists of: 1-Resident/Chief Engineer (Japanese), 1-Site Inspectors (Local),1-Clerk (Local).and 1-Utility Personnel (Local). A construction supervision engineer is dispatched at the time of construction of bridge repair and asphalt pavement.

A safety control officer is necessary to supervise, talk and cooperate with a construction contractor's safety manager so that occurrence of an accident may be prevented.

2.2.4.5 Quality Control Plan

The tasks to be carried out for quality control during the construction period are as follows:

- Concrete Works
- Reinforcing Bars and Formworks
- Earthwork
- Pavement Works

Based on the above, the quality control of main items for concrete works is presented in Table 2.2.4-2 while the quality control of main items for pavement is presented in Table 2.2.4-3.

Table 2.2.4-2 Concrete Quality Control Plan

Item	Test Items	Test Method (Specifications)	Test Frequency
Concrete	Cement Property/Physical Test	AASHTO M85	Once before trial mix and once every 500m^3 batch of concrete; or once during production of cement (Mill sheet)
Aggregate	Property/Physical Test	AASHTO M6	Once before trial mix and once every 500m^3 batch of concrete; and every change of source/quarry location (check supplier data)
	Property/Physical Test	AASHTO M80	Once before trial mix and once every 500m^3 batch of concrete; and every change of source/quarry location (check supplier data)
	Sieve Analysis	AASHTO T27	Once a month

Item	Test Items	Test Method (Specifications)	Test Frequency
	Alkali-silica Reactive Test(Mortar Bar Method)	ASTM C1260	Once before trial mix and every change of source/quarry location (check supplier data)
	Mineral Composition Test	ASTM C295	Once before trial mix and every change of source/quarry location (check supplier data)
Water	Water Quality Test	AASHTO T26	Once before trial mix and when necessary
Admixture	Quality Test	ASTM C494	Once before trial mix and when necessary (Mill Sheet)
Concrete	Slump Test	AASHTO T119	Once every 75m^3 or per batch
	Air Content Test	AASHTO T121	Once every 75m^3 or per batch
	Compressive Strength Test	AASHTO T22	6 Samples per batch or 6 samples for every 75m^3 of concrete (3 samples each for 7-day strength and 28-day strength)
	Temperature	ASTM C1064	Once every 75m^3 or per batch

Table 2.2.4-3 Quality Management Plan for Earthwork and Pavement Work

Item	Test Items	Test Method (Specifications)	Test Frequency
Embankment	Density Test (Compaction)	AASHTO T191	Every 500m^2
Base course	Material Test (Sieve Analysis)	AASHTO T27	Once before placing and once every 1,500m^3 or change in source/quarry location.
	Material Test (CBR Test)	AASHTO T193	Once before placing and once every 1,500m^3 or change in source/quarry location.
	Dry Density Test (Compaction)	AASHTO T180	Once before placing and twice every 1,500m^3 or change in source/quarry location.
	Field Density Test (Compaction)	AASHTO T191	Every 500m^2
Asphalt paving	Material Test (Sieve Analysis)	AASHTO M43,M80	Once before placing and once every 1,500m3 or change in source/quarry location.
	Material testing (density and percentage of absorption).	AASHTO T84	
	Density-in-situ examination.	AASHTO T209	Every 200m
	Temperature survey		Every track

2.2.4.6 Procurement Plan

(1) Procurement of Major Construction Materials

The available constructions materials which can be procured from Kiribati are coral aggregate for concrete. And, the available construction materials can be procured from third countries are aggregate for concrete, aggregate for asphalt paving and cement. Other materials will have to be procured from Japan. In addition, the Coral Sea material is supplied by 20\$/m^3 from a local government-related company. Table 2.2.4-4 presents the major construction materials for procurement.

Table 2.2.4-4 Procurement of Major Construction Materials

Item		Procurement Area			Procurement Reason	Procurement Routes
Item Name	Description	Local	Third Countries	Japan		
Materials for Structures						
Cement	40kg bag		○		Economic efficiency	Fiji→Betio port
Steel Sheet Pile	Type II			○	Quality and certainty,	Japan→Betio port
The escape prevention sheet of	A nonwoven fabric, t= 5 mm.			○	-Ditto-	Japan→Betio port

Item		Procurement Area			Procurement Reason	Procurement Routes
Item Name	Description	Local	Third Countries	Japan		
sand.						
Fabric Form	t= 20-30 cm			○	-Ditto-	Japan→Betio port
Reinforcing bars				○	-Ditto-	Japan→Betio port
Reinforcing bars	Epoxy Rasin-coated			○	-Ditto-	Japan→Betio port
Aggregate for concrete	Coral	○				
Aggregate for concrete	Crushed stone		○		Economic efficiency	Fiji→Betio port
Bottoming	Base, Sub Base- course	○				
Aggregate for asphalt concrete	Crushed stone		○		Economic efficiency	Fiji→Betio port
Straight asphalt			○		-Ditto-	Fiji→Betio port
Asphalt emulsion			○		-Ditto-	Fiji→Betio port
Gasoline		○				
Diesel oil		○				
Joint filler				○	Quality and certainty	Japan→Betio port
Admixture			○		Economic efficiency	Fiji→Betio port
Corrugated hard synthetic resin pipe	FEP ϕ =15cm			○	Quality and certainty	Japan→Betio port
Sand bag				○	-Ditto-	Japan→Betio port
Crack repairing material				○	-Ditto-	Japan→Betio port
Concrete repairing material				○	-Ditto-	Japan→Betio port
Temporary Materials						
Temporary Steel				○	Quality and certainty	Japan→Betio port
Large-sized sandbag				○	-Ditto-	Japan→Betio port
Plywood formwork				○	-Ditto-	Japan→Betio port

*The coral rock plan to be used to concrete for Fabric Form and parapet. More than 21N/mm^2 of the compressive strength of concrete with the coral rock was confirmed by concrete mix test in the project.

- Aggregate for cement, asphalt, and asphalt, aggregate for ordinary concrete
- These are not produced in Kiribati. It is considered as the supply from [from economic efficiency] Fiji.
- The third country and the port of discharge of a Japanese procured item.
- Supply materials from the third country which needs marine transportation, and Japan unloads in the approaching Betio Port.

(2) The machine for construction

There is no construction machinery market in a spot and the most possible is from Fiji, also most machines for construction are considered as a supply from Japan. Table 2.2.4-5 classifies and summarizes the necessary construction equipment for procurement for this project.

Table 2.2.4-5 Major Construction Equipment to be Procured

Item		Rent/ Buy	Where to Procure		Reason for Procurement	Procurement Route
Equipment	Specification		Local	Japan		
Back hoe	$0.28m^3$	Rent		○	Certainly	Japan→Betio port
	$0.45m^3$	Rent		○	-Ditto-	-Ditto-
	With a crane function.$0.45m^3$	Rent		○	-Ditto-	-Ditto-
	Super-long arm.$0.45m^3$	Rent		○	-Ditto-	-Ditto-
Dump Truck	10t Cap.	Rent		○	-Ditto-	-Ditto-
Bulldozer	15t	Rent		○	-Ditto-	-Ditto-
Road Roller	10～12t	Rent		○	-Ditto-	-Ditto-
Tire Roller	8～20t	Rent		○	-Ditto-	-Ditto-
Asphalt Finisher	2.4～6.0m	Rent		○	-Ditto-	-Ditto-
Agitator-Body Truck	4.4m3	Rent		○	-Ditto-	-Ditto-
Trailer	28tCap	Rent		○	-Ditto-	-Ditto-
Heavy Weight Breaker	Hydraulic Type 600~800kg class	Rent		○	-Ditto-	-Ditto-
Pay Loader	1.2m3	Rent		○	-Ditto-	-Ditto-
Truck Crane	35~50t	Rent		○	-Ditto-	-Ditto-
Truck with a Crane	2.9t、10tcap	Rent		○	-Ditto-	-Ditto-
Diesel Generator	20/25KVA	Rent		○	-Ditto-	-Ditto-
	125/150KVA	Rent		○	-Ditto-	-Ditto-
	350/400KVA	Rent		○	-Ditto-	-Ditto-
Mobile Concrete Pump	55～$60m^3$/hr	Rent		○	-Ditto-	-Ditto-
Concrete Plant	0.5m3	Rent		○	-Ditto-	-Ditto-
Aggregate Plant	Self-propelled Jaw crusher 10t class	Rent		○	-Ditto-	-Ditto-
Sieving Equipment	Portable screen 2 screen	Rent		○	-Ditto-	-Ditto-
Asphalt Plant	60t/hr	Rent		○	-Ditto-	-Ditto-
Vibratory Hammer	60kw	Rent		○	-Ditto-	-Ditto-
Seawater Desalination Plant	2m3/hr	Rent		○	-Ditto-	-Ditto-

2.2.4.7 Soft Component (Technical Assistance) Plan

(1) Background

Nippon Causeway (L=3.2km, W=11m) was constructed in 1985 as Batio-Bairiki Causeway Fisheries Channel Project by Japan grant. It is the only road to connect the international port at Betio island and the headquarters of administrative agencies and residential area at Bairiki island.

Aside from being old, the Causeway has incurred serious damages from king tide and strong tide brought about by the impact of climate change.

The subject roads are being maintained by MPWU. More serious collapse of revetment in the future will lose its road function due to no radical countermeasures such as protection of sand embankment, revetment repair and insufficient maintenance.

In order to have a sustainable efficiency, the Project will implement the repair and strengthening of the Causeway and adequately maintain after reconstruction. Therefore, a technical transfer will be proposed to implement the adequate operation and maintenance by MPWU.

(2) Objectives

The objective of the soft component is to implement the sustainable and effective operation and maintenance(O&M) of Nippon Causeway at Kiribati side. It is also expected that to achieve this above objectives is to effect appearance of the Japan grant project.

(3) Outputs (Direct Effects)

The Outputs to be achieve in this component are as follows:

- To understand the cause of damages and repair method for pavement and revetment
- To prepare the O&M manual for pavement and revetment
- To acquire the techniques for pavement and revetment maintenance by C/Ps

(4) Confirmation of Output

- Completion of operation and maintenance manual (included in the O&M organization and roles, inspection, repair method, etc.)
- Understanding of C/P by questionnaire

(5) Activities (Inputs)

About 10 maintenance staff in MPWU
O& M is managed directly by MPWU themselves at present. When MPWU needs the additional man-power, they contract with local community then hire the workers. It is expected that MPWU will be able to maintain the Causeway more adequately through the soft component's implementation. The soft component contains also the improvement of maintenance supervision for local community.

Table 2.2.4-6 Present Maintenance Level and Target Level for MPWU

	Pavement minor repair(Pot-hole repair)	Revetment minor repair
Present Maintenance Level	**No O&M plan** ➢ No inspection/cleaning ➢ Since there is no O&M Plan, repair works were done after the serious damage. Repair budget distribution was done after the serious damage. ➢ Since damages were left for long time, they tend to worsen. **Lack of repair technology** ➢ Currently pot holes were filled with a sand then compacted by manual.	**No O&M plan** ➢ No inspection. ➢ Since there is no O&M Plan, repair works were done after the serious damage. Repair budget distribution was done after the serious damage. ➢ Since damages were left for long time, they tend to worsen. **Lack of repair technology** ➢ Currently cavities were filled with sand then water-biding only. ➢ No use of chipping. Cement mortar was used for crack.(easy to remove the cement mortar)
Target Maintenance Level	**Conduct O&M based on the O&M Plan** ➢ Based on the O&M plan, inspection and repair will be implemented. ➢ To implement a proper O&M, necessary O&M budget will be secured in advance. ➢ Inspection and cleaning will be able to be implemented by MPWU. **Improvement of repair technology** ➢ A proper repair for pot hole(cleaning, use of cold asphalt and compaction) ➢ A proper supervision for local community	**Conduct O&M based on the O&M Plan** ➢ Based on the O&M plan, inspection and repair will be implemented. ➢ To implement a proper O&M, necessary O&M budget will be secured in advance. ➢ Inspection will be able to be implemented by MPWU. **Improvement of repair technology** ➢ A proper repair for revetment (chipping, cement mortar) ➢ A proper supervision for local community

To achieve above target, necessary activities will be done as follows

Experts:

Revetment Maintenance - 1 person, Pavement Maintenance - 1 person (Total 2.7M/M)

Activities:

Operation and maintenance manuals will be prepared in order to implement a sustainable O&M. After learning the road damaged causes and maintenance method through seminars, a site practice will be done in order to acquire the C/P's practical capability.

As Nippon Causeway will be reconstructed during soft component, site practice will be selected from other ordinary roads. Candidate locations are the asphalt pavement section with pot holes and the revetment section with many cracks.

- Formulation of O&M plan: 5days(Pavement - 5 days)
- Preparation of O&M manual: 10 days (Pavement and Revetment, 5days each)
- Preparation of Seminar: 4days (Pavement and Revetment, 2days each)

- Seminar: 4 days (Pavement and Revetment, 2days each)
- Practice for maintenance (pothole and revetment repair): 44 days（Pavement and Revetment, 22 days each）
- Others (summary of soft component expert transit):14 days

Table 2.2.4-7 Activity Schedule for Soft Component

	Expert (day)		Implementing Schedule		
Activity Items	Pavement	Revetment	2019 Jan.	2019 Feb.	2019 Mar.
Departure/Move	2	2	■		
Formulation of O&M Plan	5		■■		
O&M Manual Preparation(Revetment, Pavement Repair)	5	5	■■■		
Seminar Preparation	2	2	■		
Seminar(Revetment, Pavement Repair)	2	2	■		
Site Practice(Revetment, Pavement Repair)	22	22	■■■	■■■■■	
Summary of Soft component (inc. questionnaire)	2	2		■	
JICA Reporting/return Japan	4	2		■■	

(6) Procurement method of implementing resource

A technical transfer will be implemented by the Japanese Consultants. A counterpart from MPWU will be required in order to procure the construction machines and materials, and to support coordination with the related organization. This is the aim to implement the smooth soft component activities and to create the ownership of Kiribati side.

(7) Implementation Schedule of Soft Component

The implementation schedule of soft component is shown in Table 2.2.4-8.

(8) Output Materials

① Soft Component Plan Completion Report

② Manual of Operation and Maintenance for Asphalt Pavement, Manual of Operation and Maintenance for Revetment

(9) Responsibility of the Kiribati Side

MPWU is the responsible agency for the operation and maintenance of the Nippon Causeway reconstructed by this Project. To achieve the above goal of the soft component, the activities to be implemented by MPWU are as follows:

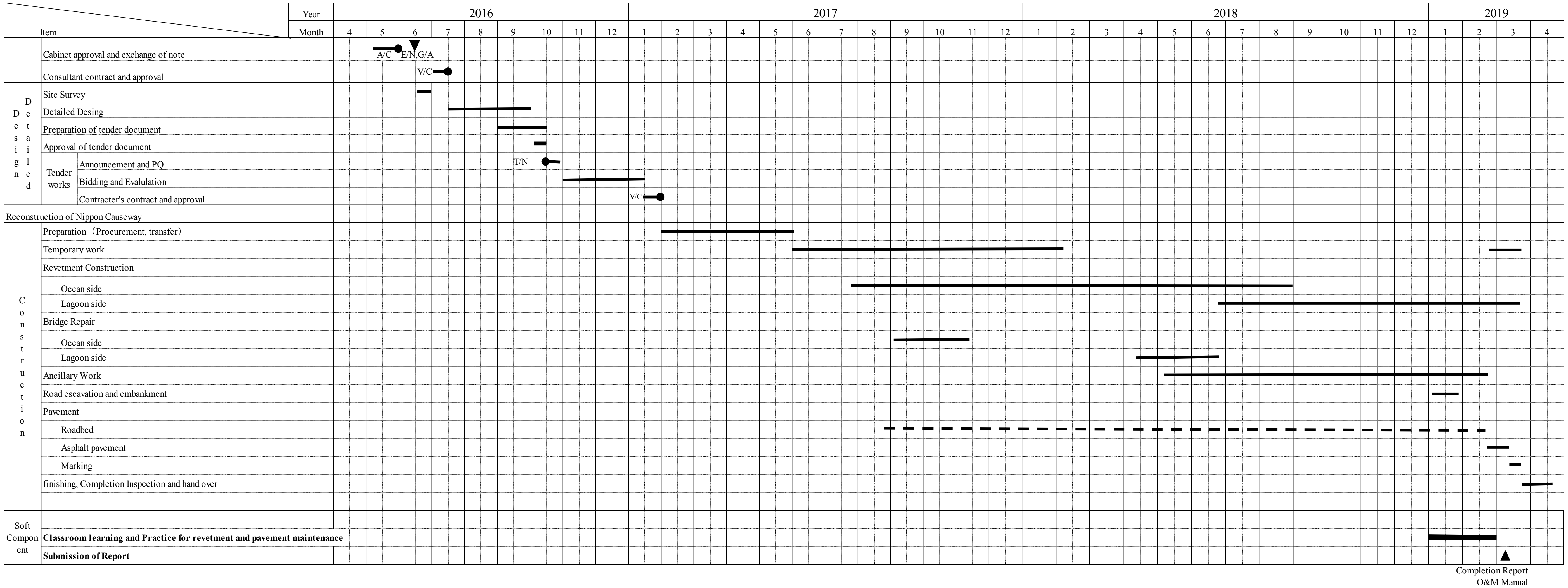
- Provision of Counter Parts (C/Ps) (from Civil Engineer Section's employee)
- Provision of training facilities (use of conference room in MPWU)
- Provision of work space for the Consultant
- Provision of materials (mortal etc.) for embankment and asphalt pavement's training
- Provision of construction machine to be use for embankment and asphalt pavement's training
- Provision of workers for embankment and asphalt pavement's training

It is important that the proper asset management impacts on the life span of the facilities and its maintenance cost.

After the reconstruction of the facilities, the Kiribati side will be required as follows

- To implement the inspection and cleaning of the facilities based on the O&M plan
- To secure the budget for the periodic inspection and maintenance

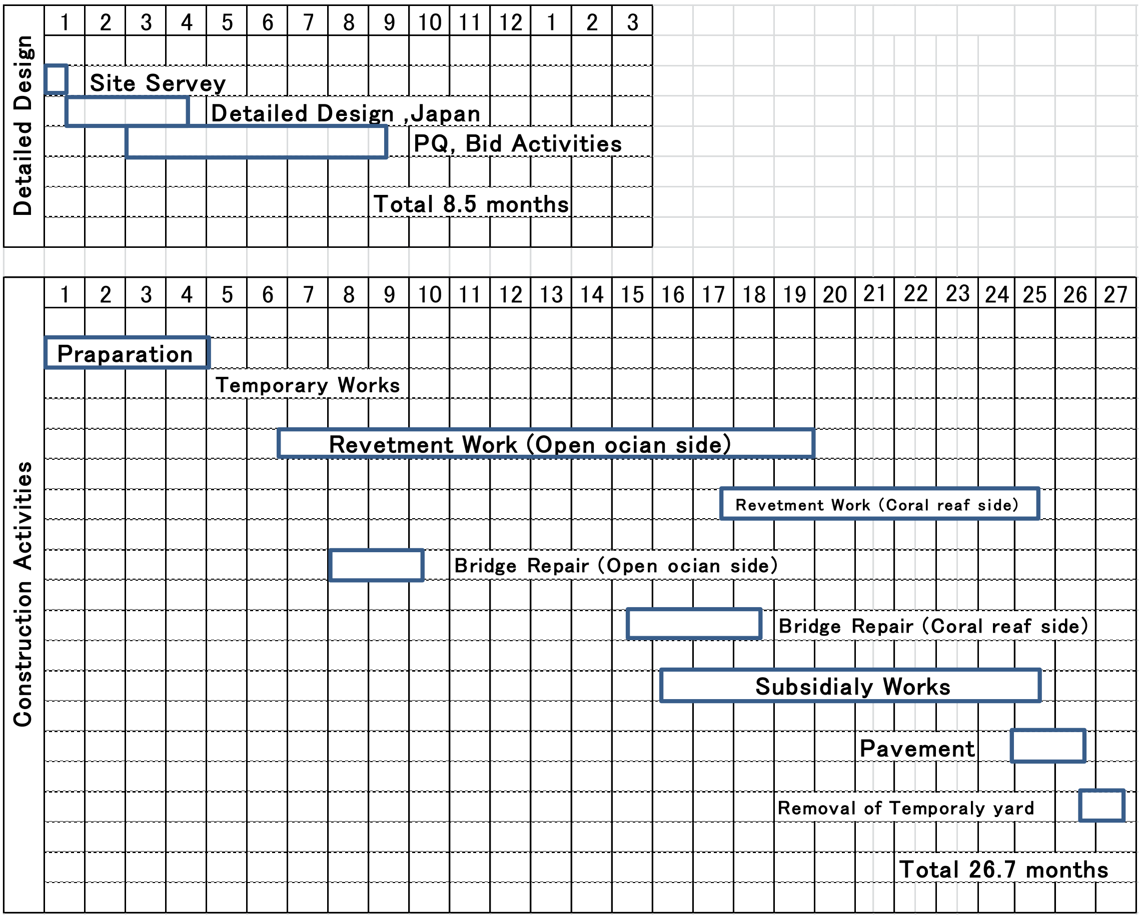
Table 2.2.4-8 Table Implementation Schedule



2.2.4.8 Implementation Schedule

Table 2.2.4-9 presents the overall implementation schedule for the detailed design and the project construction.

Table 2.2.4-9 Implementation Schedule



2.3 Obligations of Recipient Country

2.3.1 General Obligations under Japan's Grant Aid Scheme

The general undertakings of the Kiribati side in connection with the Project have been confirmed in the M/D agreed upon by the governments of the two countries. Their contents are briefly reproduced here.

- Securing of the land required for the Project
- Exemption of Japanese nationals from customs duties, internal taxes and other fiscal levies imposed in Kiribati in respect to the supply of products and services under the verified contracts
- Accordance of Japanese nationals and third country nationals (other than Kiribati nationals) whose services may be required in connection with the supply of products and services under the verified contracts such facilities as may be necessary for the entry into Kiribati and stay therein for the performance of their work

2.3.2 Specific Obligations under the Project

In addition to the general issues briefly mentioned above, there are some specific issues to be undertaken for the Project in view of the fact that it is a grant aid project.

2.3.2.1 Obtaining Permits for the Implementation of the Project

- Permit to implement the construction work for the Project
- Permit regarding the environmental impacts of the Project

2.3.2.2 Relocation of Obstacles (Buried Items such as Telephone Cables and Electric Cables) and Relocation of Street Lighting

- Concrete box for utilities such items as telephone cable, electric cable and water pipe will be constructed by the Japan side. The utilities material cost and the utilities installation cost will be owned by the Kiribati side.
- The streetlight is installed in the lagoon side along the Nippon Causeway. The foundation of streetlight will constructed by Japan side and the streetlight itself will be done by the Kiribati side. Since the existing streetlight is with sunlight panel model, it will be removed before work, be kept in the stock then after work it will be restored.

2.3.2.3 Temporary Yard

Land should be provided to accommodate the temporary construction yard of the Contractor.

2.3.3 Requests to the Recipient Country

The following requests will be made to the Kiribati side to ensure the smooth implementation of work.

2.3.3.1 Public Meeting to Explain the Project to Residents along the Nippon Causeway Sections

Following the official decision on the implementation of the Project with the signing of the E/N, the Ministry of Public Works and Utilities should organize a public meeting to explain the Project to residents along the target road sections or their representatives.

2.3.3.2 Traffic Safety

A publicity campaign should be conducted to ensure that ordinary road users follow the instructions of traffic controllers during the construction period.

2.3.3.3 Notification of Inconvenience during the Road Work

As the planned road work is expected to cause some inconvenience to road users, road users should be notified of inconvenience by means of the radio and other mass media.

2.4 Project Operation Plan

2.4.1 Operation and Maintenance Setup

Since the Civil Engineering Section (CES) of the Public Works Division, Ministry of Public Works and Utilities is the only agency able to implement road and causeway embankment maintenance in Kiribati.

The CES hardly conducts any new works but is mainly concerned with maintenance of existing facilities. It has thirty eight (38) staff members and is divided into the department in charge of Tarawa and the department in charge of outer islands.

Since Nippon Causeway are badly deteriorated and suffer from King Tide, the completed repair work is immediately followed by a need to repair other places. To combat this situation, the CES compiles an annual repair plan and conducts road repairs based on the said plan.

2.4.2 Maintenance Work following Project Implementation

In connection with the maintenance of the roads and concrete cutters in the post-project period, the following types of maintenance work will be required.

2.4.3 Routine Maintenance

The repair work, etc. which will be necessary all year round is listed below.

- Patching of the asphalt surface (pot hole patching)
- Base course repair if necessary
- Cleaning of drainage ditches and transverse drainage facilities
- Repair of fabrimat cracks

2.4.4 Periodic Maintenance

- Base course repair
- Overlay

At present, the above maintenance work is directly conducted by the Civil Engineering Section of the Public Works Division, Ministry of Public Works and Utilities and the present system should be sufficient. However, there is no asphalt plant in Kiribati, overlay works will done by foreign contractor. The key to good maintenance is the early detection of damage and the Civil Engineering Section is requested to conduct routine inspection and patrols as frequently as possible.

2.5 Project Cost Estimation

2.5.1 Initial Cost Estimation

This part is closed due to the confidentiality

2.5.1.1 Japan's Contribution

The table below indicates the costs borne by the Japan side.

Table 2.5.1-1 Approximate Cost Estimate of Japanese Contribution

This part is closed due to the confidentiality

2.5.1.2 Kiribati's Contribution

Table 2.5.1-2 Approximate Cost Estimation of Kiribati Contribution

Items		Cost (AU$)
(1)Bank Charges		8,224
(2)Relocation of Utilities	Water Pipe	340,000
	Communication Cable	170,000
	Electric Cable	170,000
	Subtotal	680,000
Total		688,224

2.5.1.3 Cost Estimation Condition

① Cost Estimate Date : September 2015

② Foreign Exchange Rate : A$ 1.00 = 94.36 Yen
US$ 1.00 = 124.40 Yen

③ Construction Period : Schedule of detailed design and construction supervision is shown in the schedule of implementation

④ Others: The project is carried out based on the Japanese Government's Grant Aid Scheme.

2.5.2 Operation and Maintenance Cost

The actual road maintenance members consist of only transport maintenance workers (4 persons) and Coastal Maintenance Workers (4 persons) under the Transport Engineer and Coastal Engineer shown in Figure 2.5.2-1.

2.5.2 Operation and Maintenance Cost

The actual road maintenance members consist of only transport maintenance workers (4 persons) and Coastal Maintenance Workers (4 persons) under the Transport Engineer and Coastal Engineer shown in Figure 2.5.2-1.

MPWU does not have budget for road maintenance. When a road maintenance is necessary, MPWU submits the request letter for road maintenance budget then will receive it from a Special Fund.

Once MPWU receives the road maintenance budget, they will contract with local community then who will hire the local people and conduct road maintenance work. Since there are no private road maintenance company in Kiribati, they utilize the local community for road maintenance.

Maintenance costs of the causeway (Embankment section and bridge section) are shown in Table 2.5.1-1 and Table 2.5.1-2.

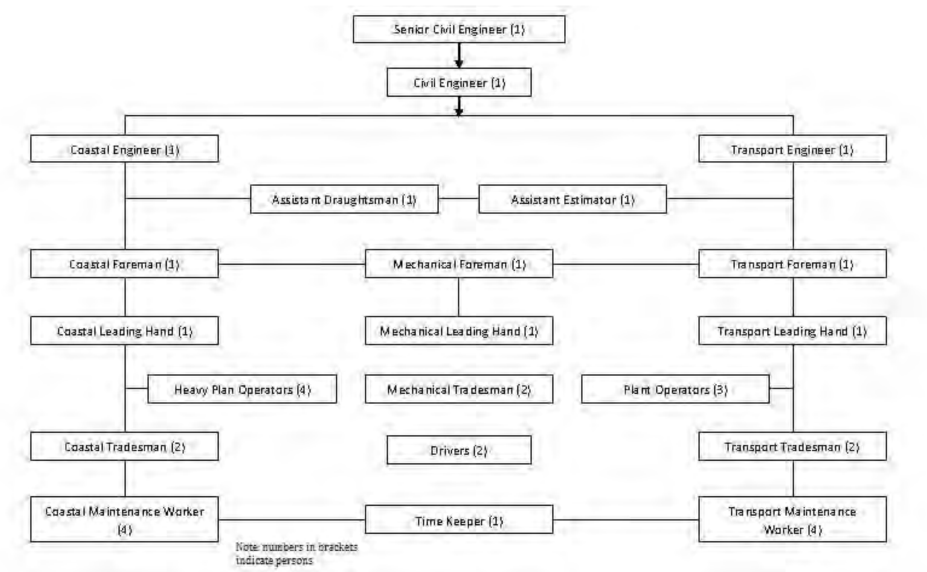


Figure 2.5-2-2 Organizational Chart of Civil Engineering Section

Table 2.5.2-1 Maintenance Items and Annual Cost of Existing Bridge

Items	Facilities	Inspection Items	Frequency	Personnel	Equipment	Total Number	Cost (AU$)
Periodical Inspection	Pavement	Crack etc.	12 times/year 1 day/time	2 persons	Scoop/Hammer/ Sickle/Barricade	24 persons/year	437
	Drainage	Sediment Deposition/Obstacle					
	Box Culvert	Damage/Deformation/Peeling etc.					
	Incidental Facilities	Railing			Pickup	24 vehicles/year	960
	Subtotal						1,397
Daily Inspection	Pavement	Cleaning	4 times/year 2 day/time	5 persons	Scoop/Barricade	40 persons/year	728
	Drainage	Removal of Obstacle or Sediment					
	Bridge	Cleaning			Small Truck	8 vehicles/year	960
	Subtotal						1,688
Repair	Pavement	Crack, etc.	1 times/year 4 day/time	6 persons	Worker	24 persons/year	437
	Drainage	Crack, etc.			Plate Compactor	4 vehicles/year	200
	Box Culvert	Crack, etc.			Small Truck	4 vehicles/year	480
	Incidental Facilities	Bridge Railing			Asphalt	1.0m^3/year	5,000
	Traffic Marking	Lane Marking			Lane Marking	5.0m/year	100
	Subtotal						6,217
Total							9,302

Table 2.5.2-2 Maintenance Items and Annual Cost of Embankment Section

Items	Facilities	Inspection Items	Frequency	Personnel	Equipment	Total Number	Cost (AU$)
Periodical Inspection	Pavement	Cracks etc.	12 times/year 1 day/time	4 persons	Scoop/Hammer/ Sickle/Barricade	48 persons/year	874
	Revetment	Cracks etc.					
	Drainage	Sediment Deposition/Obstacle			Pickup	12 vehicles/year	960
	Subtotal						1,834
Daily Inspection	Pavement	Cleaning	4 times/year 2 day/time	10 persons	Scoop/Barricade	80 persons/year	1,456
					Small Truck	16 vehicles/year	1,920
	Subtotal						3,376
Repair	Pavement	Crack, etc.	1 times/year 4 day/time	6 persons	Worker	24 persons/year	437
					Plate Compactor	4 vehicles/year	200
	Revetment	Crack, etc.			Small Truck	4 vehicles/year	960
	Incidental Facilities	Crack, etc.			Asphalt	2.0m^3/year	10,000
					Roadbed Material	30.0m^3/ year	2,250
					Lane Marking	12.0m/ year	240
	Subtotal						14,087
Total							19,297