MINISTRY OF ROADS AND PUBLIC WORKS
THE REPUBLIC OF KENYA

No.

BASIC DESIGN STUDY REPORT
ON
THE PROJECT FOR IMPROVEMENT
OF
THE NEW NYALI BRIDGE AND THE NEW MTWAPA BRIDGE
IN
THE REPUBLIC OF KENYA

August 2000

JAPAN INTERNATIONAL COOPERATION AGENCY
ORIENTAL CONSULTANTS CO., LTD.

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PREFACE

In response to a request from the Government of the Republic of Kenya, the Government of Japan decided to conduct a basic design study on the Project for Improvement of the New Nyali Bridge and the New Mtwapa Bridge and entrusted the study to the Japan International Cooperation Agency (JICA).

JICA sent to Kenya a study team from January 23 to March 7, 2000.

The team held discussions with the officials concerned of the Government of Kenya, and conducted a field study at the study area. After the team returned to Japan, further studies were made. Then, a mission was sent to Kenya in order to discuss a draft basic design, and as this result, 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.

I wish to express my sincere appreciation to the officials concerned of the Government of the Republic of Kenya for their close cooperation extended to the teams.

August, 2000

Kimio Fujita
President
Japan International Cooperation Agency

August, 2000

Letter of Transmittal

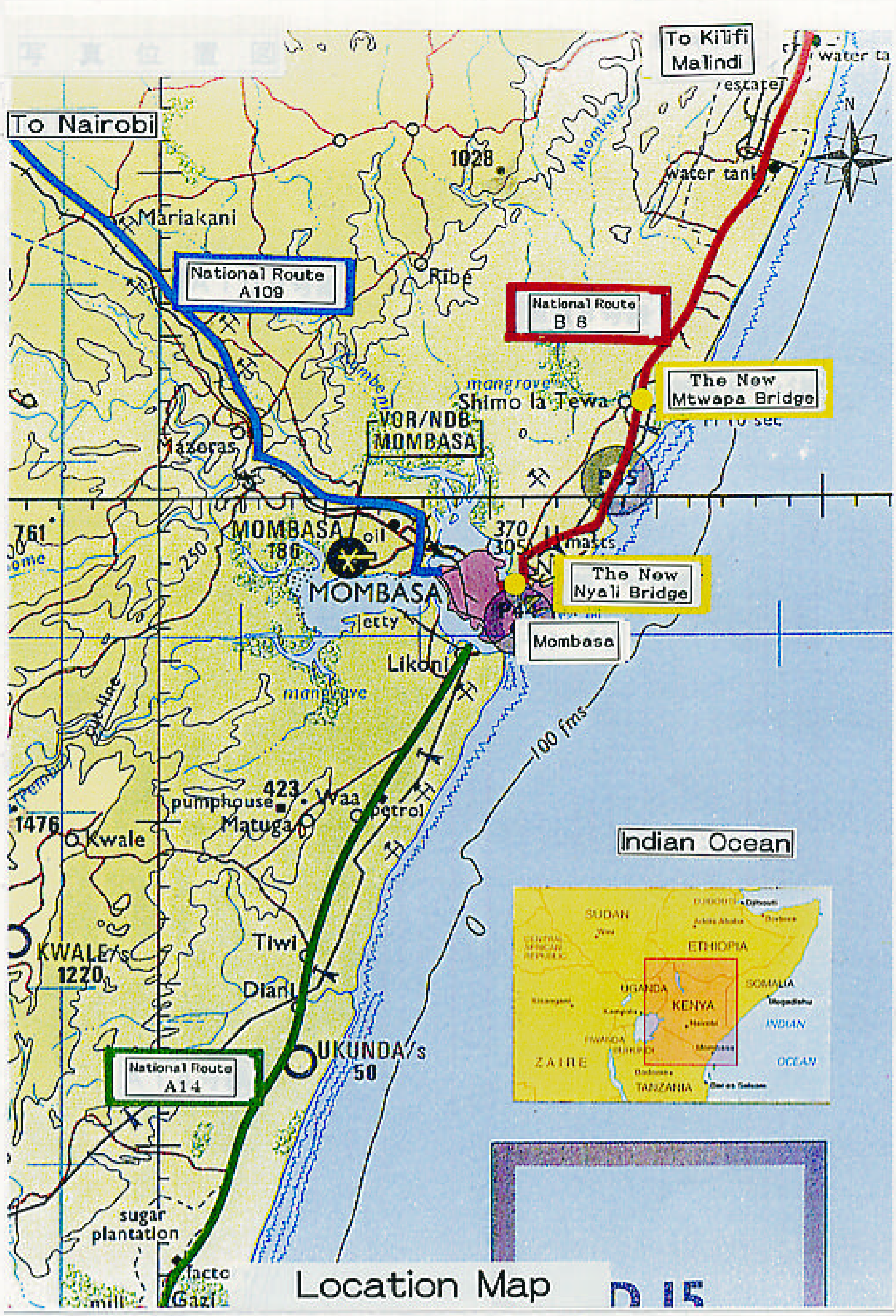
We are pleased to submit to you the basic design study report on the Project for Improvement of the New Nyali Bridge and the New Mtwapa Bridge in the Republic of Kenya.

This study was conducted by Oriental Consultants Company Limited, under a contract to JICA, during the period from January 20 to August 9, 2000. In conducting the study, we have examined the feasibility and rationale of the project with due consideration to the present situation of Kenya and formulated the most appropriate basic design for the project under Japan's grant aid scheme.

Finally, we hope that this report will contribute to further promotion of the project.

Very truly yours,

Keigo Konno
Project Manager,
Basic design study team on
the Project for Improvement on
the New Nyali Bridge and the New
Mtwapa Bridge in the Republic of Kenya
Oriental Consultants Company Limited



Location Map

NEW NYALI

NEW MTWAPA

PERSPECTIVE

Definition and Abbreviation

A Authorities and Agencies

ADB	African Development Bank
COMESA	Common Market for Eastern and Southern Africa
DAC	Development Assistance Committee
DANIDA	Danish International Development Authority
EC	European Communities
EEC	European Economic Community
EU	European Union
OECD	Organization for Economic Cooperation and Development
GNP	Gross National Product
IMF	International Monetary Fund
IDA	International Development Association
JBIC	Japan Bank for International Cooperation
JICA	Japan International Cooperation Agency
JRA	Japan Road Association
MORPW	Ministry of Roads and Public Works
NIES	Newly Industrializing Economies
OECF	The Overseas Economic Cooperation Fund
PWO	Provincial Work Office
RD	Road Department
RMI	Road Maintenance Initiative
SADC	Southern African Development Community
SIDA	Swedish International Development Agency
UNHCR	United Nations High Commissioner for Refugees

B Other Abbreviations

A	Area
AADT	Annual Average Daily Traffic
@	At the rate
B	Name of Live Load in Japan
B/D	Basic Design
BS	British Standard
C.L	Center Line
cm	Centimeter
cm^2	Square centimeter
D/F	Draft Final Report
DIN	Deutsche Industrie Normen
$	Dollar
Ec	Young's modules of concrete
Es	Young's modules of steel
Esp	Modules of elasticity
Ex	Existing
El	Elevation
H	Height
HB	Name of Live Load in BS5400
HWL	High water level
I	Coefficient of impact

Kgf/cm^2	Kilogram force per square centimeter
Kgf/cm^3	Kilogram force per cubic meter
Kgf/mm^2	Kilogram force per square millimeter
Ksh	Kenya shilling
K£	Kenya Pound
Km	Kilometer
Km^2	Square kilometer
Km/h	Kilometer per hour
KS	Kenya standard
L	Length
l	Length
LWL	Low water level
m	Meter
M	Million
m^2	Square meter
m^3	Cubic meter
m^3/s	Cubic meter per Second
MSL	Mean sea level
N	N-value or Number of wheel load application
n	Number of Ratio of Es to Ec
N/mm^2	Newton force per square millimeter
KN/mm^2	Kilo Newton force per square millimeter
%	Percent
Φ	Diameter
PC	Prestressed concrete
RC	Reinforced concrete
S	Scale
SD	Deformed Steel
σck	Allowable stress of concrete
σsa	Allowable stress of steel bar
t	Ton or Thickness
W	Width
W.L	Water level

CHAPTER 1

Chapter 1　Background of Project

Since 1963, Kenya has achieved high economic growth by fully utilizing its high education level and geographical conditions. However, due to an increase in the country's trade deficit caused by a price slump for major export products (coffee, etc.), economic difficulties began to surface in the 1980s. Although the government undertook seven national development plans to overcome the difficulty, economic growth could not be achieved. The government developed a new eighth national development plan by completely renewing conventional plans. This new development plan was different from the past plans dependent mainly on the agriculture. It aimed at both agriculture and industries with the aim of entering the ranks of the ranks of the Newly Industrializing Economies (NIES). This new plan placed its political stress on an increase in savings, expansion of investment, and the development of an investment environment in the private sector. The plan set up as important subjects the improvement of basic infrastructure, in particular, the road netowrk development, to support the national development plan, smooth physical flow, and activation of the national economy.

The Ministry of Roads and Public Works created a strategic plan for the road sector in 1997. Large-scale road sector plans in the past included the first phase from 1979 to 1983 in which stress was placed on road construction, maintenance, and traffic safety, and the second phase from 1984 to 1989 centered around strengthening of the organization and fostering of construction-related industries. In the 1990s, a more effective road network development plan was demanded to ensure a smooth flow of agricultural and industrial products to activate the economy and achieve the objectives of the national development plan. Against this background, a third road plan was proposed (Proposal for Funding of Kenya's Third Highway Sector Program). This plan established programs for the period from 1992 to 2000. The plan presented three items: ① maintenance and repair of existing roads, ② road construction for promotion of the regional development plan, and ③ construction of roads serving daily living to help enhancing the living standard of residents in the agriculture promotin region as top priorities.

National Route B8 of Kenya is a trunk road connecting Mombasa with a trade port where cargo handling is the largest in East Africa, and with Malindi, a principal city in the coastal area. The coastal area facing the Indian Sea north of Mombasa through which National Route B8 runs is the nation's largest tourism development area with several ten kilometers of sandy beach. Development of National Route B8 to ensure smooth traffic is the greatest objective of socioeconomic development for the entire coastal area facing the Indian Sea.

This National Route B8 crosses several creeks between Mombasa and Malindi. These points have become bottlenecks. For the purpose of ensuring a smooth flow of traffic, in particular, for construction of New Nyali, New Mtwapa, and Kilifi Bridges, where traffic congestion is extreme, the Kenya Government requested loan aid from the Japanese Government. Exchanges of note were concluded in 1975, 1977, and 1984. The former two bridges were completed in 1980, while Kilifi Bridge was completed in 1991. All three bridges are prestressed concrete bridges. The New Nyali Bridge is 330 m long and has six lanes; the New Mtwapa Bridge is 192 m long and has two lanes; and the Kilifi Bridge is 420 m long and has two lanes.

Upon completion of these three bridges, the traffic volume of National Route B8 (at the New Mtwapa Bridge) increased from 1,511 vehicles/day in 1978 to 3,417 vehicles/day in 1992. In particular, the number of trucks increased substantially since 1982 (756 trucks/day in 1978, 1,525 trucks/day in 1982, and 2,219 vehicles/day in 1992). Reinforced material transport capacity is therefore contributing to socioeconomic development of the coastal region.

However, maintenance of these bridges by the Kenya counterpart is not always satisfactory due to a worsening financial condition. The OECF's special post-evaluation of administration as part of monitoring and advisory services conducted in July, 1997 reported considerable damage. In particular, nearly 20 years have passed since construction of the New Nyali and New Mtwapa Bridges. The New Nyali Bridge is suffering damaged girders and piers, corroded bearings, deflected central span, damaged expansion joints, and cracked pavement. On the other hand, the New Mtwapa Bridge developed cracking in girders, deflection of the central span, and damage in the bearings. In both cases, urgent repair is necessary. These damages were detected during an inspection in 1992. They have expanded further in subsequent years. The Kilifi Bridge, which is still young after completion, showed detachment of the pavement.

Under these circumstances and on the basis of the above OECF post-evaluation and monitoring result, the Kenya Government requested grant aid from the Japanese Government to provide funds necessary for rehabilitation of the three bridges. The objective is to maintain National Route B8 (the trunk road in the Coast Province) in the sound condition over a long period. As regards the maintenance system of Kenya, which is directly responsible for above damages, the Kenya Government started, in 1998, reinforcements through reorganization, etc. on the basis of improvement proposals made after an OECF special post-evaluation.

In response to the request, the Japanese Government conducted a preliminary study in November, 1998. In this study, it was reconfirmed that the traffic volume at New Nyali, New Mtwapa, and Kilifi Bridges had increased to about 17,000 vehicles/day and about 3,700 vehicles/day respectively, indicating that construction of these bridges contributed greatly to the development of the regional economy. On the basis of the present state and cause of damage for three bridges, the investigation proposes early rehabilitation of New Nyali and New Mtwapa Bridge.

Only the pavement of the Kilifi Bridge has suffered damage. And it was proposed that the Kenya counterpart would be able to handle the repair. This study was made on the New Nyali Bridge and New Mtwapa Bridge on the basis of the results of a preliminary study.

CHAPTER 2

Chapter 2 Contents of the Project

2-1 Objectives of the Project

Since independence in 1963, Kenya initially achieved high economic growth thanks to a level of education and geographcal conditions. In 1980s, however, economic difficulties manifested themselves as trade deficits grew because of declining prices of major export products such as coffee, tea, etc., and despite the national targets that were set, economic and industrial became sluggish. Seeking solutions, Kenya has implemented seven national development plans, but each failed to lead to economic growth. Now, Kenya has newly developed an eighth national development plan by refurbishing past plans. The new plan has shifted its focus from conventional substantial dependence on agriculture to both agriculture and industry, aiming at joining the ranks of the NIES by the year 2020. This plan also stresses as policy targets the promotion of savings, expansion of investment, and development of an investment environment that will help the private sector. Furthermore, its major subject was to vitalize the national economy by developing infrastructure to support national development — in particular, the road network, and streamlining the flow of physical distribution.

MORPW developed a strategic plan for roads in 1997. Past large road projects can be divided into two phases: one from 1979 to 1983, involving mainly road construction, maintenance, and traffic safety, and the other was from 1984 to 1989, centered on strengthning the organization and fostering of construction-related industries. In the 1990s, the necessity of an effective road network development plan was acknowledged to raise efficiency of distribution of agricultural and industrial products and to vitalize the economy and achieve the targets of the national development plan. In line with this, the Proposal for Funding of Kenya's Third Highway Sector Program was made to develop the plan over the period from 1992 to 2000. Top priority was placed on (1) maintenance and rehabilitation of existing roads, (2) road construction to promote regional development, and (3) construction of roads serving daily life to enhance the living standard of residents in the agriculture promotion area.

Most bridges in Kenya have either been used for more than 30 years since construction, and are outdated, or they are small bridges located on the nationwide road network. Only a few bridges could be rehabilitated or reconstructed because of budgetary and technical restrictions. Remaining bridges are heavily damaged and have increased risk of collapse, and in certain locations where bridges have fallen, the people cross at shallow points, or when the rivers are dry during the dry season. Apart from this, many bridges are too narrow to meet growing traffic demand. Rivers of Kenya are mostly small-to- medium in size and overflow during the high-water period in the rainy season. Most of bridges are several dozen meters long.

The bridges covered by this study is located on National Highway B8, which runs north from Mombasa, the second largest city of Kenya and the largest trade port in East Africa, to a principal city of Malindi. All of development plans, including the Kenya's largest tourism development, agricultural development of the Tana River in the north, are totally dependent on whether this national highway can offer stable service. The traffic on this road is mostly mixed, including automobiles and heavy vehicles, bicycles, carts, and pedestrians. In particular, the section near the New Nyali Bridge has heavy traffic — daily traffic of about 40,000 vehicles which indicates the high necessity of stable service in terms of both residents mobility and physical distribution. However, in terms of maintenance, elaborate plans based on a long-term perspective, if established, were often postponed in implementation because of adverse financial conditions. Though easy maintenance, such as road repai, including repair of shoulders and patching of pot holes, is made, the strategy for bridge maintenance (including accessories and attachments) and specific maintenance know-how is insufficient.

As described above, the New Nyali and New Mtwapa Bridges are located in vital points of the Natonal Highway B8, a lifeline to support the traffic of Coast Province. Their rehabilitation is evidently essential to secure stable traffic, and guidelines for future bridge maintenance are needed in addition to rehabilitation.

For this project, the Kenyan Government requested the Japanese Government for grant aid for these bridges because of the difficulty of workability and maintenance. The objective of the project is to secure safe and smooth traffic through rehabilitation of the two bridges whose traffic volume is substantially growing in spite of damages due to deterioration and ageing unique to concrete.

2-2 Basic concept of the project

The original design was conducted by a German consultant, who applied the British Standard (BS) to the live load. However, the design standard for the allowable value is not clear. As a result of study on the live load and design value, it was found that these tended to be similar to cases based on the Japanese Standard. Therefore, it was decided to employ the Japanese standard as the design standard for repair and reinforcement. Concerning repair and reinforcement of locations where damage occurred, the optimum method is to be selected on the basis of comparative study in terms of structural durability, safety, economy and maintenance.

In consequence, the basic concept of this project was to ensure smooth traffic and to assure stable land transport in the area concerned to activate further the inter-regional economy by means of repair and reinforcement of damaged portions of the New Nyali and New Mtwapa Bridges.

2-3 Basic design

2-3-1 Present condition of the bridges

This section describes the present bridge condition obtained from the field survey and rearranges data and test results that are bases for design conditions of this project.

1) Structural data

a) New Nyali Bridge

This bridge was constructed, under an ODA loan from OECF (the present JBIC), by the Sumitomo Construction Co., Ltd. of Japan with design and construction supervision of a consultant, H.P. GAUFF of Germany during the period from February 1977 to July 1980. Bridge data are as shown below:

Bridge length : 391.65m

Width : 25.89m（driveway width 9.5m×2（6 lanes）＋sidewalk,curb 2.695m×2 ＋median 1.5m）

Type :

- Superstructure： Three-span continuous prestressed concrete box-girder bridge （90m+150m+90m）
  Three-span continuous reinforced concrete girder bridge (21.65m+20m+20m)
- Substructure： Abutment: Inverted T-type abutment x 2
  Pier: Wall-type pier x 5
- Foundation： Reinforced concrete piles （φ2000, φ1200）
- Attachments： Bearing support: Steel roller bearing support （Abutment, pier, and hinged-girder pier on the Mombasa side）
  Steel rocker bearing support （Abutment and pier on the Malindi side）
  Rubber bearing support （For all reinforced concrete girders excluding the abutment section）
  Expansion joint: Mauler joint
  Fence: Driveway side: Guard rail
  Sidewalk: Al railings

Construction method: Three-span continuous prestressed concrete box-girder bridge: Cantilever method
Three-span continuous reinforced concrete bridge: Erection with staging

Live load : class 60(DIN), 25 units of HA and HB of BS153

Materials :

- Concrete: Three-span continuous restressed concrete box-girder bridge and upper portion of piers : class 45(BS)
  others : class 30(BS)
- Steel bars : BS 1144
- Prestressing steel : 85/105（Dyvidag steel bar）

b) New Mtwapa Bridge

This bridge was constructed, under an ODA loan from OECF (the present JBIC), by the Sumitomo Construction Co., Ltd. of Japan with design and construction supervision of a consultant, H.P. GAUFF of Germany during the period from October 1978 to October 1980. Bridge data are as shown below:

Bridge length : 192m
Width : 12m (Driveway width 7.5m (2 lanes) + sidewalk, curb 2.25m)

Type :

- Superstructure: Three-span continuous prestressed concrete box-girder bridge (40m+112m+40m)
- Substructure:
  - Abutment: Inverted-T type abutment x 2
  - Pier: Wall-type pier x 2
- Foundation: Reinforced concrete piles (ϕ2000)
- Attachment:
  - Bearing support: Steel bearing support (abutment)
    Steel pin bearing support (Pier on the Malindi side)
    Steel roller bearing support (Pier on the Mombasa side)
  - Expansion joint: Transflex
  - Fence: Steel railings (hot dip galvanized)

Construction method: Three-span continuous prestressed concrete box-girder bridge

- Center span: Cantilever method
- Side span: Erection with staging

Live load: HA full loading and HB45 unit of BS153

Material:

- Concrete: Three-span continuous prestressed box-girder bridge and pier class 30(BS)
  Abutment class 25(BS)
- Steel bar: BS 1144
- Prestressing steels: 85/105 (Dyvidag steel bar)

2) Summary of survey results

a) Result of visual inspection

i) New Nyali Bridge

A summary of damages of the New Nyali Bridge is shown in Table 2.3.1 below.

Table 2.3.1 Damage condition of the New Nyali Bridge

Location	Major damages	Rank	Repair
Main girder	Crack, free lime, deterioration	B	Necessary
Pier	Crack, flaking, steel bar exposure	A	Necessary
Abutment	Crack	C	Necessary
Pavement	Dent, crack, rutting	A	Necessary
Bearing support	Rust, oil leakage	C	Necessary
Expansion joint	Main body deformation, noise	A	Necessary
Drain system	Faulty distribution pipe	B	Necessary
Railings	Loss of member, pillar deformation	A	Necessary

The content of ranks is shown in Table 2.3.3.

＜**Abutment on the Malindi side**＞

Only damage caused by cracking was observed.

Cracks due to shrinkage were remarkable on the wall front at 1/2, 1/4, and 1/8 points. The maximum crack width was 1.0 mm at the 1/2 point. This point corresponds to a position where the shrinkage rate is the largest because the wall surface is constrained with footing. The crack width at other points varies from 0.6 – 0.05 mm. No damage was observed on either east or west sides. These cracks require repair.

＜**Abutment on the Mombasa side**＞

No damage

＜**Pier at approach on the Malindi side**＞

No damage

＜**Intermediate piers on the Malindi side**＞

Only damage caused by crack was observed.

Damage was observed at 1/6, 1/3, 1/2, and 2/3 points from the east side on the Malindi side, with the width ranging from 0.1 to 0.3 mm. Damage was found to rise from the ground surface. On the Mombasa side, damage was observed in approximately the same points, with the width ranging from 0.05 to 0.5 mm. These cracks require repair.

＜Main pier on the Malindi side＞

Damage includes loosening and flaking of concrete, exposure and rusting of steel bars, and crack.

Loose and flaking of concrete and exposure and rusting of steel bars were found on the north, west, and south sides; all in the lower end of piers where the stress is the largest.

Cracks occurred on all north, south, east, and west sides. Uniform cracking was observed at 1/8 to 7/8 points on the north side, with the crack width ranging from 0.1 to 0.3 mm. On the west side, cracks of 0.1 – 0.25 mm width were found. Cracks on the south side were found in locations nearly similar to those on the north side. The crack width ranged from 0.1 – 0.3 mm. On the east side, cracks of 0.2 – 0.25 mm width occurred in upper and lower portions of opening. In the footing top surface, many cracks were found on west and south sides, with the width ranging from 0.1 to 0.5 mm. These cracks require repair.

＜Main pier on the Mombasa side＞

Damage includes loosening, flaking, and cracks of concrete.

Loosening and flaking of concrete occurred from the east portion of the south side to the east side, but the occurrence area is small.

Cracks occurred in all north, south, east, and west sides. Cracks on the south side were observed evenly from 1/8 to 7/8 points, with the width ranging from 0.05 to 0.3 mm. Similarly, cracks on the north side were distributed evenly from 1/8 to 7/8 points, with the width ranging from 0.05 to 0.4 mm, and slightly larger in amount than on the south side. On the east side, cracks of 0.5 to 1.0 mm were found in the southern portion. On the west side, 0.5 – 0.1 mm cracks were found in the upper and lower right portions of the opening. In the footing top surface, cracks are distributed evenly on the north side, with the width ranging from 0.1 to 0.3 mm. These cracks require repair.

＜Prestressed concrete superstructure, Deflection of main girder＞

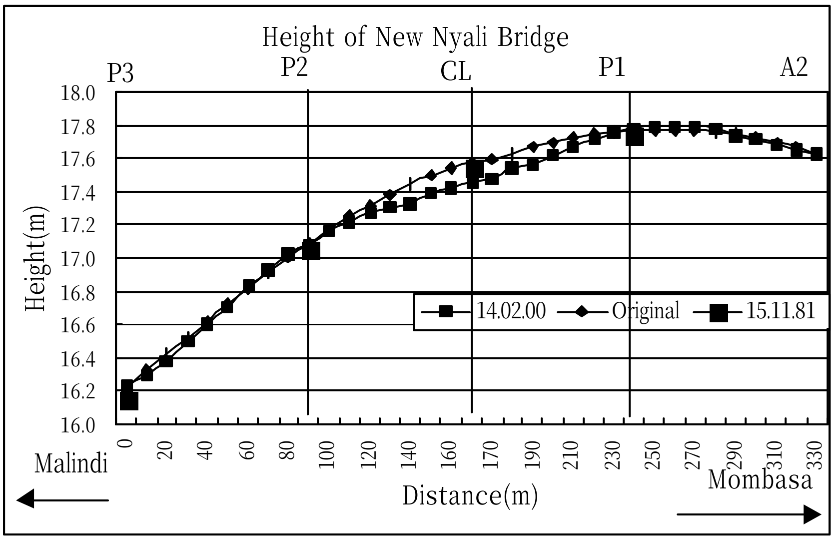


Fig. 2.3.1 Deflection of span central girder

As is known from Fig. 2.3.1, slight lowering tendency from the longitudinal alignment of original design is observed, resulting in 11 cm lowering from the original design in the middle of center span. The survey result of H.B GAUFF in December, 1981 showed 2.5 cm lowering from the original design. Though the camber is not clear, the present condition evidently indicates lowering from the original design. As crack occurrence is extremely limited, such lowering is not attributable to structural defects, but to the effect of creep. Since the longitudinal design is based on the up grade or forward and backward longitudinal grades are extremely gentle, 0.4% and 1.0% respectively, it is thought that calculated camber could not be provided during construction. The values on the side spans are approximately as designed.

＜Prestressed concrete superstructure, Crack, etc.＞

Damage found in upper and lower slabs in the east side box is limited to cracks. The number of cracks is small and their width ranges from 0.1 to 0.2 mm.

Damage in the upper and lower slabs in the west side box is limited to cracks. Many cracks occurred in the upper slab, with partial repair marks during construction detected in many points. The crack width ranges from 0.1 to 0.2 mm.

Web damage in the east side box is limited to cracks. Most occurred on the east portion. Similarly, web damage in the west side box is limited to cracks. However, on the center wall side, 0.1 to 0.2 mm cracks run over the entire length.

Almost no damage could be observed in the outside surface. It was confirmed however during concrete core sampling that water was leaking on the east side on the Mombasa side, which indicates that there are invisible cracks. These cracks require repair.

＜Expansion joint＞

The Mauler type expansion joint is used in the main bridge. On both sides, these joints suffer heavy damage, such as bending of the main body, loss of bolts, impact sound occurring due to front/rear faulting during passage of vehicles. They must be replaced as early as possible. The cut joint is used for the approach viaduct, but excessive cracks were observed in the pavement surface.

＜Bearing support＞

On the Mombasa side where roller shoes are used, the surface suffer rusting, resulting in failure of original functioning. On the Malindi side, rusting proceeded though fixed shoes are used. The same applies to other piers.

Rusting occurred in all cases, causing failure of original functions. It is therefore considered necessary to perform repairs, such as removal of rust and anti-rusting coating.

＜Drainage facilities＞

Sand and dust accumulation was found in many drain basins. Sand accumulation was also found on the shoulder. All drain pipes were short, with dripping and splash during draining being scattered directly onto concrete girders. It is therefore necessary to extend the drain pipes to a point below the girder.

＜Railings＞

Many protective fences against automobiles are bent, have lost bolts and lost bars. The same applies to the median. Railings show loss of vertical bars, bending, etc.

Damaged portions require appropriate measures, such as replacement.

＜**Pavement**＞

Pavement suffers almost all types of damage, including flaking off, potholes, cracks, settlement, etc. The pavement is intended to ensure smooth passage of vehicles and to protect structures. As damage, harmful vibration to girders or entry of rainwater into the girder may be expected. These damages must be repaired.

ii) New Mtwapa Bridge

The summary of damage of the New Mtwapa Bridge is shown in Table 2.3.2 below.

Table 2.3.2 New Mtwapa Bridge damage condition

Location	Major damages	Rank	Repair
Main girder	Crack, free lime, deterioration, deflection	A	Necessary
Pier	Crack, flaking, exposure of steel bar	C	Necessary
Abutment	Crack	A	Necessary
Pavement	Dent, crack, rutting	A	Necessary
Bearing support	Oil leakage	C	Necessary
Expansion joint	Deformation of main body, noise	A	Necessary
Drain system	Faulty drain pipe	B	Necessary
Railings	Loss of member, deformation of pillar	A	Necessary

The content of ranks is shown in Table 2.3.3.

＜**Abutment on the Malindi side**＞

Damages caused by crack were observed on west and east sides, with the width ranging from 0.1 to 0.3 mm. These cracks were also detected on the top surface and found to be continuous.

＜**Abutment on the Mombasa side**＞

Small cracks occurred on west and east sides. They were also found on the top surface, with the width being 0.1 mm to 0.5 mm in certain portions (east side).

＜**Pier on the Malindi side**＞

About 0.1 mm cracks occurred in the upper portion of the opening.

＜**Pier on the Mombasa side**＞

No damage

Table 2.3.3(a) Judgment table

Item	Rank	A	B	C	D
Principal concrete members	Crack	When concrete has broken away, possibly causing trouble in terms of traffic safety and to the third party	When the maximum width is 0.3 mm or more for RC and 0.2 mm or more for PC and the depth is 2/3 or more of the sectional dimension in the crack direction. When the maximum width is 0.3 mm or more and the crack interval is 50 cm or less. When the maximum width is 0.3 mm or more and the steel bar is rusting	When the maximum width is about 0.2 mm for RC and 0.1 mm for PC and the depth is 1/2 or more of the sectional dimension in the crack direction and when the crack interval is 50 cm or less.	When the maximum width is less than 0.2 mm for RC and less than 0.1 mm for PC or when there is no crack
	Breakaway and exposure of steel bar	ditto	When steel bar, prestressing steel, and prestressing sheath are exposed and when the prestressing steel anchorage is exposed. When the steel bar is corroded	When the steel bar is not exposed though breakaway has occurred.	When there is no breakaway and exposed steel bar.
	Deterioration, discoloration	ditto	When steel bar may be corroded or concrete may break away due to bar corrosion. When strength deficiency is detected with a Schmidt	When the strength is sufficient though deterioration has occurred.	When there is no deterioration
	Cavity, honeccomb	ditto	When steel bar, prestressing steel, and prestressing sheath are exposed When this defect has occurred near the prestressing steel anchorage	When steel is not exposed though cavity has occurred	When there is no cavity or honeycomb
	Water leakage	When there is a possibility of trouble to the third party	When there is water leakage from crack or concrete construction joint	When there is no possibility of steel corrosion though water	When there is no water leakage
Railing concrete	Exposure of steel bar and breakaway	—	When steel bar exposure and breakaway are 25% or more per span	When steel bar exposure and breakaway, if any, does not reach 25% per span.	When the damage is minor or when there is no damage
Steel railings	Film deterioration	When film deterioration is excessive	—	—	—
Drainage facility	Damage to member	Possibility of member falling or excessive water leakage	When the pipe is damaged or clogged with mud and when bracket is damaged	—	—
Splash preventive plate, blind plate, light shield fence, sound barrier, falling object preventive fence	Damage to member	When there is excessive deformation and corrosion	Corrosion or deformation of plate or net When the bolt is lost or loose	—	—
Bridge unseating preventive system	Damage to member	When there is excessive deformation and corrosion	When there is slight deformation and corrosion	—	—
Vehicle height limiting system		Excesive loss of members	Loss of member	Loose member	When there is no minor damage
Inspection approach		Hindrance to passage	Corrosion of mounting anchor bolts, damage, and water retension	ditto	ditto
Sign		Sign that may possibly fall	When a reflective sheet is fouled or wrinkled or the brown spot is fouled to damage visibility. When the sign board is bent or broken to damage visibility. When deformation and crack of pillar beam and rusting, breakaway, and blistering of anticorrosion coat exceeds 30%. Broken foundatio concrete and dislodgement of bolt pin	ditto	ditto
Cut joint, etc.		Similarly to breakage of the main body, when damage may possibly cause trouble to traffic safety or to the third party and repair must be made urgently. Abnormal sound	Dent around butt section D≧20mm l≧1,000mm W≧50mm Difference in butt D≧10mm Dislodgment of seal material l≧1,000mm	When damage is B or less and it is enough to record the damage	When there is no minor damage
Rubber joint		ditto	Loss of anchor member Dent and rise of expansion joint body D≧5mm H≧10mm Breakaway of post-placed concrete D≧20mm l≧300mm W≧100mm Breakaway of rubber l≧200mm Breakaway of filler in bolt holes Gap between pavement and post-placed concrete l≧3,000mm W≧5mm Water leakage from rubber connection Loose anchor bolt	ditto	ditto

Table 2.3.3(b) Judgment table

Item		A	B	C	D
Steel joint			Abnormality in operation of expansion joint Loss of anchor plate Damage to face plate weld Excessive clogging of drain trough with sand and damage Loss of finger Difference in finger D≧10mm Dent in post-placed concrete D≧20mm l≧300mm W≧100mm Gap between pavement and post-placed concrete l≧300mm W≧5mm	ditto	ditto
Bearing support (whole)		When the shoe body moves, rubber of rubber bearing support is broken, and there is a possibility of trouble to the third party and repair must be made	Excessive corrorion of bearing support as a whole Rubber deterioration of rubber bearing support	Rusting of rubber bearing support reinforcement	When there is no minor damage
Bearing support (bearing plate)		Breakage/crack.	Minor crack	Rust detected	ditto
Bearing support (lower shoe)		Breakage/crack	Minor crack	ditto	ditto
Bearing support (pin)		Breakage/crack.	Minor crack Rusting of rolling surface	Clogged with mud	ditto
Bearing support (roller)		Breakage/crack.	Lifting Minor crack Lost nut Rusting of rolling surface	Clogged with mud	ditto
Bearing support (sole plate)		Breakage/crack.	Minor crack Deformation	Rust detected	ditto
Bearing support (side block pinch plate)		Crack.	Minor crack Loss of bolt Breakage Crack	No allowance for interval. Loose bolt	ditto
Set bolt		Folding breakage	Rusting Dislodgement Loss	Loose bolt	When there is no minor damage
Anchor bolt		Dislodgement and cut. Loose nut. Short bolt.	Loss of washer. Inclination	ditto	ditto
Shoe seat concrete and mortar		Crushed. Heavy crack.	Minor crack Possible gap in shoe seat concrete	Minor crack. Faulty mortar grout	ditto
Shoe weld		Crack.	Excessive rusting	Minor rusting	ditto
Rutting amount Average rutting		Excessive rutting	20mm or more	Less than 20mm～15mm or more	Less than 15mm
Sum total of crack ratio		Escesive fissure	20% or more	Less than 20%～15% or more	Less than 15%
Corrugation and difference		Excessive difference	10mm or more	Less than 10mm～5mm or more	Less than 5mm

<**Prestressed concrete superstructure, Deflection of girder**>

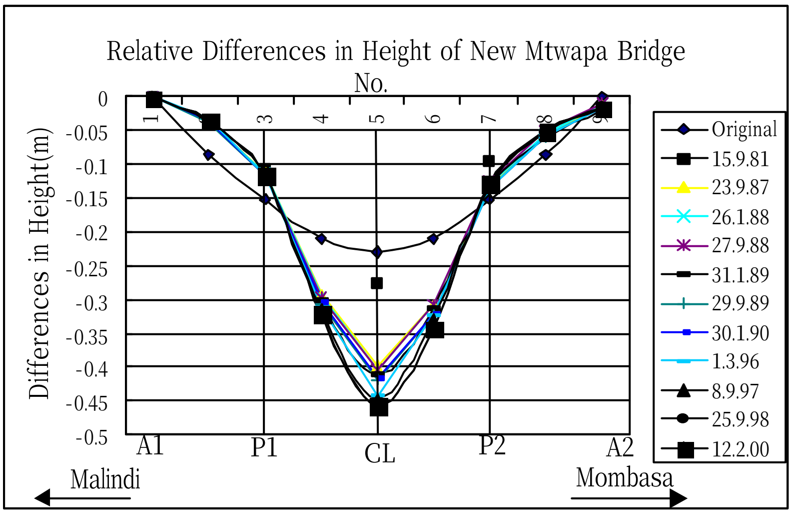


Fig. 2.3.2 Deflection of span center girder

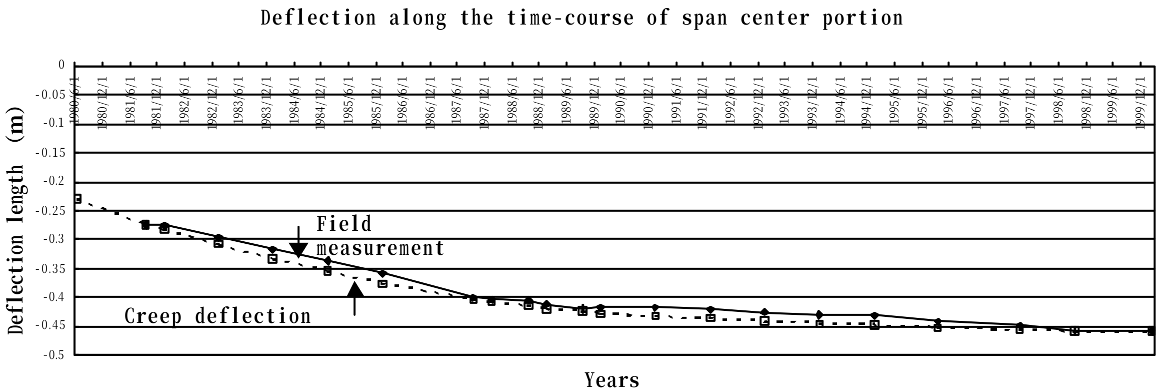


Fig. 2.3.3 Girder deflection along the time-course

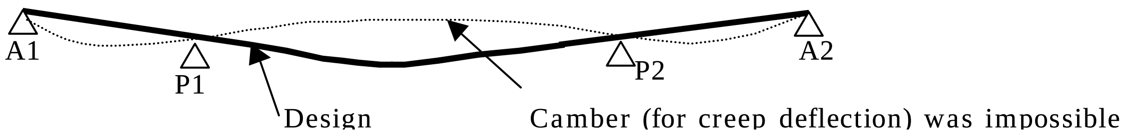
As is known from Figs. 2.3.2 and 2.3.3, 22 cm lowering from the original design was observed in the middle of span. The survey result of H.B. GAUFF in November, 1981 shows 4.4 cm lowering from the original design. For this bridge, surveys were conducted once or twice a year from 1987 to January 1990, then every year since January 1996. The results in September 1987 indicate already 17 cm lowering from the longitudinal alignment of original design, followed by further 5 cm or so in the subsequent 12 years. Namely, 77% of the total deflection occurred in seven years. The longitudinal alignment of the road is of a concave shape in the middle of the span, and creep unique to concrete caused longitudinal lowering of the main girder. In addition, measurement of the deflection under static loading during this survey resulted in approximate agreement with the calculation result. Therefore, decrease in rigidity of the girder cannot be considered. Moreover, the result of this survey is almost the same as the values obtained two years ago, so deflection is thought to have converged and will not proceed any more in the future.

According to the H.P. Gauff comments, the bridge design incorporated the two special elements cited below that ensure the economical adequacy of the bridge:

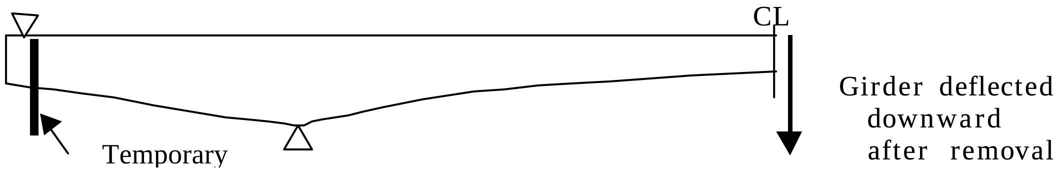
① Positioning of the sag in the center span to match the longitudinal alignment to the surrounding terrain
② Span ratio of 1 : 0.36 between center and side spans

These two points greatly affect the deflection of bridge.

① The ordinary design provides camber during construction to cope with the creep deflection. Since camber could not be provided because of the longitudinal alignment, the creep deflection became lower than the design longitudinal alignment.



② The construction proceeded in the order of erection of side spans with staging and cantilever construction of center span. As the side span was extremely short, causing uplift forces at a fulcrum of abutment during cantilever work, the prestressing steel bar was temporarily fixed to the abutment and removed when the center span was connected. It appears that change from the temporary PC steel bar fixing portion to the bearing support of abutment after removal of steel bar caused downward deflection of the center span (H.P. Gauff's comment). The deflection is considered to be approximately the value obtained from initial measurement.



Returning the deflection to the design longitudinal alignment will causes drastic change in the stress state. It will be effective if the present condition is retained as it is and repair/reinforcement is made to increase the strength.

＜Cracks, etc.＞

Most of damage is caused by cracks. Main girder web suffers many cracks in both side and center spans. Most cracks occurred at 1/3 and 2/3 points of the side span and 1/4 to central points of center span. The crack width ranges from 0.1 to 0.4 mm for side span and 0.1 to 0.3 mm for center spans. These cracks occurred mainly under influence of shear cracks.

The girder outside shows the effect of free lime in many points. Many cracks occurring on the inside are also observed on the outside and bottom side. In particular, most cracks observed on the outside may be due to penetration of cracks from the inside.

Inner slabs show cracks due to free lime or bending stress in the upper portion.

Therefore, reinforcement to prevent further cracks is necessary in addition to complete repair of existing cracks.

<Expansion joint>

The Mauler type expansion joint is used. On both sides, these joints suffer heavy damage, such as bending of the main body, loss of bolts, impact sound occurring due to front/rear faulting during passage of vehicles. Total replacement is considered necessary because of substantial structural load.

<Bearing support>

Shoes on the abutment are in positions where replacement is difficult. Internally, either the bolt is lost or bolts on the verge of dislodgement are bent. Rusting of bolts could also be confirmed.

On the pier, oil leakage was found on the Mombasa side. This damage must be eliminated during daily maintenance.

<Drainage facilities>

Catch-basins were not provided from the beginning. Though the drain pipe is embedded directly in the slab, accumulation of dirt and debris on the shoulder or rainwater splash to girders because of insufficient pipe length were observed.

Rainwater splashing to girder causes entry through cracks, resulting in rusting of steel bars, which further proceeds to flaking of concrete. It is necessary to provide the lead pipe for extension to the point below the girder to prevent direct splashing of rainwater from drain pipes.

<Railings>

Partial loss of members was observed. In addition, bent pillars and bolt loss were also found. Deflection of the main girder caused deflection of railings. Therefore, replacement of members is necessary.

<Pavement>

Pavement suffers almost all types of damage, such as stripping, potholes, crack, settlement, etc. Pavement is intended to ensure smooth passage of vehicles and to protect structures. With these damages, harmful vibration to girders or entry of rainwater into the girder may be expected, and this damage must be repaired.

b) Result of compressive strength and elastic modulus tests with sampled concrete core

i) Concrete core sampling points

Concrete core sampling points are shown in Fig. 2.3.4 and Table 2.3.4. Concrete core was sampled with a boring diameter ofφ75mm for the box-girder and φ100mm for pier and abutment. Nondestructive test with a Schmidt hammer was made at the same points. In particular, this test was conducted additionally on P4 and P5 piers of the New Nyali Bridge.

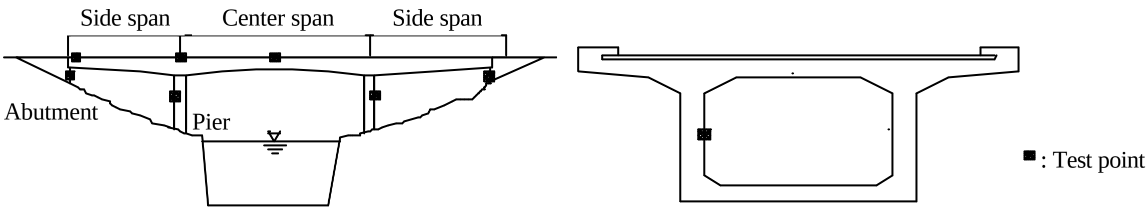


Fig. 2.3.4 Concrete core sampling and test points

Table 2.3.4 Concrete core sampling/test points Unit：pieces

		East	West	South	North	Total
New Nyali Bridge	A1 abutment			○		2
	P1 pier					—
	P2 pier					—
	P3 pier			○		2
	P4 pier					—
	P5 pier					—
	A2 abutment				○	2
	Box-girder	○				6
	Sub total	6	—	4	2	12
New Mtwapa Bridge	A1 abutment			○		2
	P1 pier			○		2
	P2 pier				○	2
	A2 abutment				○	2
	Box-girder	○				6
	Sub total	6	—	4	4	14
Total		12	—	8	6	26

ii) Nondestructive test with Schmidt hammer

The test results are attached as reference material. The nondestructive test with the Schmidt hammer showed concrete compressive strength of 70% of specified value, particularly, at the superstructure. The reason may be stain of the concrete surface of superstructure that affected the rebound hardness. Note that the specified compressive strength could be approximately confirmed with the substructure.

iii) Laboratory compressive strength/elastic modulus tests

The concrete compressive strength was above the specified strength, proving satisfactory. The elastic modulus test result was lower than the specified value. The test was conducted according to the concrete standard specification and standard (Japan Society of Civil Engineers: 1991 edition). As the height and diameter of concrete core sampled this time were nearly equivalent, the accuracy of compensated elastic modulus was poor, and the test result was lower than the specified value. Considering however that the concrete compressive strength is above the specified value, it may be assumed that the concrete quality is retained even now.

c) Carbonation test

To check for carbonation of concrete, a test was made in the field using a phenolphthalein solution. Concrete test pieces for the test were those sampled for the compressive strength test. The carbonated thickness determined from the test is shown in Table 2.3.5. Carbonation at a thickness of about 11 – 12 mm was detected in the superstructure-pier fulcrum point of both New Nyali and New Mtwapa Bridges. But no carbonation was found in other places. It may be due to dissipation of alkaline components due to certain effects during sampling of concrete core. As the progress of rusting was not observed in the bar corrosion test described later, carbonation will not present any critical problem. The test condition and result at each sampling point are shown in the attached reference material.

d) Bar corrosion inspection

To check for bar corrosion in concrete, the inspection was made in the field according to the natural current method. Inspection was made at points where the concrete core was sampled, and additionally at piers within the creek of the New Nyali Bridge. For corrosion ranks, the test results are shown in Table 2.3.6. In both New Nyali and New Mtwapa Bridges, corrosion ranks II and III were predominant, with only a few dot rusts or thin loose rust on the bar surface. The corrosion rank IV was observed in the P4 pier of the New Nyali Bridge and between the superstructure and side span on the Malindi side of the New Mtwapa Bridge. In particular, the New Nyali Bridge has steel bars exposed in portions where pier concrete broke away, so that repair such as concrete chipping and rust removal from steel bar is necessary for damaged portions. As corrosion was only limited in side span of the New Mtwapa Bridge, it is necessary to take measures to prevent rusting of steel bars, such as injection of repair agent in sufficient amount into cracks, etc.

e) Result of chloride content measurement of concrete

To check the chlorine content in concrete, a part of concrete core sample was brought back to Japan to measure the soluble chloride. For the test method, the Chlorine Content Analysis Method in Hardened Concrete, JCI-SC4 (Japan Concrete Institute) and the Exposure Test and Research on Salt Damage of Concrete Bridges (1) (July, 1982) that is reference literature of the Public Works Research Institute, Ministry of Construction.

In the test, the test piece was heated to 50°C and kept heated with hot water at 50°C, then immersed for 30 minutes to extract the soluble chlorine. After leaving the test piece in the heated condition gently, the solution was filtered. Subsequently, part of the filtered solution was batched off, to which the nitric acid solution (2N) was added to acidify the solution. The solution is then set to a potentiometric titration device with a chlorine ion selective electrode for potentiometric titration with the N/200 silver nitrate standard solution. Note that the test piece was subjected to extraction three times.

As a result of measurement of chlorine, the chlorine content was found to be extremely small as shown in Table 2.3.7. This will not affect adversely the bridge structures.

Table 2.3.5 Carbonation thickness test result

New Nyali Bridge

	Core No.	Location	Measuring point (mm)				Average Carbonation Thickness
			A	B	C	C	
Super-structure	N-SP-SS-E I	Box girder inside side span	0	0	0	0	0
	N-SP-SS-E II	Box girder inside side span	0	0	0	0	0
	N-SP-SP-E I	Box girder inside pier fulcrum	7	11	8	9	9
	N-SP-SP-E II	Box girder inside pier fulcrum	12	13	9	11	11
	N-SP-CP-E I	Box girder inside center span	0	0	0	0	0
	N-SP-CP-E II	Box girder inside center span	0	0	0	0	0
	N-SP-CP-E III	Box girder inside center span	0	0	0	0	0
Sub-structure	N-SB-A1-N I	A1 abutment	0	0	0	0	0
	N-SB-A1-N II	A1 abutment	0	0	0	0	0
	N-SB-A2-S I	A2 abutment	0	0	0	0	0
	N-SB-A2-S II	A2 abutment	0	0	0	0	0
	N-SB-P3-S I	P3 pier	0	0	0	0	0
	N-SB-P3-S II	P3 pier	0	0	0	0	0

New Mtwapa Bridge

	Core No.	Location	Measuring point (mm)				Average Carbonation Thickness
			A	B	C	C	
Super-structure	M-SP-SS-E I	Box girder inside side span	0	0	0	0	0
	M-SP-SS-E II	Box girder inside side span	0	0	0	0	0
	M-SP-SP-E I	Box girder inside pier fulcrum	12	14	13	10	12
	M-SP-SP-E II	Box girder inside pier fulcrum	10	13	12	13	12
	M-SP-CP-E I	Box girder inside center span	0	0	0	0	0
	M-SP-CP-E II	Box girder inside center span	0	0	0	0	0
Sub-structure	M-SB-A1-S I	A1 abutment	0	0	0	0	0
	M-SB-A1-S II	A1 abutment	0	0	0	0	0
	M-SB-A2-N I	A2 abutment	0	0	0	0	0
	M-SB-A2-N II	A2 abutment	0	0	0	0	0
	M-SB-P1-S I	P1 pier	0	0	0	0	0
	M-SB-P1-S II	P1 pier	0	0	0	0	0
	M-SB-P2-N I	P2 pier	0	0	0	0	0
	M-SB-P2-N II	P2 pier	0	0	0	0	0
	M-SB-P2-N III	P2 pier	0	0	0	0	0

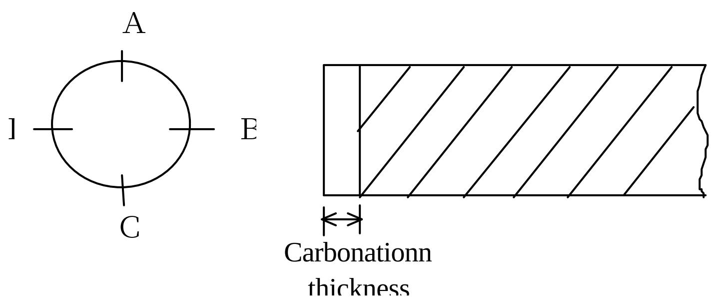


Table 2.3.6 Steel bars corrosion inspection (Self-potential method)

New Nyali Bridge

	Measuring No.	Measuring point	Potential E (mV)	Corrosion ranks
Super-structure	N-No.1	Main girder inside, side span	-250≧E>-350 -350≧E>-450	**II・III**
	N-No.2	Main girder inside, pier fulcrum	-250≧E>-350 -350≧E>-450	**II・III**
	N-No.3	Main girder inside, center span	-250≧E>-350	**II**
Sub-structure	N-No.4	A1 abutment	-250<E -250≧E>-350	**I・II**
	N-No.5	A2 abutment	-250≧E>-350 -350≧E>-450	**II・III**
	N-No.6	P3 pier	-250≧E>-350 -350≧E>-450	**II・III**
	N-No.7	P4 pier	-350≧E>-450 E≦-450	**III・IV**

New Mtwapa Bridge

	Measuring No.	Measuring point	Potential E (mV)	Corrosion ranks
Super-structure	M-No.1	Main girder inside, side span	-250<E E≦-450	**I・IV**
	M-No.2	Main girder inside, pier fulcrum	-250≧E>-350 -350≧E>-450	**II・III**
	M-No.3	Main girder inside, center span	-250≧E>-350 -350≧E>-450	**II・III**
Sub-structure	M-No.4	A1 abutment	-250≧E>-350 -350≧E>-450	**II・III**
	M-No.5	A2 abutment	-250<E -250≧E>-350	**I・II**
	M-No.6	P3 pier	-250≧E>-350	**II**
	M-No.7	P4 pier	-350≧E>-450	**III**

Steel bar corrosion evaluation standard

Potential E (mV)	Corrosion condition	Corrosion rank
-250<E	Black skin without corrosion	**I**
-350<E≦-250	Few spot rusting in bar surface	**II**
-450<E≦-350	Thin loose rust over the bar surface, with rust adhering to concrete	**III**
E≦-450	Slightly thick expansive rust, with relatively few sectional loss	**IV**
E≦-450	Excessive expansive rust over the entire bar, with sectional loss	**V**

Table 2.3.7 Soluble chlorine measurement result

Unit :cl- kg/m3

Core size (φmm)	Core No	Extraction frequency			Total
		1st	2nd	3rd	
50	2	0.035	0.021	0.013	0.069
50	3	0.040	0.016	0.010	0.066
50	5	0.026	0.014	0.009	0.049
50	7	0.029	0.014	0.008	0.051
50	10	0.025	0.015	0.009	0.050
50	12	0.038	0.010	0.006	0.054
100	1	0.065	0.034	0.021	0.121
100	3	0.019	0.018	0.011	0.048
100	5	0.027	0.016	0.010	0.053
100	7	0.026	0.014	0.009	0.049
100	9	0.022	0.016	0.010	0.049
100	11	0.026	0.009	0.006	0.041
100	13	0.021	0.021	0.013	0.055

f) Actual bridge vibration test results

The vibration test was conducted to determine the damage condition of the New Mtwapa Bridge and to obtain basic data for future rehabilitation/reinforcement and maintenance.

The test involved microtremor measurement under no load, a vehicle drop test to apply impact vibration to the bridge body by dropping a vehicle from a ladder, and a vehicle running test to measure vibration while vehicles are running to understand vibratory characteristics, such as natural frequency. The test flow is shown in Fig. 2.3.5.

Table 2.3.8 shows the analytical results using the natural frequency and concrete elastic modulus determined by the test as parameters. Analysis was made by comparing three cases of elastic modulus (Ec) of values obtained by the elastic modulus test using concrete core (Ec=23kN/mm^2), design value (Ec=28kN/mm^2), and the value (Ec=33kN/mm^2) estimated from the strength, 51N/mm^2, of actually placed concrete while assuming that the entire section is effective. The relationship between the test and analytical values is shown in Fig. 2.3.6. Analytical results show that the natural frequency in the deflection direction falls between the analytical values using the elastic modulus test result and those using design values. From the above, it may be determined that, within a range estimated from the vibration test result, no remarkable deterioration of rigidity occurred unless various assumptions are taken into account.

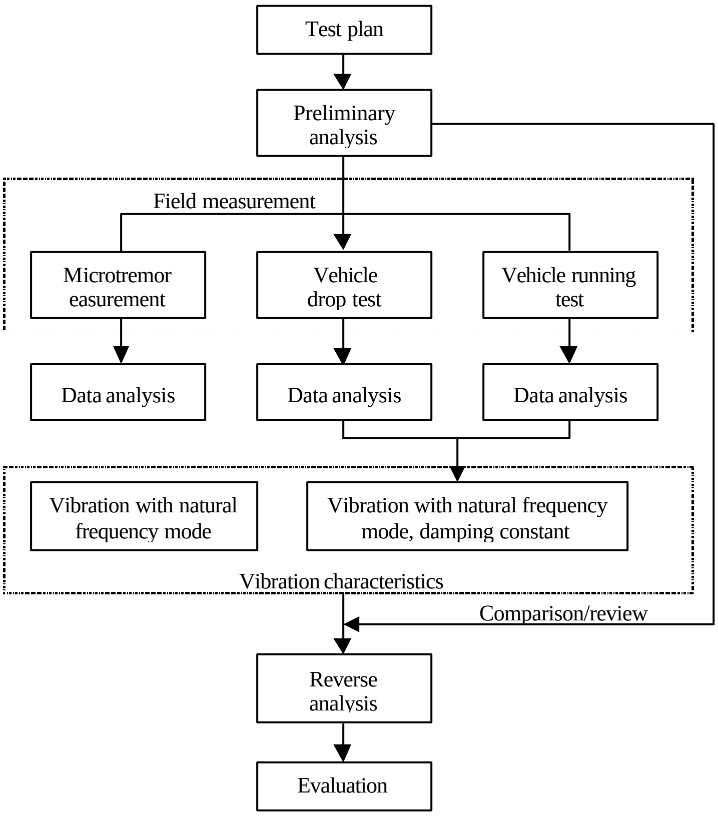


Fig. 2.3.5 Test flow

Table 2.3.8 Result of comparative review of vibration test

Vibration mode	Natural frequency (Hz)			
	Result	Analytical value		
		$Ec=23kN/mm^2$	$Ec=28kN/mm^2$	$Ec=33kN/mm^2$
Deflection, symmetrical primary mode	1.30	1.19	1.31	1.43
Deflection, antisymmetic primary mode	2.86	2.76	3.04	3.31
Horizontal symmetric primary mode	1.91	2.00	2.21	2.40

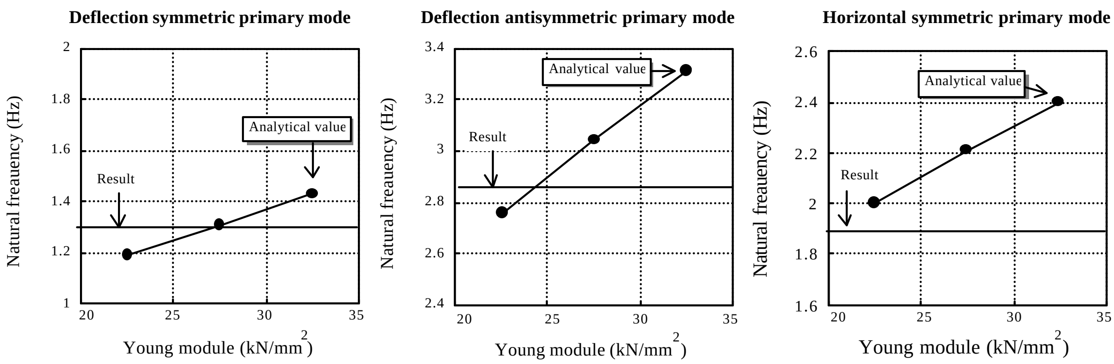


Fig. 2.3.6 Relationship between test and analytical values

g) Traffic volume

The results of the traffic volume investigation show that through traffic on the New Nyali and New Mtwapa Bridges is growing annually. The results are outlined below.

① Passenger cars and minibuses account for 81% of the daily traffic volume for both sides of the New Nyali Bridge. Utilization by pedestrians, bicycles, and carts is also high. Daytime traffic accounts for a high of 70 – 75%, but heavy vehicles ratio is as low as 3.6%.

② Morning peak traffic occurs 7:00〜9:00 a.m. in the direction of Kilifi→Mombasa. Evening peak occurs 17:00〜19:00 p.m. in the direction of Mombasa→Kilifi. The Mombasa Island area functions mainly as a place of business, while the Kisauni area（Nyali area, around Routes B8 and E949）is principally residential.

③ The distance between the New Nyali and New Mtwapa Bridges is only 11 km, but their daily both-side traffic volumes varies greatly: 38,263 vehicles for the New Nyali Bridge and 8,275 vehicles for the New Mtwapa Bridge. It is evident from this fact that the population of Mombasa extends up to the New Mtwapa Bridge.

④ As is shown in Table 2.3.9, the traffic volume has grown at a considerably high rate in recent years. This can be seen by this survey result and past data for the New Mtwapa Bridge. The growing trend is particularly remarkable with minibuses, passenger cars, and heavy freight cars.

Table 2.3.9 Traffic volume on the New Mtwapa Bridge

(Both-side traffic volume per day)

	Passenger car	Light freight car	Medium freight car	Heavy freight car	Bus	Total
1984 ＊	1,027	859	383	68	173	2,510
1988 ＊	824	1,375	272	60	162	2,693
1992 ＊	1,001	1,802	344	73	197	3,417
2000	2,701	4,967	197	264	125	8,275

（Source：＊Preparatory Study Team report）

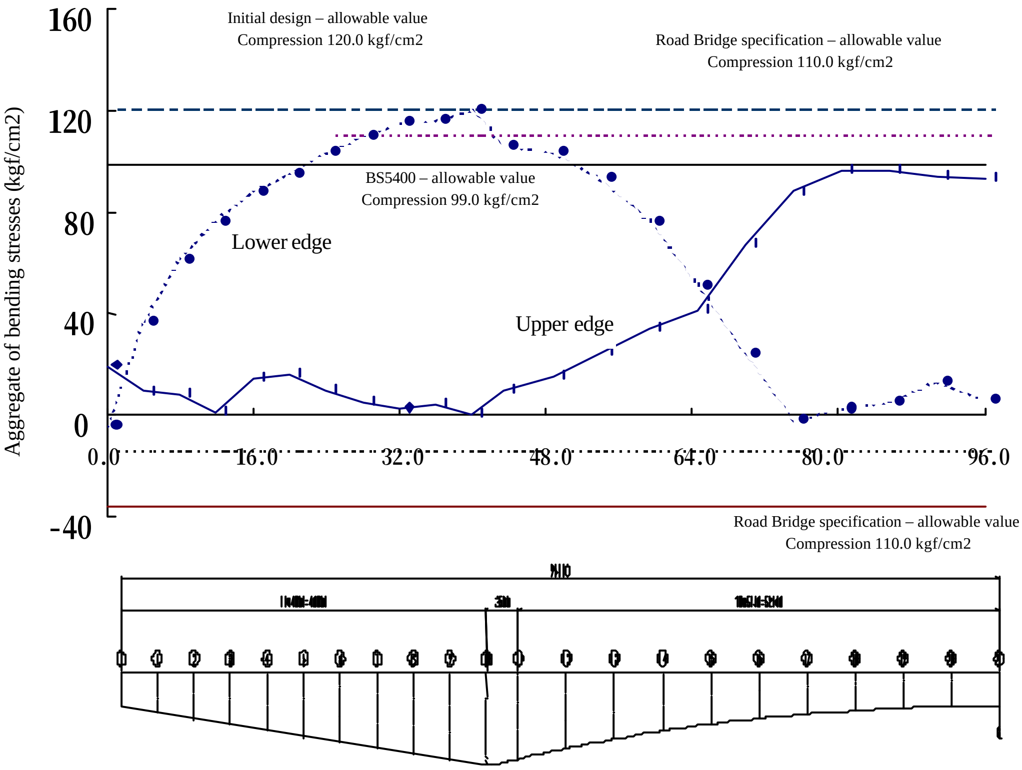
3) Design and construction history

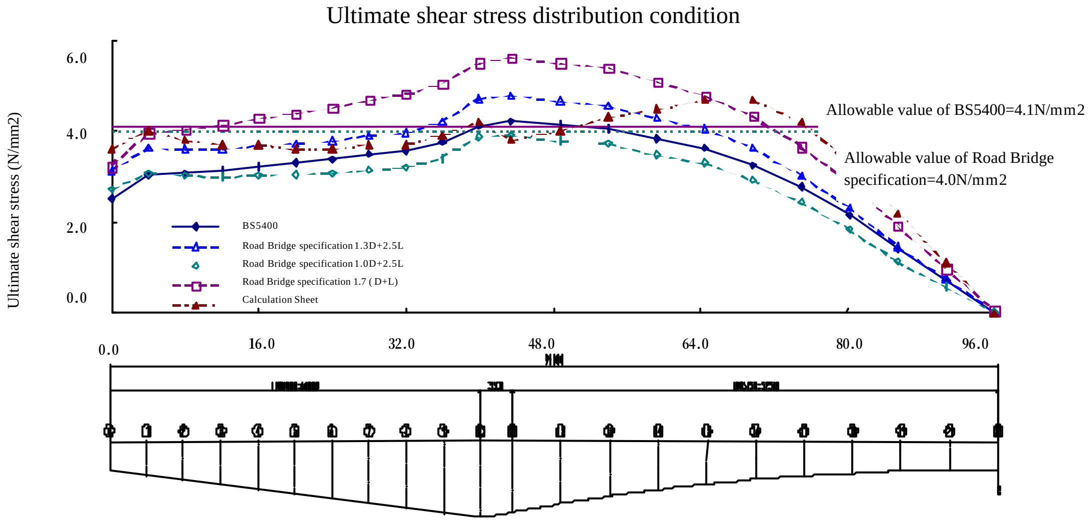
a) Design standard and generated stress

Initially, the New Nyali Bridge was planned and constructed under loan assistance. Then, the New Mtwapa Bridge was hurriedly added. Accordingly, there are slight differences in concepts used for concrete strength and live load between these bridges. The original design was implemented during a period from May 1976 to March 1977.

At the time, the design method according to the British Standard (hereinafter called "BS") was not required to specify allowance for shear stress. Therefore, shear stress occurring specifically in the New Mtwapa Bridge, which is substantially higher than currently specified allowable values, is the cause of many cracks of this bridge. On the other hand, bending and diagonal tensile stresses meet allowable values, so that there was no problem in the design in those days (Fig. 2-3-7). When compared with the Japanese standard, the bridge according to the existing BS standard (BS5400:1978) has shear, bending, and diagonal tensile stresses exceeding allowable values in several points. These points will require reinforcement in the future.

Aggregate of bending stresses – At design load





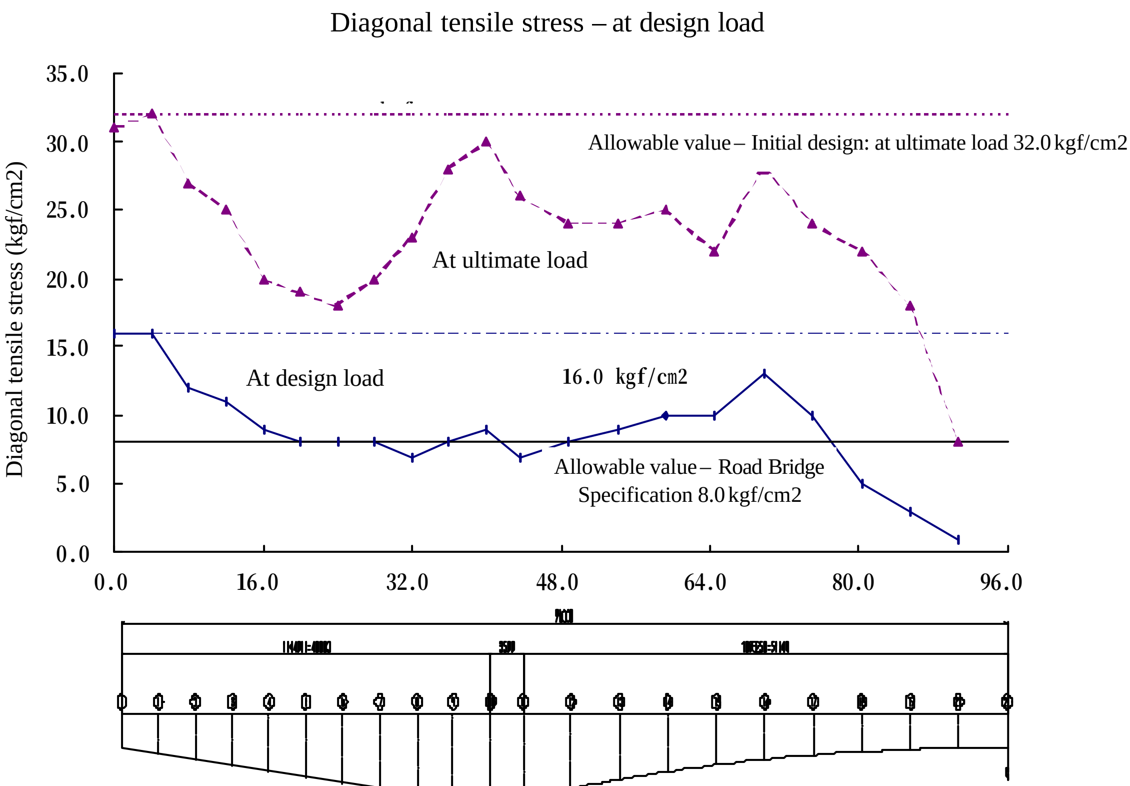


Fig. 2.3.7 The Stress and Allowable Value

b) Deflection of main girder

The longitudinal alignment of the road has a lower section in the middle of the span. Under the effect of the creep characteristic of concrete, the main girder’s longitudinal alignment tends to sink further. In spite of the progress of creep for more than 20 years, the survey this time indicated deflection that is almost the same as that of two years ago, as shown in Fig. 2-3-3. Deflection is therefore converging.

Judging from the result, deflection of the main girder does not present any problem for design and construction.

c) Live load

The statutory live load that Kenya is applying to existing vehicles utilizing the national highway is 54 tons. The maximum axle load is 8 tons and 10 tons. These live loads do not present any design problem because they do not exceed the original design load.

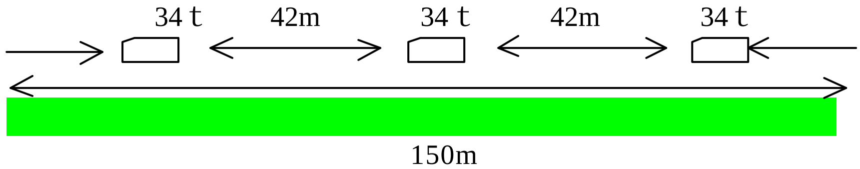
We obtained various kinds of pre-existing information on the concepts for the existing live load utilizing these bridges. Namely, it was said that the live load of the original design was TL-20 of the Japanese standard. In any case, the design live load could be confirmed using data available in the field. Comparison of live loads was made because of concern that vehicle overloading might adversely affect the bridge body.

The live load application method used for the New Nyali Bridge applied three 34-ton vehicles (maximum statutory live load – 54 t) and a 16-ton load was applied in the lane direction as shown in Table 2.3.10. This produced a load approximately equivalent to the original design load.

Table 2.3.10 Comparison of live loads on New Nyali Bridge

Live load	Employing organization	Max. axle load(ton)	Max. load on 150 m main span（ton)	Ratio to design load
①BS(25)	Load adopted at design of New Nyali	6.25	346	1.00
②TL-20	Load used in OECF（present JBIC）	8	588	1.70
③34ton	MORPW statutory load	5	342	0.99
③A live load	Existing Japanese load	8	528	1.53

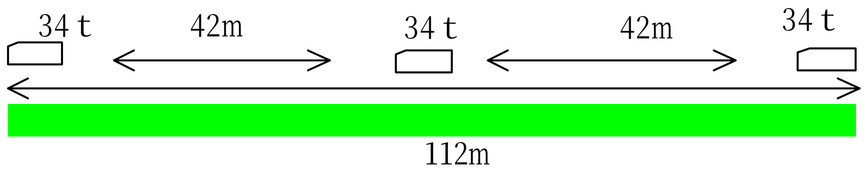
Bridge axial direction



For the New Mtwapa Bridge, the live load becomes higher when 34-ton and 16-ton loads are applied to two lanes than when the 54-ton load is applied as shown in Table 2.3.11. The former value was therefore used for comparison.

Table 2.3.11 Comparison of live load on New Mtwapa Bridge

Live load	Employing organization	Max. axle load (ton)	Max. load on 112 m main span (ton)	Raio to design load
①BS(45)	Load adopted at design of New Mtwapa Bridge	11.25	299	1.00
②TL-20	Load used in the OECF (present JBIC) survey report	8	264	0.88
③34ton	MORPW statutory load	5	150	0.50
④A live load	Existing Japanese load	8	278	0.93



As is known from the above, the vehicle load applied in Kenya is about 50% in load ratio to the original design for the New Mtwapa Bridge. The TL-20 load considered in the OECF (present JBIC) survey is about 90% of the load ratio in the original design. Therefore, it may be surmised that the statutory load currently applied in Kenya for the original design load does not cause any abnormal load.

2-3-2 Design Concept

1) Basic design considerations

This basic design was implemented to offer the most appropriate content, with due consideration of the social, economic, natural environment, legal, and other construction conditions of Kenya, in accordance with the rehabilitation grant-aid scheme. Fundamental considerations are described below:

① **Identification of causes of damage and proposals for their removal**

This survey result has made it clear that Kenya's failure to perform required maintenance is mostly responsible for damage to various locations of New Nyali and New Mtwapa Bridges. This plan is intended mainly to rehabilitate existing bridges while identifying the causes of damage in more detail and proposing eliminating causes, etc. because similar damage may occur in the future in other bridges in Kenya.

② **Clarification of cost effectiveness**

Clarification is made whether or not rehabilitation is cost effective. Namely, the basic design is made while clarifying cases, including, for example, a case when rehabilitation is less cost effective because the service life is short or when this rehabilitation is considered less practical as large-scale rehabilitations will have to be made periodically after this.

③ **Strengthened limitation of live load**

For the live load of original design of these bridges, HA full load and HB load Unit 25 (New Nyali Bridge) and Unit 45 (New Mtwapa Bridge) according to BS153 were used. Accordingly, the statutory load (full load: maximum 54 ton, axle load: maximum 8 and 10 tons) of Kenya must be limited so that it does not exceed the live load of original design. Though four fixed type axle load meters and 13 movable axle load meters were introduced in Kenya, all of fixed type meters are provided to the A class road. For National Highway B8 (B class road) on which these bridges are located, only one movable axle load meter covers the wide area including the Coast Area and the Mombasa – Nairobi road. It is proposed that Kenya acknowledge thoroughly the importance and necessity of regulation of overload vehicles and assigns one movable axle load meter for dedicated use on the Mombasa - Malindi section as soon as possible.

④ **Consideration of the rainy season and tide level**

The construction process will be planned with due consideration of the effects of the rainy season. The area where these bridges are located is at a low altitude and coastal, so the weather is different from that of inland areas. The weather is characterized by high temperatures and high humidity, with the annual highest average temperature being 30.2℃ and the lowest average temperature being 20.7℃. The relative humidity average is 82% at 9:00 in the morning and 65% at noon.

The rainy season indicates evident seasonal characteristics, with the annual average precipitation as high as 1,100 mm. The major rainy season occurs from March to July with an accuracy of 60%. Maximum precipitation is recorded from May to June. In May, monthly average precipitation of 375 mm is recorded, decreasing gradually in June. The minor rainy season begins at the end of October and ends in December to January, the following year.

Since adhesives used for repair are sensitive to humidity, due consideration must be given to humidity control during repair.

The tide level must be taken into account during repair of piers (New Nyali Bridge) in creeks because the tidal range is about 3.9 m. In particular, it is essential to grasp the time of occurrence of tidal range because the tide level affects carry-in of repair materials, and carrying in and out of temporary scaffolding, etc.

⑤ **Establishing standards that take present and future road utilization conditions into account**

This road is a trunk road with larger traffic volume than other national highways. This survey showed traffic volume exceeding about 38 thousand vehicles/day for the New Nyali Bridge. The traffic volume for the New Mtwapa Bridge exceeds about eight thousand vehicles/day, and it is increasing every year. In particular, many overload vehicles pass there, which is considered to have caused many cracks in concrete main girders of the two bridges constructed in 1981. Maintenance by MORPW is far from satisfactory, and actual repair of damage is not made at present. Note that the design of the two bridges in the 1970s was based on the live load according to the British Standard (BS) and concrete stress verification according to the German Standard (DIN), so that the design was not standardized. Since the live load system and stress verification content of both countries are considerably similar to Japanese standards, it was decided to have the design conform to Japanese Standards.

⑥ **Effective utilization of local material/equipment**

Among construction materials and equipment of Kenya, concrete-related materials, such as cement, gravel, etc. can be produced in the country. In particular, there is a cement plant near the New Mtwapa Bridge, which ensures enough supply. Construction machinery includes that owned by the government and that by private enterprises. But the availability of heavy machinery including cranes is limited, and difficult to lease depending on congestion of projects. The study will aim for maximum utilization of machines and materials available locally.

⑦ **Considerations regarding technical level of local engineers**

Local engineers of the Bridge Division of RD, MORPW, are in charge of all bridges in the country. Moreover, PWO under control of RD directly conducts maintenance of all roads including machines. Many training and lecture courses are provided to engineers, and managers. And efforts were made to have leading engineers master technology in foreign countries when they were young. There are many participants in the counterpart training held in Japan, and the percentage of engineers who have mastered technology in foreign countries is high. It cannot be denied however that there remains a gap with advanced countries. For example, it is difficult to obtain appropriate materials in terms of maintenance, deficient technical resources over the whole of maintenance field, etc. Considering these facts, this project will be implemented with the intention of helping local engineers master transferred technologies related to rehabilitation or reinforcement of existing concrete bridge body and accessories. As there are many types of works, Japanese engineers of superior ability will be dispatched to Kenya.

⑧ **Employment of the easy-to-maintain rehabilitation method**

Special maintenance expenses have been included every year for four bridges on Route B8 constructed under Japanese aid. However, these expenses were not actually used in 1998/1999 on account of damage caused by El Nino, so that thorough maintenance was not been made. For the rehabilitation method, therefore, methods, structure, and materials that can reduce maintenance cost as much as possible will be reviewed. In this case, due consideration will be given to cost performance.

⑨ **Cost reduction and shortening of the work period**

To comply with the Japanese rehabilitation grant aid program, work content that can reduce costs and the work period as much as possible will be reviewed.

2) Design standard

a) Applicable standard

Concerning the bridge design standard and its application policy, Kenya has a standard, but applies BS directly in practical cases. As described above, in the case of the New Mtwapa and Nyali Bridges, the live load was applied according to BS during design. But the standard value used to verify concrete stress is not clear. The concrete stress and allowable set value in combination with the live load show a trend approximately similar to cases designed according to the Japanese standard.

- ”Prestressed Concrete Method Design Implementation Guideline,” March, 1991, Japan Society of Civil Engineers
- ”Concrete Standard Specification,” March, 1996, Japan Society of Civil Engineers
- ”Road Bridge Specification & Explanation,” December, 1996, Japan Road Association
- ”Guidebook for Concrete Road Bridge Design,” February, 1994 Japan Road Association
- ”Guidebook for Construction of Concrete Road Bridge,” February, 1994, Japan Road Association

b) Load conditions

Loads used in bridge rehabilitation design are classified into primary load, secondary load, and particular load according to the loading method, loading frequency, and effects on the bridge. Features of each load are described below.

i) Primary load

①Dead load

The dead load is the sum of the bridge’s own dead weight and dead weight of attachments and calculated on the basis of unit weight as shown in Table 2.3.12.

Table 2.3.12 Unit weight of material

Material	Unit weight (kgf/m3)	Material	Unit weight (kgf/m3)
Iron, cast steel	7,850	Plain concrete	2,350
Cast iron	7,250	Cement mortar	2,150
Aluminum	2,800	Asphalt concrete	2,300
Reinforced concrete	2,500	Wood	800
Prestressed concrete	2,500		

②Live load

The original design used HA and HB loads (Unit 25 for the New Nyali Bridge and Unit 45 for the New Mtwapa Bridge) on the basis of BS standard. A comparison of live loads showed that calculation is possible with the load ratio for HA and HB, and B-load is used as a basic load for the design.

③Impact

The impact coefficient i for a prestressed concrete bridge is calculated as follows:

For slab design： $i = 20 / (50 + \text{span length})$

For main girder design： $i = 10 / (25 + \text{span length})$

④Prestressing force

⑤Effects of concrete creep

⑥Drying shrinkage of concrete

ii) Secondary load

This load must always be taken into account for combination of loads.

①Wind load

Not considered in this design because the effect is small.

②Effect of temperature changes（According to the temperature fluctuation of Kenya）

Concrete：15℃～40℃

③Effects of earthquake

Not considered in this design

iii) Particular loads

These loads must be specifically taken into account depending on the bridge type, construction type, and bridge point condition.

①Loads during construction

②Effects of shifting fulcrum

c) Materials used

①Concrete

Table 2.3.13 Concrete strength

Specified concrete strength	Stress σ	Application
Grade 45	45 N/mm^2	Superstructure of New Nyali Bridge
Grade 30	30 N/mm^2	Substructure of New Nyali Bridge, Superstructure of New Mtwapa Bridge
Grade 25	25 N/mm^2	Abutment of New Mtwapa Bridge

Table 2.3.14 Steel strength

Material	Design standard strength	Stress at yield point σ	Application
②Steel bars	SD295 SD345	295 N/mm^2 345 N/mm^2	Superstructure and substructure of New Nyali Bridge, superstructure and substructure of New Mtwapa Bridge
③Prestressing steel	SBPC 85/105 19S15.2	85 N/mm^2 160 N/ mm^2	New Nyali and New Mtwapa Bridges New Mtwapa Bridge
④Steel plate	SM490Y	315 N/mm^2	New Mtwapa Bridge
⑤Anchor bolt	SM490Y	315 N/mm^2	New Mtwapa Bridge

d) Superstructure design conditions

Table 2.3.15 (1) Superstructure design conditions

	New Nyali Bridge	New Mtwapa Bridge
Bridge type	Three-span continuous prestressed concrete box-girder bridge Three-span continuous reinforced concrete girder bridge	Three-span continuous prestressed concrete box-girder bridge
Width	Driveway width 9.5m×2（3 lanes ×2） Sidewalk, curb 2.695m×2 Median 1.5m	Driveway width 7.5m（2 lanes） Sidewalk, curb 2.25m
Live load	B live load（HA,HB converted）	B live load（HA,HB converted）
Horizontal alignment	∞	∞
Cross grade	←2.5%→	2.5%→
Bridge surface pavement	Asphalt pavement 55mm	Asphalt pavement 55mm
Attachments	Water pipe φ1000mm	Water pipe φ200mm
Temporary construction method	Main bridge：Cantilever Approach bridge：Erection with staging	Center span：Cantilever Side span：Erection with staging

e) Substructure design conditions

Table 2.3.15 (2) Substructure design conditions

	New Nyali Bridge	New Mtwapa Bridge
Abutment type	Inverted T type abutment x 2	Inverted T type abutment x 2
Pier type	Wall-type pier x 5	Wall-type pier x 2
Foundation type	Reinforced concrete piles (φ2000,φ1200)	Reinforced concrete piles (φ2000)

2-3-3 Basic Design

1) Concrete structure rehabilitation plan

a) Rehabilitation methods

The survey of the damage to concrete members of the New Nyali and New Mtwapa Bridges produced the following results. On the basis of the damage condition, this section selects rehabilitation methods to maintain the durability of the bridges.

For selection of the rehabilitation method, Chapter 5, Rehabilitation Method, the Concrete Crack Survey, Repair, and Reinforcement Guideline (Japan Concrete Institute) is referred to and current advanced repair technology is incorporated.

Tables 2.3.16 and 2.3.17 show the repair methods for each type of damage.

Table 2.3.16 Damage repair method

Damage	Outline of the repair method
Cracks	• The following repair methods are selected according to the crack width: Crack width 0.2mm or less — Corrosion-resistant coat 0.2mm or more — Grouting Filling
Flaking	• Repair the section after removal of loose concrete
Exposed bars	• Remove concrete from around corroded bar, provide anti-corrosion treatment to bar, and repair the section.
Water leakage	• Cut-off with grouting after confirmation of water leaking point
Deterioration (discoloration)	• No particular repair necessary, but it is desirable to provide surface painting in view of aesthetic viewpoint.
Cavity	• Confirm the size and depth of cavity, and repair through section repair, grouting, etc.
Free lime	• Confirm moisture supply portions and clean the surface after cut-off.
Corner chip	• Repair the section when corner chip is large

Table 2.3.17 Classification of repair methods for cracks

Objective of repair	Symptom and cause of crack		Crack width * (mm)	Repair method **					
				Corrosion-resistant coat	Grouting	Filling	Other methods: Application of permeable water-proofing agent	Other methods: Other	Applicable crack location
Water proof	Steel bar not corroded	Crack width fluctuation, small	0.2 or less	○	△		○		
			0.2 ～ 1.0	△	○	○			
		Crack width fluctuation, large	0.2 or less	△	△		○		
			0.2 ～ 1.0	△	○	○			
Durability	Steel bar not corroded	Crack width fluctuation, small	0.2 or less	○	△	△			Crack in main girder (0.2mm or less)
			0.2 ～ 1.0	△	○	○			Crack in main girder (0.2mm or less) Cold joint, construction joint
			more than 1.0		△	○			
		Crack width fluctuation, large	0.2 or less	△	△	△			
			0.2 ～ 1.0	△	○	○			
			more than 1.0	△	○	○			
	Bar corrosion		—			○			Rusting, loose, honeycomb
	Salt damage		—					●	
	Reactive aggregate		—					●	

Excerpt from the Concrete Crack Survey: Repair and Reinforcement Guideline (Japan Concrete Institute), P90

* Cracks 3.0 mm or more wide are often associated with structural defects.
Normally, reinforcement of structural durability is made in addition to listed repairs.

** ○: Appropriate method △: Appropriate depending on conditions
●: Method under development

b) Repair method

The following methods will be applied to these bridges according to crack width and condition.

①Corrosion-resistant coat

Grouting is difficult for fine cracks (width 0.2 mm or less). Film is applied over the cracked surface to improve water proofing property and durability.

②**Grouting**

Resin or cement material is grouted when crack width is 0.2 mm or more, improving water proofing properties and durability. Generally, resin material superior for grouting into cracks and for adhesivity is selected. In particular, epoxy resin that has often been applied with success and superior in required performance is selected. Table 2.3.18 shows the list of resins and performances.

Table 2.3.18 List of resins and performances

	Epoxy	Polyester	Polyurethane	Rubber, asphalt
Adhesivity	◎	○	○	△
Flexibility	△	△	◎	○
Durability	◎	○	○	×
Workability	○	○	○	◎
Waterproof	◎	○	○	△
Alkaline resistance	◎	×	○	△
Shrinkage	Nothing	Large	Small	Large
Economy	△	○	○	◎

Note: ◎: Superior ○: Better △: Good X: Not good

Excerpt from the Concrete Crack Survey: Repair and Reinforcement Guideline (Japan Concrete Institute), P97

③**Filling**

This method is suitable for repair of relatively wide (0.5 mm or more) cracks. Concrete is cut along crack and a repair material is filled. This method differs for two cases: one without corrosion of bar and the other with corroded bar.

＜Steel bar not corroded＞

Concrete is cut in a U or V shape to the width of about 10 mm along the crack as shown in Fig. 2.3.8. Sealing material, flexible epoxy resin, or cement material is filled in the cut portion to repair the crack. In this project, this method is employed to treat relatively large cracks.

For cracks in the inside, filling with a V-cut is made to prevent material leakage from the outside even when the crack width is not large.

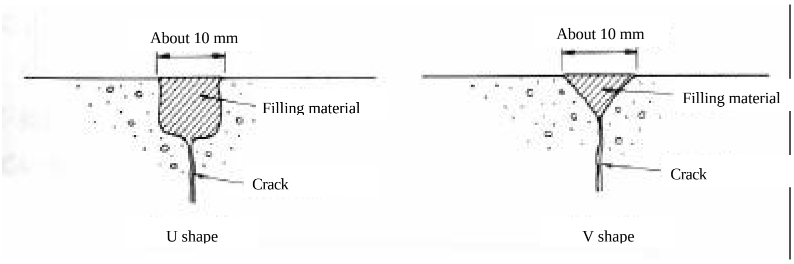


Fig. 2.3.8 Filling when bar is not corroded

<Bar corroded>

As shown in Fig. 2.3.9, the concrete is chipped sufficiently for handling of corroded bar portion. Bar rust is removed, followed by anticorrosion treatment of bars and primer application to concrete. Then, polymer cement mortar or epoxy resin is filled. According to the standard method, anticorrosion treatment is made only for the corroded portion as shown in Fig. 2.3.10. In the case of this bridge, the concrete is chipped for 100 mm or more including the backside of the corroded portion to improve the corrosion preventive effect of bars. Fiber reinforced mortar is also used for filling to ensure higher repair performance.

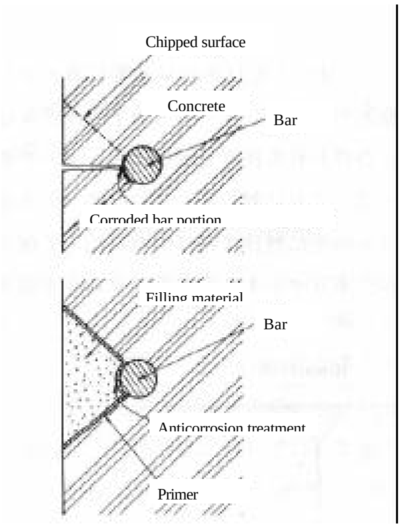


Fig. 2.3.9 Filling when bar is corroded

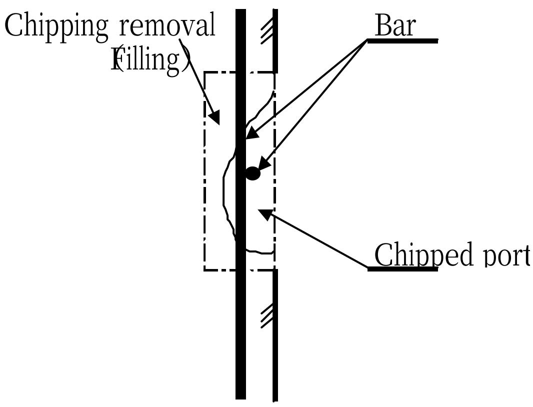


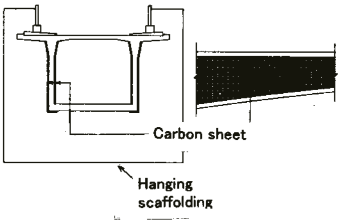
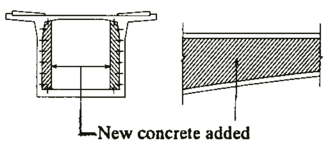
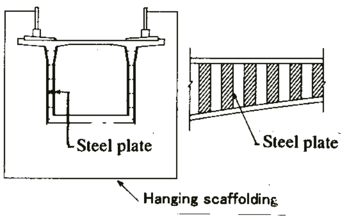
Fig. 2.3.10 Filling method for this bridge (New Nyali Bridge pier)

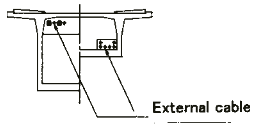
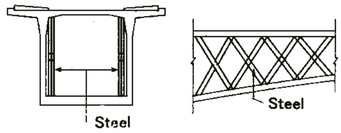
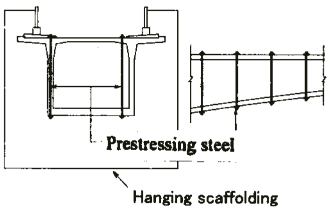
2) Reinforcement Method

a) Selection of the reinforcement method

Cracks in the main girders of the New Mtwapa Bridge occurred due to deficient shear strength. If this is left without reinforcement, the girders may become brittle and suddenly collapse. Cracks cannot be dealt with sufficiently solely through resin filling or injection, which means that other reinforcement methods will be applied at the same time to increase shear strength to prevent expansion of cracks. Table 2.3.19 shows the shear reinforcement method for the New Mtwapa Bridge.

Table 2.3.19 Shear reinforcement method

	Overall view	Outline of the method	Features	Applicability
Reinforcement with carbon fiber		Carbon fiber sheet is adhered to existing concrete while being immersed in epoxy adhesive. Through integration with existing concrete, shear strength is increased.	・No weight increase ・Surface protection necessary ・High unit price ・Periodical control necessary.	○ (To be studied)
Reinforcement through addition of new control		New concrete added to the inside of existing girder. Through integration with existing concrete, shear strength is increased.	・Weight increased ・Increase in girder deflection ・Traffic regulation necessary during work	× (Not appli-cable)
Reinforcement through adhesion of steel plate		Steel plate fixed to existing girder with anchor bolts and gap in concrete section filled with epoxy adhesive. Through integration with existing concrete, shear strength is increased.	・Slight increase in weight ・Rust-preventive treatment of steels necessary ・Periodical control necessary	○ (To be studied)

Reinforcement with external cable		Prestressing steel is arranged to existing girder in the bridge axial direction. Prestressing will enhance shear strength. However, the section is deficient if reinforcement is to be made entirely by means of cable.	･Small increase in weight ･Prevention of expansion of crack	○ (To be studied)
Reinforcement with steel framed truss		Steel-framed truss is arranged in existing main girder. Through integration with existing concrete, shear strength is increased.	･Slight increase in weight ･Stress transmission mechanism is not known. ･Rust-preventive treatment of steel necessary	× (Not applicable)
Reinforcement with shear steel		Prestressing steel is arranged to existing main girder in the vertical direction. Introduction of prestressing enables increase in shear strength.	･Small increase in weight ･Difficult to consider the existing prestressing cable.	× (Not applicable)

b) Study of reinforcement to prevent cracks

Reinforcement quantity for each reinforcement method selected in Table 2.3.19 is calculated according to the procedure described in Fig.2.3.11.

Verification of design calculation

Concrete strength : $\sigma ck = 300\ kgf/cm^2$

Verification of bending stress (BS)

- $-36\ kgf/cm^2 < \sigma ca < 0.4fcu = 120\ kgf/cm^2$

Bending stress OK

Japan: Specifications for Highway Bridges III

- Bending stress allowable value

$-12\ kgf/cm^2 < \sigma ca < 110\ kgf/cm^2$

In the case of Specifications for Highway Bridges III, the compressive stress exceeds the allowable value near the fulcrum

Verification of shear stress (DIN4227)

- Diagonal tensile stress: $\sigma Ia < -16\ kgf/cm^2$

(as described in calculation sheet)

- Ultimate shear stress: $\tau a < 62.5\ kgf/cm^2$

OK for the DIN standard

Japan: Specifications for Highway Bridges III

- Diagonal tensile stress $\sigma Ia < -8\ kgf/cm^2$

Exceeds the allowable value within the crack occurring range

- Shear stress $\tau a < 4.5\ kgf/cm^2$

Exceeds the allowable value over the entire bridge

- Ultimate shear stress $\tau m < 40\ kgf/cm^2$

Exceeds the allowable value near the fulcrum

Study of design criteria for reinforcement design

Determination is made to use which of BS, DIN, or Specifications for Highway Bridges III for the allowable value that is a basis of reinforcement design.

Comparison of allowable values shows that BS and DIN do not set the diagonal tensile stress allowable value needed for review of the shear force. Specifications for Highway Bridges III specify all of allowable values for the diagonal tensile stress, shear stress, etc.. Accordingly, Specifications for Highway Bridges III are used as a standard for reinforcement design.

Refer to the allowable value comparison table

Reinforcement policy

Bending stress

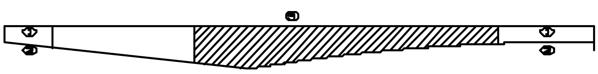
When Specifications for Highway Bridges III are used, the compressive stress exceeds the allowable value. Since concrete is highly resistant against compression, it is not necessary to improve this stress.

END

Shear stress

In certain locations, cracks had occurred. Result also shows that shear cracks are in progress.

It is therefore necessary to carry out shear reinforcement.



Rehabilitation/reinforcement locations

① Locations with shear crack
② Locations where the diagonal tensile stress allowable value is exceeded
③ Location where the ultimate shear stress allowable value

Rehabilitation/reinforcement method

① **Locations with shear crack**

Location with crack is integrated with the bridge body through epoxy resin injection, preventing decrease in strength.

② **Locations where the digonal tensile stress allowable value**

For locations where diagonal tensile stress exceeded the allowable value, introduction of external force is necessary. The following methods are considered:

a) Introduction of axial compressive force by means of external cable

b) Installation of shear steel rods

Installation of shear steel rods is physically difficult, and axial compressive force is introduced by means of external cable.

As the compressive stress of main girder concrete increases, four or more external cables cannot be installed. It is therefore impossible to eliminate all of excess diagonal tensile stress completely. In any case, compressive force can be introduced into the crack repair location, which is effective for improvement of the stress.

③ **Locations where the ultimate shear stress allowable value**

・Determination of shear compensation amount

The load combination for ultimate load to be used is the one specified in the Specifications for Highway Bridges III. The combination coefficient is 1.1 x dead load + 2.5 x live load, which is the same as for BS5400.

External cable used for improvement of stress in (1) and (2) causes the vertical component to act on (3), effectively reducing the shear force. But this component is limited and is not enough to assure shear strength against shear force under ultimate load.

To cope with deficient shear strength, either the carbon fiber or steel plate adhesion method is used for reinforcement to enhance the shear strength.

Deficient shear strength ＝ Su - Sc - Ss - Sp

Su： Shear force under ultimate load(1.1D+2.5L)

Sc： Shear force carried by concrete

Ss： Shear force carried by rebar

Sp： Shear force carried by external cable

Fig. 2.3.11 Study flow chart on reinforcement against crack

Since the main girder concrete cannot be considered as a resistance section when shear crack occurs, this is ignored during calculation of reinforcement quantity.

i) Calculation of deficient shear force

The section ⑪ is considered. The study case is for 1.1D + 2.5L at ultimate load according to Specifications for Highway Bridges III. Allowable values to be used in calculation is the one specified in Road Specification.

Section concerned	Girder height (m)	Web thickness (m)	Girder height×Web thickness	Acting shear force
Section ⑪	6.092	0.4	4.874	2199.6

○ Shear force carried by concrete

Equation： $Sc = k \cdot \tau c \cdot bw \cdot d$

$\tau c = 4.5$ kgf/cm^2

○ Shear force carried by stirrup

Equation：$Sh' = \{Aw \cdot \sigma s \cdot d \cdot (\sin\theta + \cos\theta)\} / (1.15 \cdot a)$

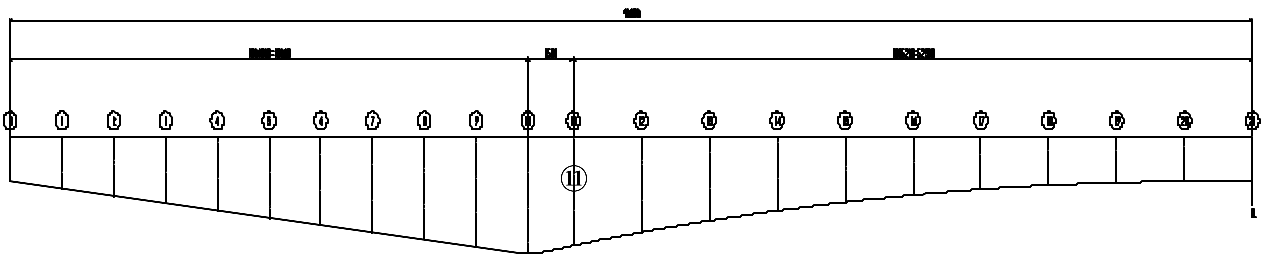
Aw：Sectional area of diagonal tensile rebar (cm^2)

σs：Yield point of diagonal tensile rebar for ultimate case

a：Diagonal tensile rebar arrangement interval (cm)

Section concerned	Girder height (m)	Web thickness (m)	Girder height× web thickness	Acting shear force (kgf)	Shear force carried by concrete (kgf)	Shear force carried by stirrup (kgf)	Deficient shear force (kgf)
Section ⑪	6.092	0.4	4.874	2199600	0	315619	1883981

※ The reinforcement quantity is calculated for the deficient shear force.



ii) Reinforcement method

The reason that the reinforcement with the method of external cable + steel plate adhesion adopted is shown in Table 2.3.20.

Table 2.3.20(a) Comparison of shearing reinforcement methods

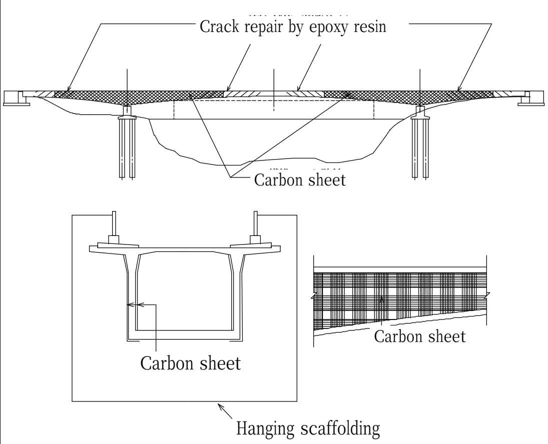
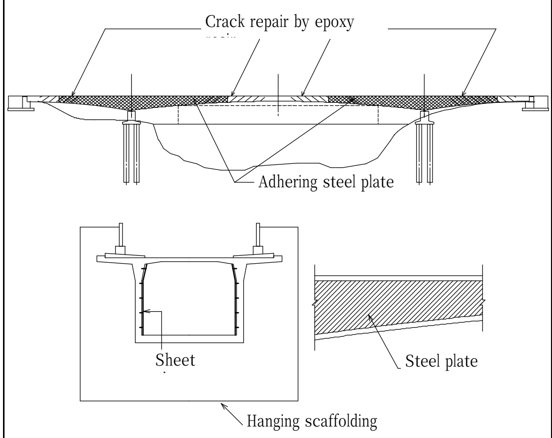
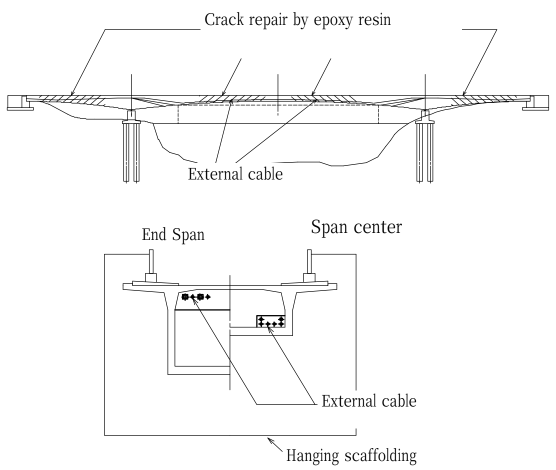
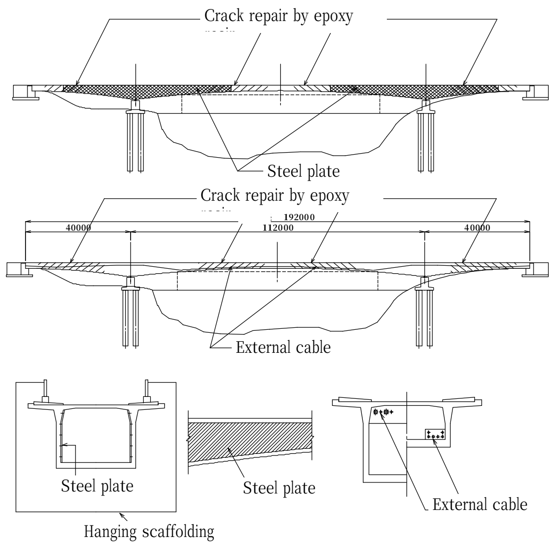
		Carbon fiber reinforcement method	Reinforcement by adhering steel plate
Conceptual view of reinforcement			
Outline		This method is to improve the shearing resistance mainly by laminating and adhering fiber reinforcing material impregnated with epoxy resin over the concrete member surface where tensile or diagonal tensile stress acts, so that the fiber is integrated with existing member.	This method is to improve the shearing resistance mainly by adhering with epoxy resin the steel plate over the concrete member surface where tensile or diagonal tensile stress acts, so that the plate is integrated with existing member.
Features	Constructive property	• Almost no increase in member dimensions and weight after reinforcement and no burden on the foundation • The anchoring location requires reinforcement with anchor and steel plate. • Reinforcement efficiency has been proved by experiment, etc. • The reinforcement amount is 2.5 layers per web on one side. • Effective only for a stress in one direction	• Anchor drilling is necessary in the web to install steel plate. (21 anchor D22 drills per 1 m width) • Increase in reinforcement results in increase in thickness of steel plate. This in turn requires division of steel plate in small pieces, which results in increase in the welding operation inside the girder. • Effective for stress in multiple directions • Abundant implementation records
	Approximate quantity	Carbon fiber sheet (weight 300g/cm^2) 239.2 m^2/layer Per point Web outside 3 layers Web inside 2 layers No. of points 4 points Total 4784 m^2	Steel plate adhesion area 1,100 m2 Steel plate weight 52 ton ＊ Use 6.0 mm thick steel plates in strip forms ＊Steel plate may be installed either to the inside or outside of girder.
	Workability	• Light weight and satisfactory workability. Favorable in places where the work space is limited • Heavy machinery or large-scale temporary work not necessary • Work impossible when the rain is falling	• Less satisfactory workability because of weight • Work possible even during rain
	Traffic regulation	• Traffic regulation not necessary for the work	• Traffic regulation not necessary for the work
	Economy	1.00	1.15
	Maintenance	• Control for surface protection necessary	• Steel plate anticorrosion treatment and periodical maintenance necessary
Example		• Tomei Expressway, Tokyo viaduct, and others	• Hanshin Expressway, No.14 Matsubara Line, Kireuriwari Bridge, and others
Overall evaluation		Though the weight does not increase much, the skilled worker is essential and the work during rain is impossible. Large-scale traffic regulation is not necessary, which is favorable, but actual application requires caution because this method provides resistance against stress in one direction only. ○:Better method	Effective reinforcement method, which has been applied in many cases and whose application is not limited to the stress direction. ○:Better Method

Table 2.3.20(b) Comparison of shearing reinforcement methods

Reinforcement with external cable	Reinforcement with external cable + steel plate adhesion
	
This method is to improve the shearing resistance through improvement of the stress condition of concrete member by introducing prestress into the member by means of the prestressing steel.	This method is to improve the stress condition of existing structures by introducing prestress by means of prestressing steel and further to improve the shearing resistance by integrating the steel plate into existing members through adhesion using epoxy resin.
･The external cable increases the axial compressive force while reducing the active shearing stress through suspension effect, thereby reducing the diagonal tensile stress in the existing concrete. A considerable number of external cables must be arranged to improve the shearing resistance, which is actually difficult to make. ･Drilling must be made in many points because a large number of φ32 prestressing steel bars must be used to install anchorage, which results in damage to existing structures. ･The use of anticorrosive external cable will ensure higher durability. ･Since the external cable causes an increase in the compressive stress, the number of cables to be installed is limited when the concrete is already subjected to severe compressive stress.	･The external cable increases the axial compressive stress and reduces the active shearing stress through suspension effect, thereby reducing the diagonal tensile stress in existing concrete. ･Anchor drilling is necessary in web to install steel plates. (21 anchor D22 drills per 1 m width on one side) ･Increase in reinforcement results in increase in thickness of steel plate. This in turn requires division of steel plate in small pieces, which results in increase in welding operation inside the girder.
External cable prestressing steel (19S15.2) 760m ←Arranged over the entire length 280m ←80 pieces for sharing reinforcement Prestressing steel bar for anchorage φ32 1600 pieces 20 - φ32 steel bars necessary per external cable	Steel adhesion area 1,100 m2 Steel plate weight 52 ton ＊Use 6.0 mm thick steel plates in strip forms. ＊Steel plate may be installed either to the inside or outside of girder. External cable steel (19S15.2) 760m ←Arranged over the entire length
• No large equipment necessary, though the work may be made within girder	• Heavy and inferior workability • Work possible even during rain
• Traffic regulation not necessary for the work	･Traffic regulation not necessary for the work
1.40	1.30
• Condition after crack repair can be confirmed.	• Steel plate anticorrosion treatment and periodic maintenance necessary
• Yashio Koshigaya Line, overpass of Aioi viaduct	• Yashio Koshigaya Line, Overpass of Aioi viaduct, etc.
The vertical component of external cable can reduce the shearing stress, which is effective as reinforcement. But 80 pieces of 19S15.2 or equivalent prestressing steel bars are necessary, which can not actually be arranged in the girder. Besides, considerable number of prestressing steel bars are necessary for anchorage. Accordingly, this method is not suitable for reinforcement with external cable only. △:Unsuitable Method	The external cable functions to suppress crack that has occurred while the steel plate is effective in resistance against new live load. By fully utilizing characteristics of the material used, this method proves most effective. ◎:Best Method

3) Content of basic design

a) Content of repair

i) New Nyali Bridge

The New Nyali Bridge rehabilitation method is shown in Table 2.3.21.

Table 2.3.21 New Nyali Bridge rehabilitation method

Location	Method
Superstructure main girder	Resin grouting, Corrosion-resistant coat
Pier	Resin grouting, Corrosion-resistant coat, Mortar grouting
Abutment	Resin grouting, Corrosion-resistant coat
Expansion joint	Replacement
Shoes	Rust remove, Corrosion-resistant coat
Drainage system	Installation of drain pipe
Railings, guardrail	Replacement of damaged portion
Pavement	Replacement

ii) New Mtwapa Bridge

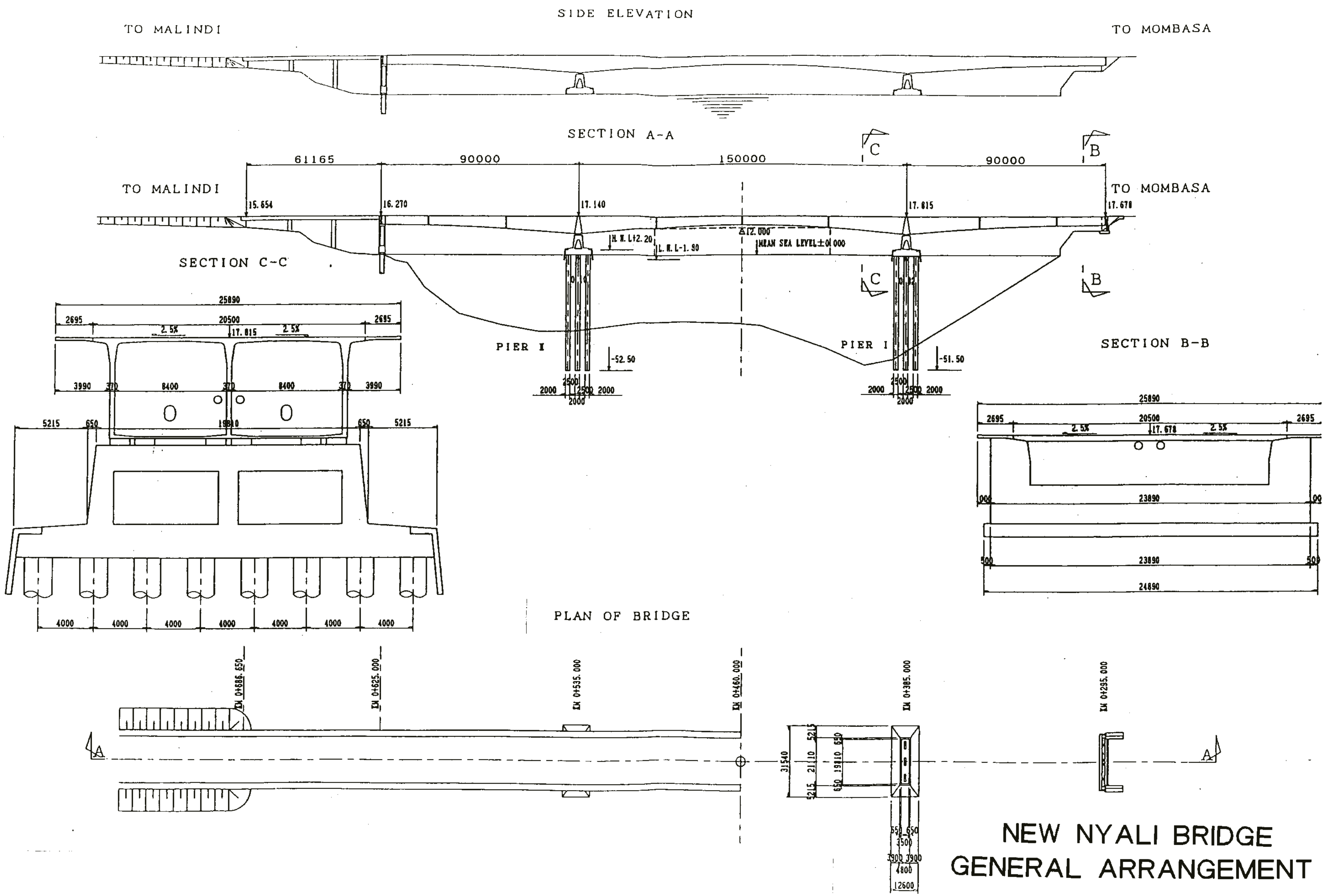
The New Mtwapa Bridge rehabilitation method is shown in Table 2.3.22.

Table 2.3.22 New Mtwapa Bridge rehabilitation method

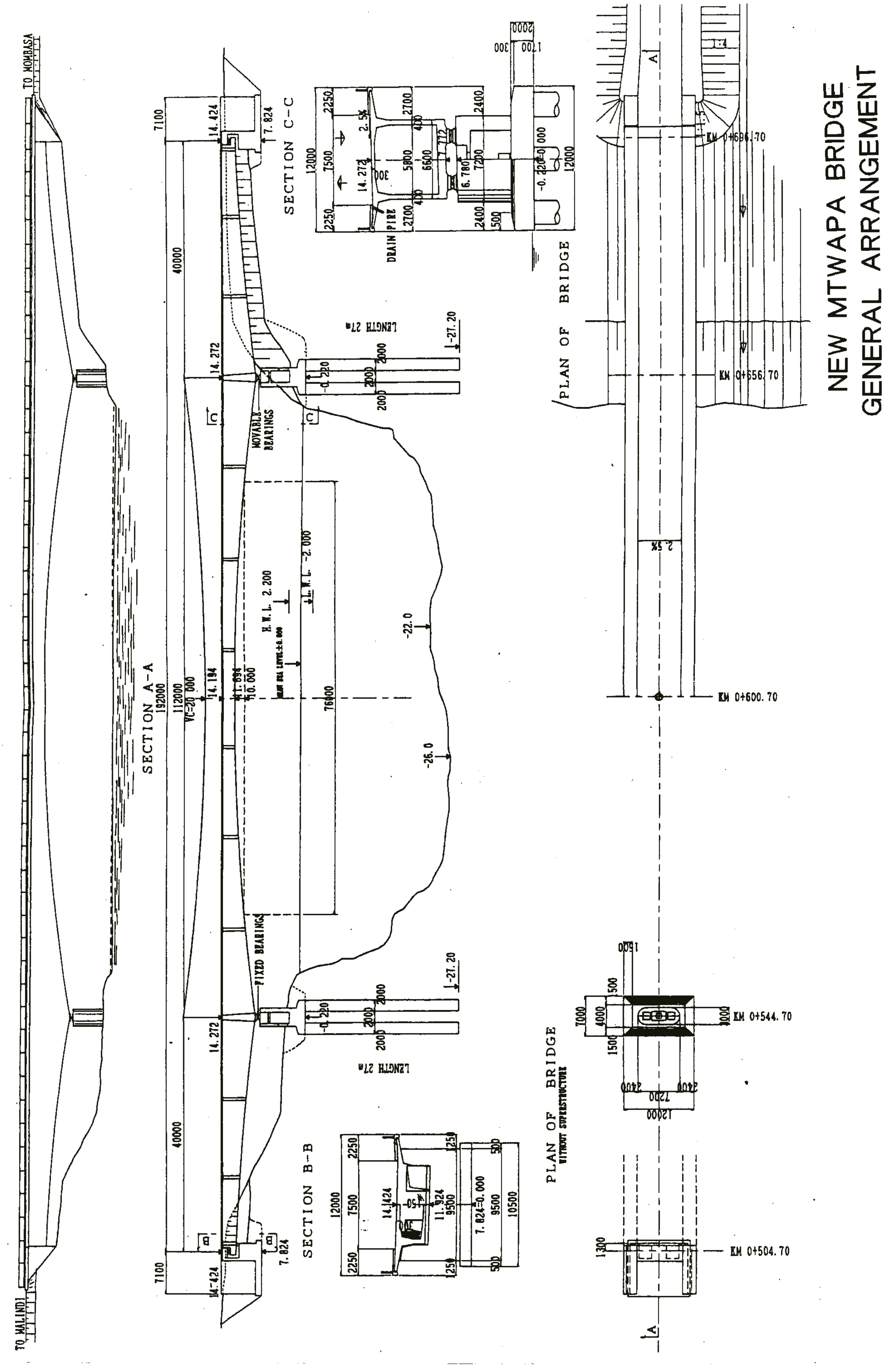
Location	Method
Superstructure main girder	Resin grouting, Corrosion-resistant coat, Reinforcement with external cable and steel plate adhesion
Pier	Resin grouting, Corrosion-resistant coat, Mortar grouting
Abutment	Resin grouting, Corrosion-resistant coat
Expansion joint	Replacement
Shoes	Rust remove, Corrosion-resistant coat
Drainage system	Installation of drain pipe
Pavement	Replacement

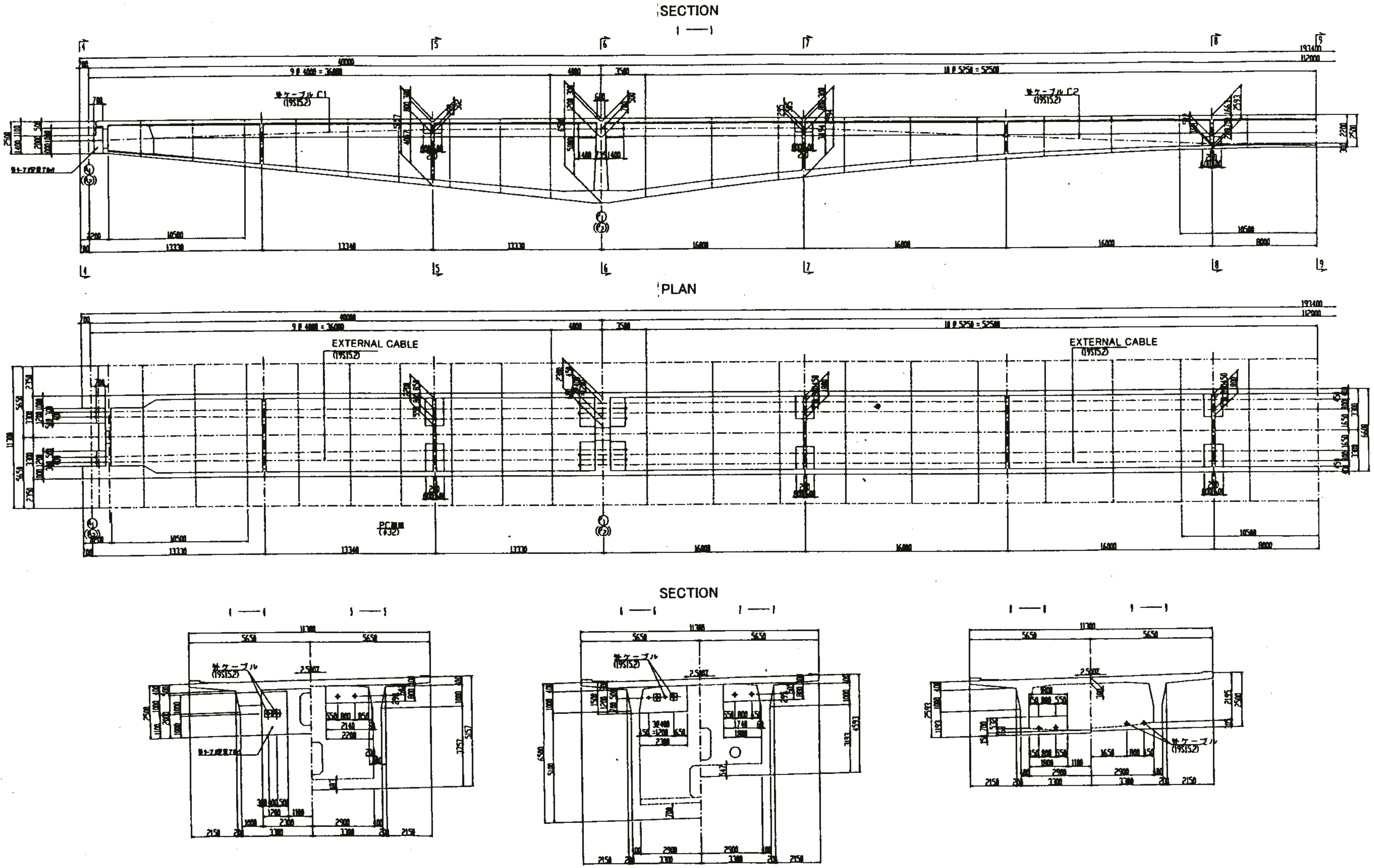
b) Basic design diagram

The general view of the New Nyali Bridge is shown. The New Mtwapa Bridge whose superstructure main girder requires reinforcement is also shown below.



NEW NYALI BRIDGE GENERAL ARRANGEMENT


NEW MTWAPA BRIDGE
GENERAL ARRANGEMENT
SIDE ELEVATION
TO MOMBASA
TO MALINDI
SECTION A-A
192000
112000
40000
40000
7100
MOVABLE BEARINGS
FIXED BEARINGS
LENGTH 27m
-27.20
H.W.L. 2.200
L.W.L. -2.000
MEAN SEA LEVEL ±0.000
-22.0
-26.0
SECTION B-B
SECTION C-C
DRAIN PIPE
PLAN OF BRIDGE
PLAN OF BRIDGE
WITHOUT SUPERSTRUCTURE
KM 0+696.70
KM 0+656.70
KM 0+600.70
KM 0+544.70
KM 0+504.70



NEW MTWAPA BRIDGE
EXTERNAL CABLE LAYOUT

c) Quantity of principal works

i) New Nyali Bridge

Structure	Location	Type	Content	Standard	unit	Quantity
Sub-structure	Pier	Resin grouting	Crack width 0．2mm>		m	92
		Corrosion-resistant coat	all paint		m2	630
		Concrete		Mix proportion	m3	3.6
Super-structure	Girder	Resin grouting inside	Crack width 0．2mm>		m	73.5
		Corrosion-resistant coat inside	Crack width 0．2mm<		m2	5.8
		Resin grouting outside	Crack width 0．2mm>		m	73.5
		Corrosion-resistant coat ouotside	Web		m2	3,156

ii) New Mtwapa Bridge

Structure	Location	Type	Content	Standard	unit	Quantity
Sub-structure	Pier, Abutment	Resin grouting	Crack width 0．2mm>		m	46
		Corrosion-resistant coat	Crack width 0．2mm<	Only cracks	m	14
Super-structure	Girder	Resin grouting inside	Crack width 0．2mm>		m	294
		Resin grouting outside	Crack width 0．2mm>		m	294
		Corrosion-resistant coat inside	Crack width 0．2mm<		m2	17
		Corrosion-resistant coat outside	All paint		m2	3,864
		External PC Cable	PC cable 19S15.2		ton	17
		Adhering steel plate			t	52

CHAPTER 3

Chapter 3 Implementation Plan

3-1 Implementation Plan

This plan will be implemented after conclusion of an exchange of notes (E/N) between governments when implementation has been approved by the Cabinet decision subsequent to a review by Japanese governmental agencies based on this report. For implementation, design and construction supervision will be conducted by the Japanese corporate consultant, and construction work will be executed by the Japanese corporate contractor, respectively upon conclusion of the contract with the Kenyan Government. These contracts will become effective upon approval by the Japanese Government.

The agency implementing this plan is the Road Bureau of the MORPW. Service coordination related to grant aid agreed upon between the two countries, preparation, and technical management, supervision, and maintenance will be under control of the Road Bureau.

The Japanese corporate consultant who has contracted with the Kenyan Government will thoroughly understand its role in this grant aid project and provide the following services:

- Detailed design including preparation of bid documents
- Prequalification of bidders and auxiliary services related to bidding and contract
- Construction supervision

The Japanese corporate contractor selected according to the grant aid system will procure equipment and materials efficiently and adequately and construct the planned facility in accordance with the plan.

3-1-1 Implementation Concept

The implementation concept for the plan takes the following points into account while assuming that the plan is implemented within the framework of Japanese grant aid:

- Maximum utilization of local laborers and materials/equipment to contribute to creation of job opportunities, promotion of technology transfer, and contributing to regional economic development
- Establishment of a close communication system among the Kenyan Government, consultant, and contractor to ensure smooth implementation
- Establishment of a practical plan based on consideration of the local rainfall pattern, the period necessary for procurement of materials/equipment, and employment of an appropriate construction method
- Employment of temporary structure plan and construction method complying with

Kenyan environmental standards and consideration of appropriate environmental measures

- Planning of the field work process which would minimize the inconvenience caused by closing the traffic

3-1-2 Implementation Conditions

1) Respect for laborer standard

The contractor will respect the appropriate laborer conditions and restrictions related to employment while complying with construction-related laws of Kenya to prevent disputes with laborers and to ensure the safety.

2) Customs clearance

Construction materials and equipment to be imported from Japan and a third country will be unloaded in Mombasa Port. In order to ensure smooth implementation, the cooperation of the Kenyan Government is indispensable during unloading and customs clearance.

3) Procurement

Materials and equipment procurable in Kenya are cement, concrete aggregate, steel bars, gas oil, gasoline, wood, and heavy construction machinery. Crack filler, steels such as prestressing steel and steel plates, expansion joint, railing members, etc. will be imported from Japan. There is a cement manufacturer called Banburi cement factory near the bridge concerned, which is sufficiently reliable in terms of production and quality.

4) Competence of local contractors

There are five construction-related quality enterprises registered in MORPW in Kenya, which are engaged as subcontractors in Japanese grant-aid and loan-aid projects. Certain local contractors have their own construction machinery, and construction machinery may be leased between private enterprises in certain cases. Accordingly, ordinary construction machinery can be procured from the private sector. Therefore, the opportunity for these enterprises to participate as subcontractors under supervision and guidance of the Japanese corporate contractor will contribute to the improvement of construction technology in Kenya.

5) Dispatch of engineers from Japan

Engineers will be dispatched from Japan for the type of work that may substantially affect the special technology or the quality of the work. This includes special technicians related to repair of cracks, prestressing work, steel plate manufacturing work, and steel plate adhesion work.

6) Securing of the land

Securing of the work yards necessary for rehabilitation and reinforcement work will be the responsibility of the Kenyan Government. For New Mtwapa and Nyali Bridges and sufficient work yards must be secured for rehabilitation. Provision of these yards is the responsibility of the Kenyan Government and the yards must be secured in time to avoid delays in commencement of the work. Possible candidate sites and applications are shown in Table 3.1.1.

Table 3.1.1 Temporary sites for the work

Location	Candidate site	Application
New Nyali Bridge New Mtwapa Bridge	① Right bank side of existing bridge ②Nationally-owned land near existing bridge	Service road and temporary office Construction material/equipment and machinery storage site, plant site, access road

7) Transfer of obstructions

MORPW will be informed of any materials that have to be transferred for the implementation plan. MORPW will coordinate with authorities concerned to determine measures that need to be taken.

8) Necessity of strict security at the site

To ensure the safety of the staff involved in this plan and to prevent theft of materials and equipment, the Kenyan Government is requested to provide special guards.

9) Traffic regulation

In order to ensure work efficiency, reduce procedures, ensure work quality, etc., it is necessary to install temporary scaffolding on the two bridges concerned. Though, in this case, temporary traffic regulations must be implemented, installation of the scaffolding will be made safely and rapidly while avoiding hindrance to the traffic flow. Before proceeding with installation, requests will be made to the Kenyan Government to take advance measures.

3-1-3 Scope of Works

The scope of the work undertaken by the Japanese and Kenyan Governments concerning implementation of the plan is outlined below.

1) Scope of work for Japan

- Provision and removal of the work yards
- Setup of the field office, warehouse, material storage yard, and work yards, including a material processing shop
- Transport of materials and equipment from Japan and a third country
- Crack repair and reinforcement (New Mtwapa Bridge) works
- Replacement of bridge accessories, such as expansion joints, railings, conduit pipes, etc.
- Replacing pavement of bridge sections

2) Scope of work for Kenya

- Providing of the work yard for the construction work. (Nationally-owned land)
- Pavement of sections other than bridge section

3-1-4 Consultant Supervision

1) Content of consulting services

The consulting service agreement will be concluded between MORPW, an implementing agency of Kenya and the Japanese corporate consultant. Major service contents included in the consultant service agreement are as follows:

a) Detailed design

The consultant will implement the detailed design for bridge rehabilitation according to the program outline determined as a result of this basic design survey; it will also prepare a complete set of bid documents.

b) Constructor selection phase

The consultant will assist MORPW concerning the following services related to selection of Japanese constructors:

- Invitation for bids
- Pre-qualification
- Bid presentation and site explanation

- Bid evaluation
- Contract negotiation

c) Consultant supervision phase

Upon receiving the notice to commence issued by MORPW to the contractor, the consultant will start supervision service. This service includes management on the work progress, quality, and safety, along with assistance to MORPW concerning the payment procedure to the contractor. The consultant will also report the work progress to MORPW, the Japanese Embassy, and JICA.

2) Implementation system

a) Detailed design implementation system

Detailed design and preparation of bid documents by the consultant require Japanese engineers as shown in Table 3.1.2.

Table 3.1.2 Detailed design staff

Engineers	Service
Chief Consultant	Generalization of all services related to rehabilitation plan, preparation of bid documents, and bidding
Bridge engineer / Rehabilitation Planner	Rehabilitation plan, implementation design, preparation of drawings, quantity calculation
Bridge engineer (superstructure)	Rehabilitation plan, implementation design, preparation of drawings, quantity calculation
Estimates／bid documents	Price survey, estimates, preparation of bid documents

b) Consultant supervision staff plan

The consultant supervision system of the consultant requires engineers shown in Table 3.1.3.

Table 3.1.3 Consultant supervision staff

Engineers	Service
Chief Consultant	Guidance of resident engineer, generalization
Resident engineer	Generalization of field services, quality, progress, payment, safety control, payment procedures, coordination and negotiation with the Kenya counterpart, and report
Bridge engineer (in charge of rehabilitation)	Quality of rehabilitation works

3) Construction implementation plan

a) Temporary work（Common to two bridges）

Upon issue of the notice to commence from MORPW, the contractor will immediately enter Kenya and set up the main office in Mombasa. The contractor will also start procurement of materials and equipment and local subcontractors while setting up the work yard at New Mtwapa bridges. Following temporary facilities will be constructed in the yard. Note that the required yard area in the site is estimated to be about 3,000m^2.

① Consultant field office
② Constructor (main contractor) site office
③ Local subcontractor offices
④ Laborer rest facility
⑤ Warehouse
⑥ Material and equipment storage yard
⑦ Material processing shop
⑧ Guard station

b) New Nyali Bridge

i) Work flow chart

The bridge rehabilitation work will be implemented in the sequence shown in Fig.3.1.1.

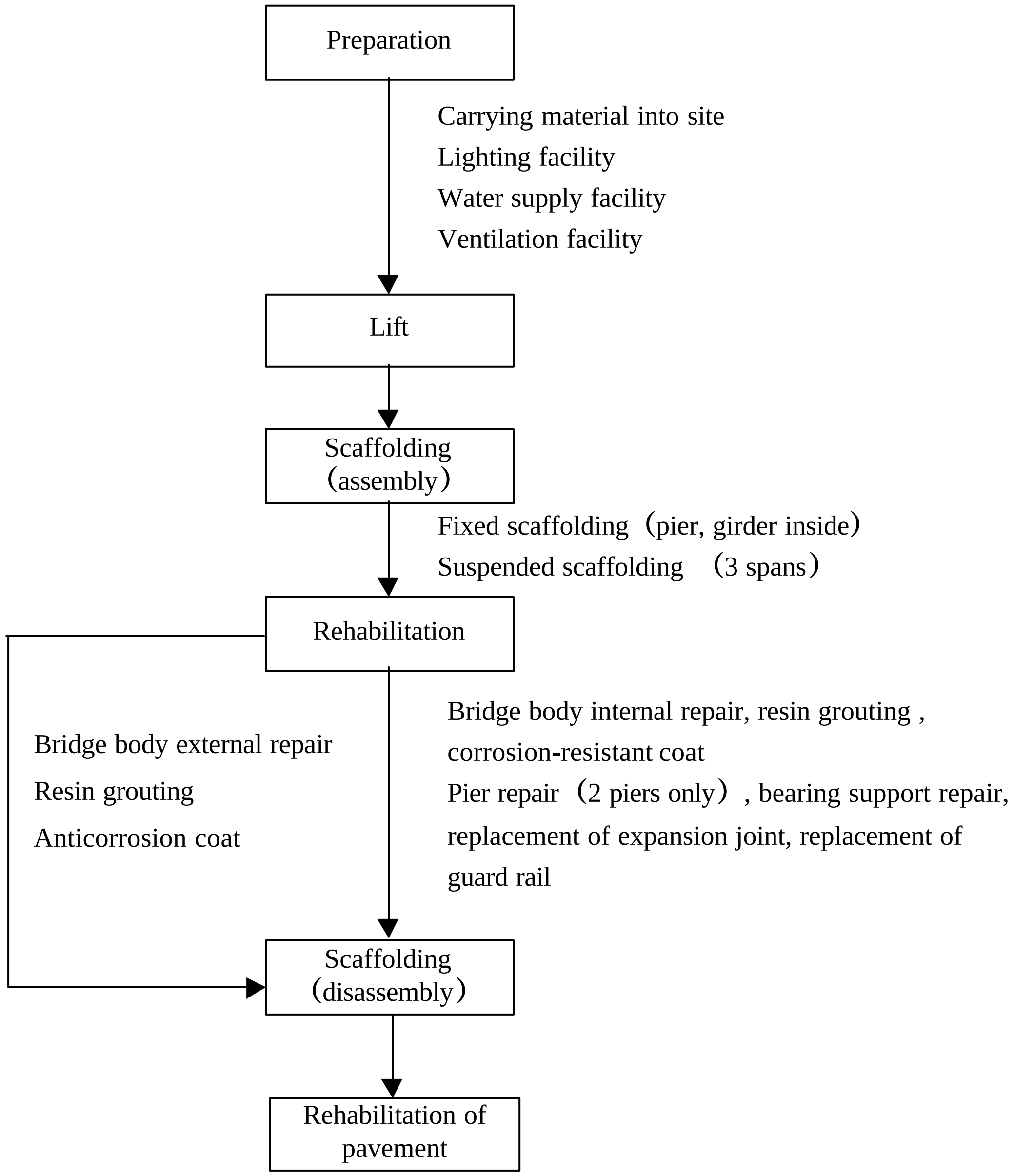


Fig. 3.1.1 Work sequence for the New Nyali Bridge

During the work, attention must be paid to the following points:

- Implementation of a well-thought-out plan because much of the work will be done while regulating traffic on roads in use.
- Ensuring safety facilities because most rehabilitation work will be done at elevated points above the sea
- Equipment and plan with due consideration of environment in terms of water pollution, etc. because this bridge rehabilitation project is within the national park

ii) Scaffolding

Fixed scaffolding to be used will be of a frame type, with pipe scaffolding used also on the slope. Temporary suspended scaffolding will be used for rehabilitation of the three spans of superstructure.

iii) Rehabilitation

① Hexagonal crack in pier concrete (Fig. 3.1.2)

- Deteriorated concrete portion will be chipped to a sound portion on the backside of steel bars.
- The form will be fixed with separators and spacers that have been welded to the steel bar, and concrete will be placed using a pump car on the bridge surface.
- After the specified curing period, the entire concrete surface will be protected with polymer cement mortar.

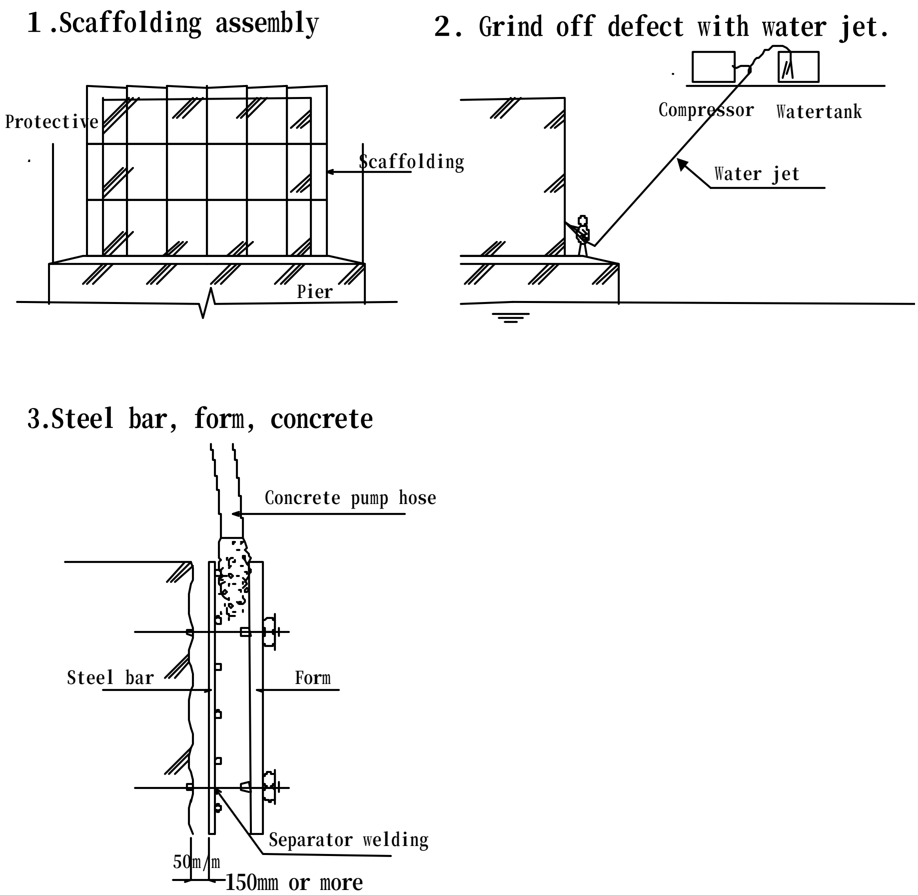


Fig. 3.1.2 Pier repair

② **Crack in girder**

- Using the temporary suspended scaffolding for repairs (repair gondola), the cracked portion will be re-checked. Epoxy resin will be grouted in the cracks for repair.（Fig. 3.1.3）
- During the work, vehicle traffic area will be secured and the traffic guide will be positioned to minimize blockage of traffic as much as possible.

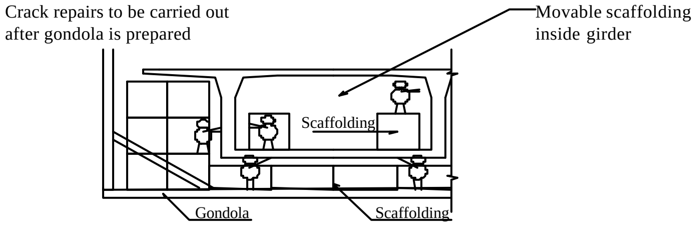


Fig. 3.1.3 Girder crack repair

③ **Bearing support**

- After assembling of the temporary scaffolding, rust in corroded portions of bearing support will be removed with chemicals and wire brush, and an anticorrosion coating will be applied.

④ **Expansion joint**

- The existing expansion joint will be removed and replaced by a non-drain type finger joint.

⑤ **Drain pipe and basin**

- The drain basins inside and along girder sides will be cleaned and repaired with polymer cement mortar.
- Steel drain pipes will be extended to a point under the girder.
- Drain pipe brackets will be hot dip zincked.

⑥ **Railings, guard rails**

- Damaged railings and guard rails will be replaced.

⑦ **Pavement**

- The work zone will be secured for replacement of pavement by regulating traffic.

c) New Mtwapa Bridge

i) Work flow chart

Bridge rehabilitation work will be made in the sequence shown in Fig. 3.1.4.

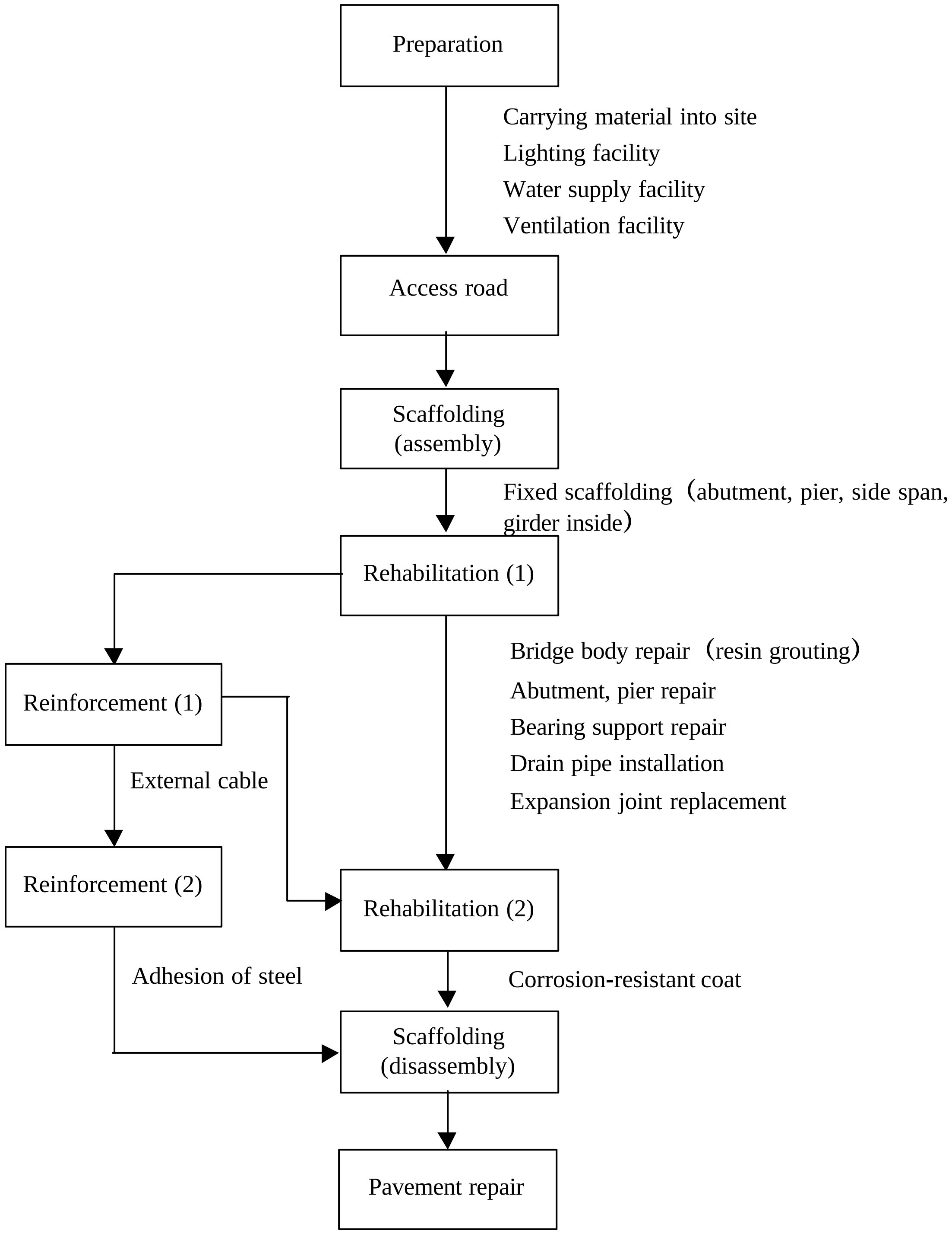


Fig. 3.1.4 Work sequence for the New Mtwapa Bridge

During the work, attention must be paid to the following points:

- Implementation of a well-thought-out plan because much work will be done while regulating traffic on roads in use.
- Ensuring safety facilities because most of rehabilitation is made at elevated points above the sea
- Equipment and plan with due consideration for the environment in terms of water pollution, etc. because this bridge rehabilitation project is made within the national park

ii) Scaffolding

- Fixed scaffolding to be used will be of a frame type, with pipe scaffolding used also on the slope.
- The repair gondola will be manufactured according to a concept similar to that for the New Nyali Bridge. The gondola will be used for the center span only and will be of a construction readily compatible with changes in girder height.

iii) Rehabilitation

① **Cracks in the outside wing wall of abutment**

- The repair scaffolding will be assembled for repair of cracks by epoxy resin grouting.

② **Cracks in girder**

- Repair of cracks in girder will be the same as for the New Nyali Bridge and the use of the repair gondola.

＜Crack width 0.2mm or more＞

As shown in Fig. 3.1.5, the special epoxy resin will be grouted deep inside the crack under low pressure and at low speed over a long period of time, integrating the bridge body.

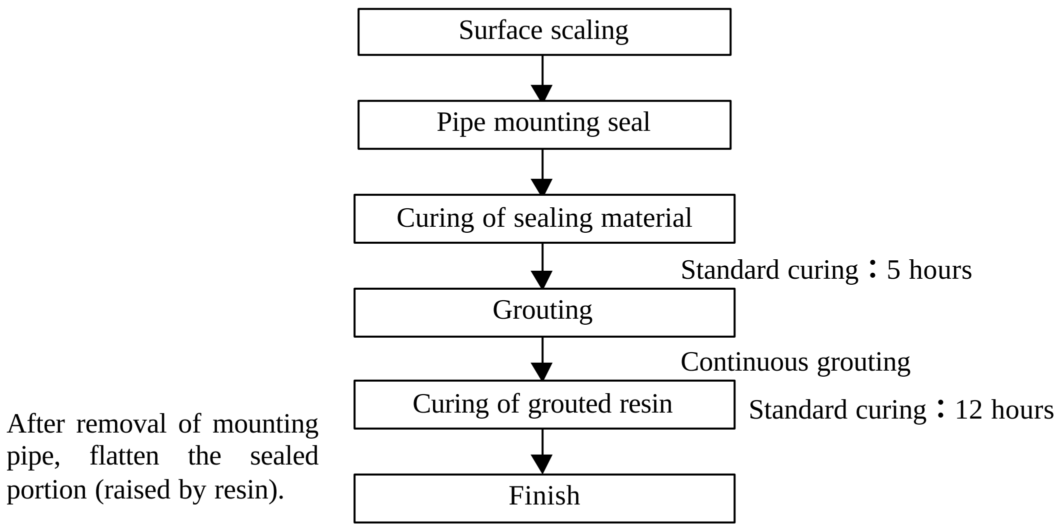


Fig. 3.1.5 Resin grouting procedure

＜Crack width less than 0.2mm＞

For cracks less than 0.2mm, the concrete surface will be protected as shown in Fig.3.1.6 to prevent deterioration of concrete. (corrosion-resitant coat)

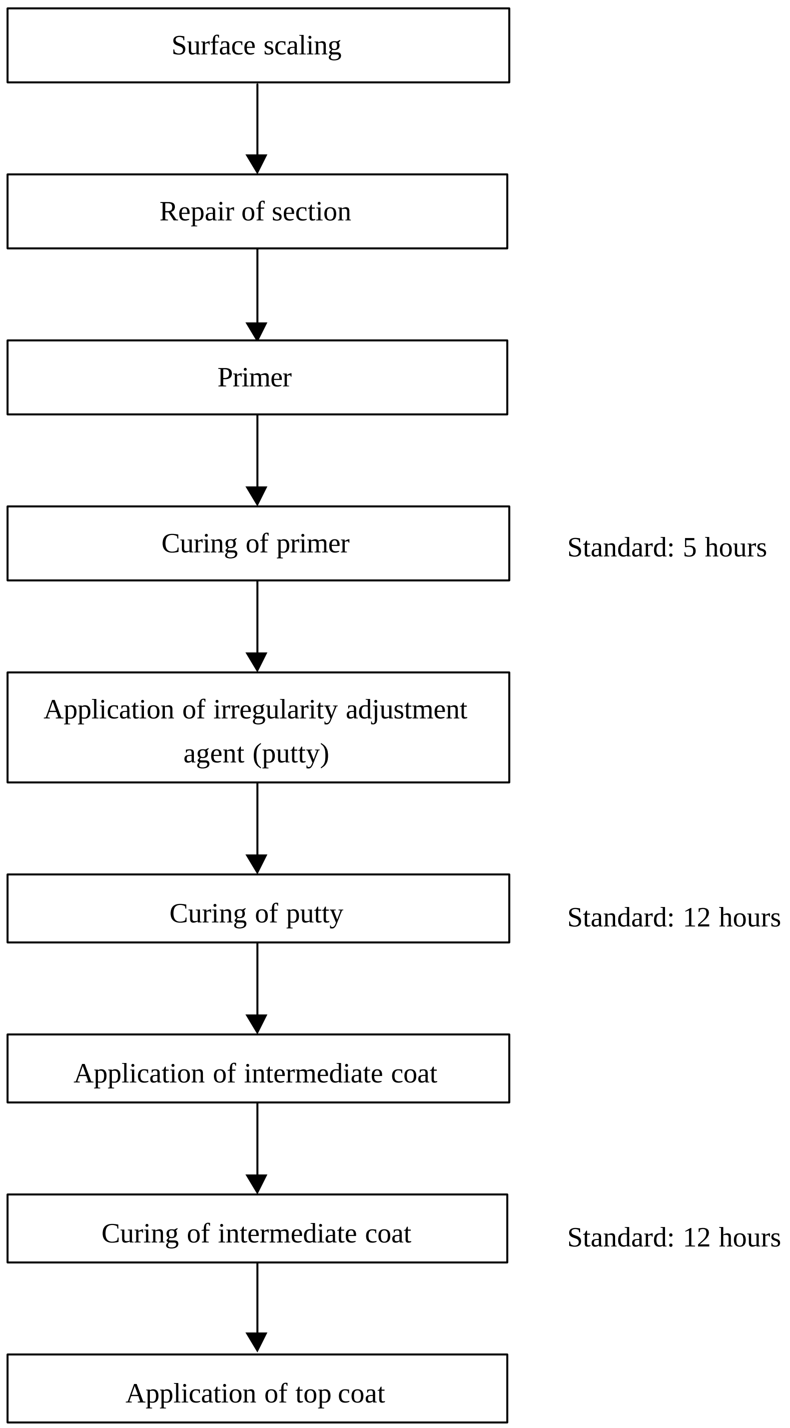


Fig. 3.1.6 Corrosion-resistant coating procedure

③ **Bearing support**

- ◆ Rust will be removed from the shoe and anticorrosion coating will be applied.

④ **Drain pipes and basins**

- ◆ The drain basins inside and along girder sides will be cleaned and repaired with polymer cement mortar.
- ◆ Steel drain pipes will be extended to a point under the girder.
- ◆ Drain pipe brackets will be hot dip zincked.

⑤ **Expansion joint**

- ◆The existing expansion joint will be removed and replaced by a new one.

⑥ **Pavement**

- ◆ The work zone will be secured by regulating traffic for replacement of pavement.

iv) Reinforcement

Bridge body reinforcement will be done using external cables and steel plate adhesion.

＜External cable work＞

The external cable will be installed to suppress opening of repaired crack.

①Layout of external cable

The reinforcement material will be installed in cable anchoring and deflection portions of existing crossbeams. Holes will be drilled in existing crossbeam to pass through the cable. (Figs. 3.1.7 and 3.1.8)

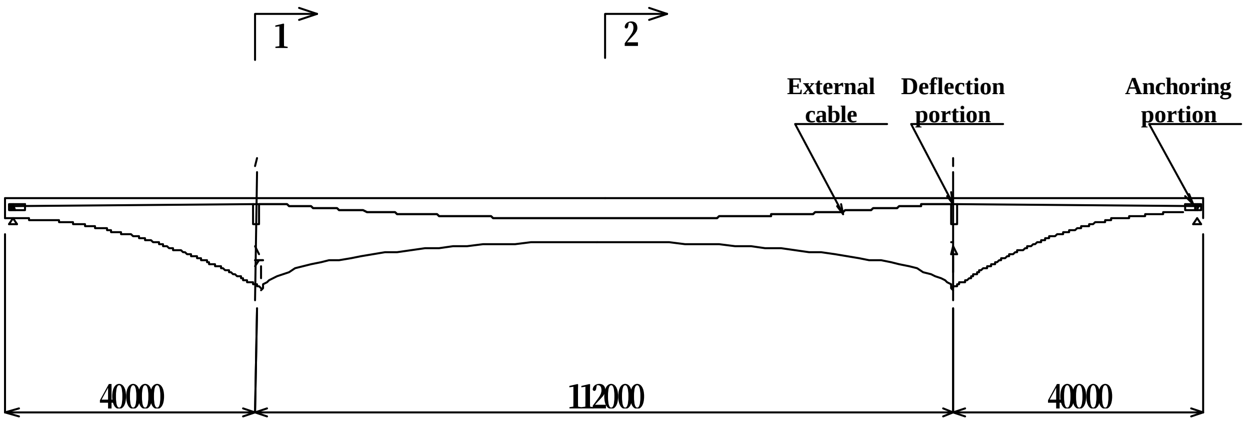


Fig. 3.1.7 External cable layout (side view)

Arrangement in girder

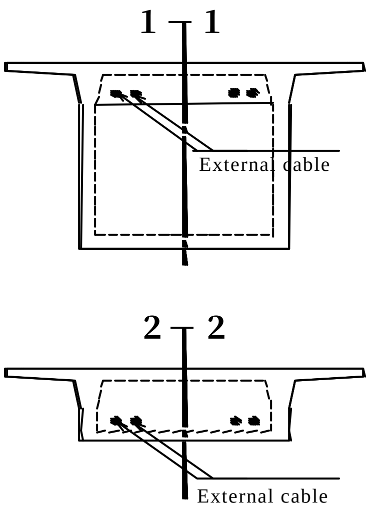


Fig. 3.1.8 External cable layout (sectional view)

②Sequence of external cable installation

The external cable installation sequence is shown in Fig.3.1.9.

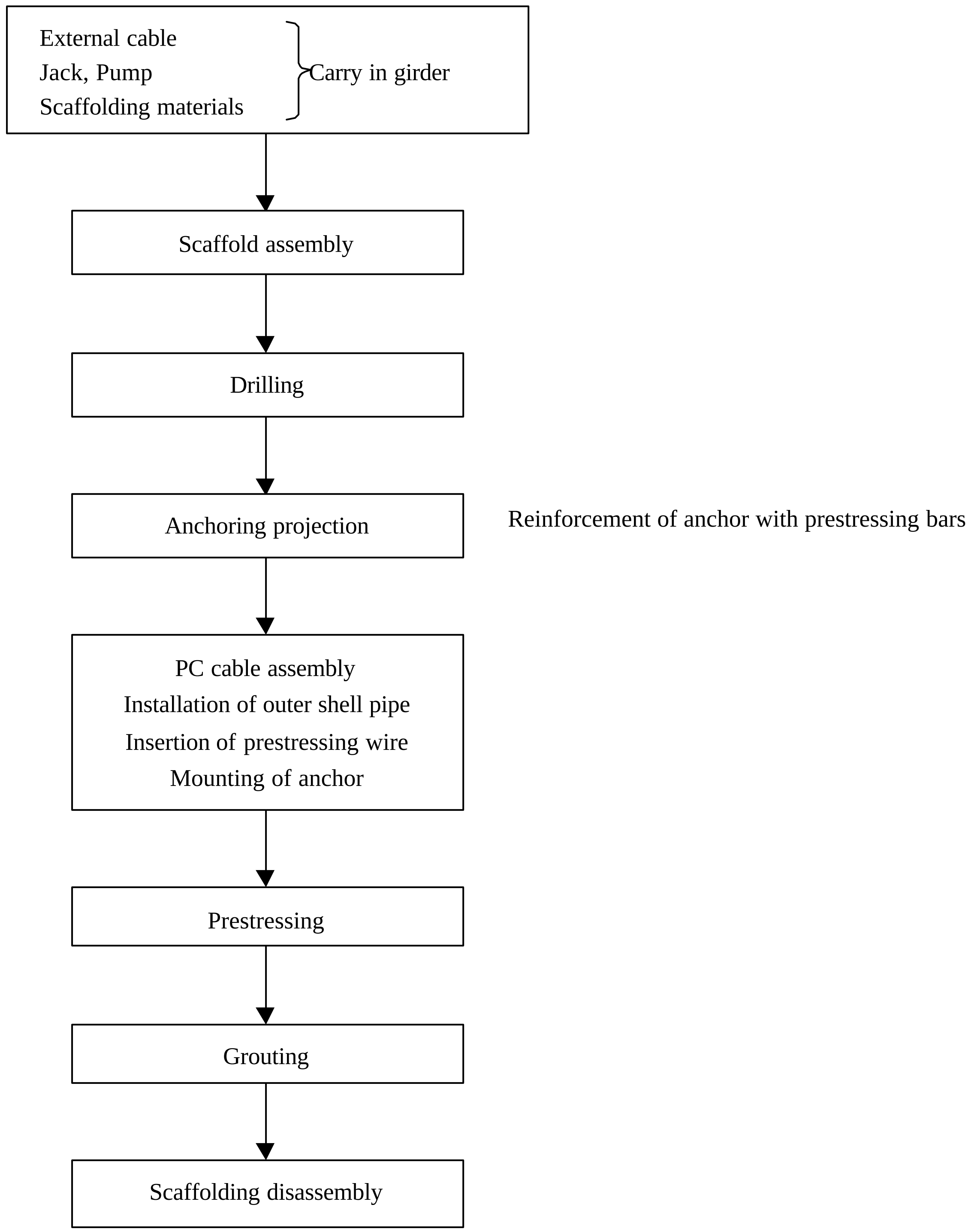


Fig. 3.1.9 External cable installation procedure

＜Shear reinforcement of girder web＞

Shear reinforcement will be made as follows. Scaffolding to be used is a simple scaffolding that will be set up only inside the girder.

①Shear reinforcement method

As shown in Fig. 3.1.10, steel plates will be adhered to the web inside.

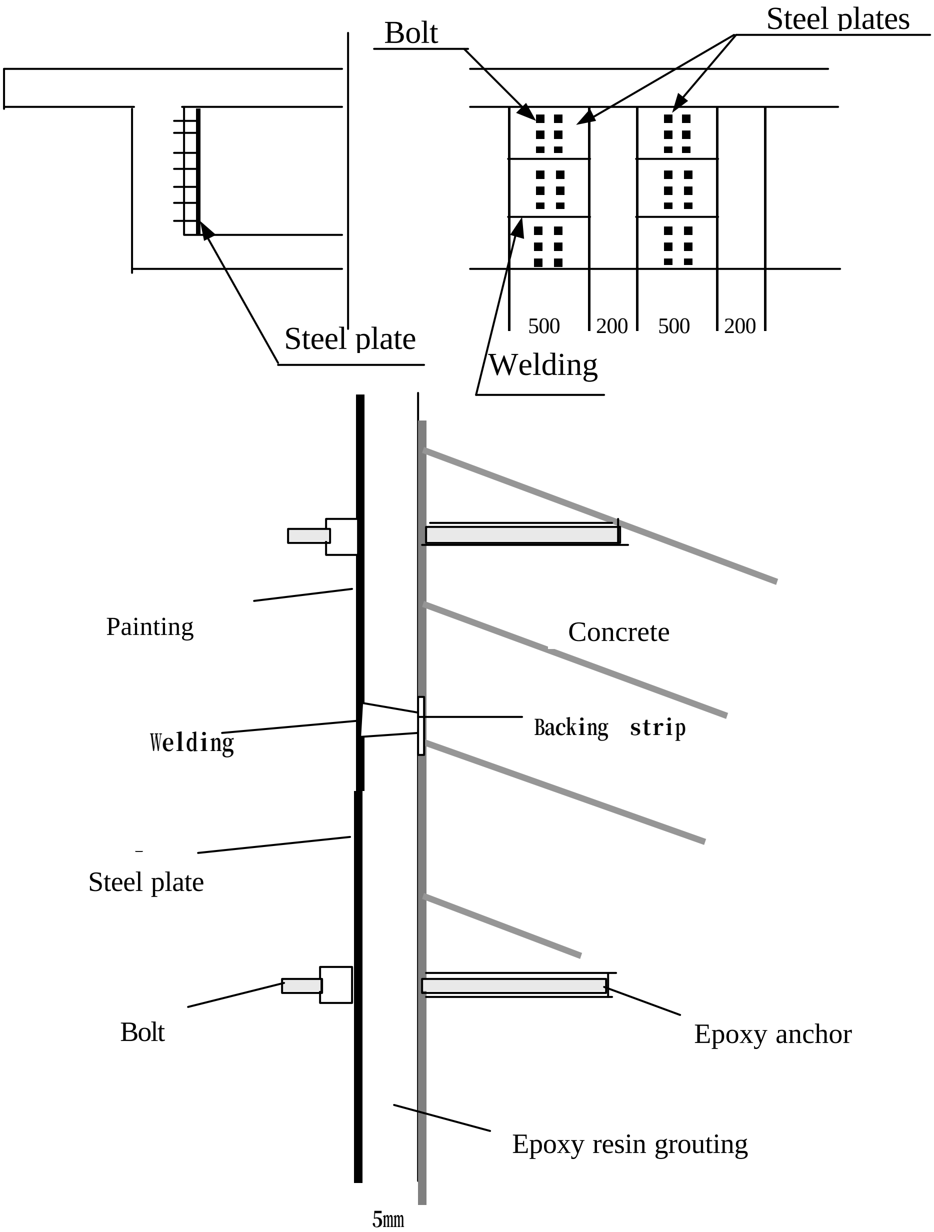


Fig. 3.1.10 Steel plate adhesion procedure

②**Steel plate adhesion work procedure**

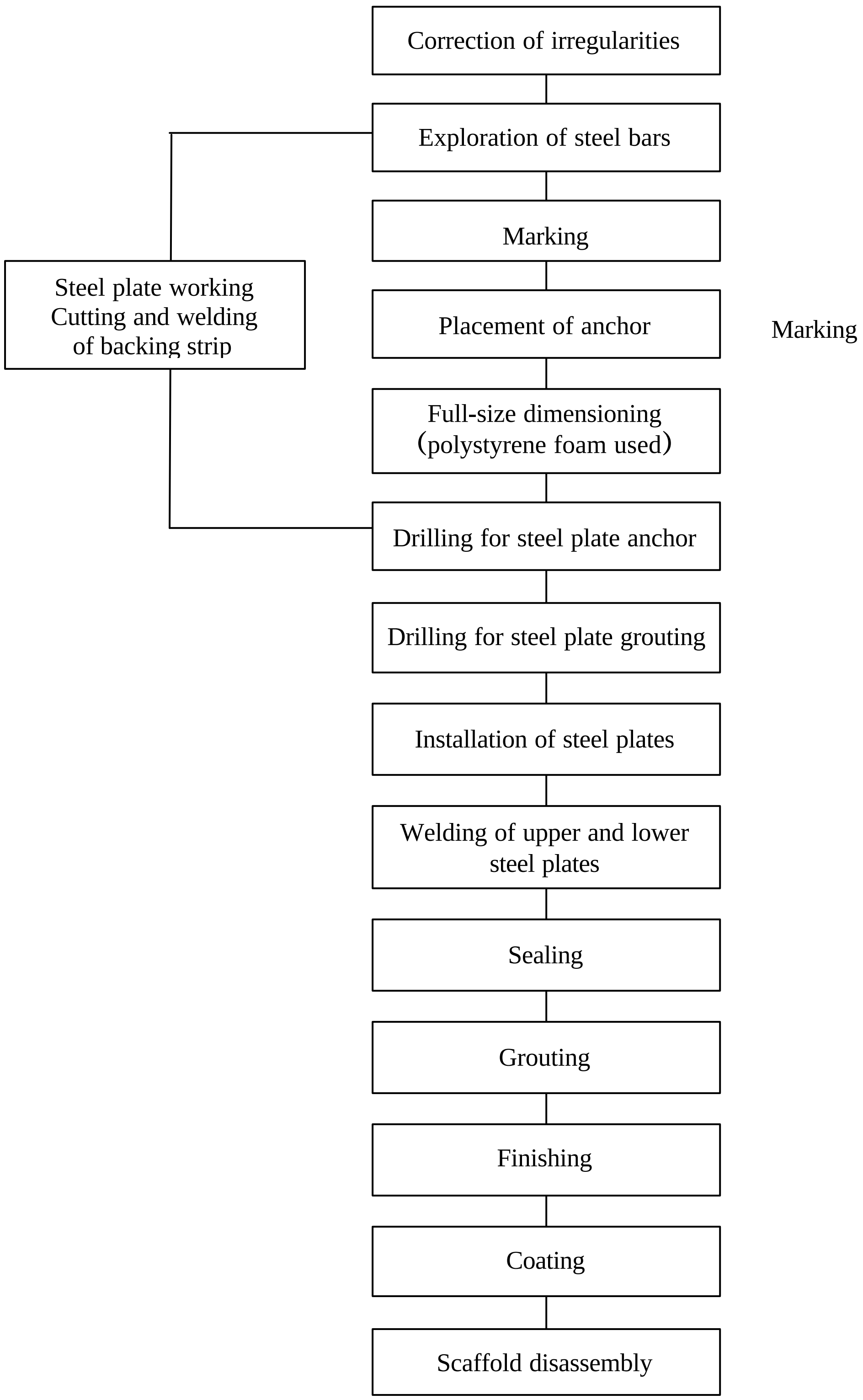


Fig. 3.1.11 Steel plate adhesion sequence

③**Work method**

◆Correction of irregularities

Scaling and correction of irregularities will ensure uniform intervals between steel plate and slab as well as to strengthen adhesion of the slab bottom surface and steel plate before installation of steel plates.

◆Exploration of steel bars

Exploration will determine position of steel bars that may hinder anchor placement.

◆Processing of steel plates

Processing of steel plates will be done using full-size foam polystyrene. (anchor, air purge).

◆Anchor placement

The anchor placement position will be marked, and anchor bolts (epoxy resin type anchor) will be installed into drilled holes.

◆Mounting of steel plates

When steel plates are installed, spacers will be installed near the anchor to secure a gap between the steel plate and slab bottom surface. Splice plate will be clamped with washers to secure a gap between reinforcement steel plates and splice plates.

◆Sealing

After grouting of sealing resin, the steel plate bottom surface will be given a smooth finish.

◆Grouting

After curing of sealing resin, epoxy resin will be grouted between the slab and steel plate. Appropriateness of the grouting condition will be confirmed with an inspection hammer.

◆Finishing

After curing, grouting and air purge pipes will be cut and finished with a sander.

◆Coating

After cleaning of the coating surface, coating will be applied by brush and attention will be given to meteorological conditions.

3-1-5 Procurement Plan

1) Labor condition

Kenya is suffering economic stagnancy due to delayed economic aid from foreign countries and international agencies as well as some outflow of a foreign capital due to fear of recent deterioration of security.

In this economic situation, the labor market is characterized by over-supply. This situation is favorable for obtaining unskilled laborers, skilled laborers, and construction technical workers with relative ease.

Among technical workers, those of engineer class are graduates from Nairobi University and Jomo Kenyatta Agricultural Engineering University. About 90 are civil engineers graduate every year.

In Kenya, both skilled and unskilled laborers tend to be excess in supply, and basically, there will be no need to seek laborers in a third country. Besides, work permits are difficult to obtain for laborers from third countries.

2) Construction machinery

Construction machinery includes that possessed by the governmental agency and that belonging to private enterprises. Generally, construction companies will conduct the work by using the machinery in their possession. Leasing of machinery from private enterprises will also be done.

There is a shortage of heavy machinery for bridge construction in Kenya and only a few skilled heavy machine operators. Accordingly, it is mandatory that these be procured from a third country. Unless thorough maintenance is ensured for construction machinery, the work progress will be hindered.

Similarly, procurement of spare parts is difficult in Kenya. It is essential to procure in advance the required quantity based on consideration of the work period and process required for procurement from a third country.

a) Construction machinery and plant held by the governmental agency

The construction machinery held by the Kenyan Governmental agency is shown in Table 3.1.4, which cannot be leased to ordinary contractors. In certain exceptional cases, machinery for the project may be leased from a competent governmental agency. In principle, private constructors cannot lease machinery.

Table 3.1.4 Construction machinery held by governmental agency

Name of machine	Type, specification, capacity	Quantity
BULLDOZER	D6	10
	D7	7
	D8	2
MOTOR GRADER	3.1M	20
	3.7M	10
WHEEL LOADER	1.5M3	10
DUMP TRUCK	5-10 TON	30

* The quantity and machinery actually held by the agency may differ to some extent.

b) Construction machinery and plant that can be procured in Kenya

Ordinary construction machinery can mostly be procured in Kenya. But the available quantity is limited and the operating rate is extremely low. During the project period, machines may have to be brought into Kenya from third countries and a sufficient quantity of replacement parts must be procured with due consideration for the limited time for the work.

The construction machinery procurable in Kenya is shown in Table 3.1.5.

Table 3.1.5 List of construction machinery procurable within Kenya

Name of Machine	**Type, specification, capacity**	**Quantity held by each enterprise**			
		CRESCENT	ASSOICIATED	BAINS	SPENCON
BACK HOE	0.5M3	1	1	2	6
BACK WHEELED	1.0M^3		1	1	10
BULLDOZER	D5		1		
	D6	1	2	1	6
	D7		1	1(D9)	5
	D8	1	2	2	2
MOTOR GRADER	3.1M		4	2	5
	3.7M		1	2	2
WHEEL LOADER	1.5M^3		1	1	5
	1.5-2.5M^3		2	2(3M^3)	2
	>2.5M^3	1	2	1	2
TRAILER	40 TON	1	20		2
COMPRESSOR	175 C.F.M		4		10
	176-349	1	1		16
	350 C.F.M				

Name of machine	Type, specification, capacity	Quantity held by each enterprise			
		CRESCENT	ASSOICIATED	BAINS	SPENCON
DUMP TRUCK	5 TON			5	
	6-9 TON		10		32
	10-15 TON		10		
	16 TON	5	10	7	
CONC. MIXER CAR	3-6M^3	1	2		
VIBRATION ROLLER	5 TON	2	2	1	6(8 TON)
	12 TON		5	1	6(10 TON)
TYRE ROLLER	8-20 TON	2	4		6
MALADAM ROLLER	10 TON		2		10
ASPHALT PLANT	100TON-200 TON/H	1	1		2
DISTRIBUTOR	2000	1			
	6000		1		4 (5000)
TRUCK CRANE	10 TON		1		2
CRAWLER CRANE	40 TON		1		
STONE CRUSHING	50 TON/H	2			2(10 T/HR)
PLANT	100 TON/H		1		
GENERATOR	50 KVA	1	2		
	50-100 KVA		1		6
	200 KVA				6

c) Construction machinery held by foreign capital contractors (Yugoslavia, Israel, UK)

Table 3.1.6 shows principal machinery held by foreign contractors (Yugoslavia, Israel, UK)(as of January, 2000). Note that most of them are held for specific projects and re-exported after completion.

Table 3.1.6 Construction machinery held by foreign constructors

	Name of company	Machines	Specification/capacity
1. Yugoslavia	PUT SARAJEVO	Backhoe	0.5M3/1.0M3
		Concrete plant	0.5M3/H
		Dump truck	15/21TON
		Motor grader	4.3m
		Vibration roller	30/60TON
		Crawlwer crane	10-20Ton
		Macadam roller	8-14Ton
		Bulldozer	
		Wheel loader	30TON/35TON
		Compressor	2.1 M3
		Generator	8-12M3/mini
		Dump truck	75,125,175,250KVA
		Concrete plant	10、15Ton
		Asphalt plant	50M3/Hr
			80Ton/hr
2. Israel	ZAKHEM	Dump truck	10.0,16TON
		Backhoe	0.5,1.0M3
		Crawler crane	40,80TON
		Bulldozer	25,35TON
		Wheel loader	1.5,2.5M3
		Motor grader	3.7M
		Generator	100KVA/50KVA
		Asphalt finisher	
		Asphalt plant	3.0m—4.0m
			120Ton/H
3. UK	MOWLEM	Backhoe	0.6M3/
		Generator	50,125KVA
		Compressor	6.0M3
		Concrete carrier	12.0M3/mini
			6.0M3

d) Construction machinery that must be procured outside Kenya

Procurement of special construction machinery is difficult in Kenya and procurement of the machinery outside of Kenya is necessary to ensure smooth execution of the work. Table 3.1.7 shows the machinery to be transported from the outside into Kenya.

Table 3.1.7 Machinery to be imported

Machine	Specification/capacity
Grout mixer and pump	1 m^3
Water jet	max2,000Kgf/cm^2
Prestressing jack and pump	200Ton class

e) Maintenance of construction machinery

Maintenance of construction machinery is an extremely important factor that determines the success of the project. All raw material for fuels and oils/greases are totally imported by Kenya. Fuel supply is enough to meet the demand, but an import plan must be established concurrently with machinery selection because certain machines use special oil and grease. Machine parts should be secured at the site after advance confirmation of the model of machines possessed by local contractors and even for machines that contractors bring into Kenya for controlled use under lease over a long period of time.

3) Construction materials

It is necessary to identify construction materials, particularly, materials necessary for bridge construction that are procurable in Kenya and those that must be imported.

a) Cement

There are two major cement manufacturers in Kenya: Banburi Cement Factory (Mombasa) and EAPC (Nairobi). Both can produce cement of reliable quality that complies with BS and EN, KS standards.

b) Concrete

There is no company in Coast province that can produce and supply ready-mixed concrete. Before start of construction, it is essential to consider procurement of the concrete plant from a third country.

c) Asphalt mixture

Enterprises that can produce and supply asphalt mixtures are shown in Table 3.1.8.
These enterprises are within a distance of 20 km from the site and ensure problem-free concrete supply.

Table 3.1.8 Asphalt manufacturers and capacity

Asphalt manufacturers and capacity	Plant location
Cresent Construction Co., Ltd. 20 Ton /h & 16 Ton /h	Militini
S.S.Metha Construction Co., Ltd. 50ton/h ＆ 40 Ton /h(Mobile)	Kokotni（Near Mazeras）

d) Steels （Steel bars and members）

Steel bars can be produced in Kenya. KUSCO in Mombasa produces steel bars of 12 m in length and 8 – 32 m in diameter in compliance with BS 449 standard. They can also obtain the required quality certificate. However, special and large steels are not obtainable within Kenya and PC steels, earth-retaining members, and steel piles must be imported from a third country.

e) Concrete aggregate and road embankment material, base course material

Concrete aggregate is available in Kenya. Within a 80 km distance from the site there are three companies of KD, KV, and Jaribuni. They have an aggregate pit, offering satisfactory quality and quantity.

f) Pavement material （bituminous material）

Eres Co. Ltd., Special Bitumens Ltd., and Shell produce and supply this material. It is satisfactory both in quality and quantity. Each company has also the means of transport to the site, supplying the material in bulk trucks and drums.

g) Wood

Almost all the wood needed is available in Kenya. Plywood 12 mm and 15 mm thick for forms is readily available. The quality is relatively good and applicable for construction of the bridge. However, water-resistant form material is not procurable in Kenya and must be imported from a third country.

h) Other construction materials

Concrete blocks and PVC pipes are supplied in sufficient quantity in Kenya. Others, particularly, special materials such as prestressing steels necessary for bridge construction must be totally imported. Table 3.1.9 shows suppliers of principal construction materials.

Table 3.1.9 Suppliers of principal construction materials

Construction materials	Procured in Kenya	Procured in Japan	Procured in a third country	Remarks
Cement	*			
Concrete chemical admixture		*	*	
Steel bars	*		*	
Structural steels		*	*	
Prestressing steel wires and steel bars		*	*	
Bituminous material	*			
Crushed stone, sand	*			
Ordinary wood	*			
Forms (plywood)	*		*	
Support and scaffolding material	*		*	
Expansion joints		*		
Bearing supports (rubber)		*		

4) Local contractors

a) Local contractors （Governmental and private）

As contractors engaged in bridge construction, there are private survey/research companies, design companies, and contractors. Enterprises listed below are quality companies registered in MORPW.

i) Construction companies

①. Private

(1) Associated Construction Co., Ltd.
(Elgon Road, Nairobi)

(2) Tm-Am Construction Co., Ltd.
(Enterprise Road, Nairobi)

(3) Spencon Construction Co., Ltd.
(Mpaka Road, Nairobi)

(4) Crescent Construction Co., Ltd.
(Militni, Mombasa)

(5) S.S. Metha Construction Co.,Ltd.
(Haile Selassie Road, Mombasa)

ii) Consulting companies

①. State-run enterprises

(1) MOPWH

(Ngong Road, Nairobi)

②. Private enterprises

(1) H. P. Gauff
(Church Road, Nairobi)

(2) Otieno Odongo & Partners
(Ngong Road, Nairobi)

(3) Howard Humphreys
(Waiyaki Way, Nairobi)

b) Foreign construction companies outside Kenya

Foreign construction companies currently in operation within Kenya are shown in Table 3.1.10.

Table 3.1.10 Foreign construction companies

Companies	Address
Skanska International Civil Engineering AB (Sweden)	Baba Dogo Road, Nairobi
Strabag A-G Lima Ltd. (Germany)	Kasarani Road, Nairobi
Stiring Civil Engineering (Italy)	Ngong Road, Nairobi

3-1-6 Implementation Schedule

This plan will be implemented as shown in schedule below after conclusion of the Exchange of Notes.

1) Implementation design

The implementation design will be made and design and bid documents prepared after conclusion of the consulting agreement.

2) Bidding and contractor agreement

The project agreement will be a direct one between the Kenyan Government and Japanese contractor. Selection of Japanese contractor will be based on open tendering addressed to Japanese contractors.

Examination items will be discussed beforehand with JICA for approval, then prequalification of Japanese contractors will be made. A consulting company that was entrusted by the implementing agency of the Kenyan Government will handle prequalification.

The contractors who have passed prequalification will be requested to submit bid documents. Bid opening will be made in the presence of the consulting company, Kenyan Governmental staff, and bidders, and witness representing JICA. The construction agreement will be concluded after bid examination and determination of successful bidders.

In parallel with conclusion of the construction agreement, the Kenyan Government will conclude the banking arrangement as soon as possible with a Japanese authorized foreign exchange bank. The banking arrangement is the basis on which the Kenyan Government will issue the Authorization to Pay (A/P) necessary for reception of aid funds from the Japanese Government and advance payment to contractors as well as for application to obtain the export license from MITI. This is also necessary to commence project implementation simultaneously with conclusion of the construction agreement.

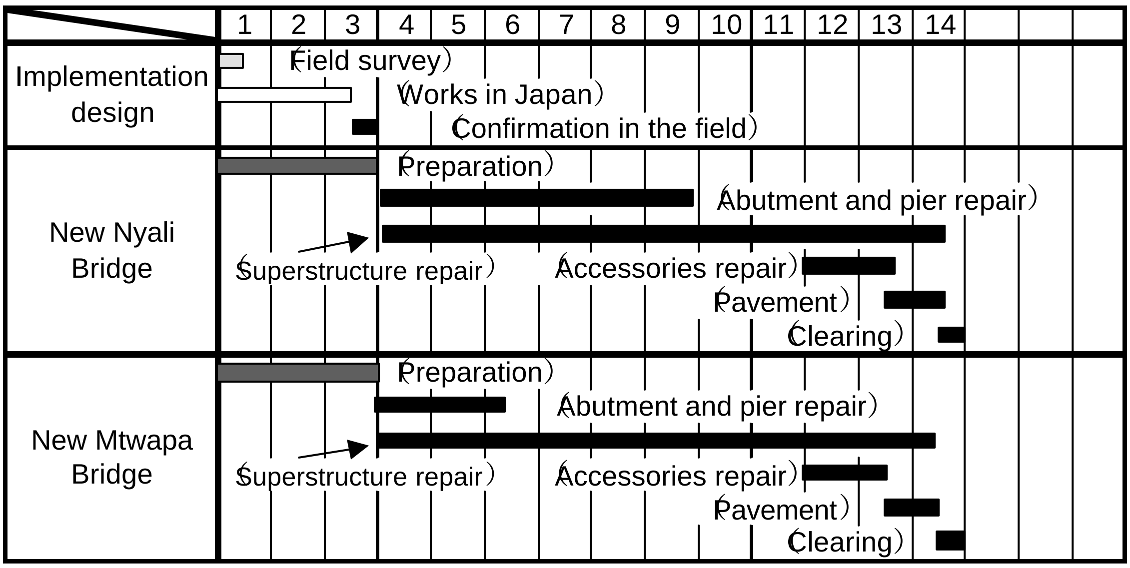
Then, approval of the contract is necessary. Approval means that the Japanese Government verifies appropriateness of the contract as an object of this grant aid. It is also a prerequisite for the contract to go into effect.

3) Construction work

The construction work begins with preparation; followed by transport of Materials into site, crack repair, bridge main girder reinforcement, bridge accessories reinforcement, railing repair, and pavement work. Around the site in Kenya, the period from March to June is a major rainy season, and the period from October to December or January the next year is a minor rainy season. During these periods, the rehabilitation work will be restricted.

The plan implementation schedule is shown in Table 3.1.11. This plan will be based on a single year, during which design and construction will be completed. The design period is estimated to be three months while the construction period will be 14 months.

Table 3.1.11 Implementation schedule



3-1-7 Obligations of Recipient Country

During implementation of this plan, the Kenyan Government will implement the following matters:

① Supply of data and information necessary for implementation of the plan
② Securing of the land (work yard, etc.) necessary for implementation and associated relocation of residents
③ Opening of a bank account in Japan and issue of authorization to pay
④ Cooperation for rapid unloading of imported materials and equipment and smooth inland transport
⑤ Exemption of Japanese corporation or Japanese people involved in this plan from duties and internal revenue levied within Kenya on the products or services supplied according to the approved contract
⑥ Entry permits for Japanese nationals into Kenya and resident permits for implementation of the work concerning supply of service based on the approved contract
⑦ Approvals and licenses necessary for implementation of the project
⑧ Appropriate and effective maintenance of facilities constructed for the project
⑨ Bearing all expenses other than those to be born under Japanese grant aid within the scope of work of the project

3-2 Operation and Maintenance Costs

Upon completion of the project, the Kenyan Government will operate and maintain the rehabilitated bridge.

3-2-1 Maintenance method

For effective utilization of Kenya's limited available funds, the maintenance method mainly comprising daily and periodic inspections will be employed to ensure early detection of damage and early countermeasures, preventing thereby major damage to the bridge main body and accessories.

① Daily inspections
An inspection vehicle will be run on the route concerned about once a month for visual appearance inspection of the bridge body and accessories, including inspection of the main girder inside. The condition will be recorded in a record form to be delivered to the engineer. The inspection crew will consist of three persons per vehicle, including an inspector, recorder, and driver.

② Periodic inspections

During the rainy season, drain basins and distribution pipes will frequently be clogged with dust. Periodic inspection will therefore be made on locations repaired in this project. The engineer will carry out inspection and report the result to the supervisor regardless of whether or not any damage is found. The repair plan will be drafted according to the damage condition.

On the basis of these inspection results, the engineer and his supervisor will judge the necessity of repair and implement repairs early to prevent worsening.

Table 3.2.1 is the table for selection of repair and reinforcement methods according to the type of damage.

Table 3.2.1 Repair and reinforcement methods

Type	Cause	Repair method								Reinforcement method					
		Surface treatment method	Resin injection method	Filling method	Mortar spray method	FRP adhesion method	Concrete replacement method	Corrosion prevention through neutralization	Corrosion prevention through salination	Additional concrete placement	Steel adhesion method	Prestress introduction method	Member lapping method	Girder addition method	Column addition method
Crack	Insufficient area of reinforcement, insufficient development length	×	×	×	△	×	×	×	×	○	◎	○	○	○	○
	Insufficient prestressing	×	×	×	×	×	×	×	×	△	○	◎	○	○	○
	Defective concrete quality, insufficient strength	×	×	×	×	×	×	×	×	△	△	△	○	○	○
	Faulty work (faulty compaction, curing)	×	×	×	×	×	×	×	×	△	△	△	○	○	○
	Increase in traffic volume, excessive wheel load	×	×	×	△	×	×	×	×	◎	◎	◎	◎	◎	◎
	Vehicle contact, etc.	×	○	×	×	×	×	×	×	×	○	×	×	×	×
	Unknown	△	△	△	△	△	△	△	△	△	△	△	△	△	△
Free line, water leakage	Insufficient area of reinforcement, insufficient development length	×	×	×	△	×	×	×	×	○	◎	○	○	○	○
	Insufficient prestressing	×	×	×	×	×	×	×	×	△	○	◎	○	○	○
	Defective concrete quality, insufficient strength	×	×	×	×	×	×	×	×	△	△	△	○	○	○
	Faulty work (faulty compaction, curing)	×	×	×	×	×	×	×	×	△	△	△	○	○	○
	Increase in traffic volume, excessive wheel load	×	×	×	△	×	×	×	×	◎	◎	◎	◎	◎	◎
	Vehicle contact, etc.	×	○	×	×	×	×	×	×	×	○	×	×	×	×
	Unknown	△	△	△	△	△	△	△	△	△	△	△	△	△	△
Honeycomb, cavity	Faulty work (faulty compaction, curing)	△	×	◎	△	×	×	×	×	×	×	×	×	×	×
Separation, exposure and corrosion of rebar and prestressing steel	Insufficient concrete cover	○	×	×	○	○	×	×	×	◎	×	×	×	×	×
	Crack development	○	○	○	○	○	×	○	○	○	○	○	○	○	○
	Defective concrete quality, insufficient strength	×	×	×	×	×	×	×	×	△	○	○	○	○	○
	Carbonation, freezing, salt damage	○	○	○	○	○	×	○	○	○	○	○	×	×	×
	Unknown	△	△	△	△	△	△	△	△	△	△	△	△	△	△
Abnormal deflection	Insufficient rigidity	×	×	×	×	×	×	×	×	○	△	○	○	○	○
	Increased dead load (overlay, added materials)	×	×	×	×	×	×	×	×	○	△	○	○	○	○
	Fatigue of prestressing steel	×	×	×	×	×	×	×	×	○	△	○	○	○	○
	Unknown	×	×	×	×	×	×	×	×	△	△	△	△	△	△

◎: Method with many successful results; considered to ensure effectiveness.
○: Method considered to be effective
△: Method with successful results, whose employment requires review beforehand
×: Method considered not effective

3-2-2 Maintenance Costs

The content and costs of maintenance expected in the ten years after completion of the project are as shown in Table 3.2.2. The bridge maintenance and repair costs will account for only a small portion of MORPW's limited budget. However, thorough maintenance is essential to extend the bridge life.

Table 3.2.2 Content and costs of maintenance

Period	Content	Cost（1000Ksh）
Yearly	① Cleaning of drain basin and pipes ② Cleaning shoes	271 110
	Sub total	381
Every five years	① Bridge body repair ② Expansion joint repair ③ Railings repair	998 101 858
	Subtotal	1,957
Costs for ten-year period		7,724

The expenses of the maintenance are estimated to be as follows:

7,724,000Ksh / 10 years = 772,400 Ksh / year

The above expenses (772,400 Ksh / year) will be about 19% of the maintenance costs for the 4 bridges in Coast province (4,000,000Ksh/year), and that Kenya Government will govern the expenses.

CHAPTER 4

Chapter 4 Project Evaluation and Recommendation

4-1 Project Effect

National Route B8 on which the bridge covered by this project exists connects Kenya's second largest city of Mombasa, the largest trade port in East Africa, with the Principal city of Malindi. All of development plans including the Kenya's largest tourism development, agricultural development of the Tana River to the north, etc. become feasible only when stable functions of this road are ensured.

The object of this project is to rehabilitate the New Nyali and New Mtwapa Bridges located at principal points along National Route B8, which urgently require rehabilitation due to damage caused by deteriorated concrete, etc. The project can therefore be expected to prevent physical and personal damage due to a bridge falling and avoid stagnation of administrative functions caused by traffic blockage.

Effects of this project are summarized in the table below.

Present state and problem factors	Project measures	Expected effects and degree of improvement
① Bridges concerned are in a condition requiring rehabilitation and reinforcement due to deterioration of concrete.	① As regards concrete crack, surface treatment and injection methods are used for repair. As cracks occurring in the main girder of Mtwapa Bridge can be attributed mostly to the effects of shear force, the steel plate adhesion method is used together with external cable.	① Safe and smooth traffic flow for about 38,000 vehicles/day and about 500,000 local residents are assured. At the same time, the project will contribute to economic development of the Coast region (about 2 million people). By performing rehabilitation at present, greater damage in the future can be prevented.
② In the present situation, it is technically impossible for Kenya to undertake large-scale bridge rehabilitation.	②In the course of implementation of this project, a seminar and field tour are held to observe the actual rehabilitation and reinforcement work.	② Technology transfer to bridge engineers of Kenya can be made.

The population benefited by this project is about 500,000 in the Mombasa District alone and amounts to about 2 million in the entire Coast Province.

It was judged that project implementation under the grant aid is acceptable, as the result of the investigation of Kenya's budget management capacity, and the engineering performance and the operation capacity of the relevant department.

4-2 Recommendation

As described above, implementation of the project is expected to offer substantial benefits and contribute widely to the improvement of residents' BHN. Accordingly, it is confirmed that implementation of the project under grant aid is acceptable.

At present, the Kenya Government has set aside a maintenance and management budget for bridges covered by this project. This budget was once accommodated for the emergency damage relief plan to cover expenses in the disaster caused by El Nino in 1998. Therefore, an important task will be further improvement of the comprehensive maintenance system and securing and application of appropriate budgeting.

Appendix

Appendix 1 Member List of the Survey Team

1-1. For the study

Mr. Atsumu IWAI	Leader	Grant Aid Project Management Department, JICA
Mr. Gohei TOKUNAGA	Technical Advisor	Honshu Shikoku Bridge Authority
Mr. Keigo KONNO	Chief Consultant / Rehabilitation Planner	Oriental Consultants Co., Ltd.
Mr. Nobuhiro KUBOYA	Bridge Designer I (Rehabilitation Designer)	Oriental Consultants Co., Ltd.
Mr. Akiyoshi TOGIYA	Bridge Designer II (Surveyor of Damaged Bridge)	Oriental Consultants Co., Ltd.
Mr. Jiro KOJIMA	Construction Planner / Cost Estimator	Oriental Consultants Co., Ltd.

1-2. For Explanation Draft Final Report

Mr.Akira NAKAMURA	Leader	Grant Aid Project Management Department, JICA
Mr. Keigo KONNO	Chief Consultant / Rehabilitation Planner	Oriental Consultants Co., Ltd.
Mr. Nobuhiro KUBOYA	Bridge Designer I (Rehabilitation Designer)	Oriental Consultants Co., Ltd.

Appendix 2 Survey Schedule

2-1. Schedule for the Survey

No				Gohei TOKUNAGA	Atsumu IWAI	Keigo KONNO	Nobuhiro KUBOYA	Akiyoshi TOGIYA	Jiro KOJIMA	Remark
1	Jan	23	Sun	12:40 Narita (BA008) → 16:35 London(H)、22:25 London(G) (BA2069) →						
2		24	Mon	→ 10:00 Nairobi 13:30 EOJ 14:30 JICA 15:30 MORPW Courtesy Call					12:40 Narita (BA008) →16:35 London(H) 22:25 London(G) (BA2070) →	
3		25	Tue	Discussion of IC/R、Minutes of Discussion					→ 10:01 Nairobi	
4		26	Wed	Minutes of Discussion					9:00Nairobi(KQ604) →10:01Mombasa	
5		27	Thu	10:30Nairobi(KQ490)→11:30Mombasa、PWO、 Site Survey (New Nyali Bri.)					Site Survey (New Nyali)	
6		28	Fri	Site survey (New Mtwapa Bri.), Route No. B8, Kilifi Bri.					Ditto	
7		29	Sat	Data Arrangement					Ditto	
8		30	Sun	15:00Mombasa(KQ609) → 16:00Nairobi、Data Arangement				12:40 Narita(BA008) →16:35 London(H),22:25	Ditto	
9		31	Mon	Minutes of Discussion				→ 10:00 Nairobi	Site Survey (New Mtwapa)	
10	Feb	1	Tue	Minutes of Discussion				9:00 Nairobi(KQ604) →10:00 Mombasa	Ditto	
11		2	Wed	Signing of Minutes of Meeting				Site survey (New Mtwapa)	Ditto	
12		3	Thu	Report to EOJ and JICA 23:00 Nairobi(BA2068) →		Report to EOJ and JICA		Ditto	Ditto	
13		4	Fri	04:55 London(G)、12:00 London(H)(BA00 5)→		Data Arrangement		Site Survey (New Nyali)		
14		5	Sat	→ 08:40 Narita		9:00Nairobi(KQ604)→10:00Mombasa		Ditto		
15		6	Sun			Team Meeting				
16		7	Mon			Preperation on Viblation Test		Ditto		
17		8	Tue			Viblation Test		Ditto		
18		9	Wed			Ditto		Site Survey (New Mtwapa)		
19		10	Thu			Site Survey (New Nyali)			Ditto	
20		11	Fri			Ditto			Ditto	Traffic Survey
21		12	Sat			Ditto			Ditto	Ditto
22		13	Sun			Team Meeting				Ditto
23		14	Mon			Site Survey (New Mtwapa)				Ditto
24		15	Tue			Ditto				Ditto
25		16	Wed			Ditto				Ditto
26		17	Thu			Ditto				Ditto
27		18	Fri			Ditto				
28		19	Sat			Ditto				
29		20	Sun			15:00Mombasa(KQ609) → 16:00Nairobi		Data Arrangement	15:00Mombasa(KQ609) → 16:00 Nairobi	
30		21	Mon			Explanation to MORPW on Site Survey Result		Ditto	Explanation to MORPW on site Survey Result	
31		22	Tue			9:00Nairobi(KQ604)→10:00 Mombasa		Ditto	9:00 Nairobi(KQ604) →10:00 Mombasa	
32		23	Wed			Site Survey				
33		24	Thu			Ditto				
34		25	Fri			Ditto				
35		26	Sat			Ditto				
36		27	Sun			Team Meeting				
37		28	Mon			15:00 Mombasa(KQ609) → 16:00 Nairobi				
38		29	Tue			Explanation to MORPW and JICA on Site Survey Result				
39	Mar	1	Wed			Data Arrangement		23:00 Nairobi(BA2068) →		
40		2	Thu			Ditto		04:55 london(G)、14:10 London(H)(BA007)→		
41		3	Fri			Ditto		→ 11:00 Nairobi		
42		4	Sat			Explanation to MORPW on Data Arrangement				
43		5	Sun			23:00 Nairobi(BA2068) →				
44		6	Mon			04:55 London(G)、14:10London(H)(BA007)→				
45		7	Tue			→ 11:00 Narita				

2-2. Schedule for Explanation of Draft Report

<table>
<tr><th colspan="4">No</th><th>Akira NAKAMURA</th><th>Keigo KONNO</th><th>Nobuhiro KUBOYA</th><th>Remarks</th></tr>
<tr><td>1</td><td>May</td><td>21</td><td>Sun</td><td colspan="3">13:05 Narita (BA008) → 17:35 London(H)、
22:15 London(G) （BA2069）→</td><td></td></tr>
<tr><td>2</td><td></td><td>22</td><td>Mon</td><td colspan="3">→ 8:40 Nairobi
14:00 JICA、15:00 EOJ Coourtesy Call</td><td></td></tr>
<tr><td>3</td><td></td><td>23</td><td>Tue</td><td colspan="3">9:00 MORPW Courtesy Call, Explanation/Discussion on DB/D</td><td></td></tr>
<tr><td>4</td><td></td><td>24</td><td>Wed</td><td colspan="3">9:00 MORPW Explanation/Discussion on DB/D, Discussion of Minutes
17:00Nairobi(KQ612)→18:00Mombasa</td><td></td></tr>
<tr><td>5</td><td></td><td>25</td><td>Thu</td><td colspan="3">8:00 Courtesy Call to Provincial Ccommissioner/PWO
9:30 Site Survey of New Nyali and New Mtwapa Bridges
15:00 Explanation/Discussion on DB/D</td><td></td></tr>
<tr><td>6</td><td></td><td>26</td><td>Fri</td><td colspan="3">Inspection on Route No.8/ Kilifi and Sabaki Bridges</td><td></td></tr>
<tr><td>7</td><td></td><td>27</td><td>Sat</td><td colspan="3">13:00 Mombasa(KQ609)→14:00 Nairobi
Data Arrangement</td><td></td></tr>
<tr><td>8</td><td></td><td>28</td><td>Sun</td><td colspan="3">Data Arrangement</td><td></td></tr>
<tr><td>9</td><td></td><td>29</td><td>Mon</td><td colspan="3">9:00 Discussion of Minutes with MORPW</td><td></td></tr>
<tr><td>10</td><td></td><td>30</td><td>Tue</td><td colspan="3">10:00 Signning of Mimutes of Meeting、
11:00 Report to JICA、14:00 report to EOJ、15:30 Report to JBIC、
22:25Nairobi(BA2068)→</td><td></td></tr>
<tr><td>11</td><td></td><td>31</td><td>Wed</td><td colspan="3">05:15 London(G)、
15:45 London(H)(BA007)→</td><td></td></tr>
<tr><td>12</td><td>Jun</td><td>1</td><td>Thu</td><td colspan="3">→ 11:30 Narita</td><td></td></tr>
</table>

Appendix 3 List of Party Concerned in the Recipient Country

◆Ministry of Roads and Public Works

1.	Eng. E.K. Mwongera	Permanent Secretary
2.	Eng. J.H.G. Wambura	Engineer in Chief
3.	Eng. M.O. Kidenda	Chief Engineer Roads
4.	Eng. M.O.A. Bajaber	Chief Superintending Engineer (Bridge)
5.	Eng. P. Wakori	Chief Superintending Engineer (Planning)
6.	Eng. J.N. Nkadayo	Chief Superintending Engineer (T/A)
7.	Eng. H. Kiragu	Superintending Engineer (Bridge)
8.	Dr. Eng. F.N. Nyangaga	Senior Superintending Engineer (Bridge)
9.	Eng. D.W. Mugambi	Chief Engineer Material
10.	Eng. J.Z. Ruwa	Former Provincial Works officer Coast
11.	Eng. W. Nambafu	Present Provincial Works officer Coast
12.	Mr. M. Ontomwa	Provincial Roads Engineering Coast
13.	Mr. E. Kabue	Provincial Bridge Officer

◆Ministry of Finance

1.C.T. Gituai   Deputy Director

◆Embassy of Japan

1. Yosuke Matsumiya   Second Secretary

◆JICA Office

1.	Eigi Hashimoto	Resident Representative
2.	Atsushi Matsumoto	Deputy Resident Representative
3.	Mitsuo Yoshitoku	Assistant Resident Representative
4.	Mr. J.M. Sabai	Programme Officer

◆JBIC

1.	Shunei Shinohara	Former Chief Representative
2.	Katsunori Sawai	Present Chief Representative
3.	Akihiro Hanazawa	Senior Representative

◆Gauff Ingenieure

1.	Mr. J. Pfeffer	General Manager in Kenya
2.	Mr. B.Streit	Resident Engineer Rufji Bridge Project

Appendix 4 Minutes of Discussion

4-1. Discussions on 2 Feb 2000.

MINUTES OF DISCUSSIONS
ON THE BASIC DESIGN STUDY
ON THE PROJECT FOR IMPROVEMENT
OF THE NEW NYALI AND NEW MTWAPA BRIDGES
IN REPUBLIC OF KENYA

Based on the results of the Preparatory Study, the Government of Japan decided to conduct a Basic Design Study on the project for improvement of the New Nyali and New Mtwapa Bridges (hereinafter referred to as "the Project") and entrusted the study to the Japan International Cooperation Agency (hereinafter referred to as "JICA").

JICA sent to Republic of Kenya (hereinafter referred to as " Kenya") the Basic Design Study Team (hereinafter referred to as "the Team"), which is headed by Mr. Atsumu Iwai, Third Project Management Division, Grant Aid Management Department, JICA and is scheduled to stay in the country from 24 January 2000 to 3 February 2000.

The Team held discussions with the officials concerned of the Government of Kenya and conducted a field survey at the study area.

In the course of discussions and field survey, both parties confirmed the main items described on the attached sheets. The Team will proceed for further works and prepare the Basic Design Study Report.

Nairobi, 2 February 2000

Mr. Atsumu Iwai
Leader
Basic Design Study Team
Japan International Cooperation Agency
Japan

Eng. Erastus K. Mwongera, EBS,OGW
Permanent Secretary
Ministry of Roads and Public Works
Republic of Kenya

ATTACHMENT

1.Objective of the Project

The objective of the Project is to secure the smooth and safety traffic on the New Nyali and New Mtwapa Bridges by rehabilitating their damaged parts.

2.Project sites

The sites of the Project are shown in ANNEX- I .

3.Responsible and Implementing Agency

3-1.The Responsible Agency is Ministry of Roads and Public Works (MORPW).
3-2.The Implementing Agency is Roads Department, MORPW.

4.Items requested by the Government of Kenya

The items requested by the Government of Kenya are as follows;
- To improve the New Nyali Bridge
- To improve the New Mtwapa Bridge

5.Japan's Grant Aid Scheme

5-1.Kenyan side understands the Japan's Grant Aid Scheme explained by the Team , as described in ANNEX 2.
5-2.Kenyan side will take the necessary measures, as described in Annex-3, for smooth implementation of the Project, as a condition for the Japanese Grant Aid to be implemented.

6.Schedule of the Study

6-1. The consultants will proceed to further studies in Kenya until 5 March 2000.

6-2. JICA will prepare the draft report in English and dispatch a mission in order to explain its contents around May 2000.

6-3. In case that the contents of the report is accepted in principle by the Government of Kenya, JICA will complete the final report and send it to the Government of Kenya by end of August 2000.

7. Safety Measure

The Kenyan side will take all possible measures to secure the safety of the Team during the field survey.

8. Other relevant issues

8-1. Kenyan side understood the followings;

a) The objective of the Project is to rehabilitate the damaged parts of the Bridges, and the reconstruction of the Bridges is out of the objective of the Project.

b)The necessity of the rehabilitation of the damaged parts of the Bridges under the Grant Aid scheme shall be appraised through further studies.

c) The damaged parts due to the failure of the original design and/or failure of the original execution of the work shall be excluded from the Project in principle.

8-2. In order to minimize the maintenance cost and to maximize the life span of the Bridges, the Team requested that the load capacity control on the Mombasa - Malindi Road (B8) shall be strenghtened more. Kenyan side assured to arrange one mobile axle unit exclusively for the B8 in the near future to strengthen the load capacity control.

8-3. The Team observed a certain improvement on the road and bridge maintenance system such as the implementation of the "Highway Management System". The Team requested that proper budgets for maintenance and routine inspection for the Bridges (including Kilifi and Sabaki bridges) shall be allocated and disbursed each fiscal year in order to expand their life span.

A. Iwai

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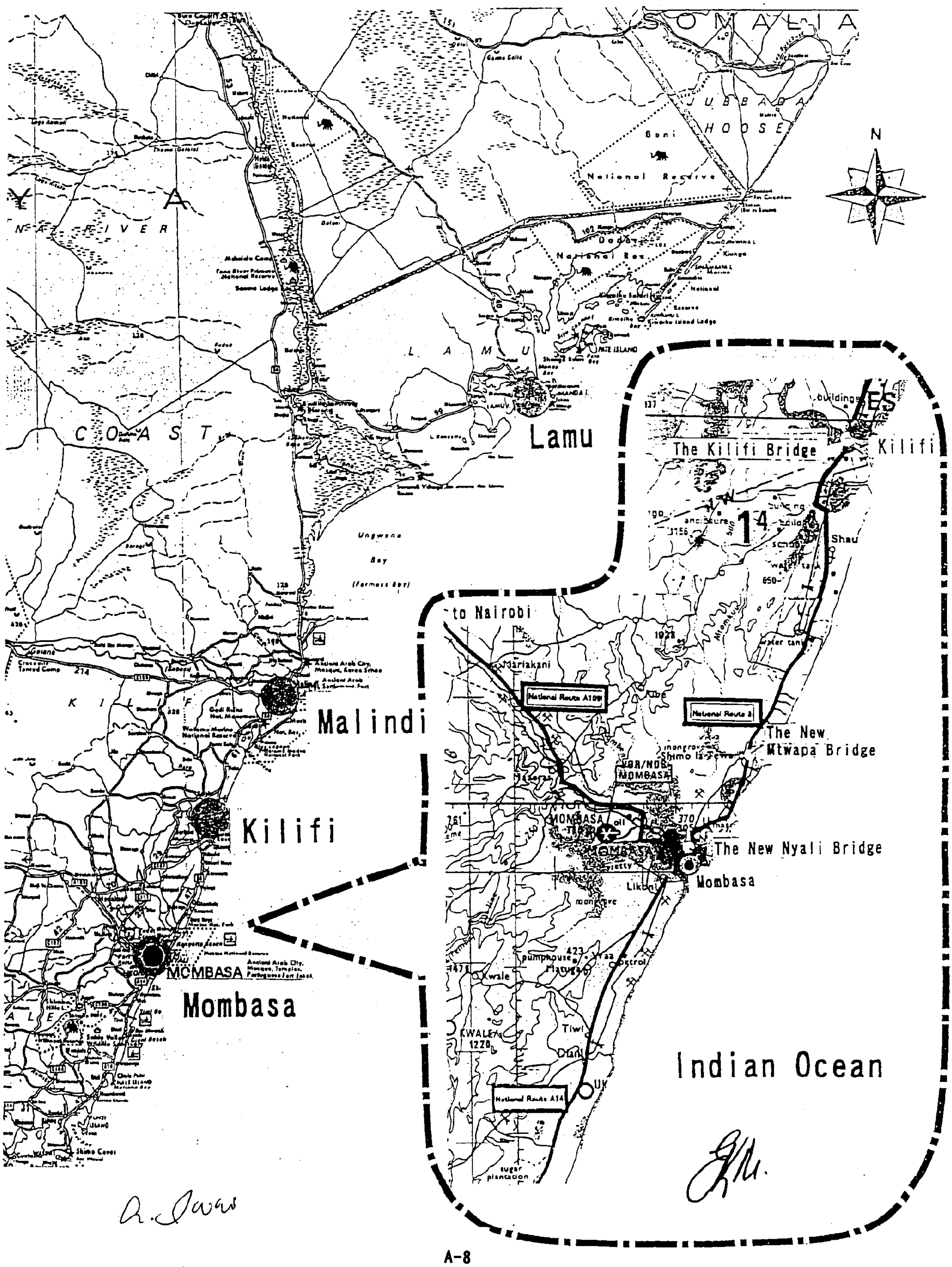
The Sites of the Project

Annex-1



JAPAN'S GRANT AID SCHEME

1. Grant Aid Procedures

1) Japan's Grant Aid Program is executed through the following procedures.

- Application	(Request made by a recipient country)
- Study	(Basic Design Study conducted by JICA)
- Appraisal &Approval	(Appraisal by the Government of Japan and Approval by Cabinet)
- Determination of Implementation	(The Notes to be exchanged between the Governments of Japan and the recipient country)

2) Firstly, the application or request for a Grant Aid project submitted by a recipient country is examined by the Government of Japan (the Ministry of Foreign Affairs) to determine whether or not it is eligible for Grant Aid. If the request is deemed appropriate, the Government of Japan assigns JICA (Japan International Cooperation Agency) to conduct a study on the request.

Secondly, JICA conducts the study (Basic Design Study), using (a) Japanese consulting firm(s).

Thirdly, the Government of Japan appraises the project to see whether or not it is suitable for Japan's Grant Aid Program, based on the Basic Design Study report prepared by JICA, and the results are then submitted to the Cabinet for approval.

Fourthly, the project, once approved by the Cabinet, becomes official with the Exchange of Notes signed by the Governments of Japan and the recipient country.

Finally, for the implementation of the project, JICA assists the recipient country in such matters as preparing tenders, contracts and so on.

2. Basic Design Study

1) Contents of the study

The aim of the Basic Design Study (hereafter referred to as "the Study"), conducted by JICA on a requested project (hereafter referred to as "the Project") is to provide a basic document necessary for the appraisal of the Project by the Government of Japan. The contents of the Study are as follows :

a) Confirmation of the background, objectives, and benefits of the Project and also institutional capacity of agencies concerned of the recipient country necessary for the Project's implementation.

b) Evaluation of the appropriateness of the Project to be implemented under the Grant Aid Scheme from a technical, social and economic point of view.

c) Confirmation of items agreed on by both parties concerning the basic concept of the Project.

d) Preparation of a basic design of the Project.

e) Estimation of costs of the Project.

The contents of the original request are not necessarily approved in their initial form as the

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contents of the Grant Aid project. The Basic Design of the Project is confirmed considering the guidelines of the Japan's Grant Aid Scheme.

The Government of Japan requests the Government of the recipient country to take whatever measures are necessary to ensure its self-reliance in the implementation of the Project. Such measures must be guaranteed even though they may fall outside of the jurisdiction of the organization in the recipient country actually implementing the Project. Therefore, the implementation of the Project is confirmed by all relevant organizations of the recipient country through the Minutes of Discussions.

2) Selection of Consultants

For smooth implementation of the Study, JICA uses (a) registered consulting firm(s). JICA selects (a) firm(s) based on proposals submitted by interested firms. The firm(s) selected carry(ies) out a Basic Design Study and write(s) a report, based upon terms of reference set by JICA. The consulting firm(s) used for the Study is(are) recommended by JICA to the recipient country to also work on the Project's implementation after the Exchange of Notes, in order to maintain technical consistency.

3. Japan's Grant Aid Scheme

1) What is Grant Aid?

The Grant Aid Program provides a recipient country with non-reimbursable funds to procure the facilities, equipment and services (engineering services and transportation of the products, etc.) for economic and social development of the country under principles in accordance with the relevant laws and regulations of Japan. Grant Aid is not supplied through the donation of materials as such.

2) Exchange of Notes (E/N)

Japan's Grant Aid is extended in accordance with the Notes exchanged by the two Governments concerned, in which the objectives of the Project, period of execution, conditions and amount of the Grant Aid, etc., are confirmed.

3) "The period of the Grant Aid" means the one fiscal year which the Cabinet approves the Project for. Within the fiscal year, all procedures such as exchanging of the Notes, concluding contracts with (a) consulting firm(s) and (a) contractor(s) and final payment to them must be completed. However, in case of delays in delivery, installation or construction due to unforeseen factors such as weather, the period of the Grant Aid can be further extended for a maximum of one fiscal year at most by mutual agreement between the two Governments.

4) Under the Grant Aid, in principle, Japanese products and services including transport or those of the recipient country are to be purchased.

When the two Governments deem it necessary, the Grant Aid may be used for the purchase of the products or services of a third country.

However, the prime contractors, namely, consulting constructing and procurement firms, are limited to "Japanese nationals". (The term "Japanese nationals" means persons of Japanese nationality or Japanese corporations controlled by persons of Japanese nationality.)

5) Necessity of "Verification"

The Government of recipient country or its designated authority will conclude contracts

A. Iwai

denominated in Japanese yen with Japanese nationals. Those contracts shall be verified by the Government of Japan. This "Verification" is deemed necessary to secure accountability to Japanese taxpayers.

6) Undertakings required of the Government of the Recipient Country

In the implementation of the Grant Aid Project, the recipient country is required to undertake such necessary measures as the following:

a) To secure land necessary for the sites of the Project and to clear, level and reclaim the land proior to commencement of the construction.

b) To provide facilities for the distribution of electricity, water supply and drainage and other incidental facilities in and around the sites.

c) To ensure all the expenses and prompt excursion for unloading, customs clearance at the port of disembarkation and internal transportation of the products purchased under the Grant Aid.

d) To exempt Japanese nationals from customs duties, internal taxes and other fiscal levies which will be imposed in the recipient country with respect to the supply of the products and services under the Verified Contracts.

e) To accord Japanese nationals whose services may be required in connection with the supply of the products and services under the Verified contracts, such facilities as may be necessary for their entry into the recipient country and stay therein for the performance of their work.

7) "Proper Use"

The recipient country is required to maintain and use the facilities constructed and the equipment purchased under the Grant Aid properly and effectively and to assign staff necessary for this operation and maintenance as well as to bear all the expenses other than those covered by the Grant Aid.

8) "Re-export"

The products purchased under the Grant Aid should not be re-exported from the recipient country.

9) Banking Arrangements (B/A)

a) The Government of the recipient country or its designated authority should open an account in the name of the Government of the recipient country in a bank in Japan (hereinafter referred to as "the Bank"). The Government of Japan will execute the Grant Aid by making payments in Japanese yen to cover the obligations incurred by the Government of the recipient country or its designated authority under the Verified Contracts.

b) The payments will be made when payment requests are presented by the Bank to the Government of Japan under an Authorization to Pay (A/P) issued by the Government of the recipient country or its designated authority.

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Major Undertakings to be taken by Each Government

NO	Items	To be covered by Grant Aid	To be covered by Recipient side
1	To secure land		●
2	To clear, level and reclaim the site when needed		●
3	To bear the following commissions to a bank of Japan for the banking services based upon the B/A		
	1) Advising commission of A/P		●
	2) Payment commission		●
4	To ensure prompt unloading and customs clearance at the port of disembarkation in recipient country		
	1) Marine(Air) transportation of the products from Japan to the recipient country	●	
	2) Tax exemption and customs clearance of the products at the port of disembarkation		●
5	To accord Japanese nationals whose services may be required in connection with the supply of the products and the services under the verified contract such facilities as may be necessary for their entry into the recipient country and stay therein for the performance of their work		●
6	To exempt Japanese nationals from customs duties, internal taxes and other fiscal levies which may be imposed in the recipient country with respect to the supply of the products and services under the verified contract		●
7	To maintain and use properly and effectively the facilities constructed and equipment provided under the Grant Aid		●
8	To bear all the expenses, other than those to be borne by the Grant Aid, necessary for construction of the facilities		●
9	To Coordinate and solve any issues related to the Project which may be raised from third parties or inhabitants around the Project area		●

A. Iwai

4-2. Discussions on 30 May 2000.

MINUTES OF DISCUSSIONS
ON THE BASIC DESIGN STUDY
ON THE PROJECT FOR IMPROVEMENT
OF THE NEW NYALI AND NEW MTWAPA BRIDGES
IN REPUBLIC OF KENYA
(EXPLANATION ON DRAFT REPORT)

The Japan International Cooperation Agency (hereinafter referred to as "JICA") dispatched a Basic Design Study Team on the Project for Improvement of the New Nyali and New Mtwapa Bridges (hereinafter referred to as "the Project") to Rebublic of Kenya (hereinafter referred to as "Kenya") from January 24 to March 5, 2000, and through the discussion with people concerned of the Project, field survey, and technical examination of the results in Japan, JICA prepared a draft report of the study.

In order to explain and discuss the components of the draft report, JICA sent the Basic Design Study Team (hereinafter referred to as "the Team") headed by Mr. Akira Nakamura, Deputy Director, Project Monitoring and Coordination Division, Grant Aid Management Department of JICA to Kenya from May 22 to May 30, 2000.

As a result of discussions, both parties confirmed the main items described in the attached sheets.

Nairobi, May 30, 2000

Mr. Akira Nakamura	Eng. Erastus K. Mwongera, EBS, OGW
Leader	Permanent Secretary
Basic Design Study Team	Ministry of Roads and Public Works
Japan International Cooperation Agency	Republic of Kenya
Japan	

ATTACHMENT

1.Components of the Draft Report

The Kenyan side agreed and accepted in principle the components of the draft report as explained by the Team.

2.Japan's Grant Aid scheme

The Kenyan side has understood the Japan's Grant Aid Scheme and the necessary measures to be taken by the Government of Kenya as explained by the Team and described in Annex-2 and Annex-3 of the Minutes of Discussions signed by both parties on February 2, 2000.

3.Schedule of the Study

JICA will complete the final report in accordance with the confirmed items and send it to the Government of Kenya by August 2000.

4.Other relevant issues

4-1. Ministry of Roads and Public Works (hereinafter referred to as "MORPW") requested the Team to provide them with the technical guidance for the methods introduced for rehabilitation and improvement works of the two bridges. The Kenyan side will take advantage of experience gained in the Project for future maintenance works of bridges.

4-2. The Team has confirmed that MORPW has secured the necessary areas for the construction yard near the bridges.

4-3. The Team has confirmed that there is no need of relocating the existing power line and telephone line.

4-4. The Team has confirmed that a mobile axle load unit has been deployed on the Mombasa -Malindi Road(B8) to restrict overloading vehicles.

4-5. The Team reiterated that the regular inspection and maintenance are indispensable to maintain the function and to maximize the life span of structure.

Inspection Result data

1. Schmitd hammer test

New Nyali Bridge

Structure	Locatikon		Position	Ro	α	△R	R	σC	σCz	fc	Remark
Sub-structure	P3 Pier	P3	West side	54.4	0	0	54.4	523.2	329.6	32.3	
			Mombasa	60.1	0	0	60.1	597.3	376.3	36.9	
			East	54.9	0	0	54.9	529.7	333.7	32.7	
			Malindi	56.4	0	0	56.4	549.2	346.0	33.9	
	P4, P5 Pier	P4	Malindi	58.8	0	0	58.8	580.4	365.7	35.9	
			Mombasa	57.7	0	0	57.7	566.1	356.6	35.0	
		P5	Malindi	45.0	0	0	45.0	401.0	252.6	24.8	
			Mombasa	51.1	0	0	51.1	480.3	302.6	29.7	
	A1Abutment	A1	East	52.8	0	0	52.8	502.4	316.5	31.0	
			Malindi	47.0	0	0	47.0	427.0	269.0	26.4	
			West side	47.0	0	0	47.0	427.0	269.0	26.4	
		A2	West side	48.9	0	0	48.9	451.7	284.6	27.9	
			Mombasa	55.5	0	0	55.5	537.5	338.6	33.2	
			East	47.8	0	0	47.8	437.4	275.6	27.0	
Super-structure	East side	(NO.1)	East	53.0	0	0	53.0	505.0	318.2	31.2	
	Side span		Bottom	33.8	-90	2.9	36.7	293.1	184.7	18.1	
	Malindi side		West side	52.7	0	0	52.7	501.1	315.7	31.0	
			Malindi	43.8	0	0	43.8	385.4	242.8	23.8	
	East side	(NO.4)	East	55.4	0	0	55.4	536.2	337.8	33.1	
	P4 Pier fulcrum		Bottom	38.3	-90	2.8	41.1	350.3	220.7	21.6	
			West side	49.8	0	0	49.8	463.4	291.9	28.6	
			Malindi	52.8	0	0	52.8	502.4	316.5	31.0	
	East side	(NO.5)	East	53.2	0	0	53.2	507.6	319.8	31.4	
	center span		Bottom	35.8	-90	2.9	38.7	319.1	201.0	19.7	
			West side	50.2	0	0	50.2	468.6	295.2	28.9	
			Mombasa	55.4	0	0	55.4	536.2	337.8	33.1	
	East side	(NO.6)	East	51.1	0	0	51.1	480.3	302.6	29.7	
	center span		Bottom	36.5	-90	2.8	39.3	326.9	205.9	20.2	
			West side	45.7	0	0	45.7	410.1	258.4	25.3	
			Malindi	53.2	0	0	53.2	507.6	319.8	31.4	
	East side	(NO.7)	East	51.7	0	0	51.7	488.1	307.5	30.2	
	P5 Pier fulcrum		Bottom	29.4	-90	3.1	32.5	238.5	150.3	14.7	
			West side	46.3	0	0	46.3	417.9	263.3	25.8	
			Mombasa	52.4	0	0	52.4	497.2	313.2	30.7	
	East side	(NO.10)	East	44.1	0	0	44.1	389.3	245.3	24.1	
	Side span		Bottom	37.5	-90	2.8	40.3	339.9	214.1	21.0	
	Mombasa side		West side	45.0	0	0	45.0	401.0	252.6	24.8	
			Mombasa	50.0	0	0	50.0	466.0	293.6	28.8	

Ro = Average Rebound Value
α = Impact Angle (deg.) →0 ↑+90 ↓-90
△R = Adusting Value in α
R = Standard Value
σC = Copmressive Strength at Present (Kg/cm²)
σC = (-184 + 13 R)
σCZ = Copmressive Strength after carcuration Age revision coefficient (Kg/cm²)
σCZ = σC×0.63
fc = Copmressive Strength at Present (N/mm²)
fC = σCZ×9.8/100
Remark = Designed strength

Nyali Bridge

Structure	Locatikon		Position	Ro	α	△R	R	σC	σCz	fc	Remark
Super-structure	West Malindi Side span	(NO.1)	East	46.9	0	0	46.9	425.7	268.2	26.3	
			Bottom	33.4	-90	3.0	36.4	289.2	182.2	17.9	
			West	50.1	0	0	50.1	467.3	294.4	28.9	
			Malindi cross beam	49.4	0	0	49.4	458.2	288.7	28.3	
	West P4 pier fulcrum	(NO.4)	East	53.6	0	0	53.6	512.8	323.1	31.7	
			Bottom	37.4	-90	2.8	40.2	338.6	213.3	20.9	
			West	54.8	0	0	54.8	528.4	332.9	32.6	
			Malindi P4 fulcrum	52.9	0	0	52.9	503.7	317.3	31.1	
	West center span	(NO.5)	East	51.3	0	0	51.3	482.9	304.2	29.8	
			Bottom	23.8	-90	3.3	27.1	168.3	106.0	10.4	
			West	51.6	0	0	51.6	486.8	306.7	30.1	
			Mombasa cross beam	54.9	0	0	54.9	529.7	333.7	32.7	
	West center span	(NO.6)	East	45.8	0	0	45.8	411.4	259.2	25.4	
			Bottom	35.9	-90	2.9	38.8	320.4	201.9	19.8	
			West	51.0	0	0	51.0	479.0	301.8	29.6	
			Malindi cross beam	53.8	0	0	53.8	515.4	324.7	31.8	
	West P5 pier fulcrum	(NO.7)	East	53.1	0	0	53.1	506.3	319.0	31.3	
			Bottom	35.7	-90	2.9	38.6	317.8	200.2	19.6	
			West	51.8	0	0	51.8	489.4	308.3	30.2	
			Malindi P4 fulcrum	53.7	0	0	53.7	514.1	323.9	31.8	
	West Monbasa Side span	(NO.10)	East	51.0	0	0	51.0	479.0	301.8	29.6	
			Bottom	35.2	-90	2.9	38.1	311.3	196.1	19.2	
			West	49.7	0	0	49.7	462.1	291.1	28.5	
			Mombasa cross beam	52.1	0	0	52.1	493.3	310.8	30.5	

Ro = Average Rebound Value
α = Impact Angle (deg.) →0 ↑+90 ↓-90
△R = Adusting Value in α
R = Standard Value
σC = Copmressive Strength at Present (Kg/cm²)
σC = (-184 + 13 R)
σCZ = Copmressive Strength after carcuration Age revision coefficient (Kg/cm²)
σCZ = σC×0.63
fc = Copmressive Strength at Present (N/mm²)
fC = σCZ×9.8/100
Remark = Designed strength

New Mtwapa Bridge

Structure	Location		Position	Ro	α	△R	R	σC	σCz	fc	Remark
Sub-structure	Pier	P1	Malindi	48.4	0	0	48.4	445.2	280.5	27.5	
			Mombasa	52.1	0	0	52.1	493.3	310.8	30.5	
		P2	Malindi	57.9	0	0	57.9	568.7	358.3	35.1	
			Mombasa	54.6	0	0	54.6	525.8	331.3	32.5	
	Abutment	A1	West	49.8	0	0	49.8	463.4	291.9	28.6	
			Mombasa	49.5	0	0	49.5	459.5	289.5	28.4	
			East	46.8	0	0	46.8	424.4	267.4	26.2	
		A2	West	45.0	0	0	45.0	401.0	252.6	24.8	
			Malindi	41.8	0	0	41.8	359.4	226.4	22.2	
			East	44.1	0	0	44.1	389.3	245.3	24.1	
Super-Structure	Malindi Side span	(NO.1)	East web	55.6	0	0	55.6	538.8	339.4	33.3	
			Bottom slab	45.1	-90	2.4	47.5	433.5	273.1	26.8	
			West web	57.1	0	0	57.1	558.3	351.7	34.5	
			Upper slab	57.0	+90	-2.5	54.5	524.5	330.4	32.4	
	Malindi P1 pier fulcrum	(NO.4)	East web	53.0	0	0	53.0	505.0	318.2	31.2	
			Bottom slab	39.9	-90	2.7	42.6	369.8	233.0	22.8	
			West web	51.4	0	0	51.4	484.2	305.0	29.9	
			Malindi cross beam	49.2	0	0	49.2	455.6	287.0	28.1	
	Malindi center span	(NO. 7)	East web	58.2	0	0	58.2	572.6	360.7	35.4	
			Bottom slab	33.6	-90	3.0	36.6	291.8	183.8	18.0	
			West web	56.5	0	0	56.5	550.5	346.8	34.0	
			Upper slab	59.6	+90	-2.3	57.3	560.9	353.4	34.7	
	Mombasa Center span	(NO. 7)	East web	55.6	0	0	55.6	538.8	339.4	33.3	
			Bottom slab	36.9	-90	2.8	39.7	332.1	209.2	20.5	
			West web	55.2	0	0	55.2	533.6	336.2	33.0	
			Upper slab	58.3	+90	-2.4	55.9	542.7	341.9	33.5	
	Mombasa P2 pier fulcrum	(NO.10)	East web	55.2	0	0	55.2	533.6	336.2	33.0	
			Bottom slab	36.7	-90	2.8	39.5	329.5	207.6	20.4	
			West web	51.8	0	0	51.8	489.4	308.3	30.2	
			Mombasa cross beam	51.0	0	0	51.0	479.0	301.8	29.6	
	Mombasa Side span	(NO.13)	East web	53.5	0	0	53.5	511.5	322.2	31.6	
			Bottom slab	37	-90	2.8	39.8	333.4	210.0	20.6	
			West web	55.4	0	0	55.4	536.2	337.8	33.1	
			Upper slab	55	+90	-2.7	52.3	495.9	312.4	30.6	

Ro = Average Rebound Value

α = Impact Angle (deg.) →0 ↑+90 ↓-90

△R = Adusting Value in α

R = Standard Value

σC = Copmressive Strength at Present (Kg/cm²)

σC =(-184 + 13 R)

σCZ = Copmressive Strength after carcuration Age revision coefficient (Kg/cm²)

σCZ σC×0.63

fc = Copmressive Strength at Present (N/mm²)

fC = σCZ×9.8/100

Remark = Designed strength

2. Compressive strength test

Compressive Strength Test

1) Diameter ϕ 100mm
Substructure

No.	Bridge name	Marking	Sampling point
1	New Mtwapa	A1-1	Abutment at Malindi side
2	New Mtwapa	A1-2	Abutment at Malindi side
3	New Mtwapa	P1-1	Pier at Malindi side
4	New Mtwapa	P1-2	Pier at Malindi side
5	New Mtwapa	P2-1	Pier at Monsaba side
6	New Mtwapa	P2-2	Pier at Monsaba side
7	New Mtwapa	A2-1	Abutment at Monbasa side
8	New Mtwapa	A2-2	Abutment at Monbasa side
9	New Nyali	A1-1	Abutment at Malindi side
10	New Nyali	A1-2	Abutment at Malindi side
11	New Nyali	P3-1	Pier at Malindi side
12	New Nyali	P3-2	Pier at Malindi side
13	New Nyali	A2-1	Abutment at Monbasa side
14	New Nyali	A2-2	Abutment at Monbasa side

2) Diameter ϕ 50mm
Superstructure

No	Bridge name	Marking	Sampling point
1	-	SS-E1	-
2	New Mtwapa	SS-E2	Web of girder at Malindi side span
3	New Mtwapa	SP-E1	Wab of girder on Malindi side pier
4	New Mtwapa	SP-E2	Wab of girder on Malindi side pier
5	New Mtwapa	CP-E1	Wab of girder at center span
6	New Mtwapa	CP-E1	Wab of girder at center span
7	New Nyali	SS-E1	Web of girder at Monbasa side span
8	New Nyali	SS-E2	Web of girder at Monbasa side span
9	New Nyali	SP-E1	Wab of girder on Monbasa side pier
10	New Nyali	SP-E2	Wab of girder on Monbasa side pier
11	New Nyali	CP-E1	Wab of girder at center span
12	New Nyali	CP-E2	Wab of girder at center span

Test result on compressive strength and elastic modulus (ϕ 100)

Name of bridge	New Mtwapa Bridge								
Test piece No.	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8	Average
Diameter d(mm)	101.8	101.6	101.6	101.7	101.8	101.3	101.7	101.7	
Height h(mm)	94.4	125.7	95.7	98.5	116.2	98.8	93.5	112.9	
h/d	0.93	1.24	0.94	0.97	1.14	0.98	0.92	1.11	
Weight (kg)	1.73	2.282	1.776	1.830	2.212	1.866	1.700	2.049	
Max. load (kN)	587.4	532.7	450.5	447.1	356.3	485.5	446.3	340.7	
Correction factor	0.879	0.927	0.881	0.886	0.913	0.887	0.877	0.908	
Compressive strength(N/mm2)	63.4	60.9	49.0	48.7	40.0	53.4	48.2	38.1	46.3
1/3 maximum load(kN)	195.8	177.6	150.2	149.0	118.8	161.8	148.8	113.6	
Above stress (N/mm2)	24.06	21.90	18.52	18.35	14.59	20.08	18.31	13.98	
Load of vertical strain 50×E-6 (kN)	16.70	13.70	21.28	6.18	7.26	17.85	5.10	11.87	
Above stress (N/mm2)	2.05	1.69	2.62	0.76	0.89	2.21	0.63	1.46	
Vertical strain of 1/3 maximum load	1.00E-03	9.71E-04	6.76E-04	9.08E-04	7.91E-04	6.68E-04	1.04E-03	7.12E-04	
Elastic modulus	2.20E+04	2.09E+04	2.54E+04	1.95E+04	1.74E+04	2.69E+04	1.71E+04	1.77E+04	1.87E+04

Note 1) To determine the correction factor, the quadratic regression equation (1) concerning the correction factor of h/d=1.00～2.00 was determined and applied to the entire range of h/d.

Regression equation (1) Correction factor C=0.66857+0.27943×h/d-0.057143×(h/d) ^ 2

Note 2) The basis strain gauge length is 30 mm or 10 mm when the test piece height is so small as to make adhesion difficult. The concrete standard specification and standard (pp.443) sets forth that the measurement length of strain gauge must be three times the maximum aggregate dimensions, but 1/2 or less of the test piece height.
As this test does not comply with this standard, the accuracy of elastic modulus is inferior, and the empirical value may be preferred in certain cases.

Note 3) Average was calculated for the test piece with h/d>1.0.

Name of bridge	New Nyali							
Test piece No.	No.9	No.10	No.11	No.12	No.13	No.14	No.15	Average
Diameter d(mm)	101.5	101.4	101.4	101	101.2	101.3	101.8	
Height h (mm)	148.9	172.3	122.9	176.3	88.8	150.1	103.1	
h/d	1.47	1.70	1.21	1.75	0.88	1.48	1.01	
Weight (kg)	2.808	3.235	2.347	3.368	1.681	2.818	1.949	
Max. load (kN)	441.2	409.1	410.5	356.5	529.4	416.1	504.7	
Correction factor	0.956	0.978	0.923	0.982	0.870	0.957	0.893	
Compressive strength(N/mm2)	52.1	49.6	46.9	43.7	57.2	49.4	55.4	49.5
1/3 maximum load(kN)	147.1	136.4	136.8	118.8	176.5	138.7	168.2	
Above stress (N/mm2)	18.18	16.89	16.94	14.83	21.94	17.21	20.67	
Load of vertical strain 50×E-6 (kN)	10.40	14.22	6.96	10.98	4.60	9.61	8.24	
Above stress (N/mm2)	1.29	1.76	0.86	1.37	0.57	1.19	1.01	
Vertical strain of 1/3 maximum load	6.71E-04	8.08E-04	7.72E-04	6.44E-04	8.10E-04	6.63E-04	9.80E-04	
Elastic modulus	2.54E+04	1.88E+04	2.10E+04	2.11E+04	2.65E+04	2.43E+04	2.02E+04	2.18E+04

Test result on compressive strength and elastic modulus (ϕ 50)

Name of bridge	New Mtwapa Bridge						Name of bridge						
Test piece No.	No.2	No.3	No.4	No.5	No.6	Average	No.7	No.8	No.9	No.10	No.11	No.12	Average
Diameter d(mm)	49.9	49.9	50	49.7	49.8		49.8	49.9	49.9	49.7	49.9	49.8	
Height h (mm)	47.2	67.2	55.9	62.2	37.8		75.7	55.2	38.4	46	36.8	47	
h/d	0.95	1.35	1.12	1.25	0.76		1.52	1.11	0.77	0.93	0.74	0.94	
Weight (kg)	0.205	0.300	0.249	0.297	0.162		0.359	0.255	0.171	0.193	0.164	0.212	
Max. load (kN)	89.1	102.8	101.5	139.3	76.5		103.6	146.7	118.2	78.6	103.1	123	
Correction factor	0.882	0.941	0.910	0.929	0.848		0.961	0.908	0.850	0.878	0.844	0.881	
Compressive strength(N/mm2)	40.2	49.5	47.0	66.7	33.3	54.4	51.1	68.1	51.4	35.6	44.5	55.7	59.6
1/3 maximum load(kN)	29.7	34.3	33.8	46.4	25.5		34.5	48.9	39.4	26.2			
Above stress (N/mm2)	15.19	17.52	17.23	23.93	13.09		17.73	25.00	20.15	13.51			
Load of vertical strain 50×E-6 (kN)	0.70	2.60	2.70	2.20	1.30		2.80	4.60	3.90	2.30			
Above stress (N/mm2)	0.36	1.33	1.38	1.13	0.67		1.44	2.35	1.99	1.19			
Vertical strain of 1/3 maximum load	8.77E-04	7.93E-04	7.06E-04	9.56E-04	6.96E-04		6.81E-04	9.86E-04	4.93E-04	8.89E-04			
Elastic modulus	1.70E+04	2.18E+04	2.26E+04	2.40E+04	1.80E+04	2.28E+04	2.41E+04	2.31E+04	3.72E+04	1.39E+04	-	-	2.36E+04

Note 1) To determine the correction factor, the quadratic regression equation (1) concerning the correction factor of h/d=1.00~2.00 was determined and applied to the entire range of h/d.

Regression equation (1) Correction factor C=0.66857+0.27943×h/d-0.057143×(h/d) ^ 2

Note 2) The basis strain gauge length is 30 mm or 10 mm when the test piece height is so small as to make adhesion difficult.
The concrete standard specification and standard (pp.443) sets forth that the measurement length of strain gauge must be three times the maximum aggregate dimensions, but 1/2 or less of the test piece height.
As this test does not comply with this standard, the accuracy of elastic modulus is inferior, and the empirical value may be preferred in certain cases.

Note 3) Average was calculated for the test piece with h/d>1.0.

3. Carbonation thickness test

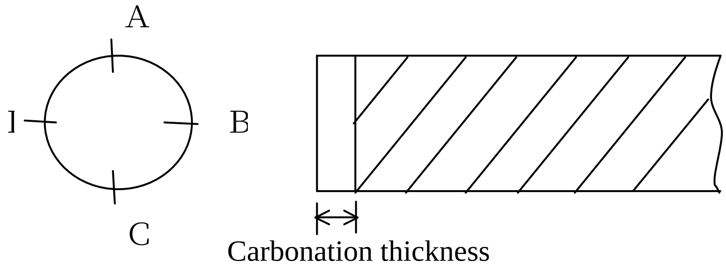
Carbonation thickness test result

New Nyali Bridge

	Core No.	Locatikon	Measuring point (mm)				Average Carbonation Thickness
			A	B	C	D	
Super-structure	N-SP-SS-E Ⅰ	Box girder inside side span	0	0	0	0	0
	N-SP-SS-E Ⅱ	Box girder inside side span	0	0	0	0	0
	N-SP-SP-E Ⅰ	Box girder inside pier fulcrum	7	11	8	9	9
	N-SP-SP-E Ⅱ	Box girder inside pier fulcrum	12	13	9	11	11
	N-SP-CP-E Ⅰ	Box girder inside center span	0	0	0	0	0
	N-SP-CP-E Ⅱ	Box girder inside center span	0	0	0	0	0
	N-SP-CP-E Ⅲ	Box girder inside center span	0	0	0	0	0
Sub-structure	N-SB-A1-N Ⅰ	A1 Abutment	0	0	0	0	0
	N-SB-A1-N Ⅱ	A1 Abutment	0	0	0	0	0
	N-SB-A2-S Ⅰ	A2 Abutment	0	0	0	0	0
	N-SB-A2-S Ⅱ	A2 Abutment	0	0	0	0	0
	N-SB-P3-S Ⅰ	P3 Pier	0	0	0	0	0
	N-SB-P3-S Ⅱ	P3 Pier	0	0	0	0	0

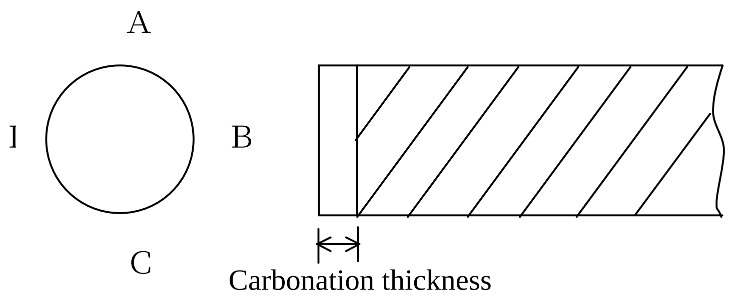
New Mtwapa Bridge

	Core No.	Locatikon	Measuring point (mm)				Average Carbonation Thickness
			A	B	C	D	
Super-structure	M-SP-SS-E Ⅰ	Box girder inside side span	0	0	0	0	0
	M-SP-SS-E Ⅱ	Box girder inside side span	0	0	0	0	0
	M-SP-SP-E Ⅰ	Box girder inside pier fulcrum	12	14	13	10	12
	M-SP-SP-E Ⅱ	Box girder inside pier fulcrum	10	13	12	13	12
	M-SP-CP-E Ⅰ	Box girder inside center span	0	0	0	0	0
	M-SP-CP-E Ⅱ	Box girder inside center span	0	0	0	0	0
Sub-structure	M-SB-A1-S Ⅰ	A1 Abutment	0	0	0	0	0
	M-SB-A1-S Ⅱ	A1 Abutment	0	0	0	0	0
	M-SB-A2-N Ⅰ	A2 Abutment	0	0	0	0	0
	M-SB-A2-N Ⅱ	A2 Abutment	0	0	0	0	0
	M-SB-P1-S Ⅰ	P1 Pier	0	0	0	0	0
	M-SB-P1-S Ⅱ	P1 Pier	0	0	0	0	0
	M-SB-P2-N Ⅰ	P2 Pier	0	0	0	0	0
	M-SB-P2-N Ⅱ	P2 Pier	0	0	0	0	0
	M-SB-P2-N Ⅲ	P2 Pier	0	0	0	0	0



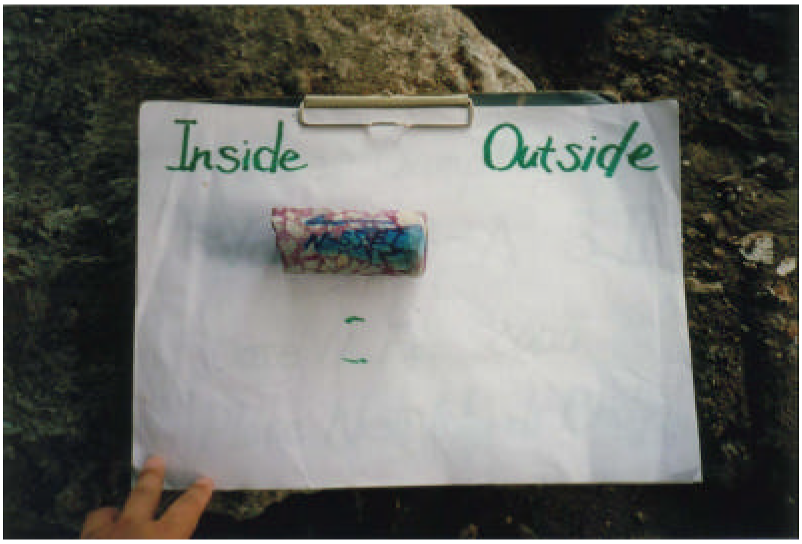
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-SS-E I	Box girder inside side span(Box No.10)



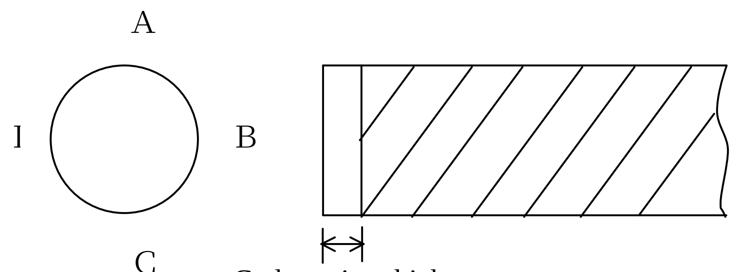
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



Carbonation thickness test result

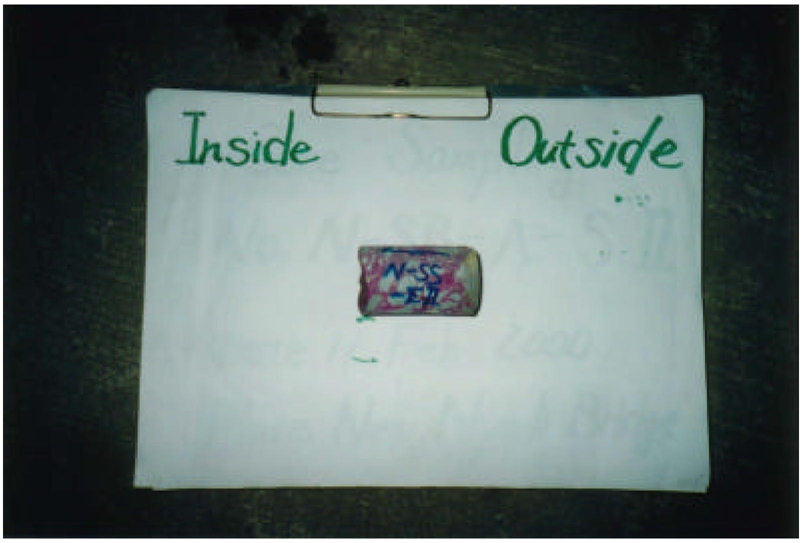
Bridge Name	Core No.	location
New Nyali Bridge	N-SP-SS-EⅡ	Box girder inside side span(Box No.10)



Carbonation thickness

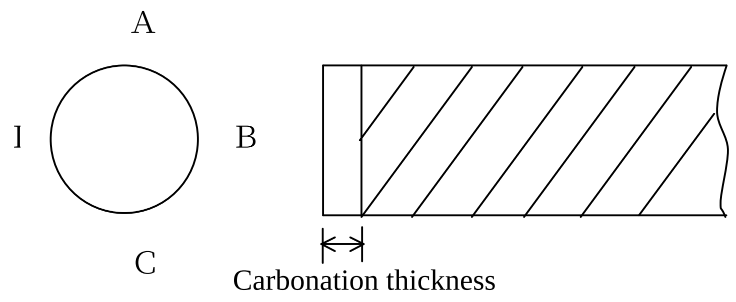
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



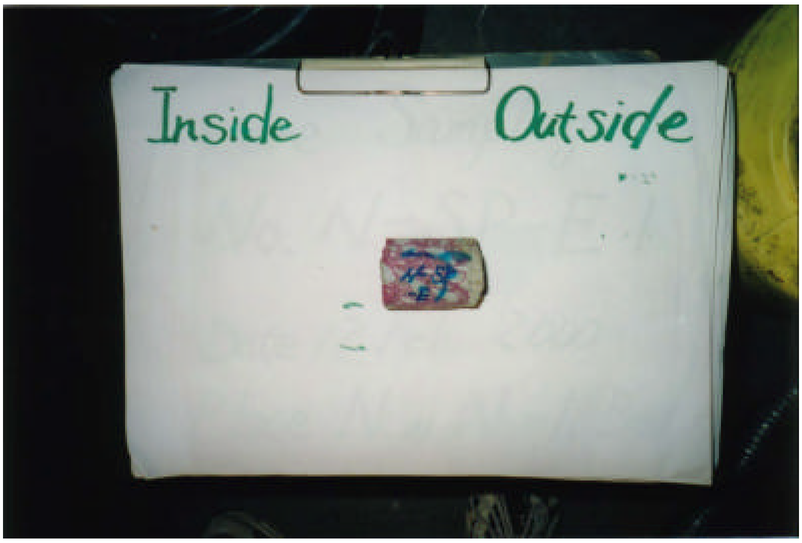
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-SP-E I	Box girder inside pier fulcrum (Box No.7)



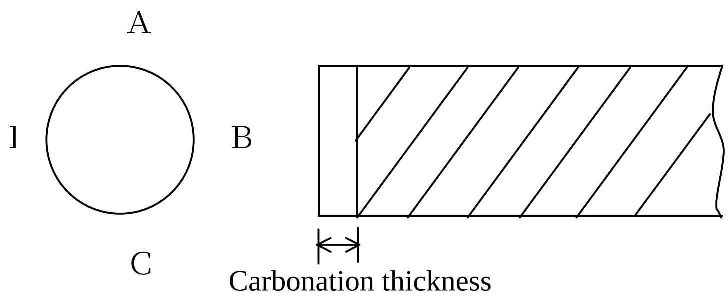
A	B	C	D	Average carbonation thickness
7mm	11mm	8mm	9mm	9mm

Photo of carbonation thickness test condition



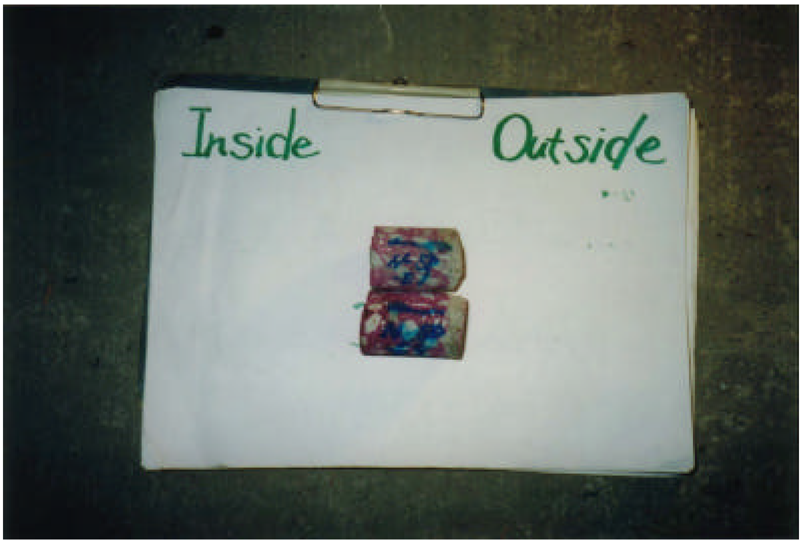
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-SP-EⅡ	Box girder inside pier fulcrum (Box No.7)



A	B	C	D	Average carbonation thickness
12㎜	13㎜	9㎜	11㎜	11㎜

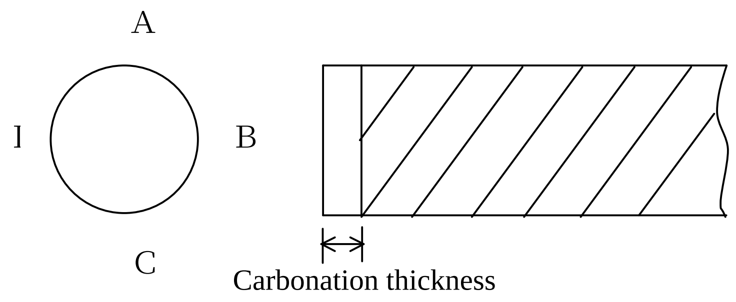
<u>Photo of carbonation thickness test condition</u>



N-SP-SP-EⅡ

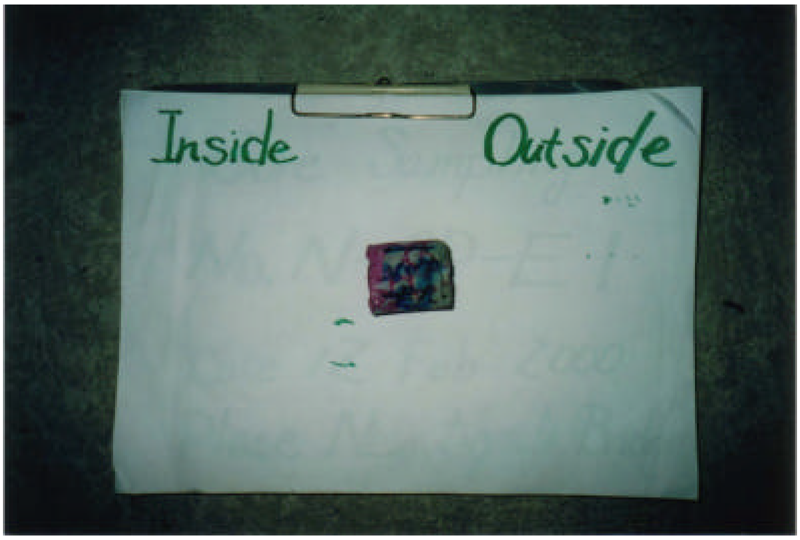
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-CP-E I	Box girder inside center span (Box No.6)



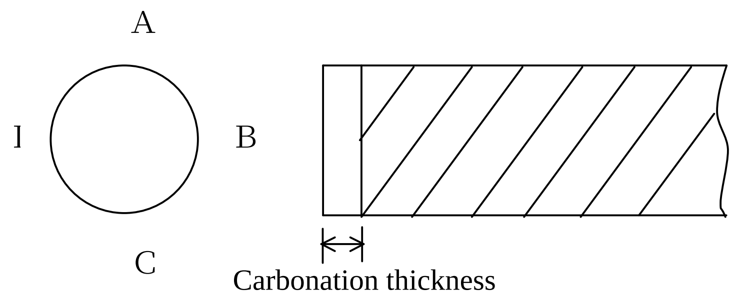
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



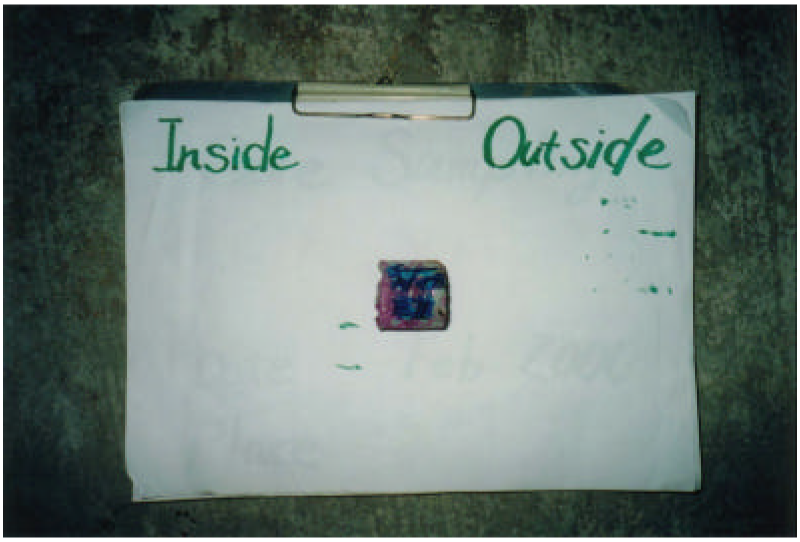
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-CP-EII	Box girder inside center span (Box No.6)



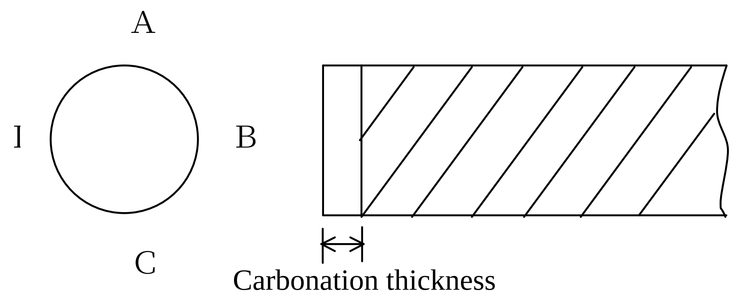
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



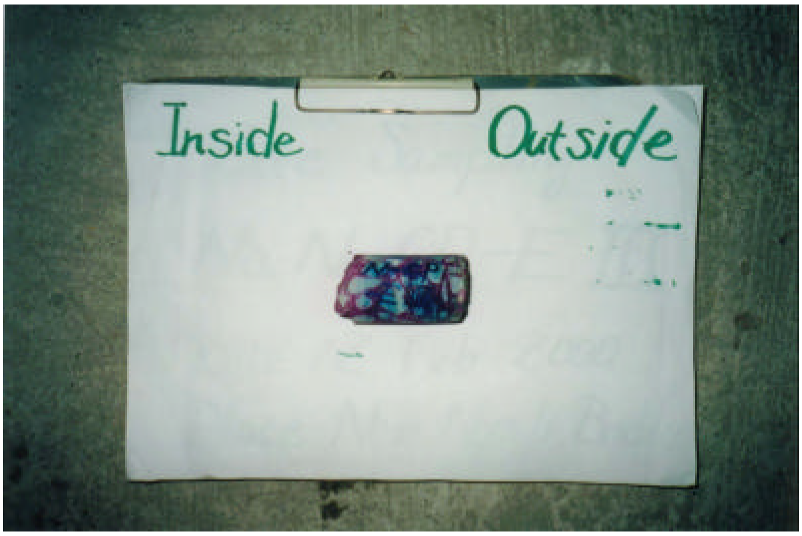
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SP-CP-EIII	Box girder inside center span (Box No.6)



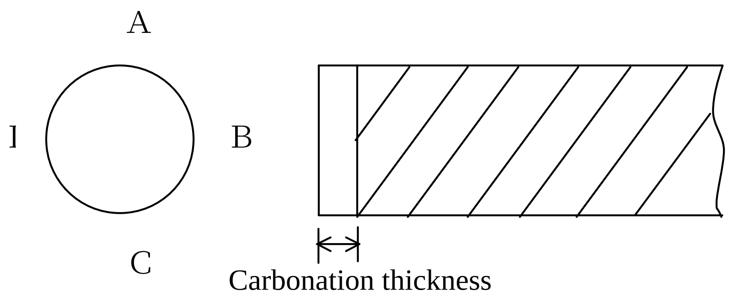
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



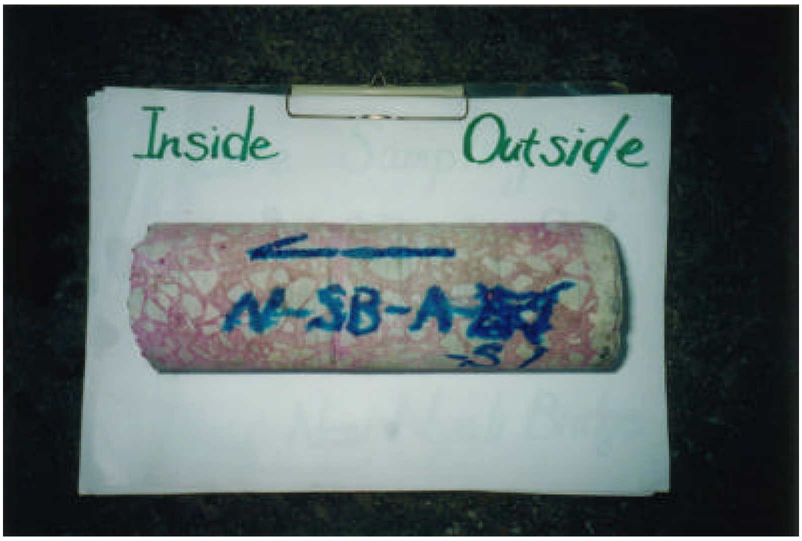
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-A1-N I	A1 Abutment



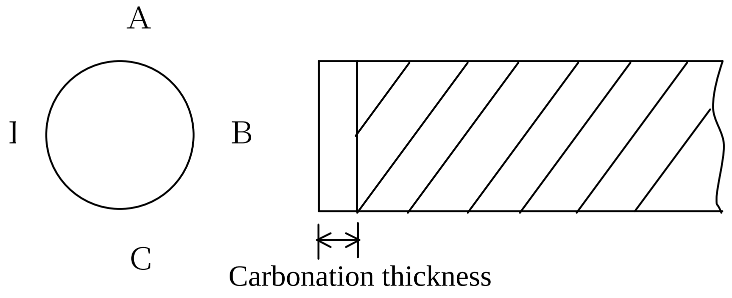
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



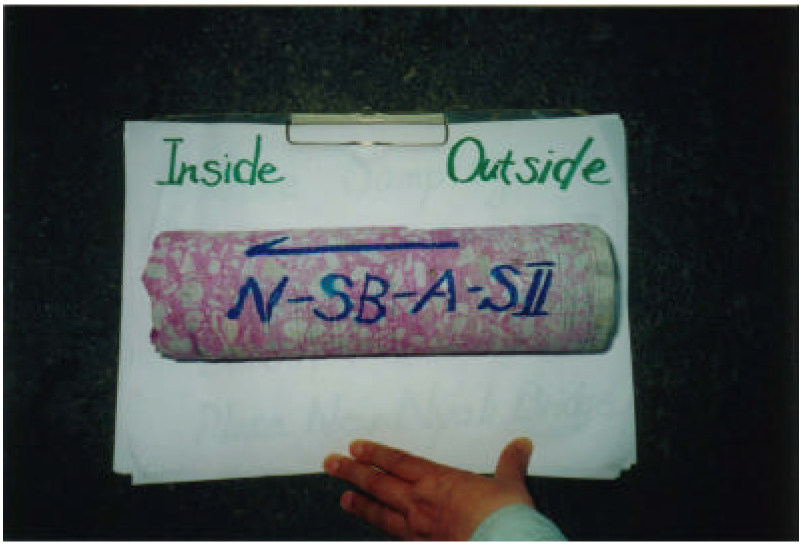
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-A1-NII	A1 Abutment



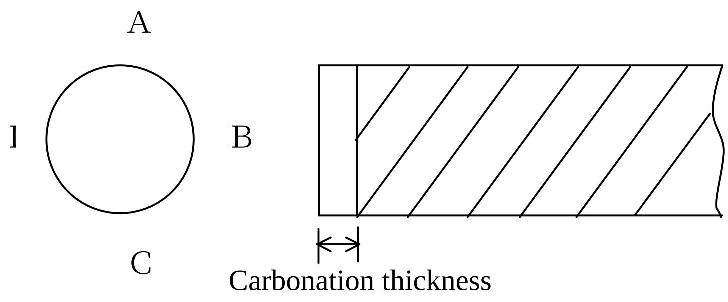
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



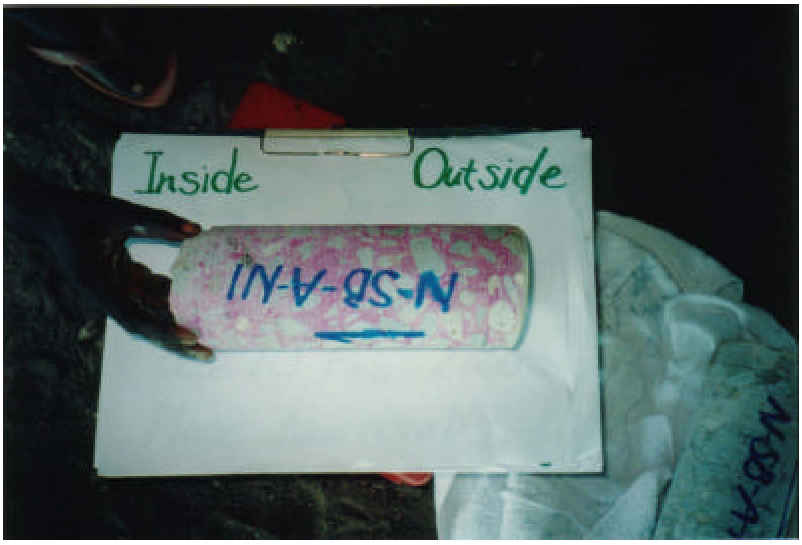
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-A2-S I	A2 Abutment



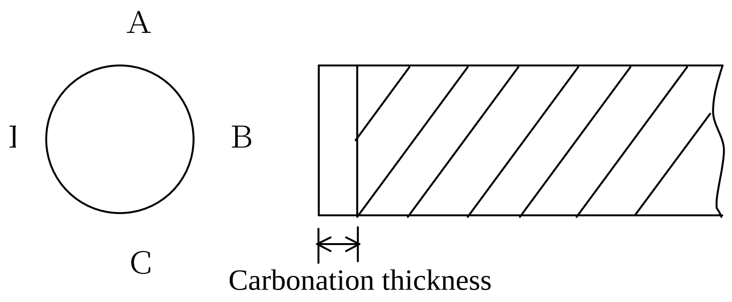
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



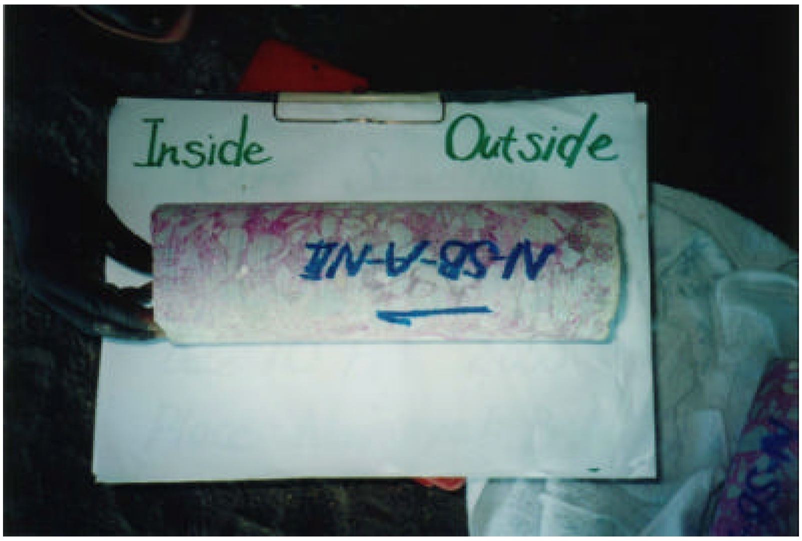
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-A2-SII	A2 Abutment



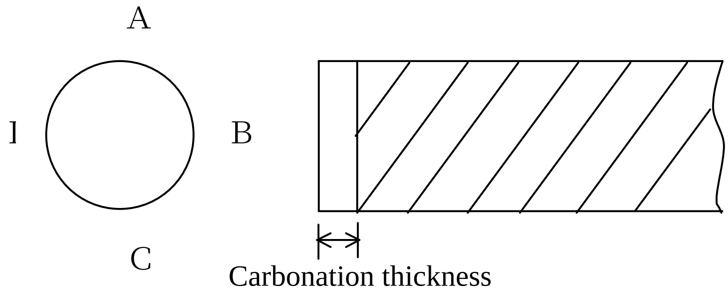
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



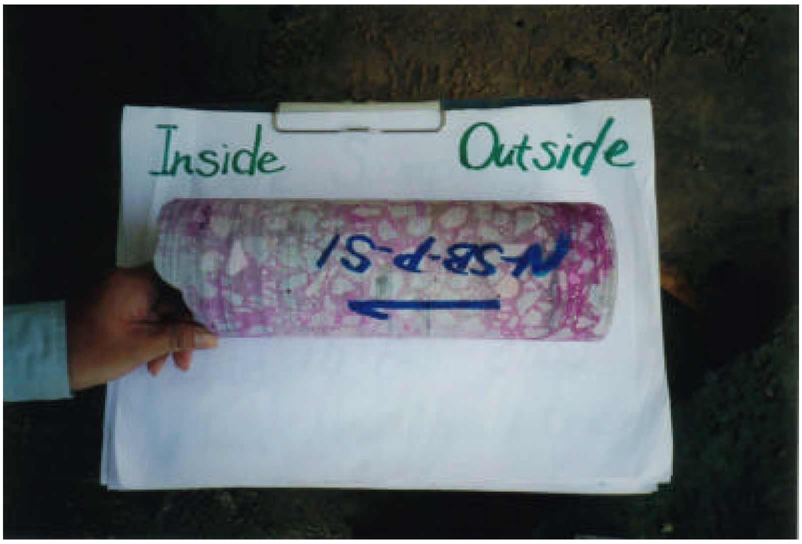
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-P3-S I	P3 Pier (Monbasa side)



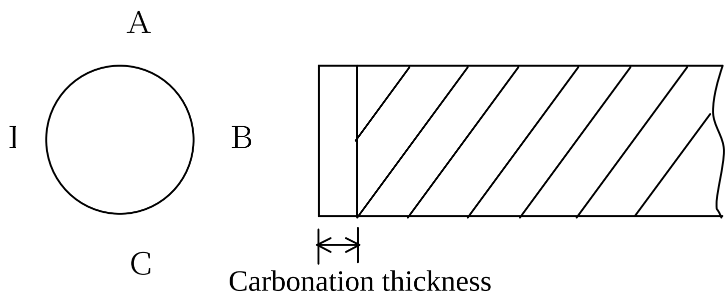
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



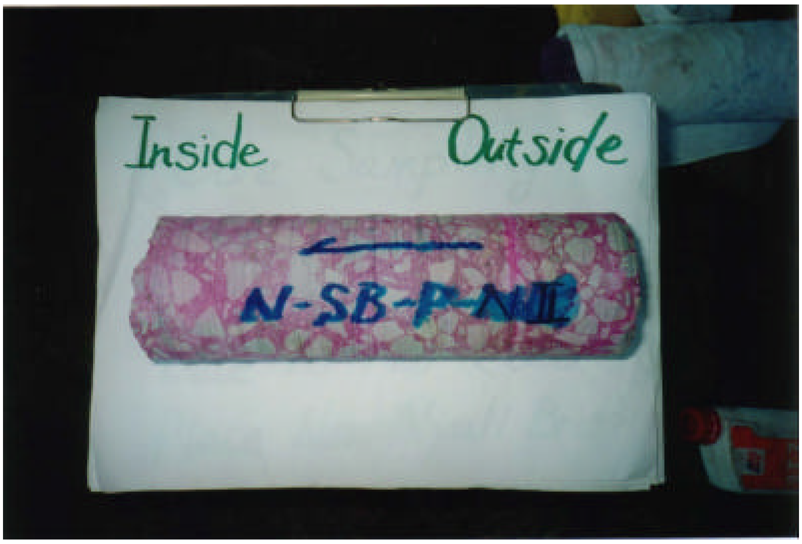
Carbonation thickness test result

Bridge Name	Core No.	location
New Nyali Bridge	N-SB-P3-SII	P3 Pier (Monbasa side)



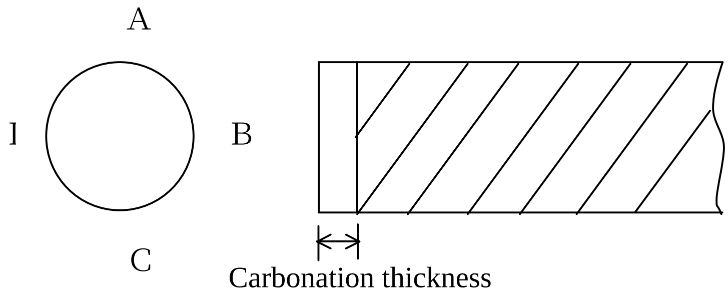
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



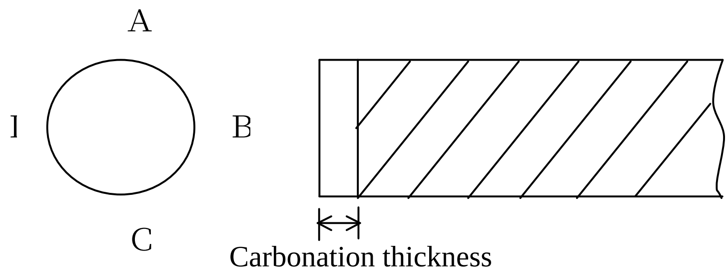
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-SS-E Ⅰ	Box girder inside side span



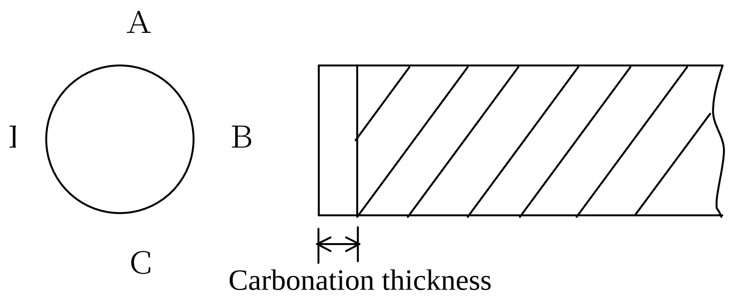
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



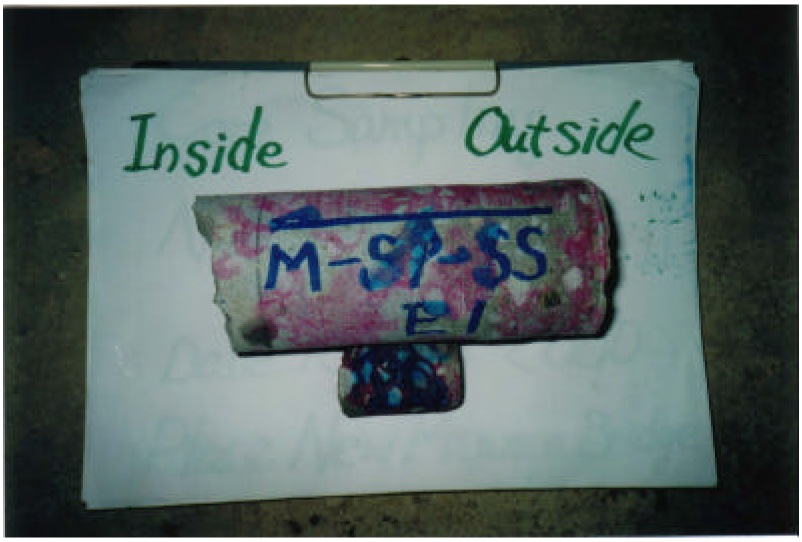
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-SS-EII	Box girder inside side span



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

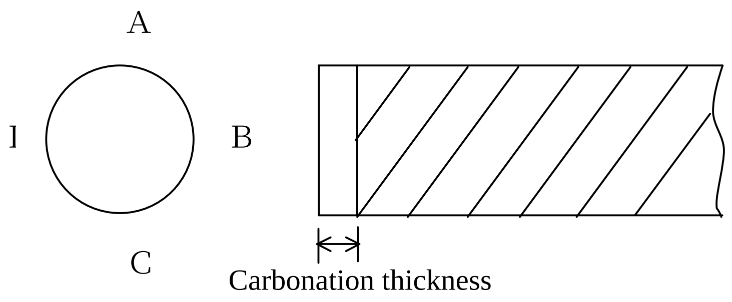
Photo of carbonation thickness test condition



M-SP-SS-EII

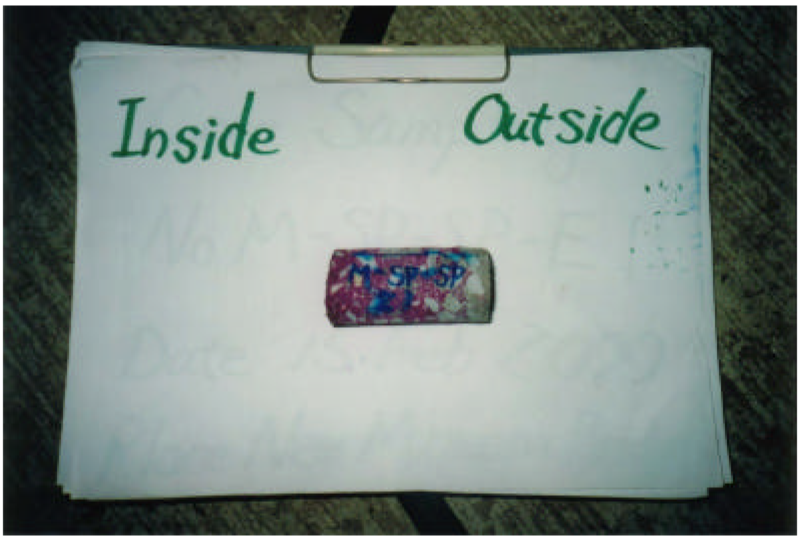
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-SP-EI	Box girder inside pier fulcrum



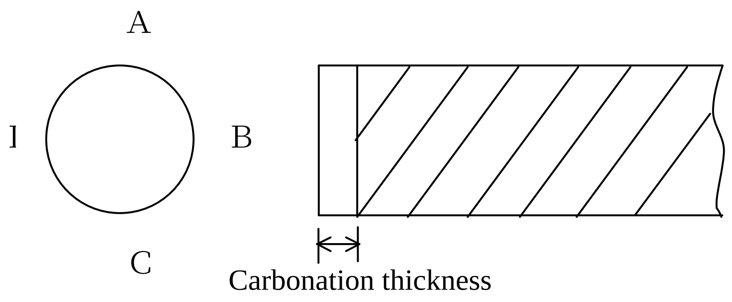
A	B	C	D	Average carbonation thickness
12mm	14mm	13mm	10mm	12mm

Photo of carbonation thickness test condition



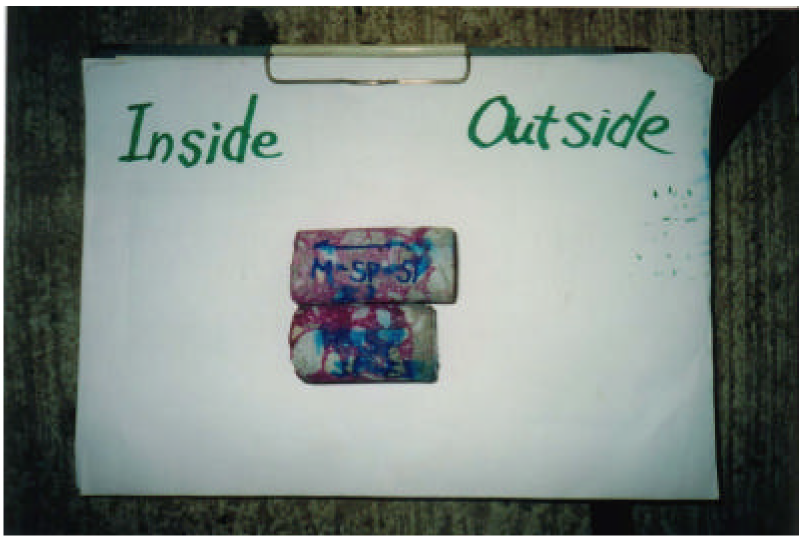
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-SP-EII	Box girder inside pier fulcrum



A	B	C	D	Average carbonation thickness
10㎜	13㎜	12㎜	13㎜	12㎜

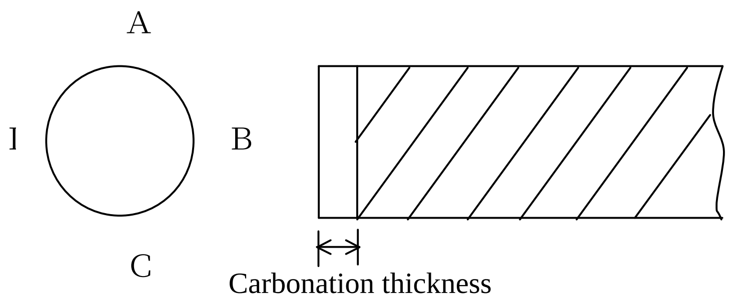
<u>Photo of carbonation thickness test condition</u>



M-SP-SP-EII

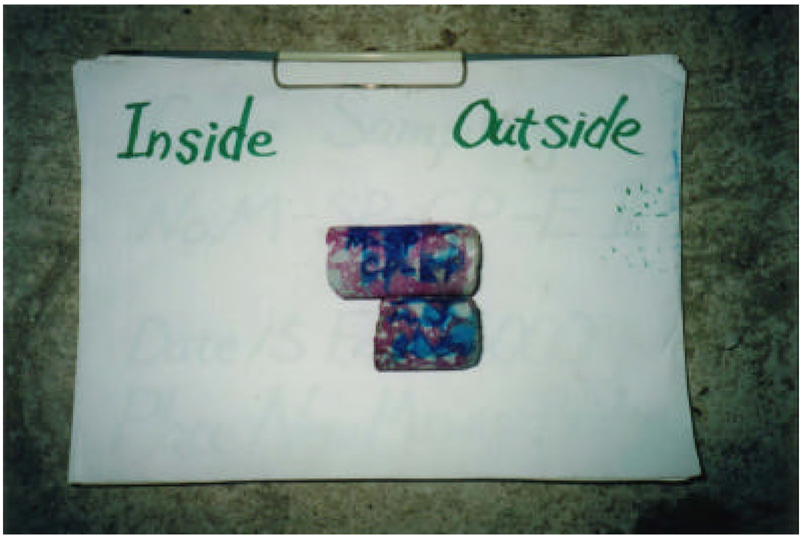
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-CP-EI	Box girder inside center span



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

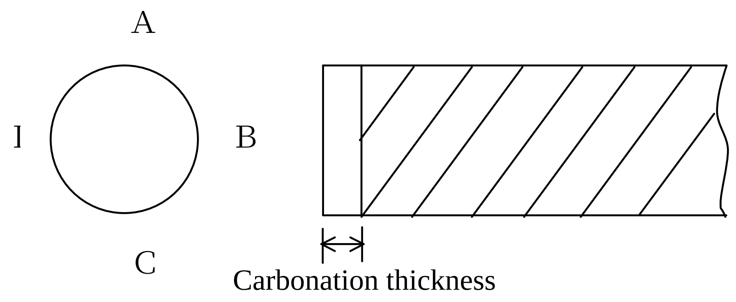
<u>Photo of carbonation thickness test condition</u>



M-SP-CP-E I

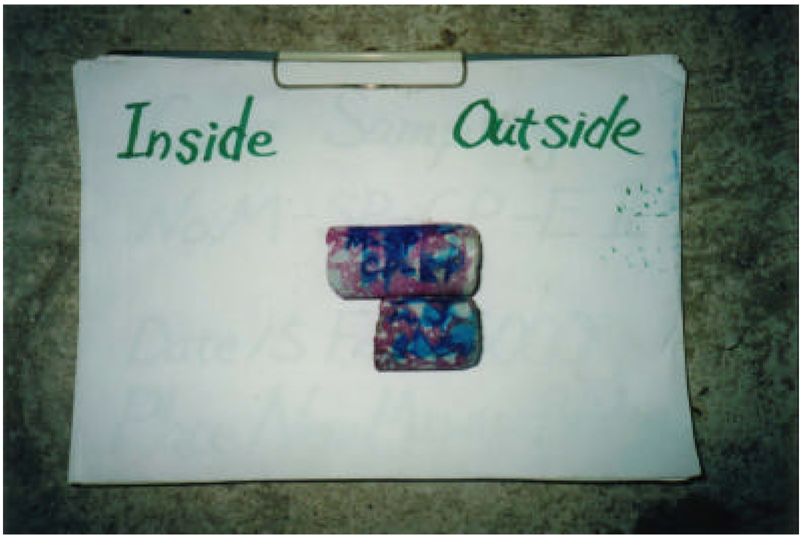
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SP-CP-EII	Box girder inside center span



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

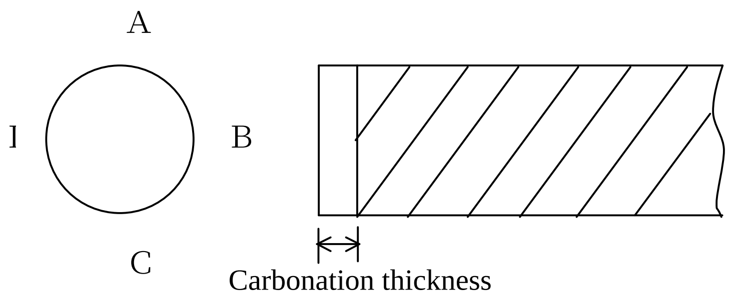
<u>Photo of carbonation thickness test condition</u>



M-SP-CP-EII

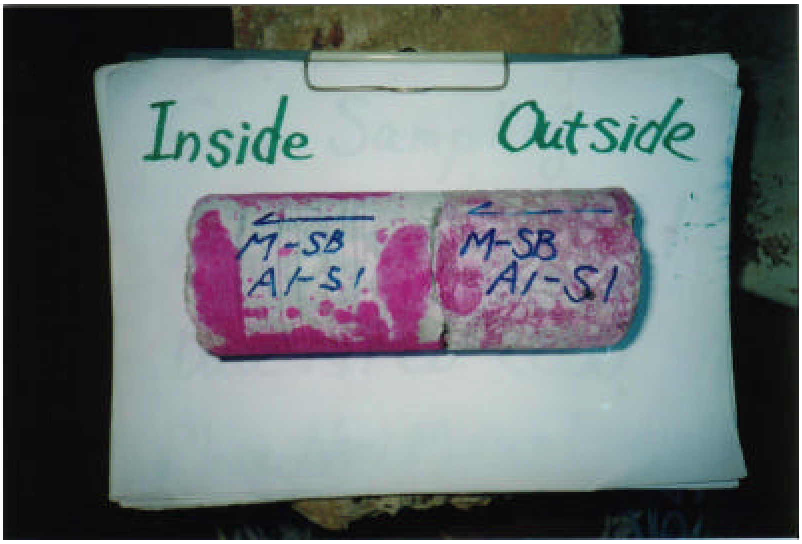
Carbonation thickness test result

Bridge Name New Mtwapa Bridge	Core No. M-SB-A1-SI	location A1 Abutment



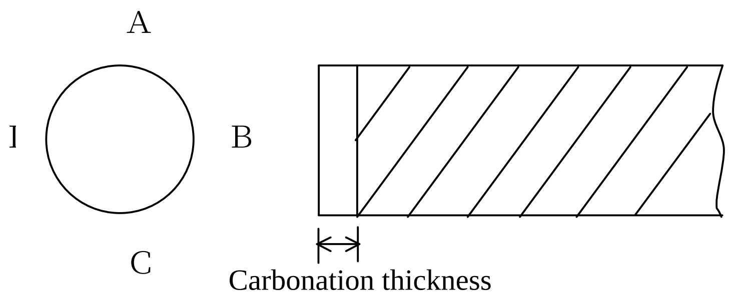
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



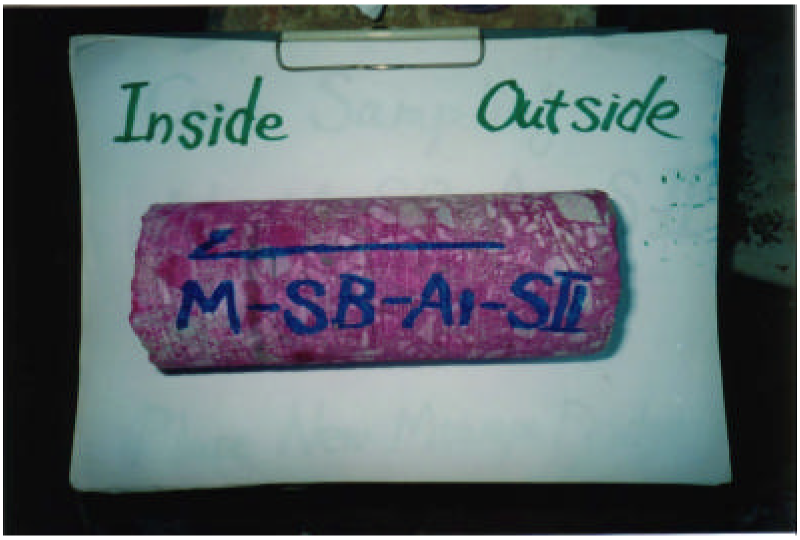
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-A1-SII	A1 Abutment



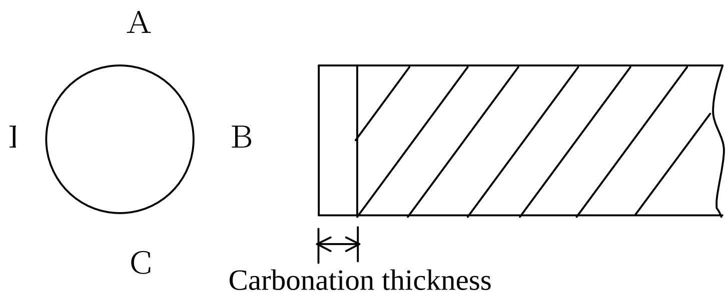
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

中性化深さ試験状況写真



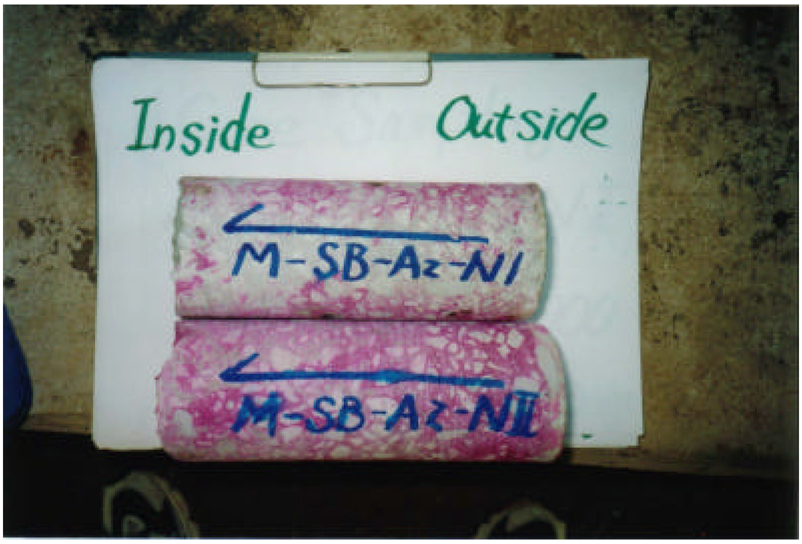
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-A2-NI	A2 Abutment



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

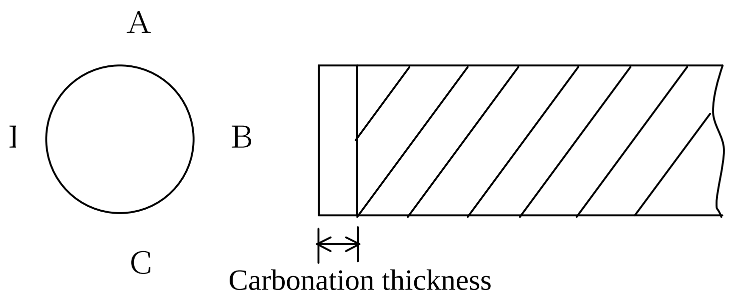
<u>Photo of carbonation thickness test condition</u>



M-SB-A2-N I

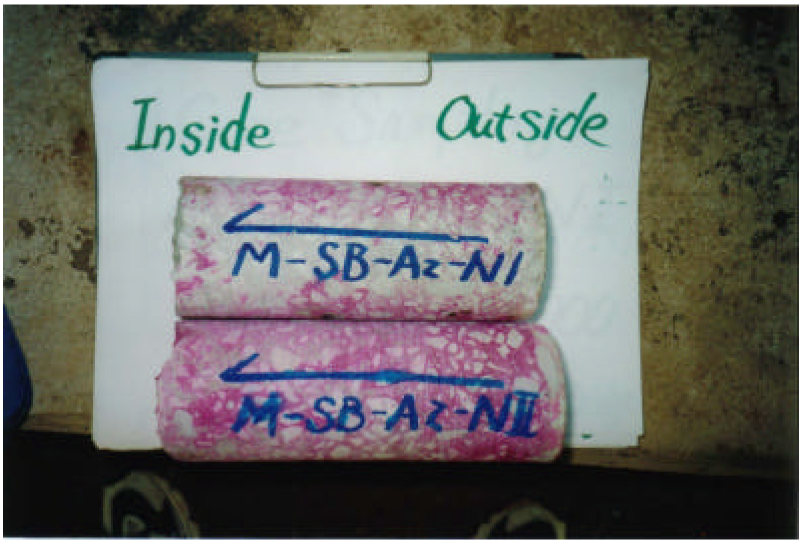
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-A2-NII	A2 Abutment



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

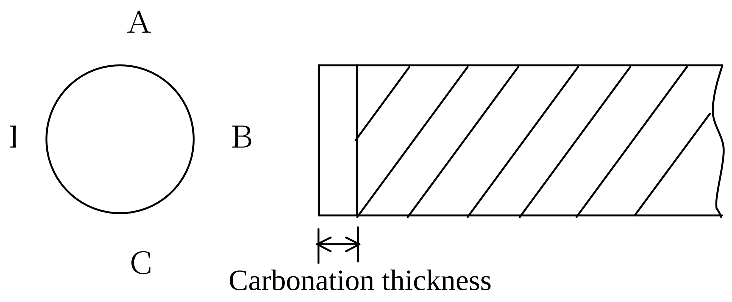
<u>Photo of carbonation thickness test condition</u>



M-SB-A2-NII

Carbonation thickness test result

Bridge Name New Mtwapa Bridge	Core No. M-SB-P1-SI	location P1 pier



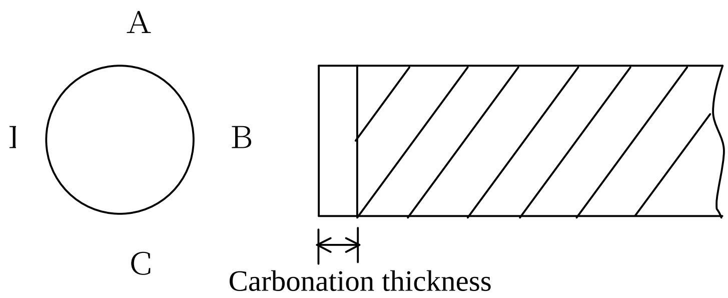
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition

M-SB-P1-SI

Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-P1-SII	P1 pier



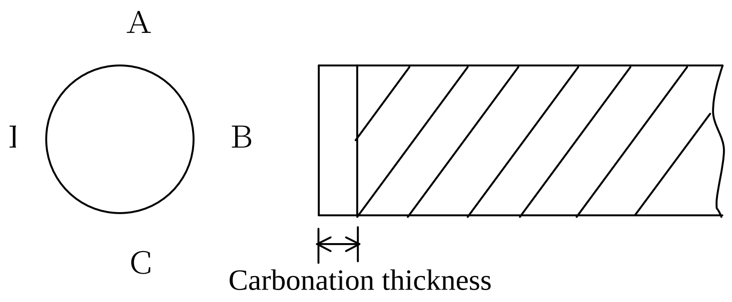
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition

M-SB-P1-SII

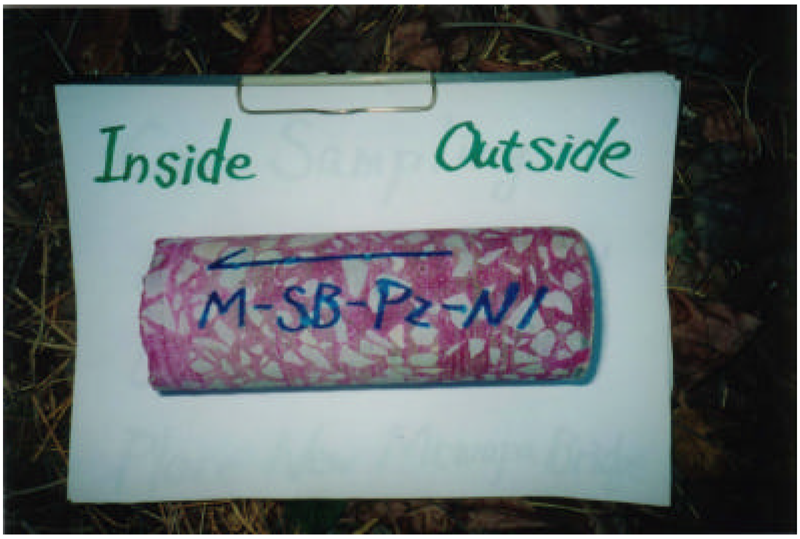
Carbonation thickness test result

Bridge Name New Mtwapa Bridge	Core No. M-SB-P2-NI	location P2 pier



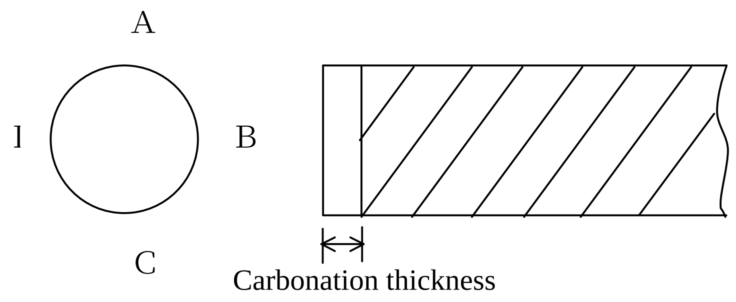
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

Photo of carbonation thickness test condition



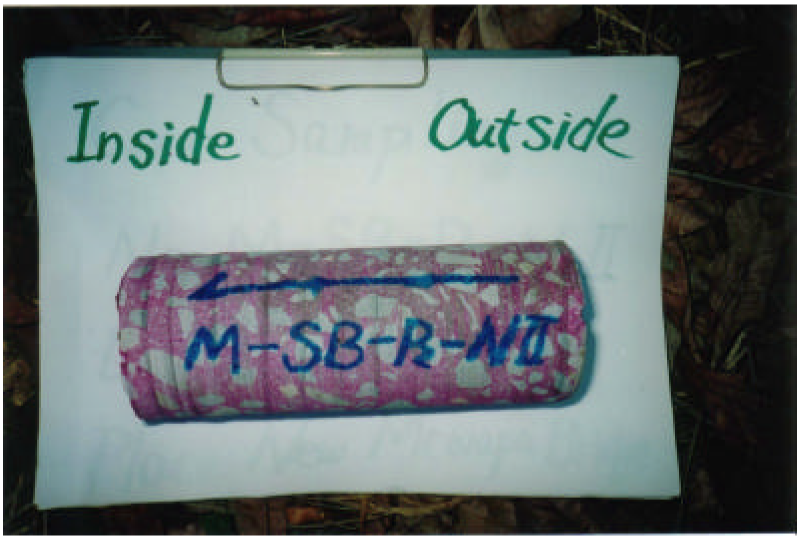
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-P2-NII	P2 pier



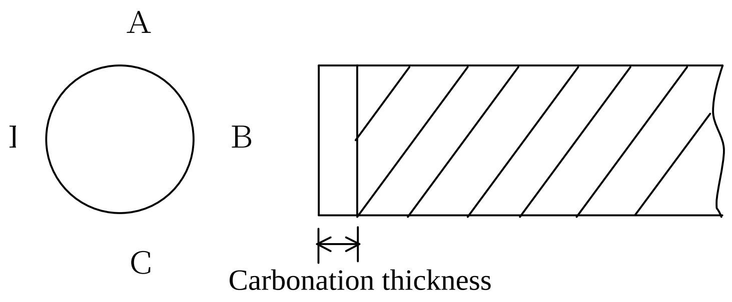
A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



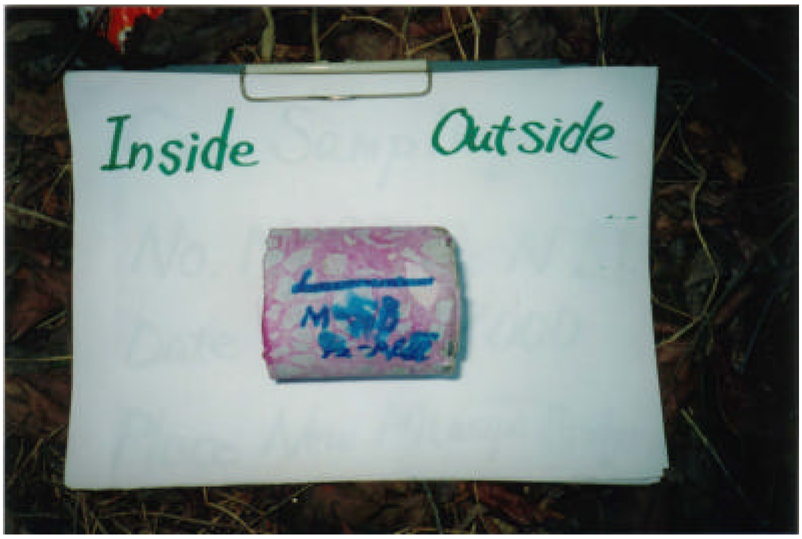
Carbonation thickness test result

Bridge Name	Core No.	location
New Mtwapa Bridge	M-SB-P2-NIII	P2 pier



A	B	C	D	Average carbonation thickness
0mm	0mm	0mm	0mm	0mm

<u>Photo of carbonation thickness test condition</u>



4. Steel bars corrosion test

Steel bars corrosion inspection Self-potential method)

New Nyali Bridge

	Measuring No.	Measuring point	Potential E (mV)	Corrosion ranks
Super-structure	N-No.1	Main girder inside, side span	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	N-No.2	Main girder inside, pier fulcrum	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	N-No.3	Main girder inside, center span	-250≧E＞-350	Ⅱ
Sub-structure	N-No.4	A1 abutment	-250＜E -250≧E＞-350	Ⅰ・Ⅱ
	N-No.5	A2 abutment	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	N-No.6	P3 pier	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	N-No.7	P4 pier	-350≧E＞-450 E≦-450	Ⅲ・Ⅳ

New Mtwapa Bridge

	Measuring No.	Measuring point	Potential E (mV)	Corrosion ranks
Super-structure	M-No.1	Main girder inside, side span	-250＜E E≦-450	Ⅰ・Ⅳ
	M-No.2	Main girder inside, pier fulcrum	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	M-No.3	Main girder inside, center span	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
Sub-structure	M-No.4	A1 abutment	-250≧E＞-350 -350≧E＞-450	Ⅱ・Ⅲ
	M-No.5	A2 abutment	-250＜E -250≧E＞-350	Ⅰ・Ⅱ
	M-No.6	P3 pier	-250≧E＞-350	Ⅱ
	M-No.7	P4 pier	-350≧E＞-450	Ⅲ

Steel bar corrosion evaluation standard

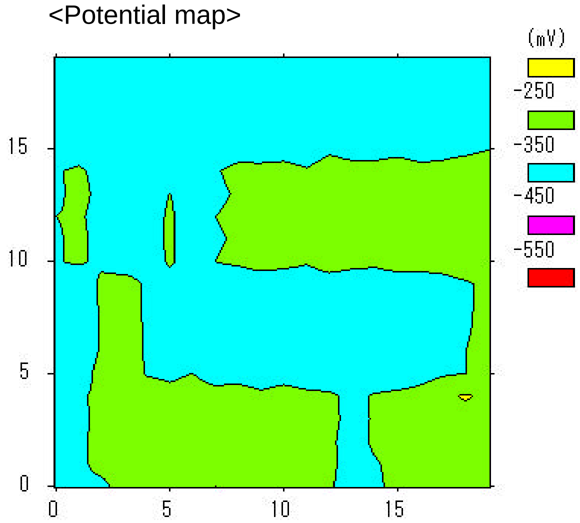
Potential E (mV)	Corrosion condition	Corrosion rank
-250＜E	Black skin without corrosion	Ⅰ
-350＜E≦-250	Few spot rusting in bar surface	Ⅱ
-450＜E≦-350	Thin loose rust over the bar surface, with rust adhering to concrete	Ⅲ
E≦-450	Slightly thick expansive rust, with relatively few sectional loss	Ⅳ
E≦-450	Excessive expansive rust over the entire bar, with sectional loss	Ⅴ

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.1	Box-girder insideside span(box No.6)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

Corrosion ranks	Corrosion condition	Natural current E(mV)
I	Black skin without corrosion	－250＜E
II	Few spot rusts in bar surface	－350＜E≦－250
III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

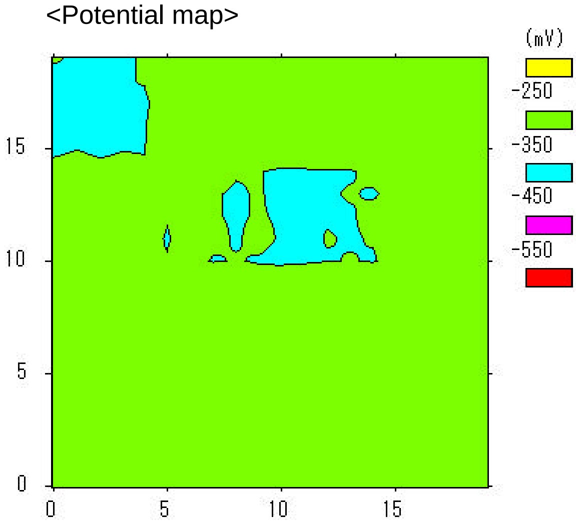
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremer	Location
New Nyali Bridge	N - No.2	Box-girder insideside span(box No.7)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

Corrosion ranks	Corrosion condition	Natural current E(mV)
I	Black skin without corrosion	－250＜E
II	Few spot rusts in bar surface	－350＜E≦－250
III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

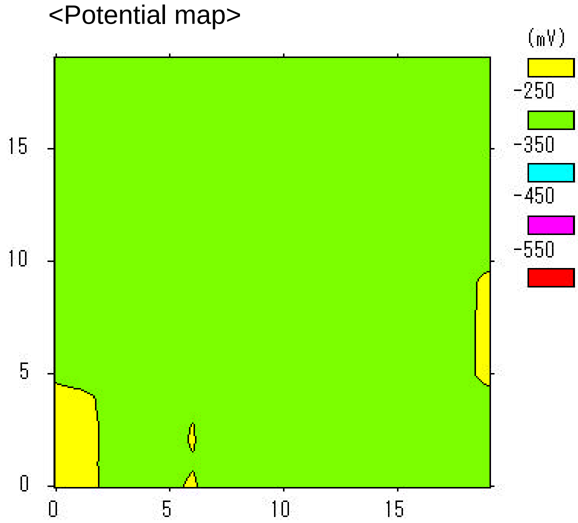
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.3	Box-girder insideside span(box No.10)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

Corrosion ranks	Corrosion condition	Natural current E(mV)
I	Black skin without corrosion	－250＜E
II	Few spot rusts in bar surface	－350＜E≦－250
III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

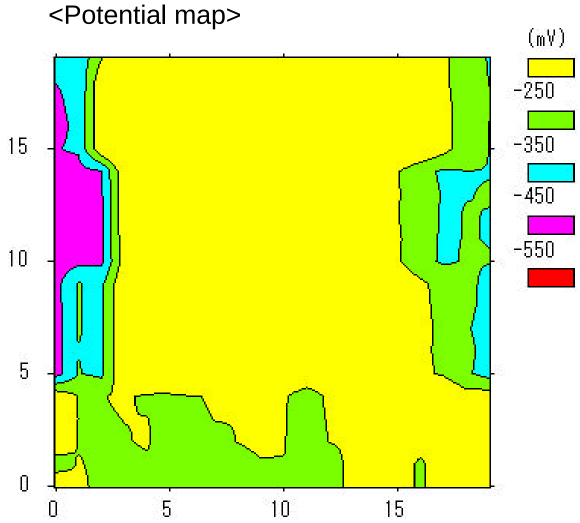
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.4	Abutment A1

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

Corrosion ranks	Corrosion condition	Natural current E(mV)
Ⅰ	Black skin without corrosion	－250＜E
Ⅱ	Few spot rusts in bar surface	－350＜E≦－250
Ⅲ	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
Ⅳ	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
Ⅴ	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

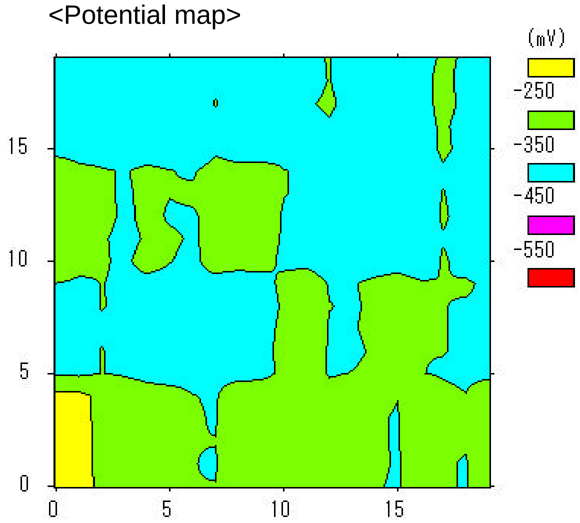
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.5	Abutment A2

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

Corrosion ranks	Corrosion condition	Natural current E(mV)
I	Black skin without corrosion	－250＜E
II	Few spot rusts in bar surface	－350＜E≦－250
III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

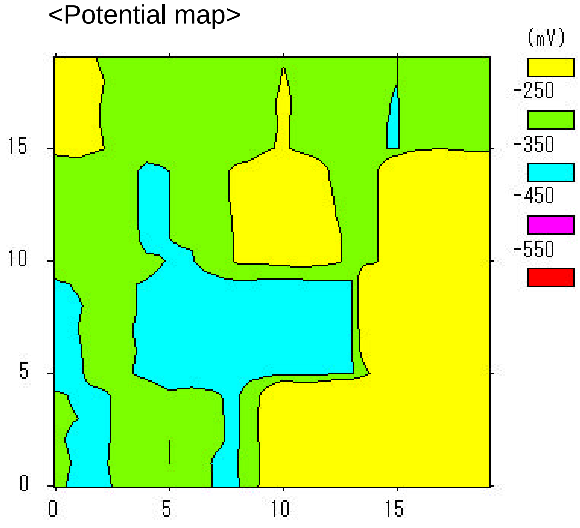
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.6	Pier P3(Monbasa side)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
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III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

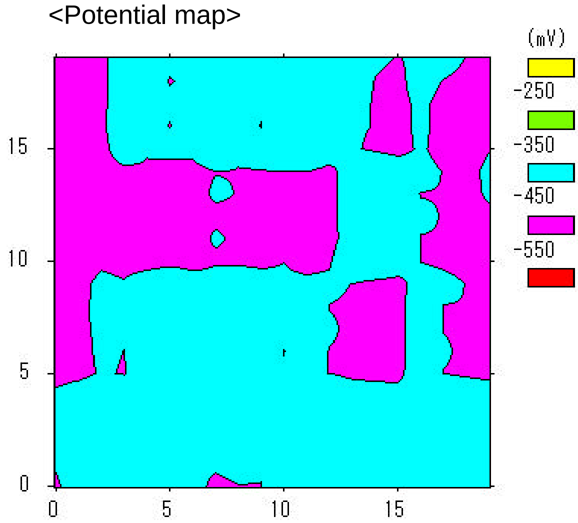
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Nyali Bridge	N - No.7	Pier P4(Malindi side)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
-350 < Potential ≦ -250	10	2.5
-450 < Potential ≦ -350	20	5.0
-550 < Potential ≦ -450	107	26.8
Potential ≦ -550	12	3.0

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I	Black skin without corrosion	－250＜E
II	Few spot rusts in bar surface	－350＜E≦－250
III	Thin loose rust spread over the bar surface, with rust adhering to concrete	－450＜E≦－350
IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

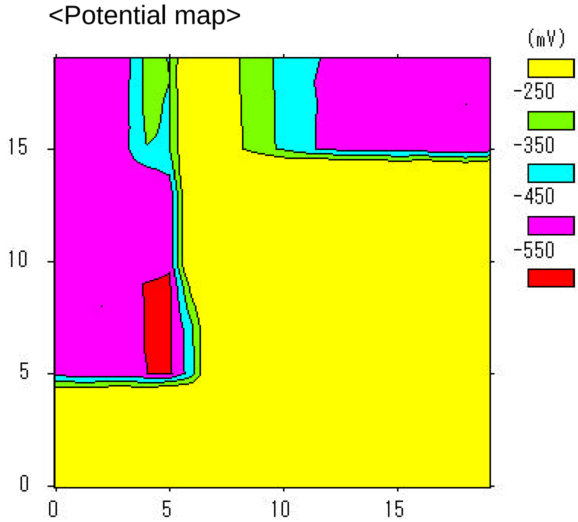
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.1	Box-girder insideside span(box No.1)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
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IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

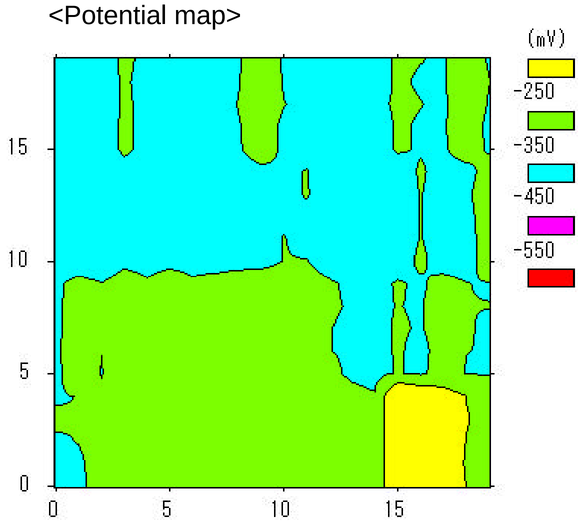
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.2	Box-girder insideside span(box No.4)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
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V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

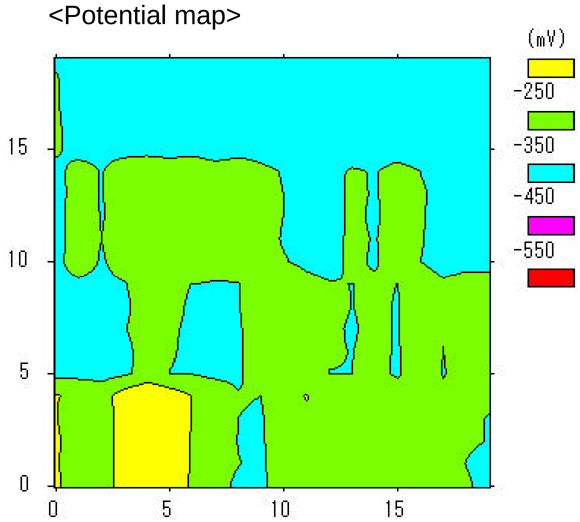
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.3	Box-girder insideside span(box No.7)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
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II	Few spot rusts in bar surface	－350＜E≦－250
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IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

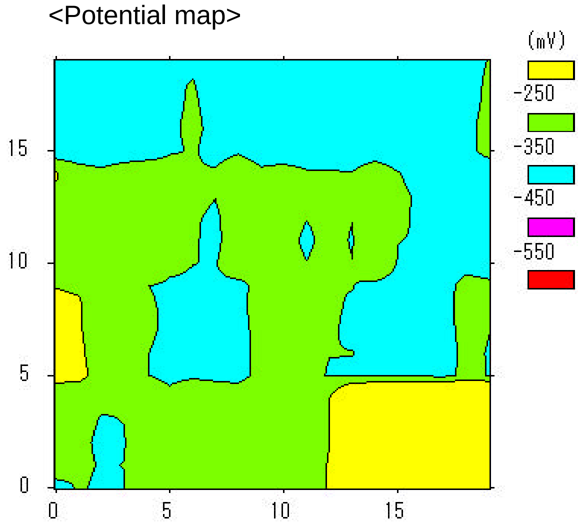
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.4	Abutment A1

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
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IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

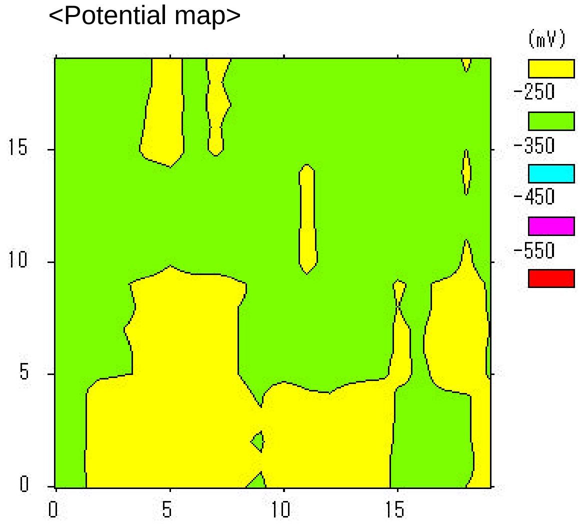
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremer	Location
New Mtwapa Bridge	M - No.5	Abutment A2

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
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V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

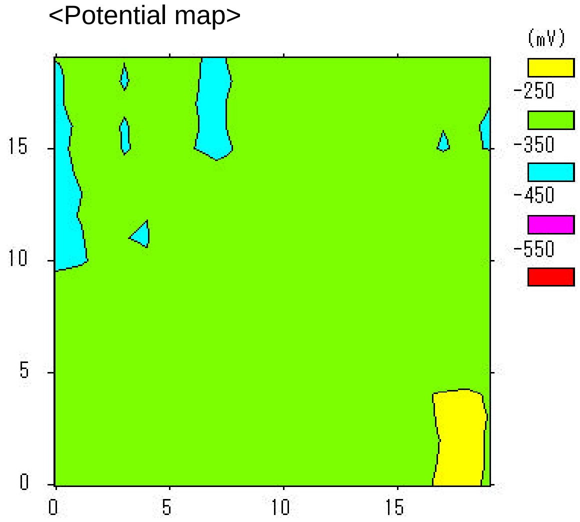
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.6	Pier P1(Monbasa side)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution
Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
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IV	Slightly thick expansive rust, but with relatively few sectional loss	E≦－450
V	Excessive expansive rust over the entire bar, with sectional loss	E≦－450

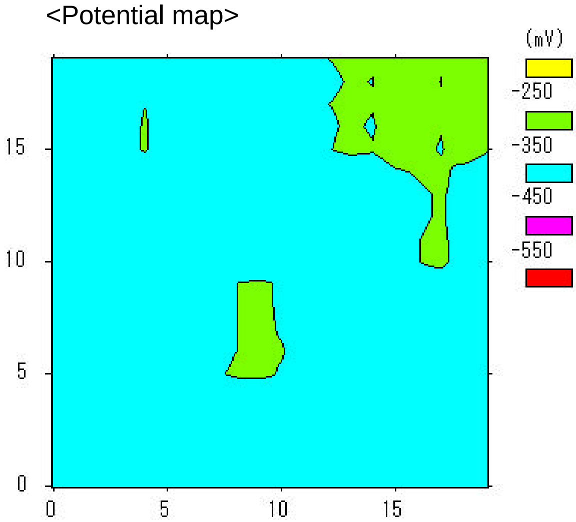
Photo of earth steel bar condition

Steel bar corrosion measurement　Self-potential method）

Bridge Name	Measuremei	Location
New Mtwapa Bridge	M - No.7	Pier P2(Malindi side)

Potential map and measurement data

<Potential map>



File name	m-b11.trd
File No.	10
Location	Bridge
Equivalent electrode	Copper sulfate
Potential, max	36 mV
Potential, average	-212 mV
Potential, min	-575 mV
Potential comrrection value	0 mV
No. of effective data	400

Distribution type　Spontaneous distribution

Distribution of potential counts

Distribution range(mV)	Counts	Indication ratio(%)
-250 < Potential ≦	251	62.7
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Photo of earth steel bar condition

5. Soluble chlorine measurement test

1. Content of Test

The soluble chloride was measured on 13 pieces of concrete cores (Table 1) sampled from actual structures in Kenya, Africa.

Table 1 Concrete core No.

ϕ size(mm)	Core No.
50	2
50	3
50	5
50	7
50	10
50	12
100	1
100	3
100	5
100	7
100	9
100	11
100	13

2. Measurement Method

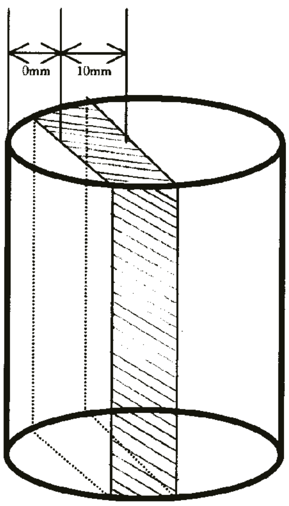
2.1 Sample preparation method

(1) ϕ 50mm concrete core

All cores were crushed for use as samples.

(2) ϕ 100mm concrete core

The core was cut in the axial direction, then the portion of Fig. 1 was cut for use as sample.



: Portion used as sample

Fig. 1 ϕ 100mm core sampling position

2.2 Test method

(1) Method

For the test method, the Analysis Method of Chloride in Hardened Concrete, JCI-SC4 and the reference literature of the Public Works Research Institute, Exposure Test Research on Salt Damage to Concrete Bridges (I) (July, 1982) were referenced.

In this test, the sample is heated to 50℃ and kept at this temperature by adding hot water of 50℃. The sample is shaken for 30 minutes to extract soluble chloride. After leaving the sample while keeping the temperature, the solution is filtrated. A part of solution is batched off, to which the nitric acid solution (2N) is added for acidification. The sample is then set to the potentiometric titration unit with a chloride ion selective electrode and subjected to potentiometric titration with N/200 silver nitrate standard solution. Note that extraction was made three times.

(2) Calculation formula

1) Nacl wt% in concrete

$$S\% = \frac{V_0 \times 0.000292 \times F}{W} \times \frac{R}{X} \times I \times 100$$

where、 V_0 : Consumption of N／200 silver nitrate standard solution (ml)

F : Factor of N／200 silver nitrate standard solution

W : Weighing amount of sample (g)

X : Batch-off amount of sample solution (ml)

I : Dilution magnification

R : 100

2) Conversion to the chloride content in $1m^3$ of concrete

Assuming that the unit weight of concrete per $1m^3$ is $2350kg/m^3$, calculate as follows:

①Nacl(kg/m^3) in $1m^3$

$$\text{Nacl in } 1m^3 \quad B = 2350 \times \frac{A}{100}$$

②cl^- (kg/m^3) in $1m^3$

cl^- in $1m^3$ $C = B \times 0.607$

0. 607 : Nacl/ cl^-

3. **Test Result**

Table 2 Soluble chloride measurement result

Unit：cl^- kg/m^3

Core size (φmm)	Core No	Extraction frequency			Total
		First	Second	Third	
50	2	0.035	0.021	0.013	0.069
50	3	0.040	0.016	0.010	0.066
50	5	0.026	0.014	0.009	0.049
50	7	0.029	0.014	0.008	0.051
50	10	0.025	0.015	0.009	0.050
50	12	0.038	0.010	0.006	0.054
100	1	0.065	0.034	0.021	0.121
100	3	0.019	0.018	0.011	0.048
100	5	0.027	0.016	0.010	0.053
100	7	0.026	0.014	0.009	0.049
100	9	0.022	0.016	0.010	0.049

6. Vibration test of existing bridge

Contents

1 Vibration Test

1.1 General

1.1.1 Objective

Generally, structures decrease in rigidity due to cracks or material deterioration. The natural frequency of structures decreases along with the decrease in the rigidity. Theoretically, therefore, measurement of the natural frequency and response value enables nondestructive evaluation of secular changes of rigidity. For example, the Railway Technical Research Institute is currently evaluating the soundness of substructures (piers, etc.) of JR companies by means of changes in the natural frequency[3)]. Our vibration test was conducted to obtain fundamental data for future repair and reinforcement work as well as to understand the extent of existing damage.

1.1.2 Method

Various tests were conducted to obtain vibration characteristics, such as a natural frequency. These include mircotremor measurement to measure microtremor under no load, a vehicle drop test to apply impact and vibration to a bridge body by dropping a vehicle from a ladder, and a vehicle running test to measure vibration while the vehicle is running. The test flow is shown in Fig. 1-1.

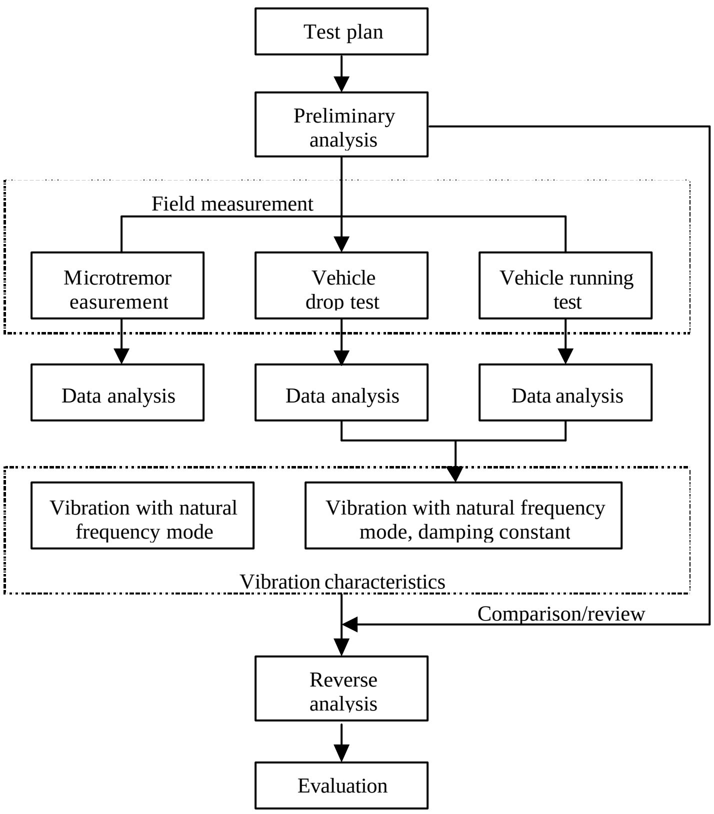


Fig. 1-1

1.1.3 Results

Table 1-1 shows the result of analysis using as parameters the natural frequency determined from a vibration test and the concrete elastic modulus (Ec). The analysis assumed that the entire section is effective and used the elastic modulus value (Ec=23kN/mm^2) determined from the elastic modulus test with concrete cores, the design value (Ec=28kN/mm^2), and the value (Ec=33kN/mm^2) estimated from the concrete strength of 51N/mm^2. The relationships between tests and analytical values are shown in Fig. 1-2. It is evident that the natural frequency in the deflection direction falls between the analytical value using the elastic modulus test value and that using the design value. It may be surmised from the above that no remarkable decrease in rigidity occurred within the range estimated from the vibration test result.

Table 1-1 Result

Vibration mode	Natural frequency (Hz)			
	Result	Analytical value		
		Ec=23kN/mm^2	Ec=28kN/mm^2	Ec=33kN/mm^2
Deflection, symmetrical primary mode	1.30	1.19	1.31	1.43
Deflection, antisymmetic primary mode	2.86	2.76	3.04	3.31
Horizontal symmetric primary mode	1.91	2.00	2.21	2.40

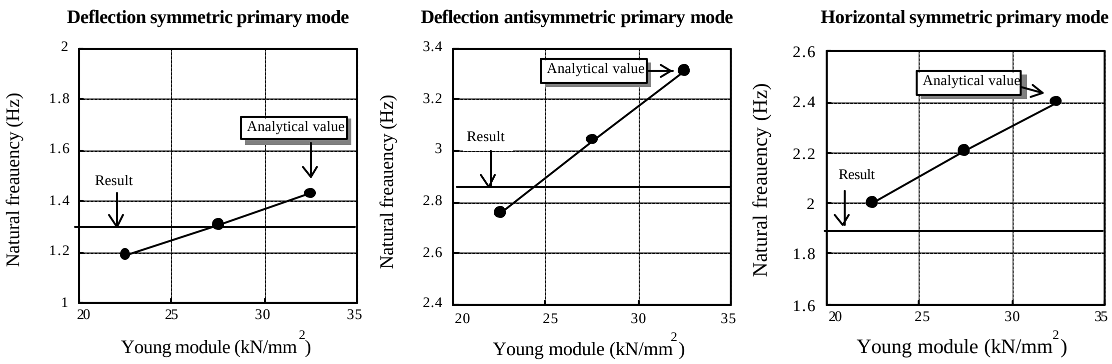


Fig. 1-2 Relationships between test and analytical values

1.2 Test Method

1.2.1 Test Items

This test involved the microtremor measurement, vehicle drop test, and vehicle running test to learn vibration characteristics of the superstructure.

Table 1-2 Test items

Test items	Vibration direction	Vibration	Objective
Microtremor measurement	Vertical	None	Natural frequency, vibration mode
Vehicle drop test	Vertical distortion	Dump truck (gross weight 19.92tf)	Natural frequency, vibration mode, modal damping
Vehicle running test	Vertical		

1) Natural frequency

Using a point with high response as a reference measuring point, signal data at this reference measuring point was applied to the equation (1). The frequency "f" was changed slightly each time, and "f" at which the sum of squares of the error term ε (t) in the equation (1) became a minimum value was assumed to be the natural frequency fr.

$$F(t) = A \cdot \cos(2\pi f \cdot t) + B \cdot \sin(2\pi f \cdot t) + C + \varepsilon(t) \qquad (1)$$

Where, F(t) : Time-series data A,B,C : Coefficients
ε (t) : Error term f : Frequency
t : Time

Using coefficients A and B, the phase θ_0 of the reference measuring point was determined from the equation (2).

$$\theta_0 = tan^{-1}(A/B) \qquad (2)$$

2) Vibration mode

By substituting the natural frequency fr determined in (1) and the signal data at each measuring point into the equation (1), coefficients A and B that are components of fr were determined. The amplitude α and phase difference θ were determined from equations (3) and (4), which were then plotted to obtain the vibration mode diagram. In this test, the response on the pier was measured in part of test cases. Since the response on the pier in each mode was about 1/100 of that at the measuring point where the response was maximum, points on abutments and piers were assumed to be steady points when the mode diagram was prepared.

$$\alpha = \sqrt{(A^2 + B^2)} \qquad (3)$$

$$\theta = tan^{-1}(A/B) - \theta_0 \qquad (4)$$

3) Modal damping

Measurement data were mode-decomposed for each vibration mode to obtain the free damping waveform of each mode. Using this waveform, the logarithmic decrement and damping constant were determined according to the free damping method. These were called the modal damping.

Logarithmic decrement $\delta = \frac{1}{n}\ln\left(\frac{X_i}{X_{i+n}}\right)$ (5)

Damping constant $h \approx \frac{\delta}{2\pi}$ (6)

1.2.2 Test Method

Testes were made as described below.

(1) Microtremor measurement

Microtremor includes the ground vibration caused by the source around bridge and the vibration that is a ground tremor of natural energies such as waves and wind. The microtremor of bridge bodies was measured continuously for 30 seconds during periods when there were no running vehicles.

(2) Vehicle drop test

A test vehicle (15tf truck crane) was allowed to ride over the ladder with its front wheels, then dropped from it. The impact vibration and damping free vibration were measured. The test condition is shown in Photo 1. The test car drop position was set in the middle of the width to excite vibration in the deflection direction. It was set also in the middle of the seaside lane to cause torsion and horizontal vibration. As shown in Fig. 13, four points were used: two points in the deflection direction (drop points ① and ②) and two points in torsion and horizontal directions (drop points ③ and ④). The test level is shown in Table 1-3. The drop height was set to 150 mm and a scaffolding board was placed at drop points to prevent localized loading (Fig. 14). For the test vehicle, the gross weight and front axle, intermediate axle, and rear axle weights, and natural frequency were measured (Table 1-4). The natural frequency of the test vehicle was measured on a firm ground so that it is not affected by the natural frequency of bridge. An accelerometer was installed to the test vehicle to measure the acceleration response when the vehicle was dropped, then the spectrum analysis was made on waveforms after an elapse of five seconds to determine the natural frequency of the vehicle.

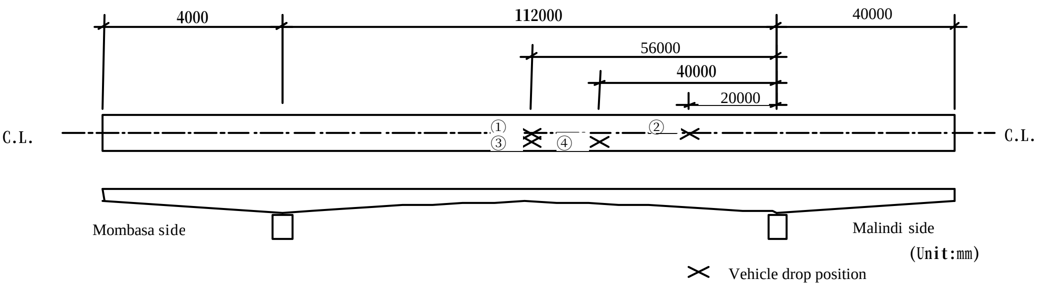
 

Photo 1　Vehicle drop test condition

Table 1-3　Vehicle drop test level

Drop position			Arrange-ment of accelero-meter	Measure-ment frequency
Case	Bridge axis direction (Center span)	Width direction		
①	Middle of span	Middle of width	Vertical	5
②	20 m from the Malindi side pier			5
③	Middle of span	Middle of seaside lane	Horizontal	5
④	40 m from the Malindi side pier			5

Fig. 1.3　Vehicle drop position



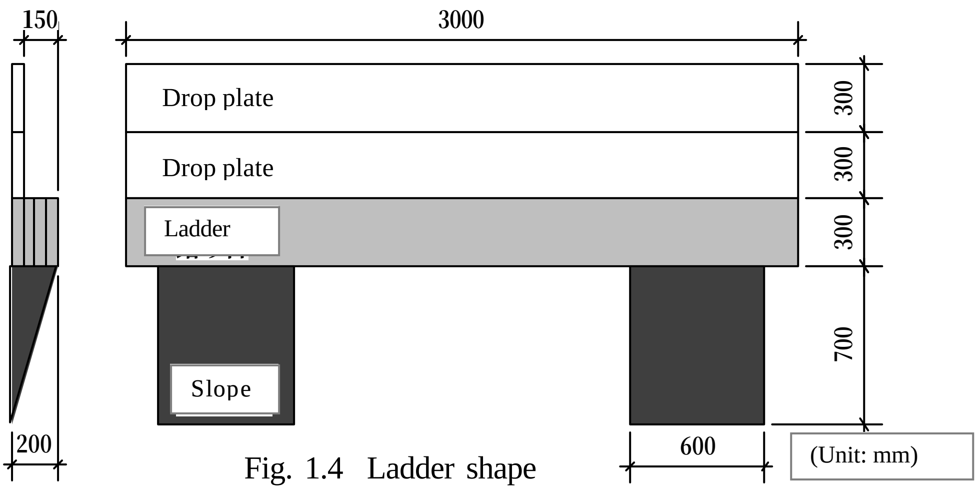


Fig. 1.4　Ladder shape

Table 1.4 Vehicle data

Vehicle type	Large three-axle (with back tandem) 15tf truck crane
Gross weight	19.92tf
Front axle weight	4.88tf
Intermediate and rear axle weight	15.04tf
Suspended natural frequency	1.39Hz
Non-suspended natural frequency	3.23Hz

(3) Vehicle running test

One test vehicle was run in the middle of the width or middle of each lane, during which vibration was measured. The running speed was nearly constant on the bridge, with target speeds set at three levels, 20, 40, 60 km/h (Table 1-5). The actual running speed of the test vehicle was determined by using a stopwatch and counting the time required to cross the bridge. The average speed per hour was calculated by assuming that the bridge length was 192 m.

Table 1-5 Vehicle running test level

<table>
<tr><th>Accelerator arrangement</th><th>Running position</th><th>Vehicle speed</th><th>Measurement frequency</th></tr>
<tr><td rowspan="3">Vertical
Torsion
direction</td><td rowspan="3">Middle
of width
Middle
of lane</td><td>20km/h</td><td>3</td></tr>
<tr><td>40km/h</td><td>3</td></tr>
<tr><td>60km/h</td><td>3</td></tr>
</table>

1.2.3 Measurement method

(1) Installation of measuring equipment

A servo type accelerometer (ASQ-ICA made by Kyowa Dengyo) was used for measurement. Its specifications are shown in Table 1-6. Accelerometers were arranged in two patterns according to the vibration mode to be measured (Fig. 1-5). The one is (a) vertical to measure the torsion vibration mode and the other is (b) in the torsion direction to measure torsion and horizontal vibration modes.

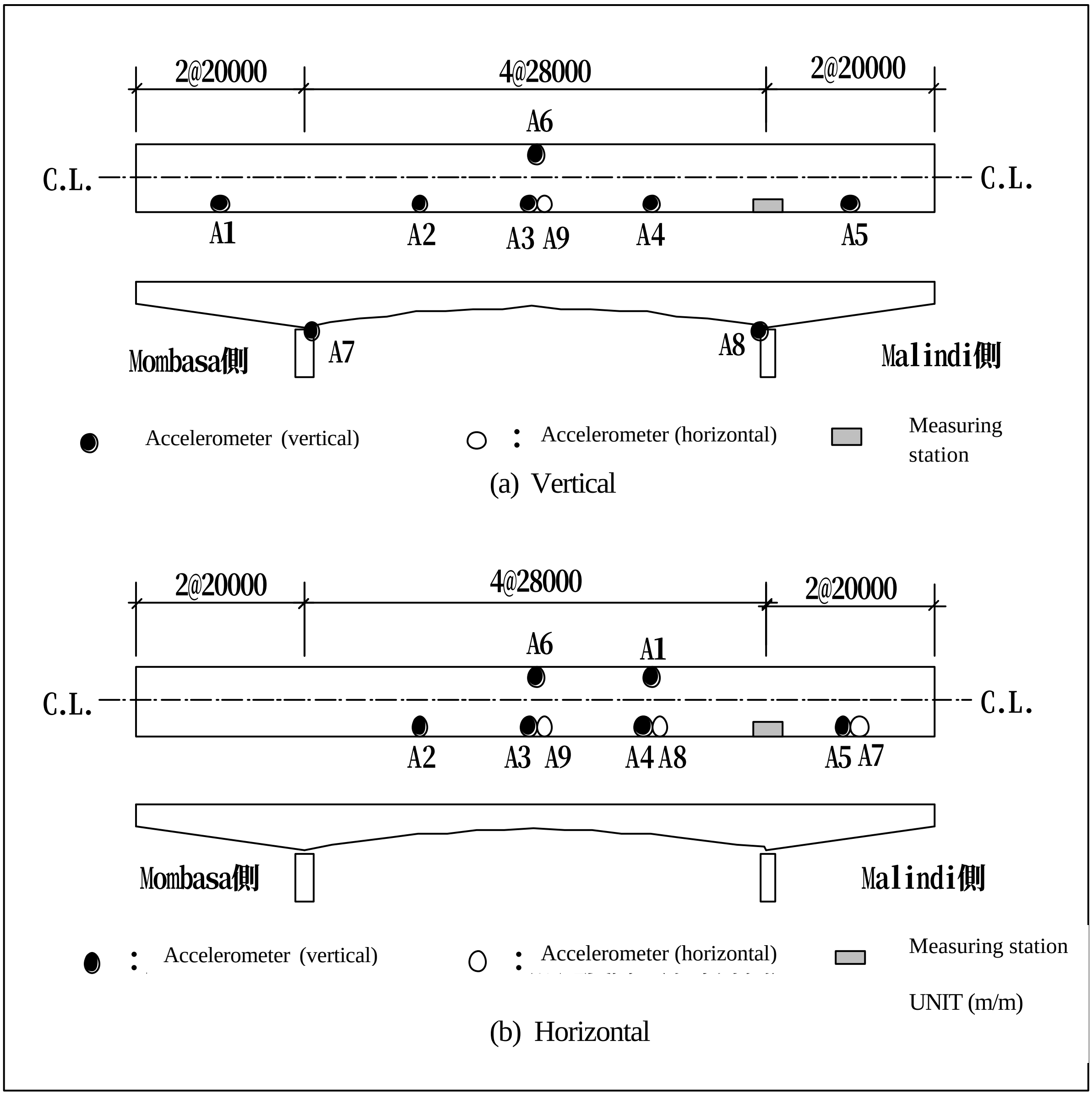


Fig. 1-5 Accelerometer installation position

Table 1-6 Specifications of accelerometer

Rated capacity	±1G	Rated output	5V±5%
Response frequency range	DC～100Hz	Cross sensitivity	Cross sensitivity
Non-linearity	±0.03%RO	Hysteresis	±0.05%RO
Temperature compensation range	-10～60℃	Mass	About 240 g

(2) Measurement method

The measurement system is shown in Fig. 1-6. Measurement was made within the acceleration range, with sampling intervals of $\Delta t=1$msec and a 30Hz low-pass filter. The signal data measured with the accelerometer was amplified in a dedicated amplifier, then converted to digital data with an A/D converter, and saved in the hard disk of a computer. A data recorder was used as a backup recorder. In the test field, the waveform was output in real time on a monitor to check the data. Concurrently, spectrum analysis was made to check approximate natural frequency and vibration mode.

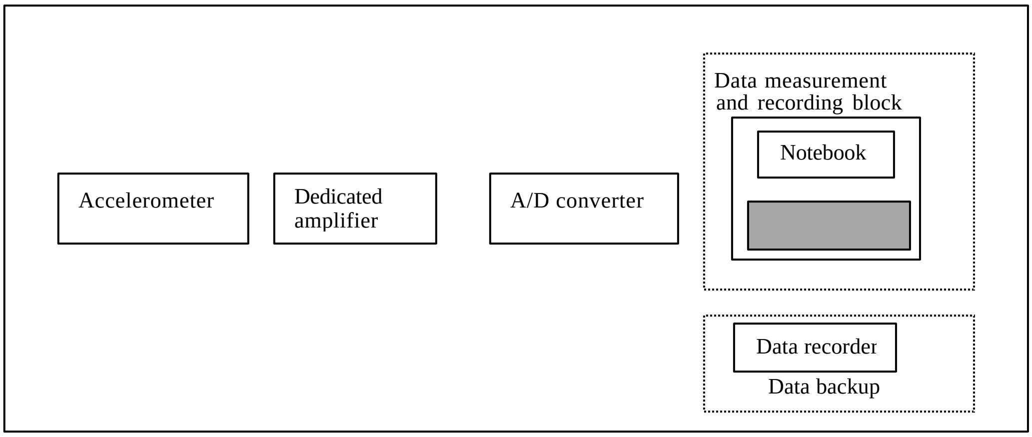


Fig. 1-6 Measurement system

1.3 Test results

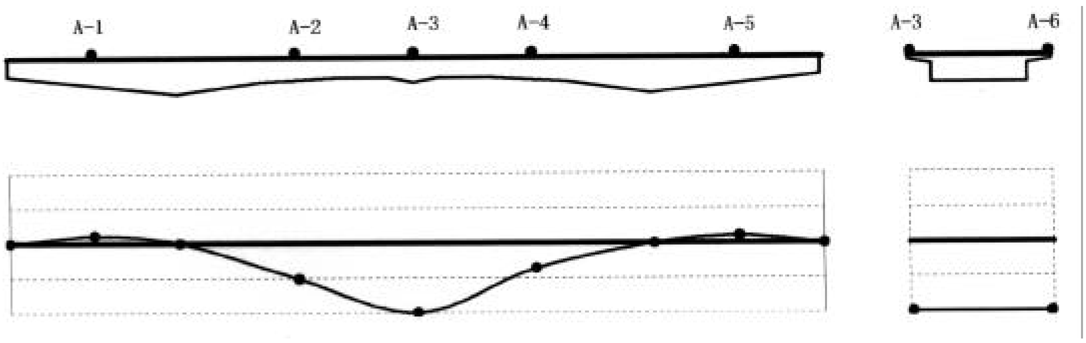
1.3.1 Result of microtremor measurement

(1) Natural frequency and vibration mode

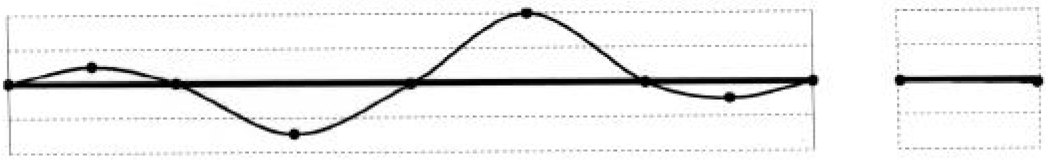
An example of the acceleration waveforms and spectrums obtained from microtremor measurement is shown in Fig. 1-7. The dominant frequency was observed at about 1.3 Hz and 2.9 Hz for torsion vibration and at about 1.9 Hz for horizontal vibration. These were confirmed to be the torsion symmetric primary mode, torsion antisymmetical primary mode, and horizontal symmetric primary mode, respectively. The test results are shown in Table 1-7 and the vibration mode in Fig. 1-8. Since measurement for the horizontal mode was made on a half span because of the number of sensors available, the mode shape as shown in Fig. 1-8 was obtained. However, the mode was judged to be a horizontal symmetric primary mode from a common sense viewpoint because this bridge is not of a singular construction.

Table 1-7 Microtremor measurement result

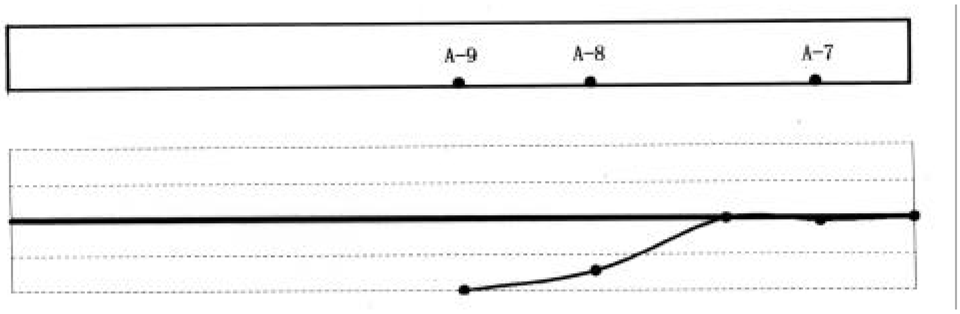
Natural frequency (Hz)	Torsion vibration		Horizontal vibration
	Symmetric primary mode	Antisymmetric primary mode	Symmetric primary mode
Average	1.30	2.86	1.91



Torsion symmetric primary mode（1.30Hz）



Torsion antisymmetical primary mode（2.86Hz）



Horizontal symmetric primary mode（1.91Hz）
Fig. 1-8 Vibration mode diagram (microtremor measurement)

1.3.2 Results of vehicle drop test

(1) Natural frequency and vibration mode

An example of the acceleration waveforms and spectrums obtained from the vehicle drop test is shown in Fig. 1-9. The dominant frequency was observed at about 1.4 Hz and about 2.9 Hz for torsion vibration and at about 2.0 Hz for horizontal vibration. These were confirmed to be the torsion symmetric primary mode, torsion antisymmetric primary mode, and horizontal symmetric primary mode, respectively. The test results are shown in Table 1-8 and the vibration mode in Fig. 1-10.

The vibration mode agrees approximately well with the result of microtremor measurement.

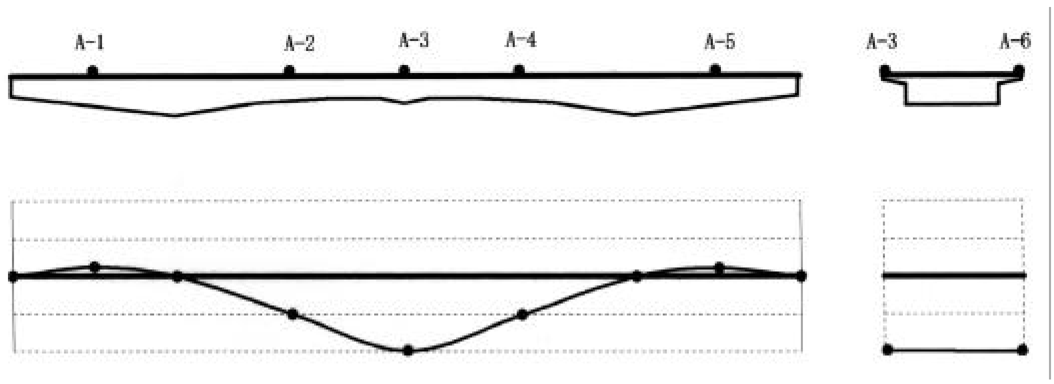
The natural frequency was slightly higher than the microtremor measurement result in the torsion symmetric and horizontal symmetric primary modes. In the torsion symmetric primary mode of the vehicle drop test, the frequency approximately similar to that of symmetric primary mode was dominant. This means that , in the torsion symmetric primary mode of the vehicle drop test, the natural frequency changed slightly because of inclusion of the vibration component in the surface outside direction. Similarly, change in the frequency in the horizontal symmetric primary mode is considered attributable to the effects of the vibration component in the surface inside direction because the vibration was in a vertical direction.

The vehicle drop test cannot be free from a certain amount of change in the natural frequency because it involves forced vibration caused by the impact of the dropped vehicle. In addition to torsion vibration caused by slight off-centering at the drop point, artificial off-center loading was attempted in cases ③ and ④ to observe vibration characteristics in horizontal and torsion directions.

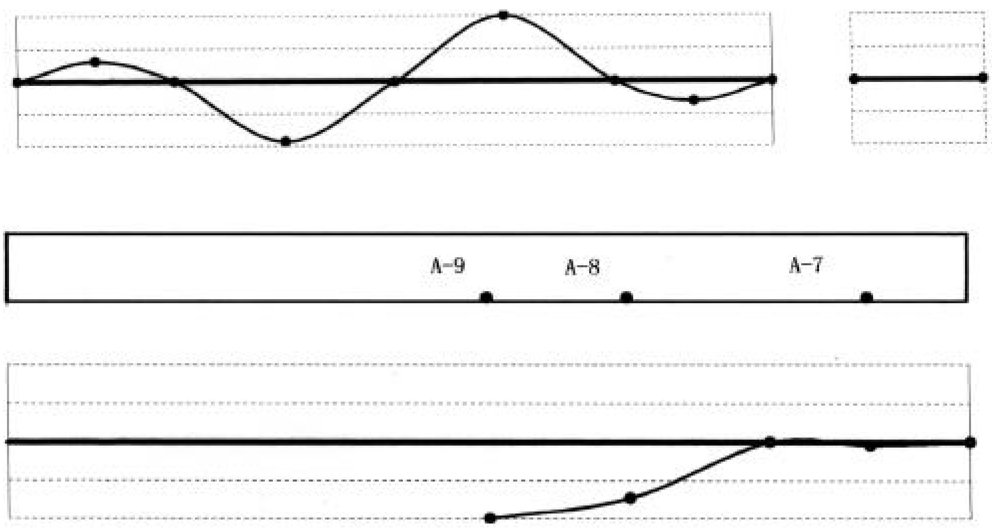
Table 1-8 Vehicle drop test result

Natural frequency(Hz)		Torsion vibration		Horizontal vibration
		Symmetric primary mode	Antisymmetric primary mode	Symmetric primary mode
Average	①	1.37	2.86	1.97
	②	1.39	2.85	1.95
	③	1.37	2.86	1.97
	④	1.38	2.81	1.97
Average		1.38（1.30）	2.85（2.86）	1.97（1.91）

Note: Values in parentheses in the bottom row indicate microtremor measurement results.



Torsion symmetric primary mode（1.38Hz）



Horizontal symmetric primary mode（1.97Hz）

Fig. 1-10 Vibration mode diagram （Vehicle drop test）

(2) Modal damping

From damping free vibration waveform after the vehicle drop test, mode decomposition was made for each mode and the damping constant was determined according to the free damping method. The damping constant was evaluated at measuring points where the response of each mode was the highest. Modal damping is shown in Table1-9.

Table 1-9 Modal damping

(a) Torsion symmetric primary mode

A3 standard		Natural frequency(Hz)	Logarithmic decrement	Damping constant
Average	①	1.37	0.039	0.006
	②	1.39	0.033	0.005
	③	1.37	0.041	0.007
	④	1.38	0.038	0.006
Total average		1.38	0.038	0.006

(b)Torsion antisymmetrcial primary mode

A2 standard		Natural frequency(Hz)	Logarithmic decrement	Damping constant
Average	①	2.86	0.037	0.006
	②	2.85	0.039	0.005
	③	2.86	0.036	0.007
	④	2.82	0.044	0.006
Total average		2.85	0.039	0.006

(c)Horizontal symmetric primary mode

A9 standard		Natural frequency(Hz)	Logarithmic decrement	Damping constant
Average	①	1.97	0.054	0.009
	②	1.95	0.028	0.004
	③	1.97	0.052	0.008
	④	1.97	0.048	0.008
Total average		1.97	0.046	0.007

From the above, it was found out that the damping factors of these bridges is about 4-5% in terms of logarithmic decrement and about 0.6 – 0.7% in terms of damping constant, regardless of the mode, for both torsion and horizontal vibrations. The damping constant is affected greatly by the type of superstructure main members, structure type, span length, and type of slab. It is generally said that the damping constant increases as damage proceeds, making comparison with other bridges difficult. In the case of the Yui Port Bridge (span: 30+70+30 m) of a three-span continuous prestressed concrete box-girder type whose structure is relatively similar to this bridge, however, the damping constant of torsion symmetric primary mode is reported to be 0.005 (natural frequency 1.7Hz)[1]. Namely, it is presumed that this bridge has damping factors generally observed with prestressed concrete box-girder bridges.

(3) Maximum response

The maximum response at each measuring point is summarized in Table 1-10. The displacement response was determined through second-order integration of acceleration data.

Table 1-10 Maximum response (Vehicle drop test)

Response	Case	Vertical・bridge surface						Vertical・pier		Horizontal	Horizontal	
		A1	A2	A3	A6	A4	A5	A7	A8	A9	A7	A8
Acceleration (gal)	①	4.38	8.32	11.04	11.15	7.82	3.96	0.28	0.70	3.77	—	—
	②	4.70	10.19	8.60	10.60	19.89	5.27	0.28	0.61	2.94	—	—
	③	9.81	9.17	13.27	12.99	9.02	5.46	—	—	2.08	1.36	1.72
	④	13.42	11.98	11.72	12.21	12.03	5.57	—	—	2.34	1.39	2.39
Displacement (㎜)	①	0.155	0.582	1.125	1.119	0.588	0.136	0.004	0.012	0.270	—	—
	②	0.117	0.462	0.872	0.874	0.498	0.118	0.003	0.010	0.127	—	—
	③	0.845	0.844	1.664	1.633	0.845	0.190	—	—	0.088	0.011	0.062
	④	0.741	0.727	1.235	1.200	0.727	0.174	—	—	0.159	0.018	0.117

From Table 1-10, it is known that the response, that is, the acceleration response, on the pier is on a microtremor level of about 1/5 of the maximum response on the bridge surface. Namely, settlement of piers did not occur due to the impact of the dropped vehicle.

1.3.3 Results of vehicle running test

(1) Natural frequency and vibration mode

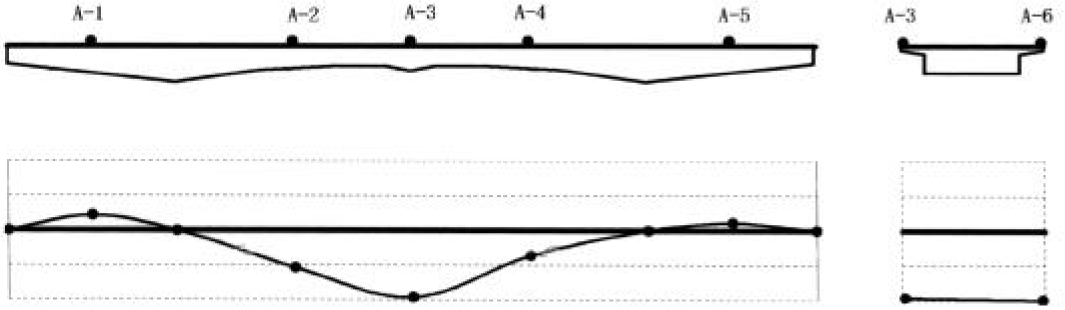
An example of acceleration waveforms and spectrums obtained from the vehicle running test is shown in Fig. 1-11. Dominant frequency was observed at about 1.3 Hz and about 2.9 Hz for torsion vibration and at about 1.9 Hz for horizontal vibration. These were confirmed to be torsion symmetric primary mode, torsional antisymmetric primary mode, and horizontal symmetric primary mode, respectively. The test results are shown in Table 1-11 and the vibration mode in Fig. 1-12. The vibration mode is approximately similar to that of the two other tests. Comparison of the test results with microtremor measurements in terms of natural frequencies indicates that the natural frequency is approximately similar for torsion symmetric and horizontal symmetric primary modes, but slightly lower for the torsion antisymmetric primary mode. As the approximately similar frequency is dominant on the horizontal direction accelerometer in the torsion antisymmetric primary mode, the mode containing the torsion vibration component appeared, resulting in a change in the natural frequency. Similarly to the case of the vehicle drop test, this may be attributed to off-center of the load relative to the width and inevitable error due to the nature of the test. Note that the natural frequency in the torsion antisymmetric primary mode was observed to be 2.81 Hz in the vehicle drop test case ④ in which torsion and horizontal vibrations were caused.

Change in the natural frequency as compared with the microtremor measurement appeared in the torsion symmetric primary mode for the vehicle drop test and in the torsion antisymmetric primary mode for the vehicle running test. Since these agree with the dominant mode in each test method, error caused by off-center of exciting force is considered to affect greatly the dominant mode in the test.

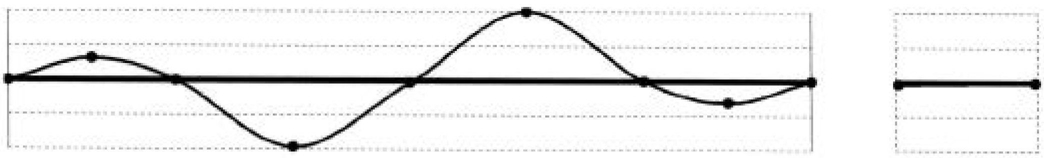
Table 1-11 Vehicle running test result

Natural frequency(Hz)		Torsion vibration		Horizontal vibration
		Symmetric primary mode	Antisymmetric primary mode	Symmetric primary mode
Average	20km/h	1.30	2.81	1.91
	40km/h	1.30	2.80	1.91
	60km/h	1.29	2.82	1.91
Average		1.30（1.30）	2.81（2.86）	1.91（1.91）

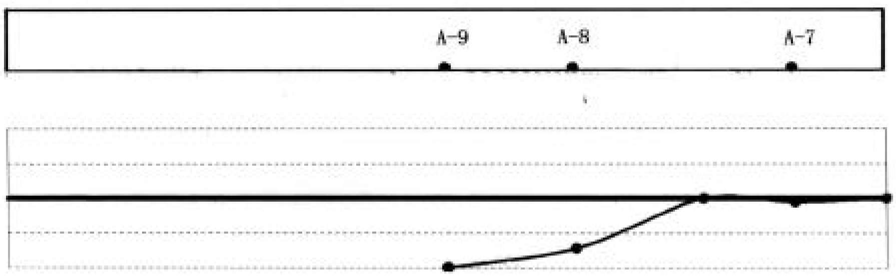
Note: Values in parentheses in the bottom row indicate microtremor measurement results.



Torsion symmetric primary mode（1.30Hz）



Torsion antisymmetric primary mode（2.81Hz）



Horizontal symmetric primary mode（1.91Hz）

Fig. 1-12 Vibration mode diagram（vehicle running test）

(2) Modal damping

From the damping waveform obtained from the vehicle running test, each vibration mode was evaluated according to a method similar to the case of vehicle drop test. Modal damping is shown in Table 1-12.

Table 1-12 Modal damping

(a) Torsion symmetric primary mode

A3 standard		Natural frequency (Hz)	Logarithmic decrement	Damping constant
Average	20km/h	1.37	0.027	0.004
	40km/h	1.39	0.028	0.005
	60km/h	1.39	0.031	0.005
Total average		1.38	0.029	0.005

(b) Torsion antisymmetric primary mode

A2 standard		Natural frequency (Hz)	Logarithmic decrement	Damping constant
Average	20km/h	2.84	0.034	0.005
	40km/h	2.85	0.037	0.006
	60km/h	2.85	0.041	0.006
Total average		2.85	0.037	0.006

(c) Horizontal symmetric primary mode

A9 standard		Natural frequency (Hz)	Logarithmic decrement	Damping constant
Average	20km/h	1.96	0.028	0.004
	40km/h	1.95	0.030	0.005
	60km/h	1.95	0.032	0.005
Total average		1.95	0.030	0.005

The damping factors determined from the vehicle running test are about 3 – 4% in logarithmic decrement and about 0.5 – 0.6% in damping constant, which are approximately similar to those of the vehicle drop test. It is also observed that the damping factors tend to increase with running speed. Since the response increases with increasing running speed (see (3) Maximum response), this can be interpreted as indicating the so-called amplitude dependence of damping.

(3) Maximum response

Table 1-13 shows the maximum response as measured vertically at each measuring point. Table 1-14 shows it as measured in the torsion direction. Speed and displacement responses were determined through two-order integration of acceleration data.

Table 1-13 Maximum response of vehicle running test (vertical)

Response	Case (km/h)	Vertical・bridge surface						Vertical・pier		Horizontal
		A1	A2	A3	A6	A4	A5	A7	A8	A9
Acceleration (gal)	20	9.81	14.28	10.01	8.61	14.27	11.09	0.28	0.65	1.14
	40	19.63	28.41	11.21	13.24	16.52	16.33	0.87	1.24	3.48
	60	26.03	38.38	24.96	23.44	34.66	26.70	0.94	1.43	5.21
Speed (cm/s)	20	0.246	0.680	0.291	0.308	0.695	0.228	0.008	0.017	0.061
	40	0.238	0.534	0.266	0.280	0.545	0.279	0.009	0.014	0.135
	60	0.375	1.095	0.715	0.530	1.331	0.416	0.011	0.022	0.188
Displacement (㎜)	20	0.116	0.370	0.178	0.176	0.369	0.124	0.004	0.007	0.055
	40	0.108	0.328	0.237	0.217	0.340	0.100	0.003	0.007	0.120
	60	0.202	0.599	0.464	0.351	0.713	0.197	0.005	0.012	0.157

From Table 1-13, it is known that, similar to the case of vehicle drop test, the response on the pier is much smaller than the response on the bridge surface. Namely, it is presumed that vehicle running has not caused deflection of piers.

Table 1-14 Maximum response of vehicle running test（Measured in torsion direction）

Response	Case (km/h)	Vertical・bridge surface						Horizontal		
		A5	A1	A4	A3	A6	A2	A7	A8	A9
Acceleration (gal)	20	10.72	15.23	15.40	7.71	13.96	16.64	2.65	2.38	1.48
	40	21.23	14.38	16.46	10.75	11.51	17.02	5.02	4.52	3.31
	60	38.39	20.69	22.39	10.93	16.31	22.13	7.22	4.48	3.31
Speed (cm/s)	20	0.267	0.781	0.793	0.265	0.470	0.753	0.076	0.113	0.066
	40	0.312	0.683	0.679	0.382	0.448	0.688	0.061	0.102	0.129
	60	0.453	1.096	1.068	0.472	0.438	1.156	1.156	0.118	0.097
Displacement (㎜)	20	0.135	0.402	0.405	0.165	0.226	0.400	0.027	0.057	0.059
	40	0.120	0.394	0.393	0.233	0.256	0.392	0.020	0.069	0.092
	60	0.222	0.681	0.650	0.381	0.408	0.686	0.019	0.074	0.084

Fig. 1-13 shows relationship between vehicle running speed and maximum response. The abscissa indicates actual running speed, the ordinate indicates the maximum response of acceleration, speed, and displacement.

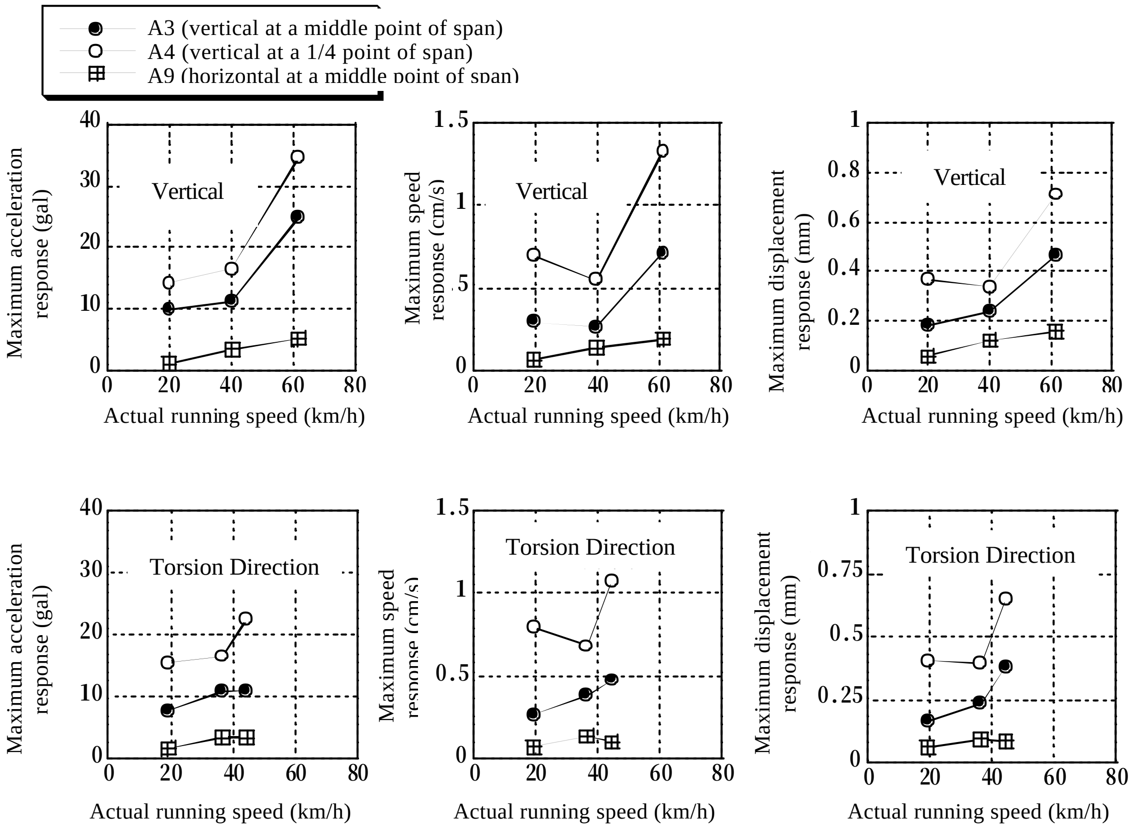


Fig. 1-13 Relationship between vehicle running speed and maximum response

From Fig. 1-13, it is evident that the response tends to increase with increasing running speed for both torsion and horizontal vibrations. In the case of torsion vibration, the response was higher at a 1/4 point of span than at the middle point of span. This means that the bridge tends to cause the torsion antisymmetric primary mode more readily than the torsion symmetric primary vibration in the service condition, that is, when vehicles are running. In other words, the bridge body develops resonance with ease because the natural frequency in the torsion asymmetric primary mode is about 2.9 Hz, which is rather similar to the approximate 3.2 Hz of the test vehicle or the vehicle non-suspended natural frequency that is said to exert a substantial effect on torsion vibration of bridges. This in turn is considered to excite the torsion antisymmetric primary mode. The spectrum diagram shows the dominant frequency at about 3 Hz. This trend is not much detected in microtremor measurement. It may be that the external forces caused by running test vehicles contains many frequency components of about 3 Hz that are equivalent to the natural frequency of vehicles.

1.4 Reverse Analysis

1.4.1 Analysis model

This vibration test was made mainly for torsion vibration of the superstructure. This bridge is a three-span continuous prestressed concrete box-girder bridge. The pier height is only 7.2 m. Generally, vibration characteristics of bridge superstructures, specifically in the torsion direction are substantially affected by the main members, type of structures, rigidity, and weight of superstructure. Effects of substructures such as piers and ground spring are considered small. Accordingly, an analysis model was designed as a three-dimensional frame model that represents only the superstructure, as shown in Fig. 1-14. In this case, the longitudinal alignment and pre-stressing in the bridge axis direction are not taken into account. The eigenvalue analysis was made using a general-purpose FEM program, ADINA, ver. 6.1.3.

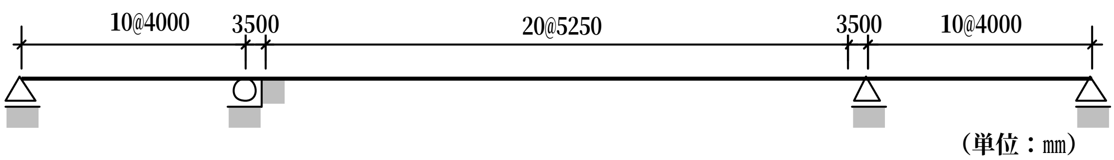


Fig. 1-14 Analysis model

1.4.2 Analysis conditions

1) Boundary conditions

Rotation of abutment and pier was not constrained.

2) Sectional rigidity

The concrete was assumed to be effective in the entire section. Prestressing steel was considered as converted to the concrete section, and steel bars were ignored. The rigidity of each element was the average of sectional rigidities at nodes. Note that railings, curb, and pavement were not taken into account.

The following three values were used for elastic modulus Ec:

- Case-1： Value based on the elastic modulus test result with concrete cores sampled this time from the superstructure（$Ec=23kN/mm^2$）
- Case-2： Design value（$Ec=28kN/mm^2$）
- Case-3： Value estimated from the concrete strength（$Ec=33kN/mm^2$）

Note that the Poisson's ratio was 0.17.

3) Mass

The mass was the concentrated mass at each node and calculated based on the unit weight of $2.5t/m^3$. Weight of railings, curb, and pavement was not taken into account.

4) Analysis method

For the model linearized in the initial state (zero hour), the natural frequency was determined according to the sub-space iteration method.

1.4.3 Analytical result

(1) Natural frequency

The analytical result is shown in Table 1-15 and comparison between test and analysis values shown in Fig. 1-15.

Table 1-15 Analytical result

Vibration mode	Natural frequency(Hz)			
	Test value	Analysis value		
		Ec=23kN/mm2	Ec=28kN/mm2	Ec=33kN/mm2
Torsion symmetric primary mode	1.30	1.19	1.31	1.43
Torsion antisymmetric primary mode	2.86	2.76	3.04	3.31
Horizontal symmetric primary mode	1.91	2.00	2.21	2.40

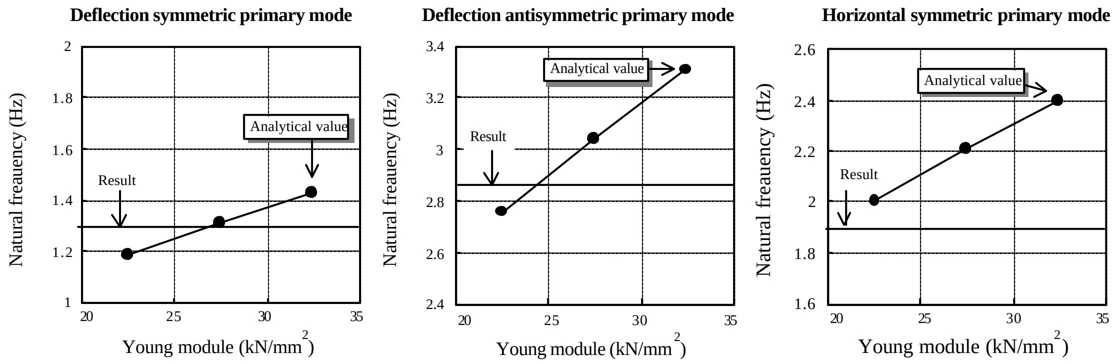


Fig. 1-15 Comparison between test and analysis values

From Fig. 1-15, it is evident that the natural frequency of torsion vibration falls between the analysis value based on the elastic modulus test value and the design value. As regards the natural frequency of horizontal vibration, the analysis value was higher than the test value in all cases. This may be attributed to the fact that the analysis model did not consider the substructure, resulting in evaluation of the structure more rigid.

At present, a soundness evaluation of the bridge by means of vibration tests is under way mainly on the substructure. [3)] As an example of test on the superstructure, Shimada, et al reported the case with a two-span continuous prestressed concrete box-girder rigid-frame road bridge with the span length of 90 m[4)]. According to this report, the natural frequency of torsion vibration decreased by 5 – 10% in 11 years, but without any problem in terms of soundness. As no measurement data is available on the initial condition (immediately after construction) for this bridge, the secular change of natural frequency is not definite. The test value on torsion vibration is 1 – 6% lower than the natural frequency when the elastic modulus was used as the design value. From comparison between test and analysis values and the above report example, it may be surmised that this is within the range presumed from the vibration test result, namely, that this bridge has not developed remarkable rigidity deterioration.

(2) Vibration mode

The vibration mode obtained from analysis is shown in Fig. 1-16. The mode shape is similar to the test value (Figs. 1-8, -10, and -12), and thus the analysis model is considered to be approximately adequate.

Torsion symmetric primary mode

Torsion antisymmetric primary mode

Horizontal symmetric primary mode

Fig. 1-16 Vibration mode diagram (analytical result)

1.5 Summary

The findings from vibration tests are summarized below.

① The natural frequency of this bridge was 1.30 Hz for the torsion symmetric primary vibration, 2.86 Hz for the torsion antisymmetric primary mode, and 1.91 Hz for the horizontal symmetric primary mode.

② The modal damping was about 0.6% of the damping constant, regardless of the vibration mode, for torsion and horizontal vibrations. This value is nearly equivalent to those observed with other prestressed concrete box-girder bridges.

③ The result of analysis of the eigenvalue using elastic modulus as a parameter shows that the natural frequency of torsion vibration falls between the analysis value based on elastic modulus test value and that based on the design value.

④ From the results of the vehicle drop and vehicle running tests, it is evident that the vertical vibration of piers is much smaller than the response level of the superstructure.

⑤ From the vehicle running test result, it is observed that the response of acceleration, speed, and displacement tends to increase with vehicle running speed.

For this bridge, evaluation on the soundness of present condition was attempted through comparison between the analysis value using the elastic modulus estimated from the concrete elastic modulus test result, design value, and concrete strength of the superstructure and the test value. This was done because the data on the initial condition of the bridge was not available. In the future, the measurement data obtained this time may be used as reference data for evaluation of the effectiveness and ageing after rehabilitation and reinforcement if the same level of measurement is made after rehabilitation and reinforcement or as a part of maintenance to grasp changes in natural frequency and response.

《Literature》

1) Japan Society of Civil Engineers： "Vibration Handbook for Civil Engineers," October 1985, pp.101～104
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