6.4 Assessment of Preliminary Inspection Results

The purposes of the assessment are to study the general tendency of the damage observed in each bridge member based on the results of the damage condition rating and to assist in the derivation of possible rehabilitation works.

A summary of assessment of preliminary inspection results from a structural viewpoint is tabulated in Table 6.8.

Table 6.8 Summary of Assessment of Preliminary Inspection Results

Main Bridge Member	Dominant Damage Detected	Cause of Damage	Possible Rehabilitation Plan
Steel Girder Steel Truss	Corrosion	- Poor maintenance - Water leak no weep hole	- Repainting - Making weep hole - Partial replacement - Cantilever slab
Steel Buckle Plate or Corrugated Plate Slab	Corrosion	- Water leak - Poor maintenance	- Repainting - Redecking to RC slab
RCB	Flaking/Rebar exposure	- Inadequate cover - Poor workmanship	- Injection - Patching - Prepacked concrete
PSC/PRE	Flaking/PC tendon exposure	- Inadequate cover - Poor workmanship	- Patching
Abutment	Crack Wear abrasion Scouring	- Loose wedge stone - Non protection - Non masonry	- Injection - Cement grouting
Pier	Crack Wear abrasion Scouring	- Loose wedge stone - Non protection - Non masonry	- Protect concrete - Stone masonry - Mat gabion

CHAPTER 7 SELECTION OF BRIDGES FOR DETAILED SURVEY

7.1 General

The main purposes of this chapter are to select 10 typical bridges out of the 100 bridges shown in the Bridge Inventory made based on the Preliminary Bridge Inspection and to select 3 bridges out of those 10 bridges for a static loading test.

The detailed survey results on the 10 bridges were used to carry out preliminary rehabilitation design of these bridges and to reflect in formulating the maintenance and rehabilitation program of the remaining study bridges as well as in preparation of the inspection, maintenance, and rehabilitation guideline, while the primary purpose of loading test is to estimate structural loading capacity of main beam which could be used in preliminary structural analysis.

7.2 Selection of 10 Bridges for Detailed Survey

7.2.1 Selection Procedure

10 bridges for the Detailed Survey shall be selected from the Bridge Inventory made based on the Preliminary Bridge Inspection. The following 3 items were considered in the selection.

1) Various rehabilitation methods considered from damage/defect shall be selected as much as possible.
2) Bridges shall be selected, which have enough design data/information such as drawings, design condition, material strength, etc.
3) Bridges shall be selected, which have high priority for their rehabilitation.

However, difficulty has been found on item 2), because most of bridges from the list of 206 bridges have almost no data/information except 15 bridges which even have a little information. Therefore, 1) and 3) shall be mainly considered to select 10 bridges as follows:

<u>for item 1)</u>

Typical type of bridges in their structural aspects shall be selected among existing bridges, and various rehabilitation methods shall also be considered in the selection of 10 bridges.

<u>for item 3)</u>

Priority of road rehabilitation shows its necessity based on its functional aspect. Taking it in consideration, a graph of which X axis is for bridge damage degree, and Y axis is for road rehabilitation priority shall be made for determination of bridge rehabilitation priority.

7.2.2 Selection Based on Bridge Types and Rehabilitation Method

After the Preliminary Bridge Inspection, the Study Team has made a presentation towards the RDA regarding selection principles and bridges to be detailed-surveyed, which the Study Team had selected. Selection principles are as follows:

(1) Condition for Selection

1. Structural type with capability of widening
2. Structural type with capability of redecking
3. Member with capability of rehabilitation
4. Structural type with capability of adding footway
5. Bridges adjacent to the city of Colombo

(2) Typical Bridge Type

1. RSJ / BUC	Main girder : RSJ, Deck slab : BUC
2. RSJ / COR	Main girder : RSJ, Deck slab : COR
3. RSJ / RCS	Main girder : RSJ, Deck slab : RCS
4. ST.TR / T	Steel through truss bridge (incl. half-through) Deck slab : BUC, COR, RCS
5. ARCH / BR	Brick arch bridge
6. ARCH / ST	Stone arch bridge
7. PSC / PRE	Prestressed pretensioned concrete beam bridge
8. RCB	Reinforced concrete beam bridge
9. CAUSEWAY	Causeway bridge

(3) Selection of Bridges for Detailed - Surveyed

The following bridges have been selected based on the results of visual inspection, (1) and (2) above.

1. SER 85 (ARCH / BR)
   This bridge is a brick arch bridge constructed in 1984. Although width of carriageway (2-lane) is sufficient, footway width needs to be widened because of security for pedestrians. Since the bridge is quite aesthetic, consideration should be given for its widening in this aspect.

2. SER 1 (RCB)
   This bridge locates on the AA002 route along the coast, so salt injury is easy to affect on the bridge. Results of the Preliminary Bridge Inspection say that main girder is damaged badly, and main reinforcing bars and stirrup are broken. For the rehabilitation of the bridge, appropriate countermeasure against salt injury has to be taken, so that detailed survey is needed to know a degree of the damage. Substructure is well enough to be used.

3. SER 27 (ARCH / ST)

This bridge is a stone arch bridge locating near sea. Although arch abutments are damaged, it is possible to be repaired. As natural stone is used for the bridge, it is quite aesthetic. Therefore, where footway is added, it should be at land side, between the bridge and a railway bridge.

4. SER 77 (ST. TR / T + RSJ / BUC)

This bridge crosses Maha River, and it is a long bridge which consists of ST. TR / T for center span and RSJ / BUC for both side spans. Although widening of its width is needed, consideration against its compound structural style has to be taken. Further survey shall be carried out including studying of widening method.

5. SER 53 (ST. TR / T)

This bridge was constructed in 1899. It has narrow width which causes a difficulty for heavy vehicles to cross on. Although footway made of timber is added outside of main truss beam, it is terribly damaged and nobody crosses it. For the main truss beam, part repair shall be enough to recover its function, however, a new bridge should be constructed next to the bridge if widening is taken place. There are many bridges of this type in Sri Lanka, therefore, it is chosen for the detailed survey.

6. SER 33 (ST. TR / T + RSJ / RCS)

This bridge was mentioned by the Preparatory Study Team and it consists of ST. TR / T for center span and RSJ / RCS for both side spans. Rehabilitation is hoped for its narrow width (3.3m) and remarkable deflection.

7. SER 59 (RSJ / BUC)

This bridge is one of the most common type of bridge inspected for the Study and is also suitable bridge to do Widening Planning and Rehabilitation Planning. Traffic volume is not so big comparatively, so that the bridge is appropriate for loading test.

8. SER 20 (RSJ / COR)

This is the most simple structure type of bridge which COR plate is placed on RSJ girder, and concrete deck slab is on the COR plate. Outside girder of the bridge is badly damaged as the same as RSJ / BUC bridge, however, inside girder is usable. The bridge is a typical one for its rehabilitation method which RSJ girder is covered with concrete and reinforcing bars, and its deck slab is replaced with RCS.

9. SER 70 (RSJ / RCS)

This bridge is a RSJ girder bridge with RC structure of its deck slab. It is possible to widen its superstructure without any reinforcement of substructure. Therefore, it is selected.

10. SER 139 (CAUSEWAY - RCS)
This type of bridge is submerged during flood season, and its superstructure is RC slab. Detailed survey shall be implemented to plan a reconstruction of a new bridge considering that new bridge surface level would be above H.F.L.

11. SER 7 (RCB + PSC / PRE)
This is a compound bridge of RCB and PSC and crosses Negombo lagoon. Terrible salt injury is found in the superstructure. Substructure is pile bent type, and their damages are not so big. Since this crosses lagoon, detour route shall be a problem during rehabilitation implementation. Many kinds of countermeasure to avoid the problems shall be studied in the detailed survey.

12. SER 211 (RSJ / RCS)
This bridge is a RSJ girder bridge with its deck slab of RC structure. Piers consists of 8 columns, and they are RC structure. Widening is needed due to its narrow width, therefore, the bridge shall be selected to the detailed survey as a representative type of widening for this kind of one.

13. SER 212 (PSC / PRE)
This bridge is a quite new bridge constructed in 1975. It locates adjacent mouth of river. So main girder is damaged quite badly for salt injury. However, remarkable salt injury is not observed on abutments and piers. This occurrence which is inconsistency has to be researched in the detailed survey.

Table 7. 1 shows the summary of above assessment results.

(4) Result of Meeting with RDA

RDA has made a comment on 13 bridges selected by the Study Team shown in the Table 7.1. Details are as follows:

- Bridges which RDA agrees, out of 13 bridges,

No. 85	No. 70	No. 77	No. 7
No. 53	No. 211	No. 33	No. 212
No. 59	No. 20		

Table 7.1 List of Bridges for Detailed Inspection

SER No.	Route No.	Bridge No.	Traffic Volume	Year of Const.	Type of Bridge	Length (m)	Width (m)	Treatment	RDA's List	
									Existing Defect/s	Proposed Treatment
85	AA001	91/2K	7,100	1894	ARCH/BR	68.90	6.30	Extension of footway at both sides	Narrow, Poor	RECONST/APP
1	AA002	98/1K	6,700	-	RCB	30.00	7.80	Repairing of main beam or Reconstruction of superstructure	Narrow, Damaged	WIDEN/REDECK
27	AA002	87/1K	6,700	1898	ARCH/ST	35.20	6.80	Repairing of abutment and Construction of pedestrian bridge	Weak/Narrow	RECONST
77	AA019	3/2K	2,100	1869	ST.TR/T+RSJ/BUC	118.88	5.36	Construction of additional bridge (for widening)	Narrow	WIDEN/REDECK
53	AA021	36/3K	700	1899	ST.TR/T	39.95	3.37	Repairing of existing truss beam and Construction of additional bridge	Narrow	WIDEN/REDECK
33	B 157	12/3K	1,040		ST.TR/T+RSJ/RCS	68.85	3.30	Repairing of existing truss beam and Construction of additional bridge	Weak/Narrow	RECONST
59	B 157	43/4K	1,040	1924	RSJ/BUC	51.00	3.20	Extension of superstructure with re-decking + Extension of substructure (BUC to RCS)	Narrow	RECONST
20	B 264	16/6K	1,700		RSJ/COR	14.35	3.97	Repairing of outside main beam and redecking	Narrow	RECONST
70	B 295	3/6K	10,700	1960	RSJ/RCS	43.23	5.46	Extension of superstructure	Narrow	RECONST
139	B 312	25/1K	430	1880	CAUSEWAY	67.50	3.30	Reconstruction of bridge	Weak/Narrow	RECONST
7	B 425	20/4K	1,600		PSC/PE+PCB	139.18	5.85	replacement of superstructures	Weak	WIDEN/REDECK
211	B 146	8/3K	2,800	1942	RSJ/RCS	23.60	3.55	Extension of superstructure and substructure		
212	AA002	138/1K	4,400	1975	PSC/PRE	62.48	10.40	Repairing of superstructure		

- Bridges which RDA has already implemented their rehabilitation or will be implemented within this year

  No. 1 No. 27 No. 139

- Although the Study Team proposed the loading test should be carried out on No. 212 (AA002 138/1km PSC / PRE) and No. 59 (B 157 43/4km RSJ / BUC), both bridges cannot be closed due to their heavy traffic volume. However, since RDA has an axle weight scale, it is possible to close the bridge for a while and to implement the loading test.

7.2.3 Selection Based on Function of Road and Damage Degree of Bridges

The connection between function of roads and damage degree of 101 bridges are shown in Table 7.2. The spot traffic volume is assumed that it is a function of road.

The bridges to be detailed-surveyed are shown in the Table. The Study Team considers that the bridges selected shall not be changed. Because item 1) shall be considered the first although the bridges were not satisfactory the condition of item 3) mentioned in 7.2.1.

Table 7.2 List of Bridge Rehabilitation by Priority

Function of Road		Damage Degree of Bridge				
Roads by Priority	Traffic Volume	4.0	3.2	3.0	2.4 to 2.0	under 2.0
1 st	Over 5,000 veh./day	1, 175	-	27, 66, (70), 75, 108, 120, 197	76, 79, 84, (85), 99, 195, 201	-
2 nd	more than 3,000 veh./day	32, 86, 202, (212)	119	17, 47, 93, 102, 123, 151, 154	36	46, 106
3 rd	more than 2,000 veh./day	91	78, 80	52, 65, (77), 89, 147, 148, 173, 209,	138, (211), 216	-
4 th	more than 1,000 veh./day	(7), 18, (20),(33), 129 150	34, 40, 42, 44, 87, 178	19, 26, 30, 39, 57, 131, 135, 136	2, 43, 45, 58, (59), 103, 130	-
5 th	less than 1,000 veh./day	31, 35, 38, 61, 62, 63, 68, 72, 122, 128, 144, 208	21,24,55,56, 74,127,133	25, 41, 67, 69,	(53), 60	22, 71, 73
undergoing or planned		139				
Sub-Total		25	16	34	20	5

( ) shows the bridges to be detailed-surveyed.

7.3 Selection of Bridges for Loading Test

7.3.1 Selection Criteria

In order to select 3 bridges for the loading test out of the 10 bridges selected for detailed survey, the 10 bridges were evaluated from the following viewpoint.

(1) Applicability of the Test Results

It is essential that the loading test results of a bridge should reflect the load carrying capacity of the other same bridge types. A typical type of bridge in Sri Lanka can be expressed as follows;

Old type bridge : steel girder or truss bridge which was made from wrough iron or mild steel

New type bridge : prestressed (pretension) concrete slab

And a representative bridge having standard bridge dimension and no critical structural defects shall be selected as the bridge for the loading test.

The results of evaluation on the applicability is shown in Table 7.3

Table 7.3 Applicability of the Test Results

SER No.	Year of Cons.		Drawing		Type of Bridges		Structural Defects (Rating)			
							Super Structure	Sub Structure		
85	1894	○	-		ARCH/BR		2.0	2.0	○	2
77	1869	○	-		ST.TR/T+RSJ/BUC		3.0	2.0		1
53	1899	○	-		ST.TR/T		2.0	1.0	○	2
33	-		-		ST.TR/T+RSJ/RCS		4.0	2.0		0
59	1924	○	-		RSJ/BUC	○	2.0	2.0	○	3
20	-		-		RSJ/COR	○	4.0	2.0		1
70	1960	○	-		RSJ/RCS	○	2.0	3.0		2
7	-		-		PSC/PRE+PCB	○	4.0	3.0		1
211	1942	○	-		RSJ/RCS	○	2.0	2.0	○	3
212	1975	○	○	○	PSC/PRE	○	4.0	3.0		3

(2) Clearance of Working Space

As a workability of the loading test, clearance between the soffit of girder/slab up to water or ground level shall be considered. Evaluation results are shown in Table 7.4.

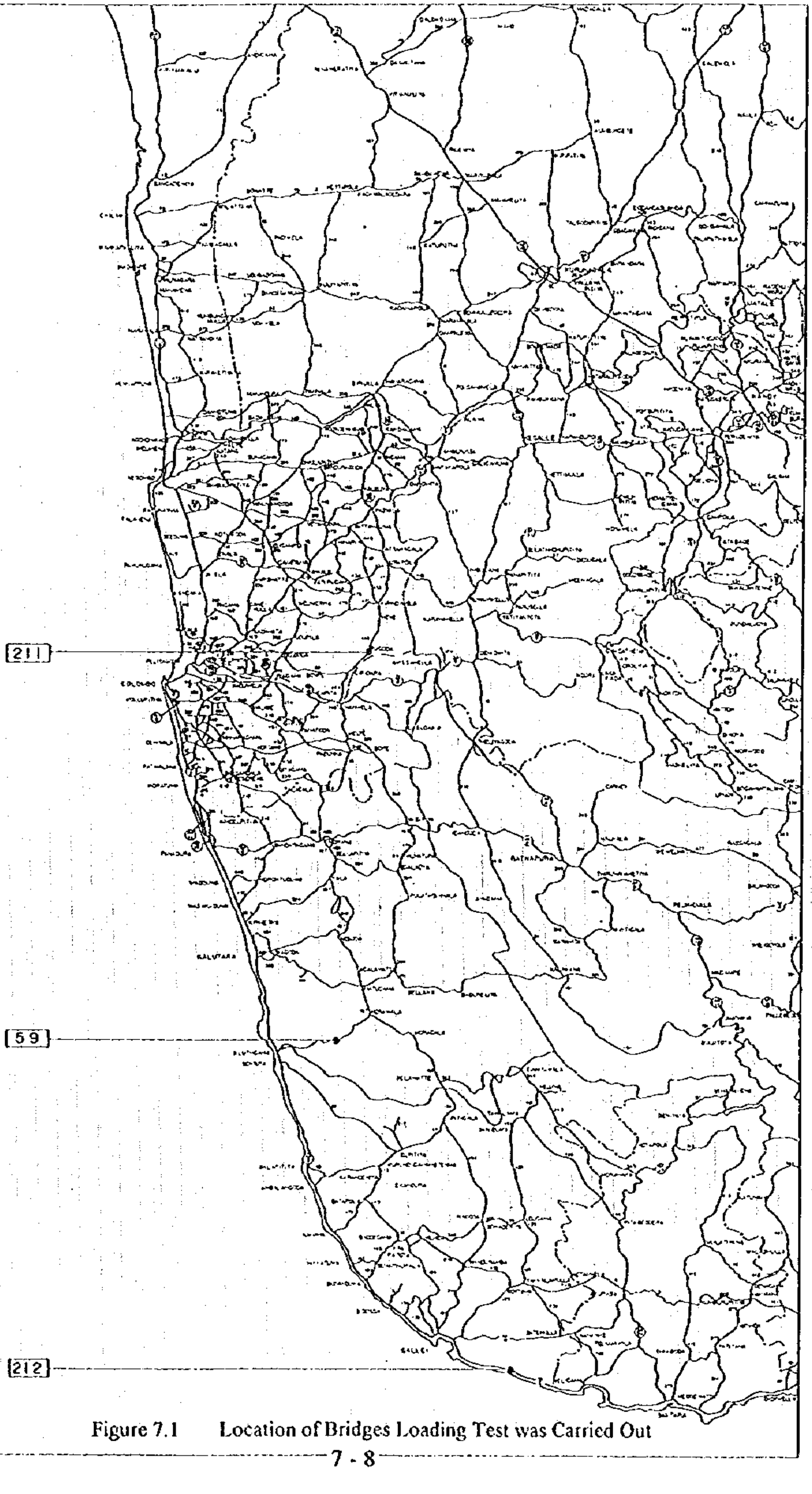


Figure 7.1 Location of Bridges Loading Test was Carried Out

Table 7.4 Clearance of Working Space

SER No.	Height of Clearance (m)		Working Condition place	water		Evaluation
85	11.0	1	river	○	1	1
77	4.5	1	ground	-	3	2
53	7.0	1	river	○	1	1
33	4.5	1	river	○	1	1
59	1.8	3	river & ground	○	2	2.5
20	3.3	2	ground	-	3	2.5
70	1.8	3	river & ground	○	1	2
7	1.4	3	lagoon	○	1	2
211	2.5	2	ground	-	3	2.5
212	1.5	3	sea shore	-	3	3

1.0 > Height ···2
1.0 < H < 2.0 ···3
2.0 < H < 4.0 ···2
4.0 < H ···1

(3) Traffic Volume

Traffic volume on the bridge to be selected shows the degree of difficulty of traffic control during the loading test. The traffic volume of 10 bridges is as follows:

Table 7.5 Traffic Volume on 10 Bridges

SER No.	Route No.	Bridge No.	Traffic Volume	Evaluation
85	AA001	91/2 K	7,100	1
77	AA019	3/2 K	2,100	3
53	AA021	36/3 K	700	3
33	B157	12/3 K	1,040	3
59	B157	43/4 K	1,040	3
20	B264	16/6 K	1,700	3
70	B295	3/6 K	10,700	1
7	B425	20/4 K	1,600	3
211	B146	8/3 K	2,800	3
212	AA002	138/1 K	4,400	2

0 < VPD < 3000 ···3
3000 < VPD < 6000 ···2
6000 < VPD ···1

7.3.2 Selection Results of the Bridge for Loading Tests

In order to select the bridges for a loading test based on the quantitative evaluation of the 3 bridges, the basic point for each of the 4 evaluation items per bridge was then added to obtain the total points for the bridge. Although the selection results are shown in Table 7.6, the obtained total points are not of an exact order, they do show the general order in which the bridges fall concerning the appropriateness for a loading test.

Table 7.6 Evaluation Results and Selection of Bridges

SER No.	Applicability	Workability	Traffic Volume	Total	Evaluation
85	2	1	1	4	
77	1	2	3	6	
53	2	1	3	6	
33	0	1	3	4	
59	3	2.5	3	8.5	○
20	1	2.5	3	6.5	
70	2	2	1	5	
7	1	2	3	6	
211	3	2.5	3	8.5	○
212	3	3	2	8	○

CHAPTER 8 TOPOGRAPHIC SURVEY

8.1 General

The topographic survey was carried out to obtain data and information of any condition at location of 10 representative bridges for preparing Bridge Rehabilitation Plan.

The survey was carried out under the following conditions:-

- The RDA did not use National Coordinates, therefore, Arbitrary Coordinates and Meridian was used.
- Temporary bench marks were set near bridge site.
- Land registration map was not existed in Sri Lanka, so that survey maps were prepared by using RDA pegs or any structure such as fences in order to clarify boundaries.

The location map and the range of the survey are given in Figure 8.1 and Table 8.1 respectively.

Hydraulic study was carried out on 9 bridges using the river cross section obtained from the topographic survey. The results of the survey is described in Section 8.5.

8.2 Objectives

The principle objectives of the topographic survey are to visualize and to illustrate the surrounding land, structures and river relative to the bridge. This topographic survey is essential since it provides the basic data for the following works:-

- Hydraulic calculations
- Determination of bridge scale for reconstruction plan
- Structural analysis for substructures

8.3 Methodology

The topographic survey work was sublet to a local survey consultant and commenced on 24th October, 1995 for a period of 30 days. Five surveying parties were mobilized to the survey works at the respective sites. The survey works were divided into reconnaissance, horizontal survey, vertical survey, setting of bench marks, plan work, river cross sections and bore holes survey for soil investigation.

8.3.1 Reconnaissance

Prior to the actual field work, a reconnaissance was carried out to confirm the exact location of the respective bridge sites and to confirm the range of survey determined in the specification prepared by the Study Team.

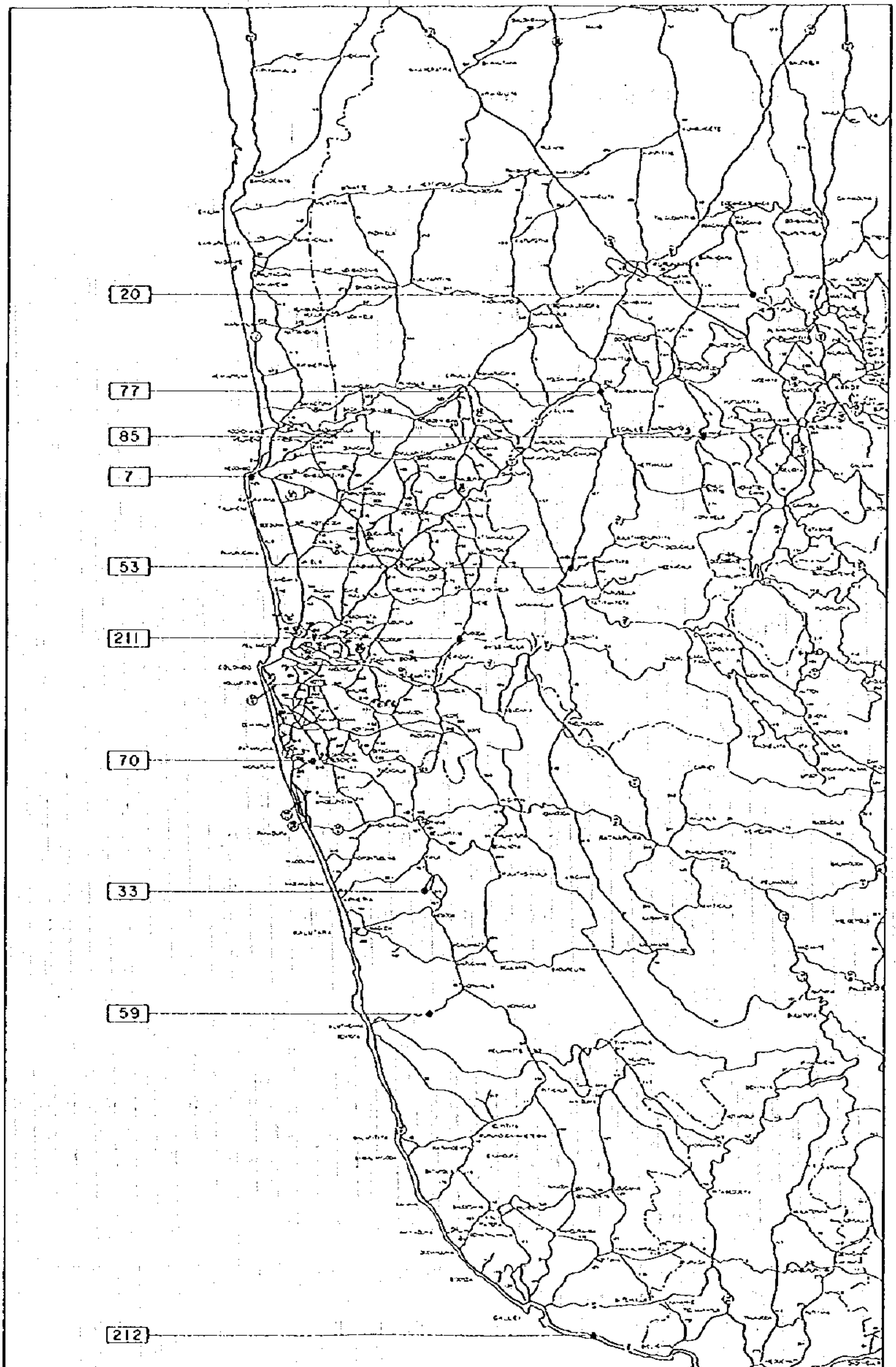


Figure 8.1 Location of Bridges for Topographic Survey

Table 8.1 Survey Area

Bridge SER No.	Bridge Length Exist. (m)	Topografhic and Utility Survey						River Cross Section Survey		Remark
		Abut to origin (m)	Abut to destination (m)	CL to Left (m)	CL to Right (m)	Area to be Covered		Length (m)	Nos.	
						L*W	=A(m2)			
85	69.30	50	50	20	30	200*50	10,000	100	3	
77	120.26	200	200	50	50	520*100	52,000	220	3	
53	38.50	200	200	50	50	440*100	44,000	140	3	
33	68.95	200	200	50	50	480*100	48,000	170	3	
59	46.22	200	200	50	50	440*100	44,000	150	3	
20	14.35	200	200	40	60	440*100	44,000	120	3	
70	43.10	200	200	50	50	440*100	44,000	140	3	
7	139.75	200	200	50	50	560*100	56,000	240	3	
211	24.00	200	200	50	50	440*100	44,000	120	3	
212	49.45	35	35	20	30	120*50	6,000	100	3	
Total							392,000		30	

8.3.2 Horizontal Survey

Horizontal survey was based on Arbitrary (assumed) Coordinates and Meridian. The two ends of each bridge centre line (each being at the abutment position) were marked, and this center line was assigned 90 degrees Azimuth from origin end to destination end. The origin end point of this was assigned 500,000N and 500,000E Cordinate value.

8.3.3 Vertical Survey

Elevations for vertical survey was basically based on Mean Sea Level (MSL) datum. This could be fulfilled by connecting the levelling at each site to a Bench Mark (BM) established by the Sri Lanka Survey Department (SD) which was based on MSL datum or to any other known BM having MSL values.

Location and elevations of Survey Department Bench Mark (SDBM) could be obtained from the SD divisional survey offices in the respective area and/or geodetic unit at the institute of surveying and mapping at Diyatalawa. Relevant information of all old SDBM in record within radius 4.5 km from the site were obtained. Thereafter, extensive search was carried out in the field to locate and reopen these old BMs.

From the investigations and searches made, it was able to reopen SDBM and connect the levelling to MSL datum at 8 bridge sites out of 10. In the case of SER No. 53 all old BMs searched were not available as they may have got destroyed due to new development and widening of roads. In the case of SER No. 20 there were no record of SDBM within 4-5 km from the site. Hence assumed datum level (ADL) were provided for these 2 bridge sites.

Reference to MSL DATUM is attached in Appendix - F.

Accordingly, all maps and drawings were endorsed referring the datum used in the site.

Sites where elevations are based on MSL datum, fore and back levelling was carried out between the newly established temporary bench mark (TBM) and the old BM reopened for the purpose. Fore and back levellings were carried out so as to maintein the required accuracy.

Levelling $\leq$ 2cm $\sqrt{s}$ (S means survey single distance in km)

On the other hand, low water ordinary spring tide (LWOST) has been used for the relevant organization of port such as tide table. This can be converted to MSL as follows:-

MSL = LWOST + 0.460m

8.3.4 Bench Marks

New TBMs were constructed at the locations of either side of each bridge, and the levels for the TBMs at the respective bridge site are given in Table 8.2.

Table 8.2 TBM Levels and Referenced Bench Marks

SER No.	Route No.	Bridge No.	BM No.	TBM Elevation (m)	Datum
85	AA001	91/2km	G1	203.185	MSL
			G2	202.953	MSL
77	AA019	3/2km	G1	69.109	MSL
			G2	68.968	MSL
53	AA021	36/3km	G1	27.948	ADL
			G2	28.156	ADL
33	B157	12/3km	G1	8.076	MSL
			G2	8.188	MSL
59	B157	43/4km	G1	3.238	MSL
			G2	3.374	MSL
20	B264	25/7km	G1	150.000	ADL
			G2	150.691	ADL
70	B295	3/6km	G1	3.771	MSL
			G2	3.661	MSL
7	B425	20/4km	G1	2.528	MSL
			G2	2.543	MSL
211	B146	8/3km	G1	11.889	MSL
			G2	11.927	MSL
212	AA002	138/1km	G1	2.643	MSL
			G2	2.853	MSL

8.3.5 Plan Work

Trigometrical levelling survey was carried out using EDM to ascertain the ajimuth and distance of each point in detail including the spot heights.

Topographical plans of bridges, SER No. 77, 53, 33, 70, and 7 were computer plotted by AUTOCAD using both DOS and WINDOW versions based on the field data obtained by the Trigometric levelling survey.

All the other topographical plans were plotted and drawn manually.

8.3.6 River Cross Sections

Three cross sections across the rivers were surveyed. One was along the centre line of existing bridge, and the other two were running parallel to the centre line of bridge with 20 meters distance from the centre line.

In most of rivers, their bed elevations were obtained by the same method adopted in levelling for land work. This was done by sprit levelling in some cases and in others by trigometrical levelling.

In bridge sites of SER No. 59, 70, and 7, it was not possible to adopt the same method, therefore such sounding was taken from boat.

8.3.7 Bore Holes

Location of bore holes were pointed out by the Study Team prior to the site survey works during the reconnaissance period. The locations and elevations of these were depicted in the topographic maps.

8.4 Results of Topographic Survey

The results of the survey were prepared in the following drawings:

-	Topographic Maps	1/500	11 sheets
		1/200	11 sheets
-	Center line and profiles		18 sheets
-	Road cross section		28 sheets
-	River cross section		10 sheets

8.5 Hydraulic Study

8.5.1 Objective of the Study

The river hydraulic study was carried out as a part of detailed survey based on the results of preliminary inspection. The main objectives of this survey are as follows:-

- To establish hydraulic rehabilitation plans based on the assessment
- To determine appropriate bridge opening for reconstruction of the bridge

8.5.2 Methodology of Hydraulic Study

The methodology of hydraulic study is divided in to the fold:-

- Field inspection from hydraulic view point
- Hydrological analysis

(1) Field Inspection from Hydraulic View Point

The inspection was carried out on the following items:-

- River water course

- Condition of embankment
- Tendency of river bed
- Local scouring

(2) Method of Hydraulic Analysis

1) Method of Analysis to be Applied

The method of analysis to be applied in the Study was determined in accordance with RDA's practice, considering river condition in Sri Lanka. The maximum discharge which a natural stream is to be designed to pass can be estimated by the following methods:

a) By using one of the empirical formula applicable to the region;
b) By using a rational method involving the rainfall and other characteristics for the Study area;
c) By the area velocity method, using the hydraulic characteristics of the stream such as cross-sectional area and the slope of the stream;
d) From any available records of flood discharges recorded in the RDA inventory or observed at the bridge site or interview from the resident at the site.

The procedure of the water course calculation is shown in Figure 8.2.

2) Empirical Formula Method

Empirical formula for flood discharge from a catchment area is written as follows:

$Q = C * A^n$ (in m^3/s)

A = Catchment Area (km^2)

C = Factor depending on the distance from the point considered to the coast

n = 2/3

Ref : Essentials of Bridge Engineering - D. J. Victor

3) Rational Formula Method

As for rational formula for flood discharge, it is necessary to take into account the intensity, and duration of rainfall as well as the area, shape, slope and permeability of the catchment. Many complicated formula are available in treatises on hydrology. A comprehension formula is adopted in RDA as follows:-

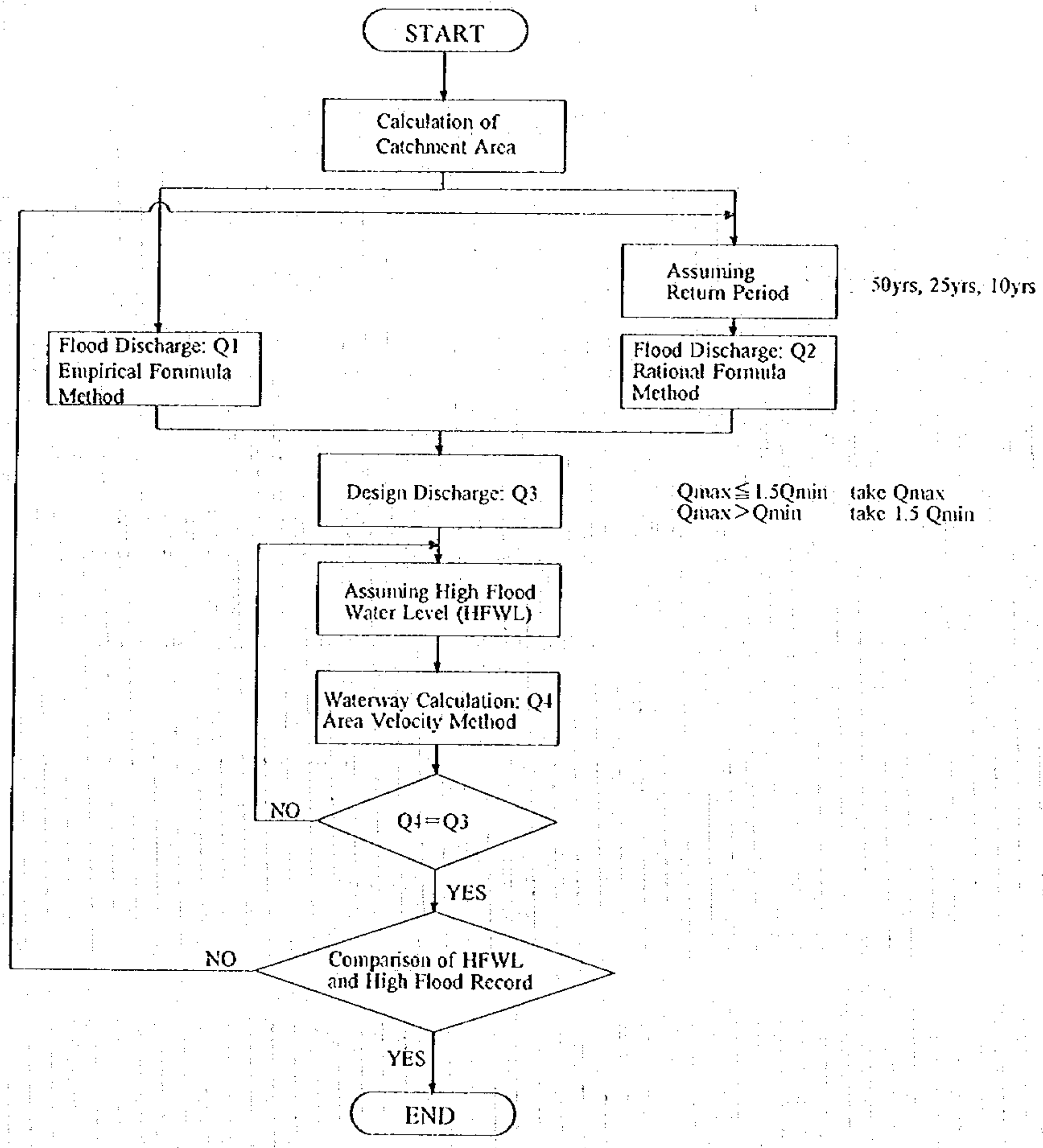


Figure 8.2 Flow Chart of Procedure of Hydraulic Analysis

Discharge $Q = C * I * A$, given in cu.m/sec.

- C : factor depending on the catchment slope
- I : intensity of rain inches/hr.
- A : catchment area in acres

$I = X * D^{-Y}$ - intensity of rain inches/hr.

- D : duration of storm in minutes
- X and Y are constants in relation to the flood return period and the zone in which the site is located.

Duration of storm (D) is calculated with the time of concentration (Tc) value which is given by:

$D = Tc = L/60v + 15$ minutes

- L : longest water course in feet
- V : velocity in ft/s - taken from the table which corresponds to the stream gradient

Ref : Design of Irrigation Headworks for Small Catchment

4) Area Velocity Method

The analysis was carried out to determine the discharge and bridge opening which need to flow the discharge.

Flow velocity is determined by the manning's formula as given by:

$V = 1/n * R^{0.67} * S^{0.5}$ (m/s)

- R : wetted perimeter
- n : manning's coefficient - depends on the type of bed material
- S : bed slope

Discharge is determined by:

$Q = V * A$

- A : area of waterway - taken by the results of river cross section survey at site and floor levels assumed.

5) Record of High Flood Water

Record of high flood water level (RHFWL) was collected by RDA inventory, drawings of existing bridge, flood mark and result of interview from residents at the site. The water levels were converted to MSL or ADL to compare with the result of assuming high flood water level derived from the analysis (DHFL).

8.5.3 Results of Hydraulic Study

(1) Results of Field Inspection

1) River Condition

The mean annual rainfall over Sri Lanka is about 2,000mm which exceeds twice the world average of rainfall.

Ground surface soil is composed of low porosity soils such as laterite. The soil promotes heavy run-off estimated to be as high as 65% for the wet zone. High run-off is promoted by a combination of factors such as steep gradients.

Rivers in the wet zone like Kelani Ganga, Kalu Ganga and Gin Ganga shows steep gradients in their upper river courses and extensive flood plain in the lower courses.

These phenomena sometime cause flood water. Dangerous floods have been recorded on the Kelani at Colombo in October 1913, May 1930, May 1940 and August 1947.

Embankments or flood bunds have been built to provide protection to density populated areas within the flood plains such as a 7km bund of Dutch Canal in Gampaha district in the 18th century and in the city of Colombo which is now going on. Others are remained mostly in natural.

Major bridges whose bridge length is more than about 50 to 60m are constructed over major river, which is called Ganga and Oya in order. Medium to small scale bridges of length less than about 50m are constructed over relatively small catchment river which is called Ela, sometimes they are dried up in dry season. Flood would damage not only to the major bridge but also to the medium to small bridges.

2) Hydraulic Defect

Some minor defect which should be rehabilitated were detected in the course of detailed survey on the 10 bridges.

a) Tendency of River Bed

Generally, river bed tends in sedimentation of soil where the bridges locate in flat area. Therefore, the foundation of substructure are relatively in good condition. In fact, taking river sand by manually was observed in the bridges of SER NO. 77, 53, and 20 near the bridge to take the surplus soil.

However, the results of river cross section survey for SER No. 33 shows the elevation of river bed MSL-8.00. This may be caused by degradation of bed due to river bed errosion. Because very fast velocity (v=4.0m/sec) was shown in the hydraulic calculation as well as observation results in the field inspection. The river should be spanned by a center span (l=50m) in the reconstruction plan. Rehabilitation of the river is out of this Study.

b) Local Scouring

Local scouring was observed in the 3 bridge sites out of the 10 bridges as follows:

- SER No. 212 : exposure of caisson foundation (A2)
- SER No. 77 : exposure of caisson foundation (P8, P9)
- SER No. 211 : exposure of spread foundation (A1)

These defects can be repaired by the concrete protection or reinforced concrete curtain wall to the foundation scoured.

c) Defects on Stone Masonry Wall

In the case of SER No. 7, both sides of stone masonry behind the abutments were seriously damaged. The cause of the defect is poor quality of the structure. The stone masonry together with mat gabions shall be provided to repair the defect.

(2) Results of Hydraulic Analysis

The results of Hydraulic Analysis are summarized in Table 8.3 and the calculation sheets are attached in Appendix - G.

(3) Consideration for Reconstruction Plan from Hydraulic Viewpoint

The following consideration shall be taken when the bridges are planned as reconstruction.

1) SER No. 212

There is a railway bridge which has similar scale at the location of upper stream side of the bridge. Both bridges have almost same elevation. Driftwoods can not impact the road bridge directly when the river flooded because the railway bridge will receive the collision. Therefore, reconstruction plan shall be done along with the existing condition.

Table 8.3 Results of Hydraulic Analysis

SER No.	Name of River	Elevation of Road (m) (1)	Depth of Road to Sofit of Beam (m) (2)	Catch-ment Area (km2)	Design Discharge (m3/s)	Bridge Length (m)	Depth of Water (m)	Elevation of Water (m) (3)	Free Board (m) (1)-(2)-(3)	Velocity (m3/s)	Flood Water Level (Record/Interview) (m)	Remark
85	Ma Oya	MSL 203.32	2.24	176.4	471.8	69.20	2.90	MSL 193.50	7.580	2.46	193.58 (Record)	
212	Goviyapana Ela	MSL 2.90	0.97	41.3	116.4	47.76	1.50	MSL 1.75	0.180	1.62	1.200 (Drawing of existing bridge)	
77	Maha Oya	MSL 68.97	0.56	562.9	1,022.7	118.65	3.60	MSL 65.01	2.400	2.50	68.970 (Interview at site)	
53	Ritigaha Oya	ADL 28.30	0.90	89.8	300.8	36.70	3.60	ADL 23.92	3.480	2.31	29.150 (Record stone at site)	5th Oct. 1913
211	Nikawala Ela	MSL 11.81	0.92	45.9	163.4	21.84	4.40	MSL 10.80	0.090	2.05	14.710 (Trace of flood mark on the wall of building in 1989)	
33	Kalu Ganga	MSL 8.22	0.54	2,624.3	2,425.7	67.48	13.80	MSL 5.80	1.890	4.00	5.885 (Record)	
59	Muhumumalwatta Ela	MSL 3.34	0.65	162.8	380.2	43.65	4.70	MSL 2.30	0.390	2.32	1.450 (Record)	
20	Ogodadola Ela	ADL 150.48	0.52	28.8	141.0	13.42	5.20	ADL 151.70	-1.740	2.03	152.880 (Record)	
70	Bolgoda Lake	MSL 3.75	0.65	54.1	139.4	42.48	2.40	MSL 0.80	2.300	1.44	None	
7	Negombo Lagoon											Omitted

2) SER No. 77

According to the result of interview from residents at the site, the flood water level reached up to the road surface. There is a milk factory (MILKD) adjacent to the origin side of the bridge. If the road elevation is raised to keep the free board (600mm), it will be difficult to go in and out of the factory. Therefore, the soffit of beam of new bridge shall be set with the elevation of existing road surface when reconstruction is planned.

3) SER No. 53

The flood mark stone at the site shows the water reached above 85cm from road surface (5th Oct. 1913). Since the bridge was constructed on the top of vertical alignment with 4% of both approach roads, the elevation of new bridge could not be raised. Both abutments were constructed into the river, so that it becomes the obstacles for fluent water current. In the reconstruction plan, it is recommended that both abutments shall be set back (approx. 4m each). Local scouring was observed around the foundation of A1 abutment. Stone masonry shall be provided around the foundation.

4) SER No. 211

Trace of flood mark was observed on the wall of shop building near the bridge (2.98m above the road surface, recorded in 1989). The cause of this flood is the flood effect of Kelani Ganga which is the main river. On the other hand, the results of hydraulic analysis reveals that the bridge opening existed is large enough to cope with the flood discharge. Therefore, reconstruction plan shall be done in accordance with the existing condition. Stone masonry shall be provided around the foundation of A1 abutment because the local scouring was observed there.

5) SER No. 33

The results of hydraulic analysis and the flood record proved that the existed bridge opening including free board is large enough to cope with. Therefore, the reconstruction plan shall be prepared in accordance with the existing condition. The results of the analysis show that the velocity is very fast (4.0m/s). So, it is recommended that the firm stone masonry work shall be done around the foundation of substructure.

6) SER No. 59

The bridge locates in the flood area. However, the flood record shows that enough free board is kept (1.5m). It seems that the flood submerges surrounding area including approach roads, but left the bridge alone. Therefore, the construction plan shall be prepared in accordance with existing condition.

7) SER No. 20

This bridge is located at a bottom of vertical alignment because the bridge and approach roads are constructed along with the natural features. Concluding from the results of the hydraulic analysis and the flood record, the existing bridge opening is not enough thus the bridge is submerged in flood. It is recommended that the bridge shall be raised and extended in the reconstruction plan. Considering the going in and out from the public facilities (a hospital) which locate adjacent to A1 abutment side, proposed elevation of new road surface is up to ADL 152.00m (1.5 higher than existing road). Alternative hydraulic analysis was carried out to search the necessary river cross section. The result shows that the bridge should be extended to 17.0m long.

8) SER No. 70

The results of hydraulic analysis and the interview at the site show that existing bridge opening is enough. Therefore, the reconstruction plan shall be done in accordance with existing condition.

9) SER No. 7

According to the flood record, the free board is sufficient fo flow the flood discharge (1.8m). The ships/boats, passing the bridge are small, and there is no request to raise the navigation clearance in order to navigate bigger ships/boats. Also, the depth of water is shallow. Therefore, existing clearance is not necessary to be changed.

Both abutments were constructed jutting out into the river in order to shorten a bridge length. Both sides of stone masonry walls behind abutments are substantially damaged, so that it need to be rehabilitated urgently. The A1 abutment shall be set back, and stone masonry shall be constructed behind the abutment. On A2 abutment side, there is dwelling houses. So, the new abutment shall be set in accordance with the existing one and the stone masonry shall be constructed as well. Mat gabions shall be provided around the foundation in order to protect the local scouring.

(4) Summary of Hydraulic Defect and Rehabilitation Plan

The major hydraulic defect detected in this Study are tabulated together with the rehabilitation plans to be taken in Table 8.4.

Table 8.4 Summary of Major Hydraulic Defects and Rehabilitation Plan

SER No.	Hydraulic Defect	Cause	Rehabilitation Plan
	Representative 10 Bridges		
212	- Exposure of caisson foundation (A2)	- Local scouring	- Protect concrete to foundation scoured
77	- Exposure of caisson foundation (P8 & P9)	- Local scouring - Insufficient embedded depth of foundation	- Reinforced concrete curtain wall
211	- Exposure of spread foundation (A1)	- Local scouring	- Protect concrete to foundation scoured
7	- Damage of wedge stone/brick wall (behind A1 and A2)	- Insufficient strength of wall	- Stone maronry and mat gabions
	Other 91 Bridges		
86	- Settlement of abutment (A1)	- Local scouring	- Protect concrete to foundation scoured
87	- Exposure of foundation	- Local scouring	- Protect concrete to foundation scoured
103	- Beams submerged	- Insufficient free board	- Raising of bridge (This rehabilitation shall be taken with raising of road in future.)
60	- Causeway	---	- Reconstruction
139	- Causeway	---	- Reconstruction
62	- Causeway	---	- Reconstruction
63	- Causeway	---	- Reconstruction
25	- Exposure of foundation (P1 to P3)	- Local scouring	- Protect concrete to foundation scoured
35	- Exposure of foundation	- Local scouring	- Protect concrete to foundation scoured

8.5.4 Recommendation from Hydraulic Aspect

(1) General

Through the field inspection carried out during the whole Study period, it was observed in some existing bridges that the hydraulic effect have not been fully taken into account in the planning of bridges from hydraulic aspect. This section presents the recommendation from hydraulic aspect.

There is not a systematic waterway improvement plan on the rivers observed in this Study. Where the plan is not existed, the bridge planning shall be individually made along with the bridge site condition but at least the followings shall be considered.

(a) Waterway cross section shall be determined considering upper and lower stream conditions of the bridge site.

(b) High Floor Water Level shall be determined considering the past flood record compared with the results of hydraulic analysis. In this case, existing embankment height is not necessary to be taken into the determination of the high flood water level.

(c) The elevation of beam sofit of new bridge shall not be lower than the existing one.

(d) The viaducts for flood relief opening shall be built at the approach road if necessary, considering past flood damege.

(e) The structures in the river shall be planned along with the recommendation as much as possible described in 8.5.4.2.

(2) Recommendation from Hydraulic Aspect

Major items to be considered in crossing requirements for waterway are as follows:-

- Bridge Opening
- Abutments
- Piers
- Bank Protection

1) Bridge Opening

<u>Crossing Requirement</u>

(a) To set bridge opening with enough width for design discharge

(b) To set bridge height with enough free board on design flood water level

Description

If the opening of bridge is smaller than the design cross section, constriction flow will be caused around the bridge. Flood condition in the upstream side of the bridge will be made worse due to backwater effect caused by the constriction flow. Furthermore, severe scouring and erosion around the bridge will be occurred.

Freeboard between the HFWL and bottom of the beam shall be adopted at least following values;

Design Discharge (m^3/s)	Freeboard (m)
less than 500	0.6
500 to 2,000	1.0
over 2,000	1.5

2) Abutments

Crossing Requirement

(a) To keep sufficient depth of foundation
(b) To set abutment at outside of the design river section
(c) To set surface direction of abutments as parallel to flow direction

Description

Sufficient depth of foundation is necessary for the stability of the abutment from foundation engineering point of view as well as the safety against scouring.

In order to keep smooth flow and to prevent constriction flow around bridge, it is necessary to keep the width of bridge opening more than the width of river cross section. Therefore, the abutments shall be located outside of the design river cross section as much as possible.

3) Piers

Crossing Requirement

(a) To adopt pier of oval shaped solid wall type or round shaped column type
(b) To keep reduction rate of flow area by piers as small as possible
(c) To keep sufficient depth of foundation

Description

As the piers are one of the obstacle to flow, occurrence of turbulence around the piers cannot be avoided. It is important to reduce the turbulence as much as possible as the turbulence often cause local scouring around the piers as well as erosion around the bridge.

Therefore, it is necessary to adopt the types of pier that have small disturbance against flow. Long and narrow oval shaped solid wall pier is the most desirable type of pier to be adopted in case of reconstruction. The direction of longer diameter of the oval shaped pier shall be set as parallel to the river flow. The footing of the pier shall be installed under the design river bed.

The piers of pile bent and the pier of solid wall with exposed pile foundations have more problem of erosion and local scouring as well as more backwater effect and clogging of floating logs than the oval shaped pier. Hence, these types shall not be adopted.

If the direction of flow is changing very much, pier of round shaped column can be adopted. Even in this case, multiple column shall not be adopted.

In order to keep flow area for the design discharge, it is desirable to keep the reduction rate of flow area by piers as small as possible. A Japanese guideline gives the reduction rate of 5% which can be considered to be adopted as a reference.

Embedded depth of the footings of the piers shall be determined based on the depth of local scouring around the piers as well as the height of sand bars. A guideline applied in Japan givers the embedded depth of footing which can be considered a reference as follows (refer to Fig. 8.3 to 8.5):

Location of Pier	Embedded Depth of Footings
a) Low water channel	More than 2m below river bed of low water channel
b) High water channel within 20m from the top of low water channel	More than 2m below river bed of low water channel
c) High water channel over 20m from the top of low water channel	More than 1m below river bed of high water channel

4) Bank Protection

Crossing Requirements

(a) To provide bank protection with sufficient length
(b) To provide bank protection with sufficient embedded depth
(c) To provide adequate foot protection

Description

Direction of the most fast portions of the current in the channel is gradually changed by the movement of sand bars. Hence, there is a tendency that the portions of the river banks where the fast current attacks are easily to be eroded even if the abutments and piers are properly installed. Furthermore, by the progressing of meandering, erosion of the banks will be easily to be occurred and the safety of the bridge will be reduced. Therefore, bank protection around the bridge with sufficient length is necessary to be provided.

The bank protection shall be provided around the bridge including abutments. Minimum length of bank protection is the width of the abutment plus 10m for both the upstream and downstream sides of the river channel (refer to Fig. 8.5). In the case of erosion of the banks by the movement of sand bar or meandering is anticipated very much, longer length of bank protection shall be provided.

Embedded depth of the bank protection shall be determined by considering the height of the sand bars as well as depth of local scouring around the bridge. One of the guideline of the embedded depth of bank protection is 0.5 - 1.0m for the rivers with middle size and more than 1.0m for the rivers with large size.

If the scouring around bank protection is anticipated, foot protection shall be provided around the bank protection using gabion packs, gabion mat and stones, etc.

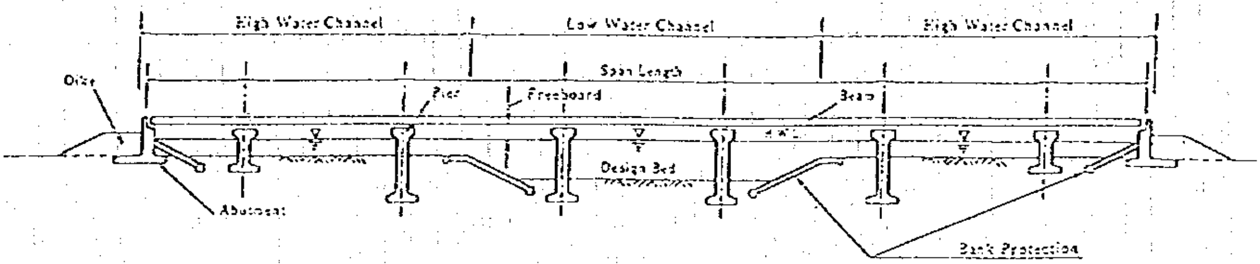
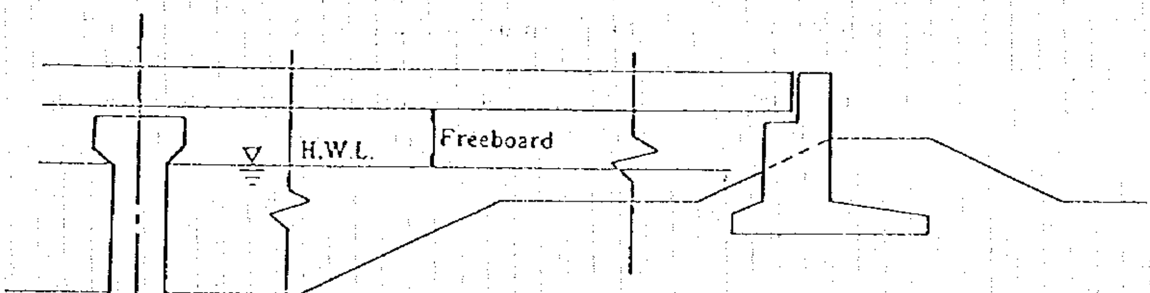


Figure 8.3 General Description of Bridge
- Bridge for Compound Design Cross Section



Design Discharge (m3/s)	Minimum Freeboard (m)
Less than 500	0.6
500 - 2,000	1.0
Over 2,000	1.5

Figure 8.4 Freeboard between HWL and Beam

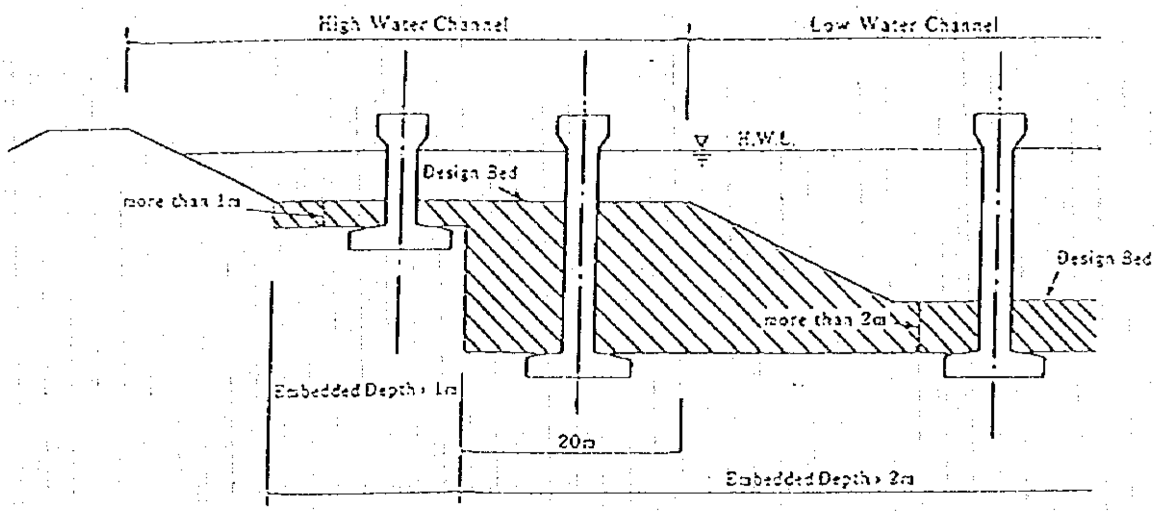


Figure 8.5 Embedded Depth of Pier

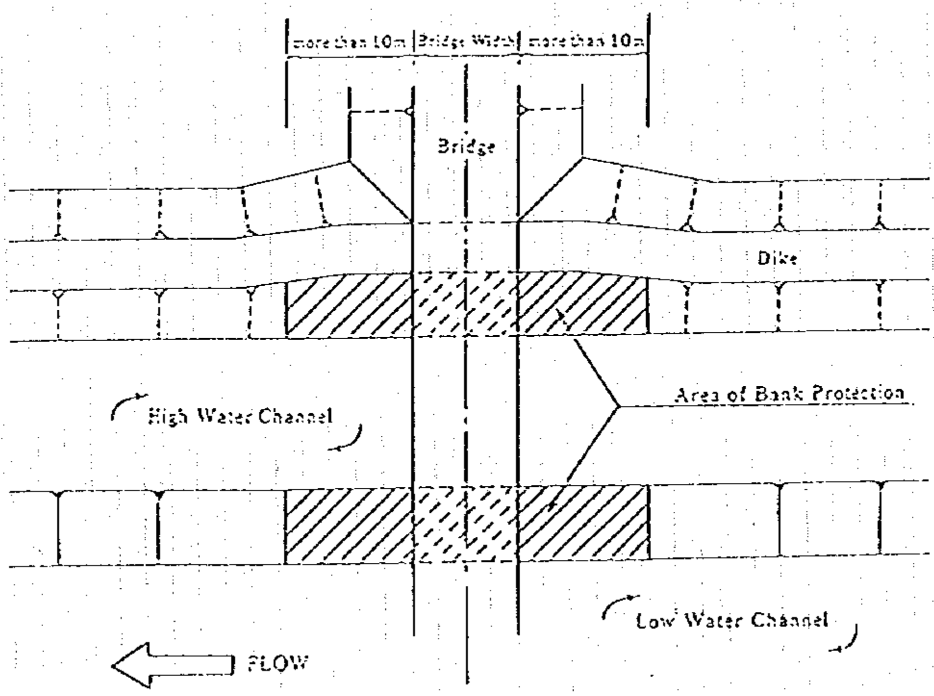


Figure 8.6 Area of Bank Protection

CHAPTER 9 GEOLOGICAL SURVEY

9.1 General

This chapter presents the results of geological survey carried out on 8 bridges out of the 10 bridges for the detailed survey. The location maps and quantities of the survey are given in Figure 9.1 and Table 9.1 respectively.

In the case of Bridge SER No. 85, the survey was not carried out because of rock-out-crops in the site. In the case of Bridge SER No. 212, the survey was not carried out because this bridge had to be repaired and strengthened without widening.

9.2 Objectives

The objective of the survey is to clarify component and characteristics of subsoil at the selected bridge site. Important subsoil engineering parameters were obtained from this survey and adopted for use of in the analysis, design, and selection of foundation types.

9.3 Methodology

The survey was sublet to a selected geological organization and commenced on 24th October, 1995 for a period of 30 days.

The geological survey is comprised of the main works as follows:-

- Rotary wash boring technique which is common practice in Sri Lanka.
- Rotary core driving technique where the wash boring could not be applied to the subsoil such as rock or boulders existed.
- Field tests such as the Standard Penetration Test (SPT), including the collection of disturbed and undisturbed soil samples to conduct laboratory tests.
- Observation of ground water levels below ground surface.
- Laboratory tests to obtain soil index properties.

9.4 Results of Geological Survey

The results of borehole investigation carried out together with N-value, and laboratory tests are given in Appendix - H.

Engineering appreciation of the ground conditions are summarized as follows:-

(1) SER No. 33

Borehole investigation indicate presence of a very loose clay silt layer down to 1.50m deep (N-value of 2). This is followed by a medium dense to dense

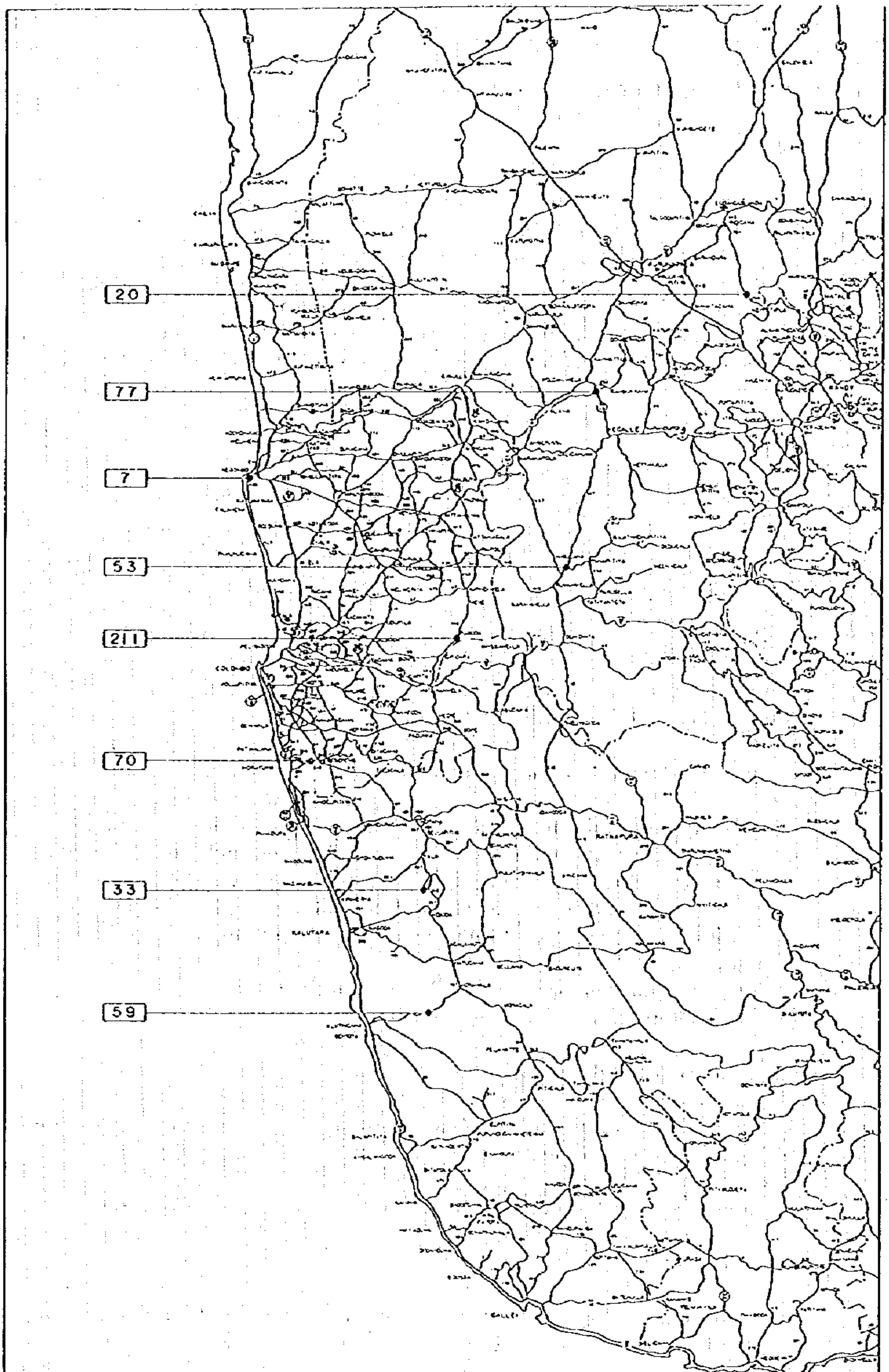


Figure 9.1 Location Map for Geological Survey

Table 9.1 Quantities for Geological Survey

	Type of Boring	Depth of Boring	Field Test		Laboratory Test					Remark
			SPT Test	Undisturbed Soil Sampling	Grain Size Test	Specific Gravity	Natural Moisture Content	Unconfined Compression Test	Liquid & Plastic Limit	
Unit	-	m	Nos.	Nos.	Nos.	Nos.	Nos.	Nos.	Nos.	
SER No. 85	-	-	-	-	-	-	-	-	-	Rock outcrops
77	Wash boring	5.59	7	0	2	2	2	0	2	Drawings of existing bridge
53	Wash boring	13.90	14	1	2	2	4	1	2	
33	Wash boring	7.05	8	0	2	2	2	0	2	Existing log of borehole
59	Wash boring	15.00	15	2	2	2	2	1	1	
20	Wash boring	13.00	9	0	3	2	3	0	3	
70	Wash boring	12.79	10	1	2	1	2	1	1	Existing log of boreholes
7	Wash boring	19.35	20	1	2	2	4	1	2	
211	Wash boring	9.80	11	1	2	2	3	1	2	
212	Wash boring	-	-	-	-	-	-	-	-	Drawings of existing bridge
Total		96.48	94	6	17	16	22	5	15	

silty sand layer (SM), clay silt (ML) and silty sand (N-value in the range of 7 to 36). Refusal to penetration of the SPT split spoon sample was noted at a shallow depth of 3.0m because of gravels existed.

Existing borehole logs shows that the bed rock could be observed down to 6m depth of river bed in the middle of river and boulder existed in the subsoil down to 3.2m to 4.7m of river bed with 0.4 to 1.1m thick.

Considering the subsoil condition, deep foundation (caisson) is recommended in this bridge. The safety bearing capacity of the hard stratum encountered at an elevation of 7m deep is estimated at 1500kN/m².

(2) SER No. 59

Borehole investigation shows that subsoils down to 7m depth are weak (N-value in the range of 0 to 3). Deep foundation (pile, etc.) is recommended to support the structure. The safe bearing capacity at 8.5m depth is 1500kN/m².

(3) SER No. 77

Borehole investigation shows that the subsoil predominantly consist of sandy soils (SM/SC/ML). (Very loose layer, N-value in the range of 0 to 3) were observed down to 5.0m depth. Hard stratum was confirmed down to 5.5m depth.

Drawings of the existing bridge shows that rock could be observed down to 2.0m depth of river bed.

(4) SER No. 53

Subsoil in this area mainly consists of silty sand (SM) and silt (ML). The top sand (SM) and silt (ML) layers are followed by a soft sandy clay layer. Very low N-value of 0.3 and 5 were recorded down to 4.0m.. Refusal to penetration of the SPT spoon sampler was noted at a depth of 7.05m because of quartzite cobbles existed.

Considering the subsoil condition, the conventional shallow foundation can not be recommended. Therefore, it is recommended to support the structure on deep foundation (caisson) because of cobbles existed in the subsoil. The safe bearing capacity of the hard stratum encountered at an elevation of 10.23m depth is calculated at as 1800kN/m².

(5) SER No. 20

Borehole investigation shows that the silty sand with mica beyond 5m depth of borehole which is hard layer (N-value>50). The bed rock encountered at 9.15m is moderately weathered to fresh horn blended bistite gneiss.

Shallow foundation can be used on the silty sand with mica layer. The allowable bearing capacity is estimated at 300kN/m².

(6) SER No. 70

Borehole investigation shows that the highly weathered/fractured rock beyond 9.6m depth. Existing borehole log shows that an incline of the rock. Upper part of the subsoil consists of very loose clayey silt and very soft organic clay having N-value in the range of 0 to 2. Lower part of the subsoil consists of silty fine sand of which N-value is in the range of 31 to 13.

Deep foundation is recommended to support structures. The bearing capacity at deferent elevations are given below:-

- 10m 2000kN/m²
- 13m 4500kN/m²

Caisson foundation is recommended because it seems that the piles cannot penetrate the fine sand layers of which N-value is in the range of larger than 30.

(7) SER No. 7

Low SPT N-value in the range of 1 to 5 were recorded within the upper layers down to a depth of 9.0m Underlying this, high plasticity clay layer exhibits N-value of 7 and 12. High N-value (26 to 54) were recorded with in the sand layer beyond 14.0m depth. The hard stratum was observed beyond 18m depth and it can be a bearing stratum.

Deep foundation (piles) on the hard stratum is recommended. The safe bearing capacity of this stratum is estimated at 1800kN/m².

(8) SER No. 211

Very low SPT N-value in the range of 0 to 3 were recorded with in the upper layers down to a depth of 8.0m which consist of fine sand and clayey silt. The hard stratum was observed beyond 9.65m, and it can be a bearing stratum.

Deep foundation (piles) on the hard stratum is recommended. The safe bearing capacity of this stratum is estimated at 15000 kN/m².

CHAPTER 10 DETAILED VISUAL INSPECTION

10.1 General

The detailed visual inspection was carried out as a part of the detailed survey on the 10 representative bridges selected. The objective of a detailed visual inspection is to obtain detailed engineering information so as to enable preliminary maintenance and rehabilitation design. The results of the inspection on these selected bridges are also used in formulating a rehabilitation program for the rest of the bridges in the study.

This inspection consists of structural detailed measurements, material strength measurements and deterioration degree measurements. The extent and type of the work vary among bridges, and it depends on the degree and type of damage in each bridge. Data gathered during the inspection was recorded in the inspection sheets, and finally all the results were transferred to a drawing. The drawings for each bridge consist of general view and crack diagram of concrete members and/or corrosion diagram of steel members. The drawings of each bridge are enclosed in Volume IV Drawings in a separate booklet.

10.2 Procedure and Inspection Equipment

Inspection team consisted three members of the study team and two counterpart engineers. Inspection equipment was as follows:

■ Measurement equipment:

2m and 3m tapes, 50m steel tapes, slide caliper, crack scales, caliper, ribbon tape, plumb bob.

■ Recording equipment:

camera, black board, chalk, plastic board.

■ Inspection equipment:

binocular, wire brush, scraper chisel, chipping hammer, torch light, electric grinder, brush and paint.

■ Safety equipment:

safety belts, goggles, cotton gloves and first aid kit.

■ Testing equipment:

ultrasonic thickness meter (steel thickness) and schmidt hammer.

10.3. Detailed Structural Measurement

Detailed structural measurement consisted of basic dimension measurement, crack/corrosion mapping and steel thickness measurement. The objective of the detailed structural measurement is to obtain actual-measured dimension of the bridge so as to examine structural characteristic of the bridge to be carried out.

10.3.1 Detailed structural measurement

Basic dimension measurement was carried out so as to record all structural details which includes overall width, carriageway width, length of span and bridge, arrangement and size of girder, layout of splice plate, bracings, abutments, piers, columns and wing walls.

These results was reflected into an actual-measured drawing. For bridges where some of the as-built-drawings were available, dimensions were compared with those in the drawing so that any differences could be identified. The measurement of dimensions was done to the nearest 5 or 10 mm.

10.3.2 Crack/Corrosion Mapping Survey

Before the crack/corrosion mapping survey, inspection sheets were prepared for each member and the extent of damages identified were recorded on those sheets in this survey. In a concrete member, damages were classified into crack, flaking, rebar exposure, free lime, etc., while in a steel member damages were classified into corrosion, lamination, paint deterioration, buckling, etc. The results of this survey were presented in the form of a crack diagram and/or a corrosion diagram. As these diagrams often give an indication of the cause of the damage, measurements and mapping help in providing a clearer picture of the condition and behavior of the structure, and they also help in preparing most practical maintenance and rehabilitation method. Surface crack width was measured with a transparent crack scale, while flaking of concrete was measured with small steel tapes. Deformation and corrosion in steel members were observed visually, it was then recorded in the form.

10.3.3 Steel Thickness Measurements

Since there was no as-built-drawing on steel bridge surveyed, it was important to provide the exact thickness of a steel member. The measurement of steel thickness was carried out with ultrasonic thickness meter and slide caliper after removing paint on a surface of main beam.

10.4 Schmidt Hammer Test

In order to obtain a concrete strength, non-destructive test was carried out by using schmidt hammer. Schmidt hammer gives an indication of the estimated strength of the concrete if at least 20 impacts is carried out per measurement point. A summary of schmidt hammer test results is shown in Table 10.1.

Table 10.1 Summary of Schmidt Hammer Test Results

(Unit: N/mm2)

SER No.	SUPERSTRUCTURE		SUBSTRUCTURE	
	BEAM	SLAB	ABUT/PIER	FOOTING/CAISSON
77			388 (P)	
			427 (P)	
				323 (C)
				271 (C)
33		544		
		544		
			375 (P)	
			375 (P)	
70		414		
		466		
			310 (A)	
			271 (A)	
			440 (P)	
			466 (P)	
7	479 (PC)			
	505 (PC)			
	505 (PC)			
	479 (PC)			
	518 (RC)			
	505 (RC)			
	440 (RC)			
	401 (RC)			
			440 (A)	
			362 (A)	
			479 (P)	
			427 (P)	
			505 (P)	
			479 (P)	
			388 (P)	
			479 (P)	
			479 (P)	
211		388		
		401		
			310 (A)	
			297 (A)	
			180 (P)	
			219 (P)	
			258 (P)	
212		492		
		479		
	505			
	466			
			453 (A)	
			453 (A)	
			401 (P)	
			440 (P)	

CHAPTER 11 FULL SCALE BRIDGE LOADING TEST

11.1 General

The main objective of a full scale bridge loading test is to confirm actual load carrying capacity of typical type of bridge in Sri Lanka and the test results on these selected bridges were used to establish a determination of applicable live load.

In order to estimate actual load carrying capacity of bridge, theoretical calculation and full scale loading test can be considered as a practical method. Theoretical calculation needs all of detailed information on the bridge design and can be used in a comparison with the results of full scale loading test, while the results of full scale loading test indicates the actual loading capacity of bridge which includes a residual loading capacity, the value of which depends on type of structure, construction materials and extent of defects or deterioration.

The primary purpose of a full scale loading test is therefore to estimate the residual loading capacity of the bridge. To achieve this, comparison of deflection will be made between theoretically calculated values and those actually measured. In the Study, the theoretical calculations were carried out by using a finite element method (F.E.M) in order to expect higher accuracy in the calculation.

Out of the 10 bridges for the detailed survey, a total of three spans at three different bridges listed below were selected for a full scale bridge loading test.

Table 11.1 List of Bridges for Loading Test

SER No.	ROUTE	BRIDGE No.	TYPE OF BRIDGE	SPAN LENGTH (m)	WIDTH (m)
59	B157	43/4 K	RSJ/BUC	8.45	3.20
211	B146	8/3 K	RSJ/RCS	7.68	3.55
212	AA002	138/1 K	PSC/PRE	16.16	10.40

The loading test carried out in the Study is static loading tests and it was to measure deflection of main component part under a known load in which loaded vehicles.

11.2 Loading Test Planning

Preparatory to loading test, detailed structural survey, selection of measurement points and determination of loading position & loading case were conducted.

11.2.1 Selection of Measurement Points

Since the main purpose of the loading test in the Study is to examine the actual loading capacity of main beam (ratio of actually measured value and theoretically calculated one under the same known loading condition), the measurement point on the main bridge component where the measurement of deflection was carried out, was mainly selected so as to coincide with the point where the maximum sectional forces induced. Next, another some points were selected in order to proof the test results on the deflection of main measurement point.

The number and position of the measurement points were determined mainly to enable comparison and verification of the theoretical calculation and the actual measurement results.

Based on the above concept, the measurement points were selected as summarized hereunder. The position of the measurement points are illustrated in Appendix - J of Volume III.

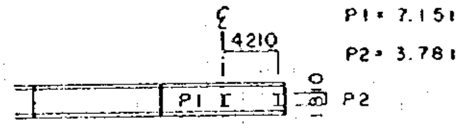
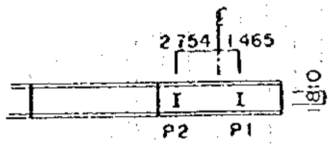
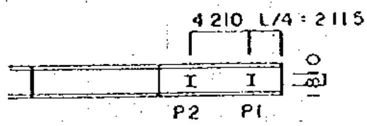
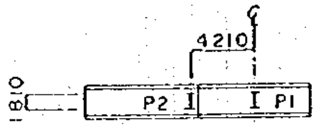
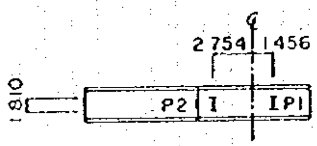
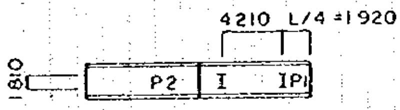
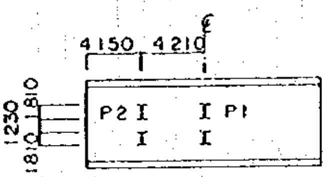
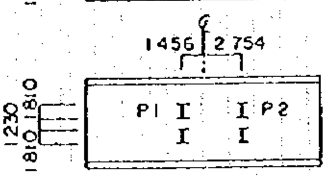
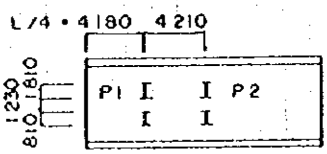
Table 11.2 Summary of Measurement Points

SER No.	TYPE OF BRIDGE	MEMBER	DISPLACEMENT
59	RSJ/BUC	Girder	L/2 : 3 point L/4 : 3 point
		Pier	: 2 point
211	RSJ/RCS	Girder	L/2 : 3 point L/4 : 3 point
212	PSC/PRE	Beam	L/2 : 3 point L/4 : 3 point

11.2.2 Determination of Loading Position and Loading Case

The loading position and number of lorries which would induce the most severe load effect to a member under consideration (that is inducing the maximum bending moment, shear force or displacement at the selected observation point) were examined based on an influence line diagram. Based on the result of this examination, the loading position of each case together with number of lorries to be used were finally determined as summarized in Table 11.3. Plan, profile and cross sections of each loading case are illustrated in Apendix - J.

Table 11.3 Loading Position and Loading Case

SER No.	TYPE OF BRIDGE	LOADING CASE	No. of LORRY	LOADING POSITION
59	RSJ/BUC	1	1	
		2	1	
		3	1	
211	RSJ/RCS	1	1	
		2	1	
		3	1	
212	PSC/PRE	1	2	
		2	2	
		3	2	

11.3 Method of Loading Test

Flowchart which shows the practical loading test procedure and the above mentioned planning is illustrated in Figure 11.1.

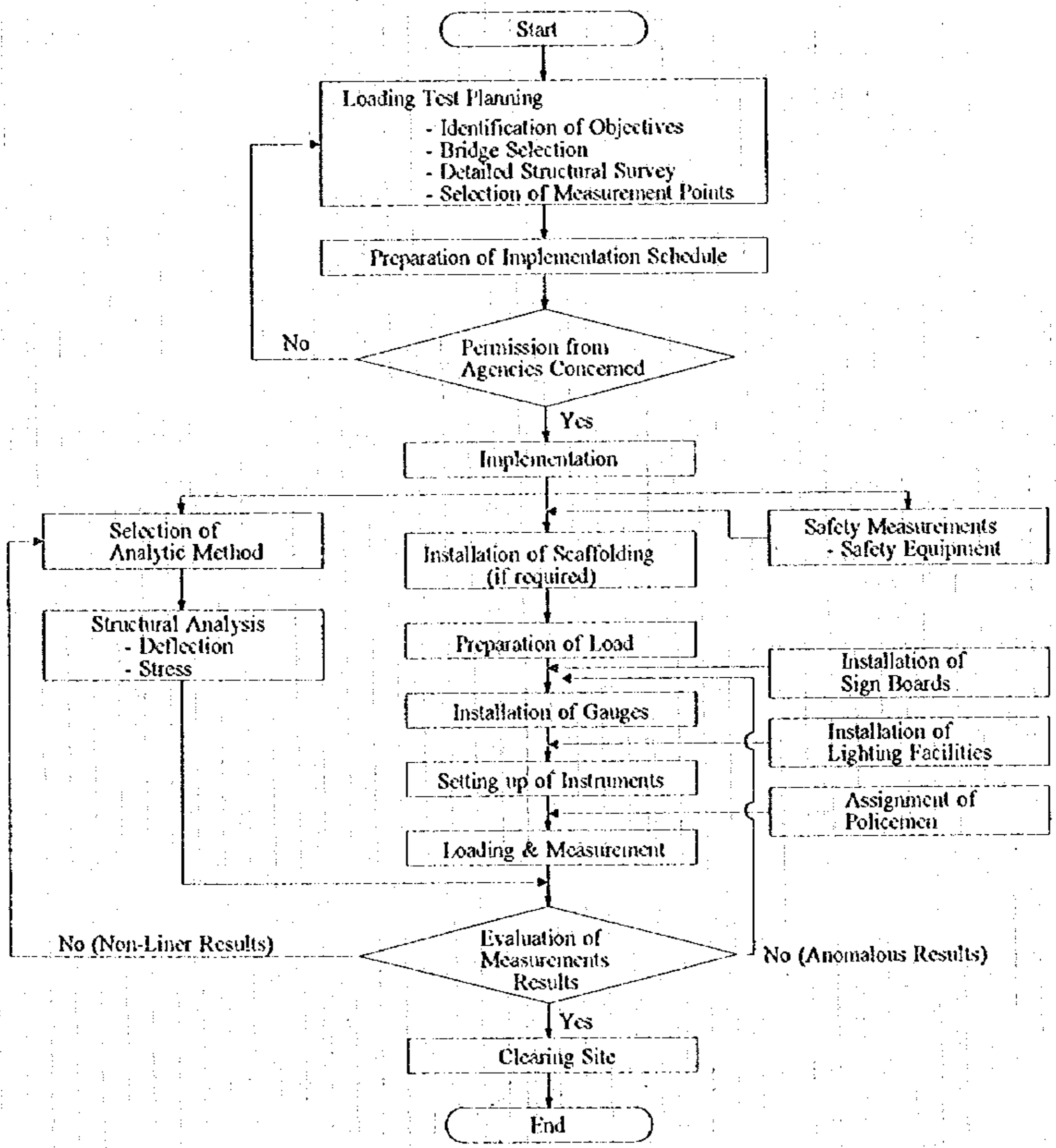


Figure 11.1 Flowchart of Loading Test

11.3.1 Preparation of Loads

The load was applied by using lorry which was loaded with crushed aggregate. The weight of lorry (11 ton) was determined considering a safety of member and configuration of the lorry including distance of axles and weight of each axle was measured with tape and portable weigh bridge in advance. This data was used for the exact determination of loading position on a bridge and for structural analysis. The exact position of the wheel was marked on the surface of the pavement using a chalk.

11.3.2 Preliminary Structural Analysis

Any damage to the bridges by the most severe load to be applied during the loading test shall be prevented. Therefore preliminary structural analysis using data collected out in advance so as to ensure that no damages on the structure arise from the loading.

(1) SER No. 59 & 211

Since these bridges have no information on as built members, the weight of load was fixed for 11 tons considering the actual weight of long wheel bus of S.L.C.T.B.

(2) SER No. 212

This bridge has as built drawing and the most severe load calculated was greater than 11 tons. However it was considered that sufficient results (deflection) could be obtained by using the above weight of lorry, so 11 tons was adopted in this bridges too.

11.3.3 Safety Management

In a full scale bridges loading test, safety management is the most important and shall be strictly executed in coordination and cooperation with the Government Agencies concerned. Out of the several important safety management items, traffic control shall be considered first and foremost. In order to control traffic flow properly and smoothly before, during and after the loading test, the following action were taken:

■ Installation of Traffic Sign Boards

The traffic sign boards such as "Load Test in progress" and "Stop" were prepared and installed in a timely manner by the local consultant concerned.

■ Assignment of Traffic Policemen

During the period of the loading test, 2 traffic policemen were assigned at the site to ensure security and to control the traffic flow.

■ Systematic Arrangement of Lorries

In order to minimize duration of the traffic interruption (10 minutes), loading sequence of the lorries was carefully studied and determined in advance.

11.3.4 Installation of Scaffolding

Installation of scaffolding is one of the major preparatory works in the loading test. Scaffolding was required to provide access to the soffit of girder and support beam of pier in SER No. 59 bridge.

11.3.5 Selection and installation of Dial Gauges

Specification of dial gauge applied are as follows:

Maker	=	Peacock
Model	=	307 G
Accuracy	=	1/100 mm (Displacement 1m/one rotation)

Firstly supporters for dial gauge were set under main girder and then dial gauge was set on the top of supporters using magnet base.

11.3.6 Loading and Measurement

After completion of the above mentioned preparatory works at the job site, traffic flow at both ends of the bridge was closed and the dial gauges were read immediately. The lorry as a static load was dived onto the proposed first location (for step 1) and the dial gauges were read after reading was stable. After driving the lorry out of the bridge, the dial gauge was read again. This practice was repeated for all the loading cases as shown in Table 11.3.

11.4 Results of Loading Test

The loading tests were carried out for three bridge sites during the period from 1 to 3 November 1995 in accordance with the plan and the method mentioned in section 11.2 and 11.3.

Assessment of the measurement results and application of the test results to rehabilitation design are presented in the following sections.

11.4.1 Assessment of Measurement Results

The measurement values obtained at the job site usually contain a residual deflection value of the gauge itself. Thus measurement values presented and used in this assessment have been adjusted by subtracting these values from the field reading values.

The assessment was carried out mainly by means of comparison of deflection between actually measured through the loading tests, and theoretically calculated using structural theories.

(1) Bridge No. 59

Table 11.4 is a comparison table in terms of deflection between actually measured and theoretically calculated.

Table 11.4 Comparison of Test Result Value and Calculated Value

(Unit: mm)

Measurement Point		Loading Case-1			Loading Case-2		
		Test Results	Calculation Results	Ratio	Test Results	Calculation Results	Ratio
		A	B	A/B	A	B	A/B
L/2	G1	0.59	0.61	0.97	0.67	0.66	1.02
	G2	0.69	0.70	0.99	0.74	0.76	0.97
L/4	G1	0.49	0.44	1.11	0.55	0.52	1.06
	G2	0.50	0.48	1.06	0.60	0.60	1.00

Measurement Point		Loading Case-3		
		Test Results	Calculation Results	Ratio
		A	B	A/B
L/2	G1	0.65	0.63	1.03
	G2	0.71	0.70	1.01
L/4	G1	0.54	0.51	1.06
	G2	0.59	0.58	1.02

- The actually measured values agree well with the calculation results as shown in Deflection Ration(Test result/Calculation result) between A and B.

- Therefore, effect of composite action in RSJ/BUC can be expected sufficiently.

(2) Bridge No. 211

Table 11.5 is a comparison table in terms of deflection between actually measured and theoretically calculated.

Table 11.5 Comparison of Test Results Value and Calculated Value

(Unit: mm)

Measurement Point		Loading Case-1				
		Test Results	Calculation Results (1)	Calculation Results (2)	Ratio	
		A	B	C	A/B	A/C
L/2	G1	0.26	0.17	0.58	1.53	0.45
	G2	0.27	0.20	0.61	1.35	0.44
L/4	G1	0.17	0.10	0.35	1.70	0.49
	G2	0.20	0.12	0.36	1.67	0.56

Measurement Point		Loading Case-2				
		Test Results	Calculation Results (1)	Calculation Results (2)	Ratio	
		A	B	C	A/B	A/C
L/2	G1	0.26	0.15	0.52	1.73	0.50
	G2	0.30	0.18	0.54	1.67	0.56
L/4	G1	0.19	0.11	0.35	1.73	0.54
	G2	0.26	0.13	0.38	2.00	0.68

Measurement Point		Loading Case-3				
		Test Results	Calculation Results (1)	Calculation Results (2)	Ratio	
		A	B	C	A/B	A/C
L/2	G1	0.26	0.14	0.49	1.86	0.53
	G2	0.30	0.16	0.51	1.88	0.59
L/4	G1	0.20	0.10	0.33	2.00	0.61
	G2	0.27	0.12	0.36	2.25	0.75

Notes: (1) is calculation results in which full composite action is considered.
(2) is calculation results in which composite action is not considered.

- Deflection ratio (Test result/Calculation result) between A and B indicated that the beams acted as a half-composited beam.

- Therefore, it can be considered that live loading is distributed into deck slab and main girders in some proportion.

(3) Bridge No. 212

Comparison of deflections between actually measured and theoretically calculated is tabulated in Table 11.6.

Table 11.6 Comparison of Test Results Value and Calculated Value

(Unit: mm)

Measurement Point		Loading Case-1			Loading Case-2		
		Test Results	Calculation Results	Ratio	Test Results	Calculation Results	Ratio
		A	B	A/B	A	B	A/B
L/2	Beam G9	0.63	1.43	0.44	-	1.48	-
	Beam G6	0.63	1.42	0.44	0.69	1.47	0.47
	Beam G3	0.62	1.41	0.44	0.66	1.46	0.45
L/4	Beam G9	0.42	0.95	0.44	-	1.04	-
	Beam G6	0.41	0.95	0.43	0.43	1.03	0.42
	Beam G3	0.40	0.95	0.42	0.40	1.03	0.39

Measurement Point		Loading Case-3		
		Test Results	Calculation Results	Ratio
		A	B	A/B
L/2	Beam G9	-	1.28	-
	Beam G6	0.48	1.28	0.38
	Beam G3	0.47	1.27	0.37
L/4	Beam G9	0.38	0.98	0.39
	Beam G6	0.35	0.97	0.36
	Beam G3	0.31	0.96	0.32

- Deflection ratio (Test result/Calculation result) between A and B indicated that the actual rigidity of the bridge is greater than their value obtained in the preliminary calculation using F.E.M analysis.

- In the F.E.M analysis, all of dimensions of the bridge was based on as built drawing, while the actual thickness of the bridge which includes a insitu concrete was greater than the designed value. The residual thickness of the insitu concrete was about 100 mm and this means that the actual rigidity of the bridge is about 150% of the designed value.

- In the F.E.M analysis, Young's modules of bridge is assumed to be 3.3 x 10E + 5 kgf/cm^2 based on the design information (precast unit concrete grade: 40 N/mm^2), while the test results of schmidt hammer shows that the concrete strength of the beam was 485 kgf/cm^2.

- Considering the differences between the actual model and the theoretical calculation, it seems to be quite all right to consider that the test result is relatively reliable and the calculation method is suitable.

- Furthermore, the deflection diagram by F.E.M analysis agrees well with the distribution of actually measured values and the stresses calculated by F.E.M analysis is only 17 kgf/cm^2. Thus it could be concluded that the pretension slab beam based on the standard design of RDA has enough durability even though stress due to dead load is added and design live load is applied.

CHAPTER 12 STRENGTH TEST OF STEEL SAMPLES

12.1 General

The steel bridges studied in the Preliminary Inspection can be assumed that they were made of wrought iron or mild steel fabricated. The number of steel bridges, as a type of superstructure, reached 73 (72.3%) out of the 101 bridges.

Since there is no design data and information about those bridges and there is no specification of steel material used in their drawings, it is impossible to know what kind of steel material used for those bridges.

Only 34 out of the 73 steel bridges are confirmed their construction year. Most of those bridges were constructed during a period of British dominion (1860-1945).

Generally, materials mainly used for steel bridges in Europe have been changed as time passing as follows:-

- approx. 1847 to 1895 : Wrought iron
- approx. 1855 to 1870 : Accid process Bessemer Steel (Accid Bessemer Steel)
- approx. 1870 to 1895 : Thomas process Bessemer Steel (Basic Bessemer Steel)
- approx. 1856 to 1930 : Low quality mild steel (early days of Open Heath Steel)
- approx. 1920 to : Mild steel (similar to Structural Mild Steel (SS400) used at the present

Although strength of steel bridges can not be determined by the visual inspection on the material used, reports prepared in U.K., Germany and U.S.A. indicate general strength of material used in old bridges as follows:

1) Some data and information from U.K. Department of Transport, Bridge Section, state some idea for material quality as follows:-

   - Although there is no reliable data, steel produced before 1955 has its yield stress of 230 N/mm2 (2,340 kgf/cm2).

   - Steel produced before 1922 has low quality.

2) According to the German calculation standard for steel Railway Bridges (B.E. 1936), the allowable stress for old girders is given as follows:-

   - Wrought iron and mild steel produced before 31.12.1894: 22,000 kgf/cm2
   - Mild steel produced after 1.1.1895 :24,000 kgf/cm2

3) The value of allowable stress is given in American old beam standard, Rules for Rating Existing Iron and Steel Bridge (1936. A.E.R.A.) as follows:-

- Tensile axle load and tensile edge of flexural bending moment (Net area).

Wrought iron	:	20,000 lbs/m2 (1,410 kgf/cm2)
Bessemer steel	:	21,000 lbs/m2 (1,480 kgf/cm2)
Open heath steel	:	24,000 lbs/m2 (1,680 kgf/cm2)

As above-mentioned, each country has deferent specification for allowable stress for steel materials, therefore, it is difficult to adopt which one particularly.

It is known that the most of steel bridges are constructed during the dominion of U.K. There should be a time lag between production of materials and construction of bridge if the girders were fabricated in U.K. and transported by ship to Sri Lanka.

On the other hand, the allowable stress for steel used in the existing bridge should somehow be determined in order to select its rehabilitation plan.

Therefore, the study team judged that the strength test would be done by taking samples from abandoned bridges of which year constructed is known, and by sending them to Japan to carry out tensile tests, hardness tests, contents analysis and micro scope observations for the purpose of classification of year constructed and its strength.

Also, further study was to be carried out in order to find a simple non-destructive test which is applicable in Sri Lanka to identify the material used in other old bridges.

12.2 Selection of Bridges for Taking Steel Sample

At first, 7 candidate bridges were selected through discussions with RDA. The samplings had to be carried out on the following bridges:

1) which were abandoned or removed for reconstruction to avoid any traffic problem;

2) of which construction year had to be identified; and

3) of which members had less fatigue effect.

Considering these conditions, a pre-inspection trip was carried out upon the candidate bridges to confirm their identification and then each member for samples to be taken.

Finally, 4 bridges were selected for the tests as follows:

Table 12.1 List of Bridges for Taking Steel Samples

No.	Name of Bridge	Year	Type	Members
2	Giriulla Bridge	1880	ST.TR (Pin Truss)	End Post, Web Plate
5	Kuruwita Bridge	1934	ST.TR (Rivet Truss)	Plate and Angle
6	Warakatota Bridge	1909	ST.TR (Rivet Truss)	End Post, Flange Plate
7	Matara Old Bridge	1860~1870	RSJ	End of girder

Table 12.2 (below) is a reference for 7 candidate bridges. The 4 bridges were selected out of them.

Table 12.2 List of 7 Candidate Bridges for taking steel samples

No.	Name of Bridge	Route No. Bridge No.	Year of Cons.	Description	Remarks
1	Dandugama Old Bridge	AA003 19/3km	1939	Colombo-Katunayake Rd. ST.TR (Rivet Truss) The bridge is on operation for light vehicles for resident.	Not applicable
2	Giriulla Bridge	B 322 38/3km	1880	Negombo-Giriulla Rd. ST.TR (Pin Truss) The bridge is now under reconstruction.	The samples can be taken from the members removed.
3	Yatiyantota Old Bridge	AA007 53km	1870~ 1880 Estimated	Avisamella-Hatton Rd. ST.TR (Pin Truss) The bridge is abandoned.	The samples can be taken.
4	Sitawaka Old Bridge	AA004	1887	23km from Avissawella ST.TR (Pin Truss) The bridge is abandoned. Difficult to take samples from Superstructure.	The samples can be taken from the lateral bracings of piers.
5	Kuruwita Bridge	AA004 87/1km	1934	Colombo-Ratnapura Rd. Old; ST.TR (Pin Truss) 1863 Reh; ST.TR (Rivet Truss) 1934 The bridge is now under reconstruction.	The samples can be taken from the members removed.
6	Warakatota Bridge	AA004 102/3km	1909	In Ratnapura town ARCH/S + ST.TR (Rivet) The bridge is abandoned.	The samples can be taken from ST.TR.
7	Matara Old Bridge	AA002 160/7km	1860~ 1870 Estimated	Colombo-Galle-Hambantota Rd. ST.TR (Rivet Truss) +RSJ The bridge is abandoned.	The samples can be taken from RSJ or ST.TR.

12.3 Methodology

The strength test was sublet to a Japanese steel consultant and carried out using the samples taken from 4 bridges as mentioned above.

The test consists of tensile test, hardness test, chemical analysis, and microscope examination.

(1) Tensile Strength Test

i) Preparation of Specimen

Five specimens, 3 for L-direction and 2 for C-direction were prepared in accordance with No. 5 specimens of JIS Z 2201 as shown in Fig. 12.1.

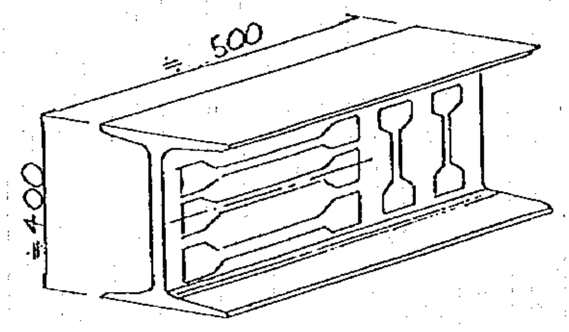
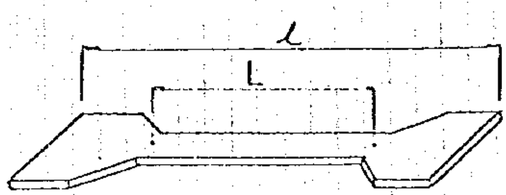


Fig. 12.1 Preparation of Specimens

All paints, rust and scaled surface were ground and finished in order to measure the thickness of section accurately.

The size of specimen is shown in Fig. 12.2.



$\ell \fallingdotseq 300$ mm
$L = 50$ mm

Fig. 12.2 Size of Specimen

ii) Tensile Strength Test

The test was carried out in accordance with JIS Z 2241.

(2) Hardness Test

Hardness test was carried out by 3 different methods, namely Brinell (JIS Z 2243), Rockwell (JIS Z 245), and Vickers (JIS Z 2244), all of which are used in general.

Measurement was also carried out on the specimens before the strength test was done.

Additional test was carried out using handy type hardness tester at the various stages of surface, such as rolled surface, ground surface and finished surface.

(3) Chemical Analysis

Chemical analysis was carried out to obtain contents of the elements such as S (Sulphur), P (Phosphorus), O (Oxygen), Carbon (C), Si (Silicon), Al (Alminium), Cr (Chromium), Mo (Molybdenum), and Mn (Mangan).

(4) Misroscope Examination

Microscope examination was carried out with magnification of 100 times and 500 times.

12.4 Results of Strength Test

For the actual test, ID numbers were put on materials from each bridge as follows:-

1) : Giriulla Bridge (1880)
2) : Kuruwita Bridge (1934)
3) : Warakatota Bridge (1909)
4) : Matara Bridge (1860 to 1870)

The following is the results of the strength test, and further details are in Appendix-K for references.

- Sample 1) is no doubt a wrought iron judging from chemical analysis, photomicroscopy and tensile tests.
- Sample 2) is thought to be a steel made by basic open-hearth, judging from construction date, high oxygen-concentration, and its application to bridges. It corresponds to the mild steel of JIS SS400 grade in tensile properties, and contains low sulfur and phosphorus contamination and has a homogeneous fine microstructure. It exhibits good properties as bridge steels.
- Sample 3) is a mild steel with poor properties for bridges. It is thought to be a Bessemer steel or steel made by earlier open-hearth judging from the construction date.
- Sample 4) contains low P, S and O, and higher YS (yield stress) than that of Sample 3), because of its fine grain structures. It is a ultra-mild steel made by acid open-hearth, judging from the stamp-mark: "SIEMENS MARTIN

ACID PROCESS". However, in the time of its construction, steels for bridges in those days had already been converted to higher strength steels by open-hearth (referred to the German Standard for Steel-Bridge Design, in which the recommendable steel after Jan.1, 1895 is strictly discriminated from wrought iron and steels before Dec. 31, 1894). Therefore Sample 4) must be an imported steel from Britain.

(1) Tensile Strength Test

Only Sample 2) exhibits sufficient strength as SS400-grade of JIS, but other samples fails in satisfying it. Sample 4) shows enough yield strength (YS), hence, the yield ratio (YS/TS) is higher than 0.9, but tensile strength (TS) is low. The tensile strengths of Sample 1), 3) and 4) exhibit 73 to 78% of that of Sample 2), and 84 to 89% of the specified minimum strength of JIS SS400. Any significant difference of strength between L- and C-directions is observed, except in Sample 1). In particular ductility in C-direction of Sample 1) is poor, because of too many inclusions.

No significant difference in Young's module of each sample is recognized.

(2) Hardness Test

The difference in hardness values between Sample 1) and Sample 2) is large in Brinell test, and in Rockwell test it becomes smaller, but in Vickers test no difference is recognized since too many massive inclusions in Sample 1) bring about difficulty in measuring accurately with Vickers test in which smaller dent is heavily affected by the inclusions. Also between Sample 2) and Sample 3)/4) the difference is discernible with every hardness test.

The ultrasonic hardness test shows the difference in hardness between Sample 2) and Sample 3)/4), only when the measurement is carried out on the machined surface, otherwise it was impossible to detect any difference.

(3) Chemical Analysis

Sample 1) shows the typical features of wrought irons, i.e., low concentrations of C, Mn and high P. The deviation of [O]-concentration measured was too large, on account of massive inclusions. Hence it was impossible to get a reliable value for them.

Sample 2) contains high concentration of C, but the concentrations of P and S are in accordance with the ranges of structural materials in JIS. Still, [O]-concentration is higher than that of modern steels of the grade.

Sample 3) and 4) are Rimmed steels, because they contain little Si and Al.

No trace of Cr and/or Mo was recognized in each sample.

(4) Microscope Examination

Sample 1) shows, again, the typical features of wrought irons, i.e., elongated massive silicate inclusions are observed in ferrite matrix.

Sample 2) shows the features of mild steels whose microstructures are ferrite-pearlite.

Sample 3) and 4) show ferrite-pearlite microstructures. Pearlite fraction in both samples is low, since [C]-concentrations are low. In the photomicrographs of Sample 3) coarser ferrite grains and more inclusions are observed.

12.4.2 Allowable Tensile Strength

In applying low strength steel like Sample 1), 3) and 4), it is designated that the design strength of wrought iron and steels by Bessemer process should be reduces by ca.20%, although that of modern mild steels can be treated as it is. these instructions are designated in the following:

1936 A.R.E.A "Rules for Rating Existing Iron and Steel Bridges"	
Axial tensile stress and bending stress;	
Open-hearth steels of 24000 lbs/in2-grade,	multiplying factor: 1.0
Bessemer steel of 21000 lbs/in2-grade,	multiplying factor: 0.88
Wrought iron of 20000 lbs/in2-grade,	multiplying factor: 0.83

Japan Railway Institute edi. "Prescriptions for replacement of civil structures"	
Wrought iron, Bessemer steel and steels before 1900,	multiplying factor: 0.75 (0.83)
S39, SS39 and steels of 1901 to 1905 incl. SS41,	multiplying factor: 0.9 (1.0)
SS41 and other steels after 1951,	multiplying factor: 1.0

> K. Tomonaga, "Maintenance and repair of railway steel-bridge"
> "the strength of wrought iron; tensile-, compression-, shear-strength, elastic stress limit and durability, should be regarded as approximately 80% of modern steels. Also for Bessemer steels these strengths should be regarded as 80 to 85%"

From the above literatures and the results of the present investigations, it is thought the allowable tensile strength of Sample 1), 3) and 4) should be reduced by 20% of its measured strength, although in Sample 2), the strength can be accepted without any reduction.

12.4.3 Identification of Steel Species by Ultra-sonic Hardness Test

It is needed to know the species of steel for maintenance and repair of steel bridges. For that purpose the sparkling-method was used as a simple method. However, it requires long experience to apply it since it owes person's sensibility.

On the other hand, Ultra-sonic hardness method is applicable to anyone regardless his experience and susceptibility. It is applicable simply by preparing surface of sample to flat polishing with rough grinding followed by fine one. It is recommended to finish the surface with whetstone. It should be noted that the Ultra-sonic method can not distinguish wrought iron from other kind of steel, because wrought iron contains too many massive inclusions. Wrought steel can be discerned from other steel, because it exhibit quite a different scale appearance on account of massive inclusions. As shown in photograph attached in the Appendix-K the scale of wrought iron appears as a layered morphology, eventually including layered cracking. Therefore, it is easy to discriminate wrought iron from other steels.

12.5 Consideration

12.5.1 Classification of Steel Material

Steel materials used in steel bridges can be classified into two major groups depending on their year of construction and allowable tensile stress as follows:

Table 12.3 Classification of Steel Material

Year of Construction	Steel Material	Allowable Tensile Stress
before 1930	Wrought iron or Low quality mild steel	80% of SS 400 (yield stress = 24kgf/mm2)
after 1930	Mild steel	equivalent to SS400

12.5.2 Simple Method to Classify the Steel Material

In order to confirm the strength of the steel material used in other old steel bridges, the field test using the handy type ultrasonic hardness tester is a useful simple non-destructive method. It could be found whether the material is low quality mild steel/Bessemer steel or mild steel. The latter shows the hardness higher than the former does.

However, this simple method cannot be applied for wrought iron because of huge nonmetallic inclusion in the wrought iron. The wrought iron rusts differently from other steel materials. The rust have layers or layer crack because of its inclusion. Thus, the wrought iron can be classified by the visual inspection.