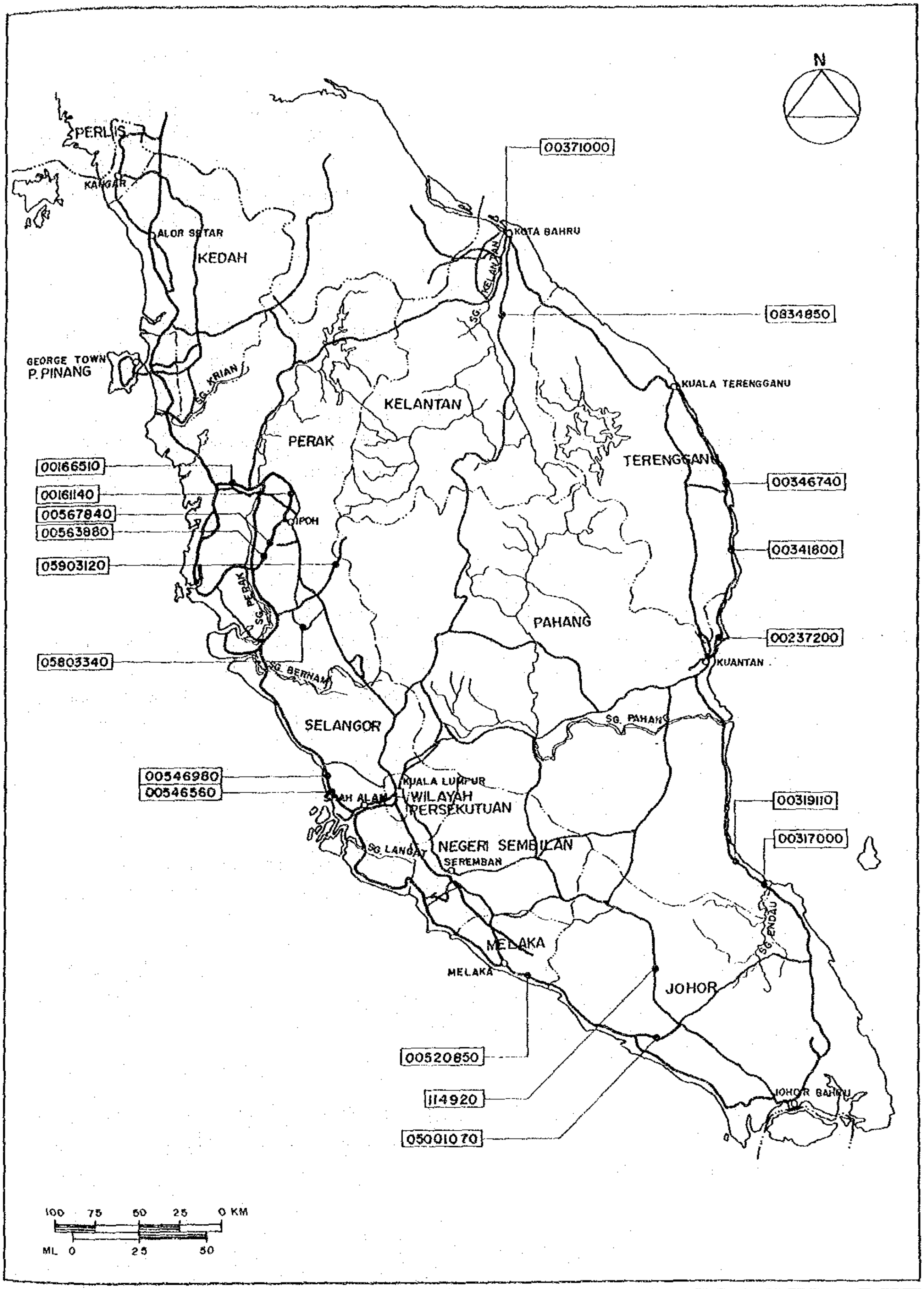


Figure 6-1 Location Map of the 18 Bridges for Topographic Survey

6.3 Methodology

The topographic survey work was subletted to a local licensed surveyor and commenced on the 26th September, 1991 for a contract period of 45 days. Upto six surveying parties were mobilized at different stages during the survey works. The field operations were divided into reconnaissance, traversing, leveling, EDM Tachometry and Echo Sounding. A brief description of each exercise is given below.

6.3.1 Reconnaissance Survey

Prior to the actual field work, a reconnaissance survey was carried out to confirm the exact locations of the respective bridge sites including the acquiring of relevant Certified Plans (CPs) from the State Survey Departments. These CPs are required in order to obtain the necessary lot boundary information near the bridge sites so as to enable the surveyor to perform connections or tie the bridge locations to the existing boundary marks. The physical locations of nearby government benchmarks as well as their integrity were also checked during this reconnaissance survey.

6.3.2 Traversing

Traversing was carried out to provide horizontal control for the bridges sites. All nearby existing boundary marks were tied to the traverse. In addition, a series of ground control pegs near the bridges were also established for the Electronic Distance Measurement (EDM) Tachometry Survey. All distances were measured using EDM while bearing datums were derived either from Azimuth of the sun or bearing from the Certified Plans computed from proven boundary marks.

6.3.3 Leveling

Height control was carried out by spirit leveling from at least two reliable and proven Survey Department benchmarks to the control traverse pegs and temporary benchmarks (TBMs) were established at the bridge site.

The TBMs were installed out of one meter length of 40mm diameter galvanized iron pipes set in concrete of dimensions 30 cm x 30 cm x 30 cm. The pipe was filled with cement mortar with a stud affixed onto the mortar at the top of the pipe and the TBM number was written on the concrete.

The accuracy of leveling was within tolerance of $\pm 0.02 \sqrt{K}$ (where K is in kilometers of line surveyed). The bench marks installed and the levels for the TBMs at the respective bridge sites are given in Table 6-2.

6.3.4 EDM Tachometry

EDM Tachometry was carried out to ascertain the bearing and distance of each point in detail including the spot height. The accuracy of this method was ± 5 centimeters in the X, Y direction and better than ± 2 centimeters in height. The interval for spot heights of points surveyed was selected on the basis of the final plotting scale. This method was also employed to determine the cross-sections of small and shallow rivers at the bridge sites.

Table 6-2 TBM Levels and Referenced Benchmarks

Bridge No.	District	State	BM No.	TBM No. Level (Meter)
00114920	Segamat	Johor	J0021	37.289
00161140	Kinta	Perak	A0210	72.92
00166510	Larut Matang	Perak	A0248	34.462
00237200	Kuantan	Pahang	C0516	3.882
00317000	Rompin	Pahang	C0220	2.919
00319110	Rompin	Pahang	CM129	6.356
00341800	Kemaman	Terengganu	T0115	3.535
00346740	Dungun	Terengganu	T0140	6.001
00520850	Jasin	Melaka	M0241	4.341
00546560	Kuala Selangor	Selangor	B0819	2.292
00546980	Kuala Selangor	Selangor	B0816	2.899
00563880	Manjung	Perak	A0606	2.703
00567840	Kinta	Perak	A0659	20.866
00834850	Kuala Krai	Kelantan	D0057	27.237
05001070	Batu Pahat	Johor	J0294	5.791
05803340	Batang Padang	Perak	A0846	26.902
05903120	Batang Padang	Perak	A1416	573.500
00371000	Kota Bharu	Kelantan	D0027	n/a

Notes :

It should be noted that at bridge site No. 00371000 there were two existing benchmarks at both ends of the bridge which fall within the survey area and were used in the height control. As such, no TBM was established at this bridge site.

Level datum for the bridge site No. 00319110 was based on the Benchmark established by the Public Work Department (Jabatan Kerja Raya) for the road maintenance and rehabilitation programme. The TBM and benchmark number is CO228 and CM129 respectively.

6.3.5 Echo Sounding

River bed profiling using the Echo Sounding technique was carried out at four bridge sites, namely bridge Nos. 00371000, 00346740, 00317000 and 00319110.

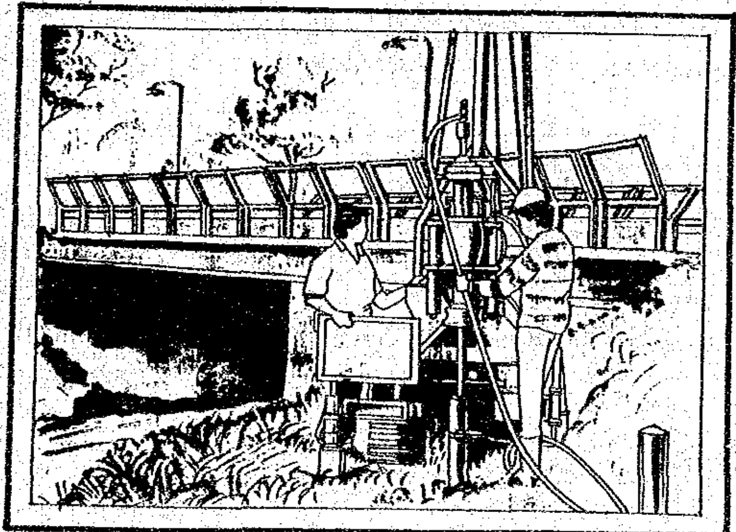
6.4 Results of Topographic Survey

All the field works were completed on the 31st October, 1991. The field books were checked and the recorded data were processed by a computer. The X, Y, Z coordinates of all recorded points were plotted using a flatbed plotter. The results of the survey were prepared in the following form :-

* Topographic Drawings (18 nos)
* River Crossings (42 Sections)
* Bridge Profile along the center line including river cross-section below the bridge center-line (18 sections).

CHAPTER 7

SUBSOIL AND WATER INVESTIGATION



CHAPTER 7

SUBSOIL AND WATER INVESTIGATION

7.1 General

The data collection exercise carried out was futile as far as the availability of subsoil information is concerned. In view of the non-availability of subsoil data, a geotechnical investigation was carried out at selected bridge sites as part of the detailed engineering survey.

A total of 5 out of the 20 bridges selected for detailed survey were found to require subsoil investigation. This is in view of the possibility of future rehabilitation plans such as the addition of sidewalks, widening of carriageway and raising of grade as well as the reinforcement of foundations. One deep borehole was sunk at each of the selected bridge sites. Their locations are shown in Figure 7-1.

In addition to the subsoil investigation at the 5 selected bridges, a river water quality survey and topsoil sampling with regard to possible sulphate were also being carried out. This exercise was carried out to confirm the NALS findings concerning the possibility of sulphate attack at a number of bridges. The locations of those bridges suspected of high degree of sulphate attack are also shown in Figure 7-1. Ten of these bridges were finally shortlisted for the sulphate survey, involving hand auger holes and river water samplings.

7.2 Objectives

The principal objectives of subsoil investigation are to determine the representative stratigraphy of the subsoil and ground water conditions at the selected bridge locations. Important subsoil engineering parameters were obtained from this investigation and adopted for use in the analysis, design and selection of foundation types. Furthermore, the safe bearing capacity of the soil or pile, the lateral load carrying capacity of pile, settlement and slope stability analysis can all be carried out if necessary.

Chemical analysis of the pH value and sulphate content in the top soil and river water samples were carried out to ascertain the concentration as well as the possible sources of these chemicals. These chemical test results will help to scrutinize the NALS findings and possibly unravel some of the causes of chemical attacks on the concrete. This will in turn help to determine the appropriate course of rehabilitation works to be taken. Immunity against further deterioration can then be given to a varying degree by protective measures.

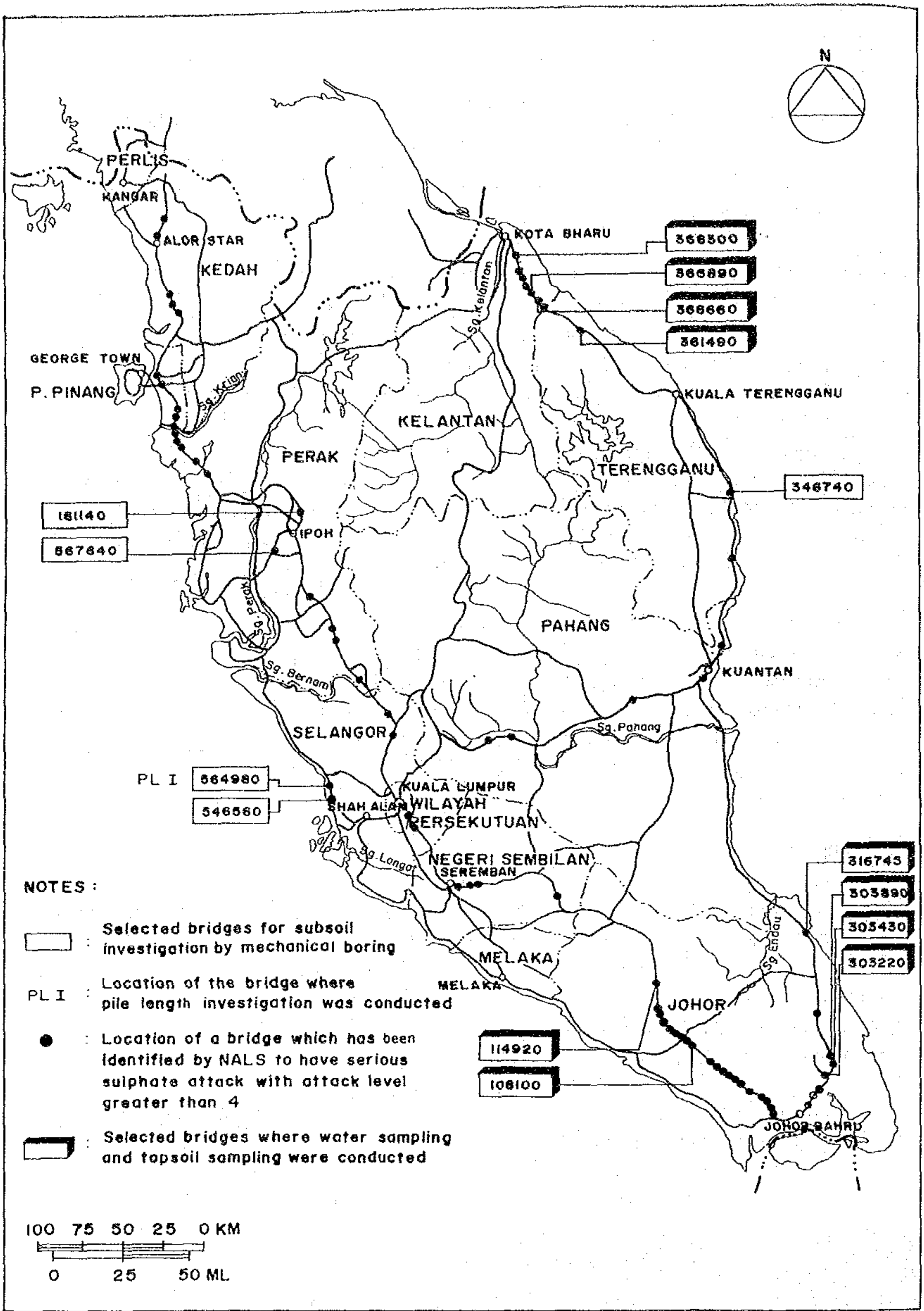


Figure 7-1 Location of Bridges where Subsoil Investigation was conducted

7.3 Scope of Works for Subsoil Investigation

One deep borehole was sunk at each of the selected 5 bridge locations as shown in Figure 7-1. The exact layout of the boreholes are given in Appendix-J. This subsoil investigation was carried out by a selected geotechnical consultant under the supervision of the JICA Study Team. The actual bill of quantities for the subsoil investigation is tabulated in Table 7-1. In general, the subsoil investigation is comprised of the following:-

* Rotary wash boring on land to a maximum length of 45.5 metres.

* Associated field tests such as the Standard Penetration Test (SPT), including the collection of disturbed samples for visual examination and laboratory testings.

* Undisturbed soil sampling using stationary piston samplers.

* Ground water and river water sampling.

* Laboratory tests to obtain soil index properties and chemical analysis of soil and water samples.

* Pressuremeter tests and pile length investigation using P/S Wave logging at bridge No. 00546980 only.

7.4 Scope of Works for Topsoil Investigation

This phase of the investigation mainly involved river water sampling as well as topsoil exploration using hand augers. A total of 10 bridge sites were selected based on NALS preliminary findings where sulphate is a possible source of chemical attack on the concrete. The list of bridges together with the bill of quantities for the topsoil investigation is given in Table 7-2.

Three hand auger holes were carried out at each bridge site using an auger of approved design to ASTM D 1452-65, 1972, "Soil Investigation and Sampling by Auger Boring". The exact locations of the auger holes and positions where water samples were collected are given in Appendix-J.. A selected soil sample from each auger hole was tested for pH value, total sulphate and water soluble sulphate content.

Two river water samples were also retrieved from each bridge site, one of which was obtained from the up-stream area. The water samples were tested for pH value and water soluble sulphate.

Table 7-1 Quantities for Subsoil Investigation at 5 Selected Bridges

SOIL INVESTIGATION AND TEST	DESCRIPTION		UNIT	NO OF SUBSOIL SURVEY/TEST FOR EACH OF THE 5 BRIDGES					TOTAL
				BRIDGE NO 00161140 PERAK	BRIDGE NO 00346740 TRENGGANU	BRIDGE NO 00546560 SELANGOR	BRIDGE NO 00546980 SELANGOR	BRIDGE NO 00567840 PERAK	TEST/ SUBSOIL SURVEY
ROTARY WASH BORING			m	4.6	18.06	45.45	45.45	14.55	128.05
FIELD	Pressuremeter Test	5m	No	–	–	–	1	–	1
	at various depth	10m	No	–	–	–	1	–	1
TEST	SPT test	0 – 20m	No	1	12	13	8	8	42
	at various depth	20 – 50m	No	–	13	17	15	–	45
AND	Water	Ground	No	1	1	1	1	1	5
	Sampling	River	No	1	1	1	1	1	5
SAMPLING	Undistured Soil Sampling		No	–	–	8	8	–	16
LABORATORY TEST ON SOIL	Soil Property	Liquid & Plastic Limits	No	–	–	8	7	–	15
		Specific gravity	No	–	–	9	7	–	16
		Unit weight	No	–	–	9	8	–	17
		Water content	No	–	–	9	8	–	17
	Index	Grain size Analysis	No	–	–	8	7	–	15
	Soil Strength or Mechanical Property	One-dimentional Consolidation test	No	–	–	8	6	–	14
		Unconsolidated-undrained triaxial compression test	sets	–	–	8	7	–	15
	Chemical Test	pH test	No	1	6	8	9	3	27
		Total Sulphate	No	1	6	8	9	3	27
		Water soluble suplhate	No	1	6	8	9	3	27
LABORATORY TEST ON WATER	Chemical Test	pH test	No	2	2	2	2	2	10
		Water soluble sulphate	No	2	2	2	2	2	10
POSSIBLE REHABILITATION PLANS				ADDING SIDE WALK	RAISING GRADE	RAISING GRADE	ABUTMENT FOOTING OR FOUNDATION	WIDENING	

Table 7-2 Quantities for Soil and River Water Quality Sampling and Testing

BRIDGE NO.	STATE	No. of Sampling / Testing				
		Soil Sample			Water Sample	
		pH Test	Total Sulphate	Water Soluble Sulphate	pH Test	Water Soluble Sulphate
00108100	Johor	3	3	3	2	2
00114920	Johor	3	3	3	2	2
00303220	Johor	3	3	3	2	2
00303430	Johor	3	3	3	2	2
00303890	Johor	3	3	3	2	2
00316745	Johor	3	3	3	2	2
00361490	Terengganu	3	3	3	2	2
00366660	Kelantan	3	3	3	2	2
00366890	Kelantan	3	3	3	2	2
00368300	Kelantan	3	3	3	2	2

7.5 Results of Subsoil Investigation

The field work for the subsoil investigation commenced on the 23rd September, 1991 and was completed on the 16th October, 1991. Laboratory tests were completed in early November, 1991. A compilation of the borelogs and laboratory tests results are given in Appendix-J.

A summary of the existing and/or recommended foundation system is given in Table 7-3.

7.5.1 Assessment of Foundation Failure at Bridge No. 00546980

The present foundation of both abutments at bridge No. 00546980 are in distress. This abutment is supported by a row of 5 Nos. 381 x 381 mm precast concrete piles. It is obvious that the crosshead and the abutment piles have tilted inwards by as much as 8° from the vertical, with longitudinal cracks visible on some piles.

Table 7-3 Summary of the Existing and/or Recommended Foundation System for the 5 Selected Bridges where Subsoil Investigation was carried out

BRIDGE NO. / STATE	EXISTING AND / OR RECOMMENDED FOUNDATION SYSTEM
161140, Sg. Chemor Perak	Spread Footing with a safe allowable bearing capacity of 500 kPa founded on fresh limestones.
346740, Sg. Dungun Terengganu	Piled Foundation with R.C. cast in place concrete piles of 900 mm diameter. The piles shall be driven through the loose silty sand stratum terminating at the weathered sedimentary rocks level. The penetration length of piles are expected to be in the range of 16 to 20 metres with an allowable axial working load of 250 to 300 tonnes each.
546560, Sg. Jeram Selangor	Piled Foundation with 300 x 300 mm driven precast concrete piles. The piles shall be driven through the thick marine clay layer terminating in the medium dense silty sand and/or very stiff silty clay stratum. The penetration length of piles are expected to be in range of 37 to 40 metres with an allowable axial working load of 40 to 50 tonnes. In view of the deep deposits of highly compressible marine clay, it is anticipated that the piles will experience some downdrag due to negative skin friction if the clay stratum is subjected to additional fill. In such instant, the pile capacity shall be downgraded accordingly. In view of the high sulphate content detected at the vicinity of the bridge, it is advisable to use low water: cement ratio concrete or sulphate resisting Portland cement for all concrete works including the construction of the piles.
546980, Sg. Buloh Selangor	Piled Foundation with 381 x 381mm driven precast concrete piles. The piles shall be driven through the thick marine clay layer terminating in the very stiff sandy / clayey silt stratum. The penetration length of piles are expected to be in the range of 42 to 45 metres with an allowable axial working load of 60 to 70 tonnes. In view of the deep deposits of highly compressible marine clay, it is anticipated that the piles will experience some downdrag due to negative skin friction if the clay stratum is subjected to additional loads. In such instant, the pile capacity shall be downgraded accordingly. In view of the high sulphate content detected at the the vicinity of the bridge, it is advisable to use low water: cement cement ratio concrete or sulphate resisting Portland cement for all concrete works including the construction of the piles.
567840, Sg. Selinsing Perak	Piled Foundation with 381 x 381mm driven precast concrete piles terminating at the granite bedrock level. The penetration length of these end bearing piles are approximately 13 to 15 metres from the existing ground level with an allowable axial working load of 70 to 80 tonnes each.

In order to identify reason(s) for the foundation failure, pressiometer tests and pile length measurement were carried out using the borehole.

(1) Pressiometer Tests

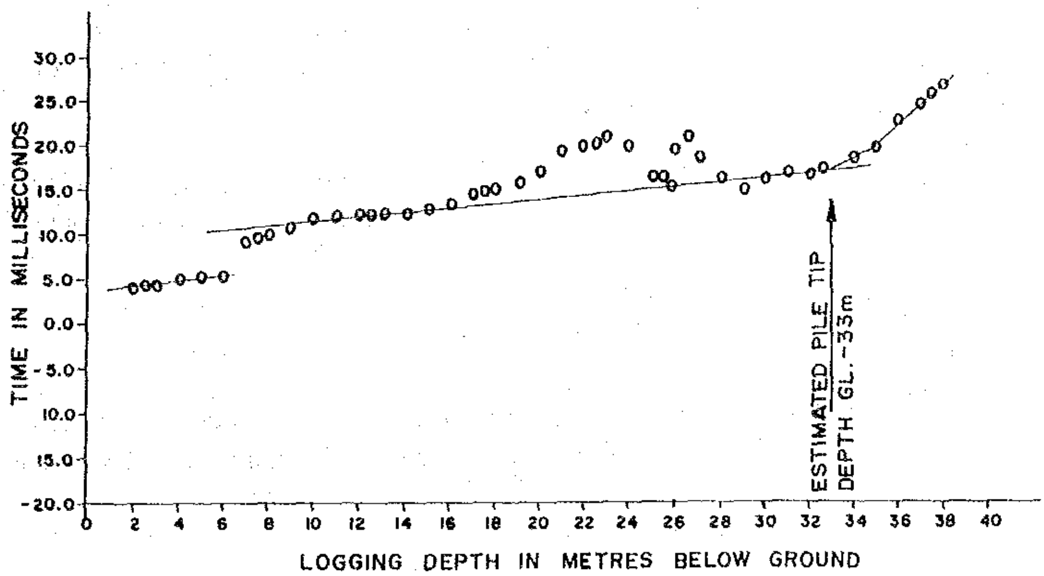
Two pressiometer tests were carried out within the borehole at depths of 5 and 10 metres below the existing ground level. The results are tabulated below while the pressiometer curves are given in Appendix-J.

Important Parameters	Depths Below Ground Level	
	5 metres	10 metres
Deformation Modulus (Ep) kN/m2	1300	960
Pressure at rest (Po) kN/m2	9	5
Yield Pressure (Pf) kN/m2	79	34
Limit Pressure (Pl) kN/m2	137	60

(2) Pile Length Test

A downhole P/S Wave logging test was also carried out to determine the length of the existing precast concrete abutment pile. Analysis result of the travel time arrival is shown in Figure 7-2. As seen from Figure 7-2 it is predicted that the pile tip is about 33m below the existing ground level or 31.5m below the pile cap level.

Figure 7-2 Travel Time Curve of Pile Length Test



(3) Assessment of Foundation Failure

The distress in the abutment foundation is probably due to a combination of the following three factors:-

- The piles did not have enough bearing capacity in the original design.
- The above condition could have been aggravated by negative friction due to consolidation settlement of the soft marine clay. These factors most likely caused the vertical settlement of the piles.
- The consolidation settlement also induces lateral soil movement and this lateral force could result in bending of the piles and accompanied by rotation of the abutment.

Based on the pile length test results, the pile tip is actually bearing on the stratum of loose to medium sand with SPT N-values ranging from 6 to 22. From theoretical calculation using pressiometer test results, the pile has an allowable bearing capacity of about 62 ton, while maxima vertical force due to the dead load of the superstructure and live load is 68 ton per a pile. Thus it is conclusive that the pile has slightly inadequate bearing capacity in the original design.

Above the design bearing stratum is mainly 27 meters of soft marine clay with zero N-value. Considering the highly compressible nature of the marine clay overlaid by 3.4 metres of fill, it is expected that downdrag due to the consolidation settlement is imposed on the abutment piles.

Furthermore, the embankment behind the abutment acts as an unsymmetrical surcharge on the foundation subsoil, inducing a lateral soil movement in the marine clay stratum. This soil movement may result in the bending of the piles and accompanied by rotation of the abutment.

Even though the piles may be able to carry the present vertical load, it is prudent to consider urgent rehabilitation measures to support the superstructure in view of bridge safety and durability of the bridge in the long term.

7.6 Results of Topsoil Investigation

Topsoil exploration using hand augers and river water sampling were carried out at the 10 selected bridge sites in order to identify the possible causes of chemical attacks on concrete.

Field work for topsoil investigation commenced on 9th October 1991 and was completed on the 16th October 1991. Soil samples were collected at three hand auger boring positions and two river water samples were also taken from each bridge site. The locations for the hand auger holes and river water samplings are given in Appendix-J. A brief description on the general geology and subsoil conditions found at each bridge site are summarized in Table 7-4.

Table 7-4 Summary of General Geology and Subsoil Conditions at 10 Bridges Sites where the Sulphate Survey was carried out.

Bridge No.	Brief General Geology	Description of Subsoil Conditions Based on Hand Augers Holes *
108100 Sg. Machap Johor	Alluvial plain underlain by lower Triassic calcareous sedimentary rock strata with possible granitic intrusives of the lower and upper Triassic age.	Alluvial deposits are encountered at hand auger points P-1 and P-2 consisting of soft brownish grey to light grey silty clay with some organic matters. The basic sedimentary rock formation was not encountered within the hand auger boring depth (1.2m). Fill material consisting of brown to light grey silty clay with some lateritic gravels and fibrous roots was encountered at the top 0.55 metres at point P-3. This fill is further underlain by similar alluvial deposits found in P-1 and P-2.
114920 Sg. Karas Johor	Similar to the above	Fill material (0.58m thick) consisting of brown silty clay with some roots was encountered at P-2. Alluvial deposits of brown sand to grey silty clay with organic matters are generally found at the other hand auger holes.
303220 Sg. Redan Johor	This area where acid and undifferentrated granitic rocks formation is predominantly found underlying residual soils	Fill material (0.85m thick) consisting of residual soil of brown silty clay was encountered at P-3. Underlying this fill layer are alluvial deposits consisting of brownish to yellowish silty clay with some organic matters. At P-1 and P-2, residual soil consisting of yellowish brown silty clay can be found.
303430 Sg. Berangan Johor	Area is slightly hilly and undulating consisting of residual soils of granitic bedrocks	Fill material with thickness ranging from 0.6m to more than 1.1m are found in all 3 hand auger holes. The fill material is a yellowish brown to brownish grey silty clay (residual soil). Alluvial deposits were encountered at P-1 at 0.6m below ground surface level. The alluvial deposits consist of light grey to dark grey silty clay with some organic matters.
303890 Sg. Tembioh Johor	Similar to the above	Fill and alluvial deposits were found at the bridge site. The fill material is a brown silty clay/clayey silt with some sand and is probably a residual soil of granitic origin. Alluvial deposits were encountered at the ground surface at P-1 and below the fill (0.4m) stratum at P-3. This soil material is composed of brown to dark grey soft silty clay with roots, decayed wood and organic matters. P-2 which is 1 m deep is entirely within the fill stratum.

continued . . .

Bridge No.	Brief General Geology	Description of Subsoil Conditions Based on Hand Augers Holes *
316745 Sg. Air Tawar Johor	The site is located within the alluvial plain near the east coast of northern Johor. Granitic bedrock of the Permian period underlies the alluvial deposits.	Fill material was found throughout hand auger P−2 and P−3 which is 0.45m and 0.95m, respectively. Alluvial deposit consisting of grey loose silty sand and soft silty clay can be found at P−1. Huge granitic boulders were found throughout the vicinity of the bridge site. These boulders were probably imported into the area during the construction of the bridge.
361490 Dungun District Terengganu	Alluvial deposits overlying granitic rock formation.	Fill materials consisting of silty clay / clayey silts of more than 1m thick were found in all 3 hand auger holes. Alluvial deposits of light brownish grey silty clay with sand and fine gravels were encountered at 1.1m below the ground surface at P−1.
366660 Kelantan	This site is located within the alluvial plain in the east coast of Peninsular Malaysia. Granitic rock formations are predominant towards the west direction from the bridge site.	Fill is generally encountered overlying the alluvial deposits. Alluvium consisting of yellowish to greyish brown silty sand was found at depths 0.65m and 0.2m below ground level at P−1 and P−2, respectively. No alluvial soil was encountered at P−3 which was terminated at 1.1m depth.
366890 Kelantan	Similar to the above	Alluvial deposits were found in all hand auger holes from the ground surface. Silty sand belonging to the older alluvium (Pleistocene period) was encountered at a depth of 0.6m at P−3. The alluvium in this region is generally clayey silt, sandy silt and silty sand.
368360 Kelantan	Similar to the above	Fill materials ranging from 0.2 to 0.4m were found overlying the alluvial deposits. The alluvium is generally yellowish brown to light grey silty sand or brownish clayey silt/silty clay.

* Note : P−1, P−2 and P−3 refers to hand auger boring points carried out at the respective bridge sites.
Please refer to Appendix−J for the exact hand auger locations.

Selected soil and river water samples collected from each of the bridge sites were tested in an approved laboratory for pH value, total sulphate and water soluble sulphate content. In addition, ground water and selected soil samples collected from the 5 bridge sites during the subsoil investigation were also tested. A summary of the results for the chemical analysis were tabulated in Table 7-5. Detailed results for each of the selected soil and water samples were given in Appendix-J.

Table 7-5 Summary of Chemical Tests Results

	pH-Value			SULPHATE CONTENT			
				Soil Sample		Water Sample	
Bridge No.	Soil Sample	Water Sample		Total SO3 (%)	Water Soluble (g/l)	River (g/l)	Borehole (g/l)
		River	Borehole				
00108100	4.1-6.6	5.6-5.7	-	<0.01	<0.01	<0.01	-
00114920	4.1-5.0	6.1-6.3	-	<0.01	<0.01	<0.01	-
00303220	4.0-4.4	5.4-5.5	-	<0.01	<0.01	<0.01	-
00303430	4.2-5.0	6.0-6.1	-	<0.01	<0.01	<0.01	-
00303890	4.2-4.9	5.7	-	<0.01	<0.01	<0.01	-
00316745	3.2-4.5	4.3-4.4	-	0-1.47	0-3.2	<0.01	-
00361490	4.6-5.3	5.3-5.4	-	0-0.04	0-0.10	<0.01	-
00366660	4.0-6.2	5.9-6.0	-	<0.01	<0.01	<0.01	-
00366890	4.7-5.1	5.0-5.1	-	<0.01	<0.01	<0.01	-
00368300	4.8-5.1	5.3-5.4	-	<0.01	<0.01	<0.01	-
00161140*	6.2	6.4	6.9	<0.01	<0.01	<0.01	<0.01
00346740*	6-8.5	5.3	5.5	0-0.04	0-0.15	<0.01	<0.01
00546560*	4.5-7.9	6.2	5.7	0-1.52	0-4.3	0.12	0.82
00546980*	5.5-7.8	5.4	6.8	0-1.1	0-2.8	0.2	0.15
00567840*	4.9-7.5	6.8	7.0	<0.01	<0.01	0.02	0.04

Note: * means bridge where mechanical boring was conducted.

7.6.1 Evaluation of Chemical Test Results

(1) pH-Value

The pH value is the most important indicator to determine the acidic condition of soil and water. Acids present in the soil or water can be very aggressive and harmful to concrete. Acidic soils including acid sulphate soils can be found throughout much of the Peninsula Malaysia, especially in coastal plains and residual soils from sedimentary rocks of marine origin. Naturally occurring organic acids are mainly lignic or humic acids which are generally found in peaty soils and waters. These acids form insoluble calcium salts by reaction with free lime of normal Portland cement. In certain swampy peats, oxidation of pyrites or marcasite can produce free sulphuric acid which is highly aggressive to concrete. In general, the presence of free sulphuric acid is indicated by pH values lower than 4.3 and a high sulphate content (Tomlinson,1986).

There is as yet no universally accepted system for the classification of aggressiveness of acids on concrete. However, a number of classification systems have been published and used by different agencies and researchers. A few of the commonly used systems are given in Table 7-6 and 7-7 below. The pH value of the selected soil and water samples were evaluated and a rating on the aggressiveness of the acids at each bridge site based on the above classification systems are summarized and tabulated in Table 7-8.

Table 7-6 Classification on the Degree of Aggressiveness of Acidic Water on Concrete (Source: German Standard DIN 4030:1969 and Building Research Establishment Digest No.174)

Classification on Degree of Aggressiveness	pH Value according to	
	DIN 4030	B.R.E. Digest No.174
Negligible	Above 6.5	9.0 to 6.0
Moderate	6.5 to 5.5	6.0 to 3.5
High	5.5 to 4.5	
Very High	Below 4.5	Below 3.5

Table 7-7 Classification on the Degree of Aggressiveness of Acidic Soils on Concrete (Source : S.R. Doran et. al, 1987)

Class	Description	pH Value
1	Not Aggressive	6.0 to 9.0
2	Mild	5.5 to 6.0
3	Moderate	4.5 to 5.5
4	High	3.5 to 4.5
5	Very Severe	less than 3.5

As can be observed from Table 7-8 , the 10 bridge sites included in the Sulphate survey are classified as "high" to "very severe" in terms of acid attack aggressiveness based on the pH values of the soil samples. However, based on the pH-Values of water samples, the aggressiveness is categorized as only "moderate" to "high" except at bridge No. 00316745. If acids from industrial sources can be ruled out, then the next most probable source of acids may be from the leaching effect (exchanging of cations around the clay minerals by hydrogen ions) at most of the bridges and /or the presence of humic acid at some bridge sites.

At the 5 bridge sites where subsoil investigation was carried out, the aggressiveness of acid attack is generally classified at "negligible" to "moderate".

Table 7-8 Summary of Ratings on the Aggressiveness of Acids at Each Bridge Site

Bridge No.	Classification on Aggressiveness of Acids Based on Table 7-6 and 7-7		Subsoil Conditions
	Soil Samples	Water Samples (per DIN)	
00108110	Not aggressive	Moderate	Fill Stratum
	Moderate to V. Severe		Alluvium
00114920	High to Very Severe	Moderate	Refer to Table 7.4 for the Description of Subsoil Conditions.
00303220	Very Severe	High to Moderate	
00303430	High to Very Severe	Moderate	
00303890	High to Very Severe	Moderate	
00316745	Very Severe	Very High	
00361490	High	High	
00366660	Not aggressive to Very Severe	Moderate	
00366890	High	High	
00161140*	Not aggressive	Negligible to Moderate	
00346740*	Not aggressive	High	
00546560*	Not aggressive to High	Moderate	Marine Clay
00546980*	Not aggressive	Negligible to Moderate	Marine Clay
00567840*	High	Negligible	Alluvium
	Not aggressive		Residual Soil

Note: * means bridge where mechanical boring was conducted.

(2) Sulphate Content

Among various acids in water and soil, sulphate is the most aggressive to concrete. Approximately 25 percent of the bridges involved in NALS were found to suffer from chemical attacks due to certain destructive agents. Although, the exact type and origin of the chemicals are unknown, it was hypothesized that sulphates produced as sulphuric acid by bacteriological action on iron pyrites in stagnant or low flow conditions in streams may be the root of the problem.

Sulphates in solution will react with tricalcium aluminate in Portland cement to form insoluble calcium sulphate and calcium sulpho-aluminate. Crystallization of the new compounds known as "Ettringite" is accompanied by an increase in molecular volume which causes expansion and disintegration of the concrete at the surface. This disintegration exposes fresh areas to attack and if there is a flow of water bringing fresh sulphates to the affected area, the rate of disintegration can be very rapid.

The chemical test results presented in Table 7-5, indicate that the sulphate content in both the soil and water samples are very low except in those samples collected from bridges No. 00316745, 00546560 and 00546980. At most of the bridge sites, the sulphate contents were generally insignificant, with less than 0.01% and 0.01 grams/litre of total sulphate and water soluble sulphate content, respectively.

A summary table showing the degree of aggressiveness of the sulphates at each bridge site is given in Table 7-9. The class or category of sulphate attack is in accordance to the British Standard, B.S 8004:1986 which is attached in Appendix-J.

Table 7-9 Summary of the Degree of Aggressiveness of Sulphate Attack Based on B.S. 8004:1986

Bridge No.	Classification on Aggressiveness of Sulphate Attack Based on B.S. 8004		Subsoil Conditions
	Water Soluble Sulphate in Soil Sulphate	Water Samples	
00108110	1	1	Refer to Table 7.4 for the Description of Subsoil Conditions
00114920	1	1	
00303220	1	1	
00303430	1	1	
00303890	1	1	
00316745	1	1	
00361490	1 to 4	1	
00366660	1	1	
00366890	1	1	
00161140 *	1	1	See Table 7−3
00346740 *	1	1	See Table 7−3
00546560 *	1 to 4	1	Marine Clay
00546980 *	1 to 3	1 to 2	Marine Clay
00567840 *	1	1	See Table 7−3

Note : * means bridge where a mechanical boring was conducted

(3) Conclusion

The chemical analysis results as summarized in Table 7-5 and interpreted in Table 7-9, clearly contradicts the NALS hypothesis regarding sulphate attack on concrete for most of the bridges under the present study. In general, the sulphate content in both the soil and water samples are very low except at one bridge site (No. 00316745) which is located in the coastal plains.

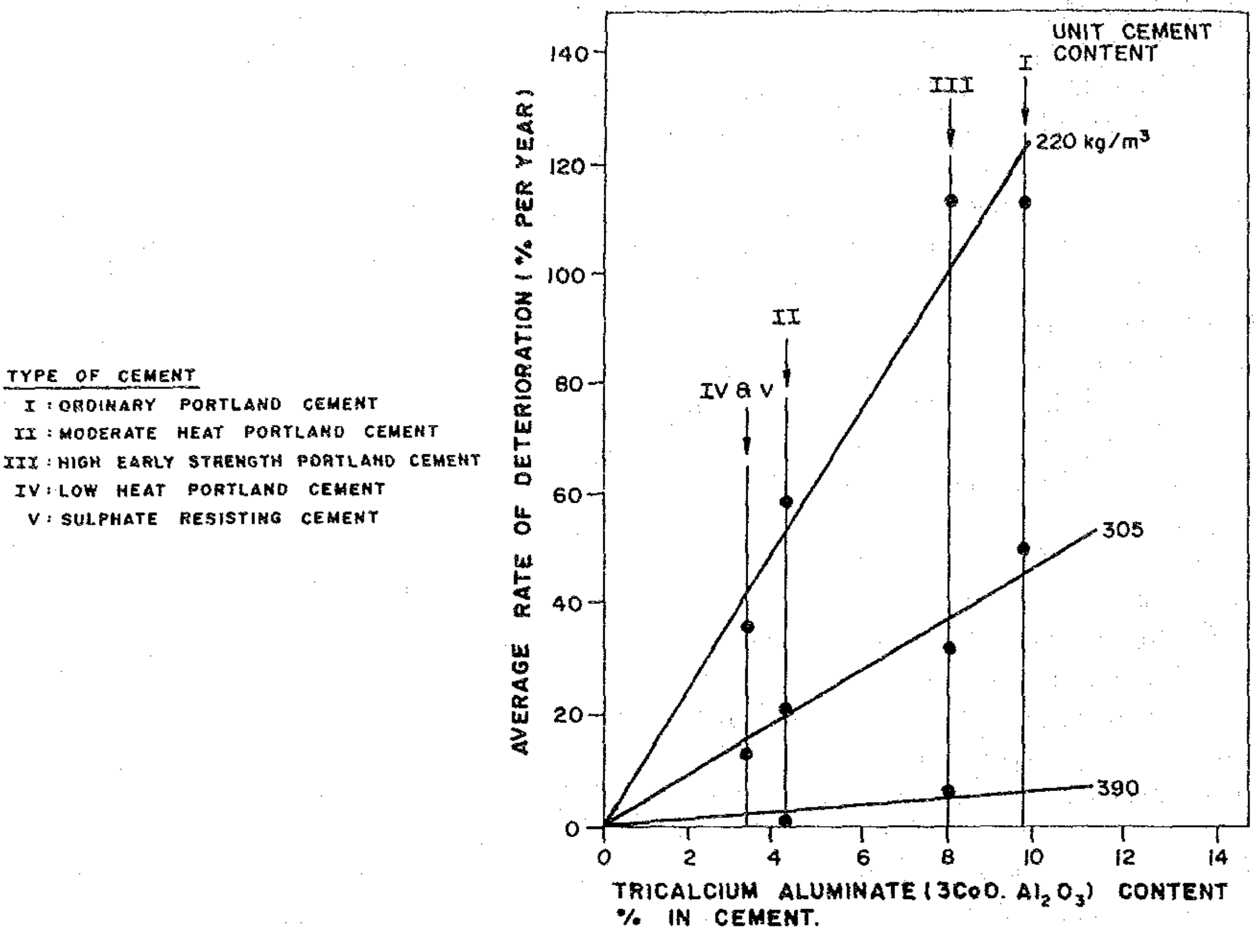
Although, the sulphate contents were found to be negligible at most bridge sites, it was clear that the cement matrix had been abraded by some form of a chemical attack. This distress, mainly in the pile concrete, could have been caused by a combination of the following two factors:-

- Presence of aids in the vicinity of the bridge sites
- High water - cement ratio in the concrete

As pH values are only indicators of the concentration of H^+ ions and not the types of acid, it is therefore difficult to ascertain the sources of the acids at each of the bridge sites. However, if acids from industrial sources can be ruled out, then the probable sources of acids may be from the leaching effect in the clay minerals, the oxidation of pyrites in the soil or the presence of humic acids which are generally found in peaty soils and waters. Acidic soils can also be found in coastal areas and residual soils from sedimentary rocks of marine clay.

In view of the nature of the distress, it is proposed that the type of concrete mix to be applied in the remedical works be low water-cement ratio and the unit cement content be more than 300kg/m^3 using ordinary portland cement with appropriate additive agent, based on the following reason. Figure 7-3 which is reproduced from an article (JCI No. 144, Vol. 15, No. 9) published by the Japan Concrete Institute (1977) shows the relationship between the rate of deterioration of concrete under sulphate/acid attacks for different types of cement. Sulphate resisting Portland Cement (Type V) complying with BS 4027 has a low proportion of tricalcium aluminate which is ideally suited for use in aggressive environments. However, as this cement is costly and not commonly available in Malaysia, it is recommended that a good quality rich mix using Ordinary Portland Cement be used to achieve a similar objective. As shown in the Figure 7-3, the use of unit cement content of 305kg/m^3 instead of 220kg/m^3 can drastically reduce the rate of deterioration by as much as three fold.

Figure 7-3 Average Rate of Deterioration of Concrete under Sulphate Attack for Various Types of Cement



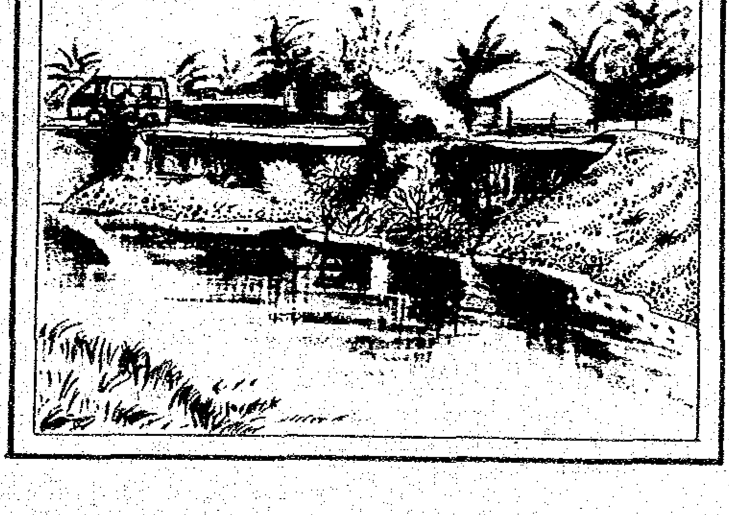
Note: Specimen soaked in soil containing sodium sulphate 10% in solution

A similar recommendation for precautionary method against acid attack was also proposed by the Construction Industry Research and Information Association (CIRIA, 1976). In an extensive review of precautionary measures adopted in various countries, CIRIA noted that in European countries reliance is generally placed on using well-compacted dense impermeable concrete as a means of resisting attack by acids rather than the use of special cements.

In conclusion, the chemical attacks on concrete at bridge sites identified during the National Axle Load Study are not due to sulphates as originally anticipated, but more likely to be caused by acids. As a precautionary measure, it is usually satisfactory to adopt a good quality rich mix of Ordinary Portland Cement to create a well compacted low permeability concrete.

CHAPTER 8

RIVER HYDROLOGICAL SURVEY



CHAPTER 8

RIVER HYDROLOGICAL SURVEY

8.1 General

This chapter presents the results of the river hydrological study as a part of detailed engineering survey. The river hydrological survey was carried out with following main objectives:

- To establish hydraulic rehabilitation plans based on the assessment, and
- To prepare basic criteria for bridge design from hydraulic viewpoint.

8.2 Methodology of Hydrological Study

The methodology applied in the hydrological study is broadly divided into two fold i.e.:

- Field inspection from hydraulic viewpoint.
- Hydrological analysis.

8.2.1 Field Inspection from Hydraulic Viewpoint

The inspection was carried out referring to the topographic maps of the twenty (20) study bridges in order to clarify the cause of the hydraulic defects observed. Principal inspection items are listed below.

- Identification of flood mark at bridge site during the past major flood.
- Confirmation of main water course of the rivers.
- Observation of river bank conditions.
- Confirmation of the tendency of riverbed whether raising or lowering.
- Identification of the local scouring location.

8.2.2 Method of Hydrological Analysis

The objective of the hydrological analysis is to assess the following river hydrological conditions around the bridge site.

- Magnitude of flood.
- Flow capacity of river at bridge site.
- Flood level.

Four (4) long span bridges with bridge numbers 00317000, 00319110, 00346740 and 00371000 were selected for the hydrological analysis out of the 20 bridges for the detailed survey. Based on data available, the method applied for the hydrological analysis is described below.

(1) River Basin Model

River basin model is necessary for flood estimation to express flood runoff traveling from upstream to downstream through the river basin. The model comprises several sub-basins and river stretches. A relatively large river basin is divided into several sub-basins to consider regional variation of runoff characteristics. In case where retardation effect of river is considerable, the river course is then divided into stretches which are defined from runoff point in the upstream sub-basin down to a runoff point in the remaining catchment.

Runoff calculation is carried out applying the storage function method which is commonly applied in Japan and has been also introduced to the flood mitigation study in Malaysia. The storage function method requires the parameters of the sub-basins and river stretches of the river basin model in order to simulate the rainfall-runoff process in the river basin. These parameters are calibrated by analyzing the past major flood and rainfall records.

(2) Rainfall Analysis

Since the flood runoff records were not sufficient for flood frequency analysis at the bridge sites, the flood frequency is analyzed from the rainfall records during the past major flood. Return period of flood is evaluated as that of basin mean rainfall during the corresponding period to flood. Basin mean rainfall is estimated by the Thiessen Polygon method from rainfall records at several raingauge stations in and around the river basin. Probable basin mean rainfall is estimated based on probability distribution of the Gumbel extreme type I.

(3) Flood Runoff Analysis

Flood runoff is estimated at the bridge site by the storage function method with the river basin model. The probable basin mean rainfall is used as an input for the runoff calculation which simulates the rainfall-runoff process in the river basin.

(4) Flood Level Analysis

Flood level at the bridge site is calculated from the estimated flood runoff peak. The non-uniform flow calculation which is a water surface routing through river channel is applied for the flood level calculation.

8.3 Field Inspection Results

8.3.1 River Condition

Major bridges of length more than 100 m are located along Federal Road 3 in the east coast of Peninsular Malaysia. Most of the bridges are affected with tidal reaches where low lying coastal plains spread. The rivers are frequently flooded during the north-east monsoon, which prevails mainly from November to January and brings heavy rainfall in the coastal plain. The river training works are limited to the near by urban area, and the river conditions around the bridges therefore remain mostly natural.

For middle scale bridges with bridge length between 20 and 100 m, the river has relatively small catchment area except for the two (2) bridges in Sabah and Sarawak.

Small scale bridges of length less than 20m mainly span artificial canals or small creeks for drainage and irrigation purposes. Therefore, it seems that the flood level of these small bridges are properly controlled.

8.3.2 Hydraulic Defects

The main hydraulic defects which were detected through the inspection of the twenty (20) bridges were classified as shown in Table 8-1. The defects are outlined below.

(1) Slope Protection Failure

The defect of slope protection failure was found at five (5) bridge sites; bridge No, 00237200, 00317000, 00341800, 00371000 and Dambai bridge. These bridges span over middle to large scale rivers.

These slope protections around abutments were commonly constructed using stone riprap. From the inspection, it can be concluded that main reason of the failure is due to inadequate depth of the footing and local scouring/bank erosion.

(2) Local Scouring

It was found that the river bed around the river piers of Bridge 00346740 have been extremely scoured based on assessment of the river cross section survey map. The extent of the scouring is over 20m long upstream and 60m downstream from the bridge with approximate 4m depth from the normal river bed at the river pier.

Table 8-1 Summary of Hydraulic Defects Observed

	Key	State	District	Bridge Length (m)	Type of Defects			
					Slope Protect-tion Failure	Local Scouring	Inadequate Bridge Opening <1	Bight of River
1	001/149/20	Johor	Segamat	12.86	-	-	-	-
2	001/611/40	Perak	Kinta	19.11	-	*	-	-
3	001/665/10	Perak	L.Matang	10.72	-	*	-	*
4	002/372/00	Pahang	Kuantan	26.70	*	-	-	-
5	003/170/00	Pahang	Rompin	397.32	*	-	*	-
6	003/191/10	Pahang	Rompin	121.96	-	-	-	-
7	003/418/00	T'ganu	Kemaman	36.14	*	-	-	-
8	003/467/40	T'ganu	Dungun	152.26	-	*	*	*
9	005/208/50	Melaka	Jasin	4.27	-	-	-	-
10	005/465/60	S'ngor	K.S'ngor	6.30	-	-	-	-
11	005/469/80	S'ngor	K.S'ngor	30.94	-	-	-	-
12	005/638/50	Perak	Manjung	41.59	-	-	-	-
13	005/678/40	Perak	Kinta	12.12	-	-	-	-
14	008/348/50	K'tan	K.Krai	13.71	-	-	-	-
15	050/010/70	Johor	Bt.Pahat	4.77	-	-	-	-
16	058/033/40	Perak	B.Padang	4.97	-	-	-	-
17	059/031/20	Perak	B.Padang	23.18	-	-	-	-
18	(Dambai)	Sabah	Penampang	50.10	*	-	-	-
19	(Samarahan)	S'wak	Samarahan	71.60	-	-	-	-
20	003/710/00	K'tan	K.Bahru	840.00	*	-	-	-

* Means defect is observed

<1 Based on interview survey results or observation of the flood evidence

(3) Inadequate Bridge Opening

Out of the 20 bridges, the following two bridges were suspected to be frequently flooded based on the field survey.

One is bridge number 00317000 at Endau. The profile of this bridge is parabolic to provide navigation clearance at the bridge center span. Because of this, on the beam web of both end spans, clear flood marks were observed which indicated the flood level reaching about one meter above the bridge seats of both abutments.

The other is bridge number 00346740 which spans over Dungun river. The bridge opening or the river flow capacity at the bridge site is less likely than the requirement because the bridge is located at a topographic constriction which seems to have caused flood levels to raise at the bridge site.

(4) Bight of River

At the site of bridge number 00166510, the river channel is sharply bent and the flow directly exposes the Taiping side abutment. This effect has caused extreme local scouring around the abutment.

The other case was observed upstream of Dungun river as mentioned above. The main water course is meandering and the left bank at the vicinity of the bridge upstream is subjected to the current flow. On the right side bank, huge sediment deposits were observed. This also causes a decrease of flow capacity.

8.4 Hydrological Analysis

8.4.1 Bridges For Hydrological Analysis

Out of the twenty (20) bridges for the detail survey, the following four (4) long span bridges were selected for the hydrological analysis, mainly considering the scale of the river.

Key	State	District	Bridge Length (m)
003/170/00	Pahang	Rompin	397.32
003/191/10	Pahang	Rompin	121.96
003/467/40	Trengganu	Dungun	152.26
003/710/00	Kelantan	Kota Bharu	840.00

Catchment area and monthly mean rainfall of the respective river basins are shown in Appendix-K. Basin hydrology of each river is summarized below.

(1) Endau River (Bridge 003/170/00)

The basin area of the Endau river is 4,740 km2 and extends over Pahang state in the north and Johor state in the south. Heavy meanders exist in lower tidal reaches.

The low lying areas about 80 km upstream from river mouth are mostly covered with swampy land. Average annual rainfall in the basin is 2,600 mm.

(2) Pontian River (Bridge 003/191/10)

The Pontian river basin has a small catchment of 300 km2 between Endau and Rompin river basins. The north east part of the catchment is mostly covered with low lying swampy land. On the other hand, hilly areas at elevation of about 200 to 600 m extend along the boundary of the Endau river basin in the south west part of the basin. Annual average rainfall is 3,000 mm in the basin.

(3) Dungun River (Bridge 003/467/40)

The Dungun river which has a catchment area of 1,875 km2 originates from the mountainous area at elevation of about 1,500 m in the central/south part of Trengganu State. Upper and middle reaches of the river flow down through the hilly area and no heavy meanders exist along the lower reaches. Annual average rainfall is abundant at 3,300 mm in the basin. Annual mean runoff at Jerangau, where the catchment area is 1,480 km2, has been recorded at 120 m3/sec.

(4) Kelantan River (Bridge 003/710/00)

The Kelantan river basin with a catchment area of 13,100 km2 occupies more than 85 % of the Kelantan State. Annual mean rainfall is 2,700 mm in the basin and varies from 3,200 mm in north west boundary of Thailand to 2,200 mm in the central south part of the basin. Annual mean runoff has been recorded at 540 m3/sec at Guillemard bridge, where a stream gauge is located just upstream of Tanah Merah, for the catchment area of 12,080 km2.

Based on the field inspection, no serious hydraulic defect was observed for bridge number 003/191/10. This bridge was, therefore, excluded from the hydrological analysis. Furthermore, the flood mitigation study which included hydrological analysis was carried out for Kelantan River in 1989. It was decided that the results of this study be utilized in the hydrological assessment for bridge 003/710/00.

Consequently, the hydrological analysis was conducted for only two (2) river basins, Endau and Dungun river, which correspond to bridge numbers 003/170/00 and 003/467/40 respectively.

8.4.2 River Basin Model

The basin division was made with due consideration to the confluence of tributary slopes of river stretches and base points of runoff calculation, such as gauging station and study bridge site.

The catchment area of Dungun river is 1,690 km2 at bridge 003/467/40 and 1,480 km2 at the DID's streamflow gauge having a station number of 4832441 which is located 35 km upstream from the bridge.

Endau river has a rather wide catchment area of 4,700 km2 at bridge 003/170/00. There is no available streamflow records along the main river course, and only one gauge station number 2235401, is located at Kg. Kenangan along the Kahang river.

The river basin model constructed for each river is illustrated in Appendix-K.

8.4.3 Rainfall Analysis

Daily rainfall records were collected from five (5) raingauge stations for the Dungun river basin and eight (8) for the Endau river basin. The collected rainfall records were retrieved in Appendix-K.

Based on assessment of the daily rainfall records, the heavy rainstorms were generally concentrated over 3 to 5 consecutive days in the east coast of Malaysia. Therefore the 5-day rainfall was applied for the design storm rainfall to estimate flood runoff. Probable basin mean rainfall for the respective return periods are shown in Table 8-2.

Table 8-2 Probable Basin Mean Rainfall

Endau River Basin

Return Period (years)	Probable Rainfall (mm)						
	1-day	2-day	3-day	4-day	5-day<1	6-day	7-day
2	138	201	237	273	291	317	342
5	191	281	337	391	426	463	489
10	225	334	402	469	515	559	587
20	259	385	465	543	601	652	681
50	302	451	547	640	712	772	802
100	334	500	608	713	795	861	893

<1 Applicable in the study

Dungun River Basin

Return Period (years)	Probable Rainfall (mm)						
	1-day	2-day	3-day	4-day	5-day<1	6-day	7-day
2	175	268	327	385	425	450	475
5	297	437	535	642	702	741	777
10	379	550	673	811	886	933	977
20	459	657	806	974	1,062	1,118	1,169
50	557	796	977	1,185	1,289	1,358	1,418
100	633	901	1,105	1,343	1,460	1,537	1,604

<1 Applicable in the Study

8.4.4 Flood Runoff Analysis

The river basin model of each river was calibrated by using the basin mean rainfall and runoff records during the past major floods. The calibration was conducted by estimation of parameters for the storage function. The estimation of parameters for the river basin model is summarized in Appendix-K.

In estimating the probable flood runoff, the hourly rainfall during the past major flood was expanded by multiplying the expansion factor obtained by the following equation.

$$\text{Expansion Factor} = \frac{\text{Probable Basin Mean 5-day Rainfall}}{\text{Basin Mean 5-day Rainfall during flood}}$$

The calculated probable flood runoff peak at each bridge site is shown in Table 8-3.

Table 8-3 Estimated Flood Runoff Peak

River (Bridge)	Endau River (3/170/00)	Dungun (3/467/40)
Return period (years)	50	50
Rainstorm Selected (year)	1989	1986
Basin Mean Rainfall (mm)	424	1192
Probable Rainfall (mm)	712	1289
Expansion Ratio	1.68	1.08
Estimated Flood Runoff Peak (m3/sec)	4,780	3,050

8.4.5 Flood Level Analysis

The peak flood runoff analyzed at 50 year return period was converted to the flood level at the bridge site. The flood water level at the bridge site was calculated using non-uniform flow calculations because retardation of river channel and tidal effect are considerable at the bridge site. The non-uniform calculation results for the estimation of the flood level was presented in Appendix-K. The results of flood level analysis are summarized in Table 8-4.

Table 8-4 Estimated Flood Level

River (Bridge)	Endau River (3/170/00)	Dungun River (3/467/40)
Return period (years)	50	50
Probably Flood Runoff Peak (ms/sec)	4,780	3,050
Elevation of Estimated Flood Level (1) (EL m)	2.79	4.00
Elevation of Beam Soffit (2) (EL m)	1.5<1	3.92
Difference (1) - (2) (m)	1.29	0.08

<1 Elevation of the lowest beam soffit at the end span.

8.5 River Assessment Results

This section presents overall assessment results based on the field findings and the hydrological analysis results described in the previous sections.

8.5.1 Overall Assessment of Major Rivers

(1) Bridge No. 00371000 (Yahya Petra Bridge)

The flood control study of the Kelantan river proposed that the bridge should be constructed with about 2m higher than that of the existing bridge because the existing channel capacity at the bridge site which is about 4,500 m3/sec is less than the requirement which is of a magnitude of 5,500 m3/sec occurring every two years.

The potential and major hydrological problems on this river are inadequate river channel capacity compared with the estimated flood runoff and high sediment yield which has been aggravating the flood problem in this area. To overcome these problems, the report recommended construction of two flood control dams upstream and to widen, raise or newly construct a levee about 50 km long on both sides.

Taking into account that the bridge elevation is not causing major and potential hydrological problems in the river and the scope of the work for this study, it is recommended not to raise the bridge grade but to provide relatively minor hydraulic rehabilitation works.

It is, however, suggested that reconstruction of the bridge shall be implemented in the flood control project.

(2) Bridge No. 00317000 (Endau River)

The hydrological analysis result has proven that beams at both end spans, which were found to have flood marks during the inspection, have been submerged. It is therefore recommended that the beams of both end spans be raised to provide adequate bridge opening.

(3) Bridge No. 00346740 (Dungun River)

From the field inspection results, topographic constriction at the bridge site has mainly resulted in local scouring around the river piers and raising of the flood level.

In this regard, the hydrological assessment results proved that the existing channel capacity at the bridge site is slightly less than that required i.e. inadequate bridge opening. This inadequacy could be aggravated by effect of the river bight in the vicinity of the bridge upstream.

Taking into account the above mentioned hydraulic defects especially on the extent of inadequacy of the bridge opening, the following remedial works are selected so as to mainly increase the channel capacity at the bridge site.

o Excavation of the right side river bank.

o Provision of revetment for both side banks.

o Provision of river bed protection around the river pier.

o Provision of spur dike at the left side river bank of the bridge upstream.

An alternative hydrological assessment, provided the above mentioned rehabilitation works was carried out, using non-uniform flow calculation method is shown in Table 8-5.

Table 8-5 Alternative Calculation Results

Condition	Present	After Rehabilitation
Return Period (Years)	50	50
Flood Runoff Peak (m3/sec)	3,050	3,050
Flood Level (EL m)	4.00	2.86
Soffit Level (EL m)	3.92	3.92
Difference (m)	0.08	-1.06

As shown in Table 8-5, the flood level, if the rehabilitation works are carried out, will be reduced from 4.00m to 2.86m. Therefore, it can be concluded that the bridge opening is adequate provided the above mentioned rehabilitation works are carried out.

8.5.2 Summary of Hydraulic Defects and Rehabilitation Plan

The hydraulic defects which were not only detected based on the field inspection but also proven by the hydrological assessments for major rivers are tabulated together with the corresponding rehabilitation plans in Table 8-6.

Table 8-6 Summary of Hydraulic Defect and Rehabilitation Plan

Key	Defect	Cause	Rehabilitation Plan
001/611/40	– Exposed pier footing	– Local scouring	– Installation of river bed protection
	– Exposed abutment footing	– Local scouring	– Installation of abutment protection
001/665/10	– Decreasing bridge opening	– Sedimentation	– Widening river channel with slope protection
	– Exposed footing	– River bight	– Realignment
	– Riverbank erosion	– River bight	– Realignment
002/372/00	– Failure of abutment protection	– Insufficient depth of slope toe	– Reconstruction of abutment protection
003/170/00	– Submergence of both end spans	– Inadequate bridge clearance at both end spans	– Raising of girders
	– Exposed footing of con-crete wall	– Local scouring	– Installation of foot protection
003/418/10	– Failure of abutment protection	– Insufficient depth of slope toe	– Reconstruction of abutment protection
003/467/40	– Inadequate free board	– Insufficient capacity of river channel	– Excavation of river channel
	– Riverbed degradation	– Local scouring	– Installation of riverbed protection
	– Riverbank erosion	– River bending in upstream	– Installation of riverbank protection
003/710/00	– Failure of abutment protection	– Local scouring	– Reconstruction of abutment protection
	– Riverbank erosion in both upstream and downstream		– Construction of slope protection
Dambai	– Riverbank erosion	– River bending in upstream	– Construction of slope and abutment protection

8.5.3 Recommended Basic Criteria for Bridge Design

Through the field inspection carried out during the whole study period, it was frequently observed that in some of the existing bridges the hydraulic effects have not been fully taken into account in the design compared with the requirements in Japan. Therefore following basic hydraulic criteria for bridge design is presented for reference purpose only.

In Japan, since bridges are considered as one of the river related structures, design of bridges should be subject to the flood control plan of the river. Several cases of flood disasters which were caused by inadequate design of river related structures have been reported. These disasters also included a case where the structure itself was damaged and washed away by flood. According to such experiences, several standards for design of river related structures including bridges have been established.

It is believed that the said standards are recommendable for further bridge design in Malaysia. The criteria introduced below are considered as principle for bridge design.

(1) Bridge Length

In general desirable river width can be estimated by the following equation:

$$B = (0.5 \sim 0.8)\, Q^{3/4}$$

where, B: River width (m)
Q: Design Discharge (m^3/sec)

Desirable bridge length which is substantially equal to the estimated river width should then be determined by the corresponding design discharge as listed below.

Design Flood Runoff (m3/sec)	Desirable Bridge Length (m)
300	40 - 60
500	60 - 80
1,000	90 - 120
2,000	160 - 220
5,000	350 - 450

(2) Minimum Span Length

Required minimum span length shall be determined by the following equation.

$$L = 20 + 0.005\,Q$$

where, L : Required minimum span length (m)
Q : Design discharge (m^3/sec)

This equation was established based on the damage reports of bridges. It is said that bridges with a span length of less than 15 m were mostly affected by wooden logs rushed from upstream during flood.

(3) Freeboard

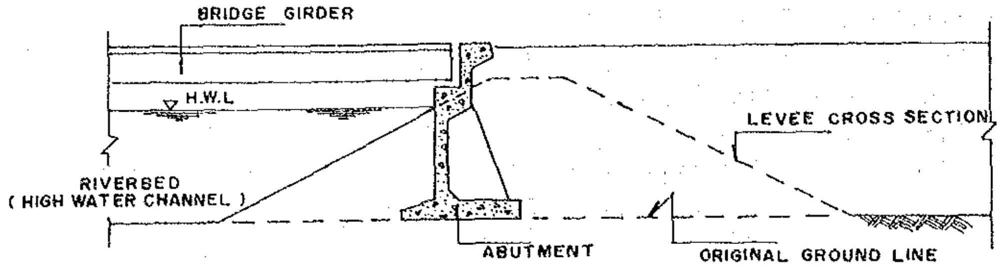
The freeboard of a bridge is considered to be the same as that of the levee. Minimum requirement of freeboard above design flood level is determined as follows.

Design Flood Runoff (m3/sec)	Freeboard (m)
less than 500	0.5
500 - 2,000	1.0
over 2,000	1.5

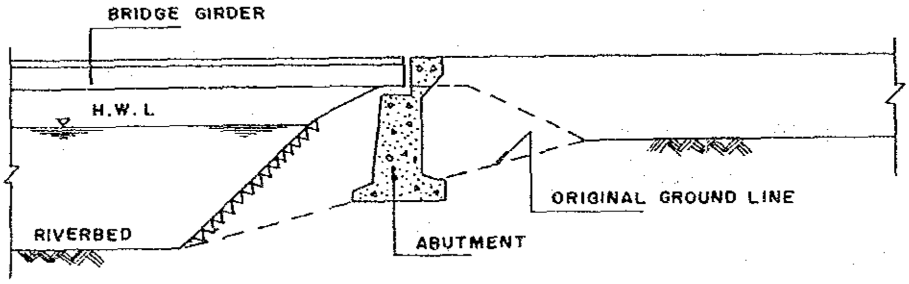
(4) Abutment

Abutment should be constructed outside of the river flow section as shown in Figure 8-1. For natural rivers without levee, abutment and approach embankments should be constructed not to obstruct flood flow under the existing river condition. The surface of the abutment on the river side should be parallel with the direction of flood flow. Foundations of abutments should be located below the original ground line.

Figure 8-1 Location of Abutment



RIVER WIDTH MORE THAN 50 m



RIVER WIDTH LESS THAN 50 m

(5) Pier

To avoid disturbances to river flows, horizontal of piers should be oval-shaped and as flat as possible. The direction of longer diameter oval-shaped piers should be in parallel with that of flood flow. Reduction of flow sectional area by installation of piers should be less than 5 % of flood flow sectional area. Pile bent type pier is not recommended because it would obstruct wooden logs flowing down from upstream. Recommended location and footing depth of pier are summarized below.

a) Minimum distance from slope toe of levee and slope top of low water channel.

Design Flood (m3/sec)	Distance (m)
less than 500	5
over 500	10

b) Embedded Depth of Footings

Embedded depth of footings should be determined based on riverbed variation and estimated local scouring depth.

Location of Pier	Embedded Depth of Footings
Low water channel	More than 2 m below riverbed
High water channel within 20 m from slope top of low water channel	- do -
High water channel over 20 m from slope top of low water channel	More than 1 m below riverbed

CHAPTER 9

DETAILED STRUCTURAL SURVEY

CHAPTER 9

DETAILED STRUCTURAL SURVEY

9.1 General

The objective of a detailed engineering survey is to obtain detailed engineering information so as to enable preliminary maintenance and rehabilitation engineering design to be carried out. The detail engineering survey was carried out as part of the detailed survey on only the 20 representative bridges selected. The results of the study on these selected bridges are also used in formulating a rehabilitation program for the rest of the bridges in the study.

The work included structural detail measurements, material strength measurements, deterioration degree measurement and assessment of bridge function. The extent and type of the work varies between bridges and it depends on the degree and type of distress of the individual bridges in the study. Data gathered during the survey was recorded in the inspection forms and finally all the results were transferred to a drawing form. The drawings for each bridge consist of general view, strength measurement diagram and crack diagram of concrete members and/or corrosion diagram of steel members. The drawings of each representative bridge type out of the 20 bridges are enclosed in Volume IV Drawings in a separate booklet.

9.2 Procedure and Inspection Equipment

Prior to the commencement of the detailed survey, two inspection teams were organized, one team for inspection of concrete bridges and the other for steel bridges. Each team consisting of a senior bridge engineer, two assistant engineers and three technicians carried a full set of basic inspection equipment which included the following:

* Measurement equipment - 3m, 50m, and 65m tapes, slide caliper, crack scale, calipers, spirit level, leveling staff and plumb bob.

* Recording equipment - camera, black board, chalk and photo-scale.

* Access equipment - ladder, rope, rubber boat and inspection vehicle (shared by two teams).

* Inspection equipment - binocular, chipping hammer, chisel, wire brush, scraper, torch light, parang, spray paint, electric drill, generator and electric grinders.

* Safety equipment - life jacket, safety belts, safety helmets, goggles, cotton gloves and first aid kit.

* Testing equipment - ultrasonic thickness meter (steel thickness), profometer (rebar detector), schmidt hammer and carbonation kit.

In addition, the following testing equipment were also used alternately between the two teams:

- Ultrasonic pulse velocity equipment (concrete strength)

- Half cell equipment (rebar corrosion)

- Echo-tip (steel strength)

Before the detailed inspection work began, all members of the inspection teams were given practical training on the procedure of recording structural data gathered, testing procedure and method of operating all the specialized testing equipment. Practical training was also given to two JKR technicians on the procedure of operating the inspection vehicle. For this purpose, a brief manual for handling of specialized testing equipment and interpretation of results obtained from the test was prepared.

9.3 Structural Details Measurement

Structural detail measurement consisted of conventional dimension measurement, crack/corrosion mapping, concrete cover and rebar size measurement, steel thickness measurement and pile length measurement. The objective of the structural detail measurement is to obtain as-measured dimension of the bridge members so as to enable structural assessment of the bridge to be carried out.

9.3.1 Structural Dimension Survey

In order to prepare an as-measured drawing, conventional dimension measurement was carried out so as to record all structural details which includes width of deck slab, length and size of girder, general layout of beam, connection details, columns, abutments, piers, diaphragm, wind bracings and transverse stiffeners details. For bridges where some of the as-built drawings were available, member sizes obtained were compared with those in the drawing so that any differences could be identified. The measurement of cross-sectional dimensions was made to the nearest millimeter, while the measurement of longitudinal dimensions of bridge component was carried out to the nearest 10mm.

9.3.2 Crack/Corrosion Mapping Survey

The extent of damages identified were recorded in the crack/corrosion mapping survey. In a concrete member, damages were classified into crack, flaking, rebar exposure, freelime etc., while in a steel member those were classified into corrosion, lamination, paint deterioration, buckling etc. The results from this survey were presented in the form of a crack diagram and/or a corrosion diagram. These diagrams often give an indication of the cause of the damage. Therefore measurements and crack mapping help in providing a clearer picture of the condition and behavior of the structure, hence its most practical maintenance and rehabilitation method. Surface crack width was measured with a transparent crack width gauge, while flaking of concrete was detected by the difference in sound when it is tapped with light chipping hammer. Deformation and corrosion in steel members were observed visually, it was then measured and recorded in the form.

9.3.3 Concrete Cover, Rebar Size, and Spacing Survey

In the case of a reinforced concrete member, structural analysis can not be carried out unless detailed data with regard to its rebar is known. Therefore, in addition to the structural dimension survey mentioned above, additional survey pertaining to rebar embedded in the concrete was carried out which includes concrete cover, rebar size and spacing. The extent of survey carried out depends on type of bridge member and the availability of as-built drawings.

The position of reinforcement bar was first located by the use of profometer II or III (an electromagnetic reinforcement detector). This equipment gives only an approximate size of the reinforcing bars. For those bridges where the as-built is available, profometer was used to check the size of the rebar. Spacing was obtained directly from the profometer.

Since the equipment gives approximate rebar size, minimal break-up of concrete cover was necessary for bridges where a drawing was not available. Steel reinforcements were exposed using an impact drill and the size of steel bar was measured, its condition noted and the concrete cover was recorded. The hacked area of the concrete was then repaired using proper repair mortar. The result of rebar size and spacing survey is incorporated in detailed survey drawings which are enclosed in Volume IV Drawings. A summary of the result of the concrete cover survey is tabulated in Table 9-1.

Concrete cover for slab soffit varies from 25 to 60mm and averaged at about 38mm. The concrete cover was found to be adequate. However, concrete cover for beam soffit averaged at about 30mm is slightly inadequate and concrete cover of 40mm in pile and in substructure is too low. This inadequacy of concrete cover may be one of main reasons for cracking which is commonly observed as a distress in concrete piles.

9.3.4 Steel Thickness Measurements

In assessment of the existing bridges, it is important to provide the exact thickness of a steel member considering possible corroded section loss. For this purpose, the measurement of steel thickness using ultrasonic thickness meter was carried out at the corroded steel section so as to obtain the effective steel section. The result of steel thickness measurements was shown in the detailed survey drawings which are enclosed in Volume IV Drawings.

Table 9-1 Concrete Cover

BRIDGE NO	BRIDGE TYPE	DEPTH OF COVER (mm)				
		SUPERSTRUCTURE		SUBSTRUCTURES		PILE
		BEAM SOFFIT	DECK SLAB SOFFIT	ABUTMENT OR PIER	CAPPING BEAM	
00114920	RCB	15	60	30	30	30
00166510	SBG&RCB	34	38	45	-	-
00237200	SBC&RCB	25	30	-	50 (Soffit)	45
00317000	PCB	25	34	36	-	38
00319110	PCB	35	40	-	30	30
00341800	RCB	30	50	35	100	40
00346740	PCB	55	40	-	30	-
00546560	RCB	33	30	-	-	-
00546980	RCS	-	49	-	-	-
00563880	IT	-	-	30	40	40
00567840	PCB	15	-	30	40	40
00834850	RCS	-	25	40	40	40
005803340	SBB	-	-	40	-	-
005903120	SBC	-	35	-	-	-
Dambai	SBC	-	30	-	-	-
Samarahan	SBC	-	30	-	-	-
00371000	RCB	30	-	-	40	30

9.3.5 Pile Length Measurement

One bridge, namely bridge No 00546980, out of the 20 bridges was found to have pile foundation failure. In order to assess the cause(s) as well as to prepare the appropriate rehabilitation plan, a pile length measurement survey was carried out using a downhole by P/S Wave logging test method. The result showed that the pile tip was about 33 meter below the existing ground level or 31.5 meter below the pile cap level. (The detailed method and the assessment result can be referred to Chapter-7)

9.4 Material Strength Measurements

Material strength measurement carried out in the study included concrete coring, pulse velocity test (UPV) and schmidt hammer for concrete strength, while steel strength measurements was carried out by using hardness tester (Echo-Tip).

9.4.1 Concrete Strength

Concrete strength was obtained either from non-destructive or semi-destructive methods. Non-destructive methods were carried out by using schmidt hammer or UPV, while semi-destructive methods by testing the concrete core sample. Concrete coring is a semi-destructive test which was carried out only if it was found to be absolutely necessary or to check the strength obtained by other non-destructive methods or where in addition to strength, the density and modulus of elasticity of concrete is also required in the assessment work.

(1) By concrete Core Test

A total of 20 concrete core samples were taken from 5 bridges in the study. Out of the 20 samples, 5 samples were used for testing alkali-aggregate reaction. The test results of the remaining 15 samples for the concrete core test are summarized in Table 9-2.

Table 9-2 Summary of Concrete Core Test Results

BRIDGE NO OR NAME	MEMBER	MAXIMUM AGGREGATE SIZE	QUALITY OF CONCRETE AND DEFECTS	SATURATED DENSITY (kg/m3)	ELASTIC MODULUS (kN/mm²)	CUBE STRENGTH (N/mm²)
00237200	Superstructure (Deck Slab)	20	-Small voids ≈ 1.5 % -Honeycombed (43x30 max)	2280	16.0	19.5
00317000 Sg. Endau Johor	Substructure (Abutment)	20	-Small voids ≈ 0.5 % -Rounded aggregate -Honeycombed (7x13 Max)	2330	26.0	33.0
00319110 Sg. Pontian Pahang	Superstructure (Deck Slab)	20	-1.5% excess voidage -aggregate well graded -Honeycombed (10x6 max.) -Crack near surface about 62mm in depth -Cold joint	2380	39.5	62.0
00834850 Sg. Kesang Kelantan	Superstructure (Deck Slab)	20	-Small voids ≈ 13 % -Flaky aggregate -Excessive honeycomb (103x100 max.) -Cold joint	2300	25.5	21.0
00371000 Yahya Petra Kelantan	Superstructure (Deck Slab)	20	-1.5% excess voidage. -Aggregate uniformly distributed. -Honeycombed (5x16 max.)	2450	30.0	50.5

(2) By Ultrasonic Pulse Velocity (UPV)

UPV test was carried out on a number of bridges. However, the test results were widely scattered and the pulse velocity measured could not be readily converted to concrete strength using the cube strength correlation formula developed in Japan. Therefore, the results obtained by UPV test were ignored.

(3) By 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. Some of the values obtained for a few bridges were slightly scattered, it is safe to take only minimum concrete strength value from conservative viewpoint. A summary of concrete test results carried out by the core sample and the schmidt hammer is tabulated in Table 9-3.

Table 9-3 Summary of Concrete Strength Test Results

BRIDGE NAME	SUMMARY OF MINIMUM CONCRETE STRENGTH (N/mm²)			
	SUPERSTRUCTURE		SUBSTRUCTURE	
	BEAM	SLAB	ABUT/PIER	PILE
00114920	14	18	11	-
00166510	36	-	25	-
00237200	37	20 ∠1 (19)	<10	-
00317000	39	23	35 (33)	-
00319110	29	35 (62)	32	27
00341800	34	36	36	45
00346740	33	27	32	36
00520850	24	<10 ∠2	-	-
00546560	16	<10 ∠2	18	-
00546980	-	24	37	-
00563880	35	-	36	-
00567840	54	-	39	-
00834850	-	<10 ∠1 (21)	11	34
05001070	-	-	24	
05803340	-	24	32	-
05903120	-	39	27	-
DAMBAI	-	28	25	-
SAMARAHAN	-	21	31	-
00371000	39	- (50)		-

Note: 1) Figure in ( ) is the minimum cube strength taken from concrete core sample.
2) ∠1 Based on reliability of the test result, value by testing the core sample is applied.
3) ∠2 The strength appears to be too low and very questionable compared with those of the beams and from the visual inspection, taking into consideration both structural members having the same design mix of the concrete. It is there fore, more prudent to take cube strength of the beam instead of the slab in the assessment.

(4) Applicable Concrete Strength in Assessment

In general, it is acknowledged that concrete strength gradually increases after casting the concrete, even after the bridge is completed, but the rate of increase is not certain depending on water/cement ratio used, quality of cement and aggregate, concrete age, environmental condition where the concrete member is located etc. Therefore, in assessment of the existing bridge, design mix applied or design concrete strength used is usually adopted as a concrete strength from conservative viewpoint.

In this regard, relevant study revealed that almost all bridges built after 1955 was constructed with 1:2:4 (Cement:Sand:Gravel) concrete mix for all in situ concrete and 1:1½:3 for precast concrete including P.C. concrete and piles of which cube strength at 28 days is about 20N/mm² and 25 N/mm² respectively.

For assessment purposes in the study, it is recommended that cube strength at 28 days for each concrete member of the study bridges shall be taken from the lowest of either 28 days designed cube strength, strength by the concrete core test, or by schmidt hammer test from a conservative viewpoint.

Based on this criteria, concrete cube strength to be used in the assessment is listed in Table 9-4.

Table 9-4 Concrete Cube Strength to be used in the Structural Assessment of Bridges (Unit N/mm)

BRIDGE NO.	BRIDGE TYPE	SUPERSTRUCTURE		SUBSTRUCTURE	
		Beam	Slab	Abut/Pier	Pile
00114920	RCB	14	18	11	-
00166510	SBG & RCB	20	20	20	-
00237200	SBC & RCB	19	19	19	-
00317000	PCB	25	20	20	-
00319110	PCB	25	20	20	25
00341800	RCB	20	20	20	25
00346740	PCB	25	20	20	25
00520850	SBE	20	20	-	-
00546560	RCB	16	16	18	-
00546980	RCS(H)	-	20	20	-
00563880	IT	25	-	20	-
00567840	PRB	25	-	20	-
00834850	RCS	20	20	11	25
05001070	SBB	-	-	20	-
05803340	SBB	-	20	20	-
05903120	SBC	-	20	20	-
Dambai	SBC	-	20	20	-
Samarahan	SBC	-	20	20	-
00371000	RCB	20	20	20	-

9.4.2 Steel Strength

(1) Structural Steel Strength

Approximate strength of the structural steel in the study bridge was obtained by using Echo-Tip which is a hardness test machine. The summary of the steel strength obtained by Echo-Tip is listed in Table 9-5.

Table 9-5 Summary of Steel Strength from Echo-Tip

Bridge Name	Average Steel Strength from Echo-tip (N/mm^2)				
	Rolled beam	plate girder	diaphragm	buckle plate	tubular pile
00166510	-	527	-	-	-
00520850	425	-	-	-	-
05001070	425	-	-	455	-
05803340	408	-	-	-	-
05903120	445	-	-	-	-
Dambai	-	424	-	-	493
Samarahan	-	444	416	-	-

On the other hand, it is commonly acknowledged that all rolled steel sections used in the study bridges were produced in United Kingdom. Therefore, with the knowledge of the beam dimension, bridge age and manufacturer of the beam, then properties of the steel beam could be obtained referring to the corresponding B.S code or relevant old engineering reference books. The list of steel standard and its yield stress in accordance with the relevant standard are shown in Table 9-6.

Table 9-6 Strength of Steel Produced in UK since 1934

Steel Standard	Title of the British Standard Code of Practice	Year published	Yield Stress (N/mm^2)
BS 15	Mild steel for general structural purposes.	1948	247.3, $t \leq 19mm$ 231.8, $t \leq 38mm$ 227.9, $t > 38mm$
BS 449	The use of structural steel in building.	1948	230 (BS 968)
BS 449	Structural Steel in building.	1969	350.0

Structural steel used in United Kingdom as listed in Table 9-6 above shows that the minimum yield stress is 230 N/sq.mm (tensile strength), while measured values using Echo-Tip is about 408 - 455 N/sq.mm. In this regard, for the purpose of the structural assessment, it is safe to assume all the structural steels in the study bridges are Grade 43 steel with yield stress of 230 N/sq.mm from a conservative viewpoint.

(2) Reinforcement Bar Strength

The strength of reinforcement bar is quite standard as the steel used in the bridges was manufactured in Britain based on the Engineering Code of Practice at that time. The code of Practice for assessment of highway bridges and structures in the UK (BD 21/84) specified that "In the absence of definite information, a characteristic yield strength of 230 N/mm^2 may be assumed for steel produced before 1955". Therefore the strength of steel reinforcement to be used in the study shall be 230 N/mm^2.

9.5 Measurement of Material Deterioration Degree

Reinforcement bars in concrete structure are protected from rusting by the alkalinity of the concrete. This protection could be reduced permanently either as a result of penetration of carbon dioxide gas into the concrete or by the penetration of chloride from sea water. The type of material deterioration measurement carried out in the study included chloride test, carbonation test, rebar corrosion test by half-cell potentiometer and alkali-aggregate reaction test. The formation analysis of bearing pads was also carried out.

9.5.1 Carbonation Test

Carbonation of concrete takes place when carbon dioxide in the air is absorbed and react with calcium hydroxide ($Ca[OH]_2$) in concrete to form calcium carbonate ($CaCO_3$). This reaction reduces the alkalinity of the concrete from pH 13 (normal condition) to about pH 9.0 or lower. Therefore the purpose of carbonation test is to determine the depth in which the concrete has been carbonated and resulted in the diminishing concrete alkalinity. The test was conducted by spraying a freshly exposed concrete with phenolphtalein solution which is colorless. The strong pink colour of sprayed concrete indicates that the concrete still retained its alkalinity but if it is colorless then the concrete has no longer enough alkalinity to protect the steel from rusting. The results of the carbonation test are summarised in Table 9-7.

Carbonation test carried out on 17 bridges shows that generally the bridge substructures are not carbonated, except for 2 bridges (bridge 00546980 and 005803340 where the substructure has carbonated slightly with carbonation depth of 3 and 14mm respectively). All PC beams are not carbonated. In-situ RC and encased I-beams have been carbonated with depth of carbonation varying from 2mm to more than 60mm. All deck slabs except for 2 bridges in the study have been carbonated with carbonation depth varies from 11mm to greater than 75mm.

Table 9-7 Maximum Carbonation Depth (mm)

BRIDGE NO.	BRIDGE TYPE	Superstructure		Substructure	
		Beam Soffit	Deck Slab Soffit	Abutment or Pier	Pile
00114920	RCB	-	12	-	0
00166510	SBG&RCB	>60	>52	0	-
00237200	SBC&RCB	13	-	-	0
00317000	PCB	0	11	-	-
00319110	PCB	0	-	0	-
00341800	RCB	0	-	-	-
00346740	PCB	-	0	-	0
00520850	SBE	25	>75	-	-
00546560	PCB	0	-	-	-
00546980	RCS(H)	-	30	3	3
00567840	PRB	0	-	-	-
00834850	RCS	-	0	0	-
05803340	SBB	-	-	14	-
05903120	SBC	-	50	-	-
Dambai	SBC	-	43	-	-
Samarahan	SBC	-	12	-	0
00371000	RCB	2	-	-	0

Note: - Means no test was carried out

9.5.2 Chloride Test

The purpose of the chloride test is to determine the concentration and depth in which the chloride ions have penetrated in the concrete, hence the risk of rebar corrosion. Generally when the chloride ions reach a critical value in the vicinity of the reinforcing bar, corrosion of the steel will start to occur. However the critical chloride concentration varies not only due to the above reasons but also on carbonation depth. Therefore there is no fixed value of critical chloride concentration. As a guide, the present of 0.4% of chloride ion by weight of cement may be taken as the critical chloride content value whereby the risk of corrosion could be high.

In the study, chloride tests were carried out mainly at bridges which are highly suspected to have been deteriorated due to chloride attack. The chloride content of the concrete was determined by the chemical analysis of concrete dust taken by drilling. The sampling of the dust was taken at increments of 20mm up to a depth of 60mm so that the profile of chloride content in the concrete member could be determined. Results of the chloride tests are given in Table 9-8.

Table 9-8 Summary of Chloride Content in Concrete Members

Bridge No.	Structural Member	% of Chloride ion at various depth (mm)			Depth of Cover (mm)
		0-20	21-40	40-60	
00237200 (SBC/RCB)	Beam	<0.01	<0.01	<0.01	25
	Deck	<0.01	<0.01	<0.01	30
	Pile	2.64	2.56	2.09	45
00317000 (PCB)	Beam	2.43	2.00	0.80	25
	Deck	1.36	1.43	0.21	34
	Abut/Pier	-	-	-	36
	Pile	-	-	-	38
00341800 (RCB)	Beam	0.24	<0.01	<0.01	30
	Deck	<0.01	<0.01	<0.01	50
	Pile	3.15	1.25	0.95	40
00546980 (RCB)	Beam	-	-	-	-
	Deck	<0.01	<0.01	<0.01	49
	Abut/Pier	0.06	<0.01	<0.01	-
	Pile	2.85	1.25	2.06	-
00371000 (RCB)	Beam	<0.01	<0.01	<0.01	30
	Deck	<0.01	0.39	<0.01	varies
	Abut/Pier	<0.01	<0.01	<0.01	40

All the results in Table 9-8 show that the chloride ion gradually decreases with depth except a few exceptional cases. This means that the chloride ion is absorbed from the concrete surface and not resident in the concrete or aggregate.

As seen from Table 9-8, it can be concluded that most of the piles have been badly attacked by chloride which is far exceeding the critical value (0.4%) at the rebar position. Furthermore, chloride attack to the beam and the slab at bridge 00317000 (flooded bridge) is quite serious and it has reached a depth of more than its concrete cover.

9.5.3 Sulphate Test

Sulphate attack on concrete is aggressive in character as a result of sulphates reacting with tricalcium aluminate (C_3A) in cement to form the compound ettringite. A sulphate test was carried out by taking samples from concrete materials as well as from top soil at the possible source of sulphates (the latter test was described in Chapter 7).

Sulphate test was carried out mainly at bridges which are highly suspected to have been deteriorated due to sulphate attack based on the NALS reports. The sulphate content of the concrete was determined by the chemical analysis of concrete dust samples. The sampling of concrete dust was taken at increments of 20mm up to the depth of 60mm so that the profile of sulphate content in the concrete member could be determined. There is no clear demarcation on the acceptable limit of sulphate content in concrete. In general, the acceptable limit of sulphate is 4.0% by weight of ordinary Portland cement. Results of sulphate test are given in Table 9-9.

Table 9-9 Summary of Sulphate Content in Concrete Members

Bridge No	% of Sulphate at Various Depth		
	0 - 20mm	21 - 40mm	41 - 60mm
00114920	1.85	1.75	1.45
00237200	2.07	1.14	1.15
00520850	2.45	2.06	1.35
05803340	2.68	2.85	2.86
05001070	1.81	1.36	1.09

The results showed that the percentage of sulphate by weight of cement was within acceptable levels. Therefore, there is a possibility that the defect observed was not caused by sulphate but it was caused by other aggressive agents as discussed in Chapter 7.

9.5.4 Rebar Corrosion Test

The purpose of carrying out rebar corrosion tests using half cell equipment is to map the area of the concrete member in which active corrosion of the reinforcement bar is taking place. However, the electrical potential does not give a definite proof of the corrosion of the rebar, but it only gives an indication of the chances that the rebar inside the concrete being tested could be rusting. Summary of the rebar corrosion test results are shown in Table 9-10.

Table 9-10 Summary of the Rebar Corrosion Test Results

BRIDGE NO	LOCATION	POTENTIAL (Volts)	COVER (mm)	CARBONATION DEPTH
00341800	Deck slab	Max:-0.01 Min:-0.23	50	-
00834850	Deck slab	Max:-0.180 Min:-0.350	25	0
DAMBAI (SABAH)	Deck slab	Max:-0.122 Min:-0.260	30	43
SAMARAHAN (SARAWAK)	Deck slab	Max:-0.041 Min:-0.107	30	12
00371000	Deck slab	Max:-0.07 Min:-0.24	Varies	-

The result of half cell test indicates that the rebar has not corroded yet or that corrosion is not serious yet. Visual inspection of the exposed rebar at the negative terminal point of half cell apparatus attest to the findings above .

9.5.5 Formation Analysis of Bearing Pads

It was often observed during the visual inspection stage that elastromeric bearing pads have deteriorated or deformed in some of the study bridges. In order to unravel the reason for the deterioration, the material sample was taken for laboratory tests in accordance with JKR's request so that an appropriate rehabilitation plan could be prepared. Two different samples were taken from Bridge No. 00701810 and Dambai Bridge in Sabah State. Only formation analysis tests were carried out due to the limited sample amount by the Chemical Inspection & Testing Institute in Japan. Summary of the test results are tabulated in Table 9-11.

Table 9-11 Summary of Formation Test Results

Test Item	Br.00701810	Dambai Br.	Requirement /1
Identification of Rubber Polymer	NR/SBR	CR	NR or CR
Total Rubber Polymer (%)	64.4	45.6	More than 50
Carbon Black (%)	22.9	38.3	15 ≈ 30
Ash (%)	4.9	2.3	Less than 10
Blend ratio	NR : SBR= 68.5:31.5	N/A	

Note: /1 Requirements in accordance with Japan Road Association
NR means Natural Rubber
SBR means Styrene Butadience Rubber
CR means Chloroprene Rubber

In general, rubber polymer used to a bearing pad is divided into three types comprising of natural rubber (NR), styrene butadience rubber (SBR), and chloroprene rubber (CR). Each type has different characteristics as summarized below:

	Aging Resistance	Ozone Resistance	Wear Resistance	Tensile Resistance
NR	Good	Bad	Very Good	Very Good
SBR	Bad	Bad	Very Good	Fair
CR	Very Good	Very Good	Good	Fair

As shown in the test results, the rubber polymer in bearing pad of Bridge 00701801 is a blended type consisting of NR and SBR which are less ozone resistance than CR. Therefore it is conclusive that the main reason of the deterioration is due to less ozone resistance of both NR and SBR and furthermore could be aggravated by water absorption leaking from defective expansion joints.

On the other hand, the quality of the bearing pad at Dambai bridge is generally at an acceptable level eventhough the total rubber polymer is slightly less and carbon black is more than the requirements. The main reason of the deformation is due to load concentration at lower flange edge since the bearing width is extremely wider than that of the lower flange width.

9.5.6 Alkali - Aggregate Reaction (AAR) Test

Alkali - Aggregate Reaction (AAR) is a mechanism of deterioration in concrete in which the alkaline pore solution in the concrete reacts with a siliceous aggregate. This reaction produces an alkali silicate gel which has the property of absorbing water and expanding, so disrupting the concrete.

The NALS reports stated cracks on the pile head of Bridge No 00319110 were due likely to Alkali Aggregate Reaction (AAR) based on the visual inspection. In order to scrutinize the NALS finding, five concrete core samples were taken from the vicinity of the pile head and the core samples were sent to Japan for AAR laboratory tests. Type of tests conducted and results are summarized below.

(1) **Visual Observation**

Test method : Code (DD4) of Japan Concrete Engineering Association

Purpose : To Identify types of coarse aggregate in the concrete and their composition.

Result : Andesite : 71%
Diorite : 29%

(2) **Mix Proportion Analysis of Hardened Concrete**

Test Method : Method recommended by Japan Cement Association

Purpose : To estimate concrete mix proportion

Results : As follows:

Unit Weight (kg/m^3)	Calcium Oxide (%)	Aggregate Amount (kg/m^3)	Cement Amount (kg/m^3)
2,284	16.3	1,818	377

(3) **Alkali Amount in Cement Used**

Test Method : Hydrothermal extraction method

Purpose : To estimate total alkali amount in cement

Result : 0.8% (Na_2O equivalent)

(4) **Total Chloride Amount Analysis of Hardened Concrete**

Test Method : Method recommended by Japan Concrete Engineering Association

Purpose : To estimate total chloride amount

Result :

Cl (kg/m^3)	NaCl (kg/m^3)
0.34	0.55

(5) **Alkali Silica Reaction Test**

Test Method : Chemical method (JIS A5308)

Purpose : To measure soluble silica amount and alkali density decrement

Result : As follows

Soluble silica amount (m mol/l)		Alkali density decrement (m mol/l)	
Measured value	Average	Measured value	Average
46, 46, 45	46	23, 29, 32	31

(6) **Polarization Microscope Observation**

Test method : Code of Japan Concrete Engineering Association (DD3 & DD4)

Purpose : To identify silica mineral and the crystal structure

Result : As follows

(i) Coarse Aggregate

o Coarse aggregate is crushed rocks of andesite and diorite.

o Ground mass of andesite replaced by micro quartz of which structure is strained.
(The micro quartz is reactive silica and is wavy extinction mineral)

o Micro quartz accounts to 80 - 85% in andesite.

o No reactive mineral in diorite.

(ii) Fine Aggregate

o Fine aggregate is natural sand.

o Composition of fine sand is as follows:

Mineral Sand Grain				Rock Sand Grain	
Quartz	Quartz /1	Alkali Felspar	Plagioclase	Granite	Quartz diorite
31%	15%	25%	tr	20%	91%

Note: /1 Wavy extinction

(iii) Hardened Cement

o Concrete carbonated upto 13mm depth from the surface

o Calcium hydroxide of which size is mostly more than 10 μm account to 10 -22% of the cement.

o The cement has 2 - 20 μm width cracks and most of them are filled in by gel.

(7) **Expansion Test**

Test method : Code of Japan Concrete Engineering Association (DD2)

Purpose : To measure expansion value

Result : An average expansion value of the concrete cores has reached 19μm after 12 weeks passed.

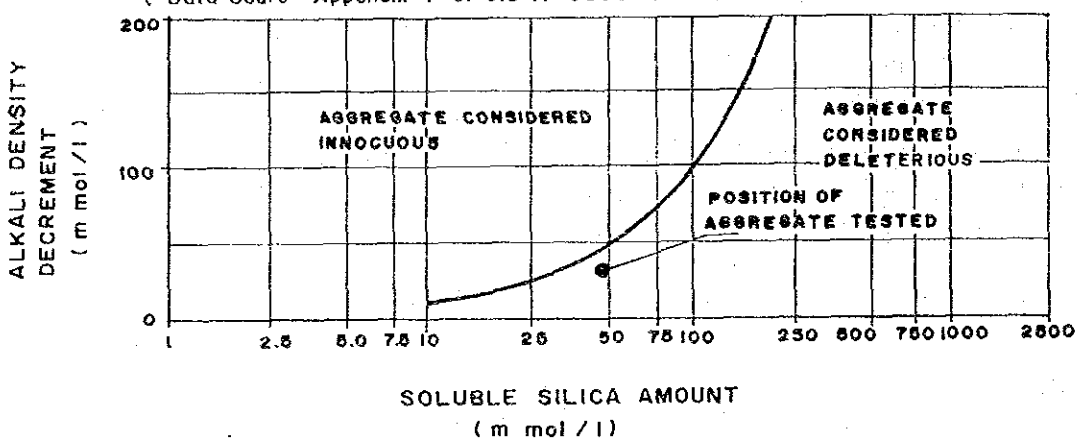
(8) **Conclusion**

- o Based on the result of the alkali silica reaction test, it is conclusive that the coarse aggregate used in the concrete is deleterious as shown in Figure 9-1.
- o Polarization microscope observation results indicate that the micro quartz which is reactive silica accounts for 80 - 85 % in andesite used as coarse aggregate and to 15% in fine aggregate.
- o Alkali amount test results show cement containing alkali accounts to 0.8% which is equivalent to 3.02kg/m^3. This value is slightly higher than the critical value of 3.0 kg/cm^3.
- o Hardened cement contains a lot of calcium hydroxide; more than that of normal cement.
- o Some of cracks were filled in by gel.
- o The core samples have been expanding.

Based on above findings, it is apparent that cracks on the pile head of Bridge No. 00319110 are due to Alkali Aggregate Reaction (AAR).

Figure 9-1 Judgement Diagram of Innocuous Degree of Aggregate

(Data Soure : Appendix 7 of JIS A - 5308 - 1989)



9.6 Bridge Function Survey

During the course of the detailed structural survey, a bridge function survey in terms of necessity of widening carriageway or adding sidewalks was carried out to scrutinize the preliminary findings presented in Chapter 2 and Chapter 4.

(1) Necessity for Widening Carriageway

Necessity for widening carriageways on all the study bridges was assessed by means of comparison with traffic capacity on a bridge (calculated by using formula given in "Highway Capacity Manual") and current demand volume at the same bridge location (based on data obtained from Traffic Volume Malaysia). Basically these assessment results were utilized in determining whether carriageway of the bridge should be widened or not.

(2) Adding Sidewalk

During the visual inspection stage, the necessity of adding a sidewalk was evaluated based on the criteria. That is whether a bridge without sidewalk is located in an urban area or not, or its proximity to public facilities such as schools, mosques and other land marks.

Applying the same criteria, the surrounding area of each bridge within about one km was surveyed to identify whether the bridge is located in the above condition or not. If a bridge is located in the above condition and there is no sidewalk, a sidewalk would be recommended.

(3) Survey Result

Table 9-12 is a survey result showing which bridge needs functional rehabilitation work, i.e. either adding sidewalk or widening carriageway.

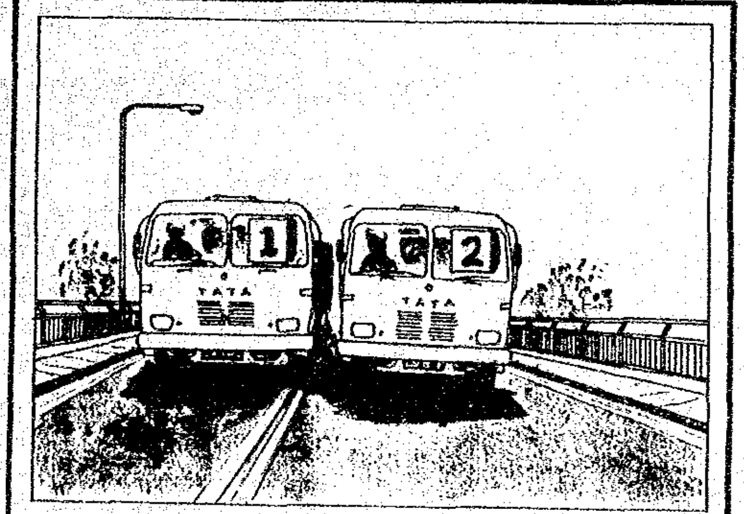
Table 9-12 Bridge Function Survey Results

Bridge No.	Survey Result	Bridge No.	Survey Result
00114920	N/A	00546560	N/A
00161140	Adding Sidewalk	00546980	N/A
00166510	Adding Sidewalk[1]	00563880	N/A
00237200	N/A	00567840	Widening carriageway
00317000	N/A	00834850	N/A
00319110	N/A	05001070	N/A
00341800	Adding Sidewalk[1]	05803340	N/A
00346740	N/A	05903120	N/A

Note : Above result is the same as preliminary results except bridge with [1], which is added based on the bridge function survey conducted in the detailed survey.

CHAPTER 10

FULL SCALE BRIDGE LOADING TEST



CHAPTER 10

FULL SCALE BRIDGE LOADING TEST

10.1 General

In most cases, actual load carrying capacity of bridges is higher than that obtained by theoretical calculation. This phenomena is caused by the difference between design and actual bridge behavior due to a result of material properties deviation, degree of difference in lateral load distribution, extent of composite action, etc. Most of bridges therefore have such a reserved residual loading capacity (RRLC), the value of which depends on type of structure, construction materials, and extent of defects or deterioration.

It is however difficult to estimate the reserved residual loading capacity by analysis alone, particularly on the effects of defects or deterioration to the overall performance of an existing bridge, but it is of importance to make it a requisite in assessment of the existing bridges. In order to overcome the difficulty, a full scale loading test could prove to be useful to determine the reserved residual loading capacity of the bridges instead of analysis.

The primary purpose of a full scale loading test in the Study is therefore to estimate the structural reserved/residual loading capacity of main component parts of the bridge. To achieve this, comparison of stress and deflection will be made between theoretically calculated values and those empirically measured. The other secondary purpose of the test is to ensure that structural theories and assumptions applied in the calculation are correct and suitable for use in the structural bridge assessment work.

Out of the 20 bridges for the detailed survey, a total of five spans at three different bridge sites listed below were selected for a full scale bridge loading test.

List of Bridges for Loading Test

Key	State	District	Bridge Type
00237200	Pahang	Kuantan	SBC & RCB
00319110	Pahang	Rompin	PCB & PCB
00834850	Kelantan	Kuala Krai	RCS

The loading tests conducted in the Study are broadly divided into 2 types; static and dynamic loading tests. Static loading test is to measure deflection and strain of main component part under a known load in which loaded vehicles are usually used. Dynamic loading test is to measure the actual working strain of the members under the existing traffic load. The static loading test was conducted at all the five spans listed above.

On the other hand, dynamic loading test was carried out only on bridge number 00237200 which is located on the heaviest trafficked among others.

In addition to above mentioned dynamic test, the Study Team also carried out dynamic loading test at bridge 00237200 to measure dynamic strain using a known load (a loaded vehicle) under different running speed, according to JKR's request.

10.2 Loading Test Planning

As preparatory work in the loading test planning, structural detailed survey, selection of measurement points, determination of loading position & loading case and traffic count survey were conducted in advance so as to ensure the final outputs meeting the purpose of the test.

10.2.1 Selection of Measurement Points

In general, the number and location of measurement points in a full scale loading test depend on the objective of the loading test, whether it is proof test, behavior test or research test such as assessment of the ultimate loading capacity. Since the main objective of the loading test in the Study is to assess the structural reserved/residual loading capacity of main bridge components (ratio of empirically measured value and theoretically calculated one under the same known loading condition), the observation point on the main bridge component where the measurement of strain and deflection was carried out, was selected so as to coincide with the maximum sectional forces induced.

The number and position of the measurement points at each selected observation point were determined mainly to enable the verification of the theoretical calculation and the measurement results.

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

SUMMARY OF MEASUREMENT POINTS

(1) STATIC TEST

BRIDGE KEY	TYPE	COMPONENT	STRAIN	DISPLACEMENT
00237200	SBC	Beam	2 points x 2 girders = 4	1 point x 2 girders =2
		Deck Slab	2 points x 1 location= 2	1 point x 1 location=1
		Beam	1 point x 2 girders = 2	1 point x 2 girders =2
		Deck Slab	2 points x 1 location= 2	1 point x 1 location=1
		Subtotal of Measurement Points	10	6
00319110	PCB(C)	Beam	6 points x 2 girders =12	1 point x 2 girders =2
	PCB(S)	Beam	6 points x 1 girders = 6	1 point x 1 girders =1
		Deck Slab	2 points x 1 location= 2	1 point x 1 location=1
		Subtotal of Measurement Points	20	4
00834850	RCS	Slab	2 points x 2 locations=4	1 point x 1 location=1
		Subtotal of Measurement Points	4	1
Total Measurement Points			34	11

(2) DYNAMIC TEST

KEY	BRIDGE TYPE	COMPONENT	STRAIN
00237200	SBC	Beam	2 Points x 2 girders = 4
	RCB	Beam	1 Points x 2 girders = 2
Total Measurement Points			6

10.2.2 Determination of Loading Position and Loading Case

The loading position and number of trucks which will induce the most severe load effect to a member under consideration (i.e inducing the maximum moment, shear force or displacement at the selected observation point) were examined based on an influence line diagram. Based on this concept, the loading position of each case together with number of trucks to be used were finally determined as summarized in Table 10-1. Plan, profile and cross sections of each loading case are illustrated in Appendix-L.

10.2.3 Traffic Count Survey

A twenty four hour traffic count survey was carried out at Bridge No 00237200 on 6 November 1991 to identify peak and low traffic hours (for both total traffic volume and heavy vehicle). Based on these traffic data as shown in Figure 10-1, time and duration of the dynamic test and time of the static test were decided in order to obtain the most severe load effect and to minimize any traffic interruption respectively during the test period.

Table 10-1 Loading Position and Loading Case

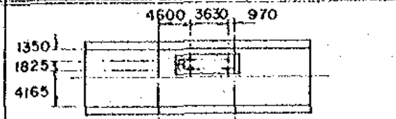
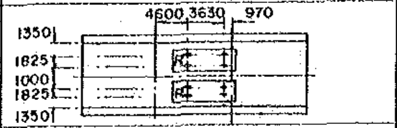
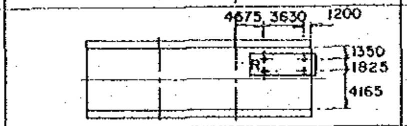
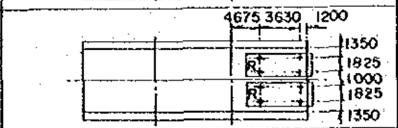
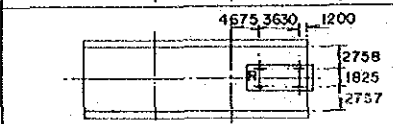
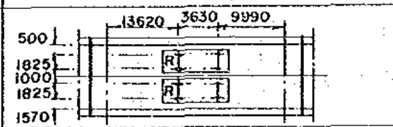
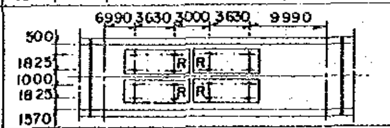
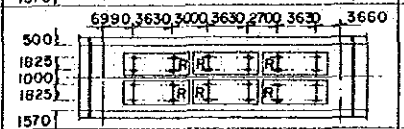
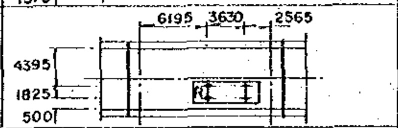
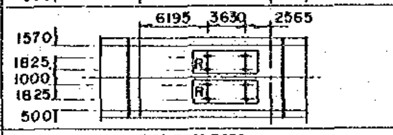
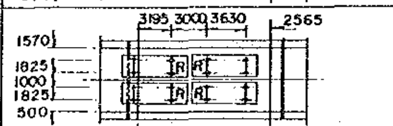
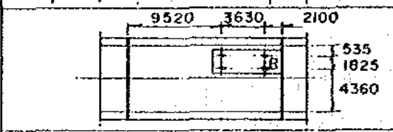
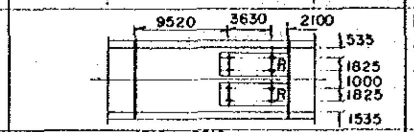
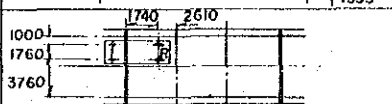
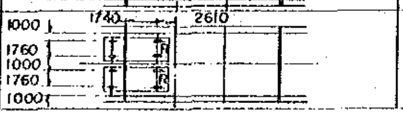
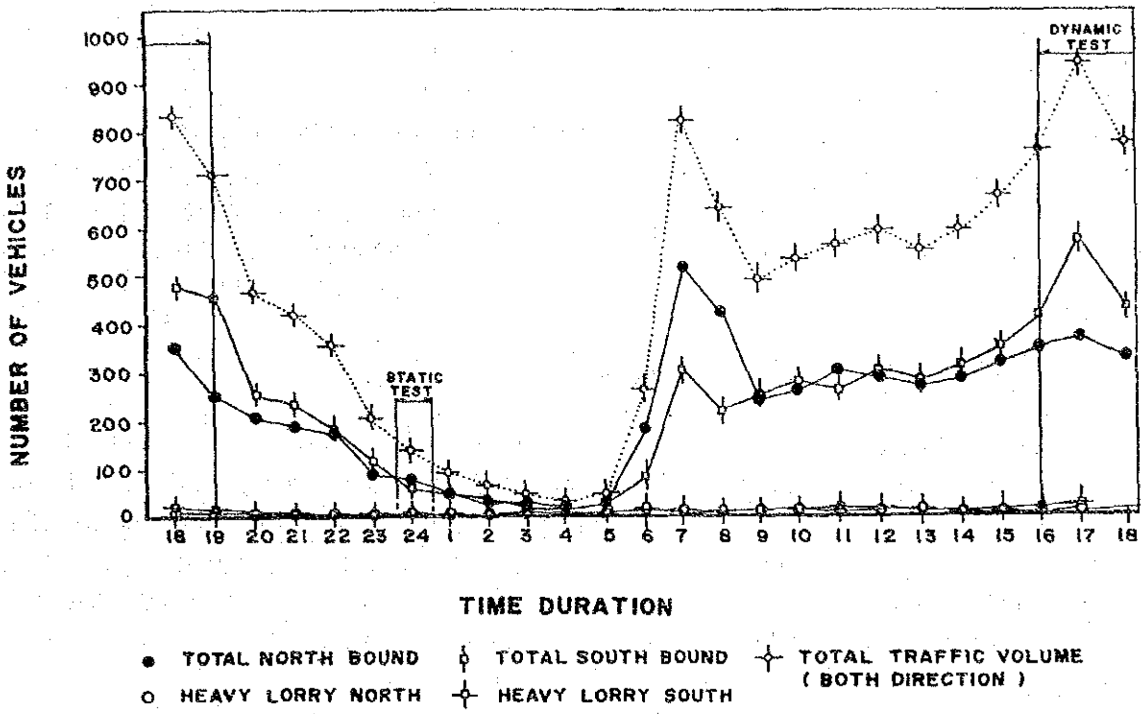
KEY	BRIDGE TYPE	LOADING CASE	NO. OF TRUCK	LOADING POSITION	REMARKS
237200	SBC	1	1		Inducing moment and deflection of main beam under consideration.
		2	2		Inducing maximum moment and deflection of main beam under consideration.
		3	1		Inducing maximum moment and deflection of deck slab under consideration.
237200	RCB	1	1		Inducing moment and deflection of main beam under consideration.
		2	2		Inducing maximum moment and deflection of main beam under consideration.
		3	1		Inducing maximum moment and deflection of deck slab under consideration.
319110	PCB (Center Span)	1	2		Inducing moment and deflection of main beam under consideration.
		2	4		Inducing moment and deflection of main beam under consideration.
		3	6		Inducing maximum moment and deflection of main beam under consideration.
	PCB (Side Span)	1	1		Inducing moment and deflection of main beam under consideration.
		2	2		Inducing moment and deflection of main beam under consideration.
		3	4		Inducing maximum moment and deflection of main beam under consideration.
		4	1		Inducing moment and deflection of deck slab under consideration.
		5	2		Inducing maximum moment and deflection of deck slab under consideration.
834850	RCS	1	1		Inducing moment and deflection of deck slab under consideration.
		2	2		Inducing maximum moment and deflection of deck slab under consideration.

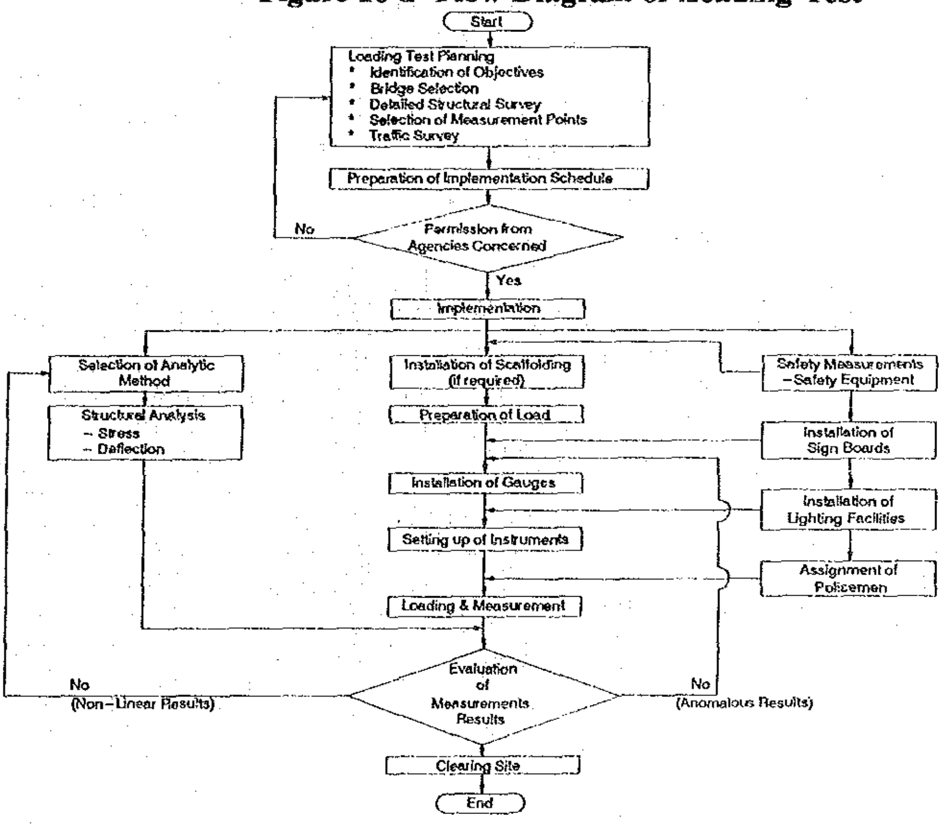
Figure 10-1 Hourly Fluctuation of Traffic Volume at Bridge No. 00237200



10.3 Method of Loading Test

A flow diagram showing the practical loading test procedure together with above mentioned planning is shown in Figure 10-2.

Figure 10-2 Flow Diagram of Loading Test



10.3.1 Preparation of loads

The load is normally applied by using loaded vehicles which are usually loaded with crushed aggregate or concrete blocks. Before placing the load on a bridge, configuration of the vehicle (usually dump truck) including distance of axles and weight of each axle must be measured in advance using a conventional tape and a weigh bridge respectively. This data is necessary for the exact determination of the loading position on a bridge and for structural analysis. The exact position of the wheel is marked on the surface of the pavement using a specific color paint.

10.3.2 Preliminary Structural Analysis

Any damages to the bridges by the most severe load to be applied during the loading test should be prevented. Hence, structural analysis using data collected through the detailed survey was carried out in advance so as to ensure that no damages on the structure arise from the loading. The assessment was made by comparison of the sectional forces due to design live load applied and due to the most severe loading condition to be applied. The result of the analysis was also presented in terms of stresses and displacements due to the load to be applied during the test.

From the result of the analysis based on the above methodology, it has been found that the most severe loading condition is equivalent to about 70 to 80 % of the design load applied, hence it is likely to be safe for the existing bridges. The following results were obtained as a reference to check the reliability of the test results.

Strain and Displacement of Considered Points

KEY	BRIDGE TYPE	BRIDGE PART	MEASUREMENT POINT	STRESS (N/sq.mm)	DISPLACEMENT (mm)	REMARK
00237200	SBC	Beam (G2)	Lower Flange	19.3	1.00	<1
		Beam (G3)	Lower Flange	20.5	1.07	<1
		Deck Slab	Main Rebar	24.9	0.18	<2
		Deck Slab	Dis. Rebar	46.0	N/A	<2
	RCB	Beam (G2)	Main Rebar	25.1	1.26	<3
		Beam (G3)	Main Rebar	28.6	1.44	<3
		Deck Slab	Main Rebar	18.9	0.09	<2
		Deck Slab	Dis. Rebar	27.9	N/A	<2
00319110	PCB (C)	Beam (G1)	Lower Flange	5.2	10.53	<4
		Beam (G2)	Lower Flange	5.4	11.63	<4
	PCB (S)	Beam (G1)	Lower Flange	3.4	1.48	<5
		Deck Slab	Main Rebar	31.9	1.93	<6
		Deck Slab	Dis. Rebar	83.9	N/A	<6
00834850	RCS	Slab	Main Bar (in)	25.5	0.43	<7
			Main Bar (in)	46.5	N/A	<7
			Main Bar (out)	6.9	N/A	<7
			Main Bar (out)	0.0	N/A	<7

Notes: Structural theories and assumption applied in above analysis are as follows:

<1. o Stiffness of deck slab, girder and cross beam is considered for lateral load distribution using othotropic plate theory of Guyon & C.Massonette.
   o Composite action is considered.
   o Young's modulus of steel is assumed to be 2.1 x 10E + 6 kgf/sq.cm.
   o Modulus ratio of steel and concrete, n = 15 is applied

<2. o Converted to infinite isotropic simply supported plate.
   o Young's modulus of deck slab is assumed to be 2.5 x 10E + 5 kgf/sq.cm.

<3. o Stiffness of deck slab, girder and cross beam is considered for lateral load distribution using othotropic plate theory of Y. Guyon & C. Massonnet.
   o Young's modulus of R.C concrete is assumed to be 2.8 x 10E + 5 kgf/sq.cm.

<4. o Stiffness of deck slab, girder and cross beam is considered for lateral load distribution using othotropic plate theory of Y.Guyon & C.Massonnet.
   o Young's modulus of P.C concrete is assumed to be 3.1 x 10E + 5 kgf/sq.cm.

<5. o No lateral load distribution was considered because it was a two girder type bridge.
   o Young's modulus of P.C concrete is assumed to be 3.1 x 10E + 5 kgf/sq.cm.

<6. o Converted to four sides fixed isotropic plate by girders and cross beams.
   o Young's modulus of deck slab concrete is assumed to be 3.1 x 10E + 5 kgf/sq.cm.

<7. o Applied finite isotropic simply supported plate theory.
   o Young's modulus of deck slab is assumed to be 2.5 x 10E + 5 kgf/sq.cm.

10.3.3 Safety Management

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

- **Installation of Traffic Sign Boards**

The traffic sign boards such as public notice sign boards, regulative sign boards were prepared and installed in a timely manner by JKR district concerned.

- **Assignment of Traffic Policemen**

During the period of the loading test, 2-3 traffic policemen were assigned at the site to ensure security and to control the traffic.

- **Systematic Arrangement of Loaded Trucks**

In order to minimize duration of the traffic interruption, loading sequence of the dump trucks was carefully studied and determined in advance.

10.3.4 Installation of Scaffolding

Installation of scaffolding is one of the major preparatory works in the loading test. Scaffolding is required to provide access to the soffit of deck slab and girders if the bridge clearance height up to ground level is more than 3-4m.

The scaffolding is divided into two types, pipe support type and hanger type which have been commonly used in bridge maintenance and rehabilitation works. In the course of the loading test, the scaffolding sustains the workers for preparation works and for observation of concrete cracks. The scaffolding applied in the test was individually designed taking into consideration surcharge load such as equipment and workers totaling 1500 N/sq.m and the dead load. The drawings are attached in Figures 10-3 & 10-4. The scaffolding in the loading test was installed and removed in a timely manner by a local contractor.

Figure 10-3 Hanger Type Scaffolding

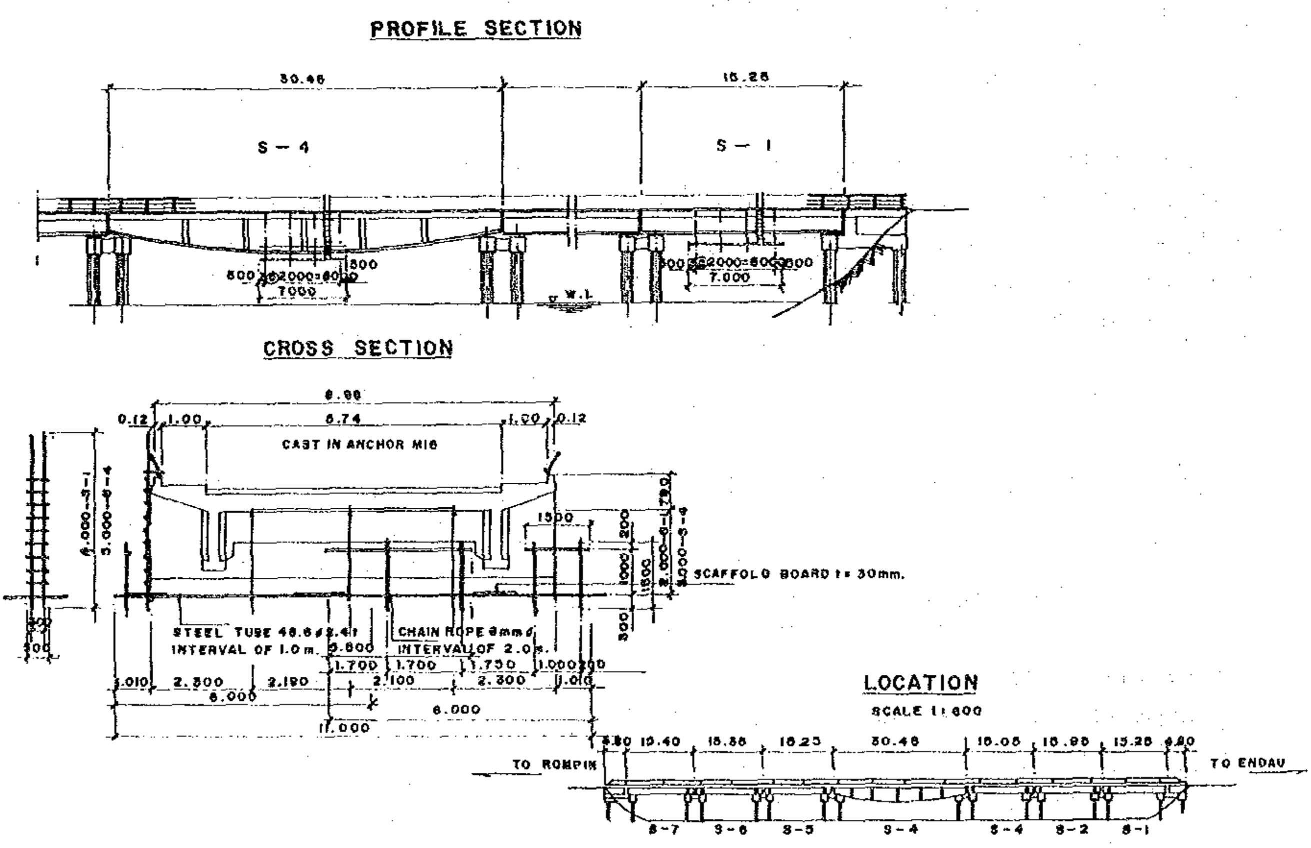
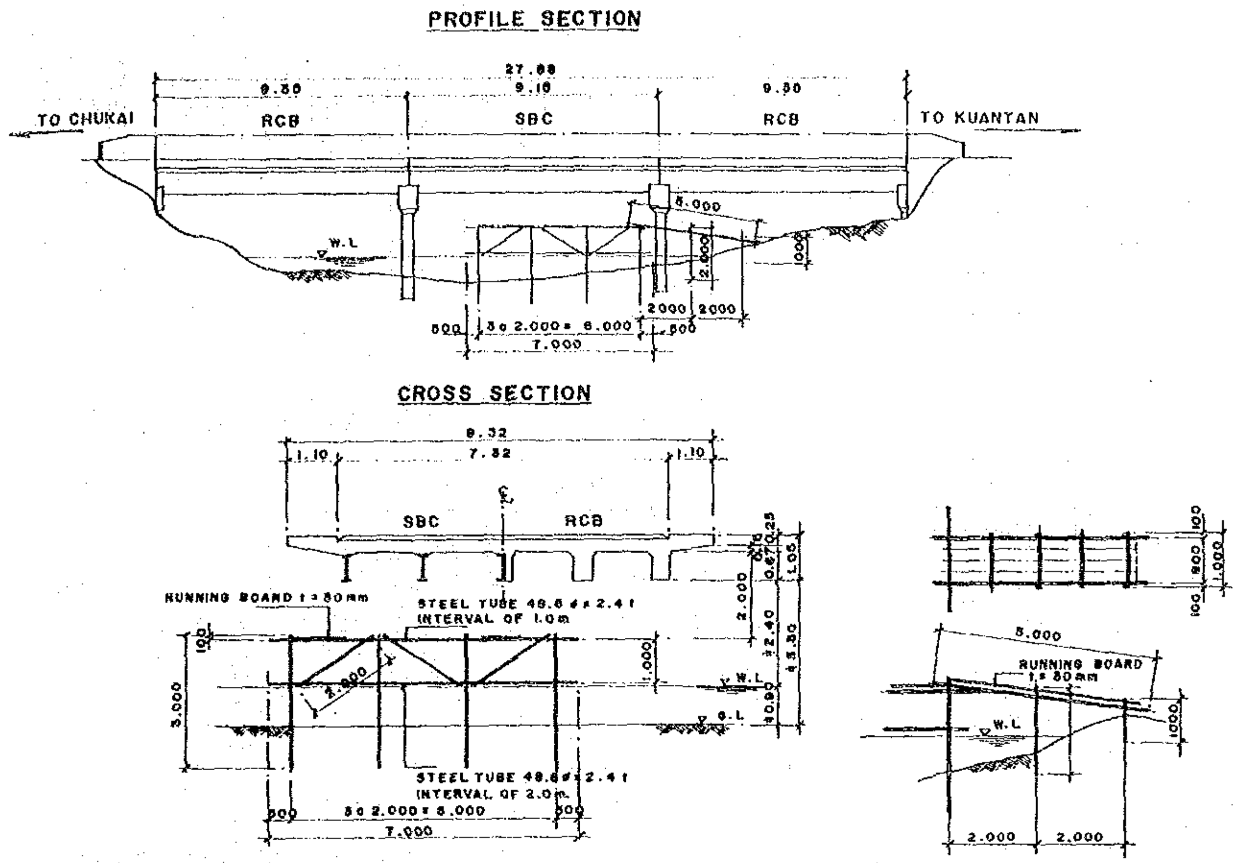


Figure 10-4 Pipe Support Scaffolding



10.3.5 Selection and Installation of Gauges

Several types of gauges can be used in conjunction with bridge loading test. Resistance wire strain gauges commonly applied in load tests in which strains are measured by the changes in the electrical resistance of the gauges and can be measured very accurately. Gauges used in the test consist of a strain gauge for concrete structures, a strain gauge for steel structures and a dial gauge to measure deflection. Specification and type of each gauge applied are as follows:

-Strain Gauge for Concrete (Type; KC-70-A1-11)

KIND / TYPE		MATERIAL		OPERATING TEMPERATURE RANGE (°C)	COMPENSATED TEMPERATURE RANGE (°C)	STRAIN LIMIT AT ROOM TEMP (%)	FATIGUE LIFE AT ROOM TEMP.	APPLICABLE ADHESIVE	MAIN APPLICATION/ FEATURES
		RESISTIVE ELEMENT	BASE						
For Concrete	Phester Gage KC	Cu-Ni Wire	Phester	-196~+150	+10~+80	1.8 %	$1.5x10^5$	PC-6, CC-33A PC-12, EP-18	General Stress

-Strain Gauge for Steel (Type; KFG-5-120-C1-11)

KIND / TYPE		MATERIAL		OPERATING TEMPERATURE RANGE (°C)	COMPENSATED TEMPERATURE RANGE (°C)	STRAIN LIMIT AT ROOM TEMP (%)	FATIGUE LIFE AT ROOM TEMP.	APPLICABLE ADHESIVE	MAIN APPLICATION/ FEATURES
		RESISTIVE ELEMENT	BASE						
General Purpose	Foil Phester Gage KFG	Cu-Ni Foil	Phester	-196~+150	+10~+100	5.0 %	12×10^6	CC-33A,PC-6	General Stress Residual Stress transducer

-Dial Gauge (Type; DT-20D)

Measuring range	20mm	Compatible bridge voltage	12V (AC, DC)
Reaction force	App. 200g	Repeatability	0.3 % FS
Output voltage sensitively	1.0mV/V+0.5%	Frequency response	App. 1 Hz
Non-linearity	0.5 % FS	Guaranteed temp. range	-10~± 50°C
Hysteresis	0.5 % FS	Weight (w/o cable)	App. 310g
Input-Output impedance	350 Ω± 2 %	Installation	By means of bolts

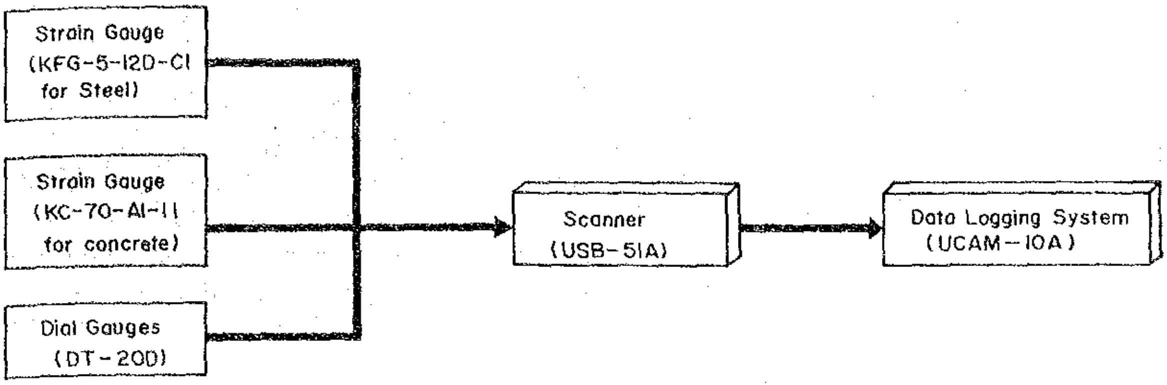
After completely cleaning the surface of the structural members to be measured by sand-paper (#100-300) and thereafter by acetone, a strain gauge was then bonded to the surface of the member using an exclusive adhesive. And later it was coated using butyl rubber material to prevent it from absorbing moisture in out-door condition. After bonding and coating the gauge completed, the lead wire was connected to the data logging system.

10.3.6 Instrumentation and Equipment

Type of instrument and equipment used in the loading test generally depend on the objectives. In the load test they are broadly divided into static and dynamic measurement equipment:

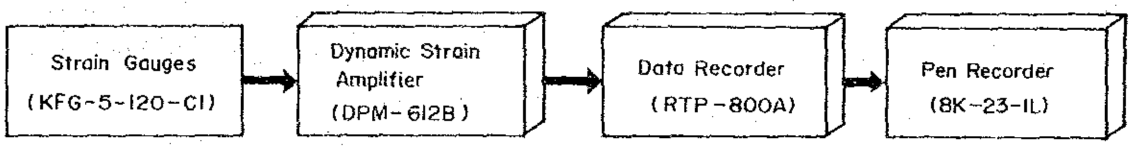
(1) Static Measurement

The measurement system shown in the following block diagram was applied to the static loading test. This method was chosen because of the considerable number of measuring points in the test and also the method minimizes recording time and prints out the physical values directly.



(2) **Dynamic Measurement**

The objective of dynamic loading test is to measure dynamic strain of members under the existing traffic over a period of peak hours. The results will reveal a fluctuation of the actual working strain. Considering the number of measuring points and the measurement period of three hours, a measurement system shown in the following block diagram was applied to the dynamic loading test.



10.3.7 Loading and Measurement

After completion of above mentioned preparatory works at the job site, the trucks were slowly moved from bridge end to the designated point. Shortly after completing decrement of the vibration, the measurement was read in the case of the static test. This practice was repeated for all the loading cases as shown in Table 10-1.

While in the dynamic test, dynamic strain due to the existing traffic was automatically recorded over the period using data recorder.

10.4 Results of Loading Test

The loading tests were conducted for a total of five spans at three different bridge sites during the period from 18 to 27 November 1991 in accordance with the plan and the method mentioned in sections 10.2 and 10.3.

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

10.4.1 Assessment of Static Measurement Results

The measurement values obtained at the job site usually contain a residual strain value or 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. Such values are stored in the Data Logger after removal of all the loads. Those are about 3 micro millimeters in steel strain gauge (which is equivalent to 0.6N/sq.mm in terms of stress) and about 16 micro millimeter in concrete strain gauge (which is equivalent to 0.5N/sq.mm).

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

(1) **Bridge No. 00237200 (SBC and RCB)**

(a) **Main Beam of SBC**

Comparison of stresses and deflections between empirically measured and theoretically calculated is tabulated in Table 10-2.

Table 10-2 Comparison of Test Result Value and Calculated Value (Main Beam of SBC)

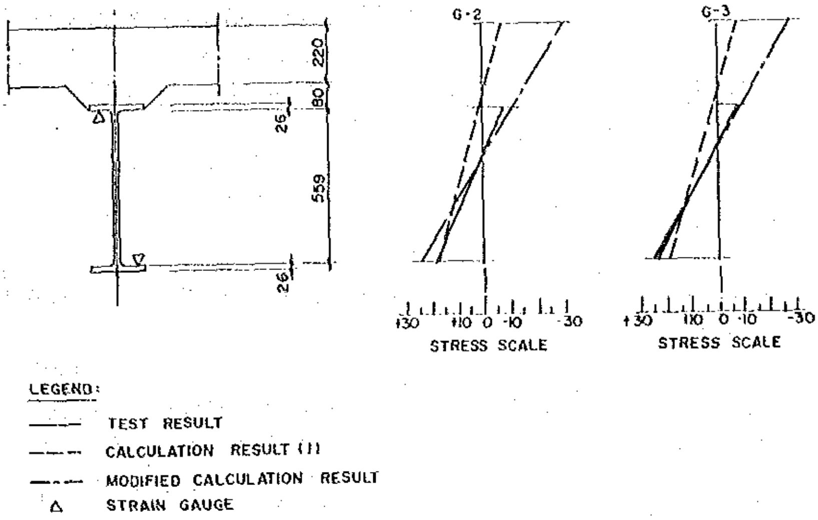
Measurement Point			Unit	Loading Case – 1					Loading Case – 2				
				Test Results	Calculation Results (1)	Calculation Results (2)	Ratio		Test Results	Calculation Results (1)	Calculation Results (2)	Ratio	
				A	B	C	A/B	A/C	A	B	C	A/B	A/C
GIRDER-2	Stress	Upper	N/mm2	−6.40	0.20	−20.30	–	0.32	−8.20	0.20	−32.20	–	0.26
		Lower	N/mm2	15.80	12.20	20.30	1.30	0.78	18.50	19.30	32.20	0.96	0.58
		Web	N/mm2	3.00	3.50	–	0.86	–	3.70	5.60	–	0.66	–
	Deflection		mm	1.03	0.64	2.14	1.61	0.48	1.34	1.00	3.37	1.34	0.40
GIRDER-3	Stress	Upper	N/mm2	−3.20	0.10	−17.40	–	0.18	−7.40	0.30	−34.00	–	0.22
		Lower	N/mm2	12.40	10.50	17.40	1.18	0.71	24.90	20.50	34.00	1.21	0.73
		Web	N/mm2	2.30	3.00	–	0.77	–	4.70	5.90	–	0.80	–
	Deflection		mm	0.87	0.54	1.83	1.61	0.48	1.75	1.07	3.60	1.64	0.49

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

* Deflection ratio (Test result / Calculation result) between A and B indicates that the actual rigidity of the beams is considerably less than their value used in the preliminary calculation. This means the beams have not enough composite action compared with that in the original design.

* In order to reveal the effectiveness of the actual composite action, a modified calculation applying the same neutral axis position from the test result was carried out and the results are illustrated in Figure 10-5.

Figure 10-5 Stress Distribution Diagram



* From the modified calculation result, it can be said that the effective deck slab widths are only 220 mm in Girder-2 and 320 mm in Girder-3, which are extremely less than those likely considered in the original composite design. This phenomenon could be caused by inadequate stiffness of shear connectors and inferior slab concrete strength. Therefore, the beams are almost working as a non-composite beam rather than a composite beam as originally designed.

* Alternative calculation as a non-composite beam based on above fact was carried out with consideration of lateral load distribution. The results are presented in Table 10-2 as calculation result (2).

* Furthermore, to ensure service level of the beams under a non composite beams, the beam stress due to dead load and HA load (which is design live load of the original design) was calculated. The result as shown in Table 10-3 indicates the beams have enough HA loading carrying capacity.

Table 10-3 Working Stress due to HA Loading and Dead Load

Member	Unit	Working Stress			Allowable Stress
		Dead Load	HA	Total	
Girder-1	N/mm^2	59.0	66.2	125.2	< 142.0
Girder-2	N/mm^2	41.0	72.2	113.2	< 142.0
Girder-3	N/mm^2	41.1	74.5	115.6	< 142.0