* Finally, the effect of the composite action in SBC is suspicious and thus from conservative viewpoint it is recommended that this effect is neglected in the assessment of SBC type bridge, i.e. steel beams of SBC are considered as a non-composite beams. Under such conditions, it can be said that this type of beam in SBC has a reserved residual loading capacity of at least 20 % in terms of the design stress as shown by stress ratio (A/C) in Table 10-2.

(b) Main Beam of RCB

Table 10-4 is a comparison table in terms of stress and deflection between empirically measured and theoretically calculated.

Table 10-4 Comparison of Test Result Value and Calculated Value (Main Beam of RCB)

Measurement Point		Unit	Loading Case-1			Loading Case-2		
			Test Results A	Calculation Results B	Ratio A/B	Test Results A	Calculation Results B	Ratio A/B
Girder-2	Stress of Main Rebar	N/mm^2	13.70	17.40	0.79	17.00	25.10	0.68
	Deflection	mm	0.70	0.88	0.80	0.95	1.26	0.75
Girder-3	Stress of Main Rebar	N/mm^2	11.10	14.60	0.76	22.30	28.60	0.78
	Deflection	mm	0.58	0.73	0.79	1.13	1.44	0.78

* Variation of both stress ratio and deflection ratio in each loading case as well as in each girder are very minimal. Thus it can be concluded that both of the test results and the calculation results are reliable. Moreover, othotropic plate theory by Y.Guyon & C.Massonnet applied in lateral load distribution is likely to be suitable to theoretically analysis of the existing bridges.

* Stress ratio (A/B) of the main rebar and deflection ratio (A/B) of the main beam in both loading cases are between 0.68 and 0.79. The reason for this phenomenon i.e. mitigating the working stress is likely to be caused by the considerable effect of secondary rebars installed, some contribution of concrete tension strength and the composite action of the asphalt surface layer. Accordingly, this type of beams in RCB, if properly constructed, has reserved residual loading capacity of about 20 % in terms of the design stress.

(c) Deck Slabs of SBC and RCB

Table 10-5 below shows the comparison in terms of stress and deflection between empirically measured and theoretically calculated.

Table 10-5 Comparison of Test Result Value and Calculated Value (Deck Slab)

Measurement Point		Unit	SBC			RCB		
			Test Results A	Calculation Results B	Ratio A/B	Test Results A	Calculation Results B	Ratio A/B
Stress	Main Rebar	N/mm^2	19.40	24.90	0.78	17.40	18.90	0.92
	Distr. Rebar	N/mm^2	5.30	46.00	0.12	5.10	27.90	0.18
Deflection		mm	0.23	0.18	1.28	0.05	0.09	0.56

* Stresses in the main rebar obtained by the test results are relatively close to those by the theoretical calculation result, while differences in the distribution rebar stresses are remarkable. This difference could be derived from nonconformity between support conditions in the theoretical calculation and in the actual behaviour.

* However, the absolute value of deflection both by the test results and by the calculation results is too small to assess the deflection ratio. Thus it is difficult to derive a definitive finding from this result.

* The working stress of the main rebar due to the loaded wheel load of 7 ton is only 19.4 N/sq.mm in SBC or 17.4 N/sq.mm in RCB, which is equivalent to about 12 to 14 % of the allowable stress.

* Thus it could be concluded that the concrete deck slabs in RCB and SBC have enough durability even though stress due to dead load is added and design wheel load of 10 ton is applied.

(2) **Bridge No.00319110 (PCB and PCB)**

(a) Main Beam of Center Span

Table 10-6 is a comparison table in terms of stress and deflection between empirically measure and theoretically calculated.

* Deflection ratio (measurement result/calculation result) of 0.93 to 1.17 indicates that both of the test results and the calculation results are reliable including structural theory and assumptions applied in the calculation.

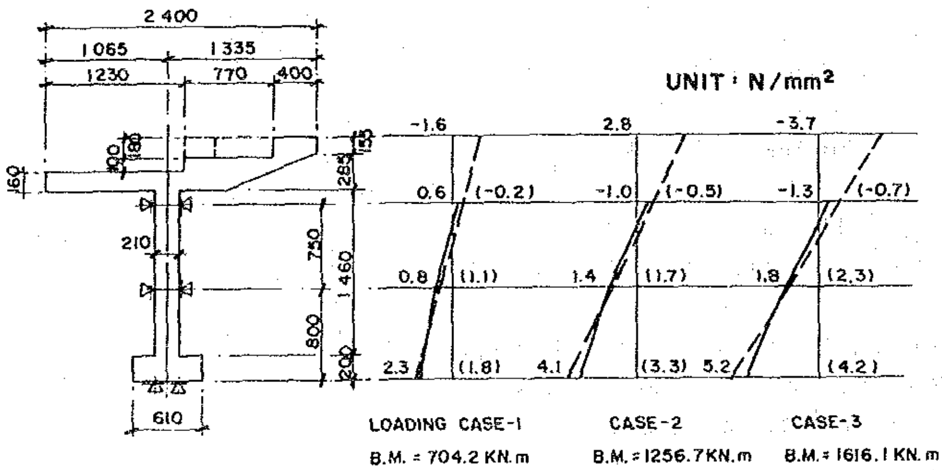
Table 10-6 Comparison of Test Result Values and Calculated Values (Main Beam)

Measurement Point			Unit	Loading Case – 1 Test Results A	Loading Case – 1 Calculation Results B	Loading Case – 1 Ratio A/B	Loading Case – 2 Test Results A	Loading Case – 2 Calculation Results B	Loading Case – 2 Ratio A/B	Loading Case – 3 Test Results A	Loading Case – 3 Calculation Results B	Loading Case – 3 Ratio A/B
GIRDER-1	Stress	Upper	N/mm^2	–0.20	–0.60	–	–0.60	–1.00	–	–0.70	–1.30	–
		Middle	N/mm^2	1.10	0.80	–	1.70	1.40	–	2.30	1.80	–
		Lower	N/mm^2	1.80	2.30	0.78	3.30	4.10	0.80	4.20	5.20	0.81
	Deflection		mm	4.76	4.06	1.17	8.13	7.80	1.04	10.63	10.53	1.10
GIRDER-2	Stress	Upper	N/mm^2	–0.10	–0.90	–	–0.20	–1.60	–	–0.40	–2.10	–
		Middle	N/mm^2	0.20	0.70	–	0.30	1.30	–	0.40	1.70	–
		Lower	N/mm^2	1.80	2.40	0.75	3.20	4.20	0.76	3.90	5.40	0.72
	Deflection		mm	4.16	4.49	0.93	7.90	8.63	0.92	10.86	11.63	0.93

* Stress distribution diagram for each beam under consideration is depicted in Figure 10-6 and Figure 10-7. The diagrams show the measurement stress at each point linearly changing in proportion to the distance from the neutral axis, except stress at middle web of G-2. It seems like the gauge at this measurement point could not work properly due to improper bonding or coating. It is however consistent that the test result is relatively reliable and the calculation method is suitable.

* Furthermore, the relation diagram between the bending moment due to the truck load in each case and the corresponding induced bending stress at lower flange is also depicted in Figure 10-8.

Figure 10-6 Stress Distribution Diagram, Main Beam (G-1) of PCB Center Span



NOTE & LEGEND:-

——— STRESS DISTRIBUTION ON THE TEST RESULT.

— — — STRESS DISTRIBUTION ON THE CALCULATION RESULT.

B. M. MEANS BENDING MOMENT

FIGURE IN ( ) SHOWS STRESS ON THE TEST RESULTS.

△ GAUGE

Figure 10-7 Stress Distribution Diagram, Main Beam (G-2) of PCB Center Span

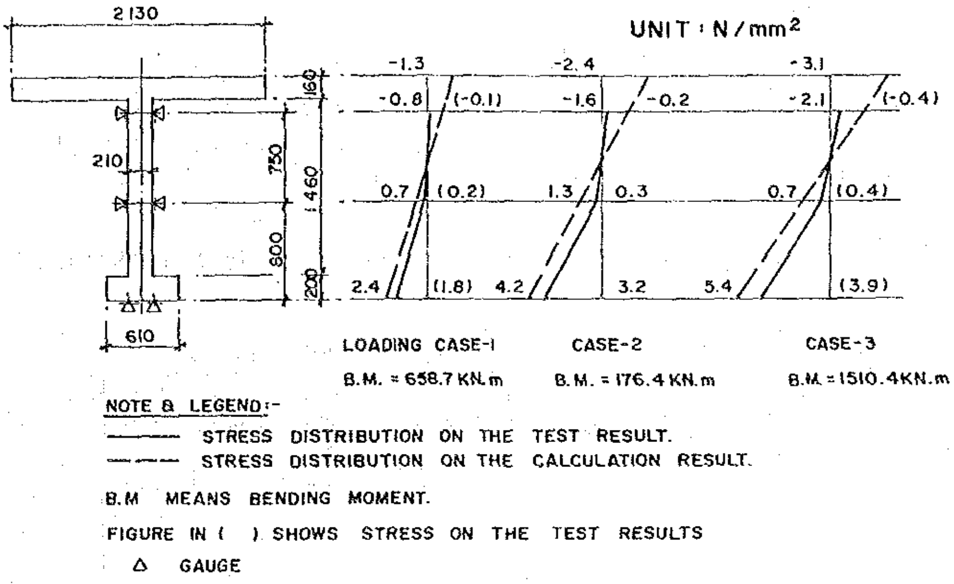
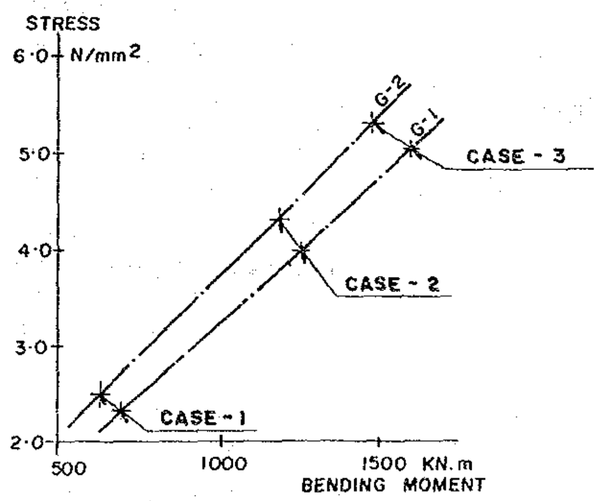


Figure 10-8 Relation Diagram between Bending Moment and Stress



* Above diagram indicates that the bending moment and the stress are entirely in proportion. Therefore, it can be concluded that the beams are working as an elastic body and the prestressing force is still working effectively.

* As seen in Table 10-6, stress ratio in G-1 and in G-2 is 0.81 and 0.72 respectively. The reason for this phenomenon, i.e.mitigating the working stress, is likely to be caused by additional rigidity of the beam due to the effect of load distribution and composite action by asphalt surface layer, side walk concrete, etc. These effects may not have been considered in the original design.

* Accordingly, this type of beam in PCB, if properly constructed, has reserved residual loading capacity of about 20% in terms of the design stress.

(b) Main Beam of Side Span

Comparison in terms of stress and deflection between empirically measured and theoretically calculated is tabulated in Table 10-7.

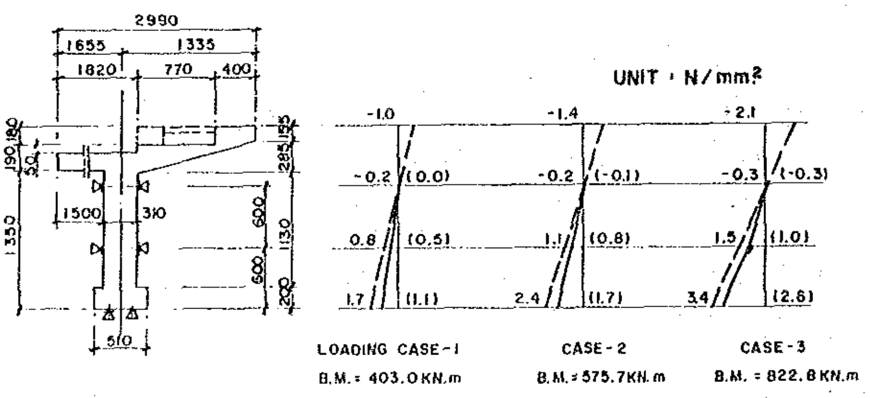
Table 10-7 Comparison of Test Result Value and Calculated Values (Main Beam, G-1)

Measurement Point		Unit	Loading Case - 1			Loading Case - 2			Loading Case - 3		
			Test Results A	Calculation Results B	Ratio A/B	Test Results A	Calculation Results B	Ratio A/B	Test Results A	Calculation Results B	Ratio A/B
Stress	Upper	N/mm^2	0.00	-0.20	-	0.10	-0.20	-	1.00	-0.30	-
	Middle	N/mm^2	0.50	0.70	-	0.90	1.10	-	1.00	1.50	-
	Lower	N/mm^2	1.10	1.60	0.69	1.70	2.40	0.71	2.70	3.40	0.79
Deflection		mm	0.69	0.67	1.03	1.00	0.95	1.05	1.70	1.48	1.15

* Deflection ratio of 1.03 to 1.15 indicates that both of the test results and the calculation results are reliable including structural theory and assumptions applied in the calculation.

* Stress distribution diagram of Girder-1 (G-1) is depicted in Figure 10-9. The diagram shows the measurement stress at each point linearly changing in proportion to the distance from the neutral axis. This fact double-ensures reliability of both results.

Figure 10-9 Stress Distribution Diagram, Main Beam (G-1) of PCB Side Span



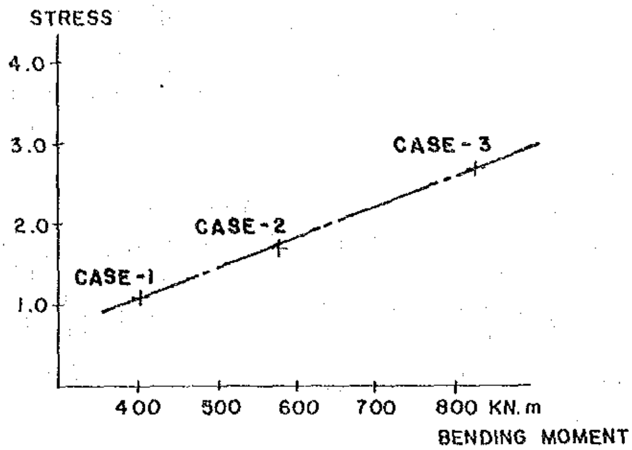
NOTE & LEGEND:-

——— STRESS DISTRIBUTION ON THE TEST RESULT.
– – – STRESS DISTRIBUTION ON THE CALCULATION RESULT.

B.M. MEANS BENDING MOMENT
FIGURE IN ( ) SHOWS STRESS ON THE TEST RESULTS.
Δ GAUGE

* Furthermore, the relation diagram between the bending moment due to the truck load in each case and the induced bending stress at lower flange is also depicted in Figure 10-10.

Figure 10-10 Bending Moment and Stress Relation Diagram



* Above diagram indicates that the bending moment and the stress are entirely in proportion. Therefore, it can be concluded that the beams are working as an elastic body and the prestressing force is still effectively working.

* As seen in Table 10-7 above, stress ratio of G-1 is 0.79. The reason for this phenomenon, i.e. mitigating the working stress, is likely to be caused by the additional rigidity of the beam due to the effect of load distribution and composite action by asphaltic surface layer, side walk concrete, etc. These effects may not have been considered in the original design.

* Accordingly, this type of beam in RCB, if properly constructed , has reserved residual loading capacity of about 20 % in terms of the design stress.

(c) **Deck Slab of Side Span**

Table 10-8 is a comparison table in terms of stress and deflection between empirically measured and theoretically calculated.

Table 10-8 Comparison of Test Result Value and Calculated Value (Deck Slab)

Measurement Point		Unit	Loading Case - 1			Loading Case - 2		
			Test Results A	Calculation Results B	Ratio A/B	Test Results A	Calculation Results B	Ratio A/B
Stress	Main Rebar	N/mm^2	12.40	26.30	0.47	12.40	31.90	0.39
	Distr. Rebar	N/mm^2	15.10	55.40	0.27	22.00	83.90	0.26
Deflection		mm	1.02	1.22	0.84	1.57	1.93	0.81

* Deflection obtained by the test results is relatively close to that by the calculation results in both loading cases. This means the assumptions and method applied in the analysis likely meet the actual deck slab behavior.

* Each stress ratio of main rebar and distribution rebar is considerably small and is less than 0.5. This mitigating working stress is likely to be caused by the effects of built-up slabs composing of deck slab and thick asphalt surface layer, and of wheel load distribution through a thick asphalt layer. These effects increase the rigidity of the deck slab more than that in the design.

* The working stress of the main rebar due to the loaded wheel load of 7 ton is only 12 N/sq.mm which is equivalent to less than 15 % of the allowable stress. Thus it could be concluded that the concrete deck slabs in PCB have enough durability even stress due to dead load is added and design load of 10 ton is applied.

(3) **Bridge No. 00834850 (RCS)**

Comparison in terms of stress and deflection between empirically measured and theoretically calculated is tabulated in Table 10-9.

Table 10-9 Comparison of Test Result and Calculated Value (Main Slab)

Measurement Point			Unit	Loading Case – 1					Loading Case – 2		
				Test Results	Calculation Results (1)	Calculation Results (2)	Ratio		Test Results	Calculation Results (1)	Ratio
				A	B	C	A/B	A/C	A	B	A/B
MIDDLE	Stress	Main Rebar	N/sq.mm	8.20	12.80	9.30	0.64	0.88	10.70	25.20	0.42
MIDDLE	Stress	Distr. Rebar	N/sq.mm	−0.60	25.20	4.10	–	–	−0.60	46.50	–
MIDDLE	Deflection		mm	0.29	0.22	0.24	1.32	1.21	0.46	0.43	1.08
EDGE	Stress	Main Rebar	N/sq.mm	4.30	4.00	5.20	1.08	0.82	5.10	6.90	0.74
EDGE	Stress	Distr. Rebar	N/sq.mm	−1.40	5.80	0.00	–	–	−1.60	0.00	–

* It is clear based from the test results that stress in the distribution rebar due to Case-1 Loading is similar to that due to Case-2 Loading, but the main rebar's stress of 8.2N /sq.mm and deflection of 0.29mm due to Case-1 Loading increase to 10.7N/sq.mm and 0.46 respectively under Case-2 Loading. From this fact, a construction joint longitudinally provided along the center line has no distribution rebar installed but it has a connector which transferrs shear force only.

* Based on above finding, an alternative calculation was made based on a model in which the slab is longitudinally separated, hence it is more severe than the actual structural condition. The results were presented in calculation (2) in Table 10-9. From these results, the measured stresses of the main rebar at each measurement point become close to that of the same point obtained by the alternative calculation.

* Concrete elastic modulus of 2.5 x 10E + 5 kgf/sq.mm which was obtained from laboratory test results of the concrete core was applied in the calculation of deflection due to both loading cases. The result shows that the values by the test are slightly higher than those by the calculation results. The reason is likely to be due to non-uniform or low quality of the concrete which was observed to have large scale honeycomb in many places on the slab soffit.

* Finally, stress ratio of the main rebar, 0.82 to 0.88 could give the inherent residual loading capacity of at least 10 % of the design stress in this type of RCS bridge, resulting from effect of built up slab action.

10.4.2 Assessment of Dynamic Measurement Results

(1) Dynamic Tests under Existing Traffic during Peak Hour

Table 10-10 presents a summary of the measurement results during a peak hour period of consecutive three hours. The detailed data consisting of traffic count data, fluctuation diagram of the stress and histogram of the stress during the period are attached in Appendix-M.

Table 10-10 Dynamic Test Results in terms of Working Stress under Existing Traffic (Unit : N/sq.mm)

Measurement Point			Static Test <1	Dynamic Test			
				Minimum	Maximum	Average	S.D
SBC	G-2	Upper Flange	-6.4	-10.39 (1.62)	-0.74	-1.63	1.574
SBC	G-2	Lower Flange	15.8	-4.75	+29.97 (1.90)	+1.22	5.160
SBC	G-3	Upper Flange	-3.2	-7.64 (2.38)	-0.42	-1.13	1.316
SBC	G-3	Lower Flange	12.4	-1.97	+23.38 (2.28)	+3.52	4.688
RCB	G-2	Main Rebar	13.7	-2.72	+27.41 (2.00)	+1.22	4.506
RCB	G-3	Main Rebar	11.1	-2.24	+25.71 (2.31)	+1.30	4.110

Notes:
* <1 Gross weight of the dump truck applied in the Static test is 17.45 tons and the results is under loading case 1.
* S.D. means standard deviation.
* Figure in ( ) shows ratio of dynamic test value and static test value

* The most dominant working stress of upper and lower flanges is -2.55 N/sq.mm and +2.57 N/sq.mm respectively in SBC, while +2.57 N/sq.mm is the most dominant stress in RCB during the period as indicated in the histogram attached in Appendix-M.

* The maximum working stress due to the existing traffic is +30.0 N/sq.mm in SBC or 27.4 N/sq.mm in RCB. These stresses are almost 2.28 to 2.31 times as large as the stress by the static test at the same measurement point. If impact stress is neglected, it is presumed that a vehicle with gross weight of about 40 ton passed on the bridge during the test period.

* The maximum working stress due to the 40 ton vehicle is equivalent to only 40 % of stress due to design live load (HA Load) in SBC, and 57% in RCB.

* Accordingly, the existing bridges have enough durability to live load derived from the existing traffic at this specific location.

(2) Dynamic Test under known Load at Different Running Speeds

Table 10-11 shows a summary of the measurement results at different running speed. A fluctuation diagram in terms of the stress drawn by a pen recorder is attached in Appendix-M.

Table 10-11 Dynamic Test Results in terms of Working Stress at Different Running Speeds (Unit : N/sq.mm)

Measurement Point			Static Test <1 (Loading Case-1)	Running Speed			
				10 km/h	20 km/h	30 km/h	50 km/h
SBC	G-2	Upper Flange	-6.40	-4.10	-3.50	-3.20	-4.00
		Lower Flange	15.80	13.20	12.40	+12.00	10.60
	G-3	Upper Flange	-3.20	-1.90	-1.90	-1.20	-3.00
		Lower Flange	12.40	9.40	9.60	+7.00	11.80
RCB	G-2	Main Rebar	13.70	12.30	12.60	+11.40	9.70
	G-3	Main Rebar	11.10	9.50	9.70	+6.40	10.60

Note : <1 The test result is under loading case-1

* In general, impact stress due to live load is related to the smoothness of the surface and running speed of a vehicle. It is, however, difficult to derive such a relation from the test results tabulated above. The reason for the above discordance is likely to be due to the wheel position under static test not coinciding with the wheel locus under the dynamic test.

10.4.3 Application of Test Results to Rehabilitation Design

It can be said that most of the study bridges were designed in accordance with HA Loading in BS 153, while LTAL Loading, which is the present JKR live load Standard applicable to the federal bridge, is almost 20 to 30 % higher than HA Loading in terms of sectional forces. Thus it is obvious that most of the stresses due to LTAL loading exceed the allowable stress applied in the original design, if LTAL is simply applied in the structural assessment without special consideration.

On the other hand, as proven by the assessment of the load test results, each bridge tested has some reserved residual loading capacity of a certain percentage against maximum design stress resulting from bridge behavior difference between in design and in actual, i.e.

mitigating actual working stress due to several effects such as composite action, built up action, lateral load distribution action and so on.

To this end, this reserved residual loading capacity (RRLC) could cover excess stress within a certain extent. Furthermore, it could be assumed from structural viewpoint that the same type of bridge has almost the same RRLC value if the bridge had been properly constructed without any major construction and design deficiencies. Therefore, the following reserved loading capacity will be considered to the corresponding type of bridge in the assessment of 20 bridges

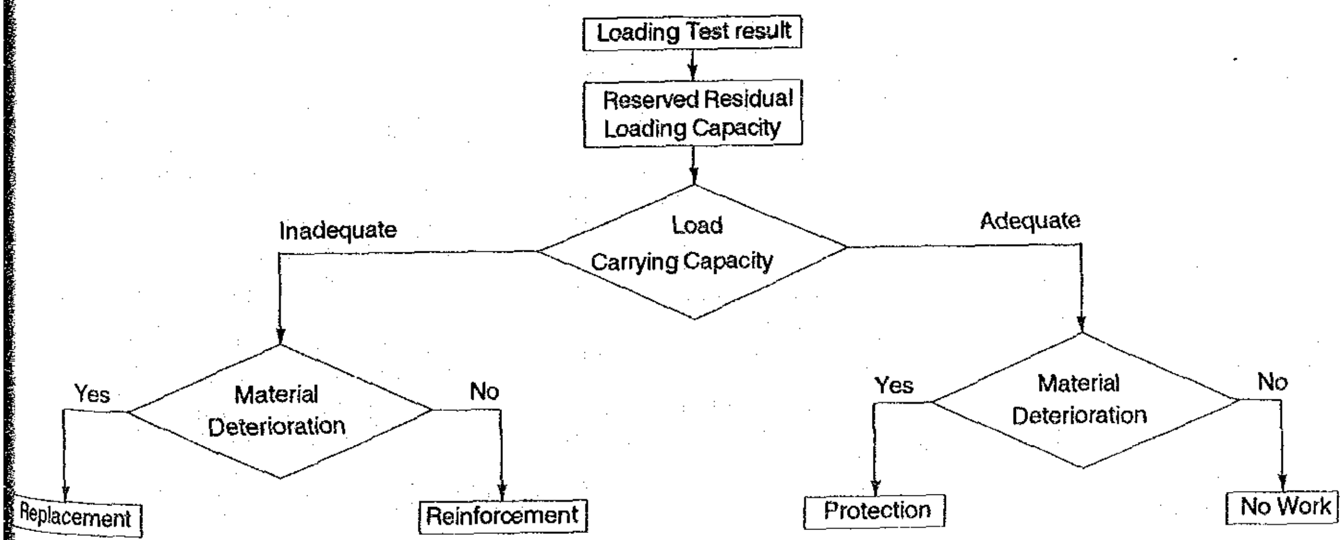
Bridge Type	Member	Reserved Residual Loading Capacity(%)
SBC	Main Beam	20 %
RCB	Main Beam	20 %
PCB	Main Beam	20 %
RCS	Main Slab	10 %

In the preliminary rehabilitation design, each of the 20 bridges will be theoretically assessed by applying an appropriate loading with due consideration of the reserved residual loading capacity to determine whether a bridge member has adequate load carrying capacity. This result will be used as one of the criteria to determine a broad rehabilitation plan. i.e. whether minor work including protection work or strengthening work including replacement is required if a bridge member has inadequate load carrying capacity.

Following the above broad classification of rehabilitation plans, the most suitable rehabilitation plan will be selected based on type, degree and extent of a damage detected as well as the cause of the damage, and cost comparison of several alternatives including type of material to be applied.

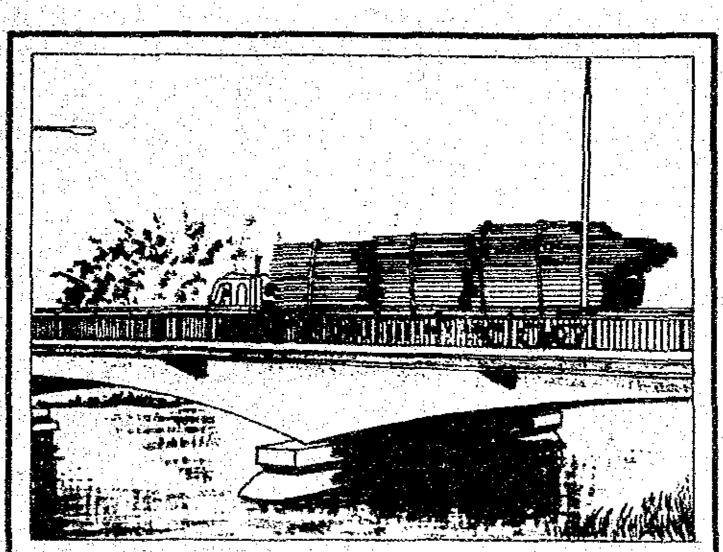
Conceptual flow of selecting a rehabilitation plan is illustrated in Figure 10-11.

Figure 10-11 Conceptual Flow of Application of Test Results in Selection of Rehabilitation Plan



CHAPTER 11

DETERMINATION OF APPLICABLE LIVE LOAD



CHAPTER 11

DETERMINATION OF APPLICABLE LIVE LOAD

11.1 General

The Study bridges were known to have been built or designed using the prevalent British Standard of the time the bridge was built. Therefore the bridges in the study were classified into several age groups which could indicate different service load even without considering any material deterioration. On the other hand, in 1990 JKR introduced and enforced a new bridge loading standard based on the recommendation made by NALS.

Assessment and determination of live loads to be applied in the Study were carried out through thorough examination of historical transition of live loads in Malaysia, linking the transition of British Standard to the present JKR live load standard. For this purpose, a comparative study in terms of sectional forces due to the several design live loads applied and the present JKR live load was carried out to reveal the extent of differences in sectional forces at various span lengths.

11.2 Historical Background of Bridge Loading in U.K.

Due to the historical linkage of Malaysia and United Kingdom, most of the bridges in Malaysia have been designed to British Standard. It is, therefore, worhtywhile to understand the brief history of the development of Standard Loading of Highway Bridges in UK which has a direct link to the design loading applied to most of the bridges covered in the Study. Most bridges built in UK in early 19th century were designed using various loadings to meet the requirements of various local authorities. After the First World War, there was a necessity to build highways to carry reasonably heavy loads such as those which were required by the British Armed Forces for the passage of its mechanical transport equipment. As a result, the Ministry of Transport (MOT) was formed immediately after the War and subsequently produced its first Bridge Loading Standard Train in 1922. In 1923 the British Standard Institution introduced the first Standard Specification for Steel Girder Bridges, BS 153 with similar live load specified as in 1922 MOT loading.

In 1932, MOT revised the Standard Loading with introduction of equivalent loading curve i.e. an Equivalent Uniformly Distributed Load (EUDL) in conjunction with Knife Edge Load (KEL) which is designated as HA load and it was derived from the previously applied standard loading train.

In 1954 BS 153 was again revised following Mr.Henderson's work in which intensity of the EUDL at 2-6 meter span length was increased considering the effect of the actual traffic and HB loading which is a hypothetical 4 axle vehicle with each axle weighting 25 to 45 ton was newly introduced to cater the passage of abnormal vehicles. This new loading code of practice allows 25% overstress on dead and live load when checking the effect of HB loading.

In 1978 new bridge loading and a design standard known as BS 5400 (1978) was introduced to incorporate the application of ultimate limit state design philosophy. The new code also introduces variable spacing between bogies in HB loading and changes in HA loading intensity i.e. reduced intensity for span length less than 30m and increased intensity for spans of more than 30 m. Another significant change was no allowance for the 25 % overstress.

In 1984 the Department of Transport introduced BD 21/84 which is to be used specifically for the assessment of highway bridges and structures in UK. The significant difference between the loading applied in this standard and those introduced earlier are higher intensity of HA loading for especially short span bridges and deletion of abnormal loading code in BD 21/84.

11.3 Historical Transition of Design Live Load Application in Malaysia

Most of the bridges in Malaysia have been designed to the British Standard, in general. However, there is a time lag between the establishment of the UK Standard and its application in Malaysia and slight modification of the Standard for its application in order to meet Malaysian local conditions. Thus the purpose of this section is to examine the historical transition of design live loads application in Malaysia which will give the Study Team a general idea concerning the assessment of load carrying capacity of the study bridges.

Even though there is no record available with regard to bridge loading application for the bridges built before 1942 in Malaysia, it could be assumed that most of the bridges in that era was designed in the United Kingdom particularly steel bridges since steel manufacturer markings were embossed on the steel beams.

It is widely known that the bridges after 1942 were designed in accordance with HA loading in the BS 153 Part 3A with reference to the relevant MOT Memoranda, while HB loading was not introduced in Malaysia until 1972. After 1975 it was certain that the bridges were universally designed to HB loading with the condition it crosses the bridge at a very slow speed and guided along the centerline of the bridge deck.

Although BS 5400 was first published in the UK in 1978, JKR formulated its application in early 1980's by requiring only the consultant executing JKR projects, while JKR Engineers still continue to design bridges by using BS 153. In consequence of these non-uniform live load application, there are 2 types of bridges in the national network i.e. bridges designed in house by JKR were designed with HA load and check with HB guided following BS 153 Standard while bridges designed by consultants were designed with HB unguided loading following BS 5400 Standard. Therefore there is a need for JKR to formulate a standard national bridge loading specification.

11.4 Present Live Load Standard in Malaysia

Taking into consideration of the above mentioned circumstance and in response to the recommendation in the NALS Phase-I, JKR formed a committee called "Bridge Loading Committee" in the Bridge Unit to review the NALS recommendations with respect to a new loading curve to be applied for future bridge design and assessment in Malaysia and to formulate a new national bridge loading standard. Finally the Committee introduced the JKR bridge loading standard in 1990 as outlined hereunder;

(1) Classification of Roads and Loading Requirement

For the purpose of bridge design, roads have been divided into 3 classes, namely Class I,II, and III, depending on the significance, traffic volume, etc. The following table shows the definition of road classification and corresponding loading requirements.

Class of Road	Classification of Road	Loading Requirement
I	Federal Roads	LTAL or 20 units SV Controlled or 7 units SV Uncontrolled
II	State Roads	MTAL
III	Other Roads	Appropriate Loading Subject to Approval

(2) LTAL & SV LOAD

The Long Term Axle Load (LTAL) consisting of a Uniformly Distributed Load (UDL) as indicated in Figure 11-1 in association with a Knife Edge Load (KEL)(100 KN per notional lane width) is intended to simulate the actual normal vehicle configuration and loading in the future. It was derived based on the loading model used in the derivation of BD 21/84 loading curve with slight modification. SV (Special Vehicle) load is an abnormal vehicle unit loading and consists of a tractor and a multi-axle trailer as depicted in Figure 11-2 with maximum total weight of 430 ton in case of 20 (guided) units (one unit of SV is taken as 1 ton per trailer axle spacing at 1.5 m).

Figure 11-1 Loading Curve for LTAL UDL

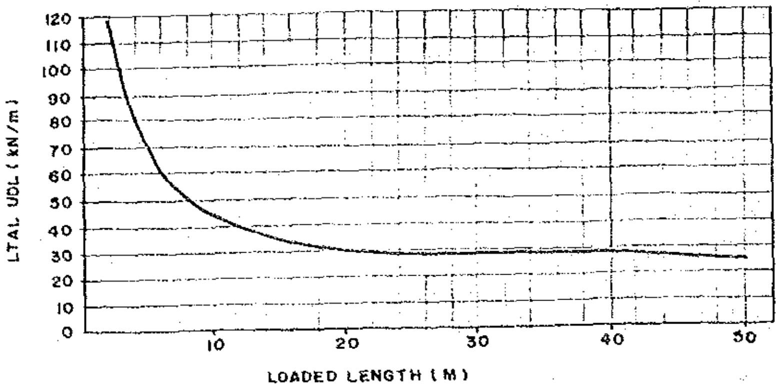
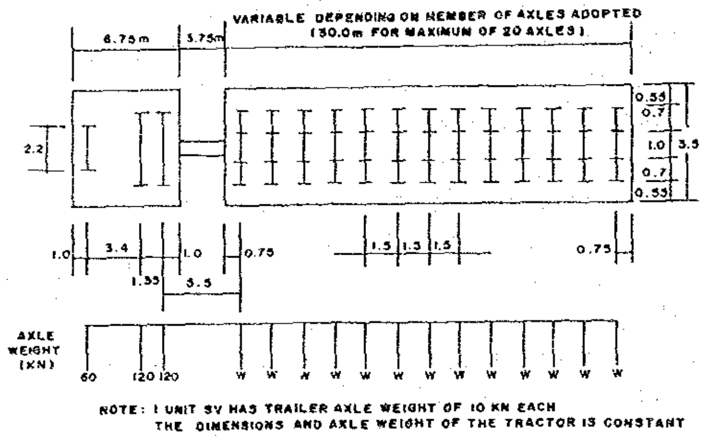


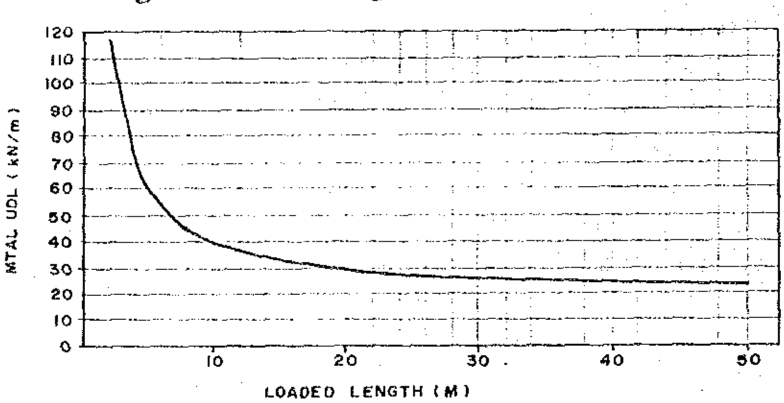
Figure 11-2 Configuration of Special Vehicle



(3) MTAL LOAD

The Medium Term Axle Load (MTAL) consists of a Uniformly Distributed Load(UDL) as shown in Figure 11-3 and a Knife Edge Load (KEL), 100 KN per notional lane, combined or a single wheel load. The MTAL-UDL loading curve was derived from the combined effects in terms of bending moment and shear from BS 153 HA load and 45 units HB loading guided. Therefore the MTAL load is almost the same as the loading previously applied by JKR as described above.

Figure 11-3 Loading Curve for MTAL UDL



11.5 Determination of Live Load to be Applied

As a summary of the above, the bridge design live loads applied or to be applied in Malaysia are broadly divided into HA, MTAL (which is almost equivalent to HB guided), LTAL and SV loading. On the other hand, all the 216 study bridges are classified into 3 age groups which are 34 nos (15.7%) built before 1945 using most probably Standard Train Loading, 180 nos (83.3%) built between 1946 to 1974 using HA loading and 2 nos (0.9%) built after 1975 using HB guided loading.

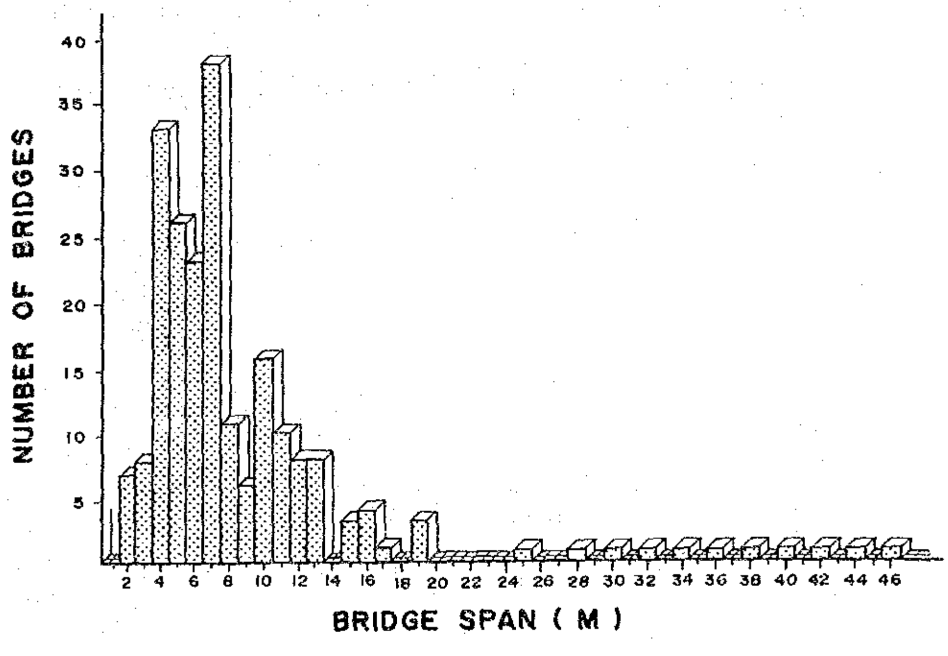
As such, comparison of sectional forces due to past design loads and present design loads at various span lengths will form a basis for determination of the applicable live load in the Study.

11.5.1 Typical Bridge Configuration

Considering two different type loadings consisting of uniform loading and wheel loading and varied application of lane width, typical carriageway width and span distribution of the study bridges were firstly assessed to obtain a representative or a typical bridge configuration in the Study for comparison of sectional forces under the same condition.

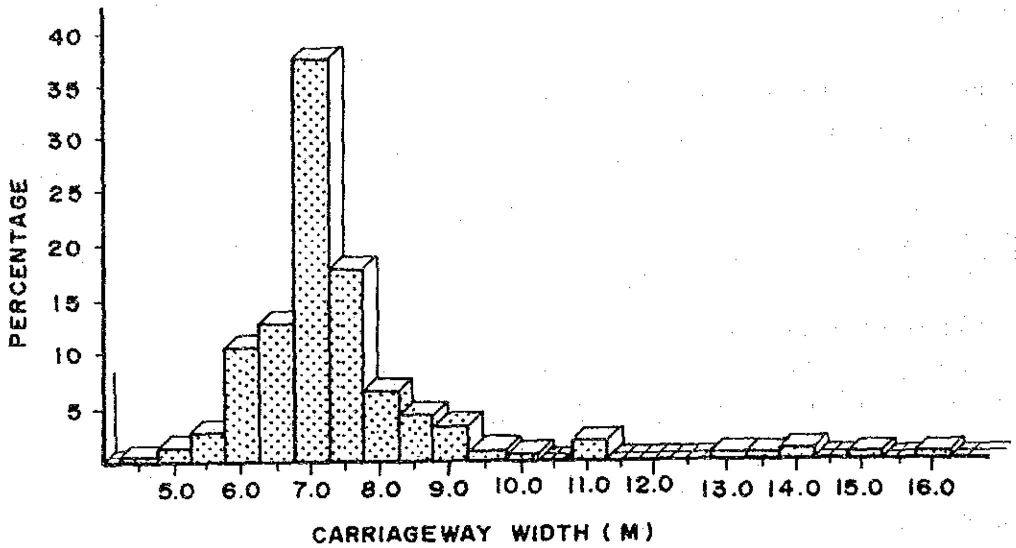
Span distribution of all the study bridges as shown in Figure 11-4 revealed that the average span length of the bridges is 8.07 m while 86% of the bridges are with span length less than 12 m and the maximum and minimum span length is 45.78 m and 1.80 m respectively. Thus the comparison of bending moment and shear force was carried out to cover a bridge span length up to 50 m.

Figure 11-4 Distribution Diagram of Span Length



Distribution diagram of the carriageway width of the study bridges is also depicted in Figure 11-5. It revealed that the average carriageway width in all the study bridges is 7.05 m. Therefore the comparison of bending moment and shear force was conducted based on a bridge with carriageway width of 7.05 m without considering lateral load distribution.

Figure 11-5 Distribution Diagram of Carriageway Width



11.5.2 Comparison of Sectional Forces due to Live Load

Bending moment at midspan and shear force at support for bridges with span lengths varying from 2 m to 50 m were calculated using different live loads i.e. HA, MTAL (Equivalent to HB guided), LTAL and 20 Unit SV loading. Figure 11-6 and 11-7 show percentage difference in bending moment and shear force compared to HA loading respectively.

Figure 11-6 Percentage Difference in B.M. Compared to HA

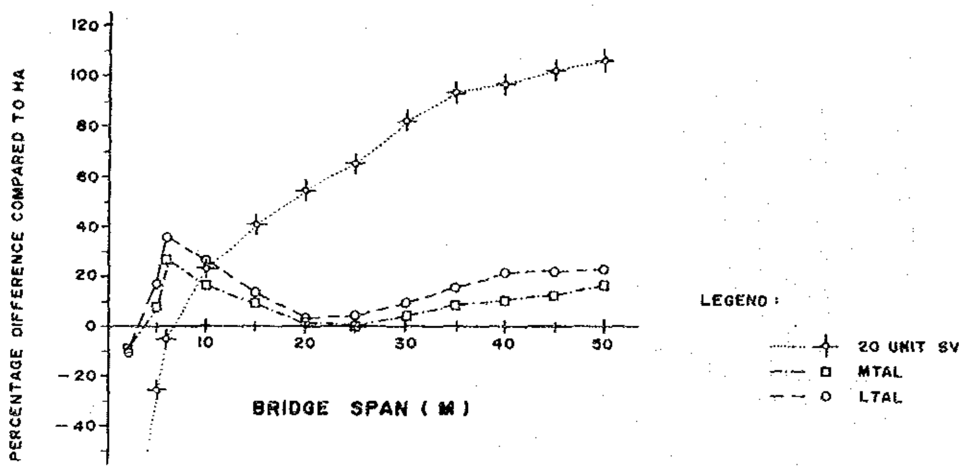
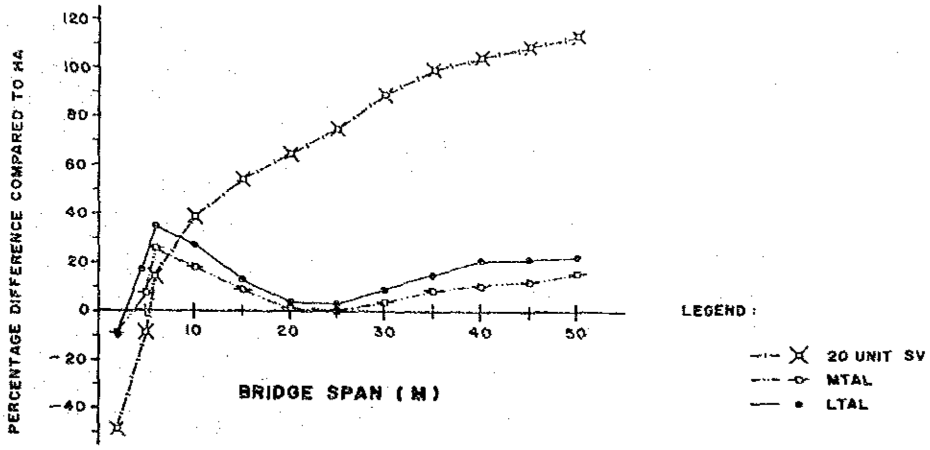


Figure 11-7 Percentage Difference in Shear Compared to HA



The bending moment due to SV Load at span length longer than 40.0m is almost double that due to HA Load. Similarly with shear force effect where shear force is caused by SV load is about double that due to HA for a bridge span longer than 35 m. Thus a comparative study on bending moment and shear force due to 20 Unit SV Loading and HA Loading indicates the Study bridges with span longer than approximately 7m will not be able to carry the SV load.

The bending moment and shear force due to MTAL and LTAL are slightly higher than that due to HA for all span ranges on average. The percentage difference in bending moment and shear due to MTAL compared to HA varies from 0.1% to 26.1% which is relatively small, while the percentage difference due to LTAL compared to HA also varies from 3.3% to 35.3%.

Based on the above study, it is obvious that application of 20 SV load has to be discarded due to unviability of this application for the purpose of the Study as most of the bridges will have to be replaced or strengthened if 20 Unit SV load is applied in the study. On the other hand, it is not conclusive to apply LTAL or MTAL in the Study since the percentage difference on span ranges between 5.5m to 13m & 39 m to 50m is more than 20% which is relatively high and quite considerable. Thus further assessment was carried out before the final selection of applicable live load is made.

11.5.3 Comparison of Sectional Forces due to Total Load

Definitive selection of applicable live loads for the Study could not be derived only from simple comparison of the sectional forces due to live loads as described above. Furthermore it is a fact that working stress in steel bridges is considerably affected by live load while conversely in concrete bridges.

In general the magnitude of the contribution in live loads depends on the weighted ratio of dead load against total load, thus comparison of sectional forces due to total loads for every typical type of study bridge has to be carried out. In this way, a clear difference in sectional forces due to LTAL or MTAL compared to HA could be obtained hence the determination of applicable live loads. The typical bridges considered are SBB of span range from 5m to 15m, SBC from 5m to 15m, RCB from 5m to 15m, RCS from 5m to 15m, IT from 10m to 20m and PCB from 25m to 50m having a fixed carriageway width of 7.05m.

Percentage difference in bending moment and shear force due to total loads consisting of dead load and LTAL or MTAL compared to HA for various bridge types is depicted in Figure 11-8 and 11-9 respectively.

Figure 11-8 % Difference in Sectional Forces due to Dead Load and MTAL Compared to HA

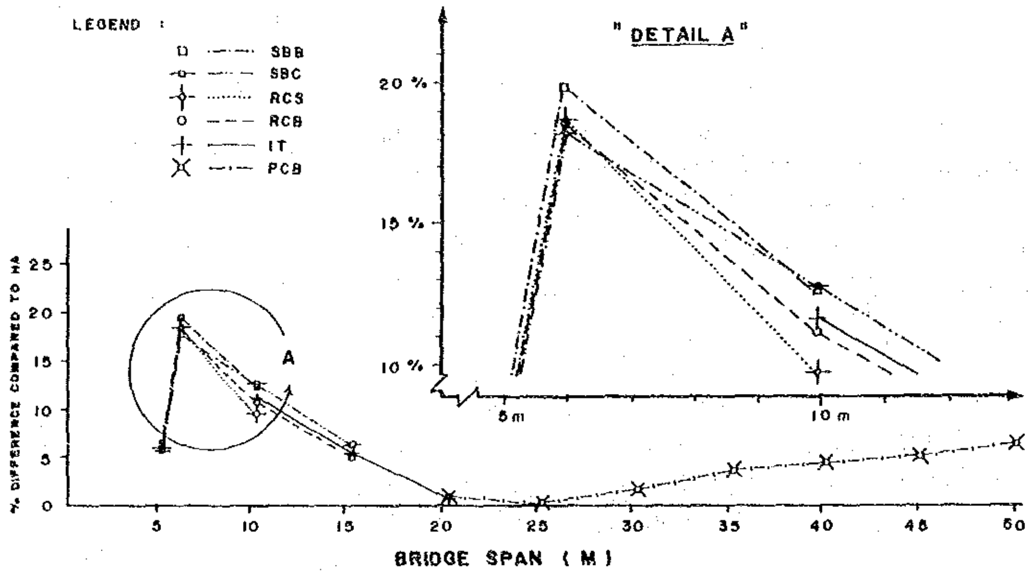
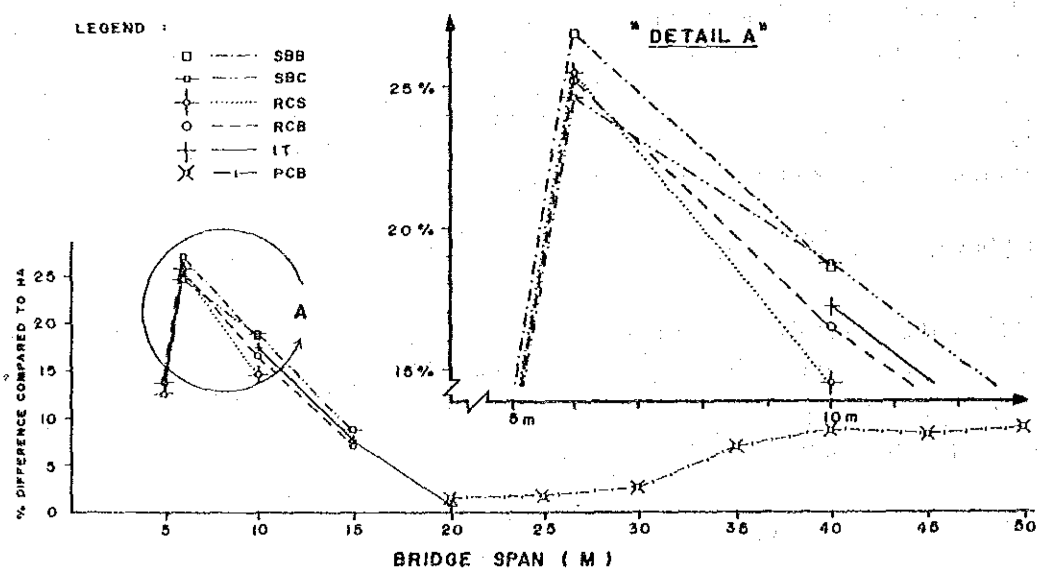


Figure 11-9 % Difference In Sectional Forces due to Dead Load and LTAL Compared to HA



As shown in Figure 11-8, each of the percentage differences in case of MTAL is within 20% and the maximum percentage difference is 19.89% in SBB with span length of 6m.

While in case of LTAL as shown in Figure 11-9, the maximum percentage difference is 26.92% in SBB with span length of 6m and the bridges of which the percentage difference exceeding 20% are SBB, SBC,RCB and RCS having span length between about 6m and 9m. These bridges amount to about 28% of all the study bridges.

In this regard, the percentage difference within 20% could be covered by the reserved residual loading capacity of each member as proven in the full scale loading test. While some of the bridges of which the percentage difference exceeds 20% up to a maximum of 26.3% could have LTAL load carrying capacity if effects of the lateral load distribution and the reserved residual loading capacity are considered. Some of those could be rehabilitated by applying standard strengthening methods.

11.5.4 Conclusion

Based on the above assessment results, it is concluded that LTAL shall be applied as the live load standard in this Study with due consideration of the following aspects:

- LTAL is the present JKR live load standard applicable to the federal bridges.
- The average percentage difference between LTAL and MTAL is only 7%.
- The percentage difference within 20% could be covered by the reserved residual loading capacity as proven in the loading test.
- The bridges of which the percentage difference in sectional force due to LTAL compared to HA exceeding 20% are only 28% of all the study bridges. Some of these could have LTAL loading capacity if effects of the lateral load distribution and the reserved residual loading capacity is considered or could be rehabilitated by applying standard strengthening methods.

CHAPTER 12

PRELIMINARY REHABILITATION DESIGN

CHAPTER 12

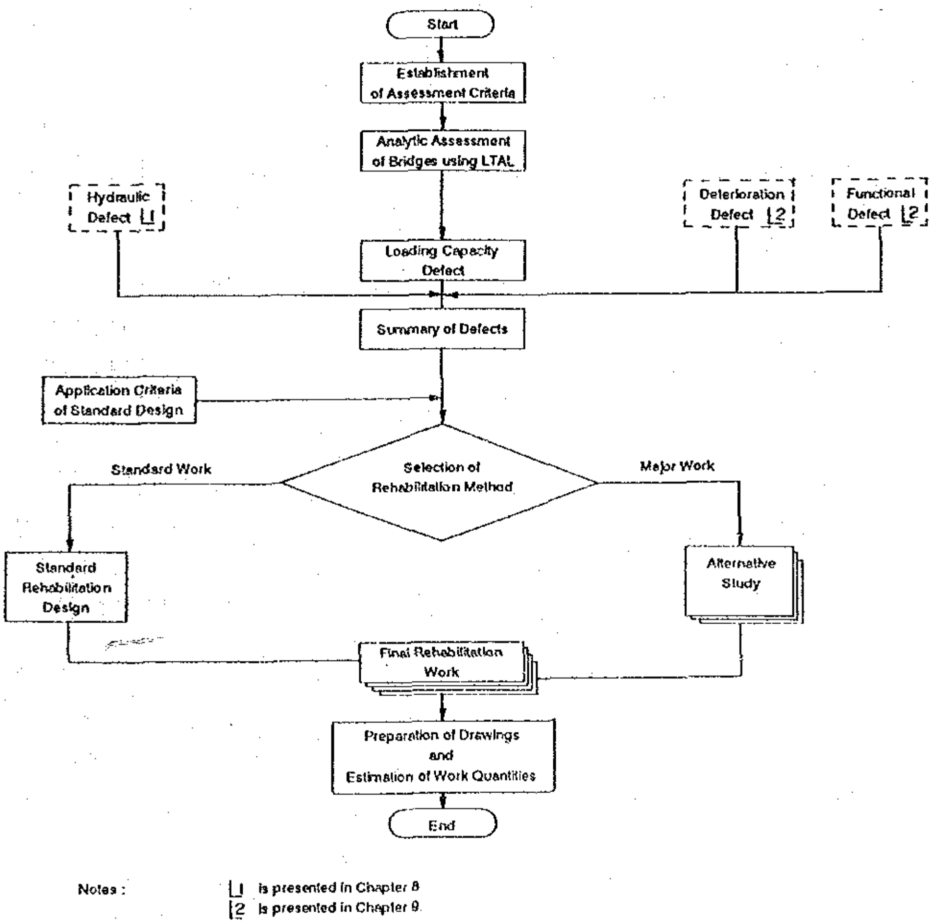
PRELIMINARY REHABILITATION DESIGN

12.1 General

The purposes of preliminary rehabilitation design covering the 20 bridges including 3 special bridges are to assess the bridges using LTAL loading, to prepare standard rehabilitation design, to carry out an alternative study for possible major work and to estimate the work quantities of each bridge. The preliminary design was carried out based on the output from the preceding detailed structural survey, bridge loading test as well as determination of applicable live load.

The design flow is depicted in Figure 12-1 which also shows the interrelationship of above work items.

Figure 12-1 Flow Chart of Preliminary Design



12.2 Establishment of Assessment Criteria

The structural assessment criteria to be applied in the Study is in principle based on JKR Design Practice except where the specification is not clear then the Bridge Design Specification in Japan is applied. The assessment criteria covers the following aspects of design:

- Geometric design standard
- Bridge width
- Bridge loading
- Material and allowable stress
- Assessment method applied
- Superstructure design
- Substructure design
- Applicable design standard

This section presents an abstract of the above items and the details are enclosed in Appendix-N.

(1) Geometric Design Standard

The geometric design standard applied is in accordance with the JKR "ARAHAN TEKNIK (JALAN) 8/86".

(2) Bridge Width

Bridge width of R5 applied is in accordance with Article 5.11 of JKR "ARAHAN TEKNIK (JALAN) 8/86".

Note: This clause is only applicable to a bridge which has functional rehabilitation works such as widening carriageway, adding sidewalk or total replacement.

(3) Free Board

The free board requirement is not clearly stated in JKR Bridge Design Manual, thus the recommendation given in River Design Standard in Japan is adopted after some modifications were made considering Malaysian river conditions. For this study the bridge soffit shall be designed in accordance with the free board requirement as tabulated below.

Free Board for Bridge Over River

Free Board	Applicable River
0.5m	Small river, design flood is less than $500m^3/sec$
1.0m	Medium river, design flood is between 500 and $2000m^3/sec$
1.5m	Large river, design flood is more than $2000m^3/sec$

(4) Bridge Loading

For preliminary design the load to be considered shall be as follows:
. Dead loads
. Live loads (LTAL loading)
. Pedestrian load (sidewalk loading)
. Tractive/braking force
. Force due to earth pressure

Above design criteria are basically in accordance with the JKR Bridge Design Manual and Standard Specification for Bridge Loading.

(5) Assessment Method Applied

The assessment of the existing bridges and the design of rehabilitation work jointed directly to the existing bridges shall be carried out in accordance with elastic design method (allowable stress design method), while for adding a sidewalk which is not attached to the existing bridge or a new bridge for total replacement, the design shall be carried out using limit state design method.

The reasons for adopting these two different design methods in the preliminary design are:

- All the study bridges were designed to BS153 which follows the elastic design principal.

- Quality of materials used in the study bridges is scattered (i.e. strength variation is very wide)

Thus, it is safe to apply elastic design method for the assessment and rehabilitation work. Contrarily, quality of material and accuracy of design for an independent structure can be controlled properly within a very low tolerance. Accordingly, it is rational to apply limit state design methods only for an independent structure which will not be attached to the existing bridge.

The elastic design method shall be based on the guidelines given in the JKR Bridge Design Manual, while for limit state design the provisions prescribed in BS 5400 shall apply.

(6) Material and Allowable Stress

- Allowable stress design

Allowable stress for reinforced concrete design shall be as specified in BE 1/73 and for steel shall be as specified in BS 153 (i) Part 3B.

- Limit state design

The design strength of materials for limit state shall be as specified in BS 5400. Nominal yield stress for steel shall comply with BS 4360.

(7) Applicable Design Standard

In deriving the assessment criteria, the JKR Bridge Design Manual was referred. In addition, reference was also made to Specification for Highway Bridges published by Japan Road Association and BS 153, BE 1/73, BS 5400 Part 1, 2,3 and 4.

12.3 Assessment of the Bridges

12.3.1 General

In selecting a suitable rehabilitation plan, it is essential to diagnose all defects in terms of material deterioration, load carrying capacity, bridge function and hydraulic adequacy together with the corresponding cause of the defect so that the rehabilitation plan is selected which will effectively rectify the cause of the defects.

To this end, all of the 20 bridges for the detailed structural survey have been assessed from hydraulic viewpoints in Chapter 8 and from material deterioration and bridge functional viewpoints in Chapter 9. Therefore, in this section, the remaining item, the analytic assessment of 20 bridges from load carrying capacity viewpoint is mainly carried out to identify which main bridge members is inadequate or adequate to carry LTAL loading based on the assessment criteria established in the previous section.

Subsequently, all the defects together with respective causes from the above four viewpoints are summarized in the final part of this section, which will assist in selecting the most suitable rehabilitation plan.

12.3.2 Analytic Assessment of 20 Bridges

The main purpose of this analytic assessment is to diagnose the LTAL carrying capacity of each bridge's main structural member based on the survey results such as the detailed structural survey and the loading test.

(1) Method of Assessment

The assessments were carried out based on the available structural data taken from the detailed structural survey results which consist of structural details such as dimensions details i.e. size of various members, size and numbers of rebars and the spacing, and material properties comprising of strength and elastic modulus.

It has been proven by the load test results that the bridge members have inherent residual loading capacity resulting from bridge behavior difference between in design and in actual. This capacity mitigates the theoretically calculated working stress to a lower value. Thus this mitigating factor was applied in the assessment where the comparison of the allowable stress with the working stress was made by deducting the latter by a certain percentage depending on the type of structure outlined as follows;

- **Steel Beam and R.C. Bridge**

In the assessment of steel beams, the maximum working stress theoretically calculated due to the most severe LTAL loading including dead load is compared with the allowable stress of the structural steel. If the working stress after reduction of 20 % which is applied as inherent residual loading capacity value of steel beam is within the allowable stress, then the steel beams are considered adequate.

The same method described above was also applied in the assessment of R.C. beams and R.C. slabs. However, the mitigating factor of working stress, 20 % for R.C. beam and 10 % for main slab were applied.

- **Prestressed Concrete Structures**

On all the P.C. beams except Endau bridge (00317000), effective prestressing forces are unknown. Due to this lack of data, the assessment method applied is by comparison of the difference between sectional forces (bending moment and shear force) due to the assessment total load (LTAL + Dead Load) and those due to the design total load (HA + Dead Load). Then if the difference is not more than 20 % which is reserved residual loading capacity of P.C. beam, P.C. beam is considered adequate.

- **Substructure**

Subsoil profile, pile length data and size of footings are requisite data in the assessment of substructure. However, due to the lack of these data, the assessment method applied is also by comparison of the difference between reaction forces due to the assessment total load and those due to the design total load. If the difference is not more than 20 - 30 % (1), then the substructure is considered adequate.

(2) Theory Applied

The followings are the main structural theories adopted in the assessment.

- o Lateral load distributions for girder type bridges are based on Y.Guyon and C. Massonet othortropic plate theory.
- o For reinforced concrete slab bridges the finite isoltropic simply supported plate theory is applied.
- o For deck slab analysis the infinite isotropic simply supported plate theory is applied.

(3) Results of Analytic Assessment

Summary of the analytic assessment for 20 bridges is tabulated in Table 12-1 and the result for each bridge is attached in Appendix-O.

Note :

(1) Normally the substructure is designed with safety factor of about 3. Therefore 20 - 30 % increase in load is quite insignificant increase to the total load carrying capacity of the substructure.

Table 12-1 Summary of Analytic Assessment on Existing Bridges

Bridge No.	Bridge Type	Ratio (%) <1			Assessment Results
		Slab	Beam	Substructure	
114920	RCB	−77.5	−2.8	+23.6	
161140	SBB	+34.2	+66.2	+12.6	Steel buckle plates and beams have inadequate capacity.
166510	SBG	−38.8	−43.7	+7.5	
	RCB	−	+86.7	−	R.C Beams have inadequate capacity.
237200	SBC	−26.7	+2.1	+20.4	
	RCB	−27.5	−27.1	−	
317000	PCB	−28.0	−20.0	+5.0	
319110	PCB	−6.6	+8.4 (−3.4) <4	+10.3	Main beams of 2−girder type bridge have inadequate capacity
341800	RCB	−80.4	−6.8	+13.5	
346740	PCB	−	<2	−	Main beams of 2−girder type bridge have inadequate capacity
520850	SBE	−55.3	−7.7	+0.8	
546560	RCB	−4.3	+19.7	+14.3	Main beams have inadequate capacity.
546980	RCS	+138.7	−	+9.4	R.C Slab has inadequate capacity.
563880	IT	−	−12.4	+0.5	
567840	PRB	−	−14.9	+24.4	
634850	RCS	−2.1	−	+7.0	
5001070	SBB	<3	+105.8	+7.9	Steel buckle plate and beam have inadequate capacity.
5803340	SBB	<3	+20.6	+18.8	Steel buckle plate and beam have inadequate capacity.
5903120	SBC	+73.9	+0.7	+16.0	Slab has inadequate capacity.
Dambai	SBC	+6.3	+34.5	+12.1	Slab and beams have inadequate capacity.
Samarahan	SBC	+99.9	+24.4	+3.6	Slab and beams have inadequate capacity.
	RCB	−	−12.8	−	
371000	RCB	−7.3	−7.6	−0.9	

Notes : <1 Percentage increase (+) or decrease (−) against allowable stress or design force.
<2 The assessment results of Bridge No. 319110 is utilized to this bridge.
<3 The assessment results of Bridge No. 161140 is utilized to this bridge.
<4 Figure in ( ) means the result of 4−girder type bridge.

12.3.3 Assessment of Pile Failure in Bridge No. 00512960

In the course of the study, JKR requested the study team to investigate the cause of the pile failure on Bridge No. 00512960 and prepare possible remedial work in addition to the 20 bridges for the detailed survey. Accordingly, the assessment was carried out in this subsection.

(1) Extent of Damage

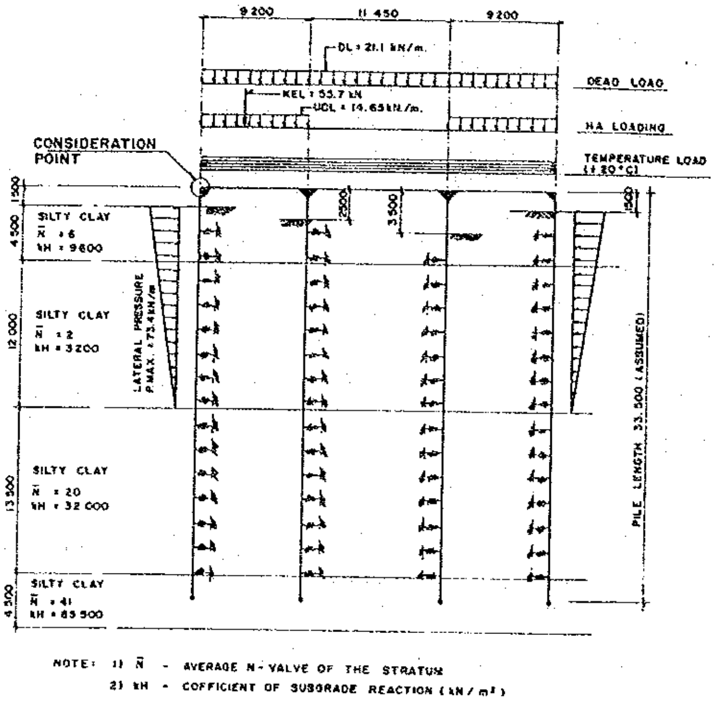
This bridge with 30.4m of bridge length is a rigid frame type with a three span continuous R.C. beam (with concrete deck slab) rigidly connected at the support with R.C. piles. Among seven (7) piles at the abutment, there are two raker piles battered outwards by as much as 10° from the vertical. It is assumed based on the boring log obtained from JKR that the pile is driven through silty clay which has a high possibility of consolidation settlement, and the penetration length is in the range of 30 to 35 meters terminating in silty clay with N-value of about 40.

All the pile caps at the abutments have cracks with width ranging from three (3) to eighteen (18) millimeters and spalling concrete as its surface. It was found that the two raker piles' pilecaps have the most severe damage, while the edge vertical piles were relatively unaffected.

(2) Method of Assessment

The analytic model for the assessment is shown in Figure 12-2.

Figure 12-2 Analytic Model for Assessment



The loads considered are dead load, live load, load due to thermal effect and lateral soil pressure caused by soil consolidation settlement of the soft silty clay. The load intensity of the lateral soil pressure mentioned in the above paragraph were calculated in accordance with the Highway Bridge Specification published by the Japan Road Association.

(3) Assessment Results

The assessment was made based on the elastic design method and the assessment results are tabulated in Table 12-2.

Table 12-2 Assessment Results of Pile Failure

	Sectional Force			Working Stress (N/mm2)		Allowable Stress (N/mm2)	
	Bending Moment (KNm)	Shear Force (kN)	Axial Force (kN)	Concrete	Rebar	Concrete	Rebar
Dead Load	22.8	10.4	81.5	2.0	19.0	10.0	140.0
Live load (HA)	15.1	6.7	93.9	1.4	6.5		
Temperature Load	43.7	29.3	7.9	3.2	63.5		
Lateral Pressure	111.8	69.5	18.1	8.1	162.9		
Total	193.4	115.9	201.4	14.8	248.9		

As indicated in Table 12-2, the working stress induced by dead and live load as well as by thermal effect is within the allowable stresses, but the total working stress caused by all of the above loads exceeds the yield stress.

It is therefore concluded that the main cause of the piles' failure is due to lateral soil pressure effect caused by soft soil consolidation.

The reason of the raker piles pilecaps suffered the most severe damage is because of rake where both the vertical settlement and lateral force induced by the consolidating soils has caused the maximum flexural stress.

(4) Remedial Work

As the soft soil strata will likely continue to consolidate and settle, it is important that the abutments be strengthened. It is, however, impractical to strengthen the abutment taking into account the extent of damage and absent data such as built drawings and boring logs. Thus it is recommended that new rigid framed abutments be constructed close to the existing abutments supported by piles of which working load is reduced to take into account the inevitable negative skin friction.

12.3.4 Assessment of Bridges under Special Heavy Vehicles (SHV)

In the Steering Committee Meeting held on 13 September 1990, the Study Team was requested to evaluate the impact of introducing heavy vehicles carrying oversized containers on 2 or 3 of the 20 selected bridges from the loading viewpoint and to look into the differential costs involved in upgrading these bridges. The assessment was carried out in this sub-section.

(1) Configuration of Special Heavy Vehicles

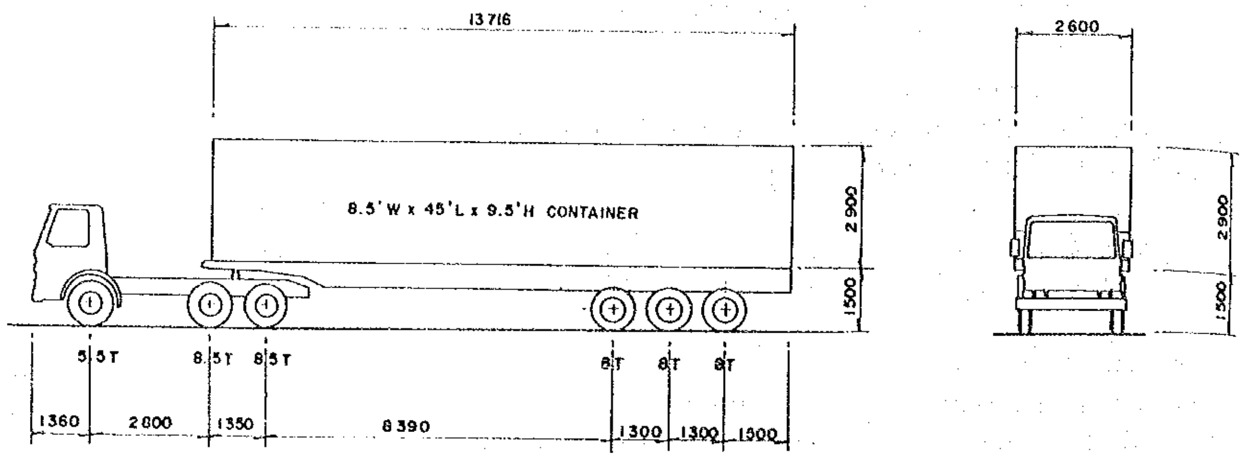
The special heavy vehicle in this report is actually an over-size container which is bigger than the International Organisation of Standardisation (ISO) standard container dimensions of 8' 6" (2.6m) high x 8' (2.4m) wide x 40' (12.2m) long.

The biggest oversized container's dimensions are 9' 6" (2.9m) high x 8' 6" (2.6m) wide x 53' (16.154m) long as reported in the Economic and Social Commission for Asia and the Pacific (ESCAP) study report carried out by the Technical and Managerial Port Assistance Office based in the Netherlands.

In the assessment of the loading effect, 45' (13.716m) long oversized container with maximum gross weight of 33 tons was applied because it will cause more severe effect in terms of loading and it is dominant in the ASEAN countries.

To carry this relatively heavy container, it will have to be mounted on a 3-axle trailer and pulled by a 3-axle tractor. The configuration of the special heavy vehicle is as shown on Figure 12-3.

Figure 12-3 6-Axle Articulated Special Heavy Vehicle

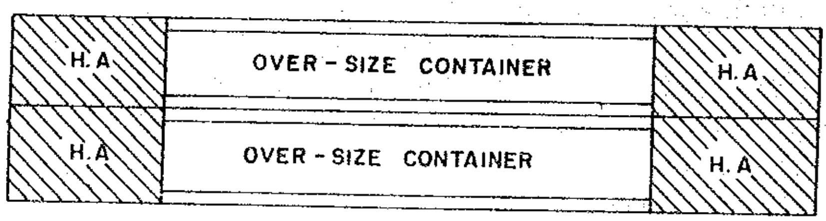


(2) Loading Criteria

Based on the research and traffic studies with respect to effect of the Special Heavy Vehicle carried out in Japan, the incidence of two special heavy vehicles passing each other on a bridge occurs frequently, while the incidence of these vehicles traveling in a convoy is a very rare exception.

Therefore, the loading application criteria adopted in the assessment is as shown in Figure 12-4 whereby one unit of the Special Heavy Vehicle in the longitudinal direction and up to two units in the transverse direction shall be applied and the rest of the loaded length with HA UDL (BS153) Loading. This criteria is consistent with the code of practice in Japan.

Figure 12-4 Loading Application Criteria



(3) Selection of Bridges for Loading Assessment

The following bridges are selected for the assessment out of the 20 bridges in order to cover all the representative characteristics in term of span length and bridge type.

Bridge No	District / State	Type	Span
00317000	Rompin, Pahang	PCB	30.48m
00341800	Kemaman, Terengganu	RCB	12.10m
05803340	Batang Padang, Perak	SBB	5.35m

(4) Results of Assessment

The effect due to SHV on the main girder was analyzed, while the effects on the deck slab were neglected because the localized wheel loads conditions of SHV is less severe than that of LTAL loading condition.

Results of the assessment by means of comparison with maximum bending moment due to LTAL loading (including dead load) and that due to SHV loading and dead load is tabulated in Table 12-3.

Table 12-3 Assessment Results of Special Heavy Vehicle Loading

Bridge No.	Maximum Bending Moment induced by LTAL on a Main Girder (KNm) (1)	Maximum Bending Moment including impact effect induced by Special Heavy Vehicle on a Main Girder (KNm) (2)	Ratio (2)/(1)
00317000	2005.0	1464.7	73 %
00341800	442.8	376.9	85 %
05803340	139.5	113.2	81 %

Table 12-3 indicates that the special heavy vehicle has not caused significant overstressing in the bridge main girders. It should be noted that analysis of the heavy vehicle loading on bridge No. 005803340 is based on the rehabilitated bridge, that is after the buckle plate has been removed and replaced with concrete deck in order to look into the differential cost involved in upgrading those bridges.

(5) Conclusion

It could be concluded that this Special Heavy Vehicle (SHV), carrying an oversize container under the loading application criteria established, has induced stresses less severe to the main beams and deck slab of the study bridges than those due to LTAL. Therefore, it is apparent that there will be no cost increase due to SHV in rehabilitating the study bridges to LTAL loading.

However, assessment of the impact due to introducing SHV from highway geometric viewpoints such as vertical clearance and vertical and horizontal alignments and from highway capacity viewpoint are not covered in this report.

In this regard, it is strongly recommended that a further study to assess the impact from above mentioned viewpoints and also to assess all remaining types of bridge should be carried out before introducing this type of oversized container.

12.3.5 Summary of Assessment

(1) Summary of Defects

Based on the assessment studies carried out in the previous chapters and sections, all the types of the defects were divided into four categories consisting of material deterioration, LTAL loading capacity, bridge function and hydraulic viewpoint, have been identified together with the corresponding possible cause of the defect.

A summary of all types of the defects detected in the 20 bridges is tabulated in Table 12-4.

Table 12-4 Summary of Defects

Category		Defect	Member	Bridge Type
STRUCTURAL DEFECT	MATERIAL DETERIORATION DEFECT	Water stain	Slab Soffit	IT
		Vertical hairline crack	PC Beam	PCB
		Honeycomb, rebar exposure and spalling	Beam Soffit	
		Water Stain and moss	Beam Web	
		Water stain	Slab Soffit	PRB
		Vertical cracks	Beam Web	RCB
		Spalling and Flaking	Cross Beam	
		Honeycomb and spalling	Deck Slab Soffit	
		Longitudinal cracks and rebar exposure	Beam Soffit	
		Spalling and water stain	Beam Web	
		Honeycomb, flaking and wear	Slab Soffit	RCS
		Water stain	Slab Soffit	
		Corrosion, paint deterioration and water stain	Steel Beam	SBB
		Corrosion, paint deterioration and water stain	Buckle Plate	
		Honeycomb and Spalling	Deck Slab Soffit	SBC
		Corrosion, paint deterioration and water stain	Steel Beam	
		Lateral crack	Deck Slab Soffit	
		Spalling and Rebar exposure	Deck Slab Soffit	
		Rebar exposure and spalling	Slab Soffit	SBE
		Rebar exposure and flaking	Beam	
		Corrosion, paint deterioration	Steel Pier	Common to All Bridges
		Flaking and Honeycomb	Abutment	
		Flaking of plaster	Abutment	
		Wear, Longitudinal crack and Rebar exposure and Spalling	Pier	
		Rebar exposure and spalling	Pier	
		Alligator cracks	Pile Cap	
		Flaking and cracks	Pier Cross Head	
	LOADING CAPACITY DEFECT	Inadequate capacity	Main Beam (2 Girder Type)	PCB
		Inadequate capacity	Main Beam	RCB
		Inadequate capacity	Main Slab (Lateral)	RCS
		Tilted Structure	Abutment	
		Inadequate capacity	Steel Beam	SBC
		Inadequate capacity	Deck Slab	
		Inadequate capacity	Steel Beam	SBB
		Inadequate capacity	Buckle Plate	
FUNCTIONAL DEFECT		Too narrow	Carriageway	PRB
		Too narrow and absent sidewalk	Bridge Width	RCB
		Too narrow and absent sidewalk	Bridge Width	SBB
HYDRAULIC DEFECT		Local scouring	Pier	Common to All Bridges
		Scouring	Abutment	
		Inadequate free board	Bridge Opening	
		Bank erosion	River Bank	
		Slope protection failure	River Bank	
		Submerged beam	Bridge opening	

(2) Summary of Selected Standard Rehabilitation Methods

The defects with the corresponding possible causes and the selected rehabilitation methods were summarized on a bridge by bridge basis and these are attached in Appendix-P.

A summary of the selected rehabilitation methods is shown in Table 12-5.

Table 12-5 Summary of Selected Rehabilitation Methods

<table>
<tr><th colspan="3">Classification</th><th>Steel Material</th><th>Concrete Material</th></tr>
<tr><td rowspan="7">STRUCTURAL REHABILITATION</td><td rowspan="3">SUPERSTRUCTURE</td><td>Protection</td><td>o Repainting</td><td>o Exposy Injection
o Protective Coating
o Patching
o Guniting
o Installation of water proof layer</td></tr>
<tr><td>Reinforcement</td><td>o Installation of Additional Beam
o Provision of Cross Beam
o Attachment of steel plate</td><td>o Prepacked concrete lining with additional rebar
o Concrete Lining by guniting w/rebar
o Steel plate bonding</td></tr>
<tr><td>Replacement</td><td>o Replacement by R.C slab</td><td>-- None --</td></tr>
<tr><td colspan="2">Incidental facility</td><td colspan="2">o Extension of drainage pipe
o Installation of water drop
o Replacement of Expansion Joint
o Replacement of bearing <2
o Replacement of railing <2</td></tr>
<tr><td rowspan="3">SUBSTRUCTURE</td><td>Protection</td><td>o Repainting (N/A)
o Concrete lining</td><td>o Concrete lining (Brick Abutment)
o Concrete lining
o Patching</td></tr>
<tr><td>Reinforcement</td><td>o Concrete lining (N/A)</td><td>o Underpining by additional pile <3</td></tr>
<tr><td>Replacement</td><td>-- None --</td><td>o Replacement of abutment by rigid flame</td></tr>
<tr><td colspan="3">Hydraulic Rehabilitation</td><td colspan="2">o Slope protection
o Foot protection
o River bed protection
o Spur dikes</td></tr>
<tr><td colspan="3">Functional Rehabilitation</td><td colspan="2">o Adding sidewalk
o widening carriageway
o Raising grade</td></tr>
</table>

Note <1 In principal, steel bearing to a steel beam while rubber bearing to a concrete beam
<2 This item is included in replacement of deck slab
<3 This item is included in raising grade
N/A means not applicable

12.4 Standard Rehabilitation Design

This section presents selected standard rehabilitation methods which are relatively well known techniques in bridge maintenance and rehabilitation work and all of which are applicable in the study.

Application criteria for each standard rehabilitation method was prepared taking into account the type, degree and extent of the defect and the cause of the defect as well as effectiveness of each method.

The standard rehabilitation designs are broadly divided into seven categories comprising of:

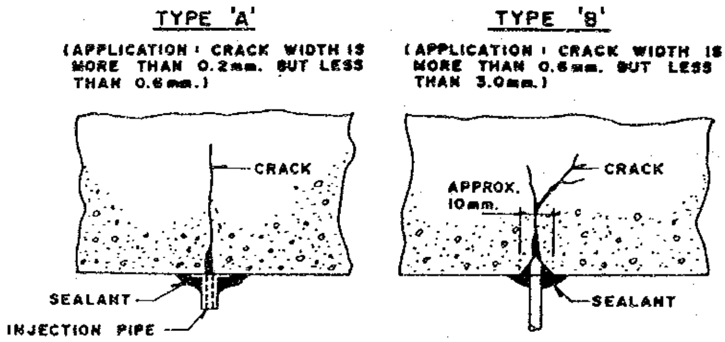
(1) Protection work to concrete
(2) Reinforcement work to concrete
(3) Protection / reinforcement to steel material
(4) Protection / reinforcement to substructure
(5) River training work
(6) Incidental facility
(7) Temporary work

The standard rehabilitation methods for each category are summarized below except category (6) and (7) of which works can be referred to STD. DWG No. 27 and 29 respectively enclosed in Volume IV Drawings.

(1) Protection work to concrete material

In general, this type of protection works is applicable to defects which are not active.

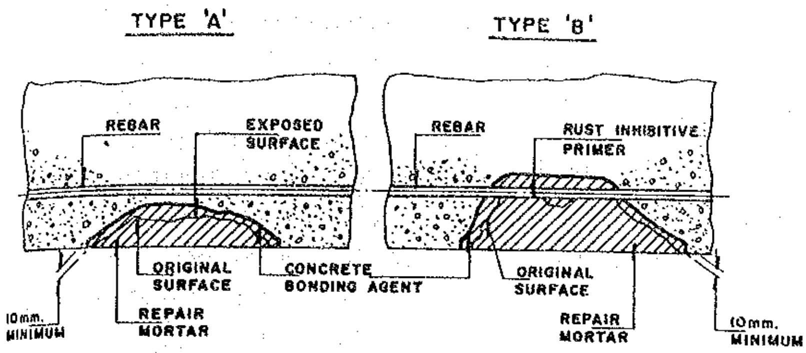
- Epoxy injection



Application criteria

- Cracks are not active and surface width is more than 0.2mm, but less than 3.0mm.
- Reason of the crack appearance is due to shrinkage or creep of concrete.
- No water leakage and no liquid rust.
- No carbonation and no chloride attack.
- If surface crack width is more than 3.0mm, apply cement paste injection.

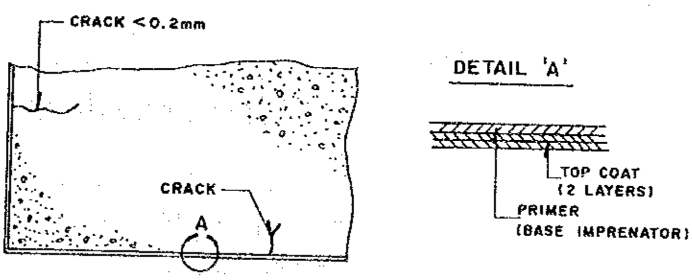
- Patching



Application criteria

- Defects such as honeycomb, flaking, and cavity, that are not active.
- Reason of these defects are mainly due to inferior concrete or poor workmanship.
- Minimal carbonation, no chloride attack and no water leakage.
- Adequate concrete cover.
- Defective area is scattered.

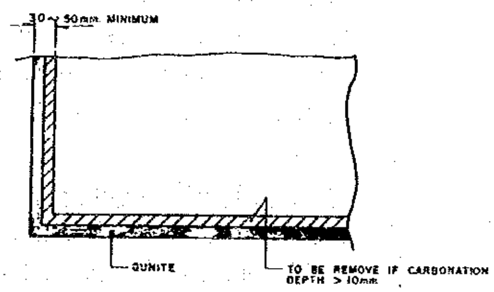
- Protective coating



Application criteria

- Cracks are not active and surface width is less than 0.2mm.
- No water leakage, no scaling and no flaking.
- Minimal carbonation and no chloride attack.
- Adequate concrete cover.

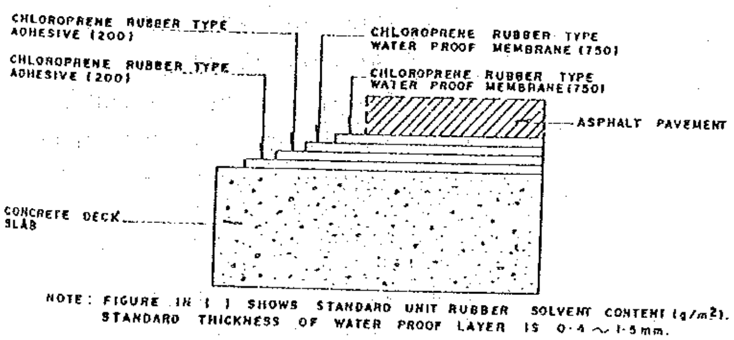
- Guniting



Application criteria

- Cracks (width is less than 0.2mm) are not active.
- Concrete is slightly carbonated.
- Minimum concrete cover is inadequate.
- No water leakage.
- Defective area is extensive.

- Waterproof layer



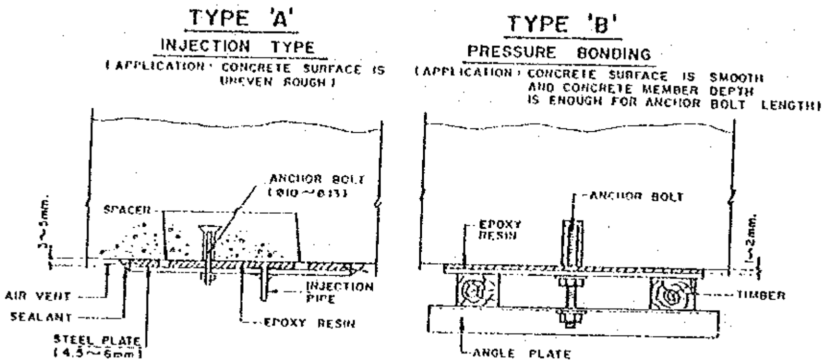
Application criteria

- Water stain, free lime and other associated defects are observed at slab soffit.
- Defects are not active.
- Water is penetrating from top of slab through defective concrete or inferior joints between precast members.

(2) Reinforcement work to concrete material

This type of reinforcement works is in principal applicable to a bridge member which has inadequate load carrying capacity or has active defects such as bending or shear crack or two way cracks etc.

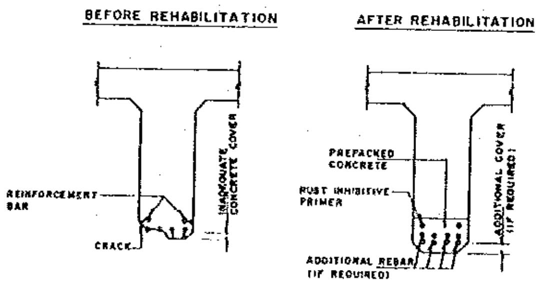
- Steel plate bonding



Application criteria

- Inadequate load carrying capacity (inadequate amount of reinforcement bar).
- No water leakage and no carbonation.
- Active cracks due to bending moment or shear force.
- Adequate concrete cover.

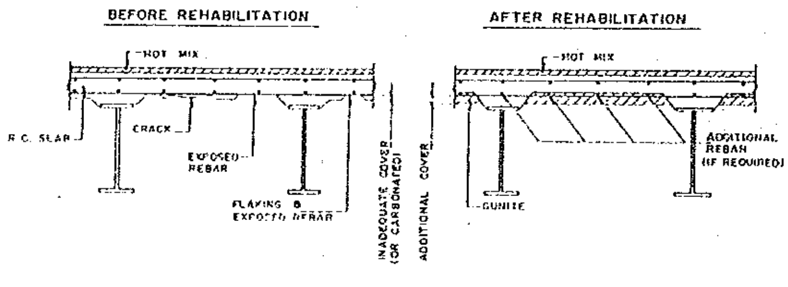
- Prepacked concrete lining with additional rebar



Application criteria

- Inadequate loading capacity.
- Various active cracks due to bending moment.
- Inadequate concrete cover.
- Suffered mild chloride attack or advanced carbonation.
- Defective area is extensive.
- Soffit of member where it is difficult to pour concrete.

- Guniting with additional rebar



Application criteria

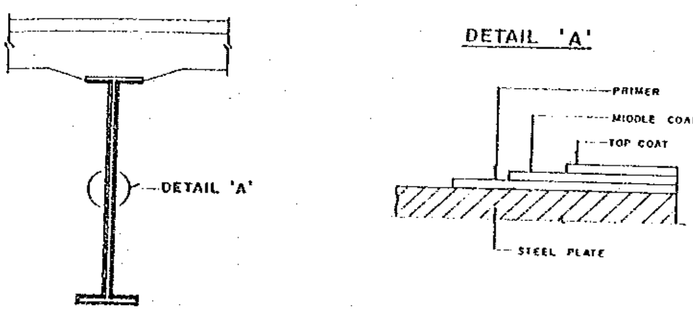
- Inadequate loading capacity.
- Various active cracks due to bending moment or shear force.
- Adequate for additional stress in beams and slab due to additional dead load.
- Bridge is located in relatively severe chloride environment.
- Advance carbonation.
- Defective area is extensive.

(3) Protection / Reinforcement to steel member

(a) Protection work

This type of protection work is applicable to steel members which have not active defects.

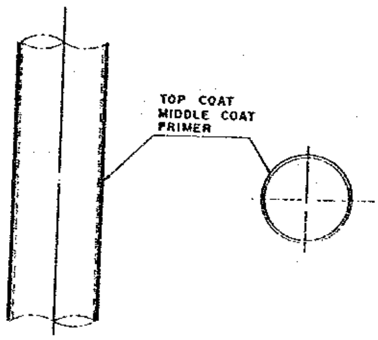
- Repainting (Superstructure)



Application criteria

- Adequate load carrying capacity.
- Non-active corrosion
- Paint deterioration.

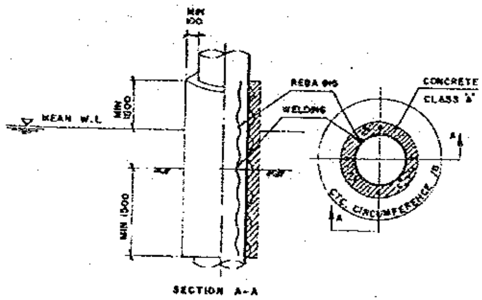
- Repainting (Substructure)



Application criteria

- Steel surface is slightly corroded but load carrying capacity is still adequate.
- Bridge is located in a non-severe environmental condition.
- No chloride attack.

- Concrete lining



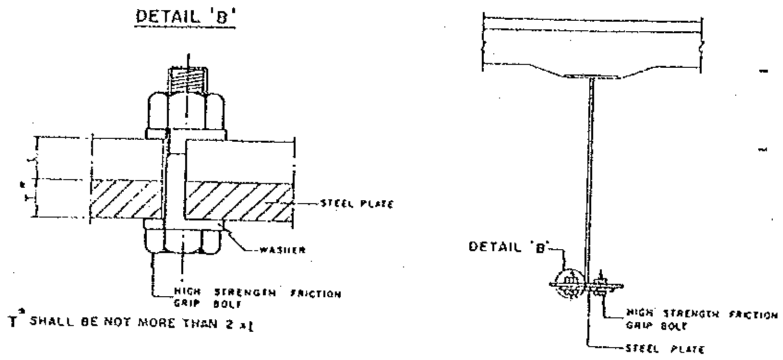
Application criteria

- Steel surface is considerably corroded but load carrying capacity is still adequate.
- Bridge is located at severe environmental condition.
- Chloride attack is considerable.

(b) Reinforcement works

This type of work is applicable to steel members which have inadequate load carrying capacity.

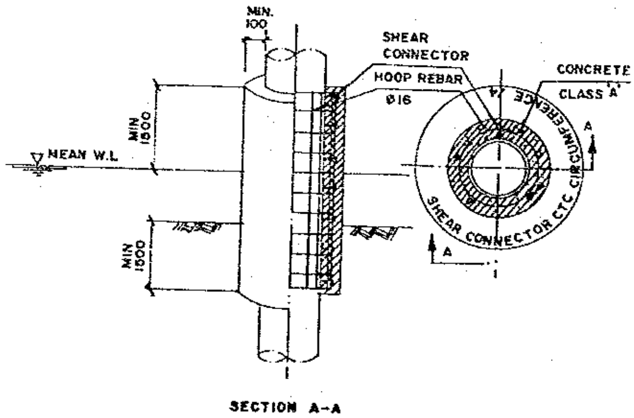
- **Attachment of steel plate (superstructure)**



Application criteria

- Inadequate load carrying capacity.
- Excess bending stress is less than 20 % of allowable stress.
- Non-active corrosion, paint deterioration.

- **Concrete lining (substructure)**



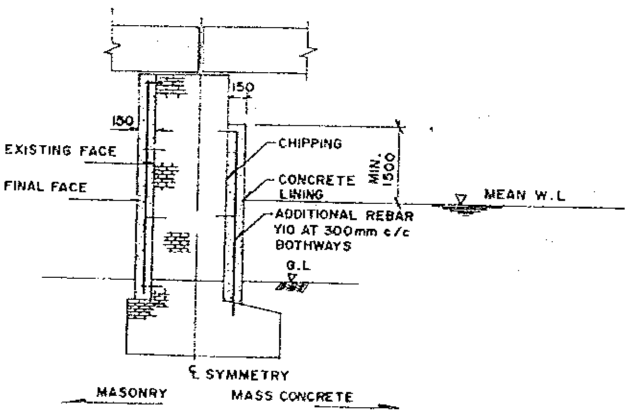
Application criteria

- Steel surface is considerably corroded and load carrying capacity is inadequate.
- Bridge is located in a severe environment.

(4) Protection / Reinforcement to Concrete Substructure

This type of works is applicable to a concrete substructure.

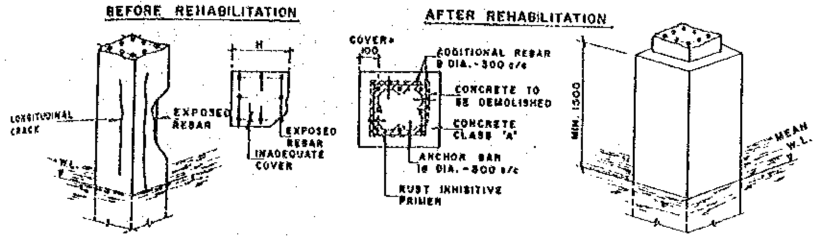
- **Concrete lining to abutment**



Application criteria

- Inadequate minimum cover or bricks are exposed.
- Abrasion of concrete surface or loss of concrete or chemical attack.
- Concrete is carbonated.

- **Concrete lining to piles**



<u>**Application criteria**</u>

- Minimum concrete cover is inadequate.
- Wide longitudinal cracks due to chloride attack or rebar exposure.
- Abrasion of concrete surface or loss of concrete matrix due to inferior concrete or chemical attack.
- Concrete is carbonated.

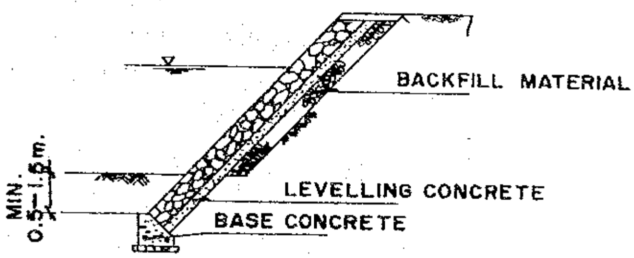
(5) River Training Works

River training work consists of slope protection, foot protection, river bed protection and river alignment depending on where protection work is provided.

(a) Slope protection

This type of work is applicable to river banks around abutments where erosion is observed.

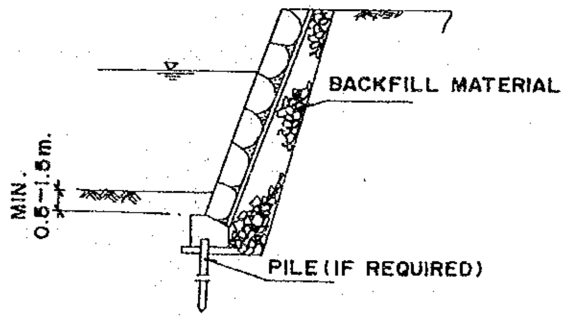
- **Stone masonry**



<u>**Application criteria**</u>

- Slope -
  1:0.5 - 1.5
- Height -
  Less than 5m
- Application -
  Small to medium scale river.

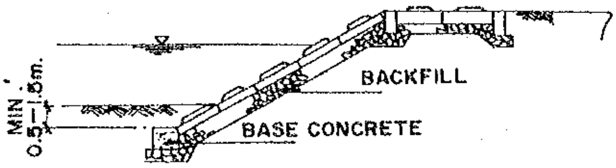
- **Concrete block masonry**



<u>**Application criteria**</u>

- Slope -
  1:0.3 - 1.0
- Height -
  Less than 3m
- Application -
  Rapid stream and small to medium scale river.

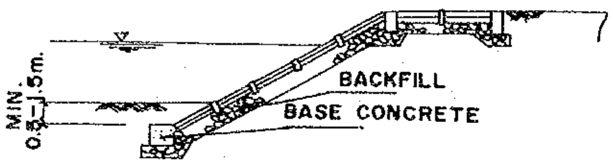
- **Concrete block pitching**



Application criteria

- Slope -
  1:1.5 - 2.0
- Height -
  Less than 5m
- Application -
  Medium to large scale river.

- **Concrete frame**



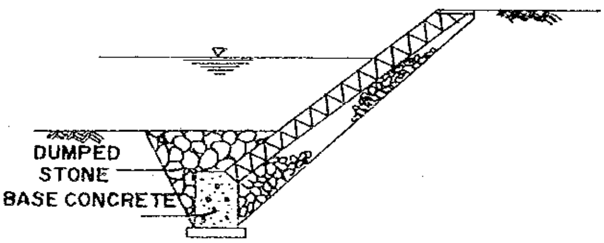
Application criteria

- Slope -
  1:1.5 - 2.0
- Height -
  Less than 5m
- Application -
  Tidal river and bank subjected to wave force.

(b) **Foot protection**

This type of work is applicable to footings for slope protection in order to prevent slope failure caused by scouring action on the river bed.

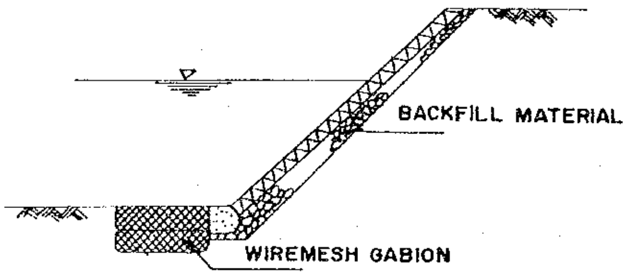
- **Dumped stone**



Application criteria

- Small to medium scale river and foundation ground is relatively solid.

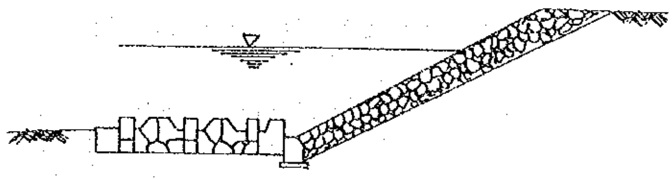
- **Wire Mesh Gabion**



Application criteria

- Small scale river and foundation ground is soft.

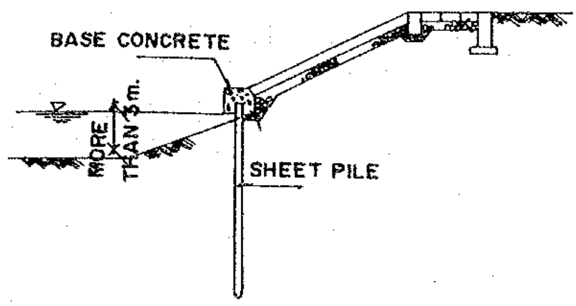
- **Concrete Block Mattress**



Application criteria

- Medium to large scale river or rapid flow velocity.

- **Sheet pile**



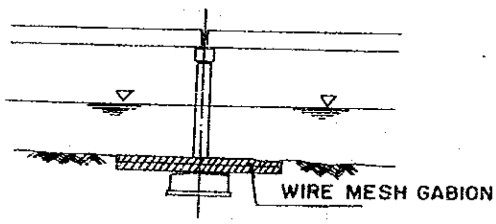
Application criteria

- Normal water level at slope toe is more than about 3.0m and it is difficult to provide base concrete under river bed at slope toe.

(c) **River bed protection**

This type of work is applicable around river piers where local scouring or river bed lowering is observed.

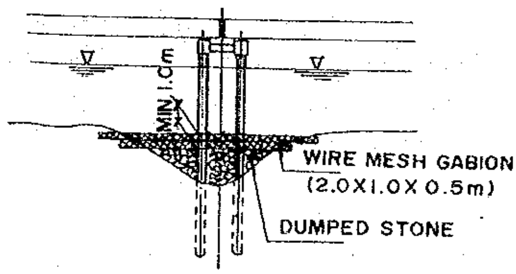
- **Wire Mesh Gabion**



Application criteria

- Foundation protection.

- **Dumped stone and wire mesh gabion**



Application criteria

- Local scouring.

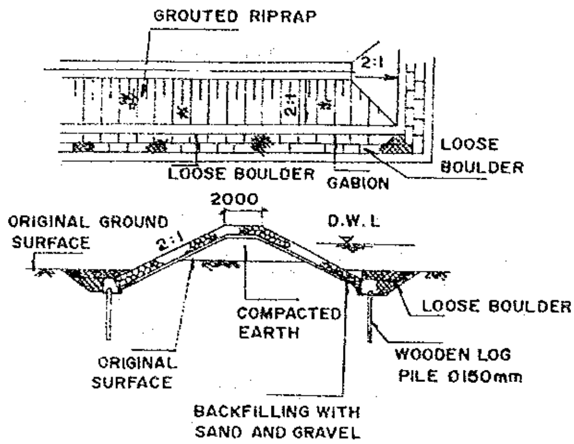
(d) **River realignment**

This type of work is applicable to extremely eroded banks of a meandering river located upstream of a bridge.

- **Spur dike by stone masonry**

Application criteria

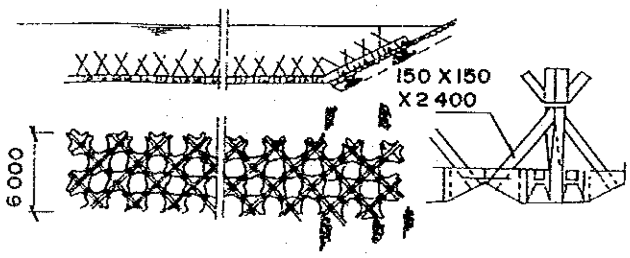
- Large scale river.



- **Spur dike by concrete pile**

Application criteria

- Medium to large scale river.



12.5 Alternative Study

Among the 20 bridges, most of the bridges can be rehabilitated applying the standard rehabilitation plans but for a few of them it is difficult to select the rehabilitation plan. These are bridges in which the selected rehabilitation plans include possible total bridge replacement, replacement of buckle plate slab and raising of bridge grade.

For these major rehabilitation works, a comparative study that is an evaluation of possible several alternatives from the structural aspect, construction cost, maintenance cost, construction period and aesthetic viewpoints were carried out to select the most optimum rehabilitation plan.

The following are alternative studies including possible total bridge replacement for Bridge No. 00166511, raising bridge grade for Bridge No. 00317000 and replacement of buckle plate slab for Bridge No. 00161140.

(1) **Bridge No. 00166510**

This bridge was originally built in 1935 with a single span steel box girder of 10.68m long and later widening was carried out using R.C beam with the same span length. The present bridge width is 7.910m wide of carriageway without sidewalk.

Major defects identified through the assessment are

- o Advance corrosion of the steel member
- o Extensive flaking at R.C. beam and deck slab soffits
- o Wide longitudinal crack along joints between RCB and SBG
- o Open crack in the abutment caused by uneven settlement due to local scouring
- o Inadequate load carrying capacity of R.C. beams
- o Too narrow carriageway and absent sidewalk
- o Local scouring and river bank erosion
- o Decreasing bridge opening due to sedimentation

Taking into account that the bridge has combined defects not only from material deterioration but also from load carrying capacity, functional and hydraulic defects, total replacement of the bridge is considered as one of the possible alternatives for rehabilitation of Bridge No 00166510.

Alt-1 ------- Total replacement of the existing bridge with prestressed inverted "T" beams supported by R.C. inverted "T" abutments with precast R.C. piles.

Alt-2 ------- Similar to Alt-1 but with steel beams as the main girders.

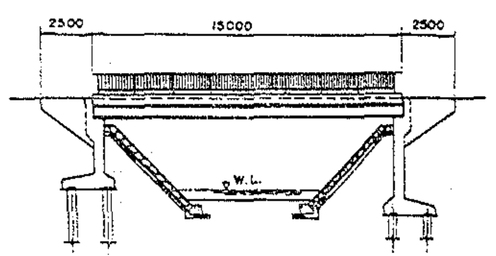
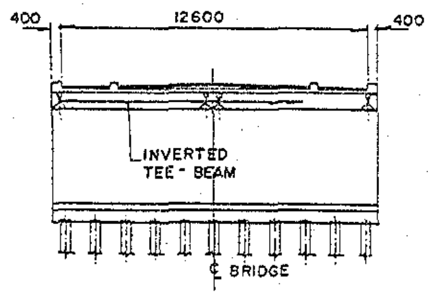
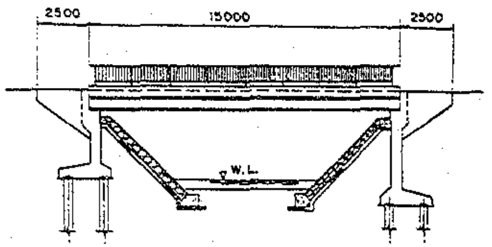
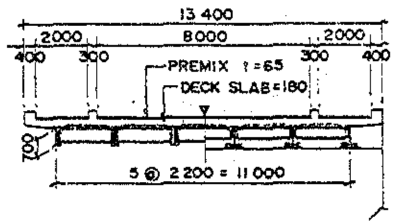
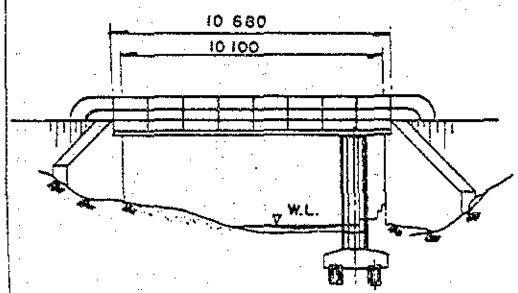
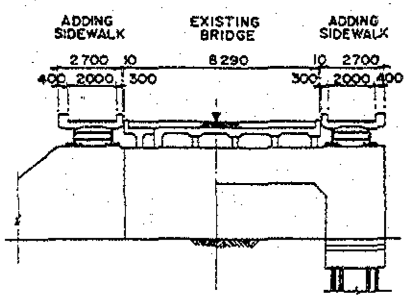
Alt-3 ------- Total rehabilitation of the existing bridge with adding rigid frame abutments adjacent to the existing abutments.

The alternative study is shown in Table 12-6. As depicted in Table 12-6, from construction cost viewpoint Alt-3 is the most superior among them. However, potential problems such as inadequate bridge opening and frequent maintenance and rehabilitation work required still remains. Thus Alternative-1 is selected as the most suitable rehabilitation plan of Bridge 00166510.

TABLE 12-6 ALTERNATIVE STUDY (1)

Bridge No. : 00166510 (SBB + RCB)

Legend : ◎ Good
○ Fair
△ Bad
X Very Bad

	Elevation	Cross Section	Assessment Items	Assessment	Assessment Results	Overall Result
Alternative No. 1			Construction Cost	$0.43 million <1	○	Selected ◎
			Structural View Point	Consistent with current practice in Malaysia.	◎	
			Maintenance View Point	Almost maintenance free.	◎	
			Easy Construction	Relatively easy	○	
			Construction Period	9 months	△	
	Description : Total replacement of the bridge by I.T		Aesthetic View Point	Quite pleasing and widely used as standard highway bridges.	○	
Alternative No. 2			Construction Cost	$0.45 million <1	△	Discarded ○
			Structural View Point	Similar to 1	◎	
			Maintenance View Point	Require maintenance	△	
			Easy Construction	Easier than alternative 1	◎	
			Construction Period	8 months	○	
	Description : Total replacement of the bridge by SBC.		Aesthetic View Point	Similar to alternative 1	○	
Alternative No. 3			Construction Cost	$0.23 million <1	◎	Discarded X
			Structural View Point	Inadequate bridge opening and possible future rehabilitation work	X	
			Maintenance View Point	Require frequent maintenance and inspection.	X	
			Easy Construction	Complicated by construction of new abutment and specialist works such as guniting etc.	X	
			Construction Period	6 months	◎	
	Description : Rehabilitation work such as repainting, patching, guniting of slab soffit and bonding steel plate at beam soffit and installation of new abutment and adding sidewalk		Aesthetic View Point	Unsightly due to the additional abutments.	X	

Note <1 : Direct cost except river training works.

(2) **Bridge No 00317000**

This is a nine span P.C Beam bridge with a total bridge length of 398.5m, spaning over Sungai Endau with parabolic bridge profile and is located at the mouth of the river.

Major defects identified through the assessment are

- o Submerged P.C. beams at both end spans
- o Rebar and P.C. cable exposure on cross beam and main beams of both end spans.
- o Advance chloride attack to the superstructure of both end spans.

Taking into account the defects mainly caused by inadequate bridge opening at both end spans, the following alternatives are considered.

Alt-1 ------ Replacement of badly deteriorated P.C. beams at both end spans to new P.C. beams together with raising the bridge seat by about 1.4m. The abutments are to be reinforced with additional piles.

Alt-2 ------ Similar to Alternative-1 but with steel beams as the main girders. No reinforcement work is required to the abutments.

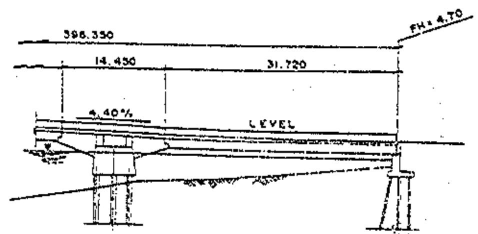
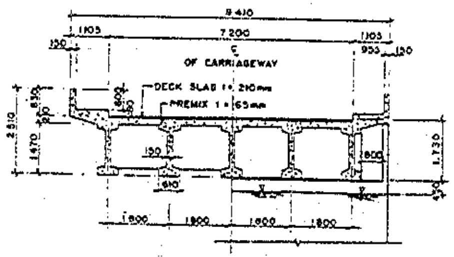
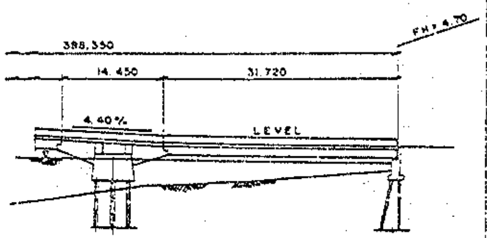
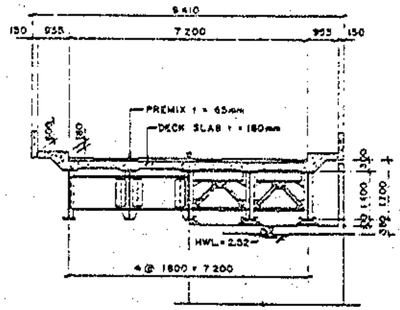
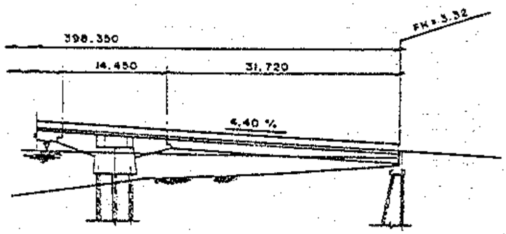
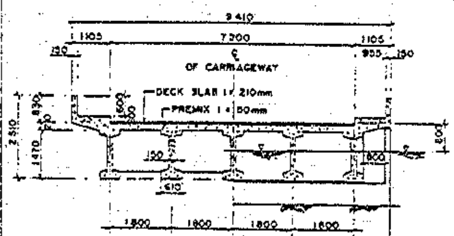
Alt-3 ------ Replacement of the P.C beams at both end spans to new P.C beams with provision of coating instead of raising the bridge seats.

The alternative study shown in Table 12-7 indicates that from construction viewpoint alternative-3 is the most superior among others. However, alternative-3 requires frequent costly maintenance by protective coating of which durability for chloride attack is questionable. Furthermore with this alternatives the beams still remain submerged which is a potential problem. Thus Alternative-1 is selected as an optimum rehabilitation plan.

TABLE 12-7 ALTERNATIVE STUDY (2)

Bridge No. : 00317000 (PCB)

Legend : ◎ Good
○ Fair
△ Bad
X Very Bad

	Elevation	Cross Section	Assessment Items	Assessment	Assessment Results	Overall Result
Alternative No. 1			Construction Cost	$1.84 million (both end span) <1	○	Selected ◎
			Structural View Point	Identical to existing bridge with adequate bridge opening	○	
			Maintenance View Point	Relatively maintenance free but require periodic inspection.	◎	
			Easy Construction	Difficulty arises from the additional piles required.	△	
			Construction Period	10 month	△	
	Description : Replacement of superstructure by PCB at both edge span. Raising bridge seat 1.4m at both abutment. Reinforcement of abutment substructure with additional piles.		Aesthetic View Point	Consistent with existing bridge.	◎	
Alternative No. 2			Construction Cost	$1.90 million (both end span) <1	△	Discarded △
			Structural View Point	This alternative will cause an imbalance pier loadings due to different dead weights.	△	
			Maintenance View Point	Require frequent maintenance.	△	
			Easy Construction	Easier than two alternatives-1 minor difficulty posed by bridge seat raising	○	
			Construction Period	8 month	○	
	Description : Replacement of superstructure by SBC at both edge span. Raising bridge seat 1.4m at both abutments.		Aesthetic View Point	Unsightly due to contrasting surfaces of steel and concrete beams.	△	
Alternative No. 3			Construction Cost	$1.82 million (both end span) <1	◎	Discarded X
			Structural View Point	Similar to alternative 1 but do not solve the flood prone problem.	○	
			Maintenance View Point	Required frequent coating maintenance.	X	
			Easy Construction	Fairly easy as it only involved beams launchings.	◎	
			Construction Period	6 month	◎	
	Description : Replacement of superstructure by PCB with coating at both edge span.		Aesthetic View Point	Similar to alternative 1 but water stains will appear after each flood.	X	

Note <1 : Direct cost

(3) **Bridge No 00161140**

Out of the 216 study bridges, steel beam buckle plate (SBB) is the most dominant bridge type amounting to a total of 76 bridges equivalent to 35.2 %. The defects detected in SBB are, in general, advance corrosion, water stain and paint deterioration and in particular inadequate LTAL load carrying capacity in the buckle plate slab and in the beam.

In addition, the buckle plate, installed with the shape of an arch anchored at each upper flange of the beam, has potential maintenance and structural defects. These cause endless repainting work required for corrosion due to accumulated water at joints between the plate and the beam and consequently leading to possible structural failure from buckling.

Considering the defects detected and potential maintenance and structural defects in the buckle plate, it is concluded to replace the buckle plate with the most suitable alternative out of three listed below.

Alt-1 --------- Replacement of the buckle plate with in-place R.C. slab.

Alt-2 -------- Similar to alternative-1 but using prefabricated reinforcements for the concrete deck slab.

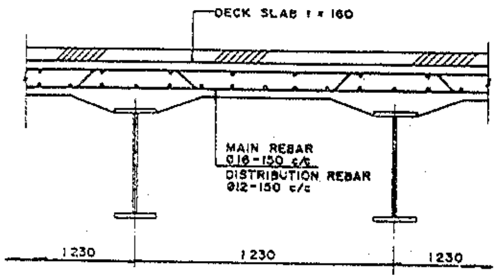
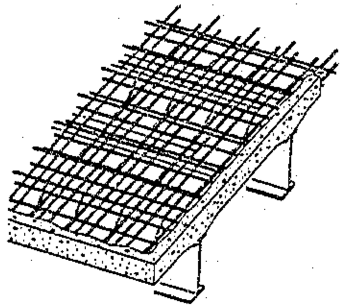
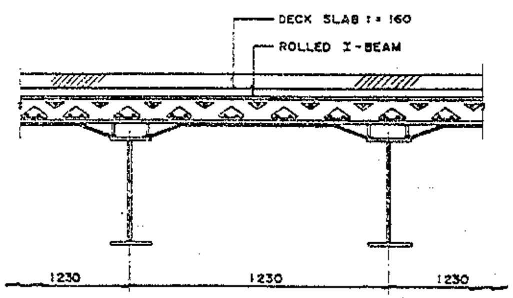
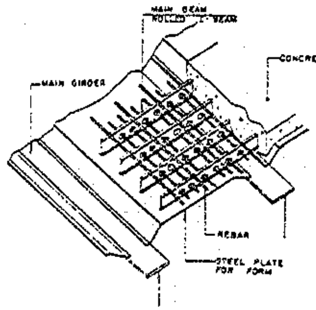
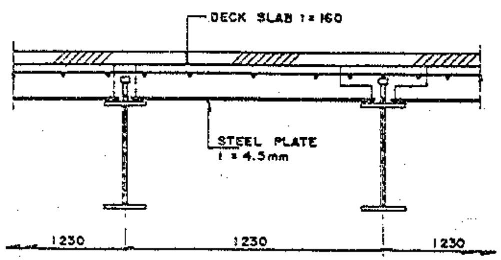
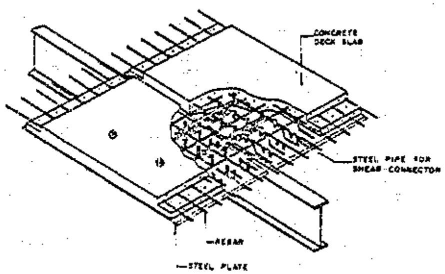
Alt-3 --------- Similar to alternative-1 but using precast reinforced concrete deck slab panels.

The alternative study is shown in Table 12-8. From the Table it is obvious that alternative-1 is the most suitable rehabilitation method from the construction cost viewpoint.

TABLE 12-8 ALTERNATIVE STUDY (3)

Bridge No. : 00161140 (SBB)

Legend : ◎ Good
○ Fair
△ Bad
X Very Bad

	Cross Section	Details	Assessment Items	Assessment	Assessment Results	Overall Result
Alternative No. 1			Construction Cost	S200/m2 <1	◎	Selected ◎
			Structural View Point	Consistent with current practice in Malaysia.	△	
			Maintenance View Point	Require little maintenance.	◎	
			Ease of Construction	A lot of temporary works and site works to be carried out.	△	
			Construction Period	3.0 month (160 sq.m)	△	
	Description : Replacement of buckle plate by reinforced concrete slab.		Aesthetic View Point	Consistent with existing bridge	○	
Alternative No. 2			Construction Cost	S340/m2 <1	△	Discarded △
			Structural View Point	Only quality control of site concreting needs supervision.	○	
			Maintenance View Point	Similar to alternative 1	○	
			Ease of Construction	Less temporary aided by less site works than alternative 1	○	
			Construction Period	1.5 month (160 sq.m)	○	
	Description : Replacement of buckle plate by prefabricated reinforcements for concrete slab.		Aesthetic View Point	Similar to Alternative 1	○	
Alternative No. 3			Construction Cost	S410/m2 <1	X	Discarded X
			Structural View Point	Quality control is much better as most works are done in the factories.	◎	
			Maintenance View Point	Similar to alternative 1	○	
			Ease of Construction	Relatively no temporary works required and there will be very little site concreting.	◎	
			Construction Period	0.75 month (160 sq.m)	◎	
	Description : Replacement of buckle plate by precast concrete deck slab panels.		Aesthetic View Point	Similar to alternative 1	○	

12.6 Results of Preliminary Design

The preliminary design of maintenance and rehabilitation works in the study was carried out in the exercises mentioned in sections 12.3 to 12.5. This section presents summaries of the rehabilitation design results and the corresponding work quantities, while all the maintenance and rehabilitation drawings prepared are attached in Volume IV Drawings. Furthermore, work sequence of the main work items and brief specification are also included in this section.

(1) Summary of Rehabilitation Design Results

Final maintenance and rehabilitation plan and work methods were decided based on the standard rehabilitation design and the alternative study of the major work items. Summary of the final rehabilitation plan for each of the 20 study bridges is tabulated in Table 12-9.

In the rehabilitation design, strengthening design was also carried out for the bridge member which has inadequate LTAL load carrying capacity revealed by the analytic assessment of the bridge. The strengthening design was in principal based on elastic design method in accordance with the assessment criteria established in section 12.2. The results of the strengthening design for the bridge having inadequate LTAL load carrying capacity are summarized in Table 12-10.

(2) Summary of Work Quantities

The work quantities for the 20 bridges were computed based on the preliminary design drawings attached in Volume IV Drawings. A summary of the work quantities for the 20 bridges is tabulated in Table 12-11.

(3) Work Sequence and Specification

Work sequence for the major works such as total replacement of the bridge, raising of the bridge grade, replacement of steel buckle plate slab, etc. is itemized in order in the general view drawing of each bridge, while the sequence of the standard rehabilitation method applied in the preliminary design is presented in each standard drawing.

An applicable specification for the maintenance and rehabilitation works is in principal based on JKR Standard Specification for Road Works (JKR/SPJ/1988), however, for some special items which are not covered in the specification, indicative specification of each special item is briefly described in the Standard Drawings.

Table 12－9 Summary of Rehabilitation Plans and Main Work Item for 20 Bridges

No	Key	State	Year Built	Bridge Type	Bridge Length (m)	Rehabilitation Plan	Main Work Item (Method)
1	114920	Johor	1955	RCB	12.86	Protection work (Slab & Beam)	Gunking to soffit of slab and beams
						Protection work (Piles)	Concrete lining of R.C piles.
2	161140	Perak	1950	SBB	19.11	Replacement (Deck Slab)	Replacement of buckle plate by R.C slab
						Protection work (Abut & Pier)	Concrete lining of substructure
						Adding sidewalk	Adding sidewalk by SBC with substructure on both sides
						Protection work (Abutment)	Installation of slope protection
						Protection work (Pier)	Installation of river bed protection
3	166510	Perak	1935	RCB/SBG	10.72	Total replacement	Total replacement by IT
4	237200	Pahang	1960	SBC/RCB	26.70	Protection work (Slab soffit)	Patching to slab soffit (Span － 2)
						Protection work (Steel beams)	Repainting to all steel beams (Span 1 & 3)
						Protection work (Pier)	Concrete lining of all pier columns
						Protection work (Abutment)	Installation of slope protection.
5	317000	Pahang	1974	PCB	397.32	Protection work (Slab & beams)	Patching to all spalled concrete member
						Raising grade (both end spans)	Raising grade of both side spans
						Protection work (Abutment)	Installation of foot protection
6	319110	Pahang	1962	PCB	121.96	Protection work (Beam & Pile caps)	Epoxy resin injection at pile caps and beams
						Protection work (All concrete surface)	Protective coating to all exposed surfaces
						Protection work (Piers)	Concrete lining of all pier columns
						Reinforcement (Beams)	Steel plate bonding at beam soffit (2 girder type)
7	341600	Terengganu	1955	RCB	36.14	Adding sidewalk	Adding sidewalk by RCB with widened substructure
						Reinforcement (Piers)	Prepacked concrete lining with additional rebar at all crosshead beams
						Protection work (Soffit of Beams & Slab)	Patching at soffit of beams and slab
						Protection work (Piers)	Concrete lining to all columns
						Protection work (Abutment)	Installation of slope protection at both side abutment
8	346740	Terengganu	1973	PCB	152.26	Protection work (Beams, Cross beams & Piers)	Patching on beams, cross beams and piers
						Protection work (Beams & Slab Soffit)	Protective coating to beams and slab soffit
						Reinforcement work (Beams)	Steel plate bonding at beam soffits (2 girder type)
						Protection work (River Bank)	Slope protection to river bank and
						Protection work (River bed)	River bed protection at piers
						Raising of grade	Excavation of both side banks
9	520850	Melaka	1950	SBE	4.27	Protection work (Slab & Beams)	Gunking to all slab soffit and beams
						Protection work (Abutment)	Partial concrete lining
10	546560	Selangor	1939	RCB	6.30	Reinforcement work (Beam Soffit)	Prepacked concrete lining with additional rebars
						Protection work (Slab Soffit)	Patching at slab soffit
11	545980	Selangor	1969	RCS	30.94	Protection work (Slab Soffit)	Patching at slab soffit
						Reinforcement work (Slab Soffit)	Steel plate bonding at slab soffit
						Replacement work (Abutment)	Installation of rigid frame type abutment.
						Protection work (Piers)	Total concrete lining of piers
12	563880	Perak	1972	IT	41.59	Protection work (Piers)	Concrete lining of all pier columns
						Protection work (Piers & Abutment)	Patching on pier crosshead and abutment
13	567840	Perak	1960	PRB	12.12	Protection work (Deck slab)	Provision of waterproof layer on top of deck slab
						Protection work (Pier & Abutment)	Patching for abutment and piers
						Widening of carriageway	Widening by PRB with widened substructures
14	834650	Kelantan	1960	RCS	13.71	Protection work (Deck Slab)	Provision of waterproof layer on top of deck slab
						Protection work (Substructure)	Patching to piers and abutment
						Reinforcement work (Slab Soffit)	Prepacked concrete lining with additional rebar
15	5001070	Johor	1919	SBB	4.77	Protection work (Abutment)	Partial concrete lining for both abutments
						Protection work (Steel Beams)	Repainting of all beams
						Replacement work (Deck Slab)	Replacement of steel buckle plate by R.C slab
16	5803340	Perak	1950	SBB	4.97	Protection work (Piers)	Concrete lining at upstand piles
						Protection work (Steel Beams)	Repainting of all beams
						Replacement (Deck Slab)	Replacement of steel buckle plate by R.C slab
17	5903120	Perak	1950	SBC	23.16	Protection work (Steel Beams)	Repainting of all beams
						Reinforcement work (Deck Slab)	Guniting with additional rebar at deck soffit
						Reinforcement work (Cross Beam)	Provision of cross beams
SPECIAL BRIDGE							
18	－	Sabah	1964	SBC	50.10	Protection work (Steel beams)	Repainting of all beams
						Replacement work (Deck Slab)	Replacement of deck slab by R.C slab
						Reinforcement work (Steel Beams)	Installation of additional beams
						Protection work (Steel Piles)	Concrete lining of all steel pier columns
						Protection work (River Bank)	Installation of slope protection
19	－	Sarawak	1965	SBC	71.60	Protection work (Deck Slab)	Epoxy injection at slab soffit
						Reinforcement work (Deck Slab)	Steel plate bonding at slab soffit
						Reinforcement work (Steel Beam)	Attachment of steel plate to lower flange
20	371000	Kelantan	1962	RCB	840.00	Protection work (Beams)	Epoxy injection for beams
						Protection work (Cross Beams)	Patching to cross beam soffit
						Protection work (River Bank)	Reconstruction of slope protection with foot protection

Table 12−10 Summary of Strengthening Design Results

Bridge No.	Structural Member	Structural Work	Stresses under LTAL	
			Working Stress	Allowable Stress
161140 (SBB) <4	Steel Buckle Plate	Steel buckle plates are to be replaced by R.C deck slab (0 16 ctc 150mm)	131 N/mm2	140 N/mm2
	Steel Beam	No work <3	129	142
319110 (PCB) <5	Main beams of 2 – girder type bridge	Bonding steel plate to beam flange soffit (steel plate 400mm x 4.5mm)	<1 16.6KN	<2 144KN
546560 (RCB)	R.C Beam	Prepacked concrete lining with additional rebar (0 25 – 3 no's per beam)	126 N/mm2	140 N/mm2
546980 (RCS)	Deck Slab (Distribution Rebar)	Bonding steel plate to deck soffit (steel plate 300mm x 10mm – 500mm ctc)	137	140
5903120 (SBC)	Deck Slab (Distribution Rebar)	Prepacked concrete lining with additional rebar (0 10 – 150mm ctc)	88	140
Dambai (SBC)	Main Beam Deck Slab	Additional main beam (2 No's) Replacement of deck slab (0 16 ctc 150mm)	127 105	140 140
Samarahan (SBC)	Main Beam	Attachment of steel plate to bottom flange soffit (steel plates 356mm x 38mm – G2,3) (steel plates 254mm x 38mm – G1,4)	168	183
	Deck Slab	Bonding steel plate to deck soffit (steel plate 200mm x 4.5mm – 500mm ctc)	52	140

Note :

< 1 Tensile force induced by LTAL in the main girders.

< 2 Capacity of steel bonding plate in resisting tension.

< 3 After replacement of buckle plate by R.C slab of which rehabilitation work increases rigidity of the slab, all beams have adequate LTAL load carrying because of effect of the lateral load distribution.

< 4 Applicable to other steel buckle plate beam bridges such as No.5001070 and No. 5803340. The beam of Bridge No. 5001070 be installed shear connectors is considered as a composite beam.

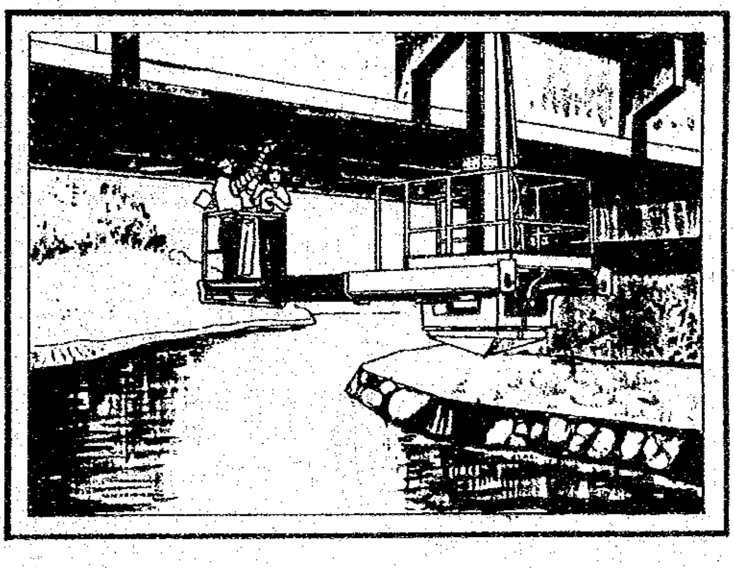
< 5 Applicable to Bridge No. 346740.

Table 12-11 SUMMARY OF WORK QUANTITIES

	MAIN PAY ITEMS	UNIT	FEDERAL BRIDGES																		SPECIAL BRIDGES		
			114920	161140	168510	237200	317000	319110	341800	346740	520850	546560	548980	563680	567840	834850	5001070	5803340	5903120	Subtotal	Dambal	Samasahan	371000
Super – Structure	1. Gunting	m2	127.00								35.90									162.90			
	2. Gunting with rebar	m2																	158.00	158.00			
	3. Patching (Type A)	m2					6.90		12.30											19.20			
	4. Patching (Type B)	m2				1.90			3.10	6.50		6.00	22.60							40.10			
	5. Prepacked lining	m2																		0.00			
	6. Prepacked lining w/ rebar	m2										6.40				104.10				110.50			
	7. Epoxy injection (Type A)	m						144.70												144.70		11.60	2378.90
	8. Epoxy injection (Type B)	m																		0.00		17.70	
	9. Protective coating	m2						356.50		3639.00										3995.50			
	10. Waterproof layer	m2													76.40	84.00				160.40			
	11. Steel bonding plate	m2						24.00		32.00			158.40							214.40		372.00	
	12. Repainting	m2				176.00													251.00	427.00			
	13. Adding cross beam (steel)	t																	0.43	0.43			
	14. Attachment of steel plate	t																		0.00		19.30	
	15. Replacement to R.C Slab	m2		168.00													69.90	39.20		277.10	530.30		
Sub– Structure	1. Epoxy Injection (Type A)	m						29.30												29.30			
	2. Epoxy Injection (Type B)	m						17.90												17.90			
	3. Protective Coating	m2						428.60												428.60			
	4. Concrete lining (Wall)	m2		111.40							9.30						55.40	7.30		183.40			
	5. Concrete lining (Column)																			0.00			
	(a) 410 diameter (Steel Piles)	m																		0.00	87.80		
	(b) 300 x 300 (R.C Piles)	m				45.00														45.00			
	(c) 310 x 310 (R.C Piles)	m							15.50									5.00		20.50			
	(d) 360 x 360 (R.C Piles)	m	34.00																	34.00			
	(e) 380 x 380 (R.C Piles)	m												46.00						46.00			
	(f) 390 x 390 (R.C Piles)	m											74.90							74.90			
	(g) 550 x 550 (R.C Piles)	m							17.40											17.40			
	(h) 610 x 610 (R.C Piles)	m							17.40											17.40			
	(i) 900 diameter (R.C Piles)	m						112.00												112.00			
	6. Crosshead lining (prepacked)																			0.00			
	(a) Prepacked lining + rebar	m2							11.30											11.30			
	(b) Prepacked lining	m2											18.60							18.60			
	7. Patching (Type A)	m2								7.00										7.00			
	8. Patching (Type B)	m2												1.18	0.90	1.20				3.28			0.12
	9. Replacement of Abut. by	m											39.00							39.00			
	rigid frame.																			0.00			
Incidental Facilities	1. Extension drainage pipes.	No's	18.00			18.00		46.00		64.00										146.00			
	2. Water drop	m						264.00		305.00			65.80	96.00						730.80			
	3. Expansion joints (Type A)	m				19.10													35.00	54.10			
	4. Expansion joints (Type B)	m					170.00	72.50	31.70				24.42	30.00						328.62			
Temporary Work	1. Detour road	m		60.00	48.50		72.00											45.00		225.50			
	2. Temporary bridge	m					162.00													162.00			
Scaffolding	1. Substructure – ground support	m3		50.40					222.60				1770.00	270.00						2313.00	220.90		
	2. Superstructure – "	m3	108.00			286.00										53.00			926.00	1371.00			
	3. Superstructure – Hanging	m2					414.00	1105.00	287.30	1299.30										3105.80	530.30	531.80	9240.00
Functional Rehabili– tation	1. Adding sidewalk	m2		84.00					195.90											279.90			
	2. Widening carriageway	m2													89.30					89.30			
	3. Raising grade	m2					597.00													597.00			
Total Replacement		m2			201.00															201.00			
River Training	1. Slope Protection																			0.00			
	(a) Type A	m2		325.00	366.00	443.00			484.00											1618.00	1466.00		
	(b) Type B	m2																		0.00			
	(c) Type C	m2								1332.00										1332.00			
	(d) Type D	m2																		0.00			2254.00
	2. Foot Protection																			0.00			
	(a) Type A	m2																		0.00			
	(b) Type B	m2																		0.00			
	(c) Type C	m2					365.00													365.00			
	(d) Type D	m2																		0.00			
	3. Riverbed protection																			0.00			
	(a) Type A	m2		108.00						3500.00										3608.00			
	(b) Type B	m2																		0.00			
	4. River Alignment																			0.00			
	(a) Type A	m			35.00					500.00										535.00			
	(b) Type B	m																		0.00			

CHAPTER 13

PLANNING OF MAINTENANCE AND REHABILITATION WORKS



CHAPTER 13

PLANNING OF MAINTENANCE AND REHABILITATION WORKS

13.1 General

Planning of the maintenance and rehabilitation works covering all the study bridges was carried out based on a supplemental bridge survey and the results of preliminary rehabilitation design.

The supplemental bridge survey covering 199 bridges, which were discarded either for visual inspections or for the detailed survey, was carried out to measure the extent of damage, to select the rehabilitation method, and to estimate the work quantity, since the NALS data did not cover the quantitative damage data which are requisite to formulate the rehabilitation program covered those 199 bridges, as originally planned.

The bridges covered in the supplemental survey are divided into 2 groups: one group is the 121 bridges which were discarded for the visual inspection during Phase I(A) and the other group is the 78 bridges which were visually inspected but discarded for the detailed structural survey during Phase II(A). The purpose of the supplemental bridge survey, thus, depends on the above two bridge categories, i.e. for 121 bridges the purposes are to rate type, degree and extent of damage, to select a rehabilitation plan and to estimate the work quantities, while for 78 bridges of which damage condition rating was completed during Phase I(A) are, thus, to select a rehabilitation plan and to estimate the work quantities.

13.2 Methodology of Supplemental Bridge Survey

13.2.1 Preparatory Work

Prior to commencement of the survey work, a standard rehabilitation plan survey sheet was prepared so as to enable systematic recording of damage information and quantity data for all 199 bridges. Each standard sheet contains bridge data, a plan and profile of each bridge as well as a blank formatted table for entering damage data, the location of damage and its extent, rehabilitation plan and the work quantity.

In addition, damage condition rating form and bridge function survey sheet which were used in the visual inspection during Phase I(A) were also adopted to quantitatively rate the damage and to obtain information as to flood water level, availability of a detour road and necessity of adding sidewalks.

13.2.2 Setting Up Criteria in Selecting Rehabilitation Plan

Prior to carrying out the field survey, criteria in selecting a suitable rehabilitation plan was set up mainly based on the results of the preliminary design so that the rehabilitation plan selected during the survey would be uniform and standardized regardless of the number of inspection teams and the selection method.

The rehabilitation plans in the study are broadly divided into three categories comprising of structural rehabilitation work, functional rehabilitation work and hydraulic rehabilitation work. The structural rehabilitation work is to rectify a deteriorated bridge member and/or to strengthen a bridge member which has inadequate load carrying capacity, while the functional rehabilitation work is to improve the bridge function by widening carriageway in terms of traffic capacity, adding sidewalk in terms of pedestrian flow capacity, or raising bridge grade from bridge opening capacity viewpoint. Moreover, hydraulic rehabilitation work is mainly to protect river bank or river bed in the vicinity of abutments and river piers.

Accordingly the selection criteria of the rehabilitation plan is also subdivided into the above three categories and is discussed in the following section:

(1) Structural Rehabilitation Criteria

The structural rehabilitation works are broadly divided into three categories comprising of protection, reinforcement (strengthening) and replacement. Each rehabilitation plan has different rehabilitation method depending on the type of defect, its extent and degree as well as the cause of defect. Therefore, the purpose of setting up the structural rehabilitation criteria is to enable the designation of a specific type of rehabilitation method for the corresponding type of structural defects consisting of material deterioration defect and load carrying capacity defect.

- **Protection Work**

Protection work is applicable to a bridge member which has adequate load carrying capacity which was determined with the aid of the analytic assessment results of the 20 representative bridges in preliminary design, but has minor material deterioration defects including inactive cracks due to shrinkage, creep or temperature or some construction deficiencies such as honeycomb, flaking, and water stain.

A specific rehabilitation method for the protection work depends on the type of defect, its extent and degree as well as the cause of defect. Each type of rehabilitation method applicable in the survey is tabulated in Table 13-1 together with the corresponding application criteria.

Table 13-1 Type of Protection Method and Corresponding Application Criteria

Rehabilitation Plan	Method	Application Criteria
o Protection work to concrete	– Epoxy Injection	o Cracks are not active and surface width is more than 0.2mm, but less than 3.0mm./1 o Reason for the crack appearance is due to shrinkage or creep of concrete. o No water leakage and no liquid rust. o No carbonation and no chloride attack. /1 : If surface crack width is more than 3.0mm, apply cement paste injection.
	– Patching	o Defects such as honeycomb, flacking, cavity etc. are not active. o Reason for these defects are mainly due to inferior concrete or poor workmanship. o Minimal carbonation, no chloride attack and no water leakage. o Adequate concrete cover. o Defective area is scattered.
	– Guniting	o Cracks of which surface width is less than 0.20 mm are not active. o Concrete is slightly carbonated. o Minimum concrete cover is inadequate. o No water leakage. o Defective area is extensive.
	– Protective Coating	o Cracks are not active and surface width is less than 0.2mm. o No water leakage, no scaling and no flaking. o Minimal carbonation and no chloride attack. o Adequate concrete cover.
	– Waterproof Layer	o Water stain, free lime and other associated defects are observed at slab soffit. o Defects are not active. o Water is penetrating from top of slab through defective concrete or inferior joints between precast members.
	– Concrete Lining (Wall)	o Inadequate minimum cover or bricks are exposed. o Abrasion of concrete surface or loss of concrete matrix due to inferior concrete or chemical attack. o Concrete is carbonated.
	– Concrete Lining (Piles)	o Minimum concrete cover is inadequate. o Wide longitudinal cracks due to chloride attack or rebar exposure. o Abrasion of concrete surface or loss of concrete matrix due to inferior concrete or chemical attack. o Concrete is carbonated.
o Protection works to steel member	– Repainting (Superstructure)	o Adequate load carrying capacity. o Non–active corrosion and paint deterioration
	– Repainting (Substructure)	o Steel surface is slightly corroded but load carrying capacity is still adequate. o Bridge is located in a non–severe environ–mental condition.
	– Concrete Lining	o Steel surface is considerably corroded but load carrying capacity is adequate. o Bridge is located in a severe environmental condition.

- **Reinforcement (Strengthening) Work**

Reinforcement (strengthening) work is applicable to a bridge member which has major material/structural defects or which has inadequate LTAL load carrying capacity judged by the analytic assessment results of the 20 bridges.

Major defects which require reinforcement work include active cracks due to bending or shear force, live or progressing settlement and serious section loss.

The analytic assessment of the 20 representative bridges for the detailed structural survey[1] revealed that the buckle plate slab of SBB, steel beam of SBB[2] and PC beam of 2 girder type have inadequate LTAL load carrying capacity. These results are applied to the other bridges of the same type in the survey.

A specific rehabilitation method of the reinforcement work also depends on construction material used, extent of excess stress, and type of associated damage together with its extent and degree. Each type of the rehabilitation method is shown in Table 13-2 together with the corresponding application criteria.

- **Replacement Work**

Replacement work is applicable to a bridge member or whole bridge of which rehabilitation work(s) is likely to be beyond economic repair.

A bridge member which has major structural/potential defects and which has inadequate load carrying capacity was designated to be replaced with an appropriate bridge member.

Furthermore, a bridge which has combined rehabilitation plans not only from structural defect but also from functional defect and/or hydraulic defect was designated to be replaced by a new bridge.

Notes:

1. The assessment results indicated that two RCSs (lateral direction only), one concrete deck slab, and RC beam have inadequate loading capacity. However, these results which are caused by extreme material deterioration or design deficiency could be considered as an exceptional case and are not applicable to other bridges of the same type. Nevertheless, these defects can be identified by visual inspection.

2. After replacement of buckle plate with R.C. Slab which increases the rigidity of slab, all beams have adequate LTAL load carrying capacity as a result of better lateral load distribution, provided there is no serious section loss.

Table 13-2 Type of Reinforcement Method and Corresponding Application Criteria

Rehabilitation Plan	Method	Application Criteria
o Reinforcement work to concrete	– Steel Plate Bonding	o Inadequate load carrying capacity (Inadequate amount of reinforcement bar) o No water leakage and no carbonation. o Inadequate for additional stress in beams and slab due to additional dead load. o Active cracks due to bending moment or shear force. o Adequate concrete cover.
	– Prepacked concrete lining w/ additional rebar	o Inadequate loading capacity. o Various active cracks due to bending moment or shear force. o Inadequate concrete cover. o Suffered mild chloride attack or advanced carbonation. o Defective area is extensive.
	– Guniting w/ additional rebar	o Inadequate loading capacity. o Various active cracks due to bending moment or shear force. o Adequate for additional stress in beams and slab due to additional dead load. o Bridge is located in relatively severe chloride environment. o Advance carbonation. o Defective area is extensive.
o Reinforcement work to steel	– Attachment of steel plate	o Inadequate load carrying capacity. o Excess bending stress is less than 20% of allowable stress, o Non–active corrosion, paint deterioration.
	– Concrete lining w/ additional rebar.	o Steel surface is considerably corroded and load carrying capacity is inadequate. o Bridge is located at severe environmental condition.

(2) Functional Rehabilitation Criteria

The functional rehabilitation plan is divided into three categories consisting of widening carriageway, adding sidewalk and raising grade. Criteria for applying these rehabilitation plans is discussed below:

- **Widening carriageway**

The necessity for widening carriageways on the study bridges was assessed in Phase I(A) stage by comparison of traffic capacity with current demand volume at the bridge location. Basically these assessment results were utilized in determining the necessity for widening the bridge.

- **Adding sidewalk**

The criteria for adding sidewalk, established in Phase I(A), was based on whether a bridge without sidewalk is located in urban area or otherwise and its proximity with institutional public facilities such as schools, hospitals, mosques and other landmarks to the bridge.

Applying the same criteria, the surrounding area of each bridge within about one km was surveyed to identify whether the bridge was located within the above specified condition. If a bridge without sidewalk is located in the above condition, it was determined that sidewalk should be provided.

- **Raising grade**

Based on interview survey with local residents living in the vicinity of the bridge site, information on whether the study bridge has been submerged during flood was obtained. If the bridge had been submerged, additional information was also obtained such as approximate flood frequency per year and flood duration time.

If a bridge is found to have been submerged, it was then determined that raising grade is required or bridge length has to be extended to cater for flood flow.

(3) Hydraulic Rehabilitation Criteria

Hydraulic rehabilitation plan includes slope protection, foot protection, river bed protection and river alignment depending on the extent and nature of the hydraulic problem encountered such as scour, erosion, flood flow at bridge site.

- Slope protection is applicable to river banks adjacent abutments where erosion is observed.

- Foot protection is applicable to footings of the slope protection in order to prevent slope failure caused by scouring on the river bed.

- River bed protection is applicable to river beds surrounding the river piers where local scouring or river bed lowering is observed.

- River alignment (rechanneling) work is applicable to extremely eroded banks of a meandering river located in the vicinity of the bridge upstream.

Selection of a specific rehabilitation method depends on the stream type, river scale, flood flow velocity, foundation type and geology of each site. Type of rehabilitation method for each plan is shown in Table 13-3 together with the corresponding application criteria.

Table 13-3 Type of Hydraulic Rehabilitation Plan and Corresponding Application Criteria

Rehabilitation Plan	Method	Application Criteria
o Slope Protection	– Stone Masonry	o Slope ; 1 : 0.5 – 1.5 o Height ; Less than 5m o Application Small to medium scale river
	– Concrete Block Masonry	o Slope ; 1 : 0.3 – 1.0 o Height ; Less than 3m o Application Rapid stream and small to medium scale river.
	– Concrete Block Pitching	o Slope ; 1 : 1.5 – 2.0 o Height ; Less than 5m o Application Medium to large scale river.
	– Concrete Frame	o Slope ; 1 : 1.5 – 2.0 o Height ; Less than 5m o Application Tidal river and bank subjected to wave force.
o Foot Protection	– Dumped Stone	o Small to medium scale river and foun– dation ground is relatively solid.
	– Wire Mesh Gabion	o Small scale river and foundation ground is under soft type.
	– Concrete Block Matress	o Medium to large scale river or rapid flow velocity.
	– Sheet Piling	o Normal water level at slope toe is more than about 3.0m and it is difficult to provide base concrete under river bed at slope toe.
o River Bed Protection	– Wire Mesh Gabion	o Foundation protection.
	– Dumped Stone & Wire Mesh Gabion	o Local scouring.
o River Realignment	– Spur Dike by Stone Masonry	o Large scale river.
	– Spur Dike by Concrete Pile	o Medium to large scale river.

13.3 Planning Results of Maintenance and Rehabilitation Works

13.3.1 Assessment of Bridge Statistics

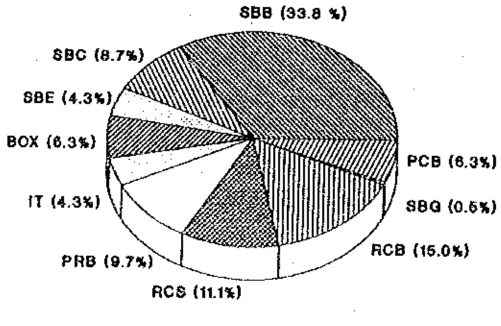
During the supplemental bridge survey covering 199 bridges, it was found out that 11 bridges have been replaced or are being constructed by the Government. Therefore the total number of the study bridges has been reduced from 216 to 205 bridges (including 17 bridges for detailed survey). The supplemental bridge survey data was recorded in the survey sheets for each of the study bridges which register the selected rehabilitation method together with the corresponding work quantities and damage rating. A summary of these data is attached in Appendix-Q of Volume III.

Statistical assessment of all the study bridges is presented in Figure 13-1 and it revealed that the most dominant bridge in the study is Steel Beam Buckle plate bridge (SBB) which made up 33.8% of all the bridges. Reinforced Concrete Beam bridges (RCB) formed about 15.0% of the bridges, while RC Slab bridges (RCS) made up 11.1% of the study bridges. The proportion of the rest in decreasing percentage is as follows; Precast RC Beam bridges (PRB) 9.7%, Steel Beam Concrete Slab bridges (SBC) 8.7%, Prestressed I-Beam with Slab bridges (PCB) 6.3%, RC Box culvert (BOX) 6.3%, Pretensioned Inverted-T Beam bridges (IT) 4.3%, Steel Beam Encased (SBE) 4.3% and Steel Box Girder bridge (SBG) 0.5%.

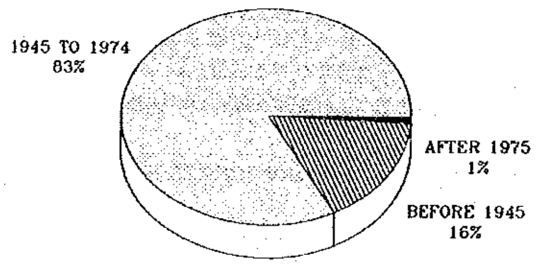
All the study bridges were grouped into three groups and the statistical assessment as presented in Figure 13-1 revealed that 16.2% of the bridges were built before 1945, 82.8% were built between 1945 to 1974 and 1.0% were built after 1975.

Figure 13-1 Statistic of 205 Bridges

Percentage of 205 bridges by type of construction



Percentage of 205 bridges by age group



Out of the 205 study bridges, four bridges were found to be in very bad condition and are assessed to be beyond economic repair, hence these bridges are recommended to be totally replaced by constructing a new bridge. The four bridges that have to be replaced are bridge 00166510 (SBG), 08603735 (SBB), 00338580 (PCB) and 06000970 (SBE).

In addition, some of the bridges have been found to be inadequate in terms of a functional viewpoint, hence they have to be rehabilitated accordingly with measures such as adding sidewalks, widening carriageways and raising grades. Four out of the 205 study bridges (i.e. 2.0%) need widening of carriageway, 17numbers (8.3%) need addition of sidewalk and 8 bridges require raising of grade. A list of those bridges which need functional rehabilitation is shown in Table 13-4.

Table 13-4 Final List of Study Bridges which Require Rehabilitation Work based on Functional Viewpoints

Type of Rehabilitation Work	No's of Bridges	List of Bridges
Widening of Carriageway	4	00567840 (PRB), 00838100 (RCS), 01800060 (RCS), 01800670 (SBC)
Adding of Sidewalk	17	00113760 (RCB), 00161140 (SBB), 00161290 (SBB), 00303890 (RCS) 00313150 (SBE), 00313520 (RCS), 00336310 (RCB), 00341800 (RCB) 00519700 (PRB), 00521710 (RCB), 00700660 (PCB), 05102670 (SBB) 06406260 (SBB), 06702060 (SBE), 07604020 (SBB), 07604160 (SBB) 08604640 (SBB)
Raising of Grade	8	00304390 (SBC), 00317000 (PCB), 00700750 (RCS), 00834950 (RCS) 00838100 (RCS), 02305970 (RCS), 05102380 (SBB), 05300960 (SBB)

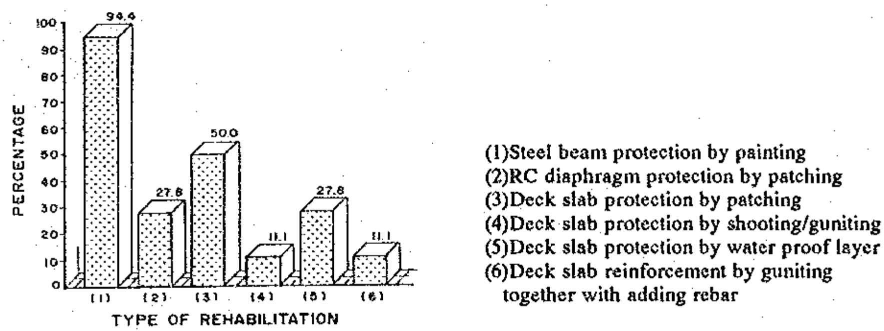
13.3.2 Assessment of Rehabilitation Plan

The purpose of assessment is to highlight the most dominant rehabilitation work which is required to be carried out on the components of each particular bridge type which has been observed to be in distress. The rehabilitation method adopted for each particular damage and bridge type is derived and applied from the criteria established in section 13-2. The assessment of the rehabilitation plan for each bridge type is presented in a X-Y graph in which the Y-axis indicates the percentage of bridge components which requires a particular type of rehabilitation method and X-axis indicates the type of rehabilitation method to be applied for the particular bridge member and type.

(1) Steel Bridge with RC Slab Bridges (SBC)

The most dominant type of rehabilitation plan for SBC bridges, as shown in Figure 13-2, is protection by repainting which constitutes of 94.4% of the bridges. This shows that 94.4% of SBC bridges have corroded and lack of maintenance.

Figure 13-2 Rehabilitation Method for SBC Type Bridges

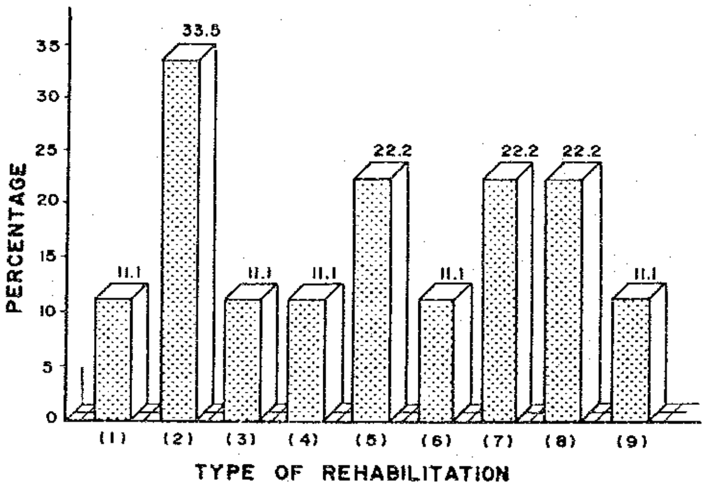


The second most dominant rehabilitation plan for this type of bridge is deck slab protection by patching which constitutes 50% of SBC bridges, followed by concrete deck slab protection by water proof layer which involves 27.8% of the bridges.

(2) Encased Steel Beam Bridges (SBE)

The most dominant type of rehabilitation plan for SBE bridges, as shown in Figure 13-3, is protection of the encased concrete by patching which constitutes of 33.3% of the bridges. Followed by reinforcement of encasing concrete by lining, deck slab protection by patching and deck slab protection by guniting where each type of rehabilitation method involves 22.2% of the SBE bridges. This phenomena is caused by inferior quality concrete and poor workmanship used in the original construction. This type of bridge has high water cement ratio, porous concrete, flaking and wide spread honey-comb.

Figure 13-3 Rehabilitation Method for SBE Type Bridges

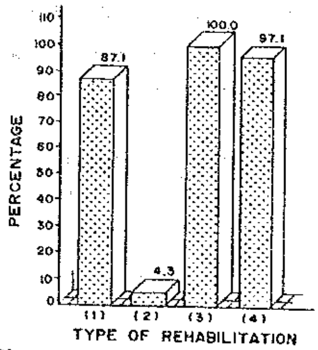


(1)Steel beam protection by painting
(2)Encasing concrete protection by patching
(3)Encasing concrete protection by guniting
(4)Encasing concrete reinforcement by bonding steel plate
(5)Encasing concrete reinforcement by lining
(6)Deck slab protection by injection
(7)Deck slab protection by patching
(8)Deck slab protection by shooting/guniting
(9)Deck slab reinforcement by guniting together with adding rebar

(3) Steel Beam Buckle Plate Bridges (SBB)

The most dominant type of rehabilitation plan for SBB bridges, as shown in Figure 13-4, is total replacement of buckle plate which involves all the SBB bridges in the study. Its replacement is necessary to avoid endless maintenance due to corrosion of the plate which is caused by water collecting at the springing of the arch plate and also most of the buckle plate has corroded very badly at the joints with the steel girder. The buckle plate is also contributing very little in terms of the lateral rigidity of the bridge. The second most dominant rehabilitation plan for this type of bridges is total replacement of the bearing which involved 97.1% of the SBB bridges. Next is the protection of steel girder by repainting which constitutes of 87.1%, followed by reinforcement of steel girder by welding additional steel plate which involves 4.3%.

Figure 13-4 Rehabilitation Method for SBB Type Bridges

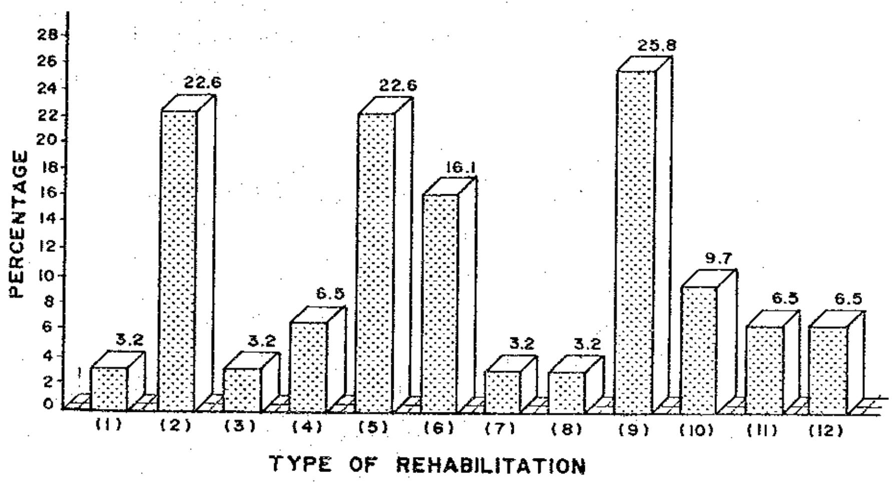


(1)Steel beam protection by painting
(2)Steel beam reinforcement by welding additional steel plate
(3)Total replacement of buckle plate by RC slab
(4)Total replacement of bearing (steel)

(4) RC Beam Bridges (RCB)

The most dominant type of rehabilitation plan for RCB bridges, as shown in Figure 13-5, is protection of deck slab by patching which constitutes 25.8% of the bridges. The second most dominant rehabilitation method for this type of bridge is protection of the concrete beam by patching which involves 22.6% of bridges, similarly with reinforcement of the concrete beam by bonding steel plate which involves 22.6% of the RCB bridges. Damaged on concrete material for this type of bridge is caused by both inferior quality concrete as a result of high water cement ratio and poor workmanship, this has resulted in numerous honeycomb.

Figure 13-5 Rehabilitation Method for RCB Type Bridges

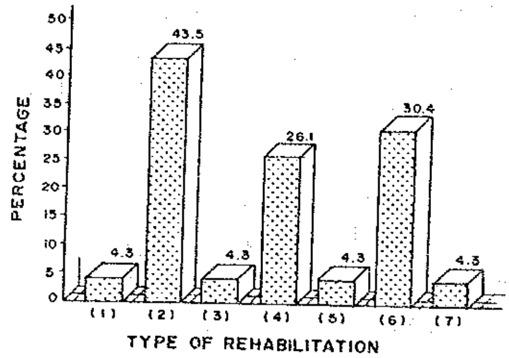


(1)Repainting of encased Steel (for RCB bridge which has SBE on some span)
(2)RC beam protection by patching
(3)RC beam protection by coating
(4)RC beam protection by shooting/guniting
(5)RC beam reinforcement by bonding steel plate
(6)RC beam reinforcement by lining
(7)RC beam reinforcement by jacketing
(8)Deck slab protection by injection
(9)Deck slab protection by patch ing
(10)Deck slab protection by shoot ing/guniting
(11)Deck slab protection by water proof layer
(12)Deck slab reinforcement by guniting together with addition of rebar

(5) RC Slab Bridges (RCS)

The most dominant type of rehabilitation plan for RCS bridges, as shown in Figure 13-6, is protection of deck slab by patching which involves 43.5% of the bridges. Damaged concrete for this type of bridge is mainly caused by flaking of inferior concrete. The second most dominant rehabilitation plan is deck slab reinforcement by guniting together with addition of rebar, which involves 30.4% of RCS bridges. Guniting is suitable at places where the rebar has been exposed and the material is badly deteriorated. The third most dominant rehabilitation plan is protection of deck slab by provision of a water proof layer particularly at top of slab which involves 26.1% of the RCS bridges.

Figure 13-6 Rehabilitation Method for RCS Type Bridges

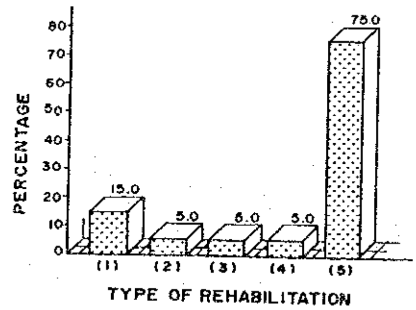


(1)RC slab protection by injection
(2)RC slab protection by patching
(3)Deck slab protection by shooting/guniting
(4)Deck slab protection by water proof layer
(5)RC slab reinforcement by bonding steel plate
(6)Deck slab reinforcement by guniting together with addition of rebar
(7)Reinforcement of deck slab by lining

(6) Precast RC Beam Bridges (PRB)

The most dominant type of rehabilitation plan for PRB bridges, as shown in Figure 13-7, is protection of deck slab by applying a water proof layer which involves about 75.0% of the PRB bridges. A water proofing layer is required for this type of bridge because of very poor longitudinal joints between adjacent beams. The second most dominant rehabilitation plan is protection of the concrete beam by patching at the flaking concrete surfaces which involves 15.0% of bridges. The other rehabilitation plans for this type of bridge include concrete beam protection by shooting 5%, reinforcement of concrete beam by bonding steel plate 5% and protection of concrete slab by patching which involves 5% of PRB bridges.

Figure 13-7 Rehabilitation Method for PRB Type of Bridges



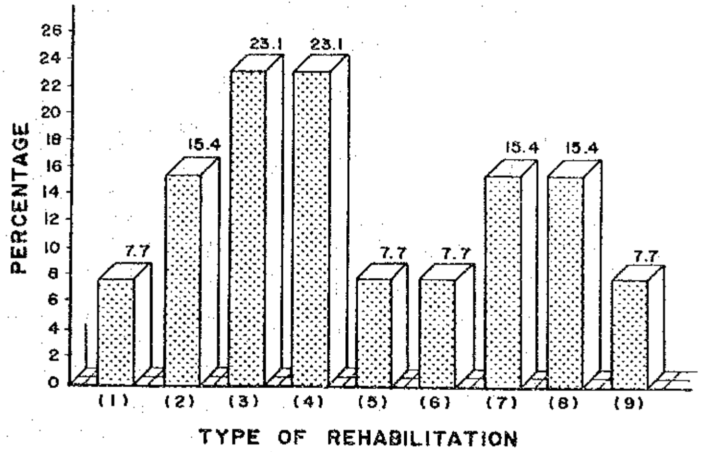
(1)RC beam protection by patching
(2)RC beam protection by shooting/guniting
(3)RC beam reinforcement by bonding steel plate
(4)Deck slab protection by patching
(5)Deck slab protection by water proof layer

(7) Prestressed I-Beam Bridges (PCB)

The most dominant type of rehabilitation plan for PCB bridges, as shown in Figure 13-8, is reinforcement of prestressed I-beams by bonding steel plates which constitutes about 23.1% of the PCB bridges. Reinforcement is required for PCB bridges of the type with only two girders per deck. The next and equally dominant rehabilitation method for this type of bridge is protection of concrete deck slab by application of water proof layer which involves

23.1% of PCB bridges. Rehabilitation by protection of prestressed I-beam by patching and coating involves 15.4% of the PCB bridges each. Protection of RC deck slab by patching also involves 15.4% of the PCB bridges. The reason for these phenomena is the same as RCB and RCS bridges mentioned above.

Figure 13-8 Rehabilitation Method for PCB Type Bridges

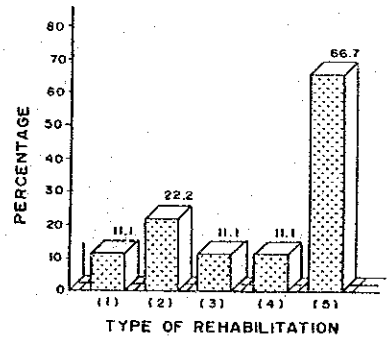


(1)Prestressed I-beam protection by injection
(2)Prestressed I-beam protection by patching
(3)Prestressed I-beam protection by coating
(4)Prestressed I-beam reinforcement by bonding steel plate
(5)Prestressed I-beam reinforcement by lining
(6)Partial replacement of prestressed I-beam
(7)Deck slab protection by patching
(8)Deck slab protection by coating
(9)Deck slab protection by shooting/guniting
(10)Deck slab protection by water proof layer
(11)Partial replacement of Deck slab

(8) Pretensioned Inverted T-Beam Bridges (IT)

The most dominant type of rehabilitation for IT bridges, as shown in Figure 13-9, is deck slab protection by application of a water proof layer which involves about 66.7% of the IT bridges. This is because the deck slab is quite porous with free lime. The next most dominant rehabilitation plan for this type of bridge is protection of Pretensioned Inv-T beam by patching at the spalled and flaking part of the soffit of beam which involves 22.2% of IT bridges.

Figure 13-9 Rehabilitation Method for IT Type Bridges

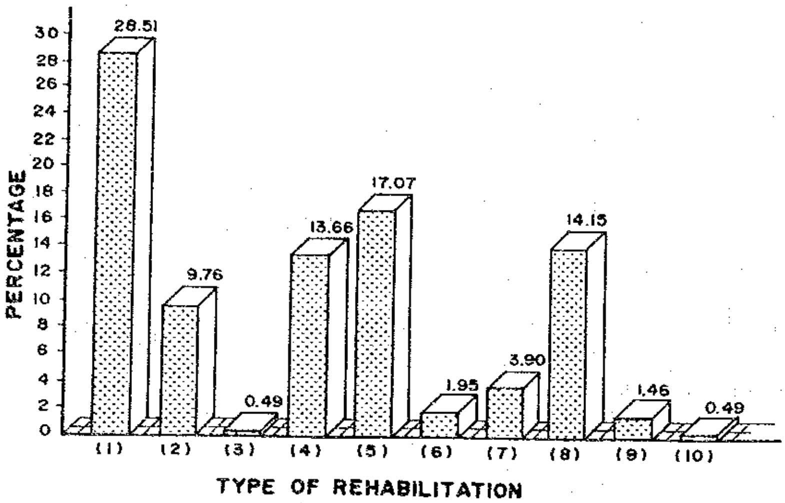


(1)Pretensioned Inv-T beam protection by injection
(2)Pretensioned Inv-T beam protection by patching
(3)Pretensioned Inv-T beam reinforcement by bonding steel plate
(4)Deck slab protection by patching
(5)Deck slab protection by water proof layer

(9) Abutments

The most dominant type of rehabilitation plan for abutments for all bridges in the study, as shown in Figure 13-10 is abutment protection by mortar injection which involves about 28.1% of the bridges. Rehabilitation by injection is required due to shrinkage cracks at construction joints on abutment walls and at its joint with wingwalls. The next most dominant rehabilitation plan for abutments is reinforcement of the damaged concrete material of the abutment by partial concrete lining which involves about 17.07% of all the bridges due to abrasion and acid or chemical attack. The third most dominant rehabilitation method for abutments is protection of the abutment foundation by revetments which constitutes 14.15% of all the bridges. Revetments is required because of serious scour at the slope of the abutment. The forth dominant rehabilitation method is protection of the main body of the abutment by guniting which involves about 13.66% of the bridges. This is mainly caused by flaking and spalling of concrete surface, and carbonation.

Figure 13-10 Rehabilitation Method for Abutments

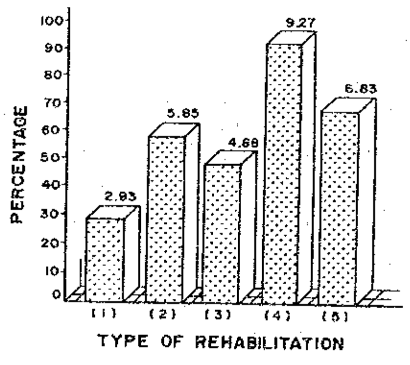


(1)Abutment protection by injection
(2)Abutment protection by patching
(3)Abutment protection by coating
(4)Abutment protection by shooting/guniting
(5)Abutment reinforcement by partial concrete lining
(6)Abutment reinforcement by total concrete lining
(7)Abutment reinforcement by sheet piling
(8)Protection of abutment foundation by revetments
(9)Protection of abutment foundation by foot protection
(10)Reinforcement of abutment foundation by underlining

(10) Piers

The most dominant type of rehabilitation plan for piers in all the bridges, as shown in Figure 13-11, is reinforcement of the damaged concrete material of the pier by partial concrete lining which involves 9.27% of all the bridges. The distress is caused by chloride attack on the rebar which has resulted in the rebar to corrode and expand, which in turn causes the concrete to crack. The next dominant rehabilitation method is pier reinforcement by total lining which involves about 6.83% of the bridges. Reinforcement of piers is required because of the seriousness of the crack, carbonation, chloride attack and loss of concrete matrix in the piers. The third most dominant rehabilitation method is pier protection by patching which involves 5.85% of the bridges in the study. Rehabilitation by patching is required due to flaking and loss of concrete matrix on the surface of the pier.

Figure 13-11 Rehabilitation Method for Piers



(1)Pier protection by injection
(2)Pier protection by patching
(3)Pier protection by shooting/guniting
(4)Pier reinforcement by partial concrete lining
(5)Pier reinforcement by total concrete lining