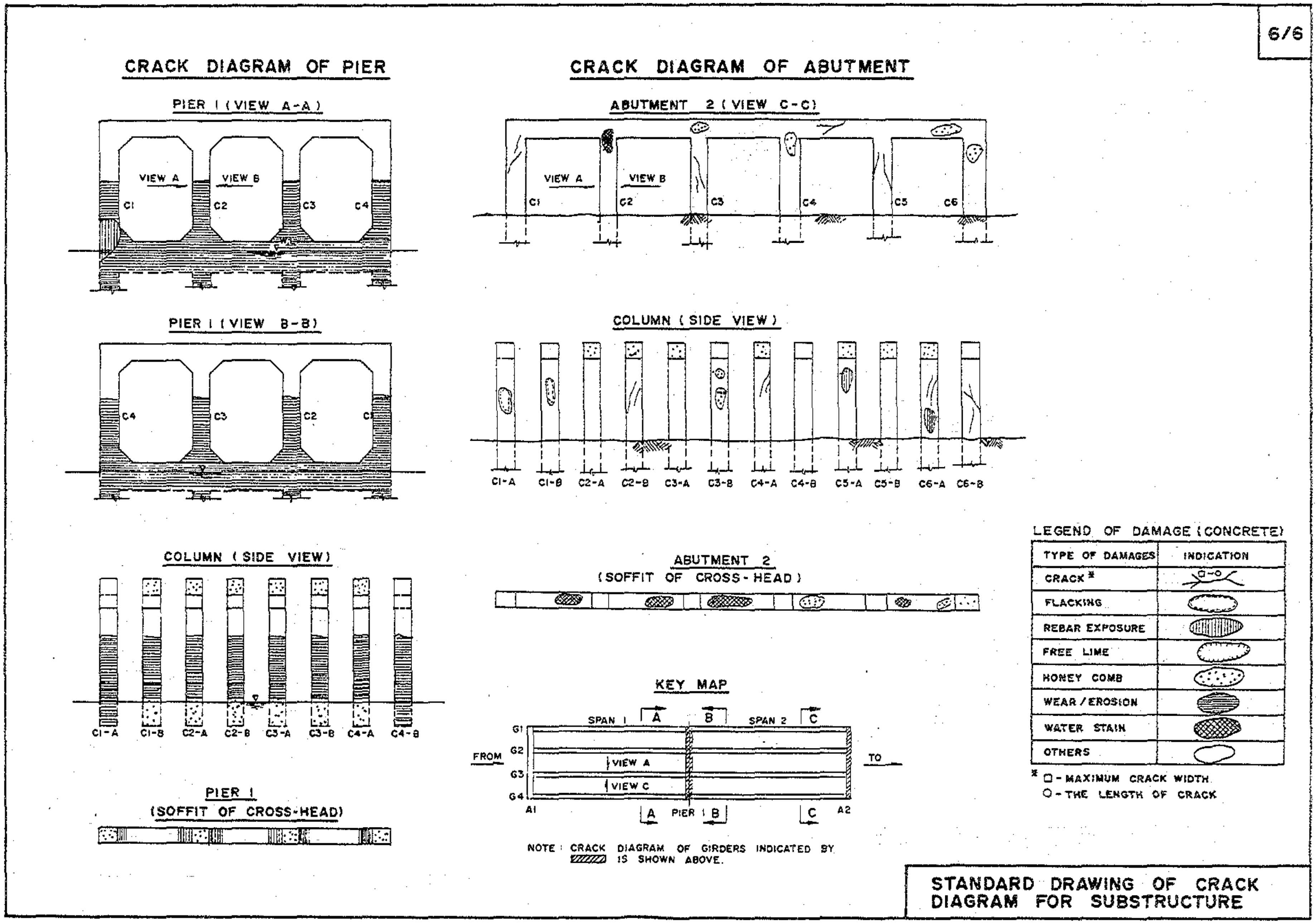

6/6
CRACK DIAGRAM OF PIER
PIER 1 (VIEW A-A)
VIEW A
VIEW B
C1 C2 C3 C4
PIER 1 (VIEW B-B)
C4 C3 C2 C1
COLUMN (SIDE VIEW)
C1-A C1-B C2-A C2-B C3-A C3-B C4-A C4-B
PIER 1
(SOFFIT OF CROSS-HEAD)
CRACK DIAGRAM OF ABUTMENT
ABUTMENT 2 (VIEW C-C)
VIEW A
VIEW B
C1 C2 C3 C4 C5 C6
COLUMN (SIDE VIEW)
C1-A C1-B C2-A C2-B C3-A C3-B C4-A C4-B C5-A C5-B C6-A C6-B
ABUTMENT 2
(SOFFIT OF CROSS-HEAD)
KEY MAP
SPAN 1
SPAN 2
A B C
G1 G2 G3 G4
FROM
TO
VIEW A
VIEW C
A1 PIER 1 A2
NOTE : CRACK DIAGRAM OF GIRDERS INDICATED BY
IS SHOWN ABOVE.
LEGEND OF DAMAGE (CONCRETE)
TYPE OF DAMAGES
INDICATION
CRACK *
FLACKING
REBAR EXPOSURE
FREE LIME
HONEY COMB
WEAR / EROSION
WATER STAIN
OTHERS
* □ - MAXIMUM CRACK WIDTH
○ - THE LENGTH OF CRACK
STANDARD DRAWING OF CRACK
DIAGRAM FOR SUBSTRUCTURE

ANNEX - E

OPERATION MANUALS AND RECORDING FORMS OF SEMI AND NON DESTRUCTIVE EQUIPMENT

TABLE OF CONTENTS

LIST OF TABLES

LIST OF TABLES

METHOD OF SAMPLING FOR SULPHATE AND CHLORIDE TEST

1. Mark the test location using maker pen or spray paint. Avoid taking samples on concrete surface when there is plastering and the test location so chosen should be free from reinforcing bar. (Use the proformeter to locate the rebar).

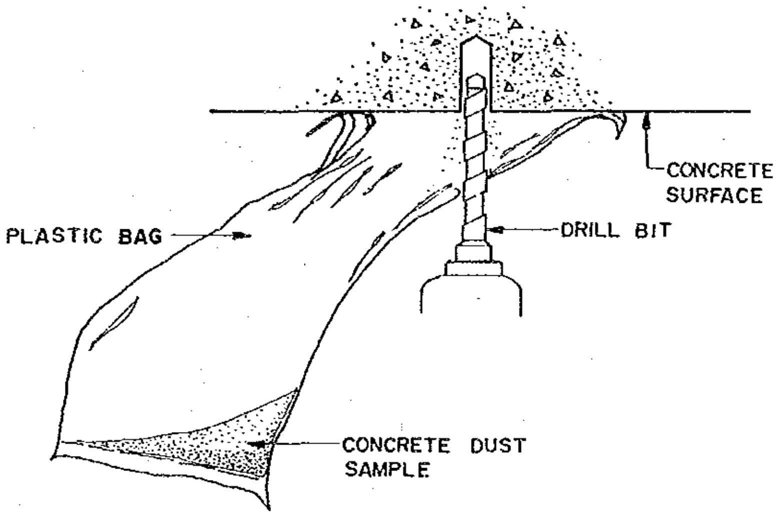
2. Prepare the concrete surface to be tested. Any mud stains, binnacles and salt built up to be removed.

3. 3 number of concrete dust samples shall be taken at various depth on each test location. The depth at which the sample is to be taken shall be 0-20, 21-40, 41-60mm and it is measured from concrete surface.

4. Method of collecting concrete sample is as follows:-

   i - Set the depth marker on the drill bit to the depth required (eg. set depth marker to 20mm for sampling depth of 0-20mm).

   ii - Drill the concrete for a depth of 20mm through a <u>new plastic bag</u> as shown in the diagram below. If drilling is not at the soffit then use the spoon provided to collect the dust sample.



   iii- Transfer the concrete dust samples to a **NEW** plastic beg, seal and marked the sample as follows:-

BRIDGE NO **SAMPLE NO/DEPTH**

iv - Discard the plastic bag used to collect the concrete sample in (i) above. (IT SHOULD NOT BE USED TWICE)

v - Repeat procedure (i) to (iv) above for concrete depth 21-40mm and 41-60mm (but set the depth marker accordingly). Concrete dust sample for each depth shall be stored in a separate plastic beg.

vi - Upon completion of taking 3 samples at each test location, put all 3 begs containing samples at various depth in a bigger plastic bag. Mark the bag as follows:-

BRIDGE NO
SAMPLE NO/MEMBER

vii -Record the test location and sample number in the appropriate sketches and table respectively in the inspection sheets.

5. Send the concrete dust sample to the laboratory for chemical analysis in accordance with BS 1881: Part 124:1988. The interpretation of sulphate and chloride test result is as follows:-

(i) Sulphate test

PERCENTAGE OF SULPHATE	PROBABILITY OF ATTACK
Less than 4%	Probability of sulphate attack is small
Greater than 4%	High probability of sulphate attack

(ii) Chloride test

PERCENTAGE OF CHLORIDE BY WT.OF CEMENT	CONDITION OF CONCRETE AROUND THE REBAR	RISK OF CORROSION
Less than 0.4%	1.Carbonated 2.Uncarbonated & cement with <8% tricalcium aluminate 3.Uncarbonated & cement with >8% tricalcium aluminate	High Moderate Low
0.4% to 1%	1.As above 2.As above 3.As above	High High Moderate
Greater than 1.0%	All cases	High

RECORD OF SAMPLING FOR CHLORIDE AND SULPHATE TEST

Bridge No :__________________________

Date of survey :__________________________

Inspector :__________________________

Sampling No	LOCATION	
	Chloride Sampling	Sulphate Sampling
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		

METHOD OF CARRYING OUT THE CARBONATION TEST

1. Mark the test location using maker pen or spray paint. Avoid carrying out the test on concrete surface where there is plastering. The test location so chosen should also be free from reinforcing bar. (Use the proformeter to locate the rebar).

2. Make a 50mm deep x 20mm diameter hole in the concrete member to be tested using electric drill.

3. Remove all concrete dust from the hole by blowing it with a rubber blower and clean the remaining dust with the brush provided.

4. Immediately spray the newly exposed concrete surface with 1% phenolphtalein solution which is a colourless solution.

5. See colour changes on the concrete surface. The area which has changed to purple indicates that the concrete is still alkaline, hence has not been carbonated.

6. Measure the depth of concrete in the hole where there is no colour change (to be measured from the surface of the concrete). The depth where there is no colour change indicates that the concrete is not more alkaline ie. it has been carbonated.

7. Place the colour reference indicator and black board (on which the location of the test, member and bridge No. has been written on) adjacent to the test location. Take the photograph of the concrete test area while maintaining measurement equipment in the hole, for record purposes.

8. Record in the inspection sheets provided, the test location, bridge No. and result of carbonation test (Carbonation depth).

9. Carbonation test could also be carried out on concrete member which has been broken out such as in the half-cell test, chloride sampling, sulphate sampling and rebar survey. In such cases spray the phenolphthalein solution on the freshly exposed concrete surface (for detail procedure of the test please Repeat item 3 to 8 above).

CARBONATION DEPTH

Bridge No :______________________________

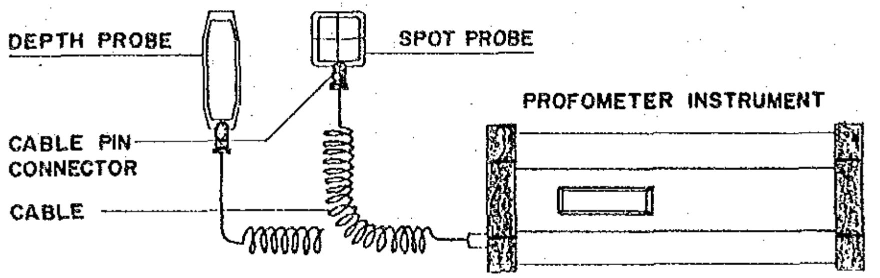
Date of survey :______________________________

Inspector :______________________________

No	TEST LOCATION	MEMBER	CARBONATION DEPTH (mm)	PHOTO NO

METHOD OF CARRYING OUT REBAR LOCATION AND REBAR COVER SURVEY BY USING PROFORMETER

1. The Proformeter is a rebar detector equipment with the setup at shown in the sketch below. (Note; the pin of the probe cable plug should be cleaned from time to time).



2. The method of using proformeter to detect the reinforcement is as follows;

   i) Connect the SPOT PROBE to the instrument by means of cable. Keep the probe away from all metal object.

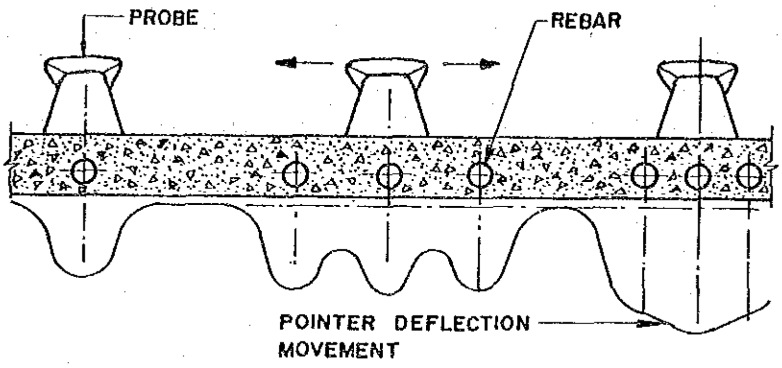
   ii) Switch on the equipment. Hold the probe away from you and be sure that it is well away from any metal object. Then reset the button in proformeter-3 and in case of proformeter-2 turn the tuning knob to bring the pointer to the "∞" mark. Set the rotary selector knob to 8.

   iii) Place the probe on the concrete surface where the test is to be carried out.

   iv) Move the spot probe slowly until a beeping sound is produced by the instrument in case of proformeter-3 and by the maximum value as indicated by the pointer in case of proformeter-2. This indicates that there is rebar directly under the cross marker of the spot probe.

   NOTE:

   There are cases where the rebar may be close together. In such cases a lot of practice and experience in using the proformeter-2 is required. The graph below illustrates how the pointer deflection of the proformeter-2 moves as a function of the interval between the rebar when the probe is moved across the structure perpendicular to the rebar position. In case of proformeter-3 there is less problem. However if the rebar is too close (similar to those shown on the right hand side of the sketch below), breakout survey may be necessary for both version of equipment (an experienced user may be able to use his/her engineering judgment in determining the location and the number of rebar detected).



v) Mark the location of the rebar detected using marker pen or chalk.

vi) Two measurements permit very precise determination of the direction of a rebar by simply joining the two measured points together. Repeat item (iii) to (v) above until all the rebar in the concrete has been detected.

3. The method of using proformeter to detect depth of cover to the reinforcement is as follows;

i) Connect the SPOT PROBE to the instrument by means of cable. Keep the probe away from all metal object.

ii) Switch on the equipment. Hold the probe away from you and be sure that it is well away from any metal object. Then reset the button in proformeter-3 and in case of proformeter-2 turn the tuning knob to bring the pointer to the "∞" mark. Set the rotary selector knob to "16" for proformeter-3 and set it to "D" in case of proformeter-2.

iii) Place the probe on the concrete surface and exactly above the axis of the rebar. Read off the cover depth.

iv) Where the cover detected in less than 10mm, remove the probe and then place the 20mm spacer between the probe and the concrete surface. Read off the depth of cover again. The actual depth of cover shall be obtained by deducting 20mm from the read off value.

v) Where the depth of cover detected is more than 60mm, switch off the instrument and replace the spot probe with DEPTH PROBE. Repeat step (ii) to (iii) above.

4. Diameter of the rebar could not be determined accurately using proformeter. Therefore minimal breakout of the concrete cover is required.

5. Record the diameter, spacing and cover of the rebar in the record form provided.

MEASUREMENT OF RE-BAR SPACING, DIAMETER AND COVERING THICKNESSBY PROFOMETER

Bridge No. :

Date of Survey :

Inspecter :

No.	MEMBER	METHOD	MAIN RE-BAR			STIRRUP OR DISTRIBUTION RE-BAR		
			DIAMETER	SPACING	COVERING	DIAMETER	SPACING	COVERING

INSTRUCTION MANUAL ON THE USE OF
HALF-CELL CORROSION POTENTIAL METER

1. Prepare the concrete surface to be tested as follows:-

   i - Layout a grid pattern of the test and rebar locations at 500mm centers covering the entire area to be tested.(use proformeter to locate the rebar)

   ii - Mark each test location with spray paint.

   iii- Remove all asphalt, waterproofing layers, paint etc. from each test location (the size of test spot shall be about 40mm diameter).

   iv - Completely expose one rebar by breaking out the concrete about 40mm x 60mm in size (the breakout area should be kept minimal).

   v - Wet the test points with the electrical contact solution at least 10 minutes before testing.

2. Mix the electrical contact solution in the plastic bucket. To ensure a standardize potential drop the mixing volume of the electrical contact solution shall be kept constant. The mixing volume shall be as follows:-

Wetting agent	:	100 ml.
Portable water	:	20 l.

3. Prepare the half cell electrode as follows:-

   i - Unscrew the top of a new electrode and fill it to the top with distilled water then screw it back tightly.

   ii - Shake the electrode for 2 to 3 minutes Ensure that excess $CuSO_4$ (copper sulphate) crystal remain at the bottom; if not, add a little bit more $CuSO_4$.

   iii- Tie the sponge to the bottom end of the electrode with a rubber band.

4. Complete the half cell electrical circuit as follows:-

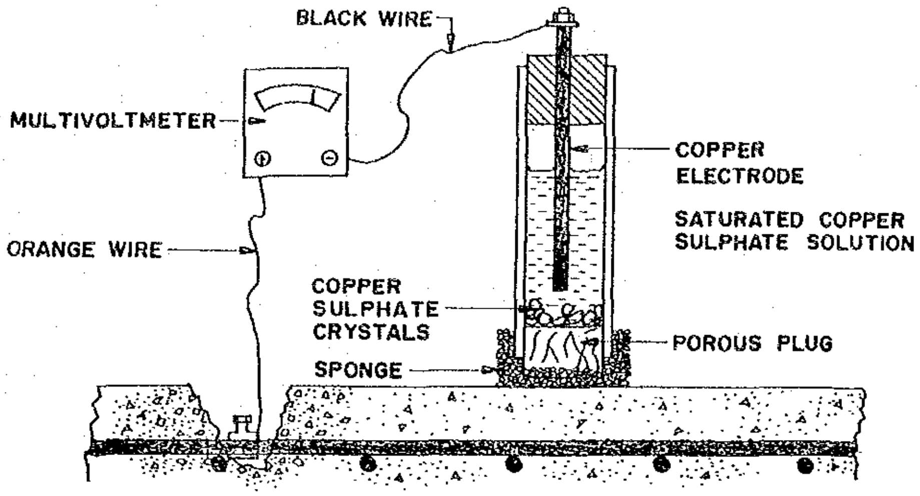
   i - Brush the reinforcing bar.

   ii - Clamp the ORANGE colour electrical wire directly to the reinforcing steel.

   iii- Ensure that the other end of ORANGE colour electrical wire is connected to the positive (RED-centre) terminal of the voltmeter.

iv - Connect one end of the BLACK electrical wire to the negative (black) terminal of the volt meter and the other end to the half-cell.

v - Ensure that proper connection has been made as shown in the diagram below;



5. Method of operating the half cell equipment is as follows:-

i - Switch the voltmeter to 'DC' position.

ii - Place the range switch to "2V" position.

iii- Place the input resistance switch in the 25 M-Ω (million-Ohm) position.

iv - Wet the sponge attached to the electrode.

v - Place the electrode against the prepared location on the concrete surface, adjacent to the marked spot.

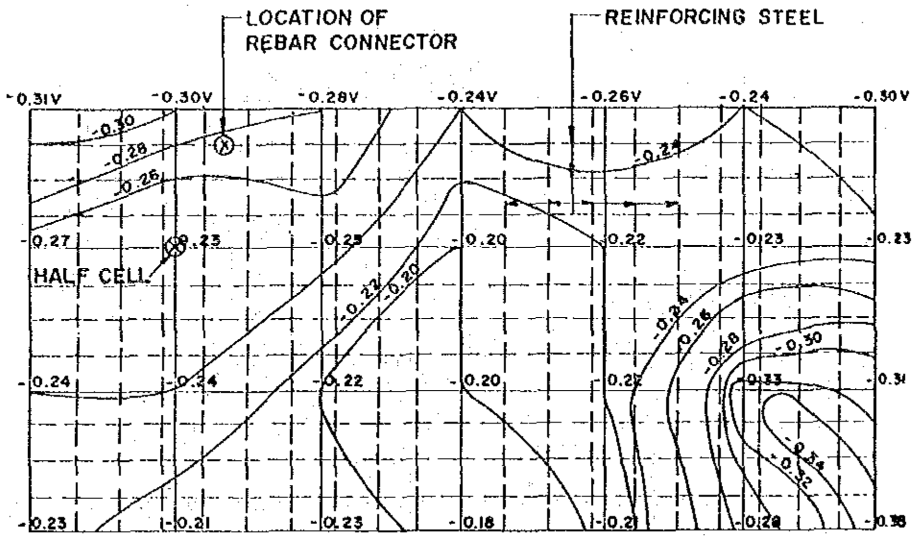
vi - Steady reading between 0 to -0.60 volts should be obtained within 10 second (slight variation in the second digit is normal). All reading must be rounded off to the nearest 0.01V.

vii- If the reading is not <u>negative</u> sign or if the reading is about <u>zero</u> then check for a disconnected or broken test lead AND repeat the test.

viii-Record the potential reading in the form attached.

6. Presentation and interpretation of result of corrosion survey is as follows:-

i) - The half-cell test result shall be presented in the form of an equipotential contour map as shown below;



ii)- The test gives the indication of area of the concrete in which active corrosion is taking place and the probability of corrosion may be interpreted based on the table below:-

ELECTRICAL POTENTIAL	PROBABILITY OF CORROSION
Greater than -0.20 V	Greater than 90% probability where corrosion will NOT occur.
-0.20 V to -0.35 V	- Not Conclusive -
Less than -0.35 V	Greater than 90% probability where active CORROSION is taking place.

ESTIMATION OR RE-BAR CORRODED BY HALF CELL POTENTIOMETER

Bridge No. :
Date of Survey :
Inspecter :

MEMBER AND LOCATION :										
COLUMN	VOLTAGE									
ROW	A	B	C	D	E	F	G	H	I	J
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										
16										
17										
18										
19										
20										

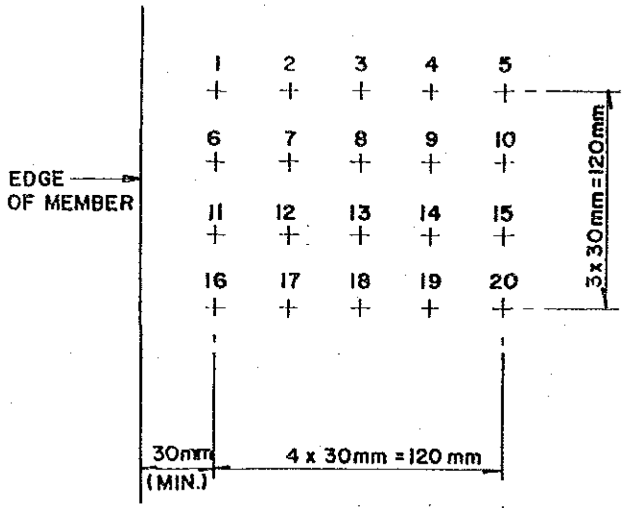
SKETCH

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
A																				
B																				
C																				
D																				
E																				
F																				
G																				
H																				
I																				
J																				

MEASUREMENT METHOD FOR ESTIMATION OF CONCRETE STRENGTH BY SCHMIDT HAMMER

1. Prepare the concrete surface to be tested. Any plasterwork, and coating must be removed. Any uneven surface shall be smoothened with carborundum stone or with power grinder.

2. Number of measurement points per member to be measured shall be 20 points and to be marked on the concrete surface using chalk or marker pen as shown below.



3. Method of operating the schmidt hammer is as follows;

   i- Hold the Schmidt Hammer at right angle to the concrete surface. Press the plunger against the concrete surface to be tested (Careful! do not touch the release button).

   ii- Remove the hammer from the test spot and it will be automatically reset and ready for next test.

   iii- Upon completion of testing at all the 20 points, open the chart housing and read the REBOUND VALUE (R).

   iv- Once the test is completed, lock the plunger by pushing the plunger all the way in while pressing the push-button.

4. Method of determination of concrete strength is as follows;

i- The rebound hammer has been calibrated with the hammer vertical down with the member to be tested positioned horizontally. If the concrete surface is not vertical, The Rebound Value "R" shall be adjusted accordingly using the correction table as shown below.

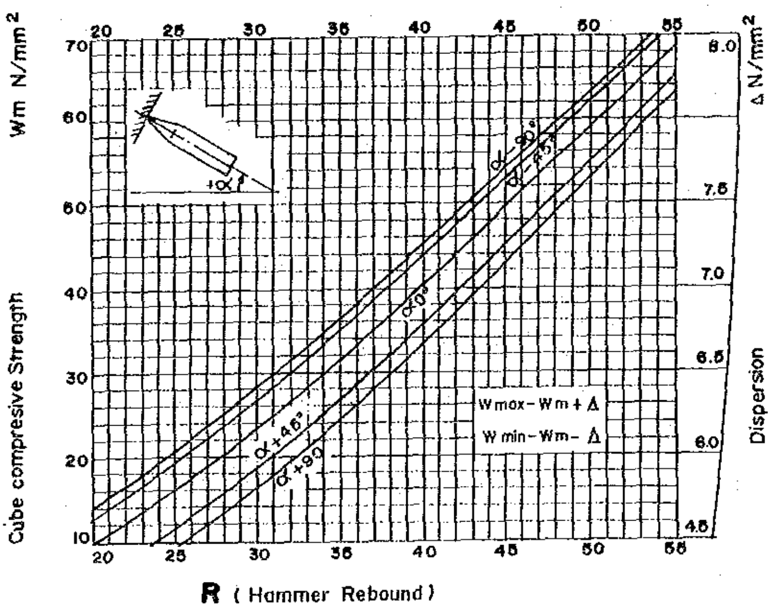
Correction of the Test Hammer Indications

for Non-Horizontal Impacts.

Rebound value $R\alpha$	Correction for inclination angle α			
	upwards		downwards	
	+90°	+45°	-45°	-90°
10	-	-	+2.4	+3.2
20	-5.4	-3.5	+2.5	+3.4
30	-4.7	-3.1	+2.3	+3.1
40	-3.9	-2.6	+2.0	+2.7
50	-3.1	-2.1	+1.6	+2.2
60	-2.3	-1.6	+1.3	+1.7

ii- Averaged out the Rebound Value "R". Any obvious "off shoots" reading are to be eliminated and repeat the impact test if the variation is too wide. Enter Rebound Value (R) in the Recording form attached (similar to UPV recording sheets).

iii- The Cube Compressive strength may be obtained based on the Mean Value of "R" using the Graph below.



ESTIMATION OF CONCRETE STRENGTH BY U.P.V. AND SCHMIDT HAMMER

Bridge No. : ____________________

Date of Survey : ____________________

Inspecter : ____________________

No.	MEMBER	U.P.V.				SCHMIDT HAMMER (R)																					CONCRETE STRENGTH
		30 CM	40 CM	50 CM	AVERAGE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVERAGE	

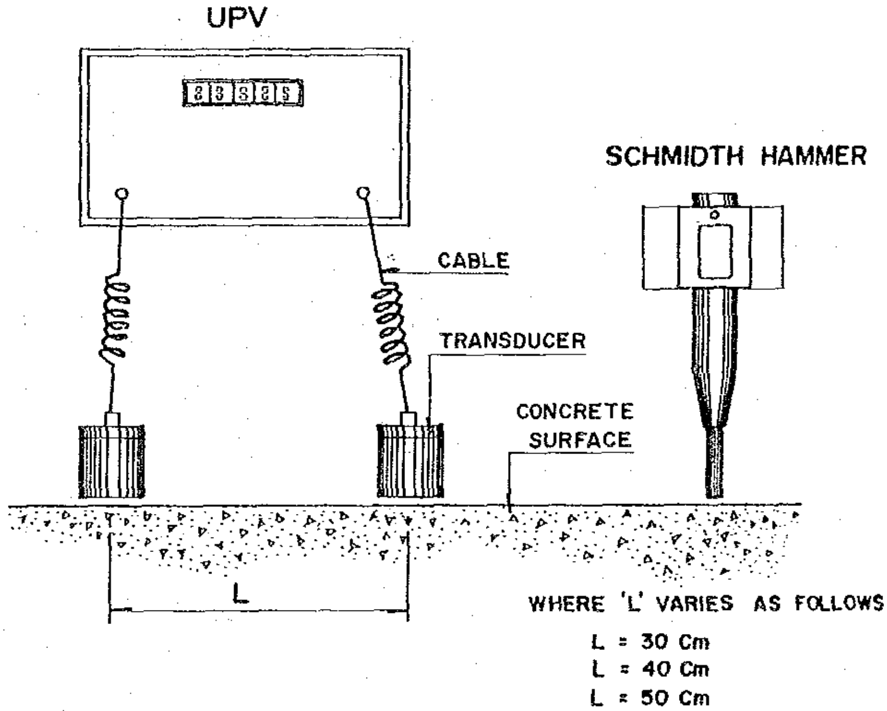
METHOD OF MEASUREMENT FOR ESTIMATION OF CONCRETE STRENGTH BY ULTRASONIC PULSE VELOCITY (UPV) MEASUREMENT

1. Locate all reinforcement bar at the test area using profor-meter and mark its location on the concrete surface.

2. Mark the UPV test location and the selected location to be tested should be free from reinforcing bar.

3. Prepare the concrete surface to be tested. Any binnacles shall be removed and the concrete surface to be smoothened and leveled with power grinder.

4. Number of measurements per member to be measured shall be 3 points and to be marked on the concrete surface using marker pen. The measurement location is as shown below.



5. Adjust the reading of the instrument each time the instrument is used or changes are made to the setup of the equipment such as transducers are interchanged, the use of different transducers and the use of different length of cable. The method of setting up and adjustment of the equipment is as follows:

   a- Apply a thin layer of couplant (petroleum jelly) to the transducers b- Press transducers against both ends of the reference bar.

c- Adjust the zero reading of the instrument accordingly.

6. Method of operating UPV instrument is as follows:

i- Apply a thin layer of couplant (petroleum jelly) to the transducers.

ii- Place the transmitting and receiving transducers on the concrete surface firmly at a fixed distance as shown in the sketch above.

iii- Read directly on digital display screen of the UPV the transmission time (UPV meter reading)

iv- Plot the transmission time (UPV meter reading) on the graph paper.

v- Draw the best straight line through the points on the graph plotted in (iv) above. If the points indicates discontinuity, it indicates that there is defects such as crack, hence result is not reliable. In such instances UPV test has to be carried out on another nearby location.

vi- Record the UPV meter reading in the recording form attached.

7. Method of determination of concrete strength is as follows:

i- Obtain the Schmidt Hammer reading (R)

ii- The concrete strength could then be calculated using the following formula:

a- In cases where the concrete cement ratio is unknown:-

Concrete Strength (Kg/cm²) = 11.8 R + 129 V_{pc} - 642

b- In cases where the concrete cement ratio is known:-

Concrete Strength (Kg/cm2) = 9.2 R + 74 V_{pc}- 4.3 (W/C) - 74

Where; R = Rebound Value of Schmidt Hammer

V_{pc} = Ultrasonic Pulse Velocity (Km/s)

$$= \frac{\text{Distance of UPV probe "L"}}{\text{(Transmission Time)}}$$

ESTIMATION OF CONCRETE STRENGTH BY U.P.V. AND SCHMIDT HAMMER

Bridge No. : ______________________

Date of Survey : ______________________

Inspecter : ______________________

No.	MEMBER	U.P.V.				SCHMIDT HAMMER (R)																					CONCRETE STRENGTH
		30 CM	40 CM	50 CM	AVERAGE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVERAGE	

CONCRETE REPAIR

1. The amount of concrete actually broken off shall always be kept minimal.

2. Ensure that all concrete repair equipment used such as steel trowel, measuring cylinder, mixing bucket, water container etc. is thoroughly cleaned.

3. Ensure that the phenolphthalene used to test for carbonation test has dried up. Use wire brush to clean the steel and any loose concrete material shall also be removed.

4. Mix a small amount of Armatec 108 based on the mixing proportion by volume as follows;

Liquid (Component A)	:	1.0 Part
Grey Powder (Component B)	:	3.0 Part.

5. Brush Armatec around the steel to a thickness of about 1mm. and all excess material shall be discarded (remember that it has a pot life of only 30 minutes).

6. Leave Armatec for 2 - 3 hours to dry.

7. Mix Sikatop 123/3 Bonding Bridge coat using the mixing proportion by volume as follows;

Liquid (Component A)	:	1.0 Part
Grey Powder (Component B)	:	3.0 Part.

8. Wet the breakout area with clean water (do not use river water).

9. Apply Bonding Bridge on raw concrete surfaces as well as on any previously patched Sikatop surfaces.

10. Mix thoroughly Sikatop 122HB mortar using the mixing proportion by volume as follows;

Liquid (Component A)	:	1.0 Part
Grey Powder (Component B)	:	4.5 Part.

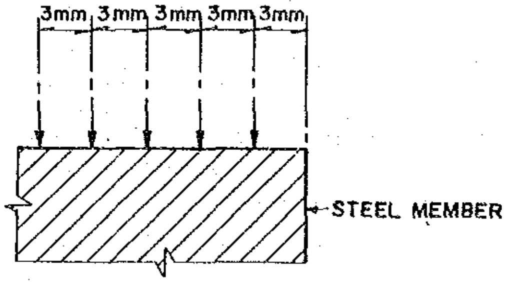
11. Apply a thin layer of Sikatop mortar with trowel (up to 10mm thick at a time) to a maximum of 40mm for vertical surface and 25mm for overhead application. All excess material shall be discarded (The mortar has a pot life of 50 minutes).

12. Wait until Sikatop mortar has set before applying a another layer. Repeat procedure 7 to 11 as required.

MEASUREMENT METHOD FOR A STEEL HARDNESS BY ECHO TIP

1. Mark the test location on the steel member to be tested. At each test location measure and mark 5 test points at centre to centre spacing not less than 3 mm as shown in Fig. E - 1.

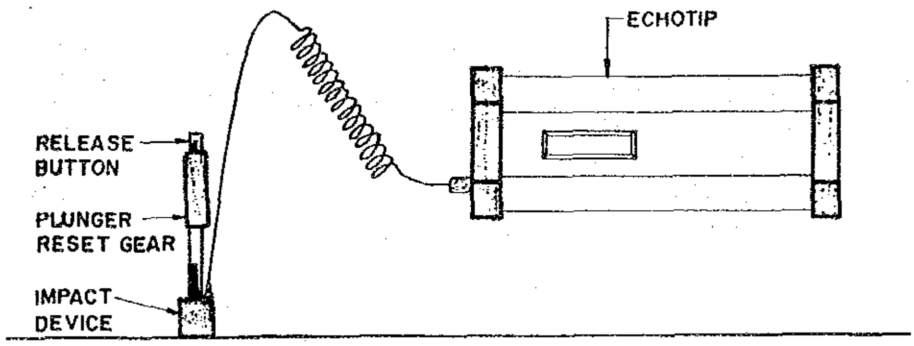
Fig.E-1 Minimum spacing for five testing points



2. Any rust or paint should be removed completely at the location where the measurement is to be taken on the steel member.

3. Method of operating the Echotip equipment is as follows;

   i - Place the impact device firmly at the surface of steel surface to be tested as shown in the sketch below.



   ii - Move the plunger reset gear down until "click" sound is heard. This will reset the instrument.

   iii- Press the impact device release button once.

   iv - Read and record the digital display of the steel hardness value ("L" value) on the Echotip display screen.

v - Repeat step (i) to (iii) for the other 4 test points at each test location.

vi - In general, the echotip equipment which is measuring the steel hardness has been calibrated with impact device at vertical position. If the position of the impact device is not vertically down as shown in the sketch above then the "L" value measured has to be adjusted as follows

Adjustment of "L" value at different position				
MEASURED "L"VALUE	Subtracting the following value from the measured value			
				
300	6	12	20	29
350	6	12	19	27
400	5	11	18	25
450	5	10	17	24
500	5	10	16	22
550	4	9	15	20
600	4	8	14	19
650	4	8	13	18

vii- If five measured "L" value differ by more than 15, please repeat the measurement again after all rust or paint around measurement location has been removed completely.

viii-Repeat step (i) to (vi) above for other test location as required.

4. Convert the "L" values to Brinell hardness by using a conversion table supplied by manufacturer of Echo Tip as shown in Table E-1.

5. Finally the Brinell hardness may also be converted to steel tensile strength by using another conversion table which is provided by JIS (Japanese Industrial Standards) as shown in Table E-2.

Table E-1 Table for converting from "L" value to Brinell hardness

" L " Value	Brinell Hardness
300	80
302	81
304	81
306	82
308	83
310	84
312	85
314	86
316	87
318	88
320	90
322	91
324	92
326	93
328	94
330	95
332	96
334	97
336	98
338	99
340	100
342	101
344	103
346	104
348	105
350	106
352	107
354	108
356	110
358	111
360	112
362	113
364	115
366	116
368	117
370	118
372	120
374	121
376	122
378	123
380	125
382	126
384	127
386	129
388	130

" L " Value	Brinell Hardness
390	131
392	133
394	134
396	136
398	137
400	138
402	140
404	141
406	143
408	144
410	145
412	147
414	148
416	150
418	151
420	153
422	154
424	156
426	159
428	160
430	162
432	164
434	165
436	167
438	168
440	170
442	172
444	173
446	175
448	176
450	178
452	180
454	181
456	183
458	185
460	186
462	188
464	190
468	192
470	193
472	195
474	197
476	199
478	200
480	202

" L " Value	Brinell Hardness
482	204
484	206
490	208
492	209
494	211
496	213
498	215
500	221
502	222
504	224
506	226
508	228
510	230
512	232
514	234
516	236
518	238
520	240
522	242
524	244
526	246
528	248
530	250

" L " Value	Converted Deviation
200	9
300	10
400	13
500	16
600	20
700	
800	
900	

Table E- 2 A table for converting from Brinell hardness to a steel tensile strength

Steel Tensile Strength MPa (Kg/mm2)	Brinell Hardness	
825 (84)	247	247
805 (82)	243	243
795 (81)	238	238
780 (79)	233	233
765 (78)	228	228
730 (75)	219	219
695 (71)	209	209
670 (68)	200	200
635 (65)	190	190
605 (62)	181	181
580 (59)	171	171
545 (56)	162	162
515 (53)	152	152
490 (50)	143	143
455 (46)	133	133
425 (44)	124	124
390 (40)	114	114
-	105	105
-	95	95
-	90	90
-	86	86
-	81	81

ESTIMATION OF STEEL STRENGTH BY ECHO-TIP

Bridge No. : ______________________

Date of Survey : ______________________

Inspecter. : ______________________

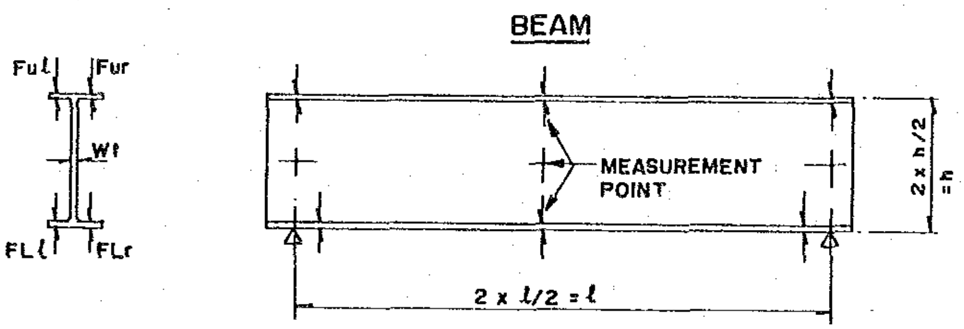
No.	LOCATION	STEEL HARDNESS						STEEL STRENGTH
		1	2	3	4	5	AVERAGE	

STEEL THICKNESS MEASUREMENT USING ULTRASONIC THICKNESS METER

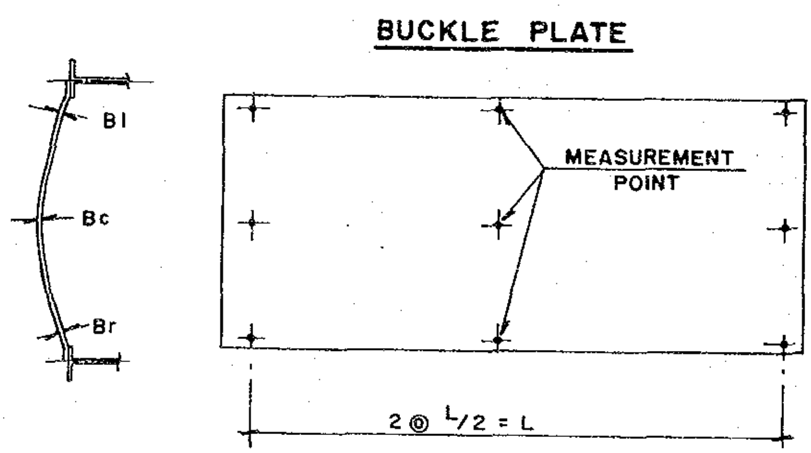
1. Mark the test location using spray paint and prepare the steel surface to be measured. The number of measurement location varies, depending on the type of steel member as shown in (2) below. Any surface rust, scaling and partial peeling of paint should be removed by using wire brush.

2. Only single measurement is required to be made at each test location. The measurement location per member to be tested is as shown below:-

   (i) Steel thickness measurement location for steel beam is as shown in the diagram below;



   (ii) Steel thickness measurement location for steel buckle plate is as shown in the diagram below;



3. Set the velocity of sound setting on top of the ultrasonic meter to 5910 (which is the velocity of sound in iron). DO NOT CHANGE THIS SETTING for the subsequent use of the equipment.

4. Method of operating the ultrasonic steel thickness meter is as follows:-

   i - Connect the probe to the main unit.

   ii - Calibrate the probe (The probe has to be calibrated very frequently). Use soft cloth or paper to clean the surface of the test piece.

   iii- Clean the probe contact surface (ie steel surface to be tested) with soft cloth or paper.

   iv - Press "PW" button on the front face of the main unit (This will switch on the unit).

   v - Coat the surface of the test piece with contact medium (ULTRASONIC PULSE CANNOT TRAVEL THROUGH THE TEST PIECE WITHOUT THE MEDIUM).

   vi - Place the probe against the test piece. Turn the "Zero Adjust Nob" situated on the left hand side of the main unit until the reading 10.00mm. is obtained.

   vii- Now the instrument is ready for measuring the thickness of the steel member.

5. Coat the probe with contact medium and place the probe against the steel to be measured.

6. Read the thickness of the steel directly from the digital display screen on the meter.

7. Record the thickness in the record sheets provided.

STEEL THICKNESS FOR BUCKLE PLATE

BRIDGE NO. : ____________________

DATE OF SURVEY : ____________________

INSPECTOR : ____________________

MEMBER	POINT	O	$\ell/2$	ℓ
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			
	$B\ell$			
	Bc			
	Br			

STEEL THICKNESS FOR BEAM

BRIDGE NO. : ____________________

DATE OF SURVEY : ____________________

INSPECTOR : ____________________

MEMBER	POINT	O	$\ell/2$	ℓ
	FUℓ			
	FUr			
	Wt			
	FLℓ			
	FLr			
	FUℓ			
	FUr			
	Wt			
	FLℓ			
	FLr			
	FUℓ			
	FUr			
	Wt			
	FLℓ			
	FLr			
	FUℓ			
	FUr			
	Wt			
	FLℓ			
	FLr			
	FUℓ			
	FUr			
	Wt			
	FLℓ			
	FLr			

ANNEX - F
DAMAGE RATING CRITERIA

TABLE OF CONTENTS

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LIST OF FIGURES

DAMAGE RATING CRITERIA

1. CORROSION

Corrosion is the most common damage in steel because the main raw material is iron which is easily oxidized in the natural environment. Although it is a progressive damage and is not difficult to detect, it is important to prevent its progress during maintenance.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Section loss	Rust on the surface
	e.g	The surface of steel material has expanded due to corrosion or sectional part is reduced due to the disappearance of the corroded part.	Rust pitting on the surface.
Range (Z)	Grade	Widely	Locally
	e.g	Corrosion or rust is expanding all over the member.	Corrosion or rust exist only locally within the water leaking area.

- RATING

(Y)	(Z)	Secondary Member	Main Member
Major	Major	4	4
	Minor	3	4
Minor	Major	3	4
	Minor	2	3

2. CRACKS

Most cracks originate around the connection points. These points should be paid careful attention during inspection.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Detected	-
	e.g	-	-

- **RATING**

Y	All Members
Major	4

4. FALLING OFF (BOLTS, NUTS & RIVETS) - LOOSE CONNECTIONS

Connections generally with high tension bolts and rivets and in particular, bearing anchor bolts are mainly subject to this problem and need checking.

		Effects on load bearing capacity and durability	
		Major	Minor
Range (Z)	Grade	Connection :Many bearing rollers, etc:fallen off	Connection: a few bearing rollers, etc. Just before falling off.
	e.g	Falling off of 2 pieces of or more per splice part. Falling off of bearing roller etc.	Falling off of 1 piece per connection part.

- **RATING**

(Z)	All Members
Major	4
Minor	3

5. RUPTURE

It is a damage often detected on attached structures such as railings, joints, etc. but as it may also be detected on superstructures and substructures, all steel materials shall be subject to this checking.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Detected	-
	e.g	Rupture caused by corrosion or collision	

- **RATING**

(Y)	All Members
Major	4

6. **PAINT DETERIORATION**

The degree of paint deterioration is expressed in terms of 'depth', the condition of paint flaking i.e. peeling is regarded as a serious effect (Major) and that of colour changes to a lesser extent (Minor). Also, the range of deterioration is expressed by 'extension', if the condition is extended to all the part, it is regarded as a serious effect (Major) and if it is limited to a local area of the part, not so serious effect (Minor).

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Paint peeling off	Paint colour changed
	e.g	Paint of girder , railings etc. peel off	Paint of girder, railings etc.changed due to deterioration
Range (Z)	Grade	Widely	Locally
	e.g	Paint deterioration extends all over the steel part.	Paint deterioration exist only locally on steel part.

- **RATING**

(Y)	(Z)	All Members
Major	Major	3
	Minor	2
Minor	Major	2
	Minor	1

7. **CRACKS**

This is the most common damage to concrete materials and there are various causes such as design, construction, materials and environment. As these causes are very important for the evaluation of the level of damage, the judgment shall be done at first in the column 'position or pattern' (X) with consideration to location of cracks and its type, with reference to typical cracks shown in the figures attached.

		Effects on load bearing capacity and durability	
		Major	Minor
Position or Pattern (X)	Grade	Critical	Uncritical
	e.g	Superstructure common to RC and PC:1,2,3,4,5,6,7,8,12 only.PC: 1,2,3,4, Substructure : 3,5,6,7,8,9, 10,11,12	Superstructure: common to RC and PC 9,10,11 only. PC : 6,7,8, Substructure:1,2,4
Depth (Y)	Grade	Wide	Hair Line
	e.g	RC Structure: 0.3mm or more PC Structure: 0.2mm or more	RC structure:less than 0.2mm PC structure:less than 0.1mm
Range (Z)	Grade	Interval	Interval
	e.g	Less than 50cm	50 cm or more

- **RATING**

X	Y	Z	Main Member	Secondary Member
Major	Major	Major Minor	5 4	4 3
	Minor	Major Minor	3 3	2 2
Major	Major	Major Minor	4 3	4 3
	Minor	Major Minor	2 1	2 1

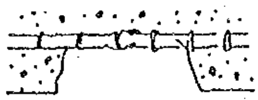
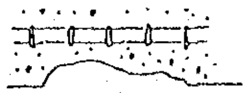
Fig. F-1 Location & Type of Typical Cracks in RC/PC superstructures

Fig. F-2 Location & Type of Typical Cracks in PC superstructures

Fig. F-3 Location & Type of Typical Cracks in substructures

8. FLAKING AND REBAR EXPOSURE

In the case of rebar exposure due to flaking of concrete or insufficient cover is considered as major and in the case of only flaking as minor. The grade of extension of damage is expressed in terms of the quantity, separately for superstructure and substructure and this can be considered as a standard.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Rebar is exposed	Flaking only
	e.g		
Range (Z)	Grade	Damage on large area	Damage on small area
	e.g	Substructure: 1.0 sq.m or more Superstructure: 0.1 sq.m or more	Substructure Less than 1 sq.m Superstructure: Less than 0.1 sq.m

- **RATING**

(Y)	(Z)	Main Member	Secondary Member
Major	Major	4	3
	Minor	3	2
Minor	Major	3	2
	Minor	2	2

9. **FREE LIME**

It refers to the phenomena in which the lime contained in concrete oozes out through cracks, etc., by the action of rainwater which has soaked into the concrete. A similar substance also oozes out from alkali aggregate reactions and it is difficult to distinguish visually one from the other. For distinguishing between these two substances at the site, hydrochloric acid is put on to the substances, the substance oozed out by alkali aggregate reaction (silica) produces hydrogen and leaves a silica deposit on the surface. The grading by extension of damage is the same as that for the case of flaking and rebar exposure.

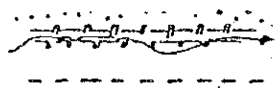
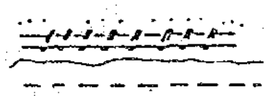
		Effects on load bearing capacity and durability	
		Major	Minor
Range (Z)	Grade	Damage on large area	Damage on small area
	e.g.	Substructure $1m^2$ or more Superstructure: $0.1m^2$ or more	Substructure: Less than $1m^2$ Superstructure: Less than $0.1m^2$

- **RATING**

Z	Main Member	Secondary Member
Major	4	3
Minor	3	2

11. WEAR AND ABRASION

It refers to the condition of damage where the surface concrete of substructures are worn and abraded by water or acid water. The extent of damage, whether it is major or minor, is indicated by rebar exposure.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Rebar is exposed	Rebar is not exposed
	e.g		
Range (Z)	Grade	Damage on large area	Damage on small area
	e.g	Substructure: 1.0 sq.m or more Superstructure: 0.1 sq.m or more	Substructure Less than 1 sq.m Superstructure: Less than 0.1 sq.m

- **RATING**

Y	Z	All Members
Major	Major	4
	Minor	3
Minor	Major	3
	Minor	2

12. SLIPPING OFF (CHIPPING)

It refers to the damage when a piece of concrete chipps off the floor deck (including filling concrete) and which is accompanied by hexagonal cracks. The reason why it is graded 4 is because the necessity to perform emergency repairs depends on the inspector's judgment relative to the position of the chip and peripheral conditions in the same way as for the case of girder rupture.

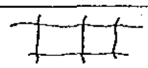
		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Concrete mass chipping off	-
	e.g.	Type of damage which can be seen on concrete floor deck accompanied with hexagonal cracks.	-

- RATING

Y	All Members
Major	4

14. SLAB CRACKS

The importance is judged depending on whether the crack is running in one direction or two directions. The depth is graded depending on the pattern of damage, i.e. the effect of those accompanied with rust liquid is considered as major, those with a water leak as intermediate and those without any sign at water as minor. The width of crack is considered in the same way as that for Item 7 `Cracks' herein. Further, the interval between cracks is indicated here as the extent of damage and the criteria for judging whether it is major or minor depends on the spacing of crack (more than 50 cm is considered minor).

		Effects on load bearing capacity and durability		
		Major	Intermediate	Minor
Position or pattern (X)	Grade	2 directions		1 direction
	e.g			
Depth (Y)	Grade	Cracks with rusty liquid or large width	Cracks with water leak or the width of gap is intermediate	Cracks without any sign at water or the width of gap is small
	e.g	Presence of rusty liquid along the rebar, or the width is 0.3m or more	Presence of water leak and free lime along the cracks, or the width is 0.2 mm or more and less than 0.3mm	RC structure: 0.2 mm
Extension (Z)	Grade	Minimum gap small	-	Minimum gap large
	e.g	Minimum width of gap is less than 50cm	-	Minimum width of gap is 50cm or more

- **RATING**

X	Y	Z	All Members
Major	Major	Major Minor	4 4
	Intermediate	Major Minor	4 3
	Minor	Major Minor	3 2
Minor	Major	Major Minor	3 3
	Intermediate	Major Minor	3 2
	Minor	Major Minor	2 2

15. **ABNORMAL SPACING OF GAP (JOINT)**

It refers to the condition where the spacing is abnormally wide or there is no opening left at all. An abnormal spacing condition is sometimes accompanied by damage to bearings and backwalls, it is necessary to check them at the same time.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Detected	-
		The spacing is so abnormally wide that the face plates are completely disconnected,or there is almost no spacing.	-

- **RATING**

Y	All Members
Major	4

16. DIFFERENCE IN LEVEL (BRIDGE APPROACH & JOINTS)

Shock produced by difference in level or corrugation cannot be ignored beyond a certain value. The value which is slightly more severe than the aimed value for ordinary roads is adopted, the value 15mm - 20mm is targeted for bridges in this criteria. (Relating to this, it has been proved by an experiment conducted by the Civil Research Institute Japan that shock greater than design value is produced when the difference in level reaches 20mm.)

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Considerably rough	Rough
		Unevenness of the direction bridge axis is 20mm or more	Unevenness of the direction of bridge axis is 10 - 20mm

- RATING

Y	All Members
Major	4
Minor	2

17. POT HOLE

Pot holes are bowl-shaped holes of various sizes in the surface course resulting from localized disintegration. As pot-holes, flaking and subsidence affect vehicles (especially two - wheeled vehicles) on the road and often cause problems to safety, particular care must be taken. Holes of less than 10mm - 50mm are considered as minor and those of 50mm or more as major. If the pot hole is 50mm or more damage to the floor deck is also possible, it is necessary therefore to carry out an inspection below the pavement.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Hole is deep	Hole is not so deep
	e.g	Hole is 50mm or more in depth	Hole is 10mm - 50mm in depth
Range	Grade	Diameter of hole is large	Diameter of hole is small
	e.g	Diameter of hole is 20cm or more	Diameter of hole is less than 20cm.

- **RATING**

Y	Z	All Members
Major	Major	4
	Minor	4
Minor	Major	4
	Minor	3

18. PAVEMENT CRACKS

It is unlikely that a crack on the bridge pavement would exceed 5mm in width, however, should such occur damage to the floor deck is possible. It then being necessary, therefore, to carry out an inspection below.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Crack width is large	Crack width is small
		Crack width is 5mm or more	Crack width on paving is less than 5mm

-RATING

Y	All Members
Major	4
Minor	2

19. RUTTING

Rutting is an unevenness on road pavement in a direction perpendicular to the bridge axis. As rutting causes accumulation of water from rainfall and produces problems such as splash and reduction of frictional resistance at high speed, it must be remedied. Unevenness of less than 20mm is not considered as damage, unevenness between 20mm to 30mm is considered as minor and that of 30mm or more as major.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Considerable uneven	Uneven
		Unevenness of direction perpendicular to bridge axis is 30mm or more	Unevenness of direction perpendicular to bridge axis is 20 to 30mm

- **RATING**

Y	All Members
Major	4
Minor	2

21. MATERIAL DETERIORATION

This covers material deterioration due to exhaust gases, salt air, etc on parts other than steel materials, such as on concrete, bearings, expansion joints, etc.

		Effects on load bearing capacity and durability	
		Major	Minor
Range (Z)	Grade	Widely	Locally
	e.g	Material deterioration extends all over the part. (Applicable to concrete, rubber, plastic, etc)	Material deterioration exist only locally on the part. (Applicable to concrete rubber, plastic, etc)

- **RATING**

Z	All Members
Major	4
Minor	3

22. WATER LEAK AND PONDING WATER

This includes water leakage from a damaged part of floor deck, expansion joint, drainage, etc. and ponding water in bearings, paving, etc.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Water leak and water accumulation detected.	-
	e.g	Water is leaking from below the floor deck, main girder, expansion joint, drainage basin jointing point, etc. Water is accumulating in the vicinity of bearings.	-

-RATING

Y	Main Members	Secondary Members
Major	4	2

23. ABNORMAL NOISE (POUNDING)

Abnormal noise produced by any structural defects shall be rectified, especially, in the case of loose bolts. In cases where the generation point cannot be determined, the presence of abnormal noises shall be noted.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Abnormal noise detected	-
	e.g	Abnormal noise is heard from stopper for shifting of movable bearings and girder	-

- RATING

Y	All Members
Major	4

24. ABNORMAL VIBRATION

It is difficult to check abnormal vibration accurately. The presence of abnormal vibration shall be checked by standing on the midspan of the superstructure and on the road in the vicinity of substructure when vehicles pass.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Abnormal vibration detected	-
	e.g	Abnormal vibration or shaking detected on main girder.	-

-RATING

Y	All Members
Major (Detected)	4

25. ABNORMAL DEFLECTION

It is difficult to check abnormal deflections accurately. The presence of abnormal deflections shall be checked by standing on the midspan of superstructure and on the road in the vicinity of substructure when vehicles pass.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Abnormal deflection detected	-
	e.g	Abnormal deflection detected on main girder etc.	-

- RATING

Y	All Members
Major (Detected)	4

26. DEFORMATION

This covers the deformation of steel parts of girders, railings, etc. caused by a traffic impact. As it is difficult to express quantitative affects on the load bearing capacity and durability, the grade of the affect is judged by whether or not deformation of the parts themselves is considerable.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Considerable deformation detected.	Deformation detected
	e.g	Girder, railings, etc. are considerably deformed by vehicles collision etc.	-

RATING

Y	All members
Major	4
Minor	2

27. SEDIMENT ACCUMULATION/VEGETATION

This refers to sediment accumulation with or without vegetation in catch basins and in the vicinity of bearings. Once catch basins become clogged with sediment, the drainage does not work properly causing leaking and accumulation of water. Besides, sediment accumulated in the vicinity of bearings not only accelerates corrosion and deterioration but also obstructs visual detection of serious defects. If sediment accumulation is reported, it shall be quickly cleaned.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Sediment accumulation and vegetation	Sediment accumulation only detected
	e.g	Sediment accumulated in catch basin and in the vicinity of bearing	-

- RATING

Y	All members
Major	3
Minor	2

28. SETTLEMENT

Foundations and bearings shall be subjected to checking. Settlement of the foundation is the easiest damage to check. As for grading of defect, settlement of 25mm or more is considered as major and that of less than 25mm as minor. In the case of a continuous girder, settlement at a point of S = L/2000mm or more is considered as major and that of less than L/2000mm as minor. As the settlement of bearings is difficult to detect it must be estimated based on the condition of anchor bolts and bearing mortar. It there is doubt about the settlement of the bearing, a detailed inspection shall be urgently carried out. (L is span length in meter)

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Settlement of bearing.Considerable settlement of foundation	There is doubt of settlement of bearing.Settlement of foundation.
e.g		Settlement of bearing is 25 mm or more for simple girder and S= greater than L/2000mm for continuous girder in case of bridge approach difference in level > 30mm.	Settlement of bearing is; less than 25mm for simple girder and S=less than L/2000mm for continuous girder. In case of bridge approach difference in level < 30mm

- **RATING**

Y	All members
Major	4
Minor	3

29. **ABNORMAL MOVEMENT**

Foundation and bearings shall be subjected to checking. For foundations, it means sideways movement. For bearings, it means abnormal displacement relative to girder or shoe due to abnormal lateral force, etc.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Abnormal movement of bearings Considerable movement of foundation	Suspected abnormal movement of bearing Movement of foundation.
	e.g	Substructure has: moved considerable due to sideways displacement of the foundation	Substructure has moved due to displacement of the foundation.

- **RATING**

Y	All members
Major	4
Minor	3

30. **DIP**

Foundations and bearings shall be subjected to checking. Regarding foundations, it covers the case of an abutment or pier becoming inclined due to displacement to the side or uneven settlement. Regarding bearings, it covers the case of the shoe become abnormally depressed due to an abnormal lateral force, etc.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth	Grade	Bearing has dipped Foundation has dipped considerably	Suspected dipping of bearing and foundation.
	e.g	Substructure has dropped considerably due to displacement	Substructure has dipped due to displacement of the side part.

- RATING

Y	All members
Major	4
Minor	3

31. **SCOURING**

This covers the case of the foundation and peripheral river bank being scoured or carried away by flowing water. In particular, for bridges located on upstream reaches of a river, special care must be taken, and checking is very important, especially, after typhoon, torrential rainfall, etc.

Further, in the case of direct foundations, the effect of scouring on the safety of the bridge is greater as compared with that of piled and caisson foundations. When scouring is considerable, in the case of direct foundations shifting of movable bearings is possible and emergency steps must be taken.

		Effects on load bearing capacity and durability	
		Major	Minor
Position or Pattern (X)	Grade	Direction foundation	Piled foundation Caisson foundation
	e.g	-	-
Depth	Grade	Considerable scouring is detected	Scouring is detected
	e.g	Foundation is considerably scoured by flowing water	Foundation is scouring by flowing water

- **RATING**

Y	Z	All Members
Major	Major	4
	Minor	4
Minor	Major	4
	Minor	3

32. **DEFECT**

This covers the case of a concrete girder, railings, rubber bearings, etc. having a partial deficiency caused by car crash or abnormal lateral force.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth	Grade	Considerable deficiency detected	Deficiency detected
	e.g	Girder, pedestrian railings, traffic etc. have considerable deficiency due to vehicle collision, etc.	Girder, pedestrian railings, traffic railings etc. have deficiency due to vehicle collision, etc.

- **RATING**

Y	All members
Major	4
Minor	2

33. **EROSION**

This covers the case of foundation and peripheral river banks being eroded or washed away by flowing water.

In particular for bridges which are located on river bends, special care must be taken.

		Effects on load bearing capacity and durability	
		Major	Minor
Depth (Y)	Grade	Considerable erosion detected	Erosion detected
	e.g	River bank up and down stream of abutment has been heavily eroded	Slightly eroded

- **RATING**

Y	All members
Major	4
Minor	3

FIG. F - 1 LOCATION & TYPE OF TYPICAL CRACKS IN RC/PC STRUCTURE

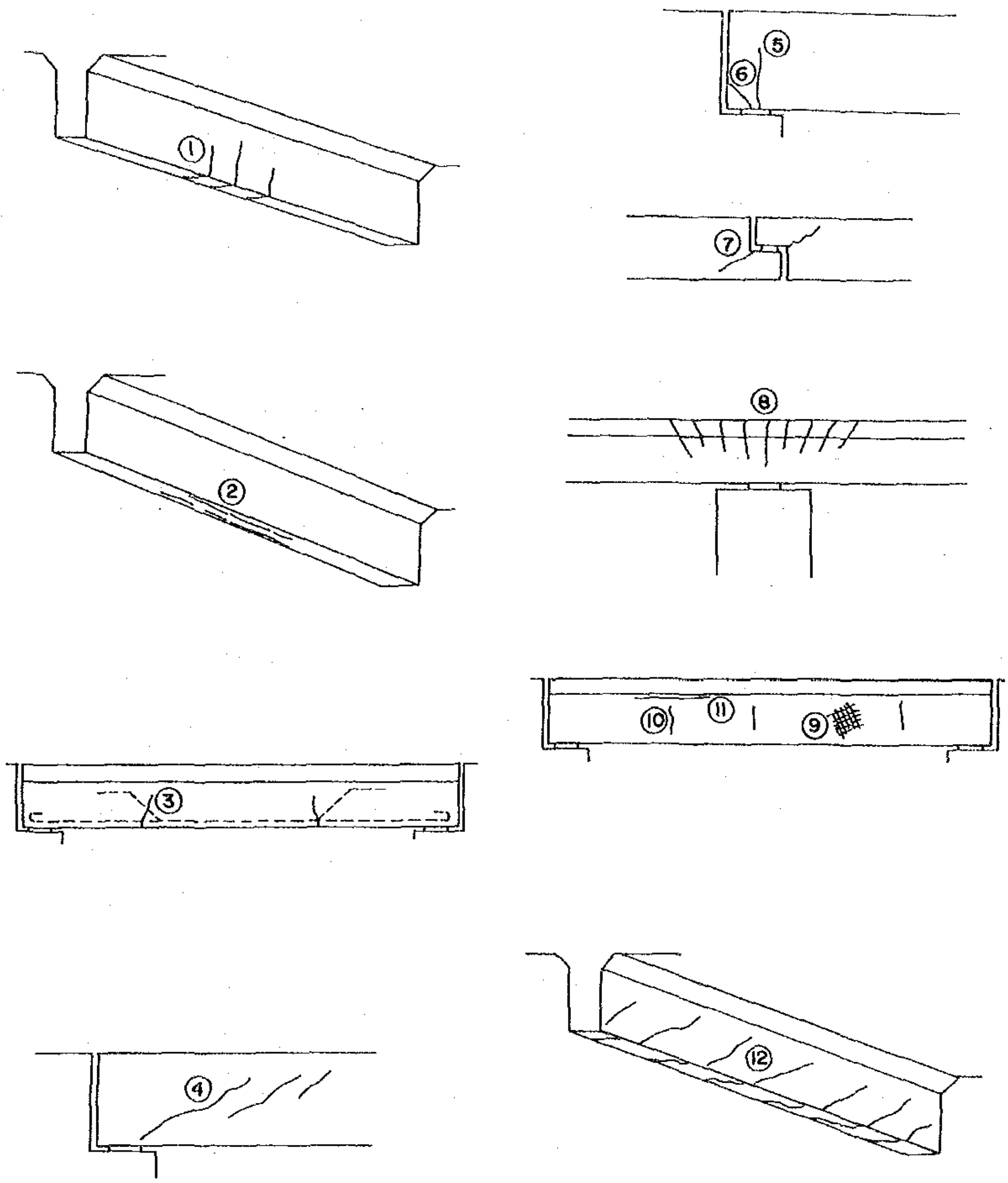


FIG. F - 2 LOCATION & TYPE OF TYPICAL CRACKS IN PC SUPERSTRUCTURE

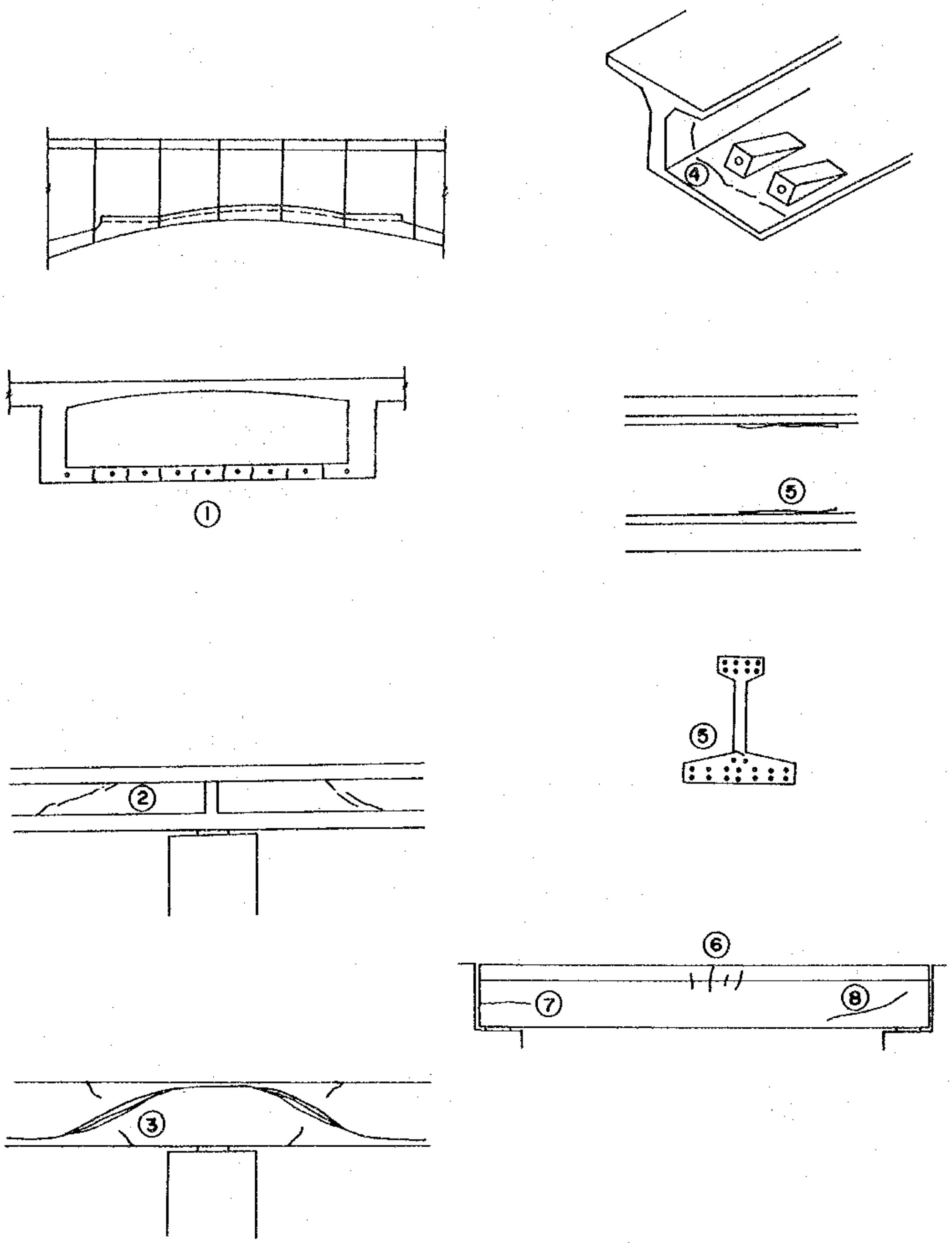
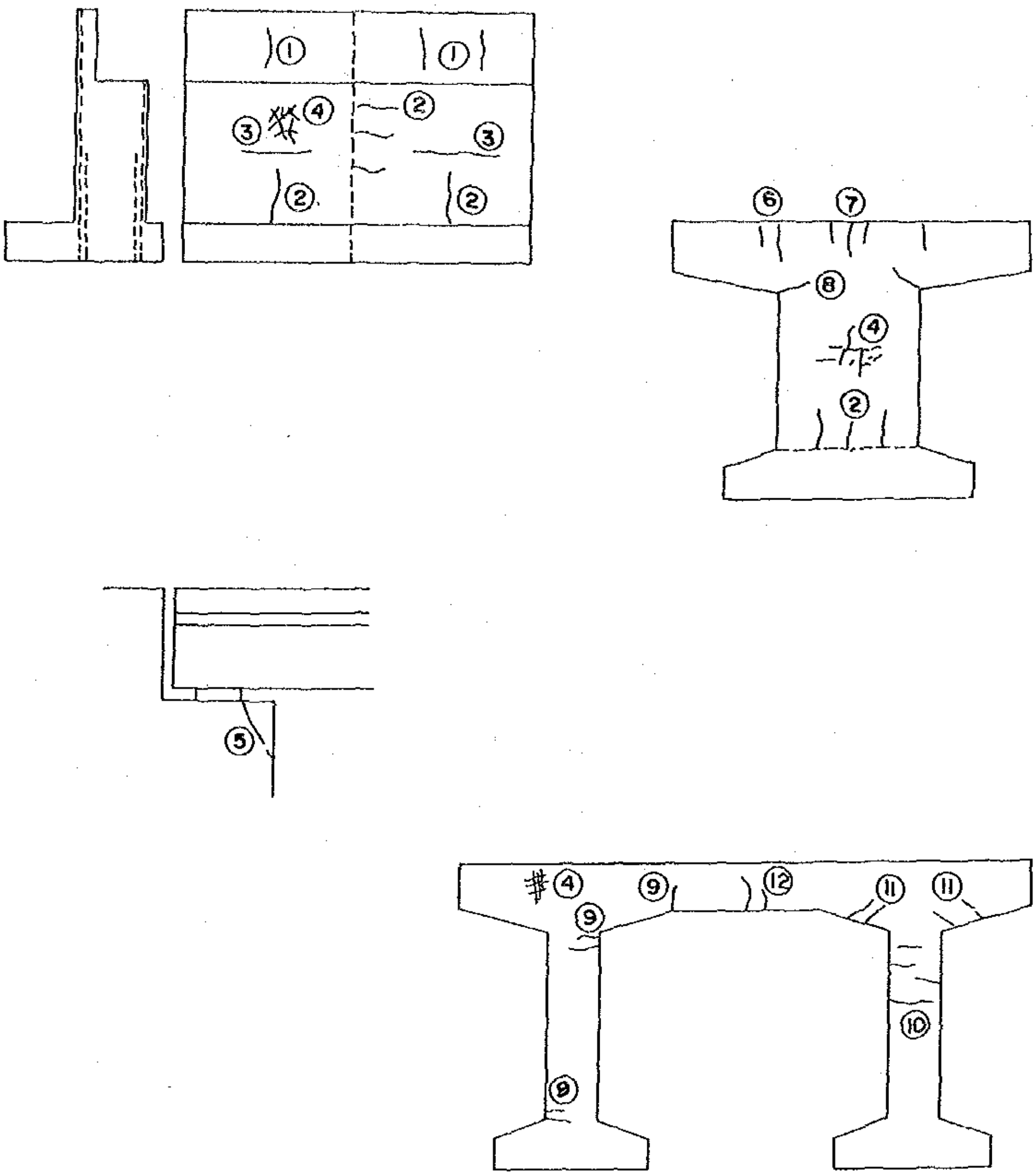


FIG. F - 3 LOCATION & TYPE OF TYPICAL CRACKS IN SUBSTRUCTURE



ANNEX - G
STRUCTURAL ASSESSMENT CRITERIA

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ABBREVIATION

(Alphabetical Order)

Bc	Concrete Bearing
Bs	Steel Bearing
Bt	Transversal Bearing
DFWL	Design Flood Water Level
DID	Drainage Irrigation Department
JKR	Jabatan Kerja Raya
KEL	Knife-Edge Load
LTAL	Long Term Axle Load
OECD	Organization for Economic Cooperation and Development
UDL	Uniformly Distributed Load

STRUCTURAL ASSESSMENT CRITERIA

1. General

Malaysia was once ruled by the British and as a result, bridges in Malaysia have traditionally been designed to British Standards. Throughout the years various revisions of the British Standard and modifications on the application of the standard to suit Malaysian conditions have been carried out resulting in bridges being designed to various loading and design specification. Even today various standards have been used in bridge design, JKR Bridge Design Manual for example, adopted BS153 as its design standard while the current applicable British Standard is BS 5400 which is used by some bridge designers in Malaysia. For the purpose of this study the design criteria to be applied are based on JKR bridge design practice except where the specification is not clear then The Japanese Bridge Design Specification will suffice. The design criteria covers the following aspect of design:-

- Geometric design standard
- Bridge clearance
- Bridge width
- Bridge loading
- Design method
- Material and allowable stress
- Superstructure design
- Substructure design
- Applicable bridge design standard

2. Geometric Design Standard

The geometric design standard to be applied is based on the JKR "ARAHAN TEKNIK (JALAN) 8/86". The summary of the design standard is as follows:-

- Design speed.....................	70 - 100 Km/Hr
- Lane width for 2-lane............	3.5m
- Shoulder (general area)	3.0m
(mountainous area).....	1.5m
- Sidewalk.........................	2.0m
- Vertical Clearance (over road)..	5.0m
(over rail)..	6.5m
- Crossfall........................	2.5%
- Superelevation rates(max)........	0.10 m/m
- Horizontal radius (min.).........	465m
- Vertical Gradient................	6%

3. Bridge Width

For the purpose of detailed assessment work, design standard of R5 road is applied in principle. However the width of the bridge depends on the availability of side walk. The difference between two types of bridge width requirements is shown in Figure G-1 and G-2 below;

Figure G-1 Normal Bridge Cross-Section

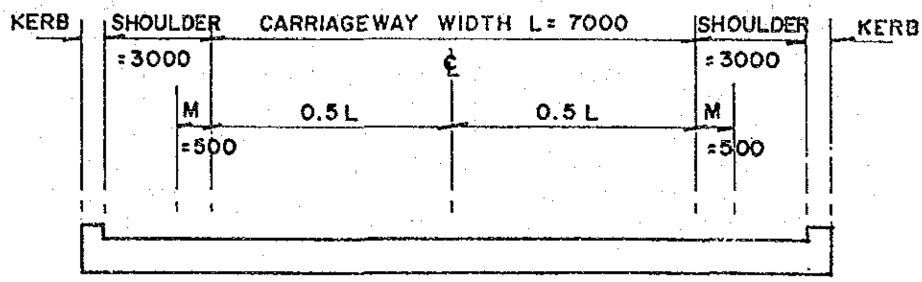
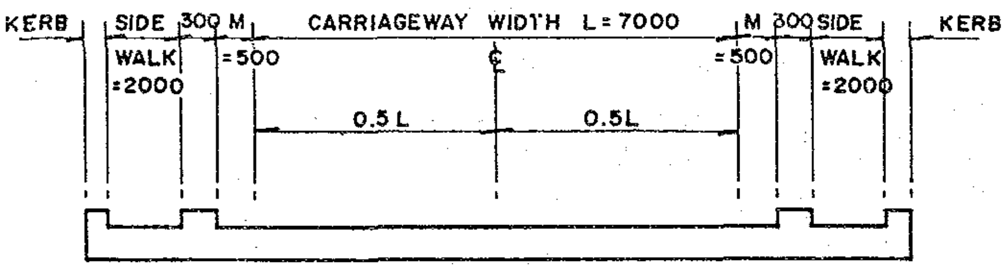


Figure G-2 Cross-Section of a Bridge With Sidewalk



4. Free Board

Hydraulic analysis shall be carried out based on DID Hydrological procedures (either Hp No.5, Hp No.11 or Hp No.4). The free board requirement is not clearly stated in the JKR bridge Design Manual, thus the recommendation given in the Japanese bridge design specification is adopted.The soffit of the bridge deck shall be designed such that it is above the designed flood level with a free board as tabulated in Table G-1.

Table G-1 Free Board For Bridge Over River

Free Board	Size of River	Design flood flow Qf (cumec)
0.50m	Small river	$Qf < 500$
1.00m	Medium river	$500 < Qf \leq 2000$
1.50m	Big river	$Qf > 2000$

5. Bridge Loading

Loads acting on the bridge structure include Dead Load, Live Load, Load due to centrifugal force, Tractive/Braking Force, collision load on bridge parapet, collision load on bridge support, Wind Load, Load due to creep, shrinkage and temperature; buoyancy or uplift force and Forces of Stream Current and Debris. In the detailed assessment the following type of bridge loadings shall be considered:

+ Dead Loads.
+ Primary Live Loads.
+ Tractive/Braking force.
+ Centrifugal force .
+ Collision load on bridge support.
+ Collision load on bridge parapet.
+ Pedestrian load (sidewalk loading).
+ Load due to temperature.
+ Forces due to stream current, debris and floating log.
+ Forces due to earth pressure.

(1) Dead Loads

The unit weight of bridge construction material as given in Table G-2 below may be used for calculation of the dead load:-

Table G-2 The Unit Weight of Bridge Construction Material

Material	Unit Weight (kN/cu.m)
Reinforced Concrete	25
Prestressed Concrete	25
Asphalt Pavement	23
Steel or Cast Steel	77
Cast Iron	71
Alluminium Alloys	28
Timber	8
Stone masonry	27
Bituminous water proofing material	11
Compacted sand, earth or gravel	19
Loose sand, earth or gravel	16

The unit weight of ancillary bridge construction material as given in Table G-3 below may be used for calculation of superimposed dead load:-

Table G-3 The Unit Weight of Ancillary Bridge Construction Material

Material	Unit Weight
100mm nom.dia. water main	0.24 (kN/m)
150mm nom.dia. water main	0.46 (kN/m)
200mm nom.dia. water main	0.73 (kN/m)
250mm nom.dia. water main	1.13 (kN/m)
300mm nom.dia. water main	1.47 (kN/m)
380mm nom.dia. water main	2.08 (kN/m)
10.0m high Lamp Post	1.31 (kN)
12.0m high Lamp Post	1.71 (kN)
RC Parapet + Handrail	7.32 (kN/m)
Std.Kerb + Handrail	4.21 (kN/m)
Std.Kerb Divider	1.80 (kN/m)

(2) Primary Live Loads

Live load to be applied shall be LTAL loading which is applied on each notional lane. Details of the application of the LTAL is as follows:-

- **Notional Lanes**

The width of each notional lane is fixed at 2.5m within the carriageway of the structure. Only integer numbers of the notional lanes shall be used. Areas of the carriageway not covered by the notional lanes shall be loaded with the minimum pedestrian loading of 5 kN/m².

- **LTAL Loading**

LTAL Loading consists of a uniformly distributed Load and a Knife-Edge Load combined, or a twin wheel load. The Nominal Uniformly Distributed Load (UDL) to be applied on a 2.5m lane width is as shown in Table G-4 below:-

Table G-4 LTAL Load For Various Loaded Length

Loaded Length L(m)	LTAL (kN/m/Lane)
$L \le 20m$	$w = 176.8 \cdot L^{-0.6}$
$20m < L \le 40m$	$w = (93.6+4.16 \cdot L) \cdot L^{-0.6}$
$40m < L \le 50m$	$w = 260 \cdot L^{-0.6}$

where:
"L" is the Loaded length in meter and "w" is the load intensity in kN per meter of notional lane width.

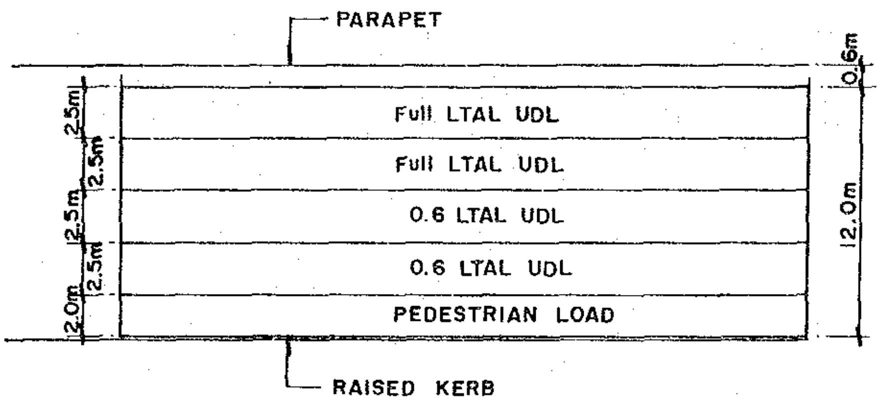
The KEL per notional lane width shall be taken as 100 kN. No dispersal shall be assumed for UDL and KEL.

Twin nominal wheel load alternative to UDL and KEL consist of two 112 kN wheels spaced at 1.8m apart. Each of the wheel is uniformly distributed over a circular or square contact area with effective pressure of 1.1 N/mm² (i.e. 360 mm diameter and 320mm side effectively). The wheel load is dispersed at spread-to-depth ratio of 1 horizontal to 2 vertical through asphalt and 1 horizontal to 1 vertical through structural concrete.

- **Application of LTAL Loading**

The UDL and KEL loads shall be applied on two notional lanes so as to give the worst effects on the structure. The rest of the notional lanes shall be loaded with 0.6 times LTAL UDL and KEL as illustrated in Figure G-3 below. The carriageway width shall be taken as the width between raised Kerbs. In the absence of raised kerbs, it is the width between safety fences, less set back of 0.6m.

Figure G-3 Application LTAL UDL and KEL Load



NOTE: LANE LOADINGS ARE INTERCHANGEABLE FOR THE MOST SEVERE EFFECTS

(3) Load Due To Temperature

Load effect due to temperature difference can generally be ignored in the preliminary design. However the following data may be used if required :-

- The overall bridge temperature shall be taken as 20°C.
- Coefficient of thermal expansion for structural steel and for concrete shall be taken as 12×10^{-6} and 10×10^{-6} respectively.

(4) Centrifugal Load

Centrifugal load on curved bridges shall be applied on any two notional lanes at 50m centres acting radially at the surface of the road and parallel to it. The centrifugal force shall be determined as follows:-

$$F_c = \frac{30000}{(r+150)}$$

where F_c = Centrifugal force (kN)
r = Radius of curvature of lane (m)

Each load F_c shall be either taken as a single load or subdivided into two parts of $^1/_3\ F_c$ and $^2/_3\ F_c$ at 5 m centres longitudinally, whichever gives the lesser effect. A vertical live load of 300 kN, distributed uniformly over the notional lane for a length of 5m shall be considered to be acting together with each F_c. Where the centrifugal load is subdivided, the vertical live load shall be subdivided in the same proportions.

(5) Collision Load on Bridge Support

The nominal collision loads on bridge support on bridges over the highways are given in Table G-5 below together with their direction and point of application.

Table G-5 Collision Load on Bridge Support

Type of load transmitted	Load normal to the carriageway below (kN)	Load parallel to the carriageway below (kN)	Point of application on bridge support.
Load transmitted from guard rail	150	50	Any one bracket attachment point or, for free standing fences, any one point 0.75m above carriage-way level.
Residual load above guard rail	100	100	At the most severe point between 1m and 3m above carriageway level.

Bridge supports shall be capable of resisting the load transmitted from the guard rail applied simultaneously with the residual load above the guard rail. Loads normal to the carriageway are to be considered separately from loads parallel to the carriageway. No other primary live loads are required to be considered on the bridge.

(6) Collision Load on Bridge Parapet

Elements supporting bridge parapets shall be designed to resist loads due to vehicle collisions. The nominal load shall be as given in Table G-6 below:-

Table G-6 Collision Load on Bridge Parapet

Type of parapet	Collision load on parapet	
	High level containment	Normal level containment
Concrete	Moment 25 kNm/m	Moment 12.5 kNm/m
Metal	Force 50 kN	Force 25 kN

For concrete parapet the moment shall be applied uniformly at the parapet base. The transverse collision force on metal parapet shall be applied equally between the number of effective longitudinal members and acting at the centroid of the members. The associated primary live load to be applied shall be twin wheel load of 112 kN each spaced at 1.8m apart.

(7) Sidewalk Loading

Sidewalk loading shall be taken as 5 kN/m² for span length up to 50m.

(8) Tractive/Braking Force

The longitudinal load resulting from traction or braking of vehicles shall be applied at the road surface and parallel to it in one notional lane only. The nominal tractive/braking load shall be taken as follows:-

$T = 8 \cdot L + 200$ (kN); *(but not more than 450 kN)*

where; L is Loaded length (m).

(9) Forces from Stream Current, Debris and Floating Log

- Force due to Stream Current

All piers and other parts of the structure which are subjected to the forces from flowing water, or debris shall be assessed accordingly. The force induced shall be calculated as follows:-

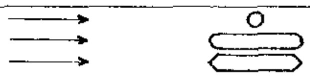
$$P = K \cdot V^2 \cdot A$$

[*The forces induced by flowing water shall be taken to be acting at 0.6H from river bed*]

where :

- P = Pressure (kN).
- V = Maximum current velocity (m/s).
- A = Vertically projected area of pier (m^2).
- H = Depth of water (m).
- K = Constant determined by the shape of the pier as shown in table G-7 below.

Table G-7 Resistance Coefficient of Bridge Pier

Shape of the end of bridge pier facing the stream	Constant
	0·07
	0·04
	0·02

- Force due to debris blockade

Where blockage by debris is likely to occur, allowance shall be made for hydrodynamic forces acting on the minimum depth of 1.2m of debris. The length of debris blockage affecting any pier shall be taken as half the sum of the adjacent spans. However, for minor bridges, the debris loadings need not be considered if the free board over the maximum flood level is more than 1.5m. The pressure P , induced by the debris on the pier shall be taken as follows;

$$P = 0.517 \cdot V^2 \quad (kN/m^2)$$

Where V is the approach flow velocity (m/s).

- **Forces due to log impact**

Where floating logs are likely, the force exerted by 10 tonne logs traveling at normal stream velocity shall be assessed. However the force due to log impact shall **not** be applied concurrently with debris force. The force due to log impact shall be calculated as follows;

$$F = 0.1 \cdot W \cdot V \quad (t)$$

Where :
- W = Weight of log (10 Tonne).
- V = Normal stream velocity (m/s).

(10) Forces due to Earth Pressure

Structures which retain earthfills shall be assessed to withstand pressure as given by Rankine's formula. In normal bridge design, because horizontal granular backfill is often used behind abutments, ground water conditions can be ignored. The earth pressure acting on the abutment depends on whether the abutment is movable type or not and also the type of soil. For preliminary design the following formula shall be used:-

- Earth pressure acting on movable walls;

(a) Sandy soil

$$P_a = K_a \cdot \Gamma \cdot h + K_a \cdot q$$

$$P_p = K_p \cdot \Gamma \cdot h + K_p \cdot q$$

(b) Cohesive soil

$$P_a = K_a \cdot \Gamma \cdot h - 2 \cdot c \cdot \sqrt{K_a} + K_a \cdot q$$

$$P_p = K_p \cdot \Gamma \cdot h + 2 \cdot c \cdot \sqrt{K_p} + K_p \cdot q$$

(2) Earth pressure acting on fixed wall;

$$P_a = K_s \cdot \Gamma \cdot h + K_s \cdot q$$

where;
- Γ = Bulk density of earth (kN/m^3).
- P_a = Active earth pressure (kN/m^2).
- P_p = Passive earth pressure (kN/m^2).
- K_a = Coulomb's active earth pressure coefficient.
- K_p = Coulomb's passive earth pressure coefficient.
- K_s = Coefficient of earth pressure at rest.
- h = Height of abutment (m).
- c = Soil cohesion (kN/m^2).
- q = Surcharge (kN/m^2).

The internal angle of friction of granular backfill material behind abutments shall be assumed to be 30°. Live Load surcharge for suitable material properly consolidated shall be taken as 20 kN/m^2.

6. Design Method Applied

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

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

- Almost all old bridges in Malaysia were designed to BS153 which follows the elastic design principal.

- Quality of materials used in these bridges is often inconsistent (i.e. strength variation is very wide)

Thus, it is safe to apply elastic design method for the assessment and the rehabilitation design. However, quality of material and accuracy of design for an independent structure can be controlled properly within very low tolerance. Therefore, it is rational to apply ultimate limit design method 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 JKR Bridge Design Manual, while for ultimate limit design the provisions prescribed in BS5400 shall be applied.

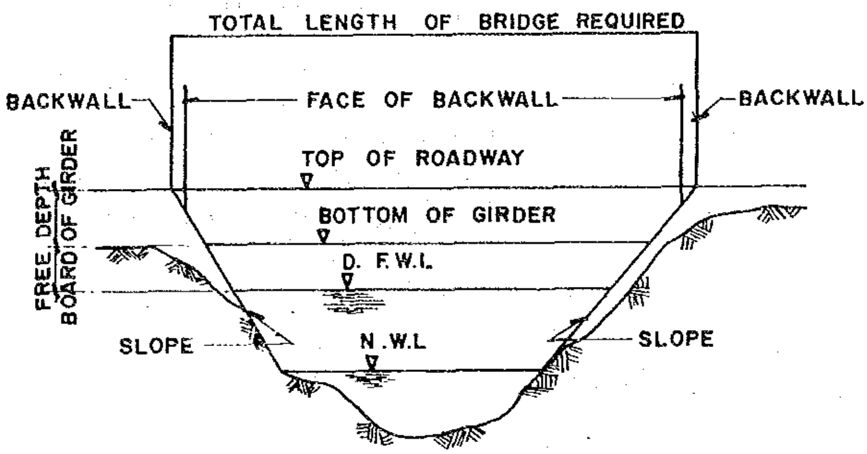
7. Bridge Planning
(Applicable to only total bridge replacement)

- Determination of Bridge Length

The clearances of a bridge controls the bridge's length as indicated in the following. From the intersection of ordinary water level and ground surface as shown in the sketch below, the proposed slopes of protection work follow the slope of the bank as close as possible, having in mind not to constrict the area of the water way required. Then the top of roadway elevation shall be determined based on the Design Flood Water Level (DFWL).

The distance between the intersections of the slopes of protection work and the top of roadway elevation represents the length of bridge required, which is the total distance between the back of backwalls. Minor adjustments shall be made, if necessary, to suit the length of the superstructure is of standard type.

Free board under a bridge shall be determined taking into consideration the necessary space needed for river navigational ways and maintenance, etc. The river administrative clearance from the bottom of the bridge girder or beam to design flood water level will vary from 0.5m to 1.5m depending on the size of river.



The design elevation of the soffit of bridge girder shall not be lower than the highest water level plus the free board.

Free board (below the bridge) - For non navigable river; general clearance between D.F.W.L. and the bottom of the lowest member of superstructure shall comply with the requirement stated in Table G-1.

Vertical Clearance (Navigable river); The DID or Marine Department shall be consulted for determining the minimum horizontal and vertical clearances under a bridge before preparing the final design and plans of the proposed bridge.

- **Applicable Bridge Types**

To select the applicable types of superstructure, substructure and foundation, the basic and important factors to be taken into consideration shall include economical construction, stability and safety, construction period and ease of maintenance and operation.

Figure G-4 Applicable Types of Concrete Bridges

TYPE	SPAN LENGTH (m.)	HEIGHT/SPAN
1. R.C SLAB	1.5 – 8	1/20
2. R.C HOLLOW SLAB	5 – 13	1/20
3. R.C – T – BEAM	10 – 20	1/15
4. P.C – HOLLOW (Pretension)	10 – 21	1/14
5. P.C – I – BEAM (Pretension)	5 – 15	1/15
6. P.C – T – BEAM (Pretension)	10 – 21	1/16
7. P.C – I – BEAM (POST)	20 – 36	1/16
8. P.C – T – BEAM (POST)	20 – 46	1/17.5
9. SIMPLE BOX GIRDER	30 – 40	1/20
10. R.C – ARCH	40 – 70	1/6.6

TYPE	SPAN LENGTH (m.)	HEIGHT/SPAN
11. CANTILEVER BOX GIRDER	60 – 240	1/15
12. P.C CABLE STAYED GIRDER	150 – 340	—

Figure G-5 Applicable Types of Steel Bridges

TYPE	SPAN LENGTH (m.)	HEIGHT/SPAN
1. STEEL I BEAM (Non-Comp.)	0 – 10	1/20
2. STEEL I BEAM (Comp.)	10 – 25	1/22
3. SIMPLE PLATE GIRDER	15 – 40	1/17
4. CONTINOUS PLATE GIRDER	25 – 65	1/18
5. SIMPLE COMP. GIRDER	20 – 50	1/18
6. SIMPLE BOX GIRDER	30 – 60	1/22
7. CONTINOUS COMP. GIRDER	40 – 65	1/19
8. CONTINOUS BOX GIRDER	50 – 100	1/23
9. SIMPLE TRUSS	50 – 100	1/8
10. CONTINOUS TRUSS	60 – 150	1/9
11. STABOGEN BRIDGE	60 – 150	1/8.5
12. CABLE STAYED GIRDER	100 – 600	1/100 1/150

Figure G-4 and Figure G-5 show the relationship between the superstructure type and the span length based on the samples of bridges. The following items are fundamental in the selection of superstructure types :

- Reinforced concrete structures are usually selected because of maintenance is considered easy except where special cases require steel structures.

- Reinforced concrete beams and steel I-beam types are applicable for short spans (length 10m to 15m).

- Prestressed concrete girders, and steel plate girder types are applicable for medium spans (length 20m to 50m).

- Prestressed concrete box girder, steel through truss and ranger girder types are to be applied for long spans (length 60m to 150m).

Figure G-6 Applicable Types of Pier

TYPE		HEIGHT(m) 0 10 20 30 40	REMARKS
P-1	COLUMN TYPE	0 15	
P-2	RIGID FRAME TYPE (1 STOREY)	5 15	
P-3	RIGID FRAME TYPE (2 STOREY)	15 25	
P-4	WALL TYPE	10 30	
P-5	WALL TYPE (1 STOREY)	25 40	

Figure G-7 Applicable Types of Abutment

TYPE		HEIGHT(m) 0 10 20 30	REMARKS
A-1	CHAIR TYPE	3	
A-2	GRAVITY TYPE	4	
A-3	SEMI GRAVITY TYPE	4 6	
A-4	INVERSE T-TYPE	6 10	
A-5	BUTTRESSED TYPE	10 15	
A-6	BOX TYPE	10 20	
A-7	SUSTAINING WALL TYPE	10 15	

Figure G-6 and G-7 show the applicable substructure types in accordance with the required structural height of a bridge. The selection of substructure types is based not only on the specified figures but also on the following considerations :

- Reinforced concrete structures.

- The cross section of pier column in the river; circular or elliptical and rectangular shape with no restricted conditions.

- Non sliding of the back fill materials behind abutment structure is considered in the selection in the abutment type to avoid the approach settlement of the approach.

Figure G-8 Applicable Types of Foundation

	TYPE		DEPTH (m.) 0 10 20 30 40 50 60 70 80 90 100	USABLE DIA. (m.)	SOIL CONDITION CLAYEY	SOIL CONDITION SANDY
F-1	SPREAD FOUNDATION		0 10	—	○	○
F-2	R.C. PILE	DRIVEN PILE	5 15 25	0.3 — 0.5	△	△
F-3	P.C. PILE	DRIVEN PILE	12 30 40	0.35 — 0.5	△	△
F-4	STEEL PIPE PILE	DRIVEN PILE	20 60	0.5 — 0.8	○	○
F-5	CAST IN PLACE W/CASING	CAST-IN PLACE PILE	10 30 40	1.0 — 1.2		△
F-6	EARTH AUGER	CAST-IN PLACE PILE	10 30	1.0 — 1.5	○	×
F-7	REVERSE CIRCULATION DRILL	CAST-IN PLACE PILE	25 60 90	1.0 — 1.2	○	×
F-8	SHINSO PILE	CAST-IN PLACE PILE	10 25	2.0 — 5.0	—	—
F-9	OPEN CAISSON	CAISSON	5 55 70	—	—	—
F-10	PHEUMATIC CAISSON	CAISSON	10 30	—	—	—

NOTE :
○ : APPLICABLE
△ : CONSIDERABLE
× : NOT APPLICABLE

Figure G-8 shows the applicable foundation types in accordance with the required effective depth to sustain the load from Supper-structures. The following are considered in selecting the foundation type :

- Possible construction depth is studied in consideration of soil conditions.

- The advantageous type is considered for works above water e.g. reverse circulation drill pile.

- The prefabricated pile types are advantageous when the bearing stratum is within a shallow range.

8. Superstructure Design

In principal, JKR standard design of superstructure shall be applied if applicable.

The design method and manners of the superstructures such as Reinforced Concrete, Prestressed Concrete and Steel Structure shall be based on the provisions prescribed in BS5400.

9. Substructure Design

The present practices of substructure design in Malaysia is based on BS8004. Since foundation design is universal and for practical purpose, Standard Specification of Highway Bridges in Japan for substructure design is adopted in this manual. Thus, followings are presented for reference.

The substructures can be founded on spread footing, caisson or pile. In general the type of foundation could be classified accordance to Table G-8 and G-9 below.

Table G-8 Classification of Spread Footing and Caisson Foundation

Type of Foundation	Ratio of D_f/B
Spread footing	$D_f/B \leq 1/2$
Caisson	$D_f/B > 1/2$

D_f: Effective embedded depth
B : Shorter width of foundation

Table G-9 Classification of Caisson and Pile Foundation

Type of foundation	Pile or Caisson Characteristic
Caisson	$\beta.L \leq 1$
Short pile	$1 < \beta.L \leq 3$
Long pile	$\beta.L > 3$

where;

L = embedded length of caisson or pile (m).
β = characteristic value of caisson or pile $=\sqrt[4]{kD/4EI}$ (m^{-1}).
EI = flexural rigidity of caisson or pile (kNm^2).
D = Diameter of caisson or pile (m).
k = coefficient of horizontal subgrade reaction of caisson or pile (kN/m^3).

note:-

1. 'k' for caisson shall be taken as a mean value from ground surface to the point of ½ depth.

2. 'k' for pile shall be taken as a mean value from ground surface to the point of $^1/_\beta$ depth.

In principle the foundation shall be designed so that it is stable against bearing, overturning and horizontal movement.

(1) Footing Foundation

The depth of footings shall be determined depending on the type and characteristic of the foundation material. In general, for footing not founded on rock, the base of footing should be founded at depth preferably not less than 1.2m below the stream bed for abutment and 1.8m for pier. This preferred minimum depth shall be increased depending on the site condition. For assessment and preliminary design purposes and where subsoil data is not available, the assumed bearing capacity and angle of internal friction for a broad basic soil type shall be as given in the Table G-10 and G-11 respectively.

Table G-10 Allowable Bearing Pressure For Spread Footing

Type of Bearing Material	Consistency	Allowable Bearing Pressure kN/m²	
		Ordinary Range	Recommended for use
Alluvial Soil	Soft Medium Very stiff to hard	0 - 80 100 - 200 200 - 400	50 150 250
Homogeneous inorganic clay, sandy or silty clay	Soft Medium to stiff Very stiff to hard	50 - 80 100 - 300 300 - 500	50 200 300
Fine to Medium Sand	Loose Medium Dense Very Dense	100 - 200 200 - 300 300 - 400	100 200 300
Gravel, gravel-sand mixtures, boulder-gravel mixtures	Loose Medium Dense Very Dense	200 - 300 400 - 600 600 - 800	200 400 700

Table G-11 Angle of Internal Friction For A broad Basic Soil Type

Type of Bearing Material	Angle of friction
Alluvial Soil	25 - 30
Moist Sand	30 - 35
Submerged Sand	25 - 30
Gravel	35 - 40

In the preliminary design of footing, an appropriate safety factor has to be applied. The allowable bearing capacity of the footing shall not be more than 1/3 the ultimate bearing capacity of the ground. The horizontal reaction of the foundation shall not exceed 1/1.5 of the passive resistance of the ground. The spread footing shall have safety factors of 1.5 against sliding. The sliding resistance at the base of the footing shall be obtained as follows:-

$$H_u = C \cdot A + V \cdot \tan \phi$$

where;

H_u = Maximum sliding resistance (t)
C = cohesion of foundation and ground (t/m^2)
ϕ = friction angle between foundation and ground (°)
A = effective loaded area (m^2)
V = vertical load (t), excluding buoyancy

(2) Pile Foundation

Generally the pile should penetrate not less than 3.0m into hard cohesive or dense granular material. In addition to that, for pile bents type pier, the pile should penetrate not less than $^1/_3$ of the total length of pile. The bearing capacity of pile shall be estimated based on the following formula;

$$Ra = \{(Ru - Ws)/n\} + Ws - W$$

where;

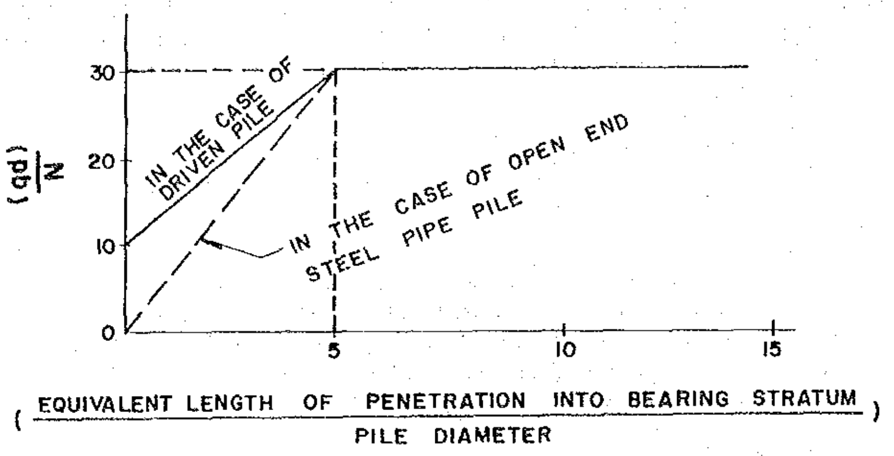
Ra = Allowable load carrying capacity of pile (t).
n = Safety factor (refer to Table G-12).
Ws = Eff. wt of soil replaced by the pile (t).
W = Eff. wt of pile and earth in it (t).
Ru = Ultimate bearing capacity of pile (t) = $q_d A + U \Sigma l_i f_i$
A = Cross-sectional pile tip.
q_d = Ultimate bearing capacity per unit area pile tip.
U = Circumference of the pile.
l_i = Penetration Length of pile/depth of stratum where skin friction is considered
f_i = Maximum skin frictional resistance (t/m^2).

Table G-12 Pile Safety Factor

Type of Pile	Safety Factor(n)
Load Bearing	3
Friction	4

In case of driven piles, the ultimate bearing capacity per unit area at the pile tip may be estimated from figure G-9 below;

Figure G-9 **Chart For Calculating The Ultimate Bearing Capacity of The Ground at Pile Tip Per Unit Area**



In Figure G-9 above 'q_d/N' is given as a function of the ratio of the length of the of the pile embedded into the bearing stratum. The bearing capacity shall be taken as the sum of the end bearing capacity and skin friction capacity. In general, the bearing stratum could be considered to be `good' when N-value for sand and gravel exceeds 30 and for cohesive soil N value is above 20 (ie q_u exceeds 0.4 N/mm²). The following formula shall be used to calculate $\bar{N}$ to be used for estimating the bearing capacity of a driven pile (ie. based on Figure G-9 above).

$$\bar{N} = \frac{(N_1 + N_2)}{2}$$

where;

$\bar{N}$ = N value of the ground for design (but $\leq$ 40).
N_1 = N value of pile tip.
N_2 = Mean N value within the range of 4D upward from pile tip.

(If N value tend to decrease from pile tip downward, the mean value within the range of 2D from the pile tip shall be taken for N_2).

The equivalent penetration length shall be taken as the distance from the pile-tip to the point where the two equal areas surrounded by the N-value distribution curve and the line of $\bar{N}$.

The friction force depends on the type of pile and soil. The maximum friction force in Table G-13 below may be used for preliminary design.

Table G-13 Skin Friction Force

Soil Type	Skin friction force (t/m²)	
	Cast in place	Cast in place driven
Sandy Soil	N/5 (≤ 10)	N/2 (≤ 12)
Cohesive Soil	C or N	c/2 or N/2

Note;

C = *cohesion of the ground surrounding the pile and it may be assumed to be ½ of the unconfined compressive strength of the undisturbed soil sample.*

For preliminary design the N value need not be modified. The minimum distance between the centers of the piles in the outermost row and the edge of the footing may be 1.25 times the pile diameter in the case of driven piles and equal to the pile diameter in the case of cast-in-place concrete piles. The centre to centre spacing of both type of pile shall be 2.5 times the diameter of pile.

(3) Caisson Foundation.

In the preliminary design of caisson foundation, the vertical loads shall be supported at the base of the caisson only. The allowable bearing capacity may be obtained based on the following formula:-

$$q_a = 1/n \cdot (q_d - \gamma_2 \cdot D_f) + \gamma_2 \cdot D_f$$

$$q_d = \alpha \cdot C \cdot N_c + \tfrac{1}{2} \cdot \beta \cdot \gamma_1 \cdot B \cdot N_r + \gamma_2 \cdot D_f \cdot N_q$$

where;

q_a = Allowable bearing capacity (t/m²)
q_d = Ultimate bearing capacity (t/m²)
n = Safety factor = 3
c = Cohesion of the soil at base of caisson (t/m²)
γ_1 = bulk density of ground at base of caisson (t/cu.m)
γ_2 = bulk density of earth surrounding the caisson
α, β= shape factor of the base of caisson as in Table G-14
D_f = effective embedded length
N_c, N_q, N_r = Coefficient of bearing capacity (Fig.G-10)

Figure G-10 Coefficient of Bearing Capacity

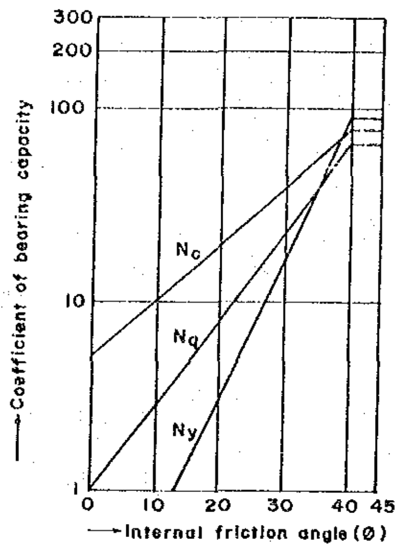


Table G-14 Shape Factor of the Base of Caisson

Shape factor	Shape factor of various shape of caisson			
	Strip	Square	Oval	Circular
α	1.0	1.3	1+ 0.3B/L	1.3
ß	1.0	0.6	1- 0.4B/L	0.6

where;

B = width of the total side Diameter of caisson (m)
L = width of front side of caisson (m)

note; *If B/L > 1 than B/L shall be taken as unity.*

The allowable horizontal bearing capacity of ground shall be similar to footing design.

10. Load Combination

- Allowable design method

Load combination for allowable stress design shall be as specified in BS 153-Part 3B and as summaries in the Table G-15 below:-

Table G-15 Load Combination For Allowable Stress Design

Load Combination	Loading	Incremental coefficient for allowable stresses
1	D + L	1.00
2	D+L+F+S	1.25
3	D+L+CS+S	1.25
4	D+L+CP+S	1.25
5	D+L+CL+S	1.25
6	D+L+BK+S	1.25

where;

D = Dead Load.
L = Live Load.
F = Centrifugal force.
CS = Collision load on bridge support.
CP = Collision load on bridge parapet.
CL = Collision load due to log impact.
BK = Tractive/Breaking force.
S = Stream current debris.

Based on engineering judgement, forces from load combination 2, 3 and 4 are not critical for all bridges in the study. Therefore for the purpose of preliminary design and assessment of bridges in the study, only load combination 1, 5 and 6 will be used.

- **Ultimate Limit Design**

For the purpose of design at Ultimate Limit State (ULS), the load combination given in Table G-16 below shall be considered:-

Table G-16 Load Combination At ULS And Appropriate Partial Factor, γ_{fL}

No	Loading	Load Combination					
		1	2	3	4	5	6
1	D(Concrete) (Steel)	1.15 1.05	1.15 1.05	1.15 1.05	1.15 1.05	1.15 1.05	1.15 1.05
2	SIDL	1.75	1.75	1.75	1.75	1.75	1.75
3	S	1.10	1.10	-	1.10	1.10	1.10
4	L	1.50	1.50/1	-	1.25/2	1.25	1.25
5	F	-	1.50	-	-	-	-
6	CS	-	-	1.25	-	-	-
7	CP	-	-	-	1.25	-	-
8	CL	-	-	-	-	1.25	-
9	BK	-	-	-	-	-	1.25

Note;

/1: Live load to be applied shall be the appropriate live load as described in (4) above.
/2: Live load to be applied shall be the appropriate live load as described in (6) above.
SIDL : Superimposed Dead Load

11. Material And Allowable Stress

(1) Allowable Stress Design

The allowable stresses for reinforced concrete design shall be as specified in BE 1/73 and for steel design shall be as specified in BS 153 : Part 3B.

- **Concrete**

The allowable compressive stresses and allowable shear stress of concrete shall be as given in Table G-17 below.

Table G-17 The Allowable Compressive and Shear Stress of Concrete

Class of Concrete denoted by specified 28 days work cube strength	Permissible Stresses in Concrete				
	Compression		Shear	Bond	
	Direct	Bending		Average	Local
N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
30	7.6	10	0.87	1.00	1.47
25	6.3	8.3	0.80	0.90	1.34
22.5	5.7	7.5	0.72	0.85	1.27
20	5.1	6.7	0.70	0.80	1.20

- **Steel Reinforcement**

The permissible stresses in steel reinforcement shall be as given in Table G-18 below;

Table G-18 The Permissible Stresses in Steel Reinforcement

Type of Stress	Permissible Stresses in rebar (N/mm^2)		
	Mild Steel Bars		All cold work & hot rolled high yield bar
	$\phi \leq 40mm$	$\phi > 40mm$	
Tensile stress other than in shear reinforcement	140	125	230
Tensile stress in shear reinforcement. That is stirrups and main bars, bent up to resist shear	140	125	175
Compressive stress	125	110	175
Range of stress	265	235	325

- **Structural Steel**

The permissible stresses in structural steel shall be as given BS 153:Part 3B which is summaries in Table F-19 below;

Table G-19 The Permissible Stresses in Structural Steel

Steel Grade	Yield Stress (N/mm²)	Permissible Stresses (N/mm²)				
		Bending			Direct/ Axial on effec- tive X- Area.	Shear
		Plate & Hollow section	Rolled section	Plate Girder		
Grade 43	215	140	133	126	129	80
	230	150	142	135	138	85
	245	160	151	144	147	91
	280	183	173	165	168	107
Grade 50	325	212	201	191	191	120
	340	222	210	200	200	126
	355	232	219	209	209	131
Grade 55	400	261	247	235	235	148
	415	271	256	244	244	154
	430	281	265	253	253	159
	450	294	278	265	265	167

(2) Ultimate Limit State Design

- **Concrete**

The design strength of materials for ultimate limit state are expressed in terms of the 'characteristic strength' of the material multiplied by r_m, the partial safety factor for material.

- o Extreme fibre stress in compression, f_c.....$0.67f_{cu}/m$
- o (r_m shall be taken as 1.5)
- o Ultimate Bearing stress, f_b................$0.4f_{cu}$
- o Ultimate shear stress, V_c shall be as follows;

$$V_c = \frac{0.27}{\gamma_m} \left[\frac{100 \cdot A_s}{b_w \cdot d} \right]^{1/3} \cdot f_{cu}^{1/3}$$

where;

A_s = Area of Longitudinal rebar.
b_w = Breadth of web or rib of member.
d = Effective depth of tension rebar.
f_{cu} = Characteristic concrete cube strength.
γ_m = 1.25.

- **Reinforcing Steel**

o The ultimate tensile strength, $f_s = 0.8f_y/m$

o (γ_m shall be taken as 1.15)

o Characteristic strength of reinforcement, f_y is as follows;

Mild steel....................250 N/mm².
High Yield steel...............410 N/mm².

- **Structural Steel**

Nominal yield stress for steel complying with BS4360 is as follows:-

Steel Grade	Nominal Yield stress (N/sq.mm)	
	$t \leq 16mm$	$16mm < t < 40mm$
40	235	225
43	275	265
50	355	345
55	450	430

12. Design Standard

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

ANNEX - H
STANDARD REPORTING FORMS

Standard Reporting Form of Superficial Inspection Results

				Date	
Key No.		State		District	
Bridge Type		Bridge Length	(M)	Bridge Width	(M)
No's of Span		Max.Span Length	(M)	Year Built	

Name of Inspector	
Date of Inspection	

Bridge Member		Rating	Major Damages	Maintenance Work Required
[ ] Beam/ Girder	[ ] Steel [ ] Concrete			
[ ] Deck Slab	[ ] Steel [ ] Concrete			
[ ] Abutment	Concrete			
[ ] Pier	[ ] Steel [ ] Concrete			
[ ] Bearing	Steel			
[ ] Drainage				
[ ] Sidewalk [ ] Kerb	Concrete			
[ ] Railing	[ ] Steel [ ] Concrete			
[ ] Pavement	[ ] Asphalt [ ] Concrete			
[ ] Expansion Joint	[ ] Steel [ ] Rubber			
[ ] River Bank				

Inspector's Comments ;

Estimated Cost for Maintenance Works ;

Detailed Inspection ; [ ] Required [ ] Not Required

Standard Reporting Form of Periodical Inspection Results

				Date	
Key No.		State		District	
Bridge Type		Bridge Length	(M)	Bridge Width	(M)
No's of Span		Max.Span Length	(M)	Year Built	
Name of Inspector					
Date of Inspection					

Bridge Member		Part Rating from Maintenance View Point	Part Rating from Bridge Safety View Point	Maintenance Work Required
[ ] Abutment	Concrete			
[ ] Pier	[ ] Steel [ ] Concrete			
[ ] Bearing	[ ] Steel [ ] Rubber			
[ ] Beam/ Girder	[ ] Steel [ ] Concrete			
[ ] Deck Slab	[ ] Steel [ ] Concrete			
[ ] Pavement	[ ] Asphalt [ ] Concrete		–	
[ ] Wing Wall	Concrete		–	
[ ] Expansion Joint	[ ] Steel [ ] Rubber		–	
[ ] Railing	[ ] Steel [ ] Concrete		–	
[ ] Drainage			–	
[ ] Bank Slope			–	
Overall Rating per the Bridge				
Traffic Capacity on the Bridge		[ ] Adequate [ ] Inadequate		Capacity Year ;
Pedestrian Flow Capacity		[ ] Adequate [ ] Inadequate		
Bridge Opening Capacity		[ ] Adequate [ ] Inadequate		

Inspector's Comments:

Estimated Cost for Maintenance Works ;

Detailed Inspection ; [ ] Required [ ] Not Required

Standard Reporting Form of Detailed Inspection Results

				Date	
Key No.		State		District	
Bridge Type		Bridge Length	(M)	Bridge Width	(M)
No's of Span		Max.Span Length	(M)	Year Built	
Name of Chief Inspector				No's of Drawings Prepared	
Date of Inspection					

Measurement Items	Measurement Results
[ ] Steel Thickness (mm)	[ ] Beam/Girder – Flang Min.; Max.; Average; – Web Min.; Max.; Average; [ ] Buckle Plate Min.; Max.; Average;
[ ] Concrete Cover (mm)	[ ] Beam/Girder [ ] Abutment; [ ] Deck Slab ; [ ] Pier ;
[ ] Pile Length (m)	Location; Length; Location; Length;
[ ] Steel Strength (N/mm2)	[ ] Beam/Girder; [ ] Buckle Plate; [ ] Reinforcing Bar;
[ ] Concrete Strength (N/mm2)	[ ] Beam/Girder; [ ] Abutment; [ ] Deck Slab ; [ ] Pier ;
[ ] Carbonation Depth (mm)	[ ] Beam/Girder; [ ] Abutment; [ ] Deck Slab ; [ ] Pier ;
[ ] Chloride Content (%)	[ ] Beam/Girder; [ ] Abutment; [ ] Deck Slab ; [ ] Pier ;
[ ] Sulphate Content (%)	[ ] Beam/Girder; [ ] Abutment; [ ] Deck Slab ; [ ] Pier ;

Bridge Member		Major Damages	Rehabilitation Work	Estimated cost
[ ] Abutment	[ ] Steel [ ] Concrete			
[ ] Pier	[ ] Steel [ ] Concrete			
[ ] Bearing	[ ] Steel [ ] Rubber			
[ ] Beam/ Girder	[ ] Steel [ ] Concrete			
[ ] Deck Slab	[ ] Steel [ ] Concrete			
[ ] Pavement	[ ] Asphalt [ ] Concrete			
[ ] Wing Wall	Concrete			
[ ] Expansion Joint	[ ] Steel [ ] Rubber			
[ ] Railing	[ ] Steel [ ] Concrete			
[ ] Drainage				
[ ] Bank Slope				
			Total Amount	
Name of Sr. Engineer in Charge ;				

Standard Reporting Form of Maintenance Works Carried out

				Date	
Key No.		State		District	
Bridge Type		Bridge Length	(M)	Bridge Width	(M)
No's of Span		Max.Span Length	(M)	Year Built	

Bridge Member		Outline of the Maintenance Works	Date	Cost
[ ] Beam/ Girder	[ ] Steel [ ] Concrete			
[ ] Deck Slab	[ ] Steel [ ] Concrete			
[ ] Abutment	Concrete			
[ ] Pier	[ ] Steel [ ] Concrete			
[ ] Bearing	[ ] Steel [ ] Rubber			
[ ] Drainage				
[ ] Sidewalk [ ] Kerb	Concrete			
[ ] Railing	[ ] Steel [ ] Concrete			
[ ] Pavement	[ ] Asphalt [ ] Concrete			
[ ] Expansion Joint	[ ] Steel [ ] Rubber			
[ ] River Bank				
Name of Engineer in Charge ;			Total Amount	

Standard Reporting Form of Rehabilitation Works Carried Out

				Date	
Key No.		State		District	
Bridge Type		Bridge Length	(M)	Bridge Width	(M)
No's of Span		Max.Span Length	(M)	Year Built	

Name of contractor	
Construction Period	
Contract Amount	
Additional cost due to Variations	
Final Contract Amount	
Number of As Built Drawing	

Classification	Outline of Rehabilitation Works	Work Items	Quantities of Works	Unit	Unit Price	Amount
Structural Rehabilitation						
Functional Rehabilitation						
Hydraulic Rehabilitation						

Name of Sr. Engineer in Charge ;

Total Amount :

ANNEX - I
ECONOMIC EVALUATION METHOD

ECONOMIC EVALUATION METHOD

I.1 Economic Evaluation Method

(1) General

The purpose of economic evaluation is to evaluate economic viability and to determine the priority of the proposed maintenance projects for bridges.

The following procedure shall be taken as shown in Fig. I-1.

1) Select a traffic count station corresponding to each bridge from the traffic census data.

2) Estimate the future traffic volume of each bridge using traffic growth rates included in the above.

3) List the possible benefits by bridge in the light of proposed rehabilitation works.

4) Formulate an evaluation model to quantify benefits included in the above.

5) Calculate economic benefits using future traffic volumes and vehicle operating costs.

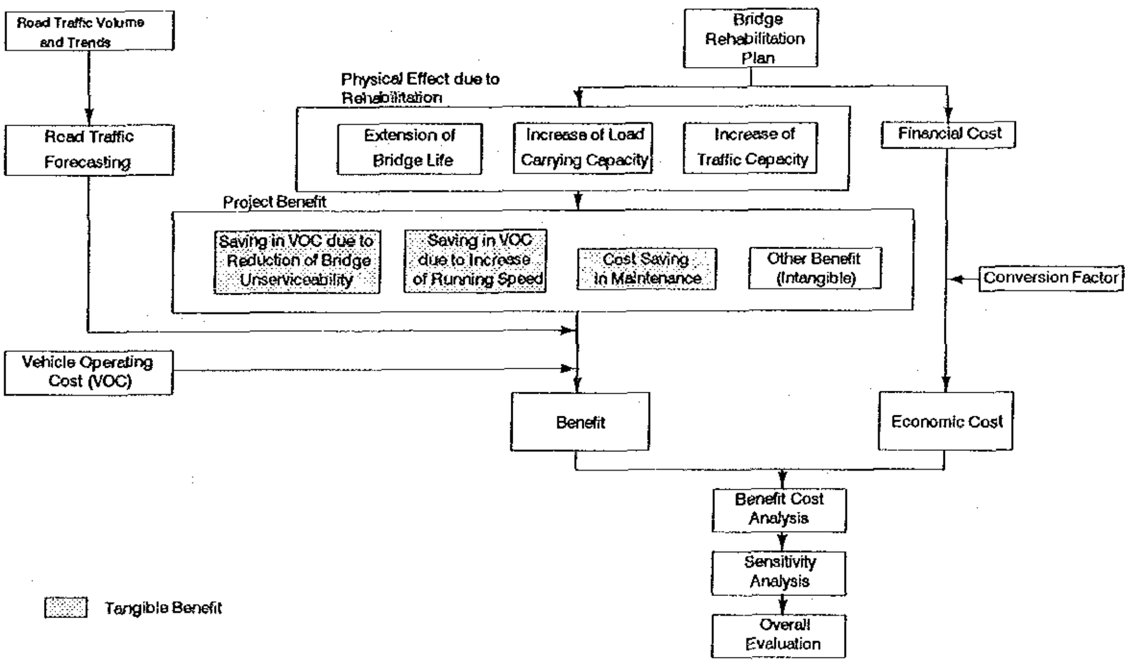
6) Convert financial cost into economic cost.

7) Work out economic evaluation by benefit cost analysis.

8) Conduct sensitivity analysis to test evaluation stability.

9) Judge project feasibility considering related aspects.

Figure I-1 Flowchart of Economic Evaluation



(2) **Traffic Projection**

Future traffic volume on the bridge shall be estimated by "trends model" based on "Traffic Volume Malaysia". The following procedure shall be taken as shown in Fig. I-2.

i) Identify a traffic count station corresponding to each bridge.

ii) Determine traffic volume (16 hours) by vehicle type and growth rate for each bridge.

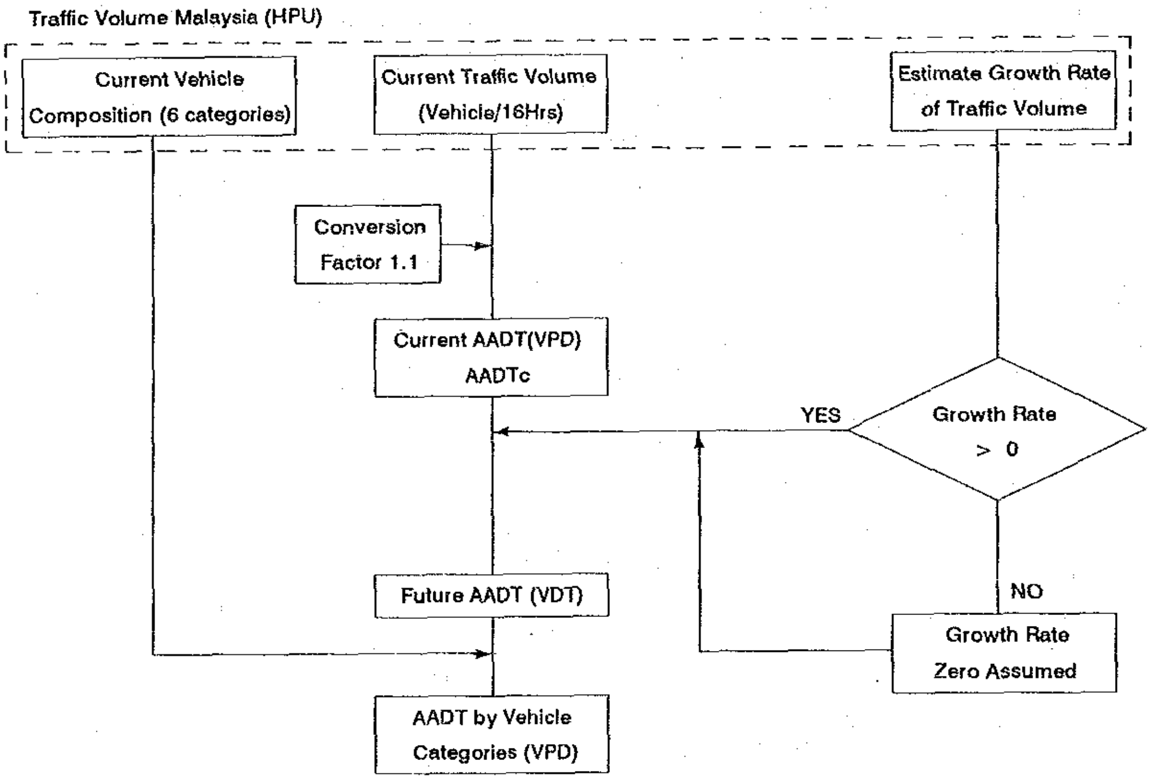
iii) Convert 16-hour traffic volume to daily value (VPD).

iv) Set growth rate at zero if it is calculated negative.

v) Calculate future traffic volume using the growth rate.

vi) Break down daily traffic volume into those by vehicle type using the present modal shares.

Figure I-2 Flowchart of Traffic Projection



(3) **Economic Costs**

The cost components to be considered in bridge rehabilitation project are:

- Rehabilitation Cost

  Investment cost in order to improve bridge durability and to enhance bridge functions.

- Maintenance Cost

  Continuously required cost in order to keep bridge serviceability after rehabilitation.

In economic evaluation, cost should be "economic cost", not "financial cost" in terms of market prices. "National Parameter for Project Appraisal" published by EPU in 1984 shows conversion factors from financial to economic cost by type of work. In this manual, a conversion factor of 0.80 is taken in accordance with the above.

Using the conversion factor, economic cost is derived according to the following formula:

$$\text{Economic Cost (EC)} = \text{Conversion Factor (CF)} \times \text{Financial Cost (FC)}$$

Annual maintenance cost is assumed to be 1 % of construction cost in this manual. In addition, standard construction cost of a bridge in Malaysia is about M$2,500 per squaremeter of deck in 1991 market prices.

Maintenance cost after rehabilitation proposed in this manual is assumed as follows:

i) Reconstruction and widening :
   2.5 % of rehabilitation Cost for 5 years (0.5%/years)

ii) Reinforcement :
   5.0 % of rehabilitation Cost for 5 years (1.0%/years)

iii) Protection :
   10.0 % of rehabilitation Cost for 5 years (2.0%/years)

These maintenance costs are also subject to conversion.

(4) **Benefit Measurement**

Tangible benefits accrued from bridge rehabilitation are :

i) Savings in vehicle operating cost due to a reduction of interruption to traffic flow.

Improvement of bridge durability reduces number of days of interruption (extends bridge life) and, therefore, saves vehicle operating cost due to detours.

ii) Savings in vehicle operating cost due to an increase in vehicle speed

Bridge widening makes it possible for vehicles to keep at constant speed on and near the bridge.

iii) Maintenance Cost Savings

The maintenance cost savings can be expected on the bridge administrator side. The benefit comes from the difference in maintenance cost between "with" and "without" project cases.

1) Probability Model For Bridges To Be Unusable

In order to quantify the benefit of reducing the interruption to traffic flow, a probabilistic model is introduced.

The "Unserviceability Probability Density" f(t) is:

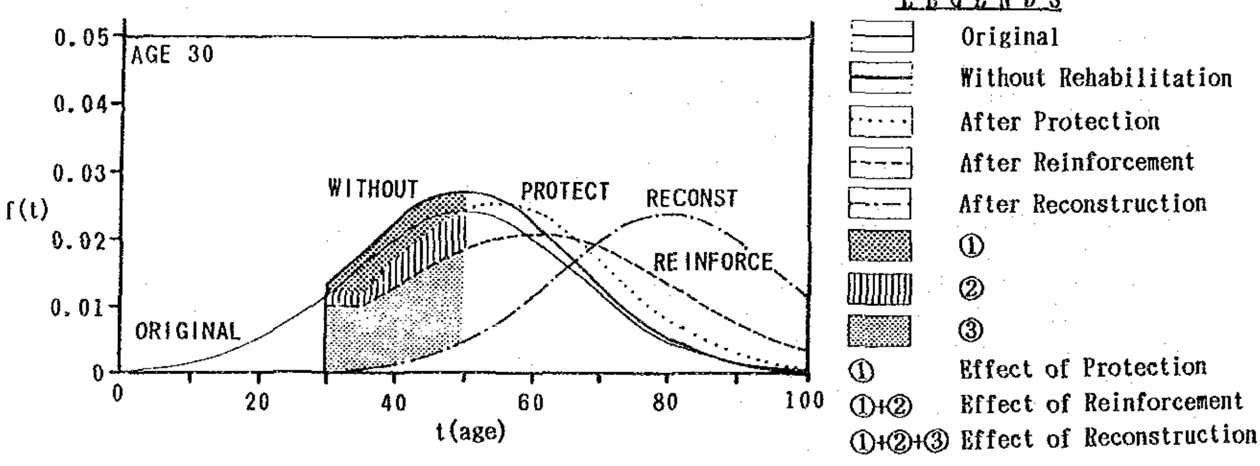
$$f(t) = \frac{1}{\sqrt{2\pi}\delta}\,\epsilon - (t-m)^2/2\delta^2 = N[m,\delta^2] = N[50,16.7^2]$$

In this manual, bridge life is determined at 50 years.

In order to quantify the benefit of rehabilitation works, a function for the bridge unserviceability probability density after rehabilitation should be assumed.

Figure I-3 presents schematically the effects of maintenance works in terms of the probability for interruptions, for example bridges of an age of 30 years.

Figure I-3 Probability Density of Bridge Unserviceability After Rehabilitation



2) Vehicle Operating Costs

The National Axle Load Study (NALS) developed the evaluation model for vehicle operating cost in Malaysia after a detailed survey in 1987. In this manual, Econom V5.1 is adopted .

Table I-1 Vehicle Operating Costs by Vehicle Type

(M$/km)

Vehicle Type	M'cycles	Cars & Taxies	Buses	S.Vans & Utilities	Medium Lorries	Heavy Lorries
Regular Route	0.046	0.184	1.517	0.498	0.785	1.059
Detour Route	0.055	0.220	1.859	0.632	0.911	1.147
Before Widening	0.066	0.266	2.761	0.867	1.248	1.579

3) Equivalent Age of Bridge

Residual life of a bridge differs by structure, traffic volume, geography, current damage and other factors even physical age. In order to assess the residual life of a bridge, a concept of "equivalent age of bridge" was introduced in relation to unserviceability probability based on the overall rating from a safety viewpoint, traffic volume and construction year. It can be defined as a normalized age on the unserviceability probability density function with an average age of 50.

Table I-2 Assumed Equivalent Age of Bridge

Overall Rating from Safety viewpoint (R)	Traffic Volume (AADT : Vehicle/day)	Year Built	
		Before 1945	After 1945
4.0 <= R	all	45	40
3.5 <= R < 4.0	AADT >= 9,000	45	40
	AADT < 9,000	40	30
R < 3.5	AADT >= 9,000	40	30
	AADT < 9,000	30	20

4) Interruption to Traffic Flow

In order to estimate the duration of bridge interrupts, the number of months required for bridge construction is assumed as a function of bridge length as follows:

$$\log(M) = 0.572 \log(L) + 0.043$$

where, M : Standard number of months required for bridge construction
L : Bridge Length (m)

Using "M", number of days of interruptions is derived as follows:

$$d = f \times M \times (365/12)$$

where, f : probability for a bridge to be unusable
d : number of days for a bridge to be unusable

5) Equation for Benefit Calculation

The equations developed for calculating benefits are based on two options, 1) bridge rehabilitation (with project) and 2) do nothing (without project).

i) Protection

$$B1 = \sum_x \sum_i (fo(x)-fwp(x))\ DBU\ AADTxi\ (DLo\ VOCio-DLw\ VOCiw)$$

where, B1 : Benefit from Protection ($)
fo(x): Probability for a bridge to be unusable in year x without rehabilitation
fwp(x): Probability for a bridge to be unusable in year x after protection
DBU : Duration of Bridge Unservice (day)
AADTxi: Average Annual Daily Traffic of Vehicle type i in year x (vehicles/day)
DLo : Length of detour route (km)
DLw : Length of regular route (km)
VOCio: Vehicle Operating Cost of Vehicle type i in detour route ($/km)
VOCiw: Vehicle Operating Cost of Vehicle type i in regular route ($/km)

ii) Reinforcement

$$B2 = \sum_{x} \sum_{i} (fo(x)-fwr(x))\ DBU\ AADTxi\ (DLo\ VOCio-DLw\ VOCiw)$$

where, B2 : Benefit from Reinforcement ($)
fwr(x): Probability for a bridge to be unusable in year x after reinforcement

iii) Reconstruction

$$B3 = \sum_{x} \sum_{i} (fo(x)-fwc(x))\ DBU\ AADTxi\ (DLo\ VOCio-DLw\ VOCiw)$$

where, B3 : Benefit from reconstruction ($)
fwc(x): Probability for a bridge to be unusable in year x after reconstruction

iv) Widening

$$B4 = \sum_{x} \sum_{i} (AADTxi\ (BL+200)/1000\ (VOCio-VOCiw))$$

where, B4 : Benefit from widening ($)
BL : Length of bridge (m)

v) Cost Saving in Maintenance

$$B5 = \Sigma_{i}\ (BL\quad BW\quad UCC\quad Pm)$$

where, B5 : Benefit from cost saving in Maintenance ($)
BW : Width of bridge (m)
UCC : Unit cost of bridge construction ($/m2)
Pm : Rate of annual maintenance cost against to initial construction cost (0.01)

(5) Economic Evaluation

Economic evaluation quantifies cost and benefit first and assesses both in comparison. This is usually called "Benefit Cost Analysis". In this analysis, the following three parameters are generally used for decision making and for determining priority:

i) Benefit Cost Ratio (BCR)

This ratio is calculated by dividing benefit by cost in terms of net present value:

$$BCR = \left(\sum_t Bt/(1+i)^t \right) / \left(\sum_t Ct/(1+i)^t \right)$$

where, Bt : Benefit in year t
Ct : Cost in year t
i : Discount rate

This quantifies the magnitude of net present benefit per net present cost.

ii) Net Present Value (NPV)

Unlike benefit cost ratio, this quantifies the magnitude of net present benefit less net present cost.

$$NPV = \left(\sum_t Bt/(1+i)^t \right) - \left(\sum_t Ct(1+i)^t \right)$$

iii) Internal Rate of Return (IRR)

This rate is defined to be a discount rate or an interest rate where net present value of the project becomes zero. In other words, this is the highest interest rate that makes the project economically feasible.

When $\sum_t (Bt-Ct)/(1+i)^t = 0$

this "i" is called "Internal Rate of Return".

In order to calculate the net present value of benefit/cost or to determine project viability when IRR of the project is calculated, "discount rate" is needed as a criterion. In this manual, a discount rate of 11% per annum is used. Benefit/cost analysis shall be done over 20 years after the commencement of the project.

I.2 Calculation

A worksheet of Lotus 1-2-3 is attached in order for a bridge maintenance engineer to carry out automatically economic evaluation by only inputting necessary data.

(1) Input data

1) Bridge Data

No. : (arbitrary taken)
Key : (Code No. of bridge inventory)
State
District
Year Built
Length : (m)
Carriageway Width : (m)
Overall Rating from Safety Viewpoint
Length of Detour Route : Da (km)
Length of Regular Route : Db (km)

2) Traffic Census Data

State
District
Station Index
Year Conducted
16-Hours Traffic
Growth Rate : (%/year)
Vehicle Composition by Vehicle Type : (%)

3) Rehabilitation Data

Rehabilitation Method : Input "1" in the column corresponding to rehabilitation method.
Rehabilitation Cost : ($/bridge in market price)
Conversion Factor : "0.80" should be specified.
Standard Construction Cost of a Bridge : ($/m2 in market price)

4) Vehicle Operating Cost

Vehicle operating cost in 1991 prices are already set on the worksheet.
Regular Route : ($/km in economic price)
Detour Route : ($/km in economic price)
Before Widening : ($/km in economic price)

5) Calculation for Indicators

Discount Rate ; do

When a discount rate do is inputted, a corresponding NPVo is calculated and, at the same time, NPV1 is also calculated corresponding to a discount rate of d1 = do + 2 (%). Using these NPVo and NPV1, a discount rate d3 when NPV3=0 shall be calculated.

Value of do shall be gradually adjusted so that NPV3 approaches to 0.

As an initial value of do, BCR x 0.1 is recommendable.

(2) **Output**

1) Indicators

BCR : Benefit Cost Ratio
NPV : Net Present Value
IRR : Internal Rate of Return (%)

2) Cost Benefit Flow

Cost benefit flow by vehicle type over the project period is calculated using the discount rate inputted.

3) Worksheet

An example of calculating economic indicators is shown in Table I-3.

Table I-3 Worksheet of Calculating Economic Indicators

< BRIDGE DATA >

No.	Key	State	District	Year Built	Length (m)	Carriageway Width (m)	Rating fro safety V.P	Equivalent Age	Detour Route (km) Da	Db
32	237200	Pahang	Kuantan	1960	26.7	7.3	3.4	30	20.0	13.0

< TRAFFIC CENSUS DATA >

Traffic Census station State	District	Stn. Index	Year	16-Hours Traffic	Growth (%/Yr)	Vehicle Composition (%) CARS & TAXIES	S.VANS & UTILITIES	MED. LORRIES	HEAVY LORRIES	BUSES	M'CYCLE
Pahang	Kuantan	J22	1988	12593	7.4%	51.7%	11.3%	3.4%	3.6%	1.5%	28.4%

< REHABILITATION DATA >

Rehabilitation Method

Protect.	Reinfor.	Reconst.	Widening
1	0	0	0

	Rehabilitation Cost ($)	Standard Const. Cost ($/m2)	Standard Const. Period (days)
finansial	375,724	2,500	220
C/F	0.8	0.8	
economic	300,579	2,000	

< Vehicle Operation Cost ($/km) >

	Cars&Taxi	S.Vans&Ut	Med.Lorry	Heavy lorr	Buses	M'cycles
regular rt	0.184	0.498	0.785	1.059	1.517	0.046
detour rt.	0.220	0.632	0.911	1.147	1.859	0.055
before wid	0.266	0.867	1.248	1.579	2.761	0.066

< ECONOMIC EVALUATION >

Evaluation Period (20 years) from 1994

Discount Rate 11.0%

	IRR	NPV	BCR
Indicator	21.21%	277,710	1.83

<PROVABILITY DENSITY OF BRIDGE UNSERVICEABILITY >

	EQUIVALENT AGE 20				EQUIVALENT AGE 30				EQUIVALENT AGE 40				EQUIVALENT AGE 45			
	WITHOUT	PROTECT	REINF.	RECONST.	WITHOUT	PROTECT	REINF.	RECONST.	WITHOUT	PROTECT	REINF.	RECONST.	WITHOUT	PROTECT	REINF.	RECONST.
1994	0.493%	0.482%	0.122%	0.027%	1.312%	1.234%	0.590%	0.027%	2.720%	2.400%	1.953%	0.027%	3.638%	3.086%	2.536%	0.027%
1995	0.546%	0.482%	0.122%	0.032%	1.407%	1.234%	0.590%	0.032%	2.815%	2.400%	1.953%	0.032%	3.697%	3.086%	2.536%	0.032%
1996	0.607%	0.482%	0.122%	0.038%	1.503%	1.234%	0.590%	0.038%	2.902%	2.400%	1.953%	0.038%	3.744%	3.086%	2.536%	0.038%
1997	0.670%	0.482%	0.122%	0.046%	1.601%	1.234%	0.590%	0.046%	2.981%	2.400%	1.953%	0.046%	3.778%	3.086%	2.536%	0.045%
1998	0.737%	0.482%	0.122%	0.054%	1.698%	1.234%	0.590%	0.054%	3.051%	2.400%	1.953%	0.054%	3.798%	3.086%	2.536%	0.054%
1999	0.807%	0.482%	0.122%	0.063%	1.795%	1.234%	0.590%	0.063%	3.112%	2.400%	1.953%	0.063%	3.805%	3.086%	2.536%	0.063%
2000	0.882%	0.536%	0.137%	0.074%	1.891%	1.323%	0.636%	0.074%	3.162%	2.483%	2.010%	0.074%	3.798%	3.137%	2.574%	0.074%
2001	0.959%	0.594%	0.153%	0.087%	1.985%	1.414%	0.684%	0.087%	3.202%	2.560%	2.065%	0.087%	3.778%	3.176%	2.607%	0.087%
2002	1.040%	0.655%	0.171%	0.101%	2.076%	1.505%	0.734%	0.101%	3.231%	2.630%	2.117%	0.101%	3.744%	3.205%	2.634%	0.101%
2003	1.123%	0.721%	0.190%	0.117%	2.163%	1.597%	0.785%	0.117%	3.249%	2.692%	2.164%	0.117%	3.697%	3.222%	2.655%	0.117%
2004	1.209%	0.790%	0.211%	0.136%	2.246%	1.688%	0.839%	0.136%	3.254%	2.746%	2.208%	0.136%	3.638%	3.228%	2.671%	0.136%
2005	1.296%	0.862%	0.234%	0.156%	2.324%	1.778%	0.894%	0.156%	3.249%	2.790%	2.247%	0.156%	3.567%	3.222%	2.680%	0.156%
2006	1.385%	0.938%	0.258%	0.179%	2.396%	1.857%	0.951%	0.179%	3.231%	2.826%	2.282%	0.179%	3.485%	3.205%	2.683%	0.179%
2007	1.475%	1.017%	0.285%	0.205%	2.461%	1.952%	1.008%	0.205%	3.202%	2.851%	2.312%	0.205%	3.393%	3.176%	2.680%	0.205%
2008	1.565%	1.098%	0.313%	0.234%	2.519%	2.034%	1.067%	0.234%	3.162%	2.866%	2.337%	0.234%	3.291%	3.137%	2.671%	0.234%
2009	1.654%	1.182%	0.344%	0.266%	2.569%	2.112%	1.127%	0.266%	3.112%	2.872%	2.357%	0.266%	3.180%	3.086%	2.656%	0.266%
2010	1.742%	1.267%	0.376%	0.301%	2.611%	2.185%	1.187%	0.301%	3.051%	2.866%	2.372%	0.301%	3.063%	3.026%	2.635%	0.301%
2011	1.829%	1.354%	0.411%	0.339%	2.644%	2.253%	1.248%	0.339%	2.981%	2.851%	2.381%	0.339%	2.939%	2.956%	2.608%	0.339%
2012	1.913%	1.442%	0.448%	0.381%	2.668%	2.315%	1.309%	0.381%	2.902%	2.826%	2.385%	0.381%	2.810%	2.878%	2.575%	0.381%
2013	1.993%	1.530%	0.487%	0.427%	2.682%	2.369%	1.370%	0.427%	2.815%	2.790%	2.384%	0.427%	2.678%	2.792%	2.537%	0.427%

< COST BENEFIT FLOW >

No.	YEAR	A.A.D.T. (VPD)	fo-fw	REHABIL. COST (WITH)	MAINT. COST (WITH)	E.COST STREAM	BENEFIT FROM PROTECTION OR REINFORCEMENT OR RECONSTRUCTION CARS & TAXIES	S.VANS & UTILITIES	MED. LORRIES	HEAVY LORRIES	BUSES	M'CYCLE	BENEFIT FR WIDENING	MAINT. COST (WITHOUT)	BENEFIT STREAM
1	1994	20,003		300,579		300,579								3,898	3,898
2	1995	21,028	0.173%			0	8,311	5,578	2,182	2,644	2,097	1,141	0	3,898	25,851
3	1996	22,053	0.270%			0	13,572	9,109	3,563	4,317	3,424	1,864	0	3,898	39,748
4	1997	23,078	0.367%			0	19,331	12,974	5,074	6,149	4,876	2,655	0	3,898	54,958
5	1998	24,103	0.465%			0	25,556	17,152	6,708	8,129	6,447	3,510	0	3,898	71,401
6	1999	25,128	0.562%		30,058	30,058	32,209	21,617	8,455	10,246	8,125	4,423	0	3,898	88,973
7	2000	26,153	0.568%			0	33,909	22,758	8,901	10,786	8,554	4,657	0	3,898	93,463
8	2001	27,178	0.571%			0	35,427	23,777	9,300	11,269	8,937	4,865	0	3,898	97,474
9	2002	28,203	0.571%			0	36,727	24,650	9,641	11,683	9,265	5,044	0	3,898	100,907
10	2003	29,228	0.566%			0	37,771	25,351	9,915	12,015	9,528	5,187	0	3,898	103,666
11	2004	30,253	0.558%		30,058	30,058	38,525	25,856	10,113	12,255	9,718	5,291	0	3,898	105,656
12	2005	31,278	0.546%			0	38,955	26,145	10,226	12,391	9,827	5,350	0	3,898	106,791
13	2006	32,304	0.529%			0	39,031	26,196	10,246	12,416	9,846	5,360	0	3,898	106,993
14	2007	33,329	0.509%			0	38,729	25,993	10,166	12,320	9,770	5,319	0	3,898	106,195
15	2008	34,354	0.485%			0	38,027	25,523	9,982	12,096	9,593	5,222	0	3,898	104,342
16	2009	35,379	0.457%		30,058	30,058	36,911	24,774	9,689	11,741	9,311	5,069	0	3,898	101,394
17	2010	36,404	0.426%			0	35,372	23,741	9,285	11,252	8,923	4,658	0	3,898	97,329
18	2011	37,429	0.391%			0	33,408	22,422	8,769	10,627	8,428	4,588	0	3,898	92,140
19	2012	38,454	0.353%			0	31,022	20,821	8,143	9,868	7,826	4,260	0	3,898	85,839
20	2013	39,479	0.313%			0	28,228	18,946	7,410	8,979	7,121	3,877	0	3,898	78,458
	TOATL			300,579	90,174	390,753	601,072	403,383	157,768	191,183	151,617	82,539	0	77,964	1,665,475

< CALCULATION FOR INDICATORS > d : discount rate(%)

NPV3= 15 NPVo= 35,799 NPV1= 184

		DISCOUNT 11.0%	COST	BENEFIT	d3 21.21%	COST	BENEFIT	do 19.20%	COST	BENEFIT	d1 21.20%	COST	BENEFIT
1	1994	1.000	300,579	3,898	1.000	300,579	3,898	1.000	300,579	3,898	1.000	300,579	3,898
2	1995	0.901	0	23,289	0.825	0	21,327	0.839	0	21,687	0.825	0	21,329
3	1996	0.812	0	32,260	0.681	0	27,054	0.704	0	27,974	0.681	0	27,059
4	1997	0.731	0	40,185	0.562	0	30,861	0.590	0	32,449	0.562	0	30,869
5	1998	0.659	0	47,034	0.463	0	33,078	0.495	0	35,367	0.463	0	33,090
6	1999	0.593	17,838	52,801	0.382	11,488	34,006	0.416	12,490	36,972	0.382	11,493	34,021
7	2000	0.535	0	49,969	0.315	0	29,471	0.349	0	32,582	0.315	0	29,487
8	2001	0.482	0	46,949	0.260	0	25,358	0.292	0	28,507	0.260	0	25,373
9	2002	0.434	0	43,786	0.215	0	21,657	0.245	0	24,758	0.215	0	21,672
10	2003	0.391	0	40,526	0.177	0	18,356	0.206	0	21,338	0.177	0	18,370
11	2004	0.352	10,586	37,210	0.146	4,391	15,435	0.173	5,190	18,244	0.146	4,395	15,448
12	2005	0.317	0	33,883	0.121	0	12,871	0.145	0	15,470	0.121	0	12,883
13	2006	0.286	0	30,583	0.099	0	10,639	0.122	0	13,003	0.100	0	10,649
14	2007	0.258	0	27,347	0.082	0	8,711	0.102	0	10,827	0.082	0	8,721
15	2008	0.232	0	24,207	0.068	0	7,062	0.086	0	8,925	0.068	0	7,070
16	2009	0.209	6,282	21,192	0.056	1,678	5,661	0.072	2,157	7,276	0.056	1,680	5,669
17	2010	0.188	0	18,326	0.045	0	4,483	0.060	0	5,859	0.046	0	4,490
18	2011	0.170	0	15,630	0.038	0	3,502	0.051	0	4,653	0.038	0	3,507
19	2012	0.153	0	13,118	0.031	0	2,691	0.042	0	3,637	0.031	0	2,695
20	2013	0.138	0	10,802	0.026	0	2,029	0.036	0	2,789	0.026	0	2,033
			335,285	612,995		318,137	318,152		320,417	356,215		318,148	318,331

BCR 1.828

NPV 277,710

NPV3= 15 NPVo= 35,799 NPV1= 184

JICA