CHAPTER 9 SELECTION OF 10 BRIDGES FOR REHABILITATION, REINFORCEMENT

9.1 Introduction

The 29 bridges are nominated to be studied for the bridge rehabilitation in this Project. A visual inspection for the bridges was carried out by the Study Team with the bridge engineers of the counterparts. The site survey including measurement of dimensions of bridges and the investigation of natural condition around bridges were also carried out simultaneously. The damages of bridges were inspected and recorded based on the inspection sheet for the evaluation of deficiency of bridges. Ten bridges were selected from the 29 bridges as the objects for the design of rehabilitation of bridges under the scope of this Study.

In the selection process of ten bridges, it is necessary to decide the weight of evaluation for each different type of damage on the different parts of bridge. Furthermore, the weight of the importance of each bridge part shall be evaluated, and the degree of damage and the importance of bridge parts shall be combined to decide the final selection.

The Analytic Hierarchy Process (AHP), a decision support method is used for the evaluation of bridge deficiency in this Study. The AHP incorporates a tangible evaluation criteria and an intangible one and provides a procedure based on pair wise comparisons that measure the criteria being considered. AHP is a set of a method, which breaks down a complex issue into a simple component and arranges these components in the simple hierarchical structure. The method of weighing the component, which is a part of the AHP, is used to evaluate the degree of the bridge deficiencies in this Project.

Ten bridges were selected based on the results of the evaluation of the bridge deficiencies.

9.2 Evaluation Method of Bridge Deficiency

9.2.1 Format for Bridge Inspection

The types of damage on each parts of a bridge and the member of a bridge, which is composed of the parts, are listed in the bridge inspection sheet for field survey as shown in Table 8.2.1. The damage of bridges were inspected through a field survey based on the inspection sheet and the grades of damage for each part of the bridge were recorded by the study team and the bridge engineers of counterparts in Costa Rica. The evaluation of bridge deficiency has been carried out based on theses bridge inspection sheets.

The definition for the damage degree of structure is as follows;

Degree 1: No Damage is observed
Degree 2: Damage is observed at a few parts.
Degree 3: Damage is observed at many parts.
Degree 4: Damage is observed at less than half of whole part.
Degree 5: Damage is observed at almost whole part

9.2.2 Steps for Evaluation of Bridge Deficiency

The evaluation of bridge deficiency is carried out through the following steps;

Step1: To list up the bridge parts, which compose an entire bridge, and to determine sets of bridge parts, which compose major members of the bridges such as superstructure and substructure.

Step2: To set the entire hierarchy of the bridge component for evaluation.

Step3: To determine the weight of each damage and each part of a bridge in Hierarchy 3.

To determine the weight should be as follows;

1. Define the type of damage on bridge parts.
2. Construct a set of pair wise comparison matrix for each of the type of damage.
3. Damages on each bridge part are compared and evaluated to fill the pair wise comparison matrix by using the scale of relative importance.
4. Pair wise comparison matrix should be constructed.
5. Multiply every element in each row of the matrix and their n^{th} root is extracted as eigenvector components where n is the number of elements.
6. Column of numbers obtained is normalized to unity as weight of damage by dividing each component by the sum of all components.

Step4: The weight of each part, which is defined as a part of the major bridge member in Hierarchy 2, is determined by the same procedure as Hierarchy 3.

Step5: The weight of major parts of the bridge member in Hierarchy 1 is also determined by the same procedure.

Step6: The degree of damage is multiplied by the weight of damage in Hierarchy 3.

Step7: Each lower level of the weight is multiplied by the weight immediately above to obtain the entire deficiency rate of the bridge.

9.2.3 Entire Hierarchy of Bridge Component

Entire hierarchy of components for evaluation of Bridge deficiencies is shown in Figure 9.2.1.

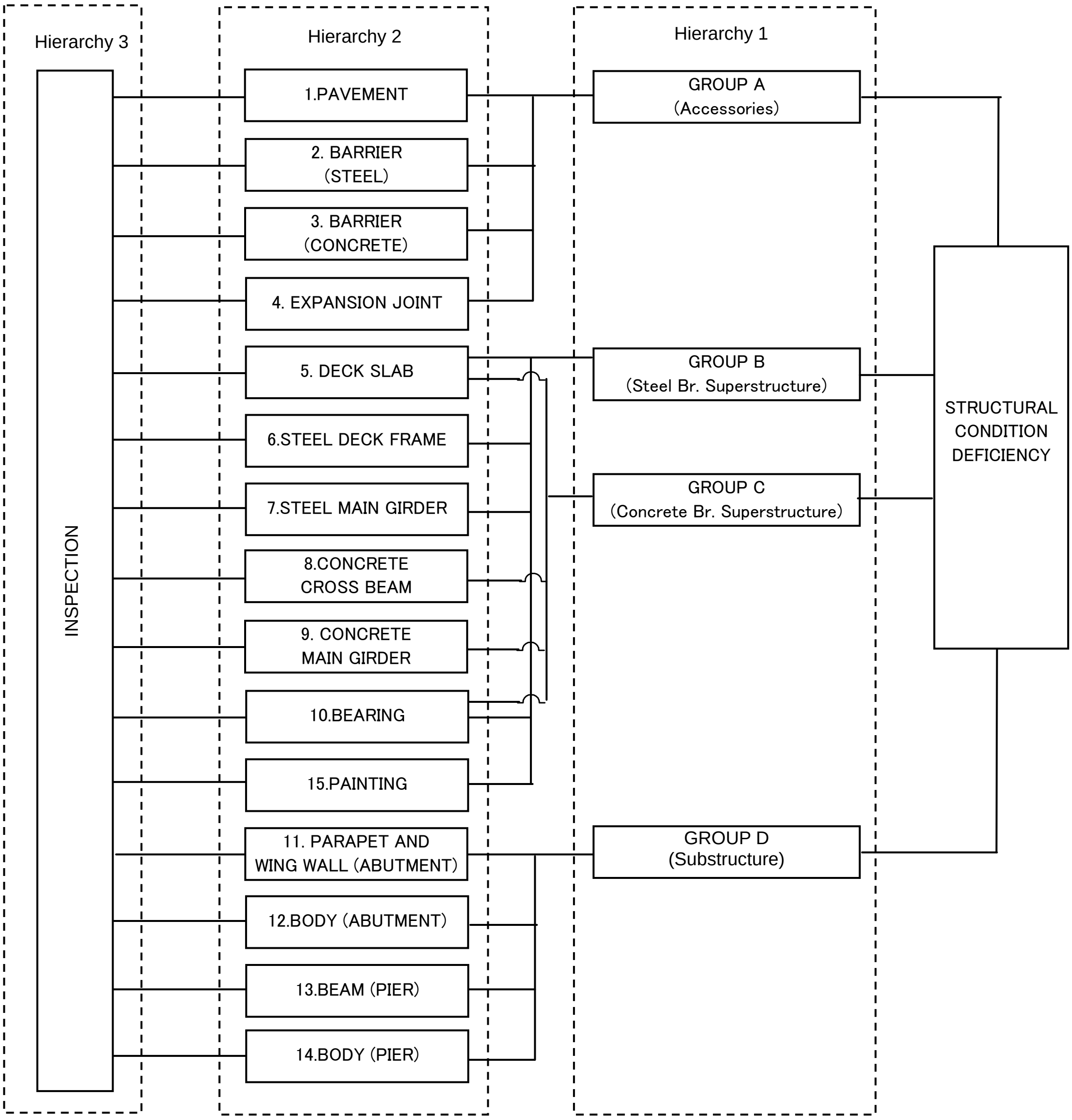


Figure 9.2.1. Hierarchies of Bridge Components

9.2.4 Evaluation of Bridge Deficiency

1) Component of the Bridge

The component of the bridge used for the evaluation of bridge deficiency shall be consistent with the content of the inspection sheet, because the results of the inspection shall be the basic data for the evaluation.

2) Determination of Weight for Damage in Hierarchy 3

(1) Type of Damage on Bridge Parts.

The types of damage are different and shall be defined for every bridge part. The types of damage are nominated on the inspection sheet.

(2) Scale of Relative Importance

The weight of each type of damage shall be determined, compared and evaluated by using the scale of relative importance which shown in Table 9.2.1.

Table 9.2.1. Scale of Relative Importance

Intensity of Relative Importance	Definition	Explanation
1	Equal importance	Two activities contribute equally to the objective.
3	Moderate importance of one over another	Experience and judgment slightly favor one activity over another.
5	Essential or strong Importance	Experience and judgment essentially favor one activity over another.
7	Demonstrated Importance	An activity is strongly favored and its dominance is demonstrated in practice.
9	Extreme importance	The evidence favoring one activity over Another is of the highest possible order of affirmation.
2, 4, 6, 8	Intermediate values between the two adjacent judgments	When compromise is needed.
Reciprocal of above Non-zero numbers	If an activity has one of the above numbers (e.g.3) compared with A second activity, then the second activity has the reciprocal value (i.e.1/3) when compared to the first.	

(3) Calculation of the Weight for Damage

The pair wise comparison matrix should be constructed in advance as preparation of the calculation. Every element in each row of the matrix shall be multiply and their n^th root is extracted as eigenvector component where n is the number of elements. Column of numbers obtained is normalized to unity as weight of damage by dividing each component by the sum of all components. Calculation method for the damage mentioned above is illustrated in Table 9.2.2. Weight calculation for pavement and Barrier are shown in Table 9.2.3. and Table 9.2.4. for the examples. The weights for the other inspection parts of bridge are shown in Appendix 5 in this report.

Table 9.2.2. Calculation Method for Weight

	The Matrix				Eigenvector Components	Vector of Priorities
	A_1	A_2	A_3	A_4		
A_1	$\frac{W_1}{W_1}$	$\frac{W_1}{W_2}$	$\frac{W_1}{W_3}$	$\frac{W_1}{W_4}$	$\sqrt[4]{\frac{W_1}{W_1}*\frac{W_1}{W_2}*\frac{W_1}{W_3}*\frac{W_1}{W_4}}=a$	$\frac{a}{Total}=X_1$
A_2	$\frac{W_2}{W_1}$	$\frac{W_2}{W_2}$	$\frac{W_2}{W_3}$	$\frac{W_2}{W_4}$	$\sqrt[4]{\frac{W_2}{W_1}*\frac{W_2}{W_2}*\frac{W_2}{W_3}*\frac{W_2}{W_4}}=b$	$\frac{b}{Total}=X_2$
A_3	$\frac{W_3}{W_1}$	$\frac{W_3}{W_2}$	$\frac{W_3}{W_3}$	$\frac{W_3}{W_4}$	$\sqrt[4]{\frac{W_3}{W_1}*\frac{W_3}{W_2}*\frac{W_3}{W_3}*\frac{W_3}{W_4}}=c$	$\frac{c}{Total}=X_3$
A_4	$\frac{W_4}{W_1}$	$\frac{W_4}{W_2}$	$\frac{W_4}{W_3}$	$\frac{W_4}{W_4}$	$\sqrt[4]{\frac{W_4}{W_1}*\frac{W_4}{W_2}*\frac{W_4}{W_3}*\frac{W_4}{W_4}}=d$	$\frac{d}{Total}=X_4$

Note: “X_1, X_2, X_3, X_4” are the weight for “A_1, A_2, A_3, A_4” respectively.

Table 9.2.3. Weight of Damage for Pavement

ITEM	1.	2.	3.	4.	Igen Vector	Weight
1. WAVING	1	4	1	1/5	**0.946**	**0.161**
2. RUTTING	1/4	1	1/5	1/7	**0.291**	**0.049**
3. CRACK	1	5	1	1/5	**1.000**	**0.170**
4. HOLES	5	7	5	1	**3.637**	**0.619**
	7.250	17.000	7.200	1.543	**5.874**	**1.000**

Table 9.2.4. Weight of Damage for Barrier

ITEM	1.	2.	3.	4.	Igen Vector	Weight
1. DEFORMATION	1	7	1/3	1/9	**0.714**	**0.100**
2. RUSTING	1/7	1	1/7	1/9	**0.218**	**0.031**
3. CORROSION	3	7	1	1/7	**1.316**	**0.185**
4. MISSING	9	9	7	1	**4.880**	**0.685**
	13.143	24.000	8.476	1.365	**7.128**	**1.000**

(4) Weight of Bridge Parts

Weight of major bridge parts which is defined in Hierarchy 2 is determined by the same procedure as Hierarchy 3. Weight of bridge parts in the group of accessories of bridge is shown in Table 9.2.5. Weight of bridge parts in the group of super structure of steel bridge is shown in Table 9.2.6.

Table 9.2.5. Weight of the Parts in the Group of Accessories

ITEM	1.	2.	3.	Igen Vector	Weight
1. PAVEMENT	1	1/9	5	**0.822**	**0.151**
2. BARRIER KERB	9	1	9	**4.327**	**0.797**
3. EXPANSION JOINT	1/5	1/9	1	**0.281**	**0.052**
	10.200	1.222	15.000	**5.430**	**1.000**

Table 9.2.6. Weight of the Parts in the Group of Super Structure of Steel Bridge

ITEM	1.	2.	3.	4.	5.	Igen Vector	Weight
1. DECK SLAB	1	3	1/3	5	7	**2.036**	**0.264**
2. STEEL DECK FRAME	1/3	1	1/5	3	5	**1.000**	**0.130**
3. STEEL MAIN GIRDER	3	5	1	7	9	**3.936**	**0.510**
4. BEARING	1/5	1/3	1/7	1	3	**0.491**	**0.064**
5. PAINTING	1/7	1/5	1/9	1/3	1	**0.254**	**0.033**
	4.676	9.533	1.787	16.333	25.000	**7.718**	**1.000**

(5) Weight of Bridge Components

Weight of major bridge component such as super structure and substructure shall be calculated by using the same method of Hierarchy 2 and 3. The weights of major bridge components are calculated as shown in Table 9.2.7.

Table 9.2.7. Weight of the Bridge Component

ITEM	1.	2.	3.	Igen Vector	Weight
1. ACCESSORLES	1	1/7	1/9	**0.251**	**0.055**
2. SUPERSTRUCTURE	7	1	1/3	**1.326**	**0.290**
3. SUBSTRUCTURE	9	3	1	**3.000**	**0.655**
	17.000	4.143	1.444	**4.578**	**1.000**

3) Evaluation of Total Deficiency Rate of Bridges

(1) Step 1

Deficiency rate at step 1 is obtained by multiplying the weights of each type of damage and the degree of damage from the inspection.

$$D_{3j} = \Sigma (W_i \times E_i)$$

Where;

D_{3j} : Calculation Results for bridge pars in Hierarchy 2
W_i : Weight of damage in Hierarchy 3
E_i : Degree of Damage correspond to the type of damage

(2) Step 2

Deficiency rate at step 2 is obtained by multiplying the weight of the bridge parts in hierarchy 2 and results of the Step 1

$$D_{2k} = \Sigma W_2 \times D_{3j}$$

Where:

D_{2k} : Calculation Results for each bridge components in Hierarchy 1
W_2 : Weight of Parts in Hierarchy 2
D_{3j} : Calculation Results in Step 3
J : Number of Bridge parts in the group of bridge component

(3) Step 3

Total deficiency rate of the bridge is obtained by the following calculation.

$$D_{total} = \Sigma \ (W_1 \times D_{2k})$$

Where:

D_{total} : Total Bridge Deficiency
W_1 : Weight of Bridge Component in Hierarchy 1
D_{2k} : Calculation Results in Step 2
K : Number of Bridge Components

9.3 Results of the Calculation

9.3.1 Weight of the Evaluation Items

The weight of the damage of bridge parts, the bridge parts and the major bridge component for steel bridges are listed in the Table 9.3.1. and 9.3.2 respectively . The weight of the barrier shall be selected depending on its material type.

Table 9.3.1. Weight of Evaluation Items for Steel Bridge

PART OF BRIDGE	PART OF DAMAGE	TYPE OF DAMAGE & EVALUATION OF DAMAGE DEGREE					
ACCESSORIES	1. PAVEMENT	1. WAVING	2. RUTTING	3. CRACK	4. HOLES		
	0.151	0.161	0.049	0.170	0.619		
	2. BARRIER (STEEL)	1. DEFORMATION	2. RUSTING	3. CORROSION	4. MISSING		
	0.797	0.100	0.031	0.185	0.685		
	3. BARRIER (CONCRETE)	1. CRACK	2. EXPOSURE OF REINFORCEMENT	3. MISSING			
	(0.797)	(0.058)	(0.207)	(0.735)			
	4. EXPANSION JOINT	1. ABNORMAL NOISE	2. WATER LEAKING	3. MISSING OR DEFORMATION	4. VERTICAL MOVEMENT	5OBSTRUCTION JOINTS	6. EXPOSURE OF RC BAR
0.055	0.052	0.042	0.077	0.499	0.178	0.020	0.183
STEEL SUPER-STRUCTURE	5. DECK SLAB	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.049	0.107	0.072	0.232	0.019	0.159
		7. ASPHALT OVERLAY	8. HOLES				
	0.264	0.034	0.328				
	6. STEEL DECK FRAME	1. RUSTING	2. CORROSION	3. DEFORMATION	4. BREAKAGE OF CONNECTION	5. BREAKAGE OF SWAY BRACING	
	0.130	0.032	0.121	0.061	0.320	0.466	
	7. STEEL MAIN GIRDER	1. RUSTING	2. CORROSION	3. DEFORMATION	4. DEFICIT OF BOLTS	5. CRACK OF WELDING OR PLATE	
	0.510	0.029	0.085	0.279	0.179	0.428	
	10. BEARING	1. BREAKAGE OF SUPPORT	2. ABNORMAL DEFORMATION	3. CLEANING REQUIRE			
	0.064	0.649	0.279	0.072			
	15. PAINTING	1. DISCOLORATION	2. RUSTING	3. RISING	4. PEELING		
0.290	0.033	0.055	0.564	0.118	0.263		
SUB-STRUCTURE	11. PARAPET AND WING WALL (ABUTMENT)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.036	0.067	0.162	0.103	0.381	0.034	0.253
	12. BODY (ABUTMENT)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.030	0.047	0.030	0.155	0.030	0.106
		7. EMBANKMENT SLOPE	8. INCLINATION	9. SOURING			
	0.400	0.071	0.309	0.222			
	13. BEAM (PIER)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.165	0.147	0.147	0.064	0.369	0.033	0.240
	14. BODY (PIER)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.033	0.072	0.033	0.160	0.033	0.108
		7. INCLINATION	8. SCOURING				
0.655	0.400	0.329	0.233				

Table 9.3.2. Weight of Evaluation Items for Concrete Bridge

PART OF BRIDGE	PART OF DAMAGE	TYPE OF DAMAGE & EVALUATION OF DAMAGE DEGREE					
ACCESSORIES	1. PAVEMENT	1. WAVING	2. RUTTING	3. CRACK	4. HOLES		
	0.151	0.161	0.049	0.170	0.619		
	2. BARRIER (STEEL)	1. DEFORMATION	2. RUSTING	3. CORROSION	4. MISSING		
	0.797	0.100	0.031	0.185	0.685		
	3. BARRIER (CONCRETE)	1. CRACK	2. EXPOSURE OF REINFORCEMENT	3. MISSING			
	(0.797)	(0.058)	(0.207)	(0.735)			
	4. EXPANSION JOINT	1. ABNORMAL NOISE	2. WATER LEAKING	3. MISSING OR DEFORMATION	4. VERTICAL MOVEMENT	5. OBSTRUCTION JOINTS	6. EXPOSURE OF RC BAR
0.055	0.052	0.042	0.077	0.499	0.178	0.020	0.183
CONCRETE SUPER-STRUCTURE	5. DECK SLAB	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.049	0.107	0.072	0.232	0.019	0.159
		7. ASPHALT OVERLAY	8. HOLES				
	0.270	0.034	0.328				
	8. CONCRETE CROSS BEAM	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.126	0.067	0.162	0.103	0.381	0.034	0.253
	9. CONCRETE MAIN GIRDER	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.565	0.067	0.162	0.103	0.381	0.034	0.253
	10. BEARING	1. BREAKAGE OF SUPPORT	2. ABNORMAL DEFORMATION	3. CLEANING REQUIRE			
0.290	0.039	0.649	0.279	0.072			
SUB-STRUCTURE	11. PARAPET AND WING WALL (ABUTMENT)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.036	0.067	0.162	0.103	0.381	0.034	0.253
	12. BODY (ABUTMENT)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.030	0.047	0.030	0.155	0.030	0.106
		7. EMBANKMENT SLOPE	8. INCLINATION	9. SOURING			
	0.400	0.071	0.309	0.222			
	13. BEAM (PIER)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
	0.165	0.147	0.147	0.064	0.369	0.033	0.240
	14. BODY (PIER)	1. ONE DIRECTIONAL CRACK	2. TWO DIRECTIONAL CRACK	3. CONCRETE FISSILITY	4. EXPOSURE OF REINFORCEMENT	5. HONEYCOMB, CAVITY	6. FREE LIME
		0.033	0.072	0.033	0.160	0.033	0.108
		7. INCLINATION	8. SCOURING				
0.655	0.400	0.329	0.233				

9.3.2 Summery of the Deficiency Rate of 29 Bridges

The result of the calculation of deficiency for the 29 bridges is summarized in the Table 9.3.3.

The 5 bridges No. 6, 9, 12, 14, and 26 consist of two types of superstructure. For these bridges, the subscripts of "(1/2)" or "(2/2)" are added after a bridge name. The material types (steel or concrete) of superstructure are the same except Bridge No.12 Rio Puerto Bridge. Two values of priority are calculated for these bridges. Therefore, the higher priories for each bridge are listed in the table for the selection of 10 bridges.

Table 9.3.3. Deficiency Rate of 29 Bridges

No.	Bridge Name	Bridge Code	Route Number	Km	Type	Acces-saries	Super-structure	Sub-structure	Priority
1	Rio Blanco	28	32	146.185	Concrete	0.087	0.373	1.519	1.979
2	Rio Cuba	27	32	134.895	Concrete	0.076	0.363	1.328	1.767
3	Rio Nuevo	16	2	327.245	Concrete	0.056	0.294	1.074	1.424
4	Rio Chirripó	17	4	0.450	Concrete	0.065	0.299	1.003	1.367
5	Rio Caracol	15	2	323.335	Steel	0.061	0.324	0.976	1.361
6	Rio Puerto Nuevo (1/2)	12	2	234.400	Steel	0.057	0.293	0.962	1.312
7	Rio Sucio	20	32	39.775	Concrete	0.065	0.430	0.817	1.312
8	Rio Toro Amarillo	21	32	59.650	Concrete	0.103	0.373	0.817	1.293
9	Rio San José	18	4	4.083	Concrete	0.056	0.296	0.921	1.273
10	Rio Chirripó (1/2)	26	32	126.220	Steel	0.060	0.551	0.655	1.266
11	Rio Sarapiquí	19	4	30.810	Steel	0.090	0.378	0.793	1.261
12	Rio Reventazón	23	32	95.050	Concrete	0.097	0.292	0.809	1.198
13	Rio Abangares	3	1	143.335	Steel	0.065	0.445	0.673	1.183
14	Rio Aranjuez	2	1	87.780	Steel	0.087	0.357	0.735	1.179
15	Rio Pacuare	24	32	100.400	Concrete	0.089	0.302	0.774	1.165
16	Rio Tempisquito	8	1	240.225	Steel	0.074	0.354	0.732	1.160
17	Rio Piedras	4	1	189.831	Concrete	0.058	0.352	0.740	1.150
18	Rio Zapote	13	2	248.400	Concrete	0.056	0.294	0.787	1.137
19	Rio Azufrado	7	1	239.845	Concrete	0.107	0.373	0.655	1.135
20	Rio Parismina	22	32	78.710	Concrete	0.056	0.318	0.751	1.125
21	Rio Curré	11	2	229.385	Steel	0.055	0.293	0.772	1.120
22	Rio Ahogados 1954 (1/2)	6	1	232.510	Steel	0.076	0.376	0.664	1.116
23	Rio Volcán (1/2)	9	2	181.820	Concrete	0.057	0.291	0.751	1.099
24	Río Torres	29	218	146.185	Concrete	0.090	0.292	0.716	1.098
25	Rio Colorado (ver dibujo)	1	1	36.605	Concrete	0.064	0.340	0.672	1.076
26	Rio Ceibo	10	2	189.150	Steel	0.058	0.323	0.692	1.073
27	Rio Terraba (2/2)	14	2	256.110	Steel	0.061	0.351	0.655	1.067
28	Rio Barbilla	25	32	116.365	Concrete	0.089	0.308	0.655	1.052
29	Rio Colorado	5	1	221.980	Concrete	0.065	0.308	0.672	1.045

9.4. Selection of 10 Bridges for Further Detailed Study

The 10 bridges for a further detailed study should be selected not only based on the bridge damage but also other factors related to bridge rehabilitation and maintenance in this Study, because the objective of this Study is to improve the capacity of the ability for bridge maintenance in Costa Rica. Therefore, the ten bridges for a further detailed study for the rehabilitation were selected thorough the following criteria. However, the bridges, which are required a reconstruction, should not be selected in this Study.

1. Different types of rehabilitation or repair method.
2. Typical damage on bridges in Costa Rica.
3. Locate on the high priority road.
4. Different structural types of bridge.
5. High priority for requirement of repair

Selected 10 bridges are shown in Table 9.4.1.

Table 9.4.1. 10 Bridges Selected for Further Rehabilitation Studies

Point of View for Selection	Bridge Code	Bridge Name	Type of Bridge	Reason for Selection
Damage of Deck Slab	2	Aranjuez River	Steel Continuous Truss	1, 2, 5
	3	Abangares River	Steel Simple Truss	1, 2, 5
Scouring	16	Nuevo River	RC Continuous Deck Girder	1, 2, 5
Earthquake-proof	12	Puerto Nuevo River	Steel Simple I-Girder	1, 2, 5
	19	Sarapiquí River	Steel Simple I-Girder	1, 2, 5
	26	Chirripó River	Steel Continuous I-Girder	1, 2, 5
	29	Torres River	PC Simple I-Girder	1, 2, 3, 5
Abnormal Deformation of Main Girder	17	Chirripó River	PC Continuous Box-Girder	1, 2, 5
	20	Sucio River	PC Continuous Box-Girder	1, 2, 5
Other Type	7	Azufrado River	RC Rigid Frame	1, 4, 5

The damage of abutments of the Bridges of No.28 and No.27 are most serious. It is expected that the reconstruction of these bridges is cheaper than the repairing in the total cost. Therefore, these bridges were not selected for the 10 bridges for further detailed study in this project.

CHAPTER 10 PLAN FOR REHABILITATION, REINFORCEMENT AND IMPROVEMENT OF 10 SELECTED BRIDGES

10.1 Identification of Deterioration in 10 Bridges

In this study, the Detailed Inspection has been carried out under two (2) technical practices. One of them is a visual inspection in site carried out using inspection instruments/tools. This is to judge accurate causes of breakage/deformation and make a plan for repair/reinforcement methods. Another one is a structural analysis to judge the stability or the load carrying capacity of each structural member in accordance with the present standard in Costa Rica.

Figure 10.1.1 illustrates work cycle for the inspection and repair/reinforcement of the bridge. This chapter is to be focused on Detail Inspection and Selection of Repair/Reinforcement method, which are highlighted with gray color in Figure 10.1.1.

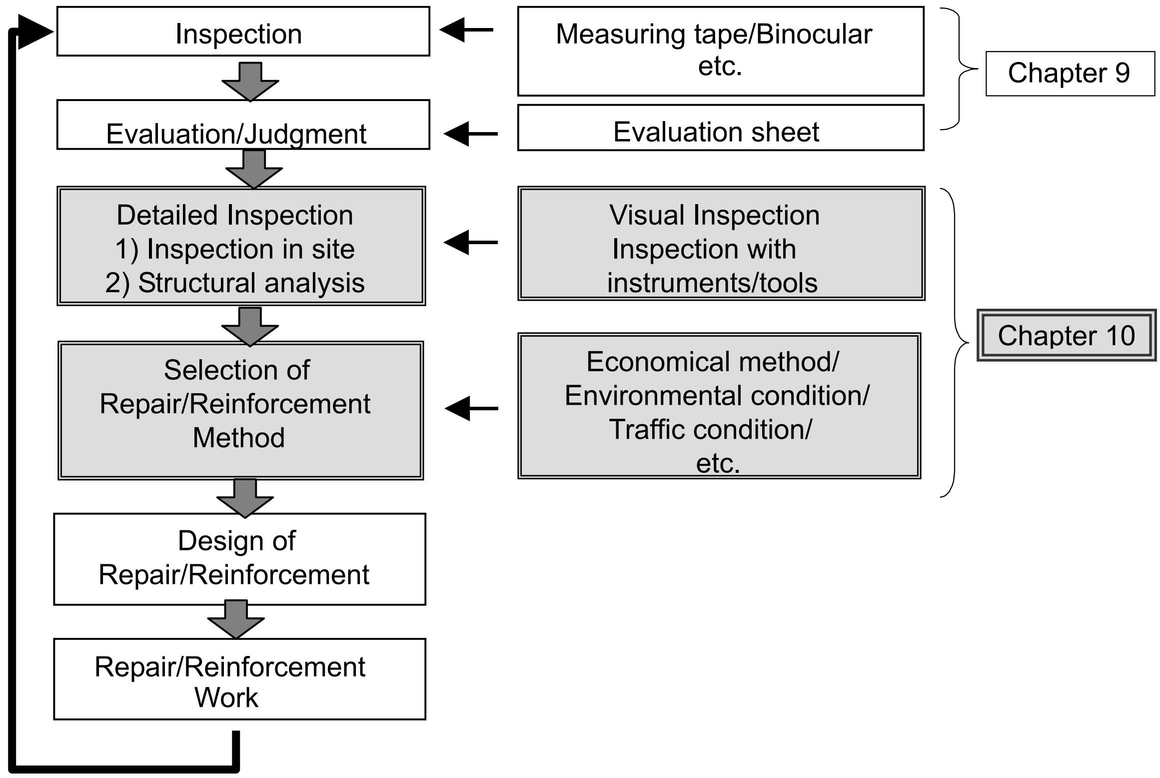


Figure 10.1.1. Process of Inspection and Repair/Reinforcement Work

10.1.1 Method to Identify Deterioration Mechanism

When any abnormality from original structural conditions are observed throughout the detailed inspection, it is a key to examine the bridge deterioration, which might cause the damage and to identify the type, the size and exact location of deformation. Figure 10.1.2 shows the work-breakdown and flow for identification of deteriorating mechanism.

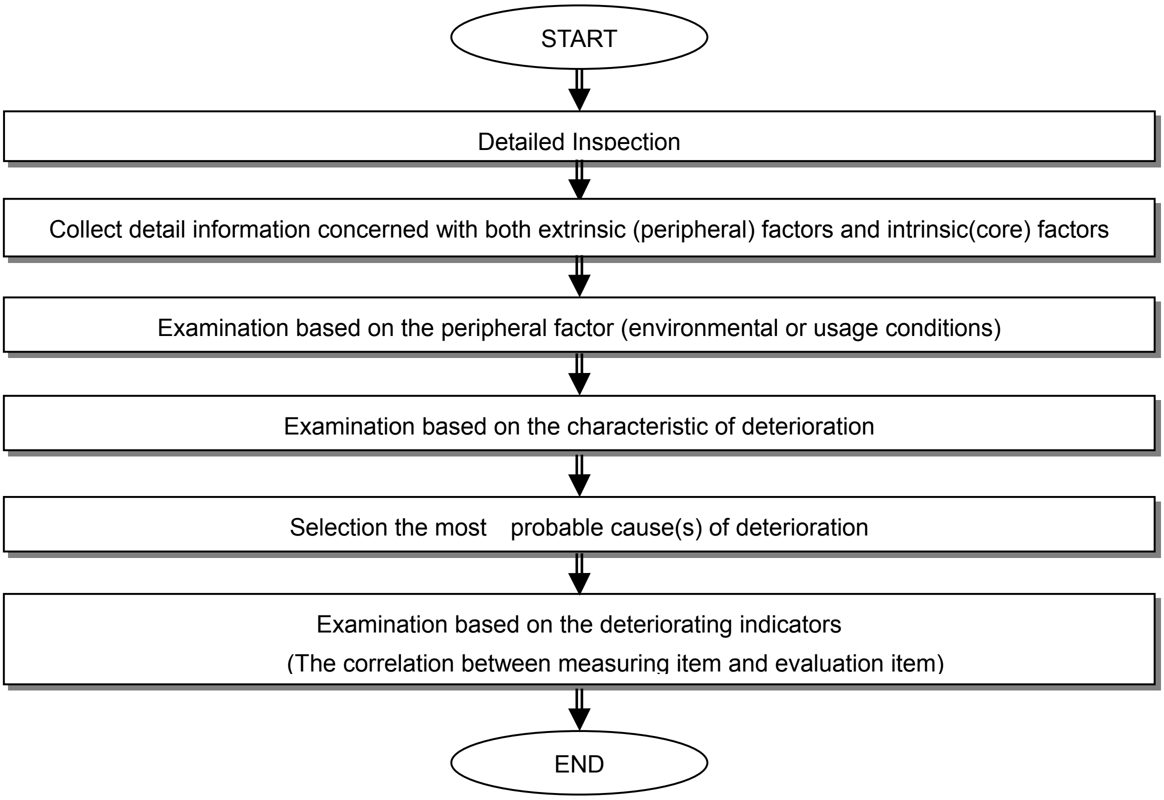


Figure 10.1.2. Work-Breakdown and Flow for Identification of Deteriorating Mechanism

Firstly, the detailed inspection must be carried out to learn the causes and the degree of bridge deteriorations. The analytical details concerning both core factors and peripheral factors , that enable to evaluate quantitatively the bridge deteriorations, must consequently be collected. The peripheral factors mean those factors, which secondarily give, occasion for the deteriorations such as environmental and/or bridge usage conditions. On the other hand the core factors mean primary influenceable factors such as its own material property (e.g. Alkali reactivity) and/or deficiencies of construction work (e.g. quality of concrete, cover for reinforcement). The core factors are generally known to accelerate the deterioration inducing carbonation or salt corrosion. Table 10.1.1 shows the correlation between the deteriorating mechanisms and the peripheral factors.

The deteriorating mechanisms will be presumed by examining both the peripheral factors and the characteristics of deteriorations.

The most probable cause(s) of deteriorations will be selected by evaluating a correlation between the peripheral factors and the structural characteristics of abnormality. And throughout an examination of above selected causes of deteriorations, it will enable the engineering judgment to be taken and to predict the deteriorating mechanisms with accuracy.

Table 10.1.2 shows the correlation among the deteriorating mechanisms, deterioration factors, deterioration indicators and structural features of deteriorations.

Table 10.1.1. Correlation between Deteriorating Mechanism and Peripheral Factor

Peripheral Factor		Presumed Deteriorating Mechanism
Environmental Conditions	Coastal area	Salt Corrosion
	Cold area	Frost damage Salt corrosion (By antifreeze agent)
	Volcanic area	Chemical attack
Operation Conditions	Repetition of wet and dry	Alkali aggregate reaction, Salt corrosion, Frost damage
	Repetition of Loading	Fatigue
	Carbon dioxide	Carbonation
	Acid rain or water	Carbonation, Chemical attack

Table 10.1.2. Correlation between Mechanism, Factors, Indicators and Characteristics of Deterioration

Mechanism	Factor	Characteristics of Deterioration	Deterioration Index
Carbonation	Carbon dioxide	- Cracking in the direction of rebar axis - Delamination of concrete	- Carbonation Depth - Corroded area or volume of steel bars
Chloride induced deterioration	Chloride ions	- Cracking in the direction of rebar axis - Rust exudation	- Chloride ion content, Corroded area or volume of steel bars
Frost damage	Freezing and thawing action	- Fine cracks, Scaling, Pop-outs, Deformation	- Depth of frost deterioration - Corroded area or volume of steel bars
Chemical attack	Acid materials Sulfate ions	- Discoloration Delamination of concrete	- Intrusion depth of deteriorating factors, Carbonation depth, Corroded area or volume of steel bars
Alkali-aggregate reaction	Reactive aggregate	- Expansive cracking in restraining directions - Distributed cracking - White gel - Discoloration	Expansion (Cracking)
Fatigue of RC slab	Traffic of large vehicles (in excess of the designed load)	- Lattice (Raft) cracking, corner disintegration, free lime	- Crack density - Deflection
Fatigue of RC beam	Repeated loads	- Cracking and rupture of tensile steel	- Accumulated damage, crack length of steel bars

10.1.2 Type and Cause of Deterioration

This Section shows the types and causes of the deterioration on both the concrete structure and the steel structure.

In general, there are two (2) cases of deterioration progress, the first case is that a single factor causes the deterioration, another case is that a multiple factors cause the deterioration. Moreover, it is noted that every single factor might intricately and mutually be linked to the others. Therefore, it is required that engineers totally understand the system of deteriorations and the correlation between effects i.e. the deterioration and causes. It is also essential that, at the stage of inspection, engineers try to predict direct causes of each deteriorations in order for the bridge conditions to be properly diagnosed. In this study, the typical and most illustrative examples in Costa Rica are taken up for examining the deterioration mechanism so as to secure the well-grounded information sources for engineers.

In the following part, the type and the causes of deteriorations, which give technical explanations of the system of occurrence, are shown in each concrete member and steel member respectively.

1) Types and Causes of Concrete Deterioration

Following table shows the general types of concrete deterioration with description of its phenomenon and photos.

Table 10.1.3. General Types of Concrete Deterioration

Type of Deterioration	Photo	Phenomenon / Causes
Crack	 	- Most popular deterioration in concrete member - It occurs by 6 causes, ➢ Extra-Force: Repeated Load ➢ Environmental Effects: Salt Effects ➢ Material Degradation : Alkali - Silica Reaction, Neutralization ➢ Volume Changes: Heat Expansion, Dry Shrinkage ➢ Defect in Construction ➢ Structural Characteristics and Defect in Design - If it is observed, engineer should recognize that this structure might have some damage and deterioration in the concrete member.
Separation Steel Exposure		- Concrete fragment drop causes dangerous for third-party beneath the structure. - It occurs by following causes ➢ Corrosion and Expansion of Steel bar ➢ Shortage for Covering - It affects the fatal damage for structure because of shortage of effective cross-section area.
Hole(s)		- It mainly occurs at the deck plate with free lime phenomenon. - Before it has been developed, there are two steps; ➢ One-direction crack caused by dry shrinkage ➢ Then, two- direction crack caused by repetition load - It causes a great loss for traffic safety without urgent rehabilitation.

Free lime Water Leaking		- It is the results of reaction between the lime in concrete and water leaked from cracks. - It affects the decline of concrete strength and the promotion of alkali – silica reaction.

These phenomena occur by the deterioration system, which are shown by deterioration causes in the flowing table.

Table 10.1.4. Deterioration System of Concrete Member

Cause of Deterioration	Deterioration System
Carbonation	- In the chemical reaction that $CaCO_3$ is generated by Ca(OH) and carbonic acid gases in atmosphere, $Ca(OH)_2$ in concrete is consumed so that pH: potential of hydrogen is declined to be neutralized. - Neutralization does not affect to the concrete material directly but to covered film of reinforcement steel to be destroyed so that reinforced steel progresses to be rusting. - Rusting of steel bar affects its expansion and develops cracks and separation of concrete.
Salt Corrosion	- Chloride ion affects steel to be rusted, then it affects expansion of bar and develops cracks, separation of concrete and shortage of cross-section area - Chloride ion is provided by the concrete member itself remained at production, and by the extra-environment such as the wind from the sea.
Alkali Aggregate Reaction	- Alkali –Silica Gel is developed with the Na_2O or K_2O in concrete and its reactor aggregate. - This gel is easy to be expanded with absorbed water then affects to its crakes.
Initial Defect	- This is the deterioration system by short of strength against the external force change (e.g. Over load, Repeated Load) and shortage of covering because of defective quality for construction. - This also occurs by shortage of cross section area when the deterioration reaches to the separation and peeling off caused by various systems such as neutralization, salt corrosion and alkali-silica reaction.

These deterioration systems are diagramed as the figure below. There are two types of deterioration system. One of the types of it is the deterioration of steel or concrete inside of structure caused by environmental conditions or material. Carbonation and Salt Corrosion deteriorate concrete or steel member developed from the surface of concrete, whereas Alkali Aggregate Reaction deteriorates concrete from the inside of concrete. These three deterioration systems basically begin after completion of construction. The rate of the deterioration development will depend on the situation such as the type of materials and environmental conditions. Another deterioration system is affected by initial defect that is caused by the increased load or repeated load due to unexpected amount of traffic volume. And also the short of covering for concrete is caused by defective construction quality.

In both of the types, first stage of the deterioration begins with a crack, and then final stage of deterioration ends with scaling off of the concrete. It is noted that the concrete fragment drop seriously causes to the lack of cross section area and induces severe development of deteriorations further.

When engineers/inspectors conduct the routine inspection of bridge, they should try to predict what is the direct cause of this deterioration based on the technical knowledge of its system.

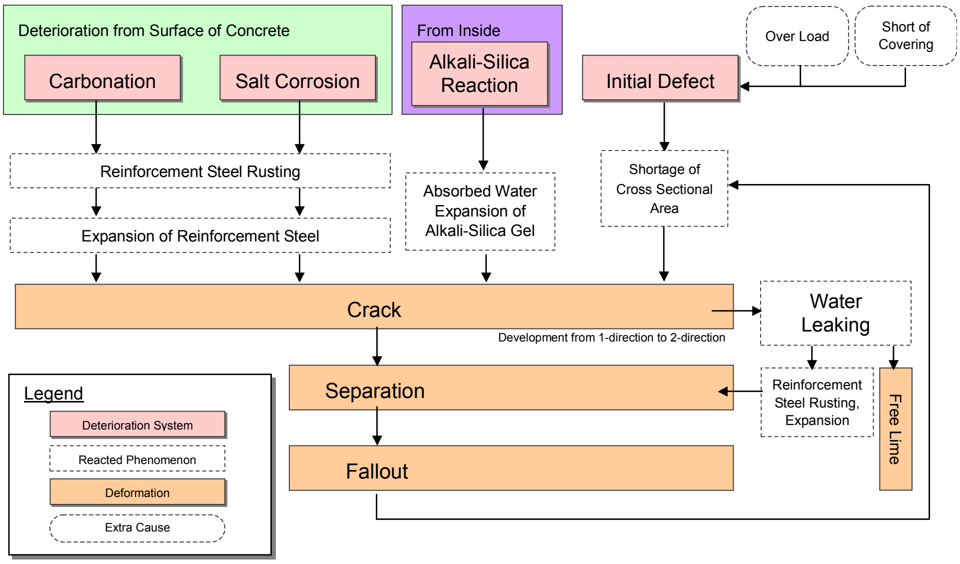


Figure 10.1.3. Deterioration System Chart for Concrete Member

2) Types and Causes of Steel Deterioration

The types of deterioration in steel member are,

- Corrosion,
- Fatigue,
- Deformation caused by friction or looseness,
- Delayed failure of High Tension Bolt (HTB),
- Damage caused by disaster (Natural disaster or Human error).

Among these deteriorations, Corrosion, Fatigue and Natural disaster are to be particularly focused as the typical deteriorations for the steel structure in Costa Rica according to results of site inspection. Following tables show these typical steel deteriorations with description of their phenomenon and photos.

a) Corrosion

Corrosion is classified into two (2) types of corrosion based on its conditions, one will be a sweeping corrosion and another one will be a local corrosion.

A sweeping corrosion is a phenomenon that the whole surface of steel member corrodes uniformly and occurs when steel member is weathered in the air.

On the other hand, a local corrosion is a phenomenon that a part of steel member is corroded when a material, a corrosive environment and a mechanism of corrosion are is different from other parts. For examples, the local corrosion occurs on parts where the drain water flows, water from expansion joint runs, water leaking from slab deck, water and dust gathered easily and drained difficulty due to structural shape. The local shape of corrosion presents is a form like a hole or a groove.

Progress of corrosion depends on the surrounding environment and type of structure, shape.

The factors to generate corrosion are humidity, temperature, rainfall, daylight and pollutant (salt particle or sulfur dioxide etc.), and a degree of these influential factors directly affects the rate of corrosion growth.

Generally the corrosion of steel material begins when humidity reaches more than 60% and the rate of corrosion growth tends to be faster as the temperature becomes higher. Among these factors, the salt particle or the sulfur dioxide is the most influential factor for corrosion.

Furthermore, the sunlight and the pollutant are also influential factors to deteriorate the paint coat and consequently accelerate the corrosion growth.

b) Fatigue

The cracks in steel members occur and grow by the high-frequent repeating inner stress even if its stress is fairly lower than the allowable stress. This phenomenon is fatigue failure.

Fatigue failure is called “sub-critical progress of crack” and this is the one of main characteristics of ductile material such as steel that demonstrates enough tenacity when cracks occurred. Accordingly, an occurrence of small-scale of cracks will not conduct the member or the structure to collapse, in addition, by taking appropriate measures in early stage of cracking, it enables the member or the structure to secure enough its safety ratio and to be prevented from unstable conditions. The most influential factors of fatigue are the fluctuation of repeated stress and frequency of repeated stress. Moreover, shape of splice plate, welding defect, residual stress and so on are added as the factors to influent fatigue stress. In case of steel material, as its strength increases higher, fatigue stress also increases higher, whereas the strength of welding dose not increase.

Most of the deteriorations caused by fatigue is observed on the splice plate, especially the splice of secondary member and the welded part.

It is generally known that some causes of fatigue failure are two-sidedly considered both in loading conditions and in structural issue since the fatigue failure occurs when inner stress exceeds fatigue strength of splice.

c) Damage caused by disaster

Disasters that affect the member or the structure are listed as natural disaster and human errors. Natural disaster is represented by earthquakes, floods etc. Human errors by fire, collide by car, dropping heavy object and etc.

The most dangerous damage is the one caused by earthquakes. It generally occurs in the substructure or the support, however, and there are a few cases that the superstructure is severely damaged by earthquakes.

3) Causes of Scouring

Scouring is a erosion that results from water flow washing and carrying away the river bed, banks as well as around piers and abutments of bridges. Scouring varies on the extent of the damage according to the materials. Loose and granular soils, e.g. sand bed, tend to be rapidly eroded by water flow, while cohesive and cemented soils present more resistance against scouring.

It is noted that the scour at a bridge crossing is composed with three scoring processes as described below. When these three components mix up, they cause the scour at a pier or an abutment.

a) Long-term aggradation and degradation of the river bed

Aggradation and degradation are phenomenon that vary an elevation of the riverbed due to natural and/or man-induced causes, and that are to be developed in long-term. Aggradation is to be caused by sedimentation of materials eroded from the river channel or juncture at the upstream, whereas degradation caused by lowering or scouring the streambed due to insufficient sedimentation with materials supplied from upper reach.

b) General scour at the bridge

General scour is a lowering of the riverbed transversely at the bridge location. This lowering may be uniform or non-uniform across the bed, that is, the depth of scour may be deeper in some parts of the cross section. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood and is more of a local issue due to a scour-inducing feature of the channel geometry (flow constriction, a bend, etc)

General scour may result from contraction of the flow, which causes removal of materials from the bed across all or most of the channel width, or from other general scour conditions such as flow around a bend where the scour may be concentrated near the outside of the bend. (Figure 10.1.3).

c) Local scour at the piers or abutments

Local scour is a removal of material from around piers, abutments, spurs and embankments, which is caused by an acceleration of the flow and vortices induced by obstructions in a current. Basically, a mechanism of local scour at piers or abutments is a formation of vortices (known as the horseshoe vortex) at the bottom part of piers or abutments. (Figure 10.1.4).

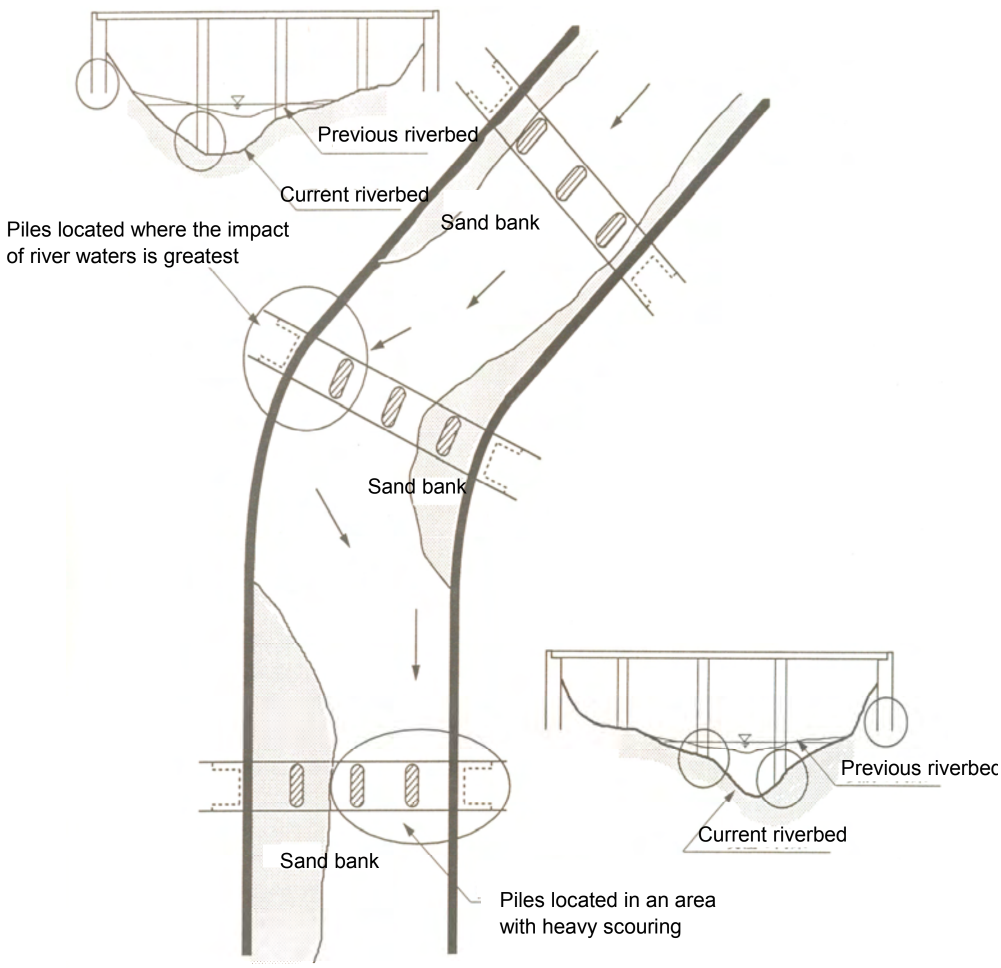


Figure 10.1.3. Schematic of Representation Biggest Impact Point of River

It is complicate that the magnitude of scouring is measured because of a nature of the cyclic process of scouring. It tends to cause a deepest scour when a flood at peak, but it hardly happens that scour holes are refilled with sediment as flood waters recede. Engineers and inspectors must assess the present state of the river stream and the watershed in order to evaluate potential changes that may affect to the river system. This assessment enables the long-term aggradation to be estimated.

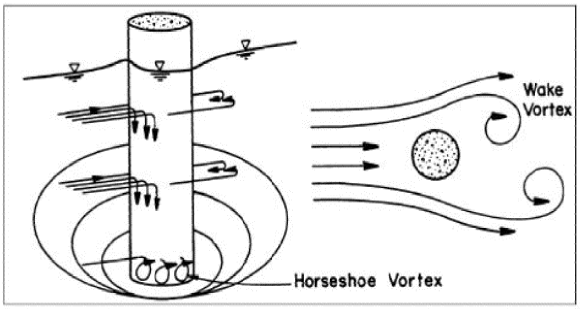


Figure 10.1.4.
Schematic of Representation of Scour at Cylindrical Pier

10.1.3 Detailed Inspection Method

This section describes the inspection method for each deterioration system for the concrete member.

Most common type of detailed inspection is visual inspections [C1]. No sophisticated, high technical apparatus are required to implement these inspections. It is accordingly to say that these are the most economical and important inspections. For the concrete members, it is solely possible for a visual inspection to gives information of the surface conditions partly based on the opinion of the inspector.

For further details such as the underlying condition of concrete members, there are several detailed inspections shown in the table below [C2]- [C7] in order to identify what deterioration system is a main cause.

Table 10.1.5. Deterioration System and Its Suitable Inspection Method

Deterioration System	Inspection Item		Inspection Method
Carbonation	Deterioration inspection	Carbonation Depth	Phenol Phthalein Method [C 2]
		Corrosion of Steel Bar	Self-potential Method [C 3]
Salt Corrosion	Deterioration inspection	Corrosion of Steel Bar	Self-potential Method [C 3]
		Degree of Salinity in Concrete	Chloride Meter [C 4]
	Cause Assumption	Density of Salinity in atmosphere	-
Alkali Aggregate Reaction	Deterioration inspection	Crack	Visual Inspection [C 1]
	Cause Assumption	Core Testing	Alkali Aggregate Reaction Test
Initial Defect	Deterioration inspection	Surface Strength	Schmidt Hummer Method [C 5]
		Compaction Testing	Concrete Core Testing [C 6]
	Cause Assumption	Thickness of Covering	Detection of Steel Bar [C 7]
		Repeated Load	Loading Testing -

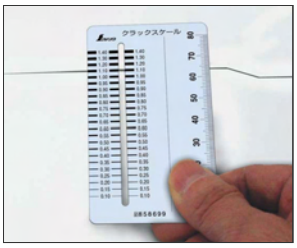
From next page, each inspection method such as [C1] to [C7] is described. The method of the degree of Carbonic acid gases and salinity in atmosphere are omitted to introduce. Loading testing will be introduced at the second year study period including the OJT (On the Job Training) activity at selected bridges.

Concrete Member | [C1] Visual Inspection

Inspection Method

Inspection Equipments

Crack Scale



Convex

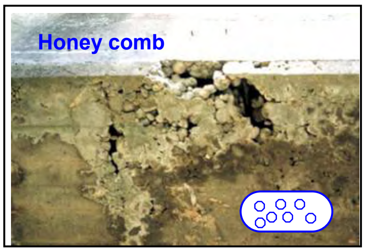
Caliper

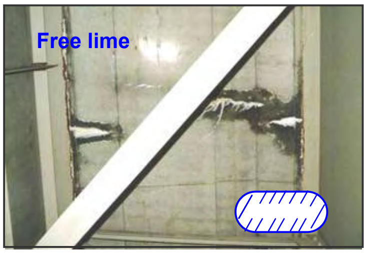
Hummer

and Camera, Bioscope, Infrared Rays Camera

Inspection Items; Types of Deterioration

Each deterioration such as crack, honey comb, fallout, separation and free lime caused by alkali-silica reaction should be measured by length, area and depth using above equipment.

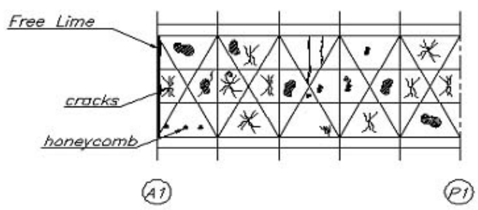




How to report

- Inspection results should be indicated as the "Deterioration Diagram" with sketch and photo, in order to identify easier in which parts of structure the deterioration appear and its size.
- Each deterioration should be used by its own legend on above figure.
- Inspection date and its method are also required to be indicated.

Deterioration Diagram



Consideration

- In the case of "Salt Corrosion", deterioration appears the place where the many chloride ion are supposed to be in-flowed.
- If the member has once been rehabilitated before, the surface is treated and covered so that deteriorations are difficult to be found. In this case, Infrared rays method should be applied.

Concrete Member	[C2] Phenol Ptyalin Method (Neutralized Depth)	For Neutralization

Inspection Method

Inspection Equipments

Caliper

Inspection Steps

1. Let the filter paper absorb the testing liquid such as Phenol Ptyalin of its 1% density with spray.
2. During drilling the concrete, concrete fragment shall be taken on the paper as above picture.
3. When the paper changed to red color, the section of concrete has been neutralized from the concrete surface to its depth.
4. The depth of neutralization will be measured with the length of drilling nozzle in accuracy of 0.1mm by Caliper.
5. This procedure should take 3 times at testing point. Result is taken by its average values.

Note: Phenol phthalein solution changes to red color with more than pH8.2 to pH10.0 in the alkaline part.

Note: If the deviation of each value against the average is over +-30%, one more procedure should be taken.

How to report

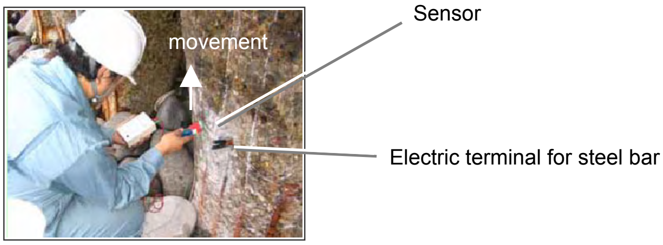
- Inspection results should include:
  - Date, Climate
  - Inspector Name
  - Inspection Point and Area
  - Inspection Results of Neutralized Depth

Concrete Member	[C3] Self-potential Method for Steel Bar Corrosion	For Neutralization, Salt Corrosion

Inspection Method

Inspection Equipments

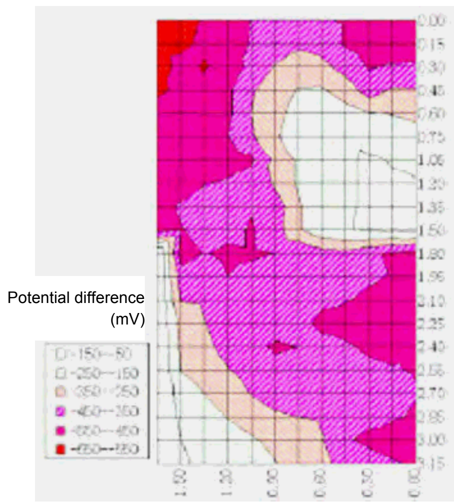
Self Potential Meter



Inspection Steps

1. Concrete surface shall always be wetted during diagnosing.
2. Electric terminal for steel bar shall be exposed to the outside.
3. Sensor shall be slowly moved on the concrete surface.
4. Result is shown as the "Potential Difference Map" by this system.

Potential Difference Map



How to report

- Steel bar corrosion is identified by the results of Self-potential difference as follows:

Potential difference (E)	Corrosion level
-250mV< E :	nothing
-350mV< E< -250mV :	point
-450mV< E< -350mV :	surface
E<-450mV :	partly short of sectional area

Concrete Member | [C4] Chloride Meter Method | For Salt Corrosion

Inspection Method

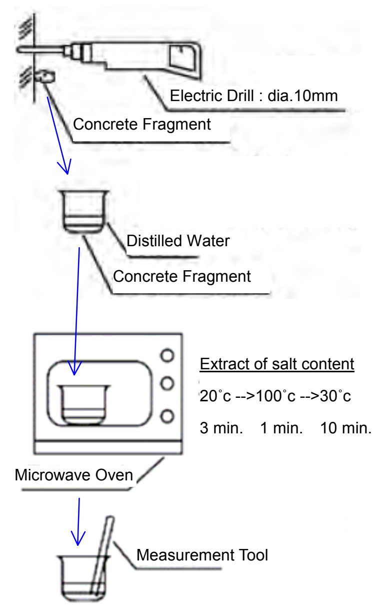
Inspection Equipments

Electric Drill : dia.10mm

Measurement Tool

Inspection Steps

1. During drilling the concrete every two (2) centimeters, concrete fragment shall be taken into a vessel more than twenty (20) grams.
2. Concrete fragment of five (5) grams shall be stirred with distilled water of ten (10) milliliters in a beaker.
3. The beaker shall be put into a microwave oven.
4. Test material shall be heated and boiled till one hundred (100) degree during three (3) minutes because of extract of salt content. And test material shall be cooled naturally.
5. Evaporated quantity of distilled water shall be revised and concentration of chloride ion will be measured with measurement tool.



How to report

- Chloride corrosion degree is identified by the results as following levels:

Level-1: concentration of chloride ion <= 1.2 kg/m3 (No corrosion)

Level-2: 1.2 kg/m3 < concentration of chloride ion (Corrosion shall start.)

Level-3: Cracks by corrosion shall occur.

Level-4: Many cracks by corrosion shall occur. Concrete around steel bars shall partly be peeled off.

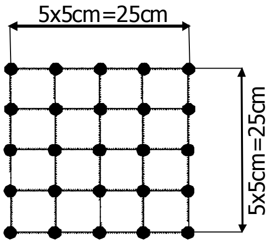
Level-5: Many cracks by corrosion shall occur. Concrete around steel bars shall almost be peeled off.

Concrete Member | [C5] Schmidt Hummer Method | For Initial Defect

Inspection Method

Inspection Equipments

Shumidt Hummer

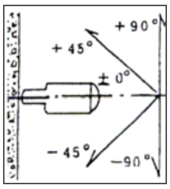


Inspection Steps

1. The Measuring part shall be prepared as smooth surface.

2. Inspector measures at 25 points/part and Measuring condition should be perpendicularity in principal.

   Note: If the points will be at an angle with concrete surface, the value "δR" shall be used for revised value.



3. Five (5) scattering values shall be deleted.

4. Remaining twenty (20) values "R" shall be averaged.

5. Standard value "Ro" can be calculated as

   "Ro = R + δR".

	δR			
R	+90deg.	+45deg.	-45deg.	-90deg.
10	—	—	—	—
20	-5.4	-3.5	+2.5	+3.4
30	-4.7	-3.1	+2.3	+3.1
40	-3.6	-2.6	+2.0	+2.7
50	-3.1	-2.1	+1.6	+2.2
60	-2.3	-1.6	+1.3	+1.7

6. Compressive strength "F" of a part will be obtained from the below formula:

   $F = -184 + 13.0 \times Ro\ (kg/cm^2)$ or

   $F = 0.098 \times (-184+13.0 \times Ro)\ (N/mm^2)$

How to report

- Inspection results should include:
  - Concrete Material Ages
  - Surface Condition of Concrete
  - Concrete Mixture Component
  - Type of Measurement Tool
  - Measurement Part and Point including the direction of penetration
  - Each Measurement Results and Average
  - Estimation Formula for Concrete Strength and Result

Concrete Member | [C6] Concrete Core Testing | For Initial Defect

Inspection Method

Inspection Equipments

Concrete Core Drill

Concrete Compaction Testing Machine

Inspection Steps

1. Core Sampling
   - Concrete core drill shall be used for acquisition of core sample.
   - Diameter of core sample should be more than thee (3) times as wide as the maximum diameter of large aggregate.
   - Height of core sample should be twice (2) of the diameter of core sample.
2. Preparation of Testing
   - Both side of sample shall be treated as capping.
   - Diameter shall be measured at the both side of sample and at the center of sample height for two directions with an accuracy of 0.1mm, then be averaged.
   - Height shall be measured at both side of sample.
   - Before the testing, sample is water curing for 40 to 48 hours.
3. Compaction Testing
   - Detail procedure is instructed by the manual of compaction machine.

Note: This method is possible to find the concrete strength directly, but it takes in consideration that core sample makes the part of lack in structure. Therefore, Acquitting of core sample should take care to avoid the place without steel bar.

Note: When the sample is already affected by chemical damaged, it is important for divided between the healthy part and damaged part.

How to report

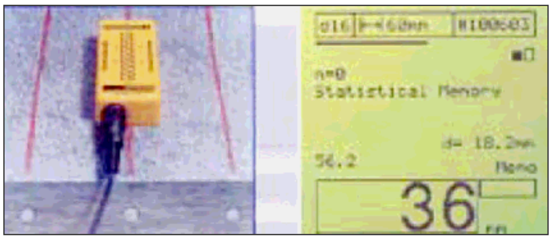
- Inspection results should include:
  - Points of Sample Acquisition and Method
  - Concrete Material Ages
  - Average Height and Diameter
  - Maximum Load Capacity
  - Compaction Strength Results
  - Curing Method and its Temperature
  - Sketch of Destruction Diagram

Concrete Member | [C7] Detection of Steel Bar: Profometer | For Initial Defect

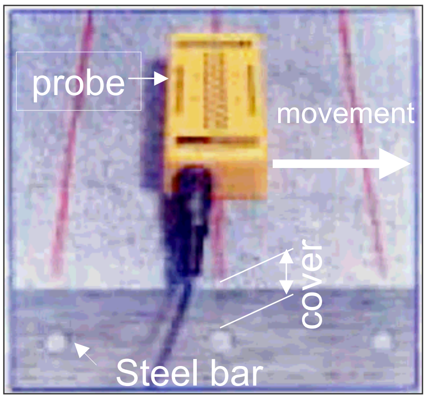
Inspection Method

Inspection Equipments

Profometer (Probe part and Display)



Inspection Steps



1. The probe will slowly be moved on the concrete surface.
2. Beep sound "pii" can be heard on steel bar.
3. Display shows the "Position of steel bars", "Cover to steel bars" and "Diameter of steel bars".

How to report

- Inspection results should include:
  - Points of Sample Acquisition and Method

10.1.4 Results of Detailed Inspections

1) Visual Inspection

A variety of damages have been observed in 10 bridge sites throughout the visual inspection. Those damages are particularly caused by multiple factors such as material deterioration, increase of the live load, scouring and lack of maintenance. Some of damages shown below are observed in all of 10 bridges. It is likely that these damages observed at only 10 bridges are common phenomenon to almost all bridges in Costa Rica.

- Asphalt overlaying has been directly carried out on surface with no removal of a previous layer.
- Expansion joints are damaged and covered by asphalt layer.
- Bearings suffer through severely inadequate conditions (weathering, soil deposited, etc)
- Some girders and some members of truss bridge have been in wet because of inappropriate drainage arrangement.
- Discoloration, rusting and peeling of paint coat on steel bridges
- Damages of deck slab (holes, cracks, free lime, water leakage were observed)

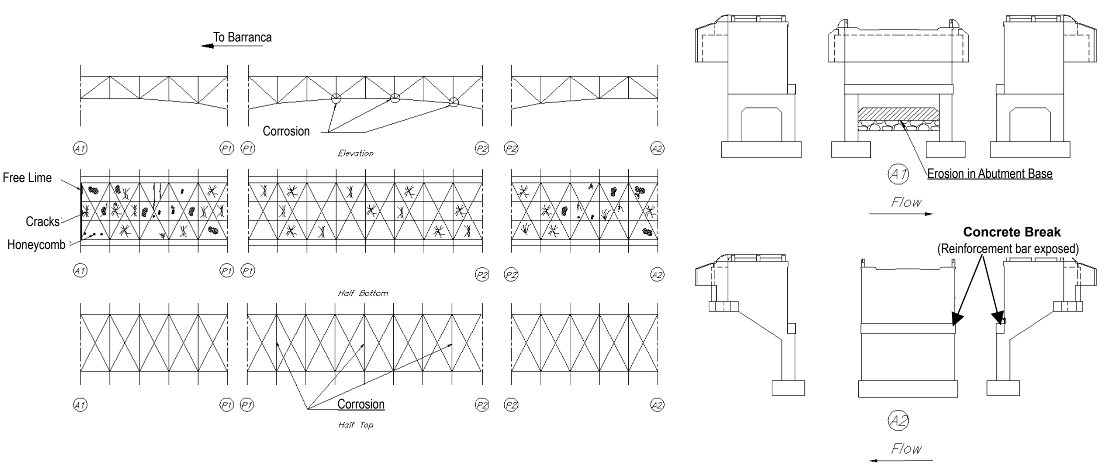
Results of the visual inspection for 10 bridges are summarized in Table 10.1.6 and more details are shown in from Table 10.1.7 to Table 10.1.16 for each bridge.

Table 10.1.6. Summary of Results of Visual Inspection for 10 Bridges

Member	Condition of deterioration	Cause of deterioration	Bridge Name Identified deterioration (Bridge No.)
Expansion joint	- Breakage	- Lack of maintenance	All 10 bridges
Railing	- Breakage	- Traffic accident	Rio Azufrado (7)
Damage of Slab	- Hole	- Fatigue by cyclic load of heavy traffic	Rio Abangares (3)
	- 2 direction crack	- Fatigue by cyclic load of heavy traffic	Rio Aranjuez (2), Rio Abangares (3) Rio Azufrado (7), Rio Puerto Nuevo (12) Rio Chirripo (26), Rio Torres(29)
	- 1 direction crack	- Fatigue by cyclic load of heavy traffic	Rio Nuevo (16) Rio Sarapiqui (19)
Steel girder	- Rising/Peeling	- Deterioration of painting	All Steel Bridges
	- Corrosion or Rusting	- Deposit soil on member	All Steel Bridges
	- Crack/Breaking	- Fatigue by cyclic load of heavy traffic	Rio Abangares (3)
	- Deformation	- Caused by earthquake	Rio Chirripo (26)
Concrete Girder	- Crack	- Fatigue by cyclic load of heavy traffic	Rio Azufrado (7) Rio Nuevo (16)
	- Deformation	- Creep of Concrete	Rio Chirripo (17)
Bearing and Bearing Base	- Corrosion/deposit soil - Leakage from expansion joint	- Lack of maintenance	All 10 Bridges
	- Breakage of bearing	- Earthquake in 1991	Rio Chirripo (26)
Pier	- Damage of surface	- Rolling stone strike against pier	Rio Sucio (20) Rio Chirripo (17)
	- Scouring of foundation	- Riverbed went down	Rio Nuevo 16)
Abutment	Collapse of slope in front of abutment		All 10 bridges
Pavement	When the pavement in earthwork section was improved or repaired, that in the bridge section also paved with 5cm overlay asphalt concrete. This maintenance work is wrong method for bridge section, because it increases dead load of bridge.		All 10 bridges

Table 10.1.7. Results of Visual Inspection for Rio Aranjuez Bridge (Route No.1 Bridge No.2)

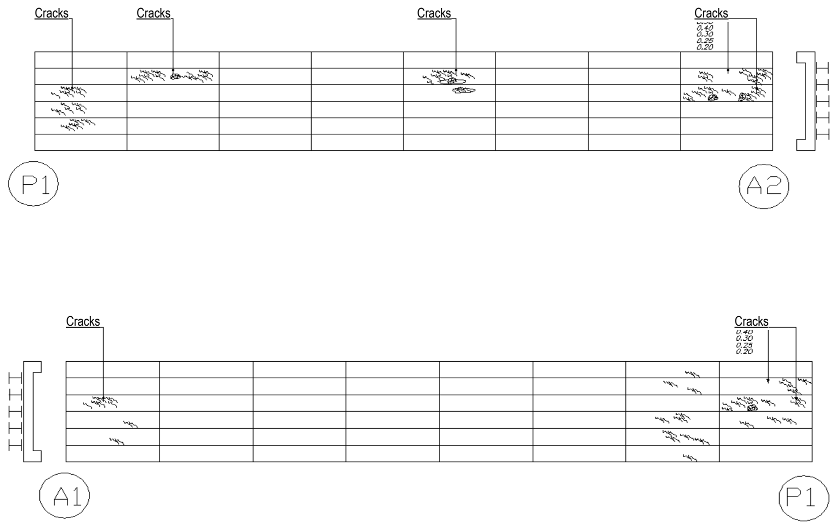
Deterioration Diagram



Accessory	- No pavement on the slab deck. Vehicles are running on slab directly. - Many cracks and holes were observed in surface of slab. - Expansion joint has been damaged.
Superstructure	- Discoloration, rusting and peeling of painting were observed in all members.
	- Many cracks and free lime were observed in slab concrete. - Cracks occurred two (2) directions (longitudinal and transversal direction)
	- Discoloration and rusting were observed in the body of Bearing. - it is very dirty condition around bearing.
Substructure	- Surface of pier is worn away by the river flow. - Cold joint was observed in Pier No 2.
	- Fissility and exposure of reinforcement were observed in the body of Abutment A2.
Foundation	- Small scouring was observed.

Table 10.1.8. Results of Visual Inspection for Rio Abangares Bridge (Route No.1, Bridge No.3)

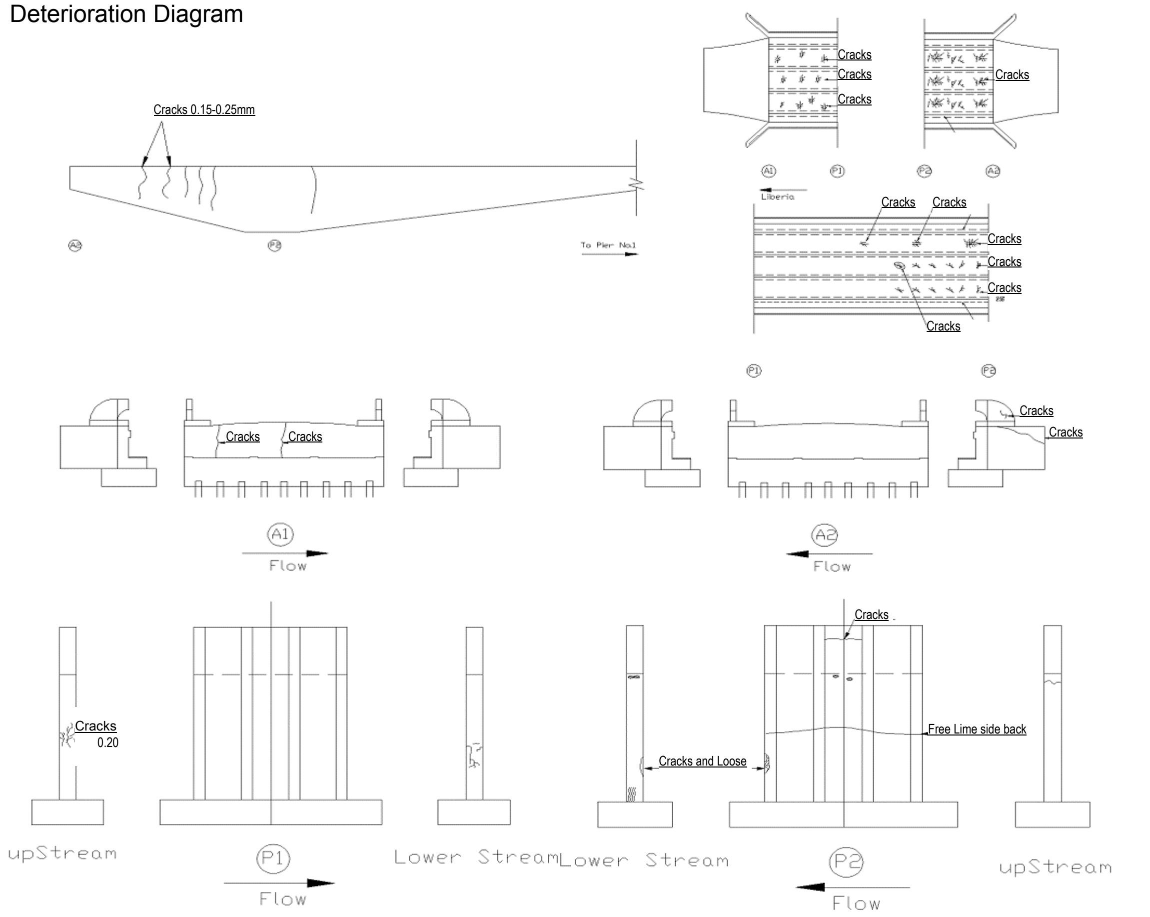
Deterioration Diagram



Accessory	- Overlay (5 cm thickness) was carried out direct on previous pavement (dead load of pavement is increased) - Many cracks and holes were observed in surface of pavement. - Water leak was observed at expansion joint and dust or soil have fill in the space of expansion joint.
Superstructure	- Discoloration, rusting and peeling of painting were observed in all members and the corrosion was observed in some parts. - Upper sway braces were damaged by hitting of big vehicles. In the section from abutment A1 to pier they have repaired.
	- Near abutment A1, big hall (3.5m(L) x 2.0m(W)) have occurred and many cracks and free lime were observed in the deck slab concrete. - Cracks occurred two (2) directions (longitudinal and transversal directions) and the width range of them is from 0.3 to 0.4 mm
	- Rivets in joint between crossbeam and stringer have been lost or repaired and some joint plates cracked.
	- Discoloration and rusting were observed in the body of bearing. - It is very dirty condition around bearings.
Substructure	- Any special damage has not been observed
Foundation	- Any special damage has not been observed

Table 10.1.9. Results of Visual Inspection for Rio Azufrado Bridge (Route No.1, Bridge No.7)

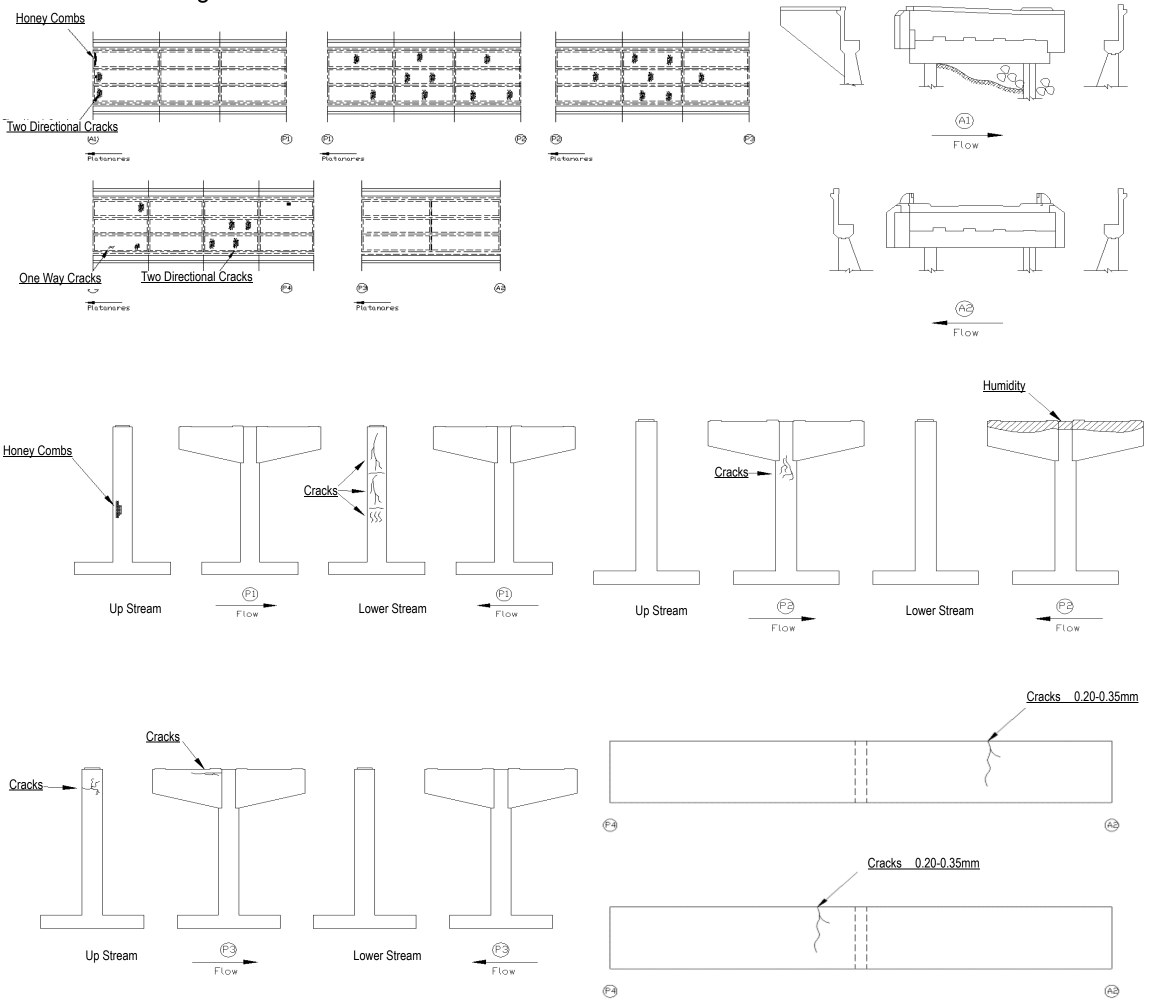
Deterioration Diagram



Accessory	- Railing has been damaged by traffic accidents.
Superstructure	- Many cracks and free lime were observed in slab concrete. - Cracks occurred two (2) directions (longitudinal and transversal directions) and the width range of them is from 0.2 to 0.3 mm - Exposure of reinforcement caused by re-bar corrosion was observed iIn overhanging slabs. - In the side of girder, many cracks were observed at upper portion of girder. These cracks run horizontal direction at 50cm interval and the width range of them is from 0.2 to 0.3 mm.
Substructure	- Some cracks were observed in Pier No 2. - There is no abutment. There is only cross beam at end of bridge, which protected soil. So girders of side span are cantilever beam.
Foundation	- Any special damages were not observed

Table 10.1.10. Results of Visual Inspection for Rio Puerto Nuevo Bridge (Route No.2 Bridge No.12)

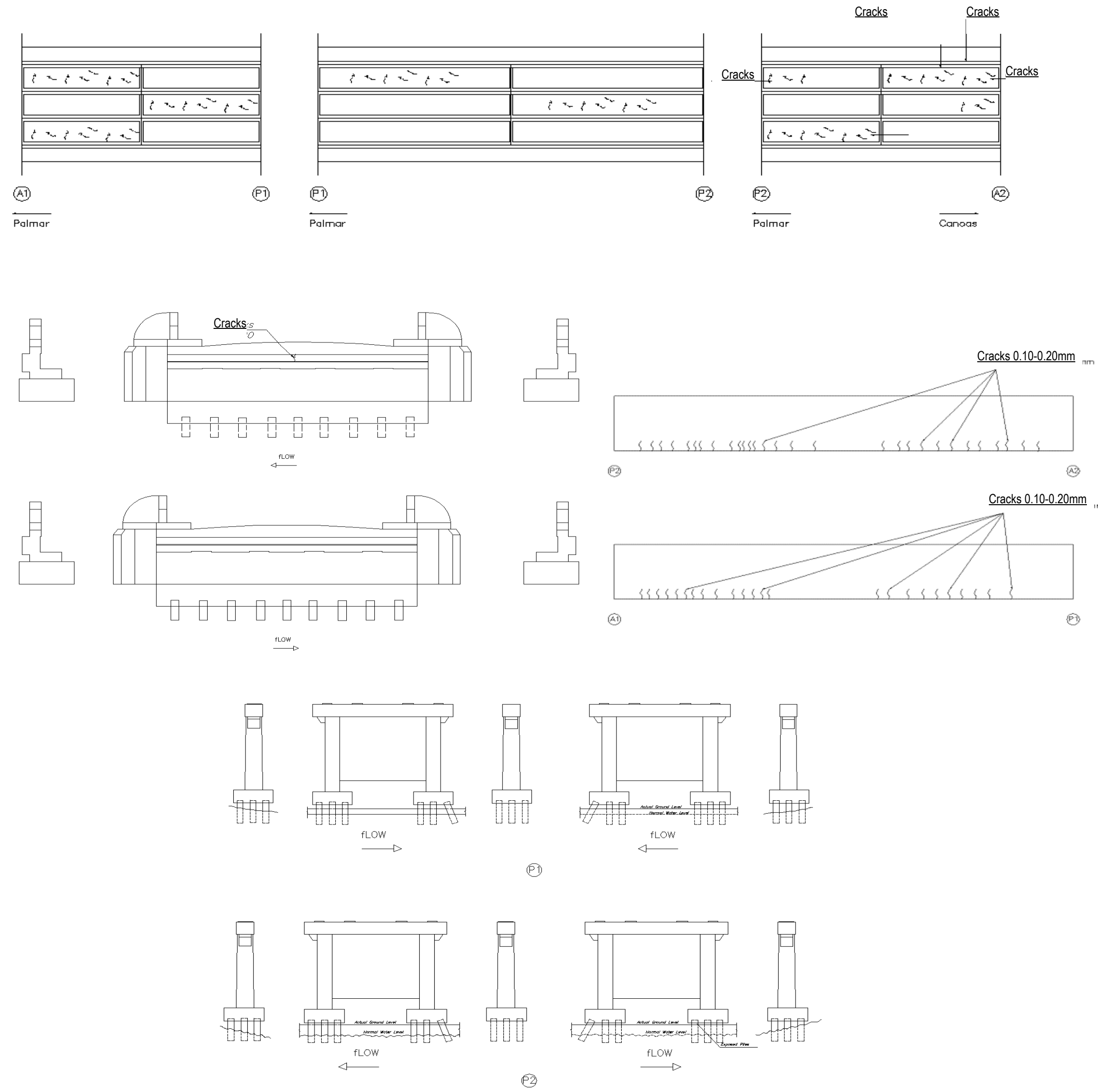
Deterioration Diagram



Accessory	- Expansion joints at both abutments A1 and A2 were covered by overlay (5 cm thickness) asphalt concrete. - There are not expansion joints at the Pier No.2 and 4. There are spaces (20 mm) for expansion. - No Pavement on the slab deck. Vehicles run directly on slab - Water leak was observed at expansion joint.
Superstructure (Concrete girder bridge)	- There is a crack which width is 0.3 mm in internal girder near abutment A2. In external girder any crack was not observed. - In the slab, cracks occurred diagonal direction and its width is 0.2 to 0.3 mm.
Superstructure (Steel girder bridge)	- In the slab, cracks occurred two (2) directions (longitudinal and transversal directions) with around 50 cm interval each and their widths are 0.2 to 0.3 mm - Peelings of painting were observed in some parts of girder and cross beam.
Substructure (Pier)	- Vertical direction cracks were observed in all piers, and in beam of Pier No.3, some horizontal cracks were observed. - Upper portion of beam of Pier No.2 was wet condition by water, which has leaked from the joint of superstructure.
Substructure (Abutment)	- Slope around abutment was collapsed - Soil has accumulated on the bearing base and some bearings were overwhelmed by an accumulation of soil.
Foundation	- Any special damages were not observed

Table 10.1.11. Results of Visual Inspection for Rio Nuevo Bridge (Route No.2, Bridge No.16)

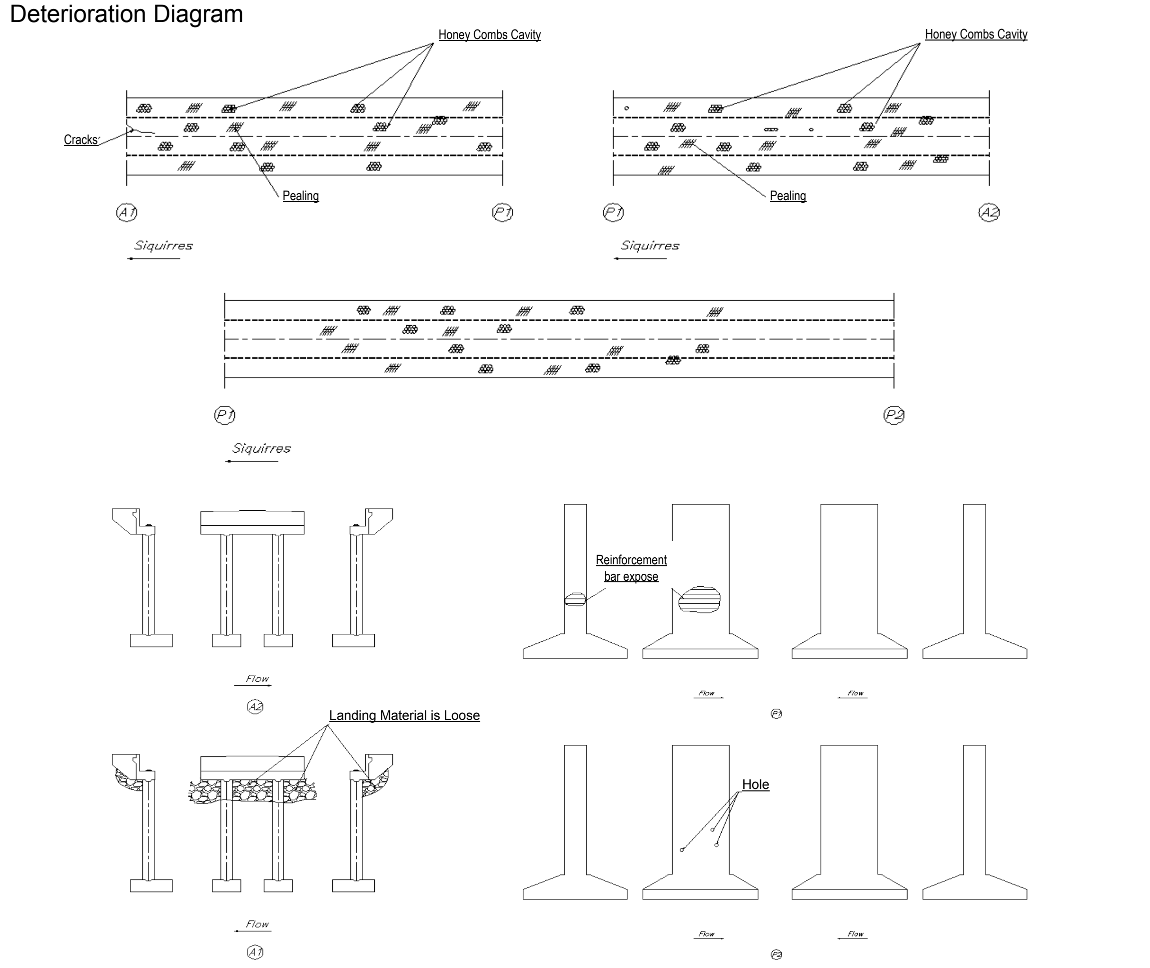
Deterioration Diagram



Accessory	- Overlay (5 cm thickness) was carried out direct on previous pavement (Dead load of pavement was increased) - Water leak was observed at expansion joint.
Superstructure	- Cracks were observed in slab concrete with width range is from 0.15 to 0.25 mm. - In the lower part of girder, many cracks were observed. Its width range is from 0.1 to 0.2mm. These cracks occurred horizontal direction and some of them run 50cm interval and its width rang is from 0.2 to 0.3 mm. - Rusting and corrosion were observed in the body of bearing. - It is very dirty condition around bearing
Substructure	- Any special damage has not been observed
Foundation	- River bed is scoured and gone down more than 3 m, so piles at pier No.1 and No.2 protrude from riverbed more than 2m.

Table 10.1.12. Results of visual inspection for Rio Chirripo Bridge(Route No.4, Bridge No.17)

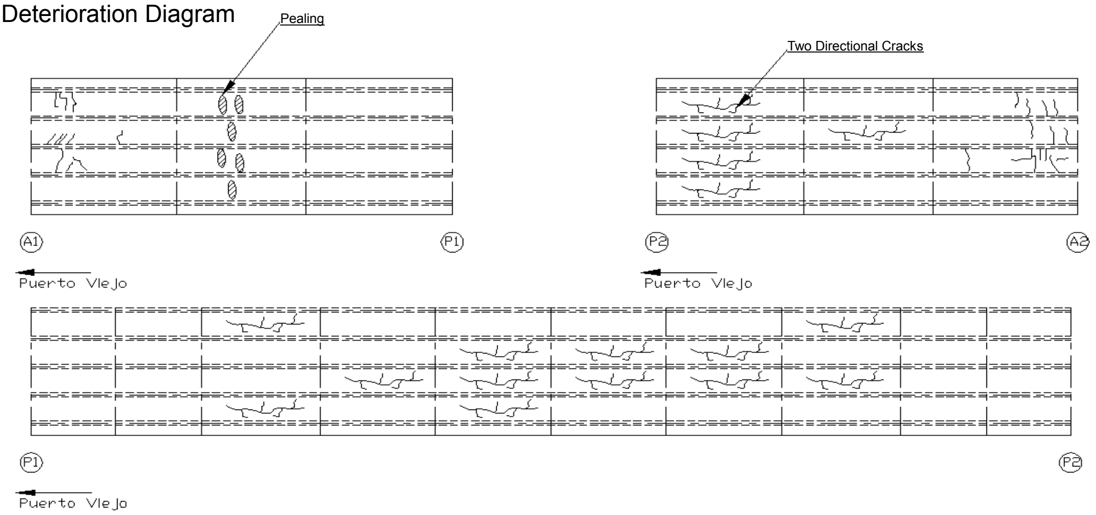
Deterioration Diagram



Accessory	-	Expansion joint has been broken and covered by Overlay asphalt concrete. Pavement has removed and slab surface has appeared in some place. Soil has deposited both side of surface and plants were grown around drain
Superstructure	-	In middle span (P1 to P2) deformation was observed Clack was observed in bottom slab near abutment No2 Free lime was observed at construction joint in bottom slab near pier No1 Concrete fissility was observed around hole in bottom slab for drain water inside box. A hole that seem to correct core in top slab was not repaired. Some defects in construction (honeycomb, not remove form etc) were observed. Grout hose protrudes from surface of web, it must be cut inside concrete of web and covered by mortal or concrete.
Substructure	-	Water leak was observed at the place of expansion joint and it is wet condition around bearing. Surface of pier was damaged by stone attacking and re-bar has been appeared in pier No2. Some holes that seem to correct core in pier No.1 were not repaired.
Foundation	-	Small scouring was observed.

Table 10.1.13. Results of Visual Inspection for Rio Sarapiqui Bridge (Route No.4, Bridge No.19)

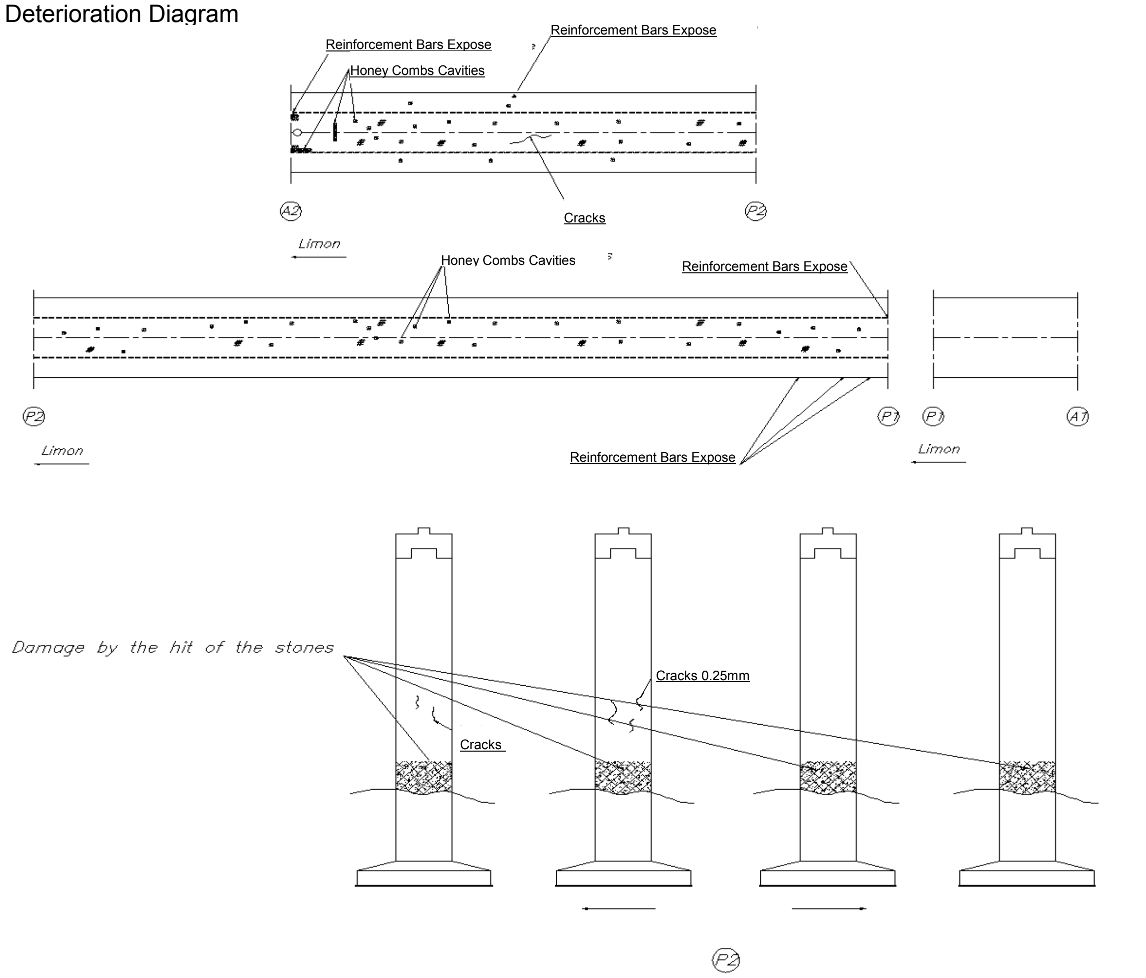
Deterioration Diagram



Accessory	- Expansion joints in both abutment A1 and A2 were covered by Overlay (5 cm thickness) asphalt concrete and they do not work. - When heavy vehicle passed middle of bridge, bridge surface lifted at the edge of bridge. - Water leak was observed at expansion joint.
Superstructure	- In the slab, cracks occurred transversal direction with width of about 0.2mm.
	- Discoloration, rusting and peeling of painting were observed in all members. And in some part, the corrosion was observed. - Losses of cross section reduced by corrosion were observed in main girder. - Main girder deformation was observed.
	- Discoloration and rusting were observed in the body of bearing. - It is very dirty condition around bearing.
Substructure	- Any special damage has not been observed
Foundation	- Small scouring was observed.

Table 10.1.14. Results of Visual Inspection for Rio Sucio Bridge (Route No.32, Bridge No.20)

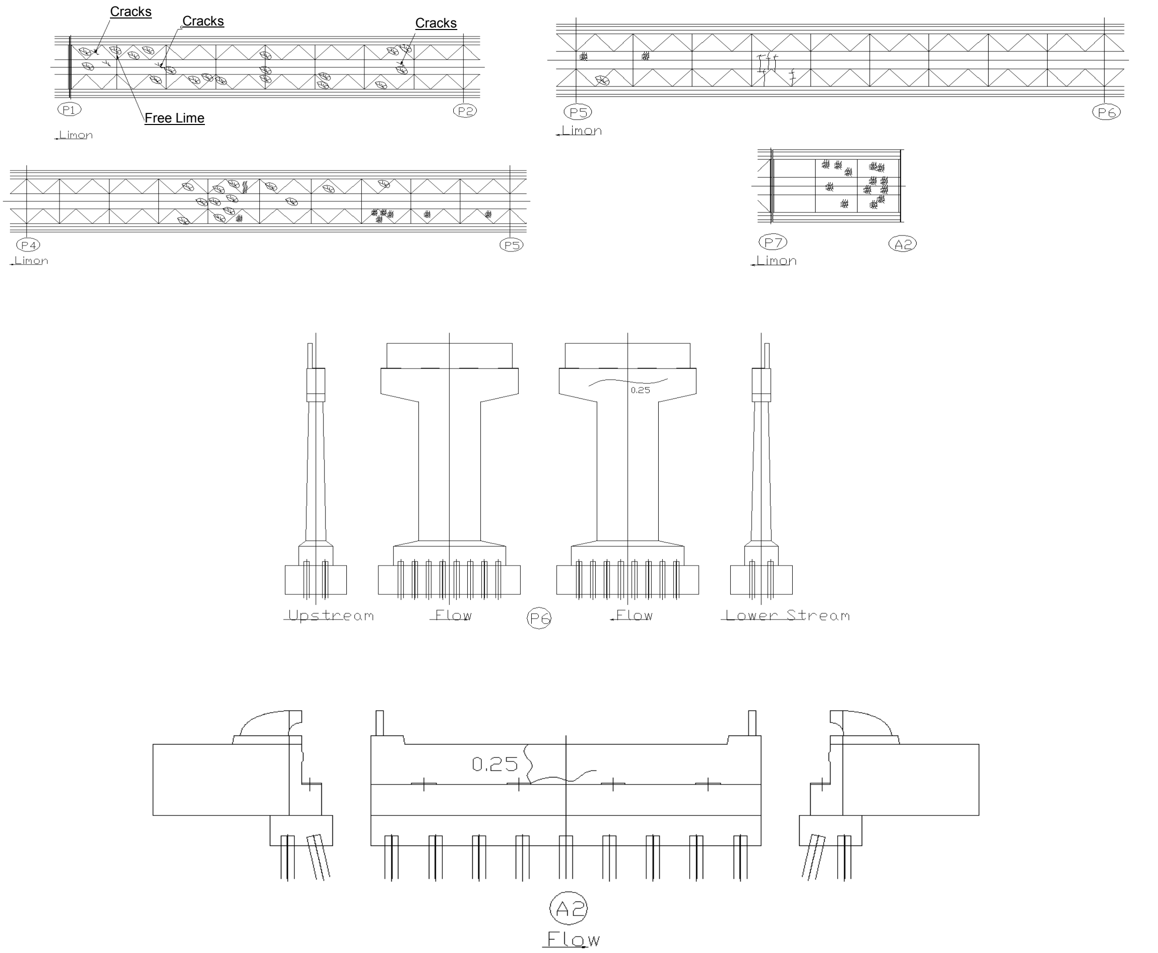
Deterioration Diagram



Accessory	- Expansion joints were broken and covered by overlay asphalt concrete.
Superstructure	- Free lime and exposure of reinforcement were observed in bottom slab - Concrete fissility was observed around hole for drainage in bottom slab inside box girder. - Some defects in construction (honeycomb, not remove form etc) were observed. - Grout hose protrudes from surface of web, it must be cut inside concrete of web and covered by mortal or concrete.
Substructure	- Some cracks were observed in Pier No.2. - Water leak was observed at expansion joint in Abutment No.2 and it is wet condition around bearings. - Surface of pier was damaged by stone attacking. Maximum depth of concrete fissility is about 10 cm.
Foundation	- Small scouring was observed.

Table 10.1.15. Results of Visual Inspection for Rio Chirripo Bridge (Route No.32, Bridge No.26)

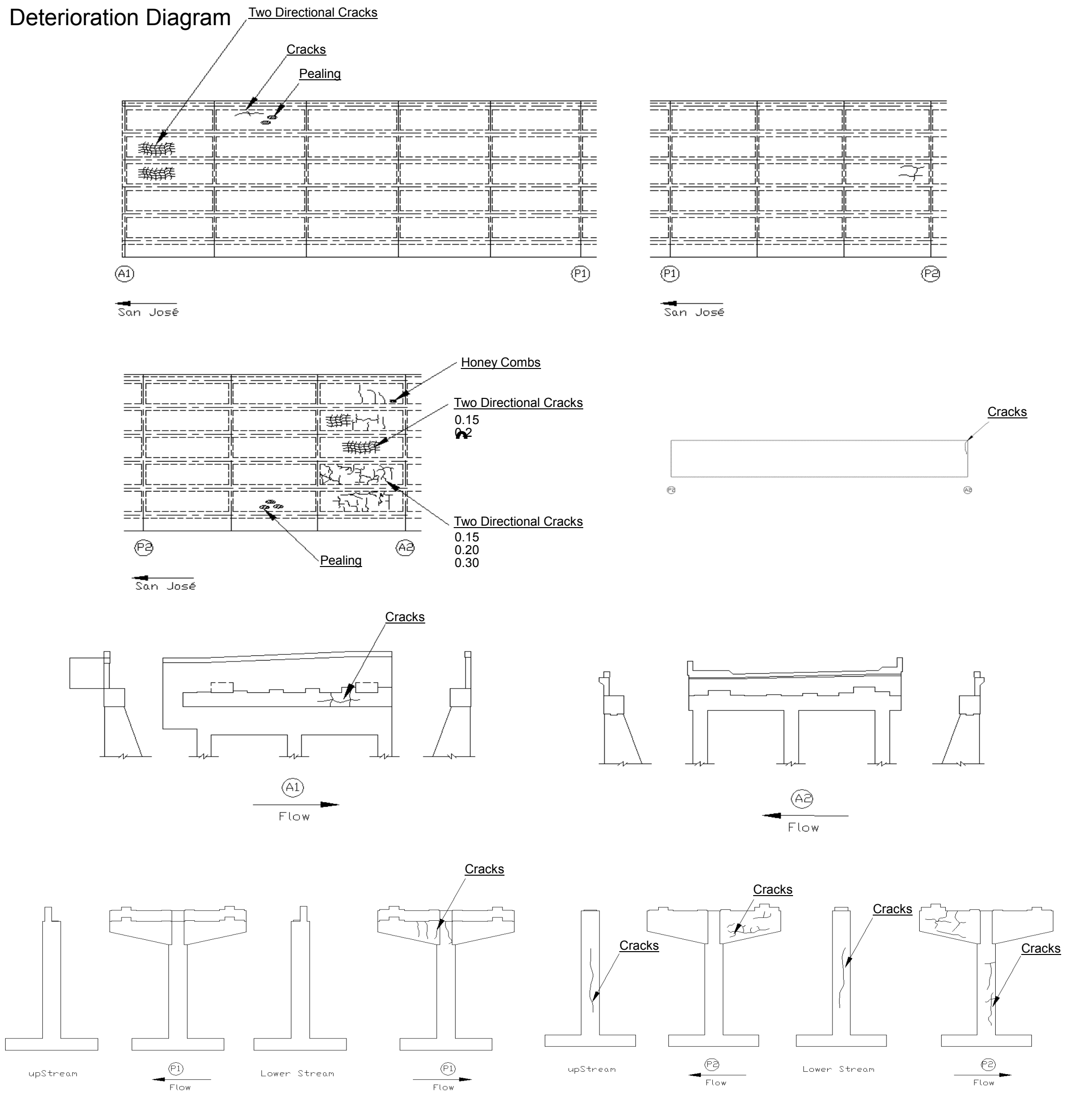
Deterioration Diagram



Accessory	- Expansion joints at both abutments A1 and A2 were covered by Overlay (5 cm thickness) asphalt concrete. And there have not worked.
Superstructure	- Continuance girder has been moved to Limon side and upstream side about 10 cm. End of continuance girder at Limon side has bumped into Pier No.1 - Side span, which is simple span bridge, has supported temporarily by H-beam pier. - In the some parts of slab, two (2) directions (longitudinal and transversal directions) cracks were observed and their widths are from 0.2 to 0.3 mm. - Water leak was observed at joint of slab concrete. - All bearings have been broken.
Substructure	- Horizontal direction cracks were observed in some piers. - Soil has accumulated on the bearing base and bearings were overwhelmed by an accumulation of soil at abutment and some piers.
Foundation	- Small scouring was observed.

Table 10.1.16. Results of Visual Inspection for Rio Torres Bridge (Route No.218, Bridge No.29)

Deterioration Diagram



Accessory	- Top pipe of railing was lost. - Expansion joint has been broken and covered by overlay asphalt concrete.
Superstructure	- Two direction cracks were observed in slab concrete. The width range of them is from 0.15 to 0.3 mm.
Substructure	- Many cracks were observed at transversal beam in Pier No.1, column of Pier No.2 and Abutment No.2. - Width of bearing base is too small.
Foundation	- Scouring was observed at pier No.1

2) Strength of Concrete and Thickness of Steel Members

a) Strength of Concrete

According to “General Notes” in original drawings, concrete of substructures is the class “A” for the bridges located in Route 1 and Route 2. Moreover, in Rio Sarapiqui Bridge, General Notes indicate that it is the class ”A”, which its design strength is 210kg/cm^2 (fc’=210kg/cm^2). In Rio Chirripo Bridge and Rio Sucio Bridge, although the design concrete strength is not clearly indicated for their superstructures in the drawings, it can be easily supposed, by reading the information from the same type bridges in year of construction, with which the concrete strength of superstructure is designed with 350kg/cm^2.

Actual concrete strength has been examined with two (2) kinds of testing method, one is the Schmidt Hammer Method, another one is the Compression Testing Method using concrete cores, taken from body part of the bridges. The Schmidt Hammer Method has been carried out three (3) or four (4) times at all 10 bridge sites and cores have been sampled at 3 bridge sites listed below.

- Rio Azufrado Bridge (No.7, RouteNo.1) : Pier No.2 upstream side
- Rio Sarapiqui Bridge (No.19, Route 4) : Abutment No2
- Rio Sucio Bridge (No.20, Route 32) : Superstructure (P1)

Figure 10.1.5 shows results of both tests. The tests result of Rio Sarapiqui Bridge and Rio Sucio Bridge are almost same except Rio Azufrado Bridge Generally, compression strength led by the Schmidt Hammer Method tends to be bigger than that of cores compression test, since conditions of surface composition i.e. size of sand and aggregate and material age of the concrete have dominant influence upon results of the Schmidt Hammer Method.

The strength of concrete is generally governed by the strength of cement paste. Accordingly, the strength of concrete tends to increase under the wet condition with its age growing up. However, the hardness of concrete surface tends not to increase, whereas the strength going up. Moreover, the hardness of surface in dry condition tends to be higher than in wet condition.

In Rio Azufrado Bridge, which is located on route 1 and near border of Nicaragua, detailed inspection was carried out in dry season when concrete is in dry condition. On the other hand, Rio Sarapiqui Bridge and Rio Sucio Bridge are located in the Rain forest area. Moreover, the Rio Azufrado Bridge was constructed in 1953, so it is older than other two bridges. This is a reason why the difference between both test results are observed.

Those results of both compression tests ensure that actual compressive strength meets the strength specified in General Notes of original drawing, and that there are thoroughly no problems regarding concrete strength in 10 bridges.

Simultaneously to implementing the “Cores” compression testing, the Static Modulus of Elasticity Tests in compression was carried out by using concrete cores that were sampled at Rio Sucio Bridge. This test results are shown in Table 10.1.16.

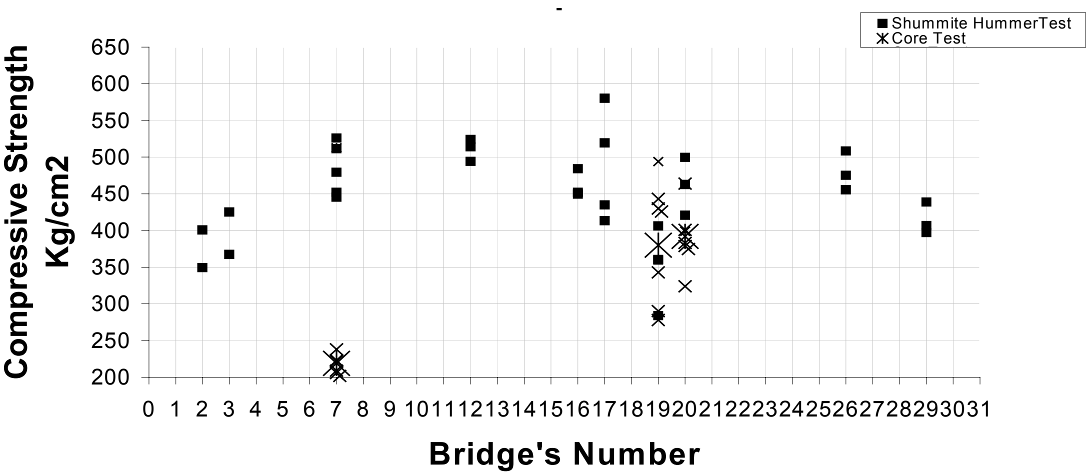


Figure 10.1.5. Result of Compression Test

Table 10.1.17. Static Modulus of Elasticity Tests of Concrete in Compression

No.	Static Modulus of Elasticity (kg/cm^2)	Compressive Strength (kg/cm^2)
1	447,013	383
2	442,337	380
3	488,416	465
4	591,419	324
5	469,100	400
Average	487,657	390

b) Thickness of Steel Members

Since there is few information in MOPT regarding dimension of members in Rio Abangares Bridge, dimension of each member has been totally measured in site so as to establish analysis models.

3) Carbonation

The Carbonation Test has been carried out at 10 bridge sites. Testing results indicate that Rio Aranjuez Bridge (No.2) in route 1 is the most affected bridge by carbonation as shown in Table 10.1.18. Results show and list that the bridges in route 1 tend to be affected more severely than those in other routes. The bridges in route 32 are the next on the list and those in route No.4 and 218 continue. The bridges in route 2 are not affected. It seems that the influence of carbonation is likely to be proportionately to traffic volume.

Table 10.1.18. Result of Carbonation Test

Route	Bridge No.	Bridge Name	Place	Depth (cm)	Average (cm)
1	2	Rio Aranjuez	Abutment No.1	6.06	4.945
			Pier No.1	3.83	
	3	Rio Abangares	Abutment No.2	3.75	3.175
			Pier No.2	2.6	
	7	Rio Azufrado	Pier No.2	3.57	3.230
			Girder	2.89	
2	12	Rio Puerto Nuevo	Abutment No.2	0.6	0.433
			Pier No.1	0.4	
			Pier No.4	0.3	
	16	Rio Nuevo	Abutment No.1	0.1	0.433
			Abutment No.2	0.2	
			Pier No.1	1	
4	17	Rio Chirripo	Abutment No.1	1.5	1.050
			Abutment No.2	0.6	
	19	Rio Sarapiqui	Abutment No.1	0.1	0.100
			Abutment No.2	0.1	
32	20	Rio Sucio	Pier No.1	1.5	1.567
			Pier No.2	1.8	
			Box Girder	1.4	
	26	Rio Chirripo	Pier No.2	4.53	1.510
			Pier No.5	0	
			Pier No.6	0	
218	29	Rio Torres	Pier No.2	1.2	0.800
			Girder	0.4	

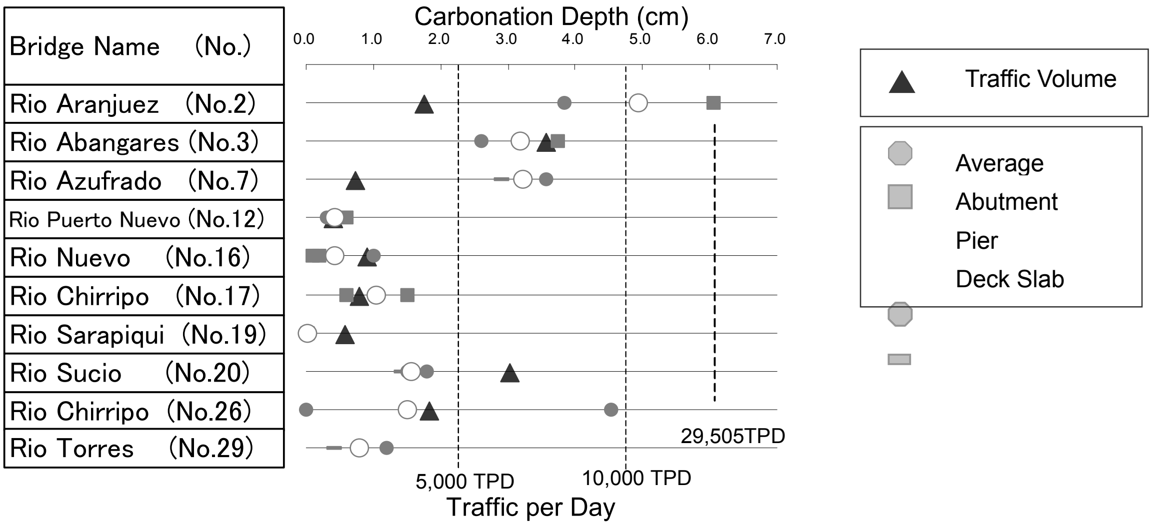


Figure 10.1.6. Result of Carbonation Test

4) Deformation of Girder

Survey team in MOPT and Study team jointly measured deformation of girder in Rio Chirripo Bridge (No.17). Figure 10.1.7 shows the survey results. The deformation in Rio Chirripo Bridge presents about 10 to 15cm in the middle of the center span between P1 and P2, and the difference between pier and abutment is around 20 to 22cm. According to design drawings of this bridge, the difference in height between the levels positioned at pier and at abutment is 22cm. Therefore, the gradient of side span is almost same as the original drawing.

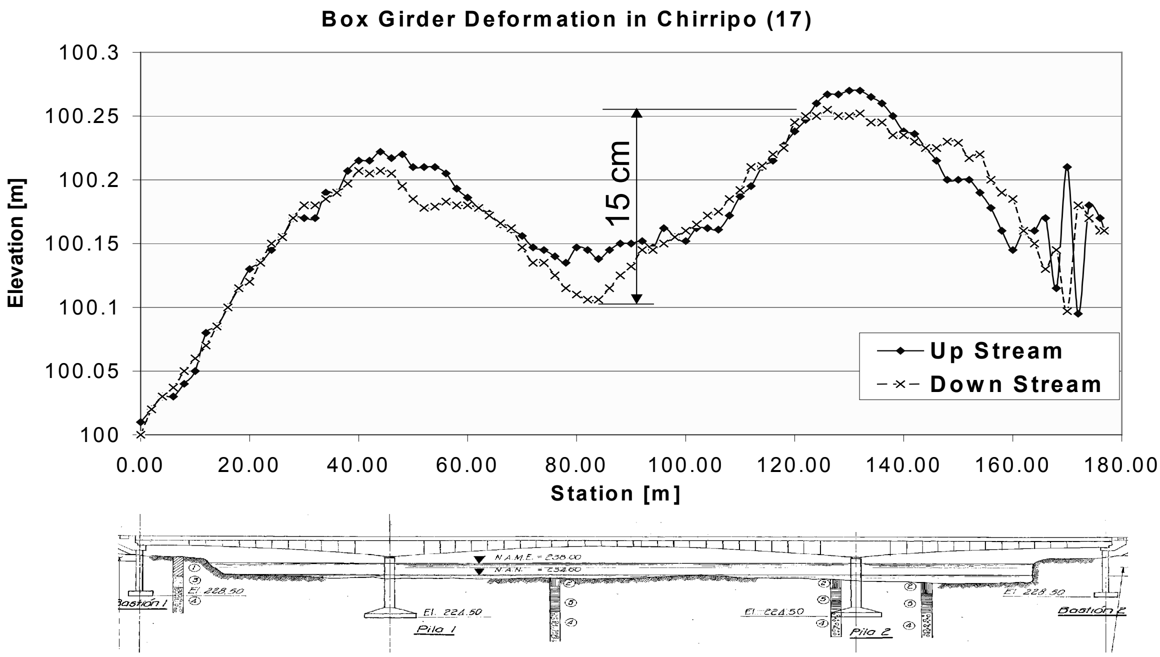


Figure 10.1.7. Deformation of Box Girder in Rio Chirripo Bridge (No.17)

10.1.5 Analysis of Load Carrying Capacity of Bridge

1) Method of Analysis

In order to examine the load carrying capacity or the safety of 10 bridges against the present live load (HS20+25%) as well as the earthquake load, a working load or a working stress should be calculated in computer-aided structural analysis for main members and compared with a resisting force or an allowable stress.

Prior to computing the working force or the working stress, the necessary drawings are to be collected to make an analysis model. The all drawings of the 8 selected bridges provide enough information to make the analysis model. However, no enough information is available on drawings of Rio Abangares Bridge and Rio Chirripo Bridge to make the model.

In the case of Rio Abangares Bridge, additional information about shape thickness and dimensions of some members, which are required to make the model has been investigated by MOPT counterparts and the study team. And in the case of Rio Chirripo Bridge, the necessary data to analyze a working load or a working stress has been obtained from drawings of Rio Virilla Bridge that has same type and same dimension as Rio Chirripo Bridge.

Outline of analysis including bridges to be analyzed and methods of evaluation are shown below.

Table 10.1.19. Outline of Analysis and Method of Evaluation

Standard		AASHTO (17th Edition 2002)		
Load	Dead Load	Original Design ; Original drawings New dead load ; Original drawings + Asphalt Pavement (5 cm)		
	Live Load	Original Design ; HS15 or HS20 Latest live load ; HS20+25%		
	Earthquake	The Seismic Code of Costa Rica		
Software		SAP2000		
Analysis bridge	Steel Girder	Truss Deck Bridge		Aranjuez Bridge (No.2)
		Truss Bridge	Deck type	Abangares Bridge (No.3)
			Through type	
		I beam Bridge	Composite	Puerto Nuevo Bridge (No.12)
			Continuous	Chirripo Bridge (No26)
			Gerber beam	Sarapiqui Bridge (No19)
	Concrete Girder	RC I Beam Bridge		Nuevo Bridge (No.16)
		RC Rigid Frame Bridge		Azufrado Bridge (No.7)
		PC I Beam Bridge		Torres Bridge (No.29)
		PC Box Girder Bridge		Chirripo Bridge (No.17), Sucio Bridge (No.20)

Method of Evaluation	Superstructure	-Truss Bridge -Composite I Beam	Compare the working force with the resisting force or Compare the working stress with the allowable

		-PC Girder -RC Bridge -Steel I Beam	stress
	Substructure	Compare the working force with the resisting force	
	Foundation	Spread Foundation	The stability was evaluated by the Load eccentricity Normal : e < B/6 Earthquake : e $\leq$ B/3 e : Load eccentricity B : Width of Footing
		Pile Foundation	Compare Axial force of pile with the axial capacity force

2) Result of Analysis

a) Deck Slab

The analysis results of deck slab for 10 bridges are as shown in Table 10.1.20. This table gives the condition of load carrying capacity of deck slab against latest live load, that is HS20+25%, and it is summarized as below.

- According to the computer-aided analysis, , the working force obtained with new live load (HS20+25%) ranks 20% or 30% bigger than the load carrying capacity of the deck slab designed with HS15 of live load in route No.1 and No.2.
- Suppose that the deck slab was originally designed with HS20, it would be less than the load carrying capacity of deck slab.

Table 10.1.20. Analysis Results of Deck Slab

Route	Bridge No.	Bridge Name	Original Condition					M(HS20+25%) +Pavement(5cm) (ft-kip)	Capacity	Required thikness of slab (cm)	M(HS20+25%)/MC	Damage Condition	year	Traffic Volume
			Live Load	Span Length (m)	Thickness (cm)	Re-bar area (cm2/m)	M (ft-kip)		Mc (ft-kip)					
1	2	Rio Aranjues	HS-15	1.951	17.78	11.13	2.9	4.6	3.4	24.1	1.35	×	1955	4,000
	3	Rio Abangares (129ft Bridge)	HS-15	1.372	16.51	11.13	2.1	3.5	3.1	18.6	1.13	× ×	1953	8,120
		Rio Abangares (200ft Bridge)	HS-15	1.372	16.51	11.13	2.1	3.4	3.1	18.1	1.10	×	1953	8,120
	7	Rio Asufrado	HS-15	1.707	16.51	11.13	2.5	4	3.1	21.3	1.29	×	1955	1,660
2	12	Rio Puerto Nuevo (Steel Bridge)	HS-15	1.585	17.78	11.13	2.4	3.8	3.4	19.9	1.12	△	1961	930
		Rio Puerto Nuevo (Concrete Bridge)	HS-15	1.707	16.51	11.13	2.5	4	3.1	21.3	1.29	○	1961	930
	16	Rio Nuevo	HS-15	1.707	16.51	11.13	2.5	4	2.97	22.2	1.35	○	1961	2,060
4	17	Rio Chirripo	HS-20	-	-	-	-	-	-	-	-	-	1978	1,800
	19	Rio Sarapiqui	HS-20	1.219	17	11.13	2.5	3.1	3.1	17.0	1.00	○	1978	1,330
32	20	Rio Sucio	HS-20	-	-	-	-	-	-	-	-	-	1978	6,900
	26	Rio Chirripo	HS-20	1.89	17	16.49	3.7	4.5	4.7	16.3	0.96	○	1978	4,185
218	29	Rio Trres	HS-20	1.554	18	15.34	3.1	3.8	6.3	10.9	0.60	×	1980	29,505

Note:
XX ; observed holl and two direction crack with 30 cm space
X ; observed two direction crack with 30 cm space
△: observed two direction crack with 50 cm space
○: observed one direction crack

b) Main Frame and Floor System of Truss Bridge

Table 10.1.21. summarizes the working forces of each member by original live loud (HS15 or HS20) and HS20+25%, and the comparison between them.

- In the case of Rio Aranjuez Bridge

  The compression members bring problems. Especially the working force in the Lower chord member and vertical member of Main Frame is 30% to 50% larger than the resisting force. And the working force of the Floor System, such as the Floor Beam and the Stringer exceeds around 15% larger than the resisting force.

- In the case of Rio Abangares Bridge

  The compression members also bring problems. The working force of the Floor System is 30% to 50% larger than the resisting force, whereas the working force of other main frame members are slightly over than the resisting force.

Table 10.1.21. Analysis Result of Superstructure (Truss Frame)

Route	Bridge No.	Bridge Name	Member	Original Condition			(HS20+25%) +Pavement(5cm)		Capacity		M(HS20+25%)/MC	V(HS20+25%)/VC	year	Traffic Volume
				Live Load	M (ft-kip) or T(kip)	V (Ksi)	M (ft-kip) or T(kip)	V (Ksi)	Mc (ft-kip) or Tc(kip)	Vc (Ksi)				
1	2	Rio Aranjues	Floor Beam	HS-15	210	40	300	60	265.3	126.2	1.13	0.48	1955	4,000
			Exterior beam	HS-15	99.6	27.4	156.6	43.2	148.7	84.8	1.05	0.51		
			Interior Beam	HS-15	110	30.1	170.1	46.9	148.7	84.8	1.14	0.55		
			Truss(Top)max	HS-15	109	−	139	−	647	−	0.21	−		
			Truss(Top)min	HS-15	−124	−	−161	−	−499	−	0.32	−		
			Truss(Vretical)max	HS-15	−252	−	−303	−	−299	−	1.01	−		
			Truss(Vretical)min	HS-15	−319	−	−413	−	−299	−	1.38	−		
			Diagonal Smax	HS-15	211	−	269	−	261	−	1.03	−		
			Diagonal Smin	HS-15	−147	−	−162	−	−157	−	1.03	−		
			Diagonal Cmax	HS-15	91	−	112	−	176	−	0.64	−		
			Diagonal Cmin	HS-15	−57	−	−84	−	−85	−	0.99	−		
			Truss(Lower S)max	HS-15	−230	−	−271	−	−252	−	1.08	−		
			Truss(Lower S)min	HS-15	−294	−	−378	−	−252	−	1.50	−		
			Truss(Lower C)max	HS-15	184	−	237	−	288	−	0.82	−		
			Truss(Lower C)min	HS-15	−191	−	−250	−	−207	−	1.21	−		
	3	Rio Abangares (129ft Bridge)	Floor Beam	HS-15	573	52	861	79	761	189	1.13	0.42	1953	8,120
			Exterior beam	HS-15	−	−	−	−	−	−	−	−		
			Interior Beam	HS-15	102	26	158	40	119	71.8	1.33	0.56		
			Truss(Top)max	HS-15	−	−	−	−	−	−	−	−		
			Truss(Top)min	HS-15	−	−	−	−	−	−	−	−		
			Truss(Lower)max	HS-15	−	−	−	−	−	−	−	−		
			Truss(Lower)min	HS-15	−	−	−	−	−	−	−	−		
			Diagonalmax	HS-15	−	−	−	−	−	−	−	−		
			Diagonalmin	HS-15	−	−	−	−	−	−	−	−		
		Rio Abangares (200ft Bridge)	Floor Beam	HS-15	700	65	1047	100	818	282	1.28	0.35		
			Exterior beam	HS-15	−	−	−	−	−	−	−	−		
			Interior Beam	HS-15	151	31	231	48	166	94	1.39	0.51		
			Truss(Top)max	HS-15	−540	−	−600	−	−610	−	0.98	−		
			Truss(Top)min	HS-15	−620	−	−700	−	−610	−	1.15	−		
			Truss(Lower)max	HS-15	300	−	310	−	362	−	0.86	−		
			Truss(Lower)min	HS-15	250	−	250	−	362	−	0.69	−		
			Diagonalmax	HS-15	−	−	−	−	−	−	−	−		
			Diagonalmin	HS-15	−	−	−	−	−	−	−	−		

Note 1) "+"; Tension, "-"; Compression

2) Shade means that the working force is larger than the resisting force.

3) is more than 30% large

c) Steel I Beam Girder Bridge

The analysis results of Steel I Beam Girder Bridge are as shown in Table 10.1.22. The table indicates the condition of load carrying capacity of the Steel I Beam Girder Bridge against live load (HS20+25%) and summarizes the stress condition as below.

- In case the original design live load is considered as HS15, the working force or the working stress computed by live load (HS20+25%) in Steel I Beam Girder and in Composite Steel I Beam Girder are 30% to 50% larger than the load carrying capacity of both type girders.
- In the case of the original design live load with HS20, the working force by HS20+25% is 10% larger than the load carrying capacity of both type girders.
- In Rio Puerto Nuevo Bridge, its working stress ranks is than the allowable stress even if the original live load is HS15.

Table 10.1.22. Analysis Result of Superstructure (Steel I Beam Girder)

Non Composite Steel I Beam Girder

Route	Bridge No.	Bridge Name	Member	Original Condition			(HS20+25%) +Pavement(5cm)		Capacity		M(HS20+25%)/MC	V(HS20+25%)/VC	year	Traffic Volume
				Live Load	M (ft-kip)	V (Ksi)	M (ft-kip)	V (Ksi)	Mc (ft-kip)	Vc (Ksi)				
2	12	Rio Puerto Nuevo (70ft Span)	Interior Beam	HS-15	1247	75	1730	105	1263	329	1.37	-	1961	930
			Exterior beam	HS-15	1130	63.2	1591	97	1350	331	1.18	-		
4	19	Rio Sarapiqui (Pier suport)	Interior Beam	HS-15	2674	143	3243	165	2910	394	1.11	0.418782	1969	1,330
32	26	Rio Chirripo (First span)	Suporting point	HS-20	10165	241	11177	263	10343	614	1.08	0.43	1978	4,185
		Rio Chirripo (Center span)	Suporting point	HS-20	10165	241	11177	263	10343	614	1.08	0.43		

Composite Steel I Beam Girder

Route	Bridge No.	Bridge Name	Member	Original Condition			(HS20+25%) +Pavement(5cm)		Capacity		f(HS20+25%)/fC	V(HS20+25%)/VC	year	Traffic Volume
				Live Load	f (ksi)	V (Ksi)	f (ksi)	V (Ksi)	fc (ksi)	Vc (Ksi)				
2	12	Rio Puerto Nuevo (80ft Span) (Composite beam)	Int bottom	HS-15	28.2	-	38.4	-	24.8	-	1.55	-	1961	930
			Int top	HS-15	25.5	-	33.8	-	24.8	-	1.36	-		
			Int slab Concrete	HS-15	0.7	-	1.1	-	1.2	-	0.92	-		
			Ext bottom	HS-15	20.4	-	31.4	-	24.8	-	1.27	-		
			Ext top	HS-15	20.7	-	32	-	24.8	-	1.29	-		
			Ext slab Concrete	HS-15	0.6	-	1.1	-	1.2	-	0.92	-		
4	19	Rio Sarapiqui (Center span) (Composite Beam)	Int bottom	HS-15	20.5	50	24.7	69	19.8	349	1.25	0.20	1969	1,330
			Int top	HS-15	15.3		16.5		19.8		0.83			
			Int slab Concrete	HS-15	0.1	-	0.2	-	1.2	-	0.17	-		
32	26	Rio Chirripo (First span)	Mid Section bottom	HS-20	24	213	26	231	23	435	1.13	0.53	1978	4,185
			Mid Section top	HS-20	17.9		18.4		23		0.80			
			Mid Section Concrete	HS-20	0.2	-	0.2	-	1.2	-	0.17	-		
		Rio Chirripo (Center span)	Mid Section bottom	HS-20	22	214	25	232	23	435	1.09	0.53		
			Mid Section top	HS-20	15.2		16		23		0.70			
			Mid Section Concrete	HS-20	0.2	-	0.2	-	1.2	-	0.17	-		

Note
1) Shade means that the working force is larger than the resisting force.
2) ▭ is more than 30% large

d) Main Girder of RC I Beam Bridge and PC I Beam Bridge

The analysis result of RC I Beam Bridge and PC I Beam Bridge are shown in Table 10.1.23. The table indicates the condition of load carrying capacity of both type of bridges against new live load (HS20+25%) and summarizes the stress condition as below.

- In the case of RC I Beam

  In the case of the original design live load with HS15, the working force computed by new live load (HS20+25%) is larger than the load carrying capacity of RC I Beam Bridge. Especially in Rio Azufrado (Bridge No.7) and Rio Nuevo (Bridge No.16) with the non-uniform section girder, the working force is 1.5 times or 2 times larger than the load carrying capacity. On the other hand, in Rio Puerto Nuevo with the uniform girder, its working force is only 20% larger than the load carrying capacity, whereas its shear force is larger than capacity of shear force.

- In the case of PC I Beam (Rio Torres)

  The working tensile stress in 30m length span with new live load (HS20+25%) is 20% larger than allowable tensile stress, whereas the tensile stress in 17m length of span is only 3 ksi.

Table 10.1.23. Analysis Result of Superstructure (Concrete Girder)

RC I Beam

Route	Bridge No.	Bridge Name	Member	Original Condition			(HS20+25%) +Pavement(5cm)		Capacity		M(HS20+25%)/MC	V(HS20+25%)/VC	year	Traffic Volume
				Live Load	M (ft-kip)	V (Ksi)	M (ft-kip)	V (Ksi)	Mc (ft-kip)	Vc (Ksi)				
1	7	Rio Asufrado	Mid Span	HS-15	388	9	574	15	419	54	1.37	0.28	1955	1,660
			Column Junction	HS-15	1057	80	1541	112	967	123	1.59	0.91		
2	12	Rio Puerto Nuevo	Interior Beam	HS-15	868	74	1147	100	962	89	1.19	1.12	1961	930
			Exterior beam	HS-15	841	72	1127	99	896	89	1.26	1.11		
	16	Rio Nuevo	Mid Span	HS-15	587	15	920	21	438	53	2.10	0.40	1961	2,060
			Column Junction	HS-15	1147	89	1554	122	1418	155	1.10	0.79		

PC I Beam

Route	Bridge No.	Bridge Name	Member	Original Condition			(HS20+25%) +Pavement(5cm)		Capacity		f(HS20+25%)/fC	V(HS20+25%)/VC	year	Traffic Volume
				Live Load	f (ksi)	V (Ksi)	f (ksi)	V (Ksi)	fc (ksi)	Vc (Ksi)				
218	29	Rio Torres 30m Span PC Beam	Top	HS-20	-110	-	-117	-	-168	-	0.70	-	1980	29,505
			Bottom	HS-20	22	-	41.4	-	32.7	-	1.27	-		
		Rio Torres 30m Span PC Beam	Top	HS-20	-71.6	-	-76.5	-	-168	-	0.46	-		
			Bottom	HS-20	-16.7	-	3	-	32.7	-	0.09	-		

Note

1) "+"; Tension, "-"; Compression

2) Shade means that the working force is larger than the resisting force.

3) $\boxed{}$ is more than 30% large

e) PC Box Girder Bridge

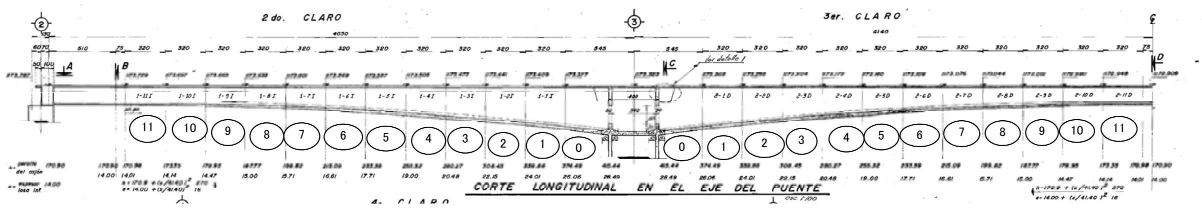
Since there are no as-built drawings of Rio Chirripo Bridge in MOPT, the necessary data to make analysis model with, such as layout of PC cable and tensioning force is needed to be presumed from the drawings of Virilla Bridge which is same type bridge with same dimension.

The working stresses of girder are listed in Table 10.1.24. The analysis has been done for three (3) different conditions of Creep strain of 0, 10 and 20% under 20% of pre-stress losses have been considered. Zero creep is an unlikely condition for the bridge in 20 years old. Therefore, the stresses do not comply with allowable stresses. The bridge with creep conditions of 10 % and 20 % is likely and its working stress is almost normal level.

Table 10.1.24. Working Stress of PC Box Girder in Rio Chirripo Bridge (No.17)

(Unit;kg/cm2)

Creap	Block No.		12	11	10	9	8	7	6	5	4	3	2	1	0	Pier	0	1	2	3	4	5	6	7	8	9	10	11
0%	top	σ max	18	18	27	43	43	53	50	55	41	41	52	60	69	87	61	51	30	26	9	2	7	−1	5	−3	−10	−30
		σ min	18	−2	−2	9	7	16	15	21	10	14	31	42	52	68	47	40	22	20	4	−5	−5	−19	−19	−33	$\boxed{-48}$	$\boxed{-69}$
	bottom	σ max	69	92	102	101	117	115	127	114	124	136	125	120	126	122	126	118	132	123	122	135	137	$\boxed{156}$	$\boxed{155}$	$\boxed{158}$	$\boxed{173}$	$\boxed{194}$
		σ min	69	60	59	50	63	61	75	66	82	100	98	98	105	99	110	105	121	115	116	125	120	130	120	113	117	133
10%	top	σ max	19	19	31	49	52	64	64	70	58	60	69	79	89	110	81	70	49	48	31	24	29	22	28	22	15	−3
		σ min	19	−1	2	15	15	27	28	36	27	32	48	60	72	91	68	60	41	42	26	17	17	4	5	−8	−22	$\boxed{-43}$
	bottom	σ max	68	91	97	92	104	98	108	93	101	112	104	98	102	95	102	94	107	94	92	104	105	123	120	121	135	$\boxed{153}$
		σ min	68	59	54	41	50	44	56	45	59	76	76	76	81	72	86	82	97	87	86	94	87	96	85	77	79	93
20%	top	σ max	19	20	35	56	60	74	74	81	69	70	78	88	99	120	92	82	61	64	49	45	53	50	59	57	53	37
		σ min	19	0	6	22	24	37	39	47	38	43	57	69	81	102	79	71	53	58	44	38	41	32	36	27	16	−3
	bottom	σ max	66	89	91	82	91	84	93	77	86	98	91	87	91	82	89	81	91	73	67	74	70	82	74	70	78	92
		σ min	66	58	48	31	38	30	41	30	44	62	64	65	70	60	73	68	80	65	61	64	52	56	39	25	23	31



; Block No.

Allowable compressinon stress 140 kg/cm2
Allowable tension stress −30 kg/cm2

Note

1) "-"; Tension, "+"; Compression
2) ▭ is exceeding allowable stress

f) Substructure

The analysis results of substructure are shown in Table 10.1.25 and Table 10.1.26. The tables indicate the condition of load carrying capacity of member of substructure against an earthquake load (EQ). There is no information regarding condition of working stress against new live load HS20+25% in this table, because the mall and column of substructure have sufficient resisting force to live load..

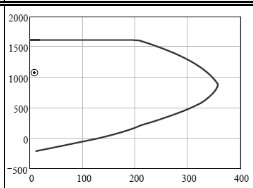
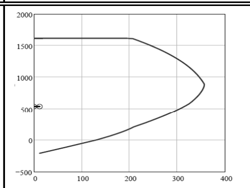
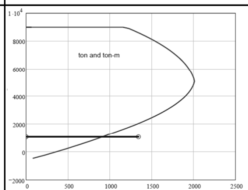
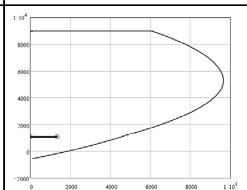
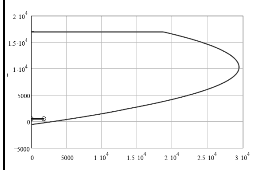
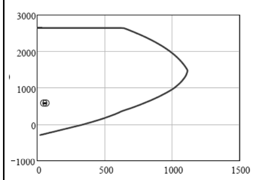
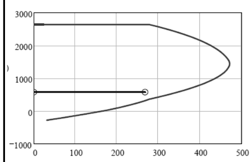
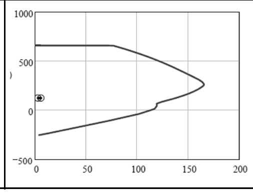
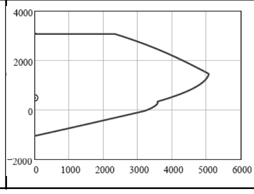
In the case of EQ, the working stress of reinforcement of pier exceeds allowable stress in Rio Aranjuez Bridge (No.2), Rio Nuevo Bridge (No.16) and Rio Torres Bridge (No.29), whereas it is considered that all abutments are able to resist.

Piers in Rio Aranjuez Bridge and Rio Nuevo Bridge are the wall type and their thickness are very thin.

Pier in Rio Torres Bridge is a slender column. Despite the fact that Rio Puerto Nuevo Bridge also has slender columns for the piers, their working stresses of pier are less than allowable stress. Both bridges differ from the type of their superstructure. The difference of superstructure induces difference in structural behavior because a steel girder is rather lighter than a concrete girder.

It is summarized that the working stress of pier tends to exceed the allowable stress when fixed supports are utilized and/or when superstructure is made of the concrete girder.

Table 10.1.25. Analysis Result of Substructure (1)

Route No.	Bridge No.	Bridge Name	Typr of superstructure	Design Live Load	Member	Type of substrucuture	Fix or Move	Longitudinal	Transversal	Evaluation
1	2	Rio Aranjues	3 Span Continuous Truss (Deck Bridge)	HS-15	Abutment 1	Rigid Frame	Exp			
					Pier 2	Wall	Fix			Not Ok
	3	Rio Abangares	2 Span Simple Truss (Through Bridge)	HS-15	Pier 1	Wall	Exp	-		
					Abutment 2	Rigid Frame	Fix			
	7	Rio Azufrado	RC Riged Frame	HS-15	Pier	Wall	Fix			

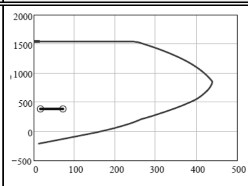
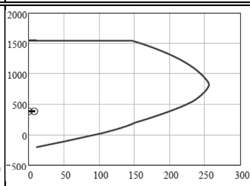
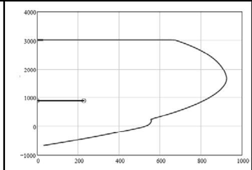
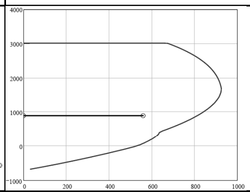
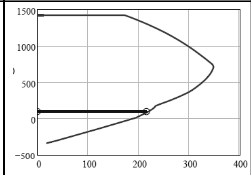
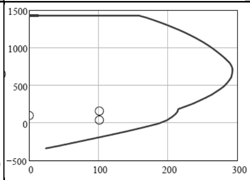
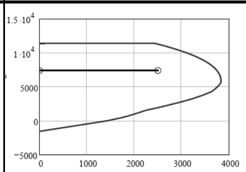
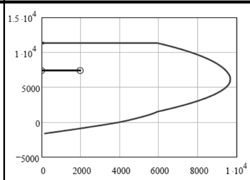
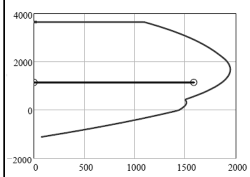
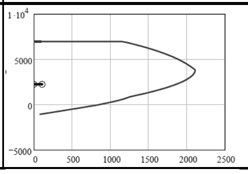
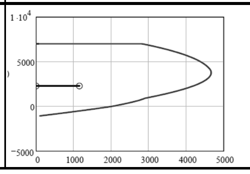
Note

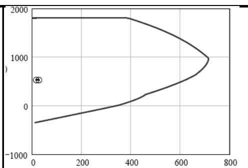
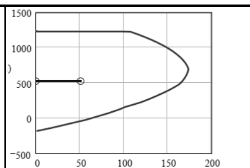
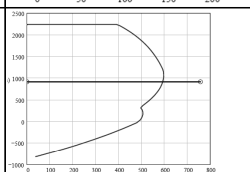
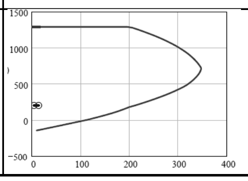
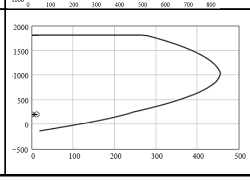
1) Horizontal axis is moment, Vertical axis is axial force

2) Curved line is resisting force combination of moment and axial force

3) Heavy line is actual force of moment and axial force

Table 10.1.26. Analysis Result of Substructure (2)

Route No.	Bridge No.	Bridge Name	Typr of superstructure	Design Live Load	Member	Type of substrucuture	Fix or Move	Longitudinal	Transversal	Evaluation
2	12	Rio Puerto Nuevo	Steel I Beam (Simple Girder)	HS-15	Abutment 1	Rigid Frame	Fix			
					Pier 2	T Type Pier (Column)	Fix			
	16	Rio Nuevo	3 Span Continuous RC Beam	HS-15	Pier 1	Rigid Frame	Fix			Not OK
4	17	Rio Chirripo	3 Span Continuous PC Box Beam	HS-20	Pier 1	Wall	Fix			
	19	Rio Sarapiqui	3 Span Gerber Beam	HS-15	Pier 1	T Type Pier (Ellipse)	Exp	–		
32	26	Rio Chirripo	6 Span Continuous Steel I Beam	HS-20	Pier 3	Wall	Fix			

218	19	Rio Toress	3 Span Simple PC I Beam	HS-20	Abutment 1	Rigid Frame	Fix			Not Ok
					Pier	T Type Pier (Column)	Move	–		
					Abutment 2	Rigid Frame	Fix			

g) Spread Foundation

The analysis result of Spread foundation is shown in Table 10.1.27. The table indicates the stability of spread foundation against new live load HS20+25% and an earthquake load (EQ). The stability is evaluated by the eccentricity of load.

There is no problem regarding the stability of foundation in 9 bridges by new live load (HS20+25%). However, the stability of foundation during earthquake must be considered. It is especially noted that the stability in Rio Aranjuez Bridge cannot be kept on both longitudinal and transversal direction, whereas other bridges are unstable on only transversal direction.

Pier tends to be less stable comparing to abutment and transversal direction tends to be less stable than longitudinal direction.

Table 10.1.27. Analysis Result of Stability of Foundation (Spread Foundation)

Route	Bridge No.	Bridge Name	Member	Live Load	Longitudinal							Transversal							Evaluation	year
					Width (m)	No EQ			EQ			Width (m)	No EQ			EQ				
						qmax (t/m²)	qmin (t/m²)	e	qmax (t/m²)	qmin (t/m²)	e		qmax (t/m²)	qmin (t/m²)	e	qmax (t/m²)	qmin (t/m²)	e		
1	2	Rio Aranjuez	Abutment 1	HS-15	7.468	-	-	-	-	-	-	3.658	30.48	30.48	0	35.78	25.17	0.107		1955
			Pier 2		4.267	20.08	20.08	0	-9.13	0		7.315	20.08	20.08	0	-42.22	0		Not OK.	
	3	Rio Abangares	Abutment 2	HS-15	4.572	42.15	34.1	0.079	53.87	22.37	0.314	2.591	38.12	38.12	0	596.91	0	1.186	Not OK.	1953
			Pier 1		2.743	-	-	-	-	-	-	12.497	18.56	18.56	0	255.52	0	5.642	Not OK.	
	7	Rio Azufrado	Pier 1	HS-15	8.23	-	-	-	26.73	26.16	0.015	-	-	-	-	-	-	-		1955
2	12	Rio Puerto Nuevo	Abutment 1	HS-15	4.267	20.69	25.21	0.07	6.13	39.77	0.521	3.048	22.95	22.95	0	25.35	20.55	0.052		1961
			Pier 2		5.486	22.93	22.93	0	40.8	5.06	0.713	5.486	22.93	22.93	0	74.81	0	1.622	Not OK.	
4	17	Rio Chirripo	Pier 1	HS-20	9.5	57.118	57.118	0	88.822	25.414	0.878	10.5	57.118	57.118	0	85.803	28.433	0.878		1978
218	29	Rio Tores	Abutment 1	HS-20	6	13.97	24.94	0.283	11.64	27.26	0.402	3.5	19.45	19.45	0	27.79	11.11	0.25		1980
			Abutment 2		3.5	20.24	24.45	0.055	14.01	30.67	0.216	2	22.34	22.34	0	32.36	12.32	0.149		
			Pier 1		-	-	-	-	-	-	-	7.5	12.59	12.59	0	39.01	0	2.137	Not OK.	
			Pier 2		-	-	-	-	-	-	-	7	11.16	11.16	0	20.8	1.52	1.009		

h) Pile Foundation

The analysis result of the pile foundation is shown in Table 10.1.28. The table indicates the stability of pile foundation against new live load HS20+25% and an earthquake load (EQ).

Three (3) bridges among 10 bridges have pile foundations, such as Rio Nuevo Bridge (No.16), Rio Sarapiqui Bridge (No.19) and Rio Chirripo Bridge (No.26), and in Two (2) bridges among those three (3) Bridges such as Rio Nuevo Bridge and Rio Sarapiqui Bridge, the axial force of pile reaction is larger than allowable bearing capacity of pile. Especially in Rio Sarapiqui Bridge, the axial force of pile reaction is larger than allowable bearing force of pile even under normal conditions.

Table 10.1.28. Analysis Result of Stability of Foundation (Pile Foundation)

<table>
<tr><th rowspan="3">Route</th><th rowspan="3">Bridge No.</th><th rowspan="3">Bridge Name</th><th rowspan="3">Member</th><th rowspan="3">Live Load</th><th rowspan="3">Pile Type</th><th rowspan="3">Pile length</th><th colspan="2">Allowable Axial Force</th><th colspan="4">Longitudinal</th><th colspan="4">Transversal</th><th rowspan="3">Evaluation</th><th rowspan="3">year</th></tr>
<tr><th rowspan="2">Normal</th><th rowspan="2">EQ</th><th colspan="2">No EQ</th><th colspan="2">EQ</th><th colspan="2">EQ</th><th colspan="2">No EQ</th></tr>
<tr><th>Pmax (KIP)</th><th>Pmin (KIP)</th><th>Pmax (KIP)</th><th>Pmin (KIP)</th><th>Pmax (KIP)</th><th>Pmin (KIP)</th><th>Pmax (KIP)</th><th>Pmin (KIP)</th></tr>
<tr><td rowspan="2">2</td><td rowspan="2">16</td><td rowspan="2">Rio Nuevo</td><td>Abutment 1</td><td rowspan="2">HS-15</td><td rowspan="2">14"x14"</td><td>16.764</td><td>18</td><td>27</td><td>-</td><td>-</td><td>12.169</td><td>10.12</td><td>-</td><td>-</td><td>-</td><td>-</td><td rowspan="2">Not OK</td><td rowspan="2">1961</td></tr>
<tr><td>Pier 1</td><td>9.144</td><td>19</td><td>28.5</td><td>13.79</td><td>13.79</td><td>62.385</td><td>-34.806</td><td>32.244</td><td>-4.665</td><td>13.79</td><td>13.79</td></tr>
<tr><td rowspan="3">4</td><td rowspan="3">19</td><td rowspan="3">Rio Sarapiqui</td><td>Abutment 1</td><td rowspan="3">HS-15</td><td rowspan="3">12"B53</td><td>24</td><td>22</td><td>33</td><td>-</td><td>-</td><td>-</td><td>-</td><td>41.44</td><td>-1.02</td><td>108.489</td><td>108.489</td><td rowspan="3">Not OK</td><td rowspan="3">1969</td></tr>
<tr><td>Pier 1</td><td>13.3</td><td>12</td><td>18</td><td>-</td><td>-</td><td>-</td><td>-</td><td>418.183</td><td>-201.205</td><td>108.489</td><td>108.489</td></tr>
<tr><td>Pier 2</td><td>14.5</td><td>13</td><td>19.5</td><td>-</td><td>-</td><td>-</td><td>-</td><td>107.239</td><td>-32.132</td><td>37.554</td><td>37.554</td></tr>
<tr><td>32</td><td>26</td><td>Rio Chirripo</td><td>Pier 3</td><td>HS-20</td><td>14"BP89</td><td>30</td><td>100</td><td>150</td><td>57.118</td><td>57.118</td><td>117.16</td><td>105.32</td><td>142.27</td><td>80.21</td><td>111.24</td><td>111.24</td><td></td><td>1978</td></tr>
</table>

10.1.6 Identification of Deterioration and Damage Mechanism of 10 Bridges

In accordance with the inspection result and the analysis result, the condition of deterioration and its cause in 10 bridges are summarized as below.

1) Deck Slab

The life of the bridge slab in route No.1 and route No.2 already passed their ages for more than forty-five (45) years. And based on the analysis result, it is apparent that required load carrying capacity of deck slab is insufficient for new live load (HS20+25%). It is noted that he slab in route No.2 is not so much damaged as in route No.1, despite the fact that these bridges in both routes have been constructed in almost same age.

On the other hand, although Rio Torres Bridge in route 218 has been constructed much newer than bridges in route No.2 and has had enough load carrying capacity for new live load (HS20+25%), its slab is severely damaged as much as bridges in route No.1.

Accordingly, it is likely that traffic volume has much more influence upon the damage of slab than age of bridge. Almost all slab thickness of 10 bridges is minimum value specified in AASHTO, it is much desirable to increase the slab thickness or to be reinforced by steel plate or FRP when it is assumed that the traffic volume still increases in bridge site in future,

2) Superstructure

a) Accessory

The items shown below shall be required to repair. It is caused by lack of maintenance

- Install new expansion joint
- Repair railings
- Pave with 5cm thick of asphalt layer and waterproof on the slab

b) Truss Bridge

According to the analysis result of floor system (shown in Table 10.1.21) in Abangares Bridge (No.3) and Aranjuez Bridge (No.2), the member of floor system dose not have enough strength and the load carrying capacity for new live load. This is same conclusion as the result of detailed inspection in site.

The stress of compression member in main flame is bigger than allowable stress, so additional member shall be required to decrease the buckling length. Moreover, some tension members of the main frame should be reinforced to ensure the load carrying capacity in Rio Aranjuez Bridge.

According to above mentioned, it is required that the truss type bridges shall be reviewed and measured against new live load (HS20+25%) as well as dead load increased by reinforcement of slab and/or new pavement layer.

c) Steel I Beam Girder

Both non-composite and composite steel I beam designed with HS15 shall be reinforced against new live load (HS20+25%) as well as dead load increased by reinforcement of slab and/or new pavement. Because its working force and working stress exceed the capacity limit by 30% to 50% (shown in Table 10.1.22).

However, in the case of bridges designed with HS20, the working stress of both top and bottom flanges exceed the allowable stress by around 10%. In consequence, reinforcement may not be required unless the dead load of girders will increase.

d) RC I Beam Girder

According to the analysis result (shown in Table 10.1.23), both the bending moment and the shear force exceed the resisting force by 10% to 30% in Rio Puerto Nuevo Bridge (No12), in which some cracks caused by shear force in the girder are estimated. This phenomenon corresponds with results of the site inspection.

In Rio Azufrado Bridge and in Rio Nuevo Bridge, bending moment is larger than the resisting force, by its 50% to 100%.

Consequently, RC I Beam girder is required to be reinforced.

e) PC I Beam

Among 10 bridges only, Rio Torres Bridge is categorized as a PC I Beam Bridge. This bridge consists of 2 simple beams with different span lengths, which are 30m and 17 m.

According to analysis result (shown in Table 10.1.23), in Rio Torres Bridge (No.29), the tensile stress in bottom flange of PC I beam with the 30m span length is about 30% bigger than allowable stress, so this girder is required to be reinforced. In case of making reinforcement plan, it is important to consider that the selected countermeasure will not increase the dead load for effective reinforcement. On the other hand, the girder with 17m span length has enough load carrying capacity for new live load (HS20+25%), so reinforcement of this girder may not be required.

f) PC Box Girder

Deflection can be observed at the central span of two pre-stressed concrete box girder bridges: Bridge No.17 (Rio Chirripo, about 20cm) and Bridge No.20 (Rio Sucio, about 27cm). Potential causes of deflection are as follows:

- Lack of concrete box girder strength
- Decrease in the elastic modulus of concrete box girders
- Lack of introduced tension of PC cables
- Abnormal progress in concrete creep
- Insufficient quality control during construction

The results of investigations into the above-mentioned potential causes of deflection for Bridge No. 17 and No. 20 are described below.

(a) According to a detailed inspection conducted by the Study the following can be stated:
- There is no abnormal cracking.
- There is no abnormal carbonation.
- There is no abnormal vibration due to vehicles passing.

(b) According to a concrete core boring test for Bridge No.20 and a Schmidt Hammer test for both Bridge No. 17 and No. 20 the following can be stated:
- The concrete strength of the bridge bodies satisfies required levels.
- The elastic modulus of boring core specimens satisfies required levels.

(c) According to a structural analysis and loading test for Bridge No.17 the following can be stated:
- Required stiffness was confirmed and is the same as that of the drawings

The above analyses indicate that Bridge No.17 and No.20 satisfy the necessary structural requirements, except for the deflections observed at the central spans, and will therefore be able to serve traffic safely. It is thought that the observed deflections are due to insufficient quality control during construction. However, this is an assumption as there are no original construction plans and construction records available. Note that the load carrying capacity of the bridges for HS20+25% was checked in the structural analysis using a model based on original design drawings.

3) Substructure

During the site inspection, it is observed that Column Pier is hardly resistible against earthquake load because of its being very thin. Nevertheless, the analysis result indicates that the column pier in steel girder bridge, such as Rio Puerto Nuevo Bridge (No.12), has enough load carrying capacity for earthquake load since the weight of steel girder is light. It is noted, however, that the column pier is not resistible the earthquake load when superstructure is the concrete girder bridge, such as Rio Tress Bridge (No.29).

In the case of wall type pier and fix support, its thickness in longitudinal direction is too thin to have enough load carrying capacity even when superstructure is a light weight of the steel girder.

4) Foundation

According to the analysis result of spread foundation, Rio Azufrado Bridge (No.7) and Rio Chirripo Bridge (No.17) among 6 bridges are steady under seismic influence because Rio Azufrado Bridge is the rigid type bridge and the footing in Rio Chirripo Bridge is the biggest size in 10 bridges, which is almost same size as width of roadway. The footing size of other bridges is less than width of roadway.

The abutment is more stable than pier since its height is lower and, moreover, its size of transversal direction is wider than pier.

In the case of Pile foundation, the pile reaction force by earthquake load in Rio Nuevo Bridge (No.16) and Rio Sarapiqui Bridge (No.19) are bigger than the allowable bearing capacity of the pile. Especially in Rio Sarapiqui Bridge, its pile reaction force in normal condition is bigger than the allowable bearing capacity of the pile. However, Rio Chirripo Bridge (No.26) is stable under seismic influence. This result supports the detailed inspection result, which indicates that the pier in Rio Chirripo Bridge has not been damaged by earthquake in 1991.

Regarding scouring, which has been observed in some bridges, the biggest scouring in 10 bridges is in Rio Nuevo (Bridge No16) around pier. It is caused by lowering down of the riverbed and it occurs not only around bridge but also the whole of riverbed. Moreover, the slop protection in abutment has collapsed in almost all bridges, which shall be required to repair.