10.2 Bridge Loading Test

10.2.1 General

In the design of bridge, the safety of bridge structure is evaluated by various analyses with structural modeling. These analyses and modeling include many premises based on lots of experiences to simplify the design methods. Therefore, there are some differences in the behaviors between design analysis and actual bridge.

The loading test on existing bridge is carried out for the purpose of the evaluation of the load carrying capacity and the durability of fatigue for steel member by measuring the deformation and stress of bridge under the actual loading on existing bridge. These results of the tests are used one of the material for making a plan to repair or reinforce the bridge, after comparing the results between a structural analyses and loading tests.

For the old bridge, which does not have its design drawings, the loading test is also carried out for the purpose to set up conditions of structural analysis for repair or reinforcing design.

10.2.2 Purpose of Test

As described above, the bridge loading test is one of the method for the bridge rehabilitation and reinforcement to evaluate the bridge load carrying capacity and the durability of fatigue for steel member. However, there is few experience about a bridge loading test on existing bridge in Costa Rica. Only deflections of the existing bridge under the full truck loading were measured by using a level.

The loading tests on existing bridges in this Project were carried out for the purpose of technical transfer about the loading test to engineers in university and private companies as well as counterparts in Costa Rica. The series of loading tests on existing bridges have been carried out by the study team together with the counterparts in Costa Rican using the latest measurement equipment.

10.2.3 Outline of Loading Test

The loading tests have been carried out on two of 10 bridges, which are selected for the object bridges of rehabilitation, aimed for following points in this study.

1) To confirm the specific characteristic of the bridge structure
   -Comparison with the results of structural analysis and results of Loading Tests

2) To confirm the specific characteristic of the fatigue of steel member
   -Fatigue Life Prediction based on the stress frequency

3) To confirm the relationship between load and stress
   - Comparison with the results of structural analysis and results of Load Test

The contents of the loading tests are listed as shown in Table 10.1.1.

Table 10.2.1. Contents of Loading Test

Test Item	Measurement Items	Loading Conditions
Static Load Test	-Deflection of bridge girder -Stress of steel member of bridge	**- Static Truck Load**
Dynamic Load Test (Stress Frequency Measurement Test)	- Stress frequency of steel member which intend to have higher possibility of breakage due to fatigue	**- Running Truck Load** **- Actual Traffic Load**

10.2.4 Objective Bridges and Loading Test Items

The bridges selected for the test are No.17 Rio Chirripo Bridge on Route 4 and No.29 Rio Chirripo Bridge on Route 32. These two bridges were selected as a typical type from concrete bridge and steel bridge respectively. The stress measurement under bridge loading tests is normally carried out for only steel bridges, and is not carried out for concrete bridges due to the reasons below.

-The concrete is composite material of sand, gravel and cement, is not single material like steel. Therefore, the strain in the small area of concrete is affected by each characteristics of material and cannot be converted to the stress of concrete.

-The tensional stress of concrete is ignored in the structural design. Therefore, the concrete stress comparison between the result of test and the structural analysis is no meaning.

Therefore, as the No.17 Chirripo Bridge is a prestressed concrete bridge type, the deflection measurements of main girder were carried out for the static load test on No.17.

1) Objective Bridges of Loading Test

The location of the bridges for the load test is shown in Figure 10.2.1

Figure 10.2.1. Location Map of Bridges

2) Loading Test Items

The loading test items for each bridge were selected as shown in Table 10.2.2. Bridge general views are shown in Figure 10.2.2. and 10.2.3.

Table 10.2.2. Loading Test Item

Test Item	Objective Bridges	
	Bridge Name (Route. No.)	Bridge Type (Length & Span Arrangement)
Static Load Test	No.17 Chirripó Br. Route 4	PC 3 Spans Continuous Box Girder 46.50 + 82.80 + 46.50 = 175.80 m
Static Load Test	No.26 Chirripó Br. Route 32	Steel Simple I Girder + Steel 6 Spans Continuous Steel I Girder 15.86 + (59.39 +67.0 +73.2 +73.2 +67.0 +59.39) = 415.52 m
Running Load Test		
Actual Traffic Load Test (Stress Frequency Measurement Test)		

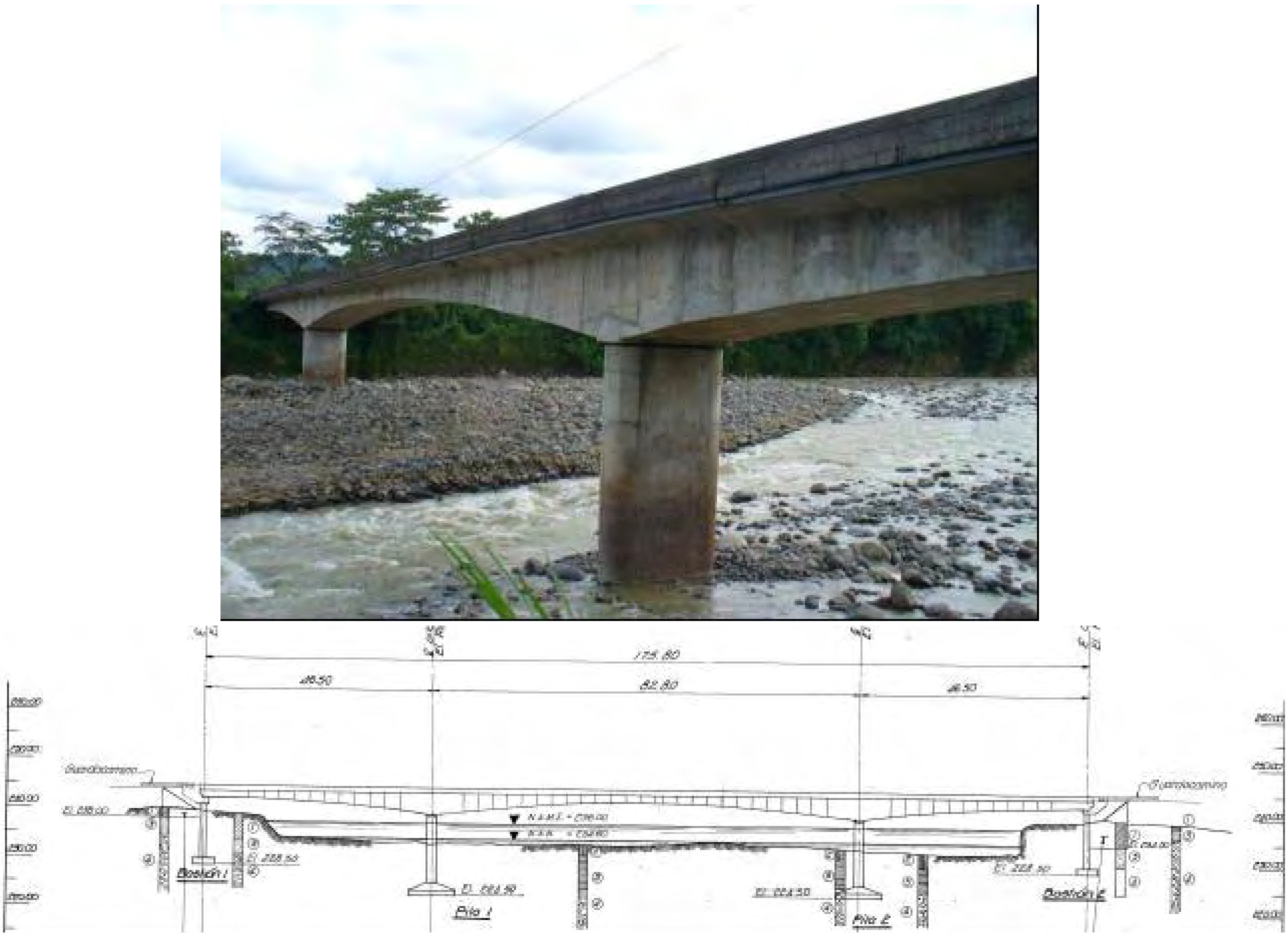


Figure 10.2.2. No.17 Chirripó Bridge

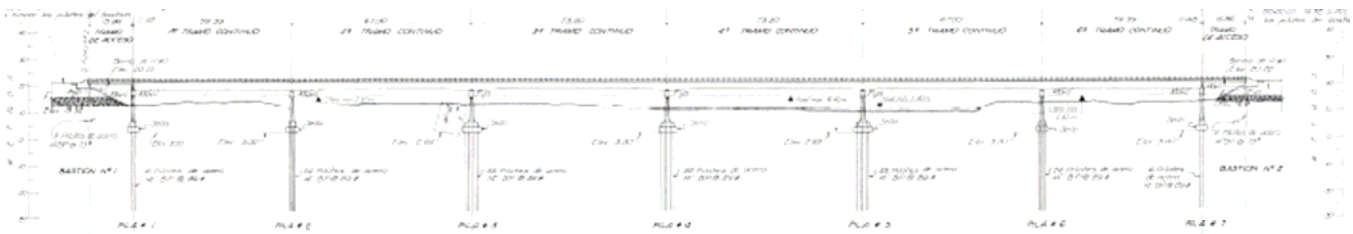


Figure 10.2.3. No.26Chirripó Bridge

10.2.5 Loading Tests Flow

The loading test flow is as shown in Figure 10.2.4.

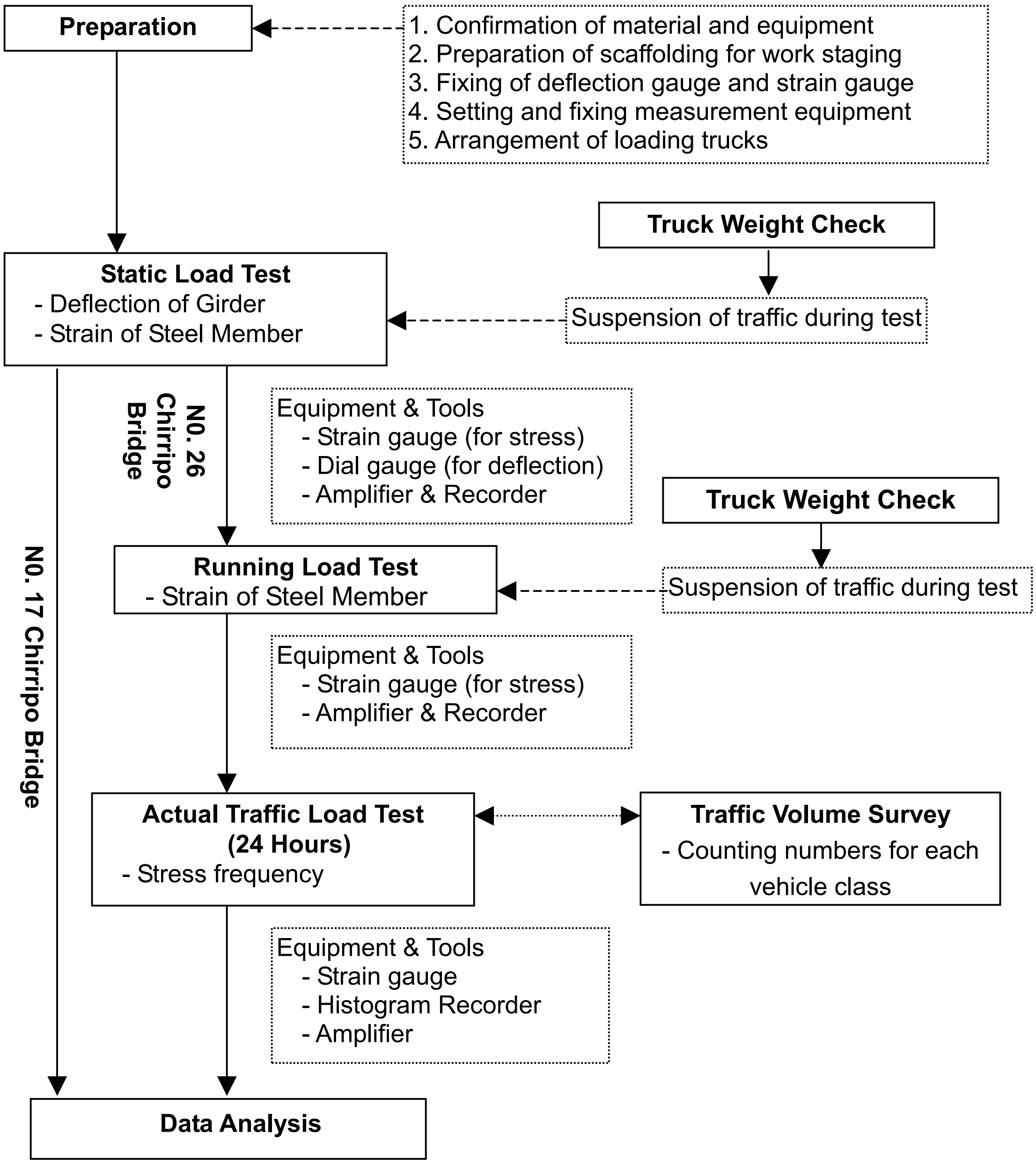


Figure 10.2.4. Loading Test Flow

10.2.6 Loading Tests Schedule

The loading tests were carried out as shown in Table 10.2.3. The preparation for the test has been started from middle of July. The local equipments and tools were used for the test as much as possible, except the strain gauges and the deflection measurement equipment, which were imported from Japan.

The Static Load Test on No.17 Chirripo Bridge was carried out in the early morning on 29th August and the Static Load Test and the Running Load Test on No.26 Chirripo Bridge were carried out in the early morning on 30th August to minimize the influence of traffic suspension during the tests. The actual traffic load test on No.26 Chirripo Bridge was started at 12:00 on 30th August and it had been continued for 24 hours and the traffic volume was also continued during same period of the test.

Table 10.2.3. Schedule of Bridge Loading Tests

Date		Work Item	
Middle of July		• Explanation about Bridge Loading Test to Counter Part • Start of Arrangement of Material Equipment • Seminar about Usage of Strain Gauge and Data Recorder	
End of July		• Selection of Contractor for Arrangement of Scaffolding and Loading Truck	
Beginning of August		• Information and Requests to Related Organizations and Government Office - Traffic Office: Traffic Suspension during the tests - ICE: Temporary Electric Power Supply - Organization and Road User: Information about Traffic Suspension	
Middle of August		• Seminar about Usage of Strain Gauge and Data Recorder • Fabrication of Deflection Measurement Tools	
August	Week	No.17 (Route 4)	No.26 (Route 32)
22	Tue		- Scaffolding Setting Start
23	Wed		- Scaffolding Setting and Completion
24	Thu	14:00: Deflection Tool Check (Pre-Setting)	09:00: Deflection Tool Check (Pre-Setting) 09:30: Electric Power Supply Setting 10:30: Truck Size & Weight Measurement
25	Fri		
26	Sat		06:00: Gauge Setting 10:00: Truck Position Marking
27	Sun	8:00 Truck Position Marking	06:00: Gauge Setting
28	Mon	11:00 Deflection Tool Setting (Anchor)	06:00: Gauge Setting Check
29	Tue	02:30 Site Meeting 03:00 Truck Weight Measurement 03:00 Deflection Tool Setting **05:00: Static Load Test till 7:00** 07:00: Removal of Test Equipment	13:00: Connection of Data Gathering System
30	Wed		01:30: Site Meeting 02:00: Check of Data Gathering System 02:00: Deflection Tool Setting 02:00: Truck Weight Measurement **04:00: Static Load Test & Running Load Test, till 8:00** **12:00: Traffic Load Test (Traffic survey)**
31	Thu		**12:00: Traffic Load Test Finish** 13:00: Removal of Test Equipment
1	Fri		08:00: Removal of Scaffolding

10.2.7 Loading Tests for No.17 Chirripo Bridge

The static load test has been carried out on the No.17 Chirripo Bridge to confirm the relationship between load and deflection of the main girder. The evaluation of load carrying capacity with the bridge loading test was proceeded in accordance with the sequence as shown in Figure 10.2.5.

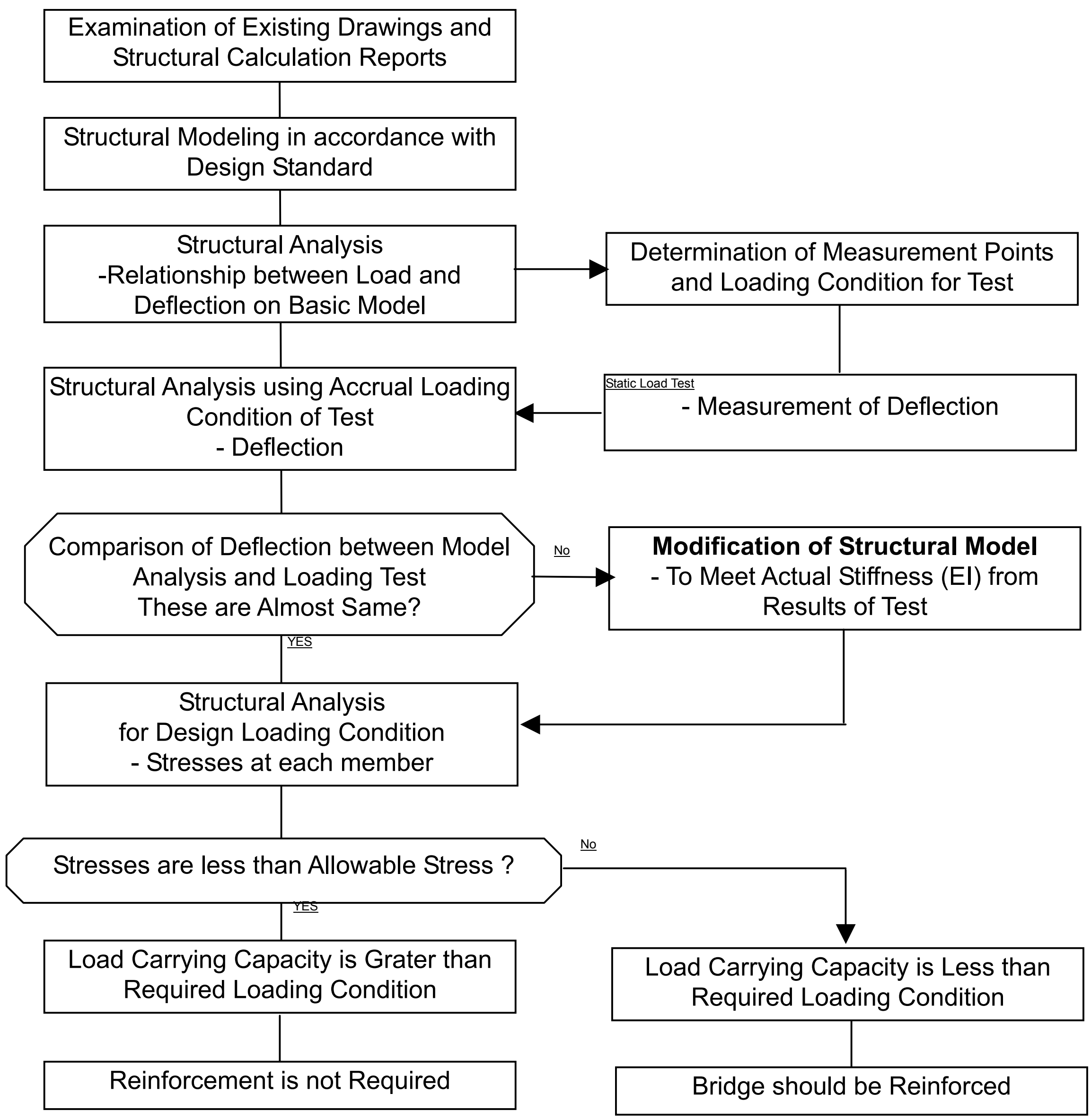


Figure 10.2.5. Evaluation Sequence for Load Carrying Capacity

1) Measurement Points of Deflection

The measurement points of deflection for No.17 Chirripo Bridge were at both sides of center of main span as shown in the Figure 10.2.6. The deflection shall be measured with deflection measurement equipment as shown in Figure 10.2.7.

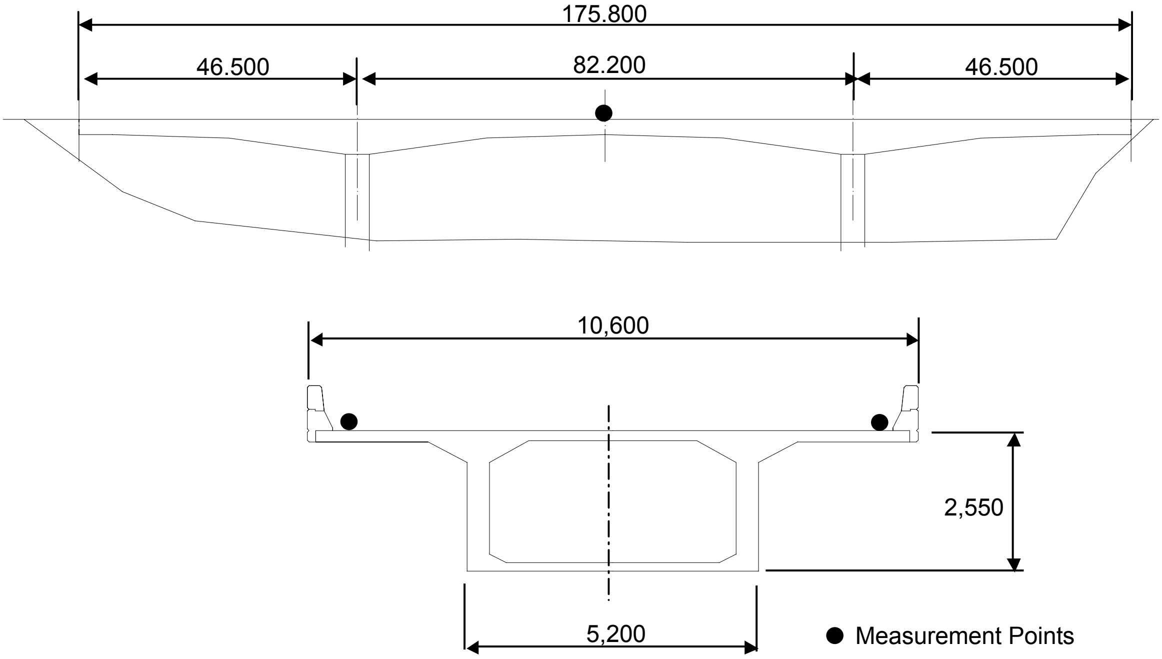


Figure 10.2.6. Deflection Measurement Points for No.17 Chirripó Bridge

Figure 10.2.7. Deflection Measurement Equipment

2) Measurement Method of Deflection

The riverbed under the measurement points of the Bridge located on a small island in the river and it was impossible to measure the deflection on the riverbed. The height of bridge surface is about 15 meter from the existing riverbed. Therefore, the deflection was measured on the bridge surface with using a measurement tool fabricated with steel pipes and angles shapes as shown Figure 10.2.8 and 10.2.9. The measurement tools were used as the fixed point and the deflections were by the deflection measurement equipments set on the both sides of bridge surface.

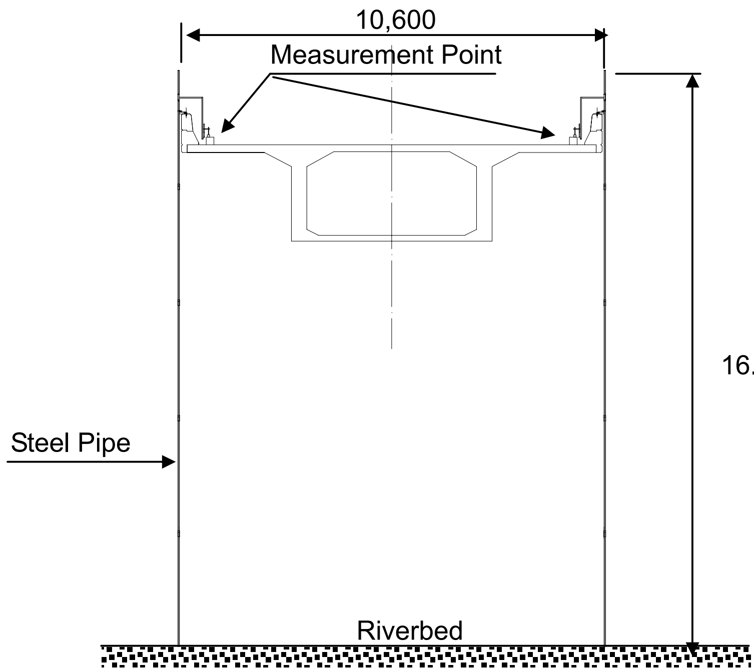


Figure 10.2.8. Deflection Measurement Method for No.17 Bridge

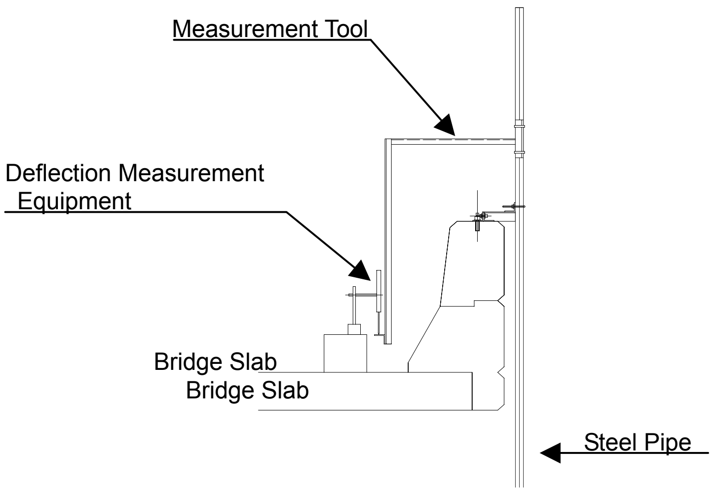


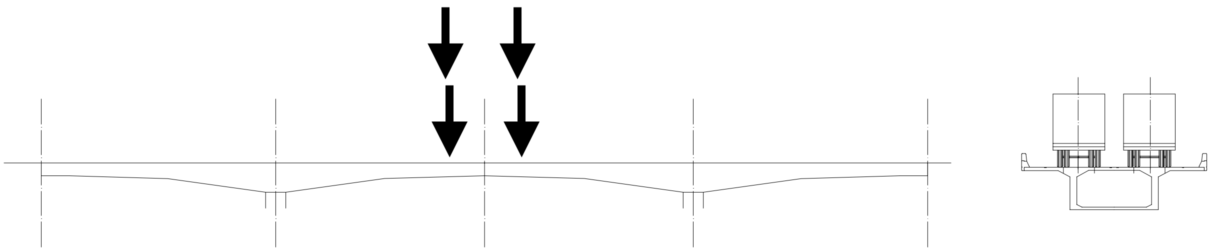
Figure 10.2.9. Deflection Measurement tool for No.17 Bridge

3) Loading Condition

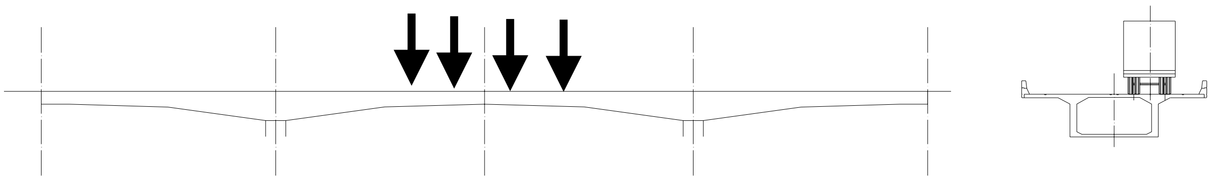
The three load cases were tested taking account with the maximum positive deflection, torsion and maximum negative deflection as shown in Table 10.2.4 and Figure 10.2.10.

Table 10.2.4. Static Load Case for No.17 Bridge

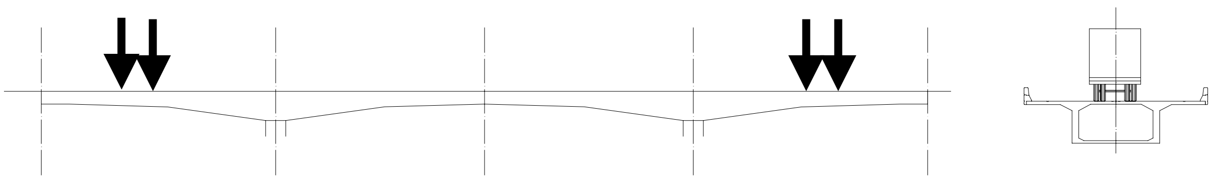
Case	View Point	No. of Trucks	Loading Points
1	Positive Deflection	4	Center of center span, Both lane
2	Torsion	4	Center of center span, One lane
3	Negative Deflection	4	Center of both side span, Both lane



Case 1. Positive Deflection



Case 2. Torsion



Case 3. Negative Deflection

Figure 10.2.10. Static Load Case for No.17 Bridge

4) Operation of Static Load Test

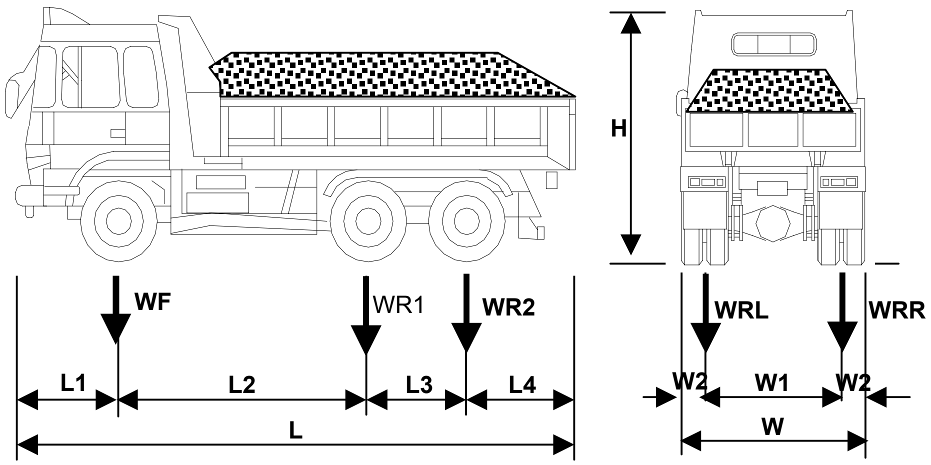
The static load test on No.17 Bridge was carried out on 29th August with the study team and the counter part of MOPT. The traffic suspension with 30 minutes was planed for each loading case, and the road was opened for traffic with 15 minute between two loading cases. After the final site meeting, the measurement of loading truck and the setting of the deflection measurement tool were carried out at same time. The trucks with weight of 25 ton including gravel for load were used for the test load. The dimensions and weights of trucks, which used for the test, are shown in Table 10.2.6.

Table 10.2.5. Detailed Time Schedule of Static Load Test

Time	Work Item
29/8, 02:30	Site Meeting: All Staff
03:00	- Truck weights and size measurement - Deflection tool setting - Lighting system setting
05:00	**-Static Load Test for 1st loading position**
05:30	Open for traffic
05:45	**-Static Load Test for 2nd loading position**
06:15	Open for traffic
06:30	**-Static Load Test for 3rd loading position**
07:00	Open for traffic
07:15	-Removal of measurement equipment & tolls

Table 10.2.6. Truck Dimensions and Weight Used for Loading

Figure



Truck			Truck 1	Truck 2	Truck 3	Truck 4
Dimensions (mm)	Length	L	7,980	7,540	8,380	8,020
		L1	1,200	1,250	1,310	1,240
		L2	4,450	3,960	4,590	4,540
		L3	1,280	1,280	1,290	1,320
		L4	1,050	1,050	1,190	920
	Width	W	2,430	2,410	2,410	2,420
		W1	1,820	1,830	1,830	1,840
		W2	305	290	290	290
Weight (tone)	Front Wheel	Left	3.430	3.055	3.380	3.985
		Right	3.340	2.950	3.580	3.645
		Total	6.770	6.005	6.960	7.630
	Rear Wheel 1	Left	4.885	5.625	4.605	5.570
		Right	4.705	4.590	4.855	4.575
		Total	9.590	10.215	9.460	10.145
	Rear Wheel 2	Left	4.815	5.330	4.635	5.220
		Right	4.330	4.220	4.905	4.290
		Total	9.145	9.550	9.540	9.510
	Total	Left	13.130	14.010	12.620	14.775
		Right	12.375	11.760	13.340	12.510
		Total	**25.505**	**25.770**	**25.960**	**27.285**

The situations of the loading test are as shown in photos in Figures 10.2.11. The test was carried out very smoothly and the traffic volume was not so large at that morning, the actual test time could be reduced.

Fabrication Work of Deflection Measurement Tool

Vehicle Axial Weight Measurement

Setting of Deflection Measurement Tool

Loading Test for Case 1

Deflection Measurement

Loading Test for Case 3

Figure 10.2.11. Situation of Loading Test on No.17 Bridge

10.2.8 Loading Tests for No.26 Chirripo Bridge

The static load test and two types of dynamic load test have been carried out on the No.26 Chirripo Bridge to confirm the relationship between load and deflection/stress of the main girder and the stress frequency of steel member. The evaluation of load carrying capacity with the bridge loading test was proceeded in accordance with the sequence as shown in Figure 10.2.5 as same as that for the No.17 Chirripo Bridge. The evaluation sequence of durability for fatigue is as shown in Figure 10.2.12.

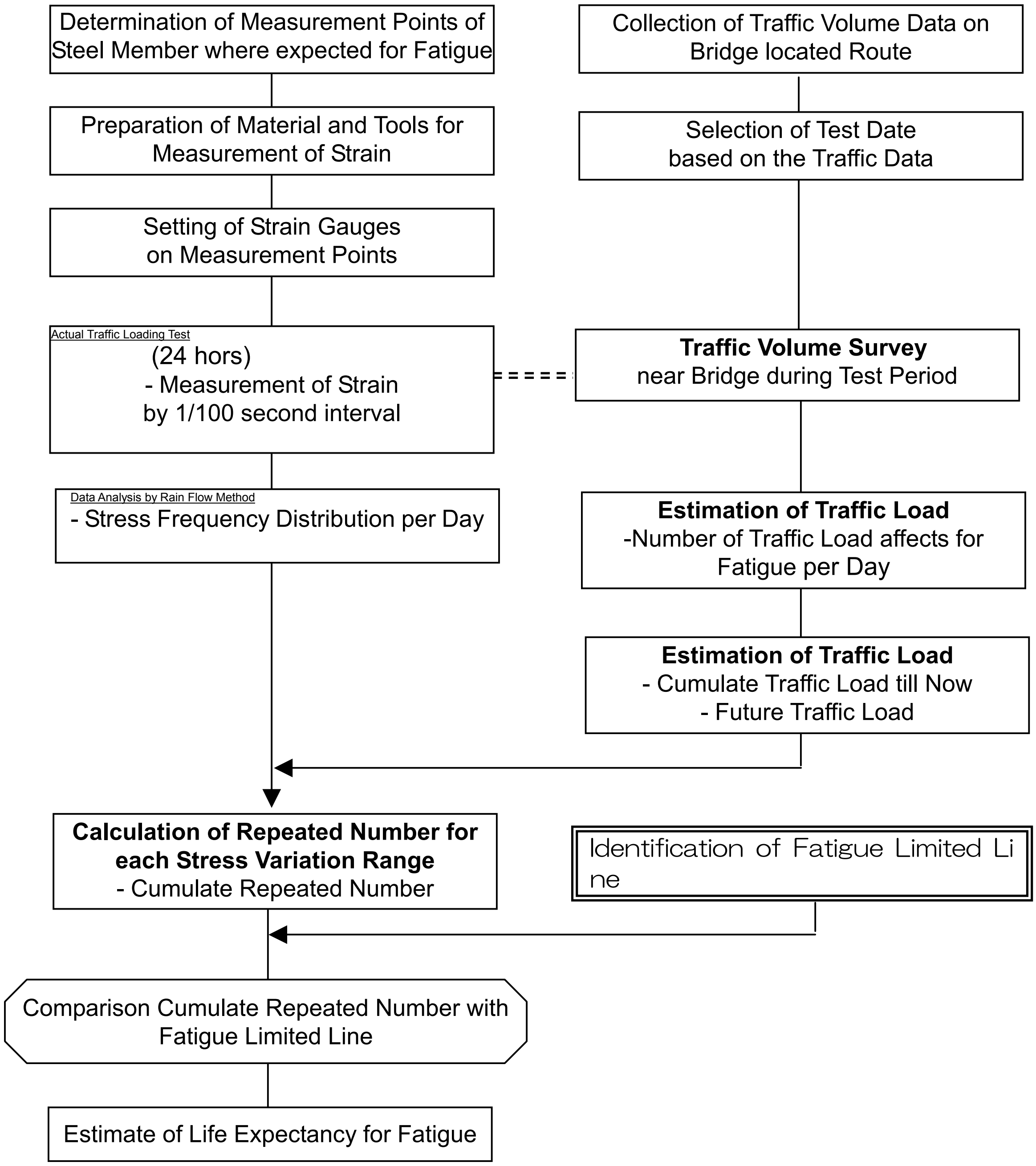


Figure 10.2.12. Evaluation Sequence for Durability for Fatigue

1) Measurement Points of Deflection

The measurement points of deflection for No.26 Chirripo Bridge were at both sides of center of span between pier P5 and pier P6 as shown in the Figure 10.2.13. The deflection was measured for only the static load test.

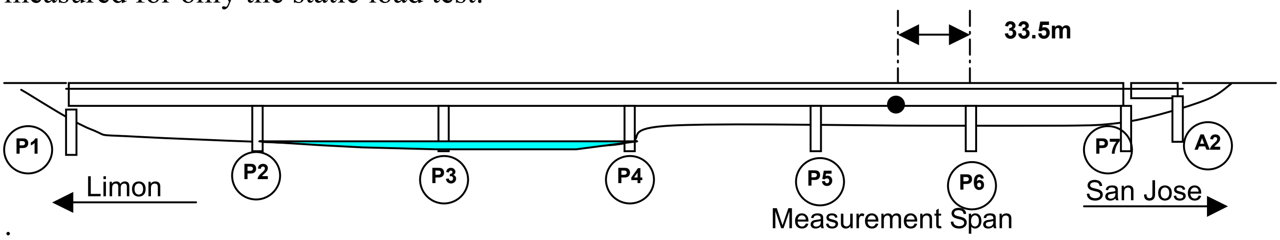


Figure 10.2.13. Deflection Measurement Points for No.26 Chirripo Bridge

2) Measurement Method of Deflection

The measurement points of this bridge locate on riverbank and the clearance under the girder is about 4 meter. Therefore, the deflection was measured on the ground surface under the bridge using the deflection measurement equipment same as bridge No.17. The steel angle shaped steel fixed at lower flange of main girder was used for the deflection measurement tool as shown in Figure 10.2.14.

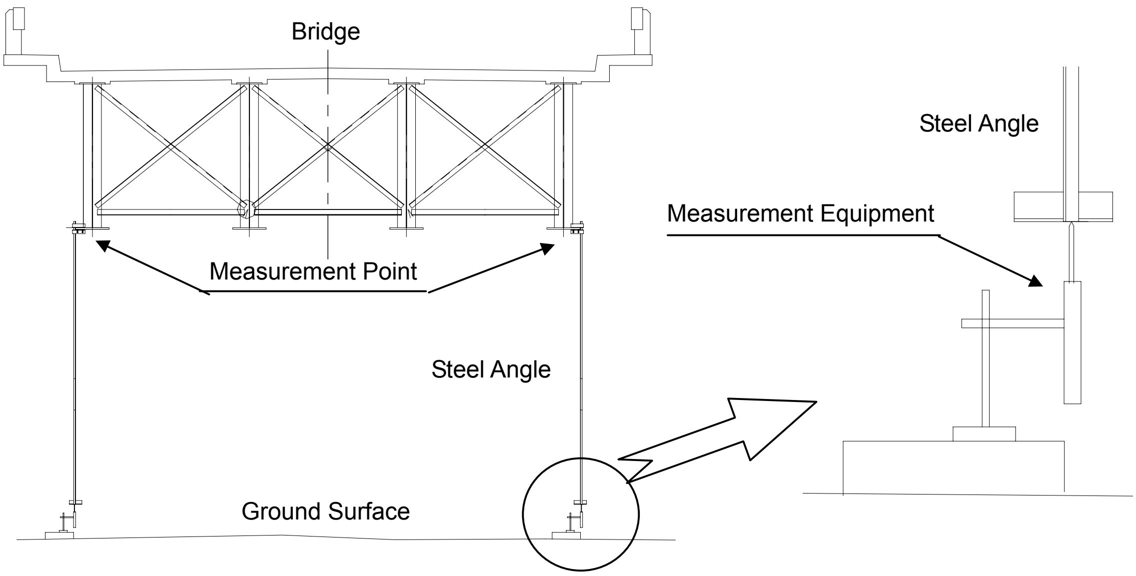


Figure 10.2.14. Deflection Measurement Tool for No.26 Chirripo Bridge

3) Measurement Point for Stress

The measurement points for stress were selected by taking account to evaluate both the load carrying capacity and the durability of fatigue. As this bridge is a continuous steel girder, the measurement points for load carrying capacity were set at both the center of span and at the pier. The measurement points for the durability of fatigue were selected at connection parts which are easy to be affected by fatigue as shown Figure 10.2.15.

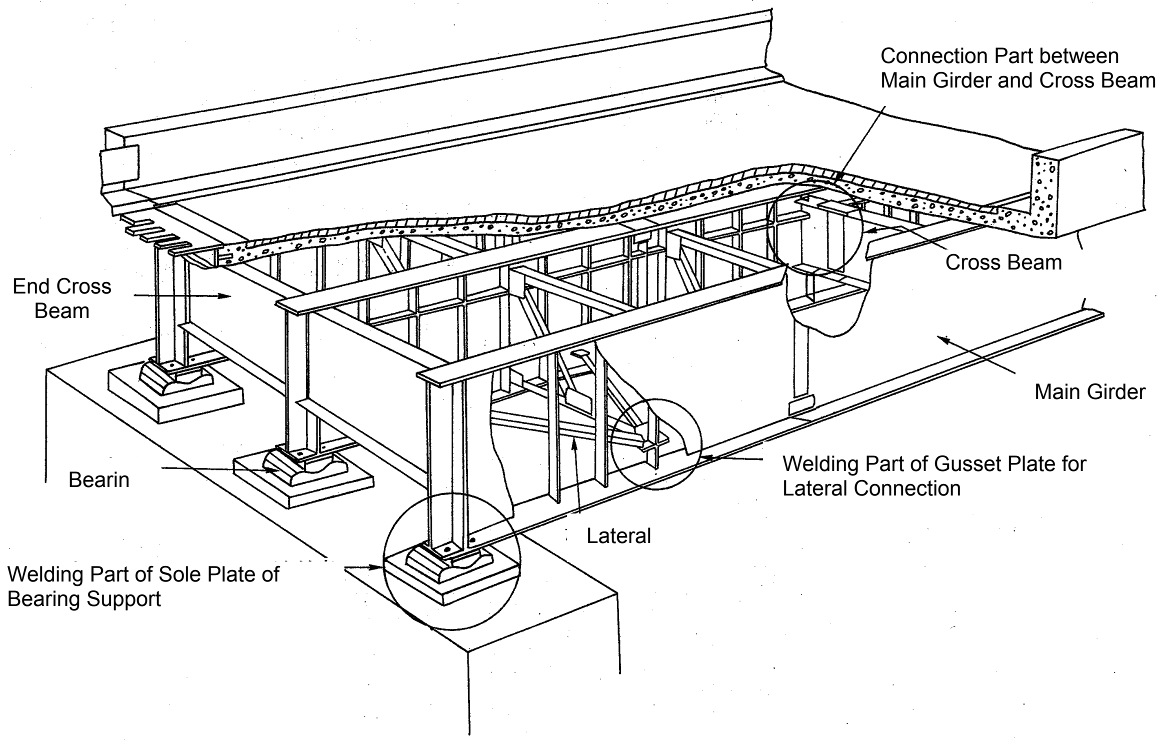


Figure 10.2.15 Easy Affected Parts by Fatigue for Steel Bridge

Selected measurement points for stress of the bridge are as shown in Table10.2.7 and Figure 10.2.16. The number of measurement points were controlled by the capacity of the data recorder, which could be arranged in Costa Rica. Therefore, No. 221 and 221 were used for only the static load test.

Table 10.2.7. Measurement Points for Stress

Point Number	Measurement Points	Purpose of Measurement	Remarks
111,112, 121,122 211,212, 221,222	8	Stress of Main Girder (Upper and Lower Flanges)	211 & 221 were only for static load
411,412	2	Fatigue of Welding at Sole Plate of Support	
311,312, 321,322,	4	Fatigue of Welding between Brace and Vertical Stiffener	
511,512	2	Fatigue of Welding of between Gusset Plate of Lateral Bracing and Web Plate	
Total	16		

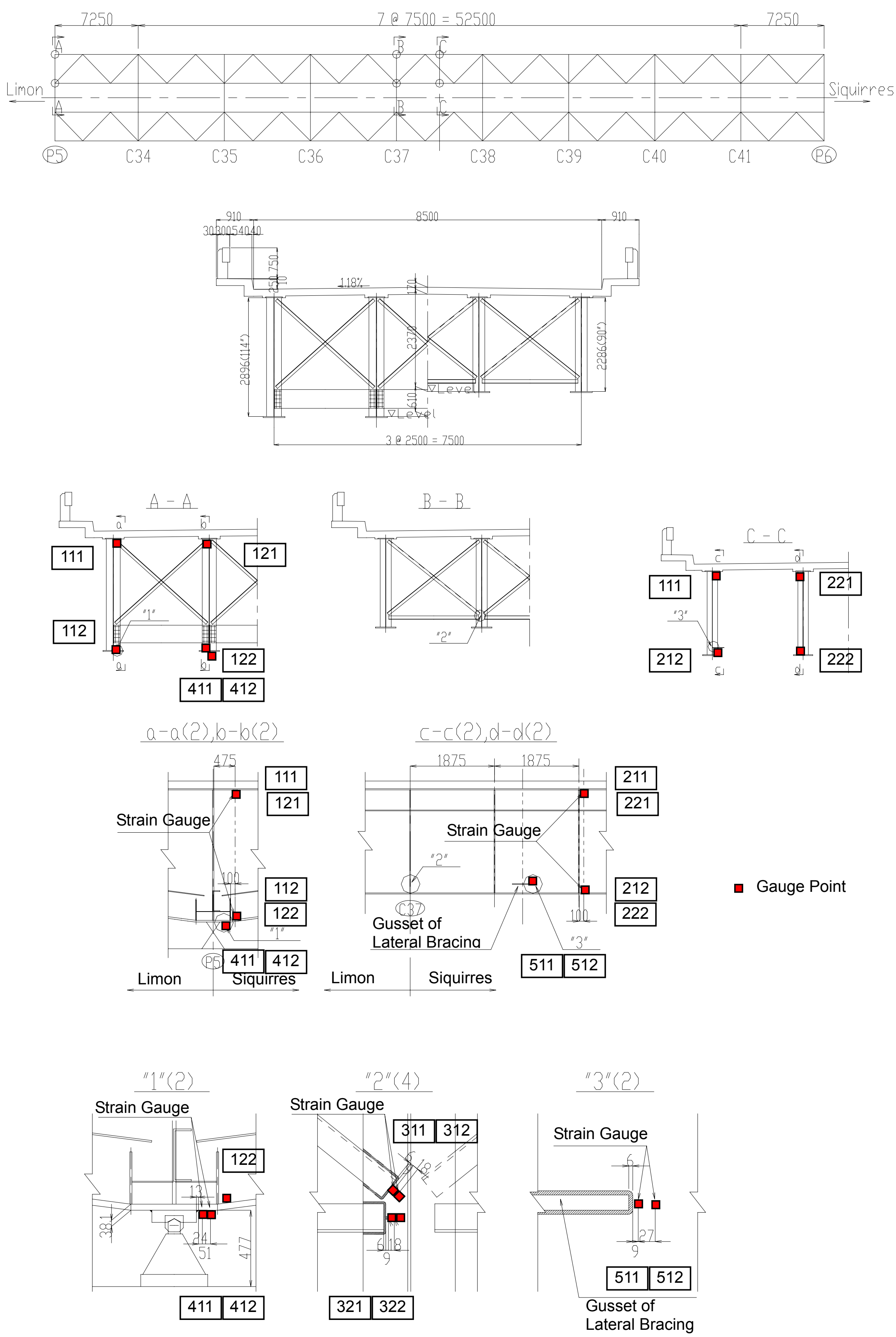


Figure 10.2.16 Gauge Setting Point on No.26 Chirripo Bridge

As most of members are connected by the welding on this bridge, the measurement points for fatigue were selected focusing on the welding parts. The stress at the welding part is evaluated with the "Hot Spot Stress". The hot spot stress is defined as the stress at the viewpoint (end of welding) in consideration of disorder of stress, which does not include the local stress concentration by welding. This hot spot stress cannot be measured directly, and calculated with the stresses at two points close to the viewpoint as shown in Figure 10.2.17. Therefore, to measure the hot spot stress, the two strain gauges were set each welding parts.

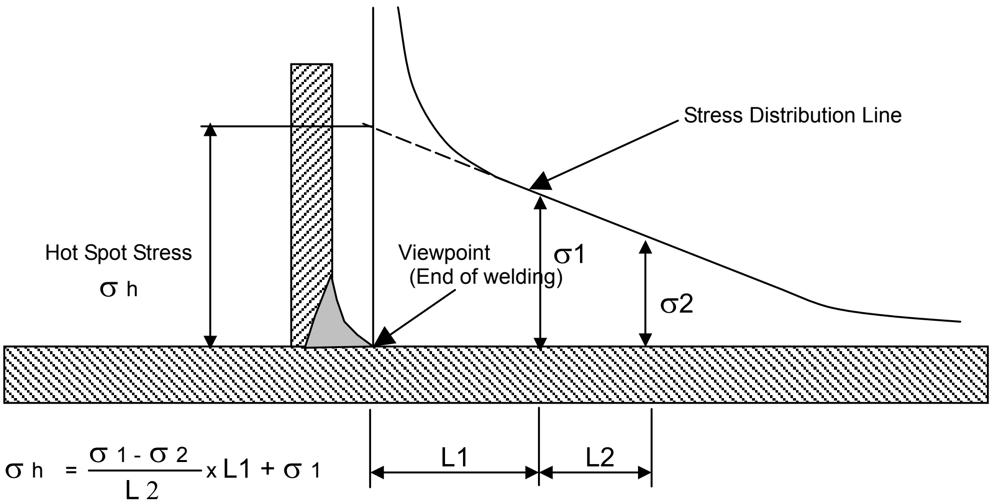


Figure 10.2.17 Hot Spot Stress

4) Measurement Equipment and Tools for Stress

The stress of member cannot be measured directory. The strain of member is measured and it can be transform into stress by multiplying the elastic modulus of material. The strain of member shall be measured by using a strain gauge. The strain gauge changes its electric resistance when it is expanded or shrieked. Using this characteristic together with the electrical circuit named Whetstone Bridge, which is set in the data recorder, the strain of material can be gotten as a change of an electric voltage.

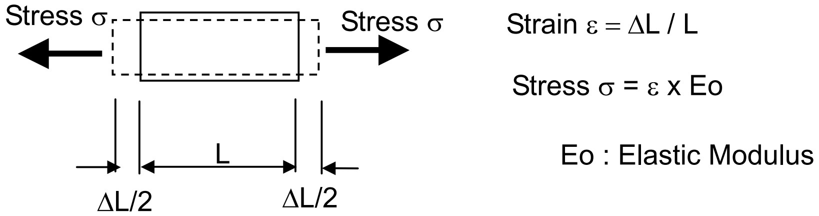


Figure 10.2.18. Stress and Strain

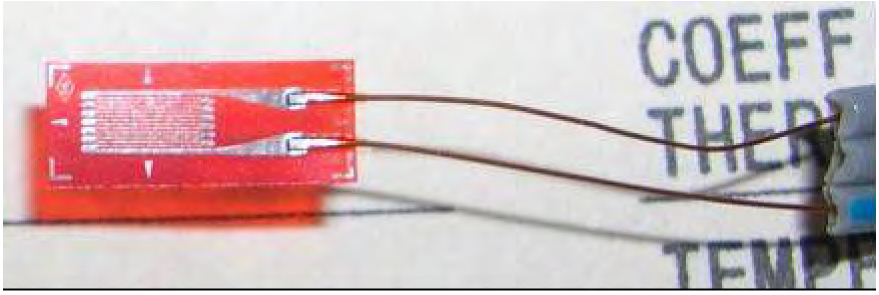


Figure 10.2.19. Strain Gauge

There are several kinds of strain gauges for the measurement purposes. For the bridge loading test, the values of stress are required for very small area, therefore the strain gauge with small gauge size are used. Especially, when the stress at welding point is required, more than 2 very small size strain gauges are used setting next to welding point with short distance, because the stress at welding point can not be measured directly. The strain gauge with 3.0 mm of gauge length was used for the stress measurement points on steel member and 0.5 mm of gauge length was used for the measurement of welding points.

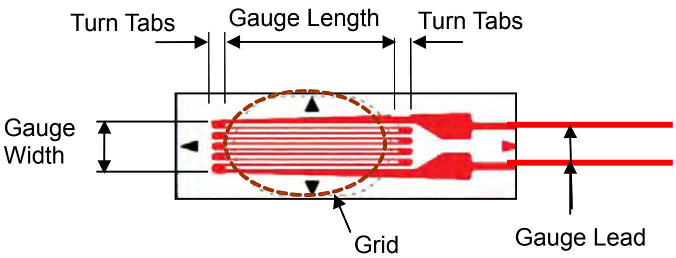


Figure 10.2.20. Detail of Strain Gauge

The strain data was recorded with the data recorder, which was borrowed from one of the concrete suppliers in Costa Rica. This recorder has 15 channels to connect the strain gauges and has a capacity to read data 10,000 times per second.

Figure 10.2.21. Data Recording System

5) Preparation Works for Test

a. Setting of Scaffolding and Temporally Electric Power Supply Line

The scaffolding for the stage of setting work yard was constructed under the bridge and the temporally electric power supply line, which can provide stable electric power to the measurement equipment and lighting system as shown in Figure 10.2.22.

Figure 10.2.22. Scaffolding for Working Stage

b. Setting of Strain Gauge and Deflection Measurement Tool

The series of loading tests was carried out by the counterparts of MOPT, Costa Rica together with the study as a part of technical transfer of bridge maintenance. After taking training and lecture for setting method of strain gauge, all gauges were set on the bridge by them. The flow of strain gauge setting is as shown in Figure 10.2.23 and 10.2.24. The deflection measurement tool was set at lower flanges of both external girders as shown in Figure 10.2.14.

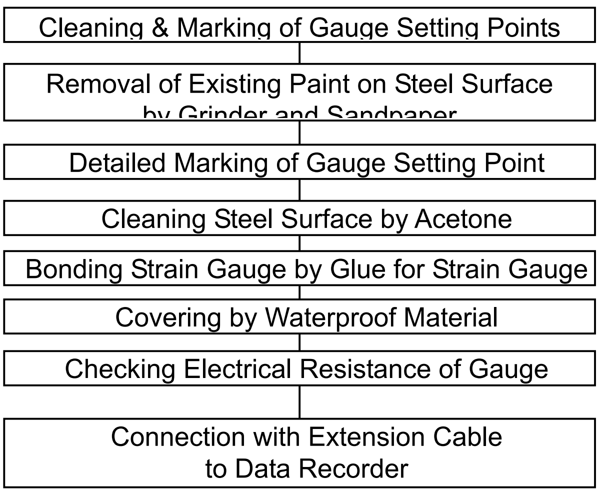


Figure 10.2.23. Strain Gauge Setting

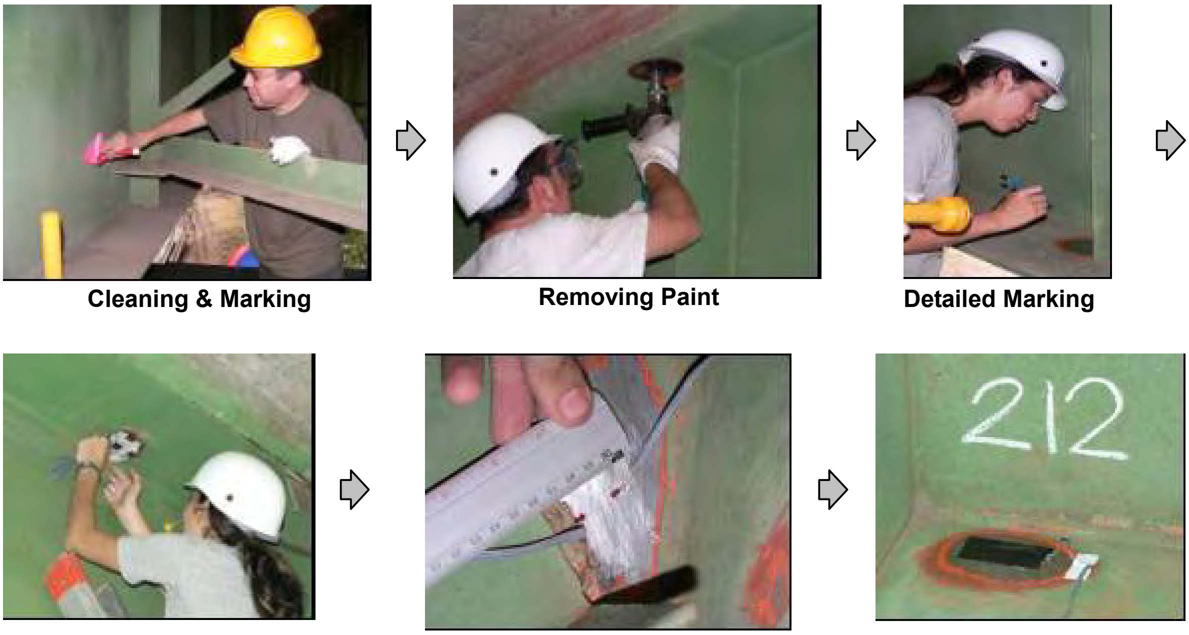


Figure 10.2.24. Strain Gauge Setting

6) Static Load Test

The four loading cases were carried out as the static load test on No.26 Chirripo Bridge to measure the stress and strain as in Table 10.2.8. and Figure 10.2.25.

Table 10.2.8. Static Load Case for No.26 Chirripo Bridge

Case	Viewpoint	No. of Truck	Loading Points
1	-Maximum Stress at Support of P5 of Main Girder	1	Center of span P6 - P7, One lane
2	-Maximum Stress at Center of Span P5 - P6 of Main Girder	1	Center of span P5 - P6, One lane
3	-Minimum Stress at Support of P5 and at center of span P5 - P6 Main Girder	1	Center of span P4 - P5, One lane
4	-Maximum Stress at Cross Frame in Span P5 – P6	1	Cross Frame in span P5 - P6, One lane

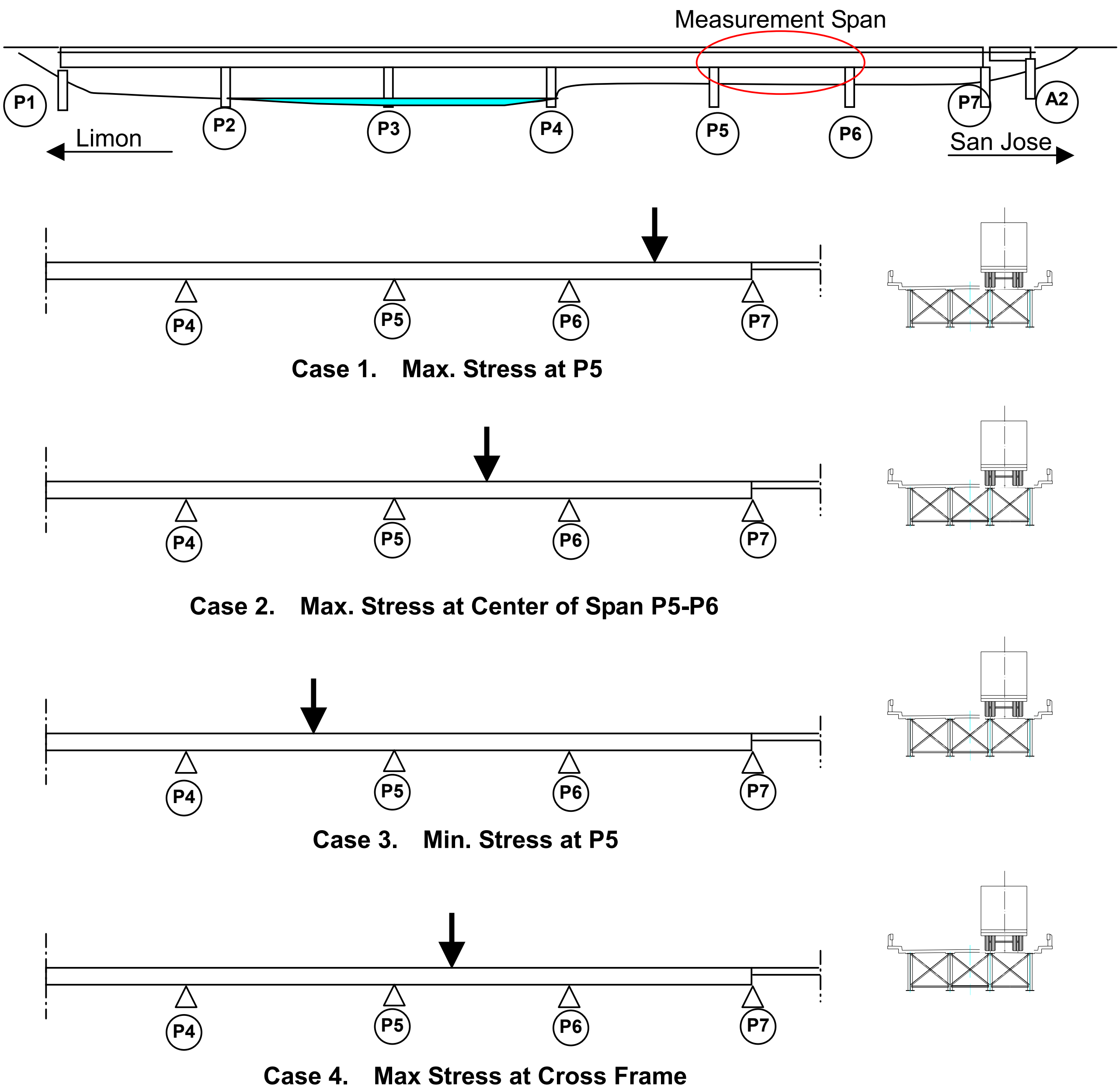


Figure 10.2.25. Static Load Cases for Bridge No. 26

7) Dynamic Load Test

Flowing Two types of dynamic load test were carried out to measure the stress frequency

- Running Load using 25 ton Load Truck
- Actual Traffic Load

a. Running Load Test

Running Load Test was carried out using Load Truck with weight of 25 ton. The stress frequency for running load was measured for three cases of truck running speeds of 20, 40 and 60 km/h.

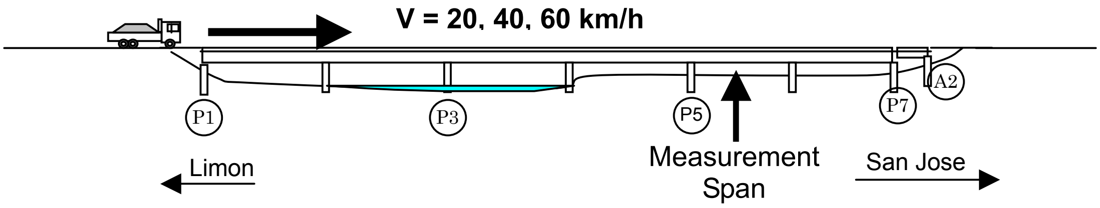


Figure 10.2.26. Image of Moving Load Test on No.27 Chirripo Bridge

b. Actual Traffic Load

The stress frequency Test for actual traffic load was measured for 24 hours. During measurements period, the traffic volume survey was carried out with the classification of standard in MOPT by traffic surveyors from MOPT as shown in Table 10.2.9.

Table 10.2.9. Classification of vehicles for Traffic Survey

	Class	Vehicle Types
Light Vehicles	1	Passenger
	2	Light Cargo
Heavy Vehicles	3	Busses
	4	2 Axial Truck
	5	3 Axial Truck
	6	4 Axial Truck
	7	5 or more Axial Truck

8) Operation of Loading Test

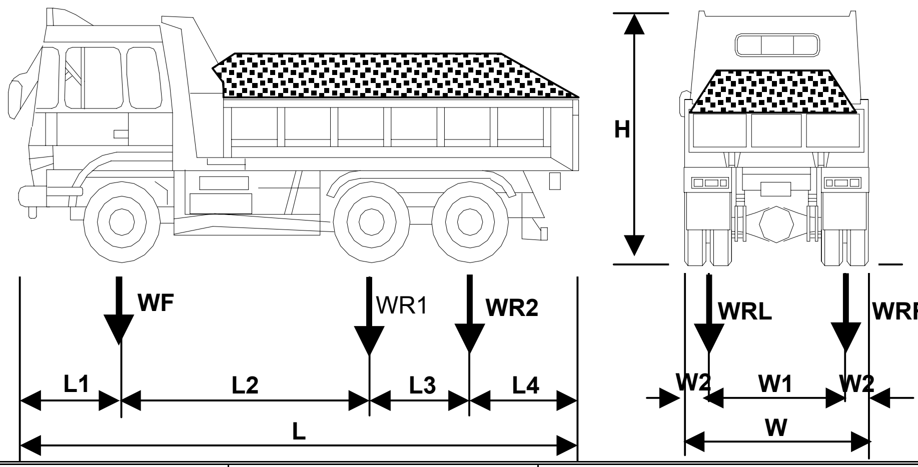
The static load test and the running load test on No.26 Chirripo Bridge was carried out on 30th August with the study team and the counter part of MOPT. The traffic suspension with 30 minutes was planed for each loading case, and the road was opened for traffic with 15 minute between two loading cases. However, to reduce the effect to the traffic on Route 32, the schedule was changed as shown in Table 10.2.10. After the final site meeting, the measurement of loading truck and the setting of the deflection measurement tool were carried out at same time. The truck with weight of 25 ton including gravel for load was used for the test load. The dimensions and weights of loading truck are as shown in Table 10.2.11.

Table 10.2.10. Detailed Time Schedule of Test

Time	Work Item
30/8, 01:30	Site Meeting: All Staff
02:00	- Truck load measurement - Lighting setting - Connection strain gauge and data gathering system - Deflection tool setting
04:00	**-Static Load Test for 1st loading position**
04:20	Open for traffic
04:30	**-Static Load Test for 2nd, 3rd and 4th loading position**
04:50	Open for traffic
05:00	**-Moving Load Test for 1st speed**
05:20	Open for traffic
05:30	**-Moving Load Test for 2nd speed**
05:45	Open for traffic
05:55	**-Moving Load Test for 3rd speed**
06:10	Open for Traffic

Table 10.2.11. Truck Dimensions and Weight Used for Loading

Figure



Truck			Truck 1
Dimensions (mm)	Length	L	8,140
		L1	1,270
		L2	4,410
		L3	1,310
		L4	1,150
	Width	W	2,400
		W1	1,800
		W2	300
Weight (tone)	Front Wheel	Left	3.405
		Right	3.640
		Total	7.045
	Rear Wheel 1	Left	5.510
		Right	5.130
		Total	10.640
	Rear Wheel 2	Left	5.445
		Right	4.985
		Total	10.430
	Total	Left	14.360
		Right	13.755
		Total	**28.115**

The actual traffic load test was carried out for 24 hours from noon on 31st August after the running load test. The traffic volume survey was also carried out during same period.

Truck Position Marking

Gauge Cable Connection

Deflection Measurement Tool

Strain Data Recorder

Running Load Test

Traffic Volume Survey

Figure 10.2.27. Situation of Loading Test on No.26 Bridge

10.2.9 Results and Conclusion for Loading Test on No.17 Chirripo Bridge

1) Results of the Test

The results of deflection for the static load test on No.17 Chirripo Bridge were as shown in Table 10.2.12 and Figure 10.2.28. The deflections on the model were calculated by the structural model using the actual load conditions in taking account of position and weight of each axle of loading trucks.

Table 10.2.12. Comparison of Deflection on No.17 Chirripo Bridge

Load Case	Upstream Side			Downstream Side		
	Actual (mm)	Model (mm)	Difference (%)	Actual (mm)	Model (mm)	Difference (%)
Case 1 Max Positive	+ 49.3	+ 60.6	81.4	+ 49.3	+ 60.6	81.4
Case 2 Torsion	+ 41.5	+ 55.1	75.3	+ 40.7	+ 53.6	76.0
Case 3 Max Negative	- 12.0	- 17.7	67.8	- 12.1	- 17.7	68.4

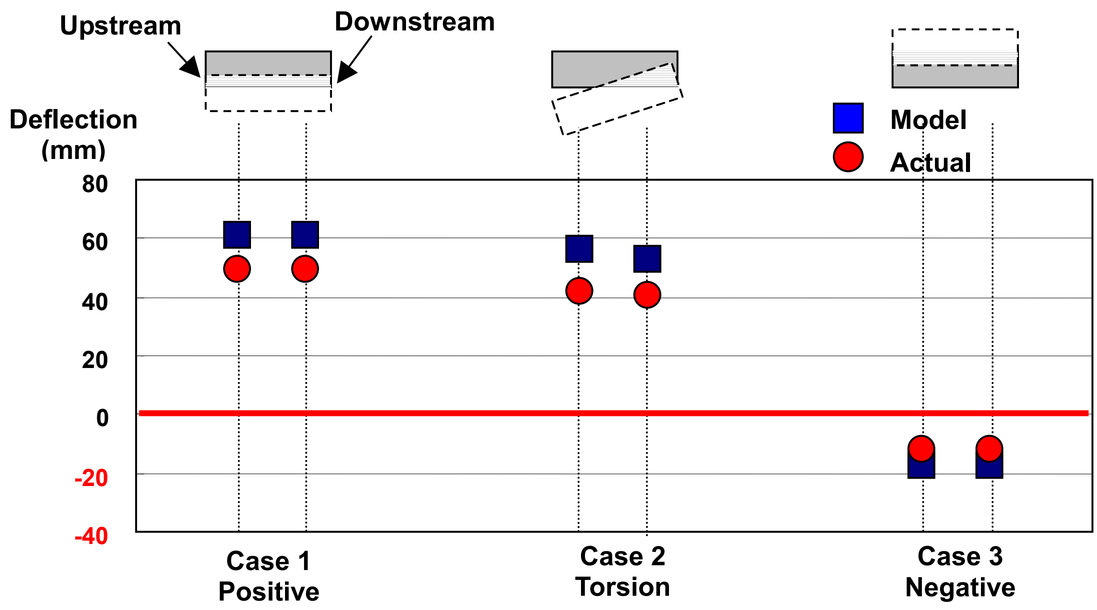


Figure 10.2.28. Comparison of Deflection on No.17 Chirripo Bridge

The actual deflections are less than the model analyzed ones in all cases with the differences between 67 to 82 %. This means that the actual stiffness of the girder is lager than that of model. The actual stiffness of the bridge includes the stiffness of concrete barriers on both sides, which are ignored in the structural analysis in the design. The stiffness of the main girder including concrete barrier is about 110 to 125% of the main girder. As the deflection is calculated by M / (E x I), the deflection in taking account with the barrier becomes 90 to 80 %. Therefore, the actual stiffness of the bridge is lager than that of the analysis taking account with the barrier.

2) Conclusion

The actual stiffness of the bridge is lager than the stiffness, which can be calculated from the drawings.

Therefore, the existing condition of the bridge main girder can be considered as followings;

1) The construction work satisfied the requirement of original design and/or specification. And the quality of main girder satisfies the requirement of original design.

2) In the girder, there are not any cracks or defects that lead decline of stiffness of the girder.

3) Decrease of elastic modulus of concrete caused by deterioration, such as carbonation, is not occurred

[Conclusion]

The load carrying capacity of the bridge for HS20+25% can be evaluated by the analysis using the model, which is made based on original design conditions.

10.2.10 Results and Conclusion for Loading Test on No.26 Chirripo Bridge

1) Results of Static Load Test

a. Comparison for Deflection and Stress

The results of deflection and stress at the center of span P5 – P6 for the static load test on No.26 Chirripo Bridge were as shown in Table 10.2.13 and 10.2.14. (Load cases are as shown in Figure 10.2.29.). The deflections on the model were calculated by the structural model using the actual load conditions in taking account of position and weight of each axle of loading trucks.

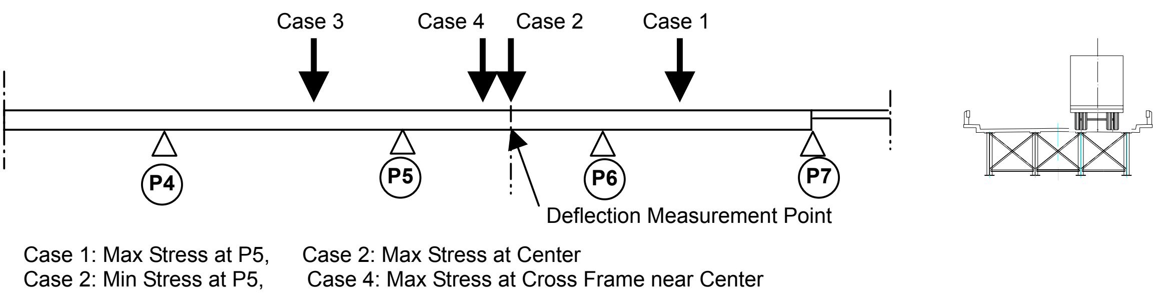


Figure 10.2.29. Static Load Case for No.26 Chirripo Bridge

Table 10.2.13. Comparison of Deflection at Center of Span

Load Case	Upstream Side			Downstream Side		
	Actual (mm)	Model (mm)	Difference (%)	Actual (mm)	Model (mm)	Difference (%)
Case 1	- 5.9	- 6.2	95.2	- 4.2	- 4.5	93.3
Case 2	+ 19.6	+ 21.0	93.3	+ 8.7	+ 9.4	92.6
Case 3	- 5.9	- 4.3	137.2	- 4.4	- 3.1	141.9
Case 4	+ 19.2	+ 20.6	94.6	+ 8.7	+ 9.2	94.6

Table 10.2.14. Comparison of Stress at Lower Flange at Center of Span

Load Case	Upstream Side			Downstream Side		
	Actual (N/mm^2)	Model (N/mm^2)	Difference (%)	Actual (N/mm^2)	Model (N/mm^2)	Difference (%)
Case 1	-5.9	-7.0	84.3	-4.2	-2.6	161.5
Case 2	19.6	22.4	87.5	8.7	4.5	193.3
Case 3	-5.9	-4.5	131.1	-4.4	-1.1	400.0
Case 4	19.2	21.3	90.1	8.7	4.4	197.7

The actual deflections is smaller the canalized results for each case except case 3. The actual differences between both girders are smaller than that of model analysis. This means that the actual stiffness of the girder is lager than that of model. This bridge is a continuous bridge and was designed as the composite girder around the part of span center and as no-composite girder around support areas. And the girders are connected together only by the sway-bracings in the design. However, the actual of the bridge is considered as a composite girder for all sections and the girders are connected strongly by the slab deck not only sway bracings. For the cases 1 and 3, which are loaded at the next span for the deflection measurement point, the deflections are small and are easy to be affected by the difference of the stiffness of the girder between actual bridge and the bridge model.

b. Comparison with Revise of Structural Model

As paying attention to the differences between deflections of two girders, the structural model was revised by increasing the stiffness of the connection member between girders in taking account of the stiffness by the slab deck. The defections and stresses by the revised model become closer to the actual ones than by the original model as shown in Figure 10.2.30 and 10.2.31.

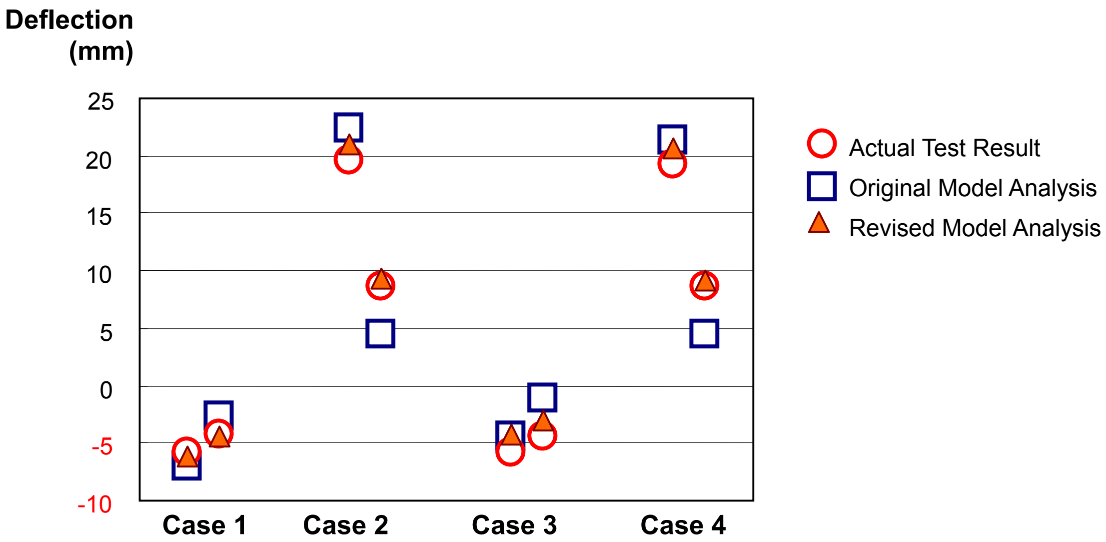


Figure 10.2.30. Comparison of Deflections Between Actual and Models

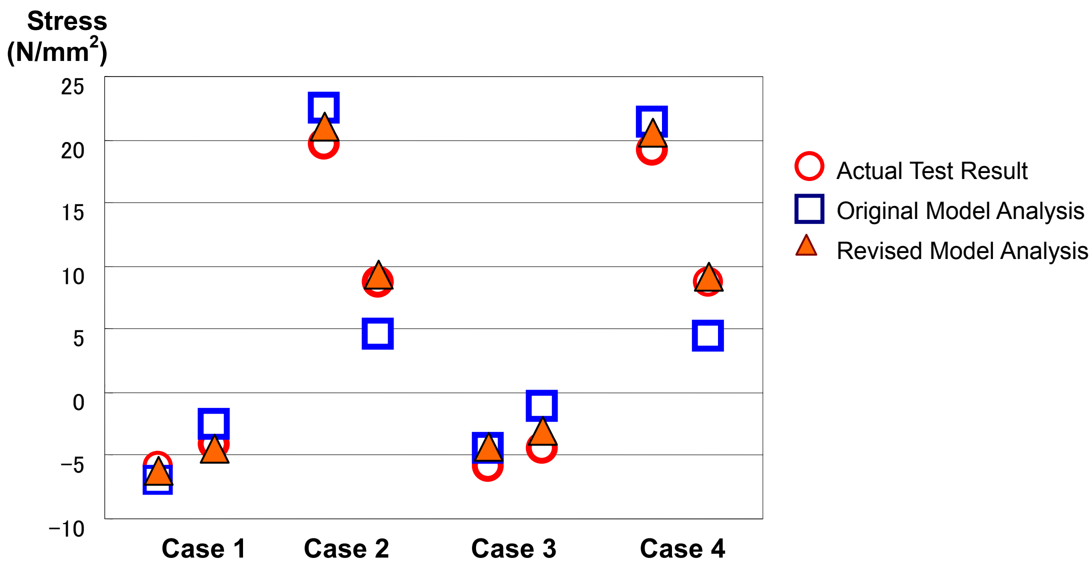


Figure 10.2.31. Comparison of Stress Between Actual and Models

c. Stress for Increased Design Load HS20+25%

Based on the revised structural model, the deflection and stresses of flange were analyzed for the increased design live load HS20 + 25%. Both deflection and stress at the center of the span are smaller than the allowable ones as shown in Figure 10.2.32.

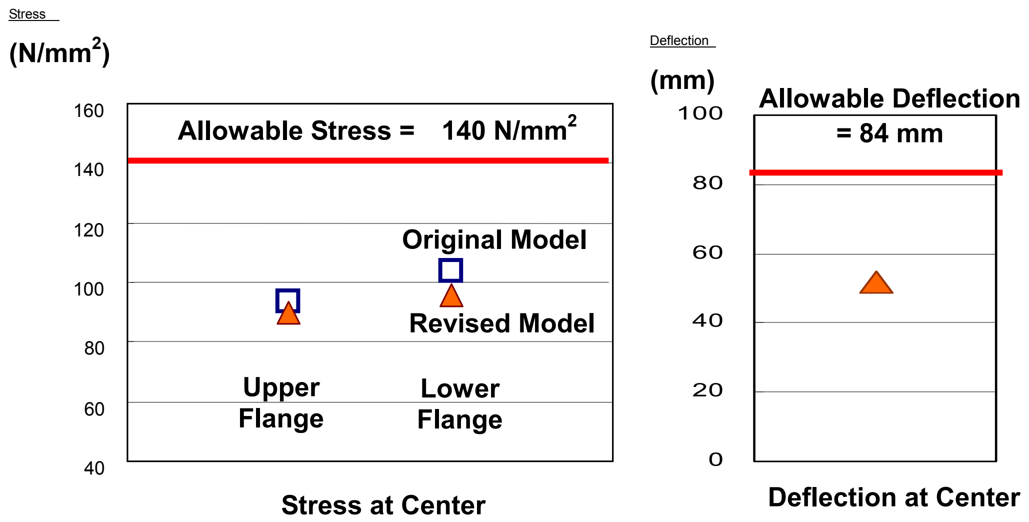


Figure 10.2.32. Stress and Deflection for HS20+25% by Revised

2) Result of Running Load Test

The results of deflection for the Running load test on No.26 Chirripo Bridge were as shown in Table 10.2.15. The deflections on the model were calculated by the structural model using the actual load conditions in taking account of position and weight of each axle of loading trucks.

The differences of the deflection between the running speeds are not observed in this test result. This means the impact, which is considered for the design as a 30 per cent of live load was not confirmed by this test.

Table 10.2.15. Comparison of Deflection for Running Speed

Speed (km/h)	Upstream Side				Downstream Side			
	Static	20	40	60	Static	20	40	60
Deflection (mm)	19.6	18.8	19.2	18.1	8.7	9.3	9.2	10.1

3) Result of Traffic Load Test

The stresses for the member to evaluate the fatigue were measured under the traffic load test for 24 hours at the points as shown in Figure 10.2.33.

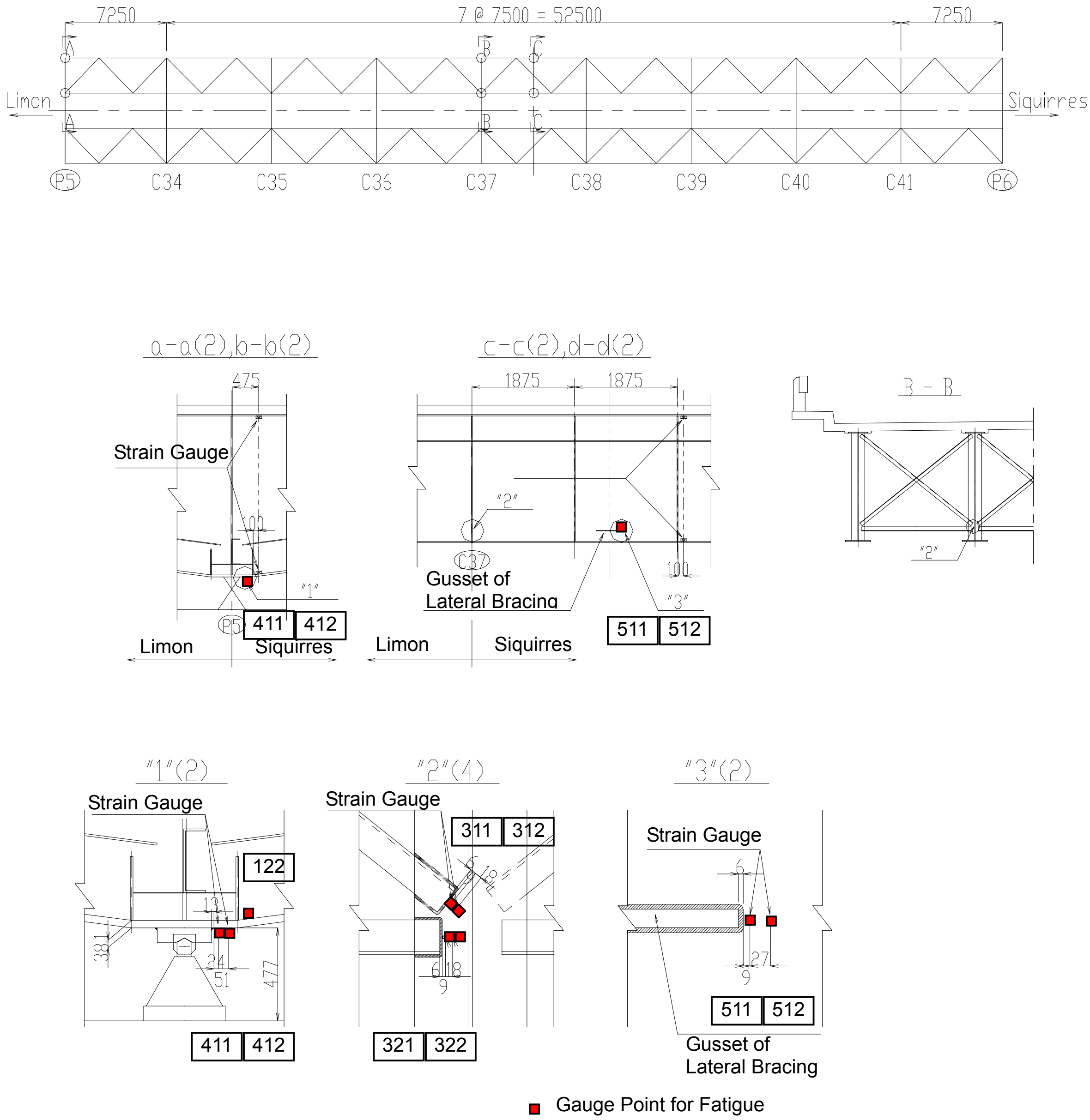


Figure 10.2.33. Stress Measurement Point for Evaluation of Fatigue

(1) Evaluation Method of Durability for Fatigue

To evaluate the durability of steel member for fatigue, the traffic data on the route where bridge locate as well as stress frequency data measured by the actual traffic load test. The evaluation sequence of durability for fatigue is as shown in Figure 10.2.34.

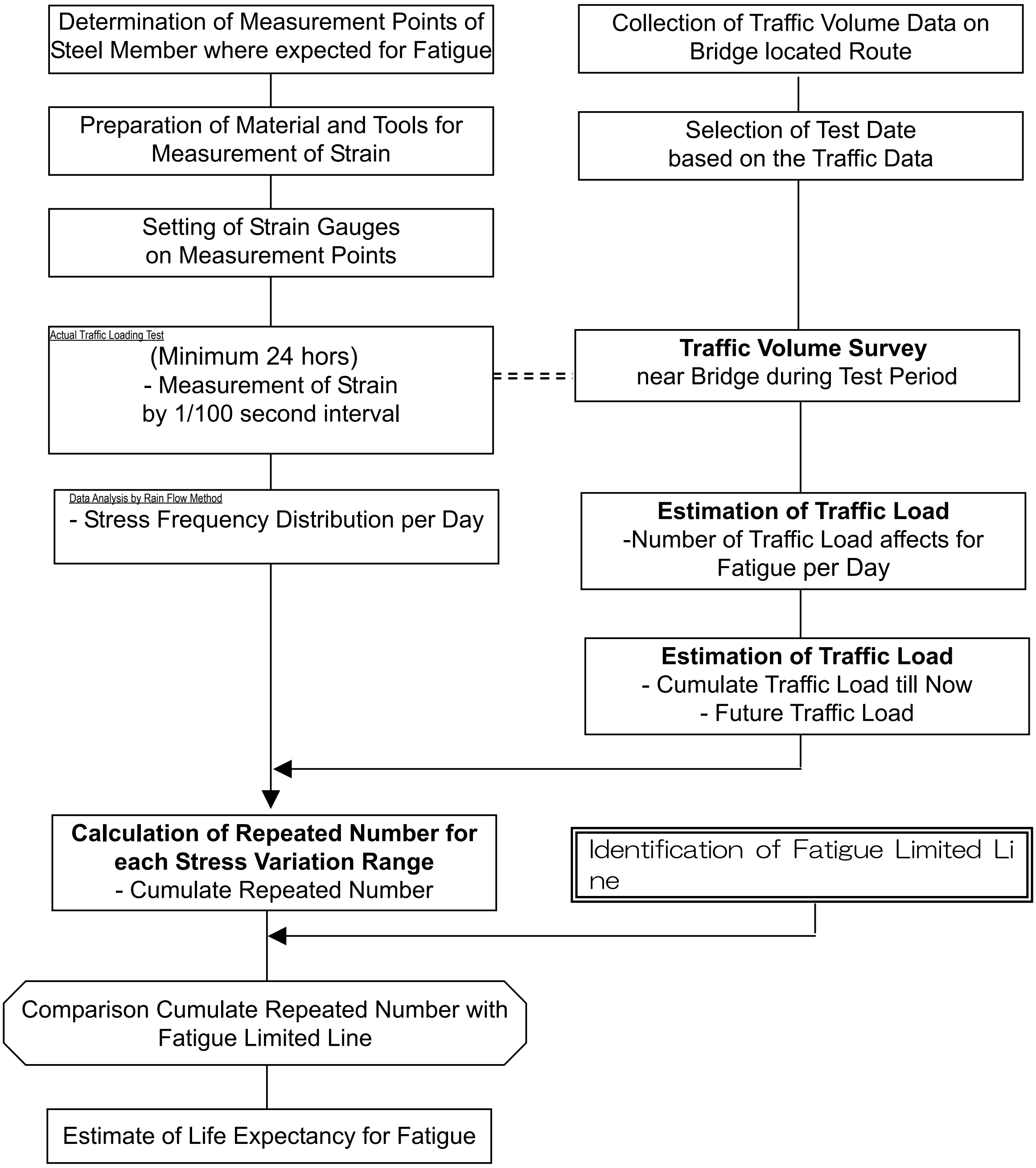


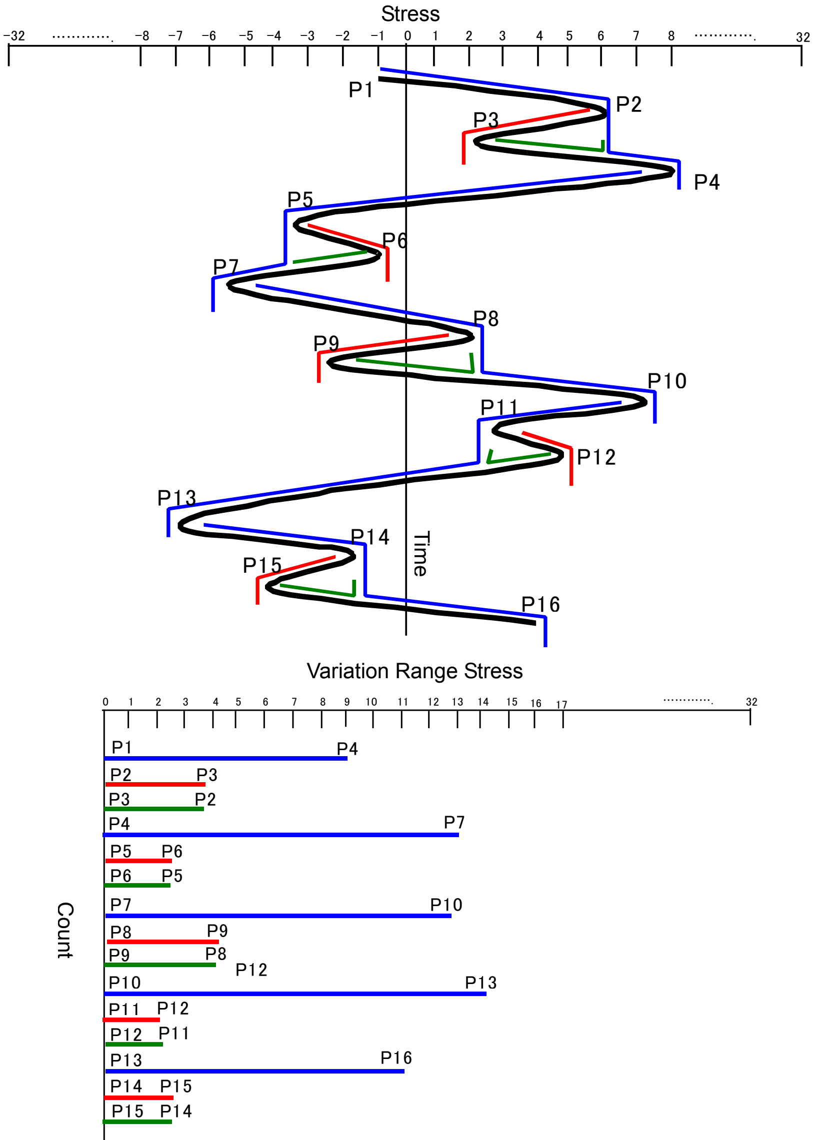
Figure 10.2.34. Evaluation Sequence of Durability for Fatigue

The stress data under the traffic load were recorded for 24 hours with the data recorder in this test. After the test, the stress data were counted for each stress variation rage by the Rain Flow Method. The counted data for each stress range were evaluated by the linear accumulation damage low (Miner's low) .

a. Method for Analysis of Stress Frequency Distribution

The stress data under the traffic load were recorded for 24 hours with the data recorder in this test. After the test, the stress data were counted for each stress variation rage by the Rain Flow Method. The counted data for each stress range were evaluated by the linear accumulation damage low (Miner's low) .

The Rain Flow Method is a method to transform the stress wave to the stress frequency distribution by attending a variation rage of stress (i.e. a difference between maximum value and minimum value). The wave pattern is considered as a multiplex roof and the differences are counted like as that raindrops flow down from the root of all roofs as shown in Figure 10.2.35. The stress frequency distribution during test period is summarized to the repeated number for each variation rang.



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	....	64	Range
0	2	4	2	2	0	0	0	0	1	0	1	1	1	1	0	0	....	0	Count

Figure 10.2.35. Image of Rain Flow Method

b. Cumulated Repeated Number for Stress Frequency Distribution

The cumulated repeated number for each stress variation range can be calculated based on the traffic volume observed during the test period and the traffic volume data for past years. The traffic volume used for the affection by fatigue.

Traffic Volume during Test Period

	Class	Vehicle Type	Number
Light Vehicles	1	Passenger	XXXX
	2	Light Cargo	XXXX
Heavy Vehicles	3	Busses	XXXX
	4	2 Axial Truck	XXXX
	5	3 Axial Truck	XXXX
	6	4 Axial Truck	XXXX
	7	5 or more Axial Truck	XXXX

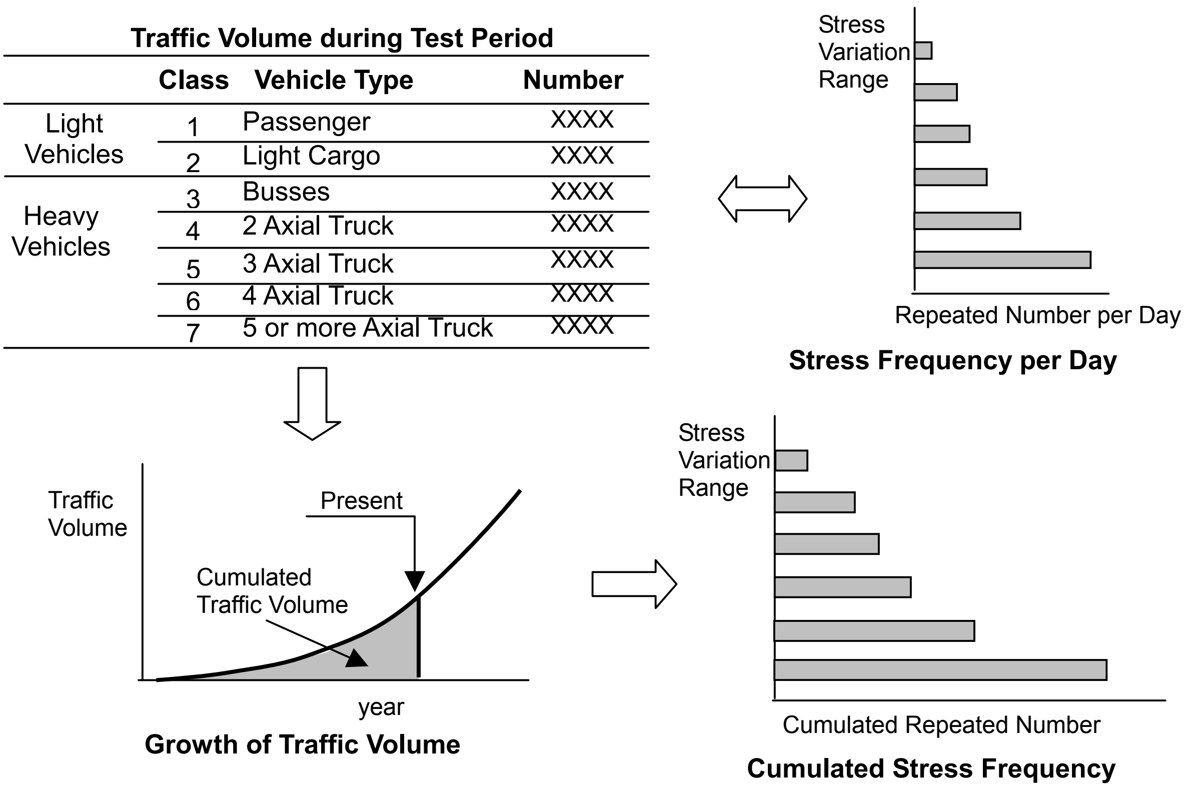


Figure 10.2.36. Cumulated Repeated Number

c. Evaluation of Durability for Fatigue

The damage by fatigue occurs when the repeated number of stress variation becomes the limit number, which depended on the stress variation range. The degree of fatigue damage is evaluated with the linier damage method. By this method, the damaged degree is evaluated by the value of cumulated degree of fatigue D calculated with the following equation. When D reaches to 1.0, damage by fatigue will be occurred.

In the case to evaluate about a fatigue, only the traffic volume of heavy weight vehicles is considered, because the influence for fatigue by lightweight vehicles can is neglect.

Where the stress variation range is smaller than the certain range, the fatigue is not occurred even if any number linear accumulation damage of stress variation is repeated.

$$D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \text{------} + \frac{n_i}{N_i} + \text{------} + \frac{n_j}{N_j} + \text{----}$$

Where, D: Cumulated Degree of Fatigue
N_i: limit repeated number, which occurs damage by fatigue for the stress variation range $\Delta\sigma_i$
$= 2{,}000{,}000 \times (\text{Ave. } \Delta\sigma_i^3 / \sigma\text{lim.}^3)$
$\sigma_{lim.}$: Basic Stress Variation Range for 2,000,000 repeated stress, depends on stress class of the position of member.
n_i: Cumulated repeated for the stress variation range $\Delta\sigma_i$
n_i / N_i: Degree of fatigue for the stress variation range $\Delta\sigma_i$

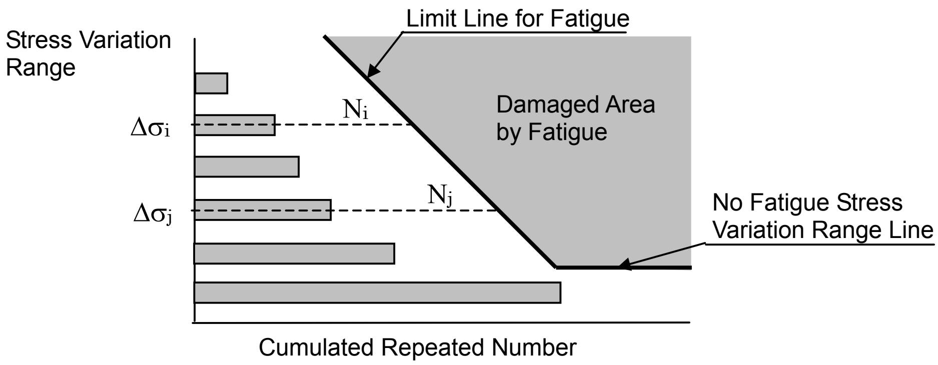


Figure 10.2.37. Image of Cumulated Degree of Fatigue

(2) Results of Actual Traffic Load Test

a. Stress frequency for Each Member

The stress frequency data for each member under the actual traffic load test for 24 hours are as shown in table 10.2.16. to 10.2.19.

Table 10.2.16. Frequency of Stress at Connection part of Sole Plate

Stress Variation Range $\Delta\sigma_i$ (N/mm^2)	Average Stress of Range Ave. $\Delta\sigma_i$ (N/mm^2)	Number of Occurrence n_i	Basic Stress Variation Range for 2,000,000 Repeated Stress σlim (N/mm^2)	Allowable Number for Average Stress of Range $N_i = 2{,}000{,}000 \times (\text{Ave. } \Delta\sigma_i^3 / \sigma_{lim.}^3)$	Degree of fatigue for Ave. $\Delta\sigma_I$ (%)
100 – 120	110	2	72	560,853	0.00036
80 – 100	90	12		1,024,000	0.00112
60 – 80	70	42		2,176,373	0.00191
40 – 60	50	138		5,971,968	0.00231
20 – 40	30	377	Below Fatigue Limit	-	-
0 - 20	10	1,963		-	-
	Total				0.00570

Table 10.2.17. Frequency of Stress at Connection part of Sway Bracing 1

Stress Variation Range $\Delta\sigma_i$ (N/mm^2)	Average Stress of Range Ave. $\Delta\sigma_i$ (N/mm^2)	Number of Occurrence n_i	Basic Stress Variation Range for 2,000,000 Repeated Stress σlim (N/mm^2)	Allowable Number for Average Stress of Range $N_i = 2{,}000{,}000 \times (Ave.\ \Delta\sigma_i^3 / \sigma_{lim.}^3)$	Degree of fatigue for Ave. $\Delta\sigma_I$ (%)
100 – 120	110	1	80	769,346	0.00013
80 – 100	90	2		1,404,664	0.00014
60 – 80	70	24		2,965,423	0.00079
40 – 60	50	66		8,192,000	0.00081
20 – 40	30	205	Below Fatigue Limit	-	-
0 - 20	10	1,331		-	-
	Total				0.00187

Table 10.2.18. Frequency of Stress at Connection part of Sway Bracing 2

Stress Variation Range $\Delta\sigma_i$ (N/mm^2)	Average Stress of Range Ave. $\Delta\sigma_i$ (N/mm^2)	Number of Occurrence n_i	Basic Stress Variation Range for 2,000,000 Repeated Stress σlim (N/mm^2)	Allowable Number for Average Stress of Range $N_i = 2{,}000{,}000 \times (Ave.\ \Delta\sigma_i^3 / \sigma_{lim.}^3)$	Degree of fatigue for Ave. $\Delta\sigma_I$ (%)
100 – 120	110	0	80	769,346	0.00000
80 – 100	90	1		1,404,664	0.00007
60 – 80	70	1		2,985,423	0.00003
40 – 60	50	13		8,192,000	0.00016
20 – 40	30	94	Below Fatigue Limit	-	-
0 - 20	10	831		-	-
	Total				0.00026

Table 10.2.19. Frequency of Stress at Connection part of Lateral

Stress Variation Range $\Delta\sigma_i$ (N/mm^2)	Average Stress of Range Ave. $\Delta\sigma_i$ (N/mm^2)	Number of Occurrence n_i	Basic Stress Variation Range for 2,000,000 Repeated Stress σlim (N/mm^2)	Allowable Number for Average Stress of Range $N_i = 2{,}000{,}000 \times (Ave.\ \Delta\sigma_i^3 / \sigma_{lim.}^3)$	Degree of fatigue for Ave. $\Delta\sigma_I$ (%)
100 – 120	110	1	80	769,346	0.00006
80 – 100	90	9		1,404,664	0.00061
60 – 80	70	41		2,985,423	0.00136
40 – 60	50	150		8,192,000	0.00183
20 – 40	30	339	Below Fatigue Limit	-	-
0 - 20	10	2,310		-	-
	Total				0.00386

b. Traffic Volume during the Test

The traffic Volume during the actual traffic load test was as shown in Table 10.2.20.

Table 10.2.20. Traffic Volume during Test (Both Sides)

Hours		Light Vehicles		Heavy Vehicles					TOTAL
From	To	Passenger	Pick Up	Bus	2 Axles	3 Axles	4 Axles	5 Axles	
12:00	13:00	137	58	12	39	8	0	81	335
13:00	14:00	144	55	19	28	6	0	113	365
14:00	15:00	165	87	14	27	10	0	103	406
15:00	16:00	168	68	17	20	5	0	129	407
16:00	17:00	162	75	16	29	17	0	145	444
17:00	18:00	224	80	16	28	6	0	133	487
18:00	19:00	162	42	11	17	4	0	135	371
19:00	20:00	105	35	11	20	5	0	108	284
20:00	21:00	87	19	1	12	1	0	140	260
21:00	22:00	57	13	1	5	6	0	99	181
22:00	23:00	32	11	2	6	1	0	77	129
23:00	0:00	19	5	1	3	2	0	71	101
0:00	1:00	5	3	0	0	1	0	52	61
1:00	2:00	7	0	0	1	0	0	37	45
2:00	3:00	4	0	0	0	0	0	20	24
3:00	4:00	5	1	0	3	0	1	19	29
4:00	5:00	29	11	4	6	0	0	40	90
5:00	6:00	54	36	15	17	7	1	70	200
6:00	7:00	94	54	23	28	5	0	63	267
7:00	8:00	133	66	24	19	10	0	65	317
8:00	9:00	131	75	21	32	7	0	64	330
9:00	10:00	141	67	16	39	8	0	85	356
10:00	11:00	162	89	11	40	11	0	78	391
11:00	12:00	148	69	15	37	12	0	69	350
TOTAL		**2,375**	**1,019**	**250**	**456**	**132**	**2**	**1,996**	**6,230**
		38.1%	16.4%	4.0%	7.3%	2.1%	0.0%	32.0%	100.0%

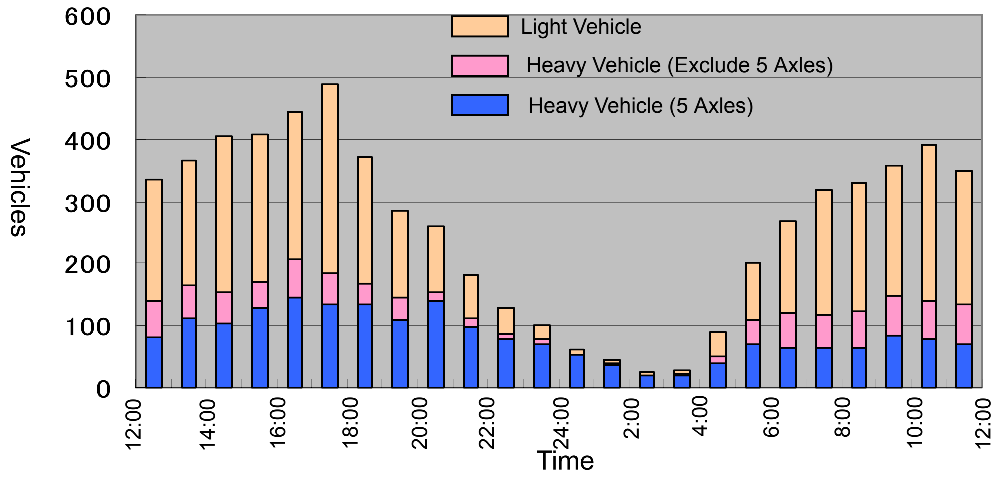


Figure 10.2.38. Traffic Volume during Test (Both Sides)

c. Estimation of Durability for fatigue

When the part of steel member is not damaged by fatigue, i.e. the cumulated degree of fatigue D is less than the value of 1.0, the life expectancy is estimated by the traffic volume prediction. The cumulated degree of fatigue per one vehicle is calculated by the results of the actual traffic load test. Based on this degree, the limit number of vehicle, which occurs the damage of fatigue, can be calculated. Then, the year when the cumulated traffic number reaches to the limit number can be estimated by cumulated traffic volume curve as shown in Figure 10.2.37. The life expectancy for fatigue is remained period to that year from now.

$$D(v) = D(day) / V(day)$$
$$N_f = 1.0 / D(v)$$

Where, $D(v)$: Degree of Fatigue by One Vehicle
$D(day)$: Degree of Fatigue for One
$V(day)$: Traffic Volume per day based on the Loading Test
N_f: Cumulate Vehicle Number occurs the Damage of Fatigue.

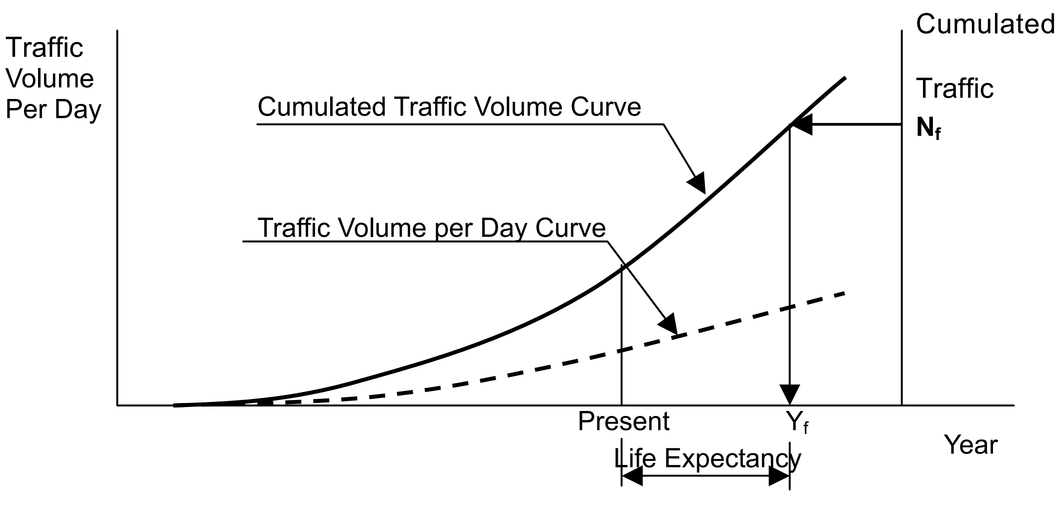


Figure 10.2.39. Life Expectancy and Cumulated Traffic Volume

As it is impossible to measure the weight of each vehicle during the test, it is assumed that these large stresses were caused by only the 5 axles truck. The number of vehicle during the test was 1,996 as shown in Table 10.2.20. Therefore, The cumulated vehicle number for each measured part are calculated as in Table 10.2.21.

Table 10.2.21. Cumulated Vehicle Number Occurs Damage

Part	Degree of Damage during Test D (day) (%)	Number of Heavy Vehicles During Test V(day) (No.)	Cumulated Vehicle Number Occurs Damages Nf (No.)
Sole Plate	0.00570	1,996	35,069,339
Sway-Bracing 1	0.00187	1,996	107,120,710
Sway-Bracing 2	0.00026	1,996	758,602,892
Lateral	0.00386	1,996	51,791,664

Based on the growth rate of traffic volume per year, the degrees of damage for fatigue at present can be assumed and the life expectancy for fatigue also estimated.

If it was assumed that the traffic volume increase at constant per year, and the traffic volume per day is Vp at the Yp years from the completion of the bridge. The growth rate A and the cumulated traffic number Np can be calculated as the following equations.

$$A = Vp / Yp$$

$$Np = A / 2 \times Yp^2 \times 365$$

The growth of the 5 axles trucks and the cumulated volume based on the number of trucks during the test and the years of 28 after completion of the bridge, is shown as Figure 10.2.40.

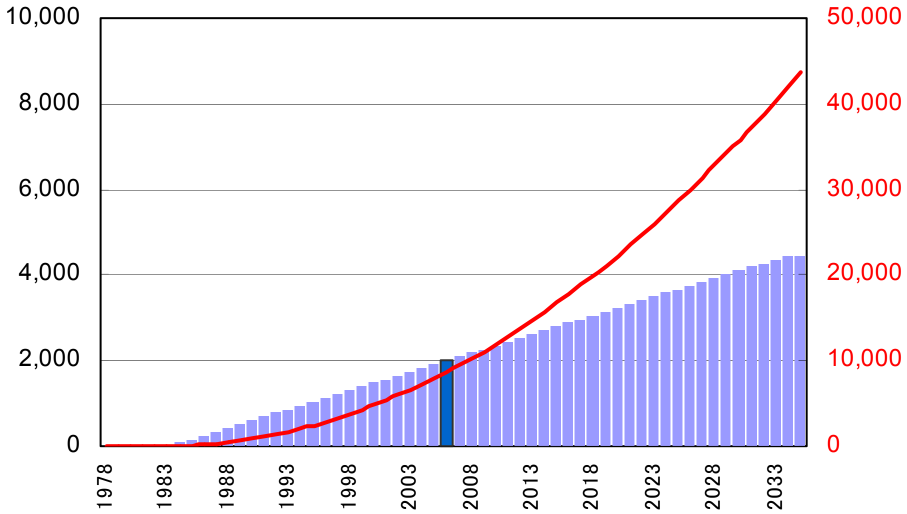


Figure 10.2.40. Growth and Cumulated Traffic Volume for 5 axles Truck

Then, the year Yf, when the cumulated traffic number reaches the cumulate vehicle number occurs the damage of fatigue Nf, is calculated as following equation.

$$Yf = (2 \times Nf \quad / (A \times 365))^{1/2}$$

Therefore, the year when the member will be damaged by fatigue are astimated as in Table 10.2.22.

Table 10.2.22. Cumulated Vehicle Number Occurs Damage

Part	Number of Heavy Vehicles per Day at present Vp (No.)	Years after completion Yp (year)	Cumulated Vehicle Number Occurs Damages Nf (No.)	Year when damage will be Occurred	
				Years after completion Yf (year)	Year
Sole Plate	1,996	28	35 million	52	2030
Sway-Bracing 1	1,996	28	107 million	91	2069
Sway-Bracing 2	1,996	28	758 million	241	2219
Lateral	1,996	28	52 million	63	2041

As the results of above, all the member measured on the test are not damaged by fatigue yet. The minimum life expectancye is estimated as 24 years for the welding part of sole plate, the other parts do not have a problems for fatigue.

4) Conclusion

As the results of the analysis based on the load tests on the No.26 Chirripo Bridge, the existing condition of the bridge main girder can be considered as followings;

1) The actual stiffness of the bridge is lager than the stiffness, which can be calculated for the design from the existing drawings.

2) The maximum deflection and stresses of main girder satisfy the allowable ones for the increased design live load of HS20 +25%.

3) The life expectancy for the fatigue is estimated 25 years from now at the connection point around the sole plate of bearing support. And the other connection parts such as sway bracing and lateral have more life expectancy

10.3 Natural Condition Survey

The river condition survey for 29 bridges and the geological survey for the selected 10 bridges were carried out as the natural condition survey in this Study.

10.3.1 River Condition Survey

The river flow condition, erosion and deposit conditions of riverbank and riverbed were investigated for 29 Study bridges. The conditions of riverbed or riverbank have been changed largely in some of the bridge site from the conditions of those when the bridges were constructed, due to the floods during long period after construction. In this section, river conditions for the selected 10 bridges are descried briefly in Table 10.3.1.

The scoring of riverbed at No.16 Nuevo River Bridge is most serious. The level of riverbed is scored around 6 m in depth from its original level, and it is not only the area around bridge but also the area in both upper and down stream sides. The piles of both P1 and P2 are exposed by scoring and the slope protection of embankment in front of A2 abutment is eroded deeply.

The erosions of backfill soil behind abutment are observed at many bridges, where, the pier type abutments are used. These erosions were occurred by the erosion of the slope protection in front of the abutment, which caused by floods or drain of rainwater around abutment. These erosions of backfill cause the settlements of road surface behind the abutment.

Figure 10.3.1. Scoring of Riverbed on No.16 Nuevo Bridge

Figure 10.3.2. Erosion of Backfill on No.12 Puerto Nuevo Bridge

The flood flow velocity is very fast in the most of rivers, because the river profiles are very steep as shown in Table 10.3.1. At the bridges, such as No.17 Chirripo Bridge and No.20 Sucio Bridge, located on steep mountain area, large size of rocks are carried by flood and cover the riverbed as a deposit. These large rocks hit the piers and damaged them.

Figure 10.3.3. Large Rock Deposit on No.17 Chirripo River Bridge

Figure 10.3.4. Large Rock Deposit on No.20 Sucio River Bridge

Table 10.3.1. River Conditions of 10 Bridges

Bridge No. & Name	River Profile near Bridge (Approx. %)	Conditions of River Flow Riverbank and Riverbed
No.2 Aranjuez	1.0	- Embankment Slope around A1 abutment is partly eroded. - Riverbed around piers is not scoured deeply. - Slope in front of A2 rises steeply covered with trees
No.3 Abangares	1.6	- Slopes of both A1 and A2 abutment are not eroded. - Riverbed near A2 side is deposited with sand and gravels, and driftwoods are entangled in pier. - No scoring around pier is found.
No.7 Azufrado	1.0	- Bridge located on an exposed rock layer - No scoring and erosion are found. - Rock riverbed is dud deeply with narrow width at center of river flow.
No.12 Puerto Nuevo	0.3	- Slopes in front of both A1 and A2 abutment are steep, and embankment behind A1 pier abutment is eroded. - Riverbed around piers is scoured, especially around P4 is scoured deeply.
No.16 Nuevo	1.2	- Riverbed is scoured deeply whole width of river flow, not only in the area around bridge site but also wide area both upper and down stream sides. - Riverbed comes down about 6m from its original revel at bridge site. - Piles of both piers are exposed by scoring. - Slope protection works in front of both abutments are broken by erosion.
No.17 Chirripo	1.5	- Slope protection of A2 abutment is eroded. - Riverbed is scoured with whole width. - Ordinary river flow area has been changed by the influence of quarries located at both up and down stream direction.
No.19 Sarapiqui	0.5	- Slope protection of A2 abutment is eroded. - Riverbed around P2 pier is scoured.
No.20 Sucio	2.4	- Riverbed is cover with huge rocks with more than 2 m diameter brought by flood. Damages caused by these rocks are found on pier. - Slope protection of A2 abutment is eroded by rain flow.
No.26 Chirripo	0.3	- Riverbed is scoured with whole width, especially around P6 pier. - A2 abutment was broken by the earthquake.
No.29 Torres	2.1	- Riverbed around P1 pier is eroded. - Ground under bridge between P1 to A2 is covered by illegal abandonment of industrial wastes

10.3.2 Geological Survey

Geological survey was carried out to understand the ground conditions, which are required for the check the structural safety of existing foundations. There were boring log data in the general drawings for some of selected 10 bridges. The boring points and numbers for each bridge were planed based on this data. Geological survey including mechanical drillings and laboratory tests for selected 10 bridges was divided into three groups to reduced the survey work time and was carried out by three local survey companies as listed in Table 10.3.2.

Table 10.3.2. Bridge Sites and Geological Survey Companies

Group	Bridges No. & Name	Number of Boring Holes	Survey Company
1	No.2 Aranjuez No.3 Abangares No.7 Azufrado No.20 Sucio	2 3 2 2	IMNSA
2	No.12 Puerto Nuevo No.16 Nuevo No.29 Torres	2 2 2	Vieto & Asociados, S.A.
3	No.17 Chirripo No.19 Sarapiqui No.26 Chirripo	2 2 2	INGEOTEC, S.A.

A total of 21 perforations were made with the purpose of determining the geological profile and evaluating the soil and rock existing in the 10 bridge sites. Rotary drilling equipment was used to make the boreholes and Standard Penetration Test (STP) was used combined with the standard sampler and shelving, to obtain the samples of soil. The method of double tube casing was used in order to get core samples. The undisturbed samples extracted from the field were analyzed in the laboratory and the characteristics of different layers, such as moisture content, unit weight, sieve analysis, specific gravity of soil, plasticity limits, liquid limits, and unconfined compression testing.

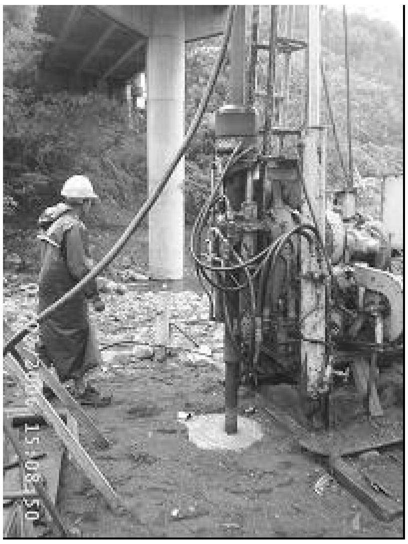


Figure 10.3.5. Drilling in Bridge Sites

Figure 10.3.6. Soil Samples of Boring by Geological Survey

The geological conditions of each bridge site, based on the results of geological surveys and the information on the as-built drawings of bridge, are as descried bellows.

1) No. 2 Aranjuez River Bridge Site

River deposits, such as igneous rock blocks, some blocks of limestone and sand are located on the base rock stratum, which is slightly weathered igneous rock with fractures. The thickness of the deposit layer is from 4 to 7 m at river path. The weathered rock layer is located higher place on the both side riverbanks. All spread type foundations are set on this weathered rock layer.

2) No. 3 Abangares River Bridge Site

Back fill materials of the A1 abutment lays on a clay silt layer with very compact to hard and with thickness of 5.5m. This clay silt layer lays on the sand and gravel layer with volcanic blocks and blocks of andesitic lava. This sand and gravel layer is considered the weathered igneous rock layer.

River deposits, such as sand and andesitic blocks with sizes up to 20cm, are located on the base weathered rock or volcanic alluvial layer. The thickness of the deposit layer is about 4 m at river path and 7m at the boring point in front of A2 abutment. All spread foundations are located on the weathered rock layers or volcanic alluvial layer.

3) No. 7 Azufrado River Bridge Site

Tufa exposes at riverbed at both pier locations. Poorly graded sand with from medium dense to dense covers the tufa layer at both abutment positions. The spread foundations of both rigid frame pier are set on the tufa layer. However, both abutments are not constructed as a structure supporting the force from superstructure. These abutments are set to stop a slide of backfill material.

4) No. 12 Puerto Nuevo River Bridge Site

Deposit of alluvium materials consists of calcareous blocks and igneous blocks with sand, covers weathered igneous rock gabbro. The river path alluvium material, where P2 is located, the condition of this material is good for bearing capacity at the layer deeper than 15 m.

At the edge of the river where the A2 located, there are layers of altered and fracturated block of calcareous rock and igneous rock that can be considered very stable. All spread foundations of piers and abutments are set on the weathered igneous rock stratum.

5) No. 16 Nuevo River Bridge Site

The riverbed is scored more than 4 m in depth from the original level and the piles of both piers are exposed by scouring. The friction capacity of piles might be reduced from the original design capacity.

The embankment of A2 abutment is filled about 10 m in height from the original ground level. The conditions of alluvium material starting from 13 m in depth from the surface of embankment (3 m from the original ground level) has enough capacity as a bearing stratum with N-values more than 56.

At the edge the river, where P1 pier is located, the N-values of STP of layers until the depth of 8m and in the depth between 18 m and 21m, are lower than 20. The layers of alluvium and clay with very dense is found at 22 m depth. All piles of foundation are driven in to the depth of more than 10 m from the original ground level.

6) No. 17 Chirripo River Bridge Site

At the both of boring points of A1 side and A2 side, the top layer from surface to 1m in depth consists of silty sand with N-value of 13 or rebound. The river deposit with blocks of andesitic lava and very coarse gravels continues to 15m in depth below the top layer at A1 side and to 12 m in depth at A2 side. All values of STP are not counted because of the hummer rebounded in this layer.

Inside of river flow area, the riverbed is covered with same river deposit with large size blocks of andesitic lava and very coarse gravels. All spread foundations of piers and abutments are set in this deposit layer at the depth between 11 m and 15 m from the ground.

7) No. 19 Sarapiqui River Bridge Site

At the boring point of A1 abutment, a gravel backfill layer, which consists fractured andesitic lava blocks and fragments, continues from the ground surface to the depth of 6 m. The medium plasticity clayed loam lies with the thickness of 9.4 m under the backfill layer. This clay layer is very soft with the N-value of 0. The sand and gravel layer with the N-value of around 40 continues with thickness of 4.6m. Under the above layer, a with N-value comes down again to 0 in the soft clay layer with the thickness of 3.9 m. A rigid clay layer with N-value of 40 continues for 1 m thickness. Finally, the andesitic basaltic rocks with completed altered very vesicular lavas is found at the depth of 25 m from the ground surface. The existing piles with 22.5 m long of A1 abutment are driven into this layer.

At the boring point in the river flow area near the P2 pier, a low plasticity clayed loam with the N-value of between 2 and 22 for the thickness of 2 m covers the river deposits of coarse sand layer. The sand layer continues in the depth from 2 m to 11 m with the N-values between 2 and 35. The N-values at the layer deeper than 11.5m, are 65 or more. The H-shape steel piles with 12.5m long are driven into this sand layer at P2 pier.

8) No. 20 Sucio River Bridge Site

Along the whole studied depth of 18m boring hole near P1 pier, andesitic lavas are encountered. The deposits consist of compact andesitic lava and fragments of lava including large size rock blocks with more than 1m diameters. The spread foundation of P1 pier is set in this layer at 15 m below riverbed.

At the boring hole near P2 pier located on the left bank, only a gravel layer exists along the whole depth of 10m. It consists of andesitic-dacitic lava fragments. The spread foundation of P2 pier is set in this layer at 6 m below ground surface.

9) No. 26 Chirripo River Bridge Site

At boring hole near the P1 pier, silty sand layer with thickness of 1.5m coves river deposit. This silty clay is very loose density with N-values between 3 and 4. The layer from 1.5 m up to 8 m in depth consists of fine gravel with 7 cm of maximum size of boulders. Rebounds of the STP are measured along this layer. This layer is susceptible to liquefaction during a earthquake. Underlying this material, the same fine sand with loose relative density (N-values are between 3 and 10) was detected. Finally between 35.0 m and the bottom of the boring, soft clay of high plasticity was detected. The unconfined compression stresses of this clay are between 0.30 and 0.40 kg/cm^2 and N-values of it are between 7 and 12.

In B26-2 boring near P7 pier, the soil profile is basically gravel material in all the bored depth. The layer from ground surface to 22 m in depth consists of alluvial andesitic lava blocks and fragments with maximum size of 13 cm of boulders. Rebounds of the STP are measured along this layer. The layer from 22 m up to 35 m in depth consists of very fine sandy gravel with 5 cm of maximum size of boulders. STP could not be measured either along this layer because of rebound. Due to the bearing stratum is so deep, all substructures of this bridge are supported by H-shaped steel piles with length of 30 m.

10) No. 29 Torres River Bridge Site

The original ground is covered with refilled material including concrete blocks. The consistency of this layer is variable between soft and very stiff. Under the foundation level, where the P1 is located, it is found a mix of gravel, boulders and a matrix of silty sand than can be susceptible to erosion. At the boring point in front of A2 abutment, refilled material is found until 4 m in depth same as P1. From 5 m in depth, a matrix of shale clay with altered igneous rock continues with N-values between 19 and 41.

All spread foundations of piers and abutments are set on the clayly sand layer at the depth between 3m and 8m from original ground level.

10.4 Selection of Rehabilitation, Reinforcement and Improvement Method

10.4.1 General

Appropriate countermeasure for rehabilitation, reinforcement and improvement should be planed for the bridges which are assessed or judged to have the risk of performance degradation at the present moment and in future by the following inspection results;

- One or more of the performances such as durability, safety, serviceability, hazard ability to third party and aesthetic/landscape have been observed at an unacceptable level.
- Although no deterioration is observed at the present, it is predicted that the bridge will have the risk of degradation of performance in the period of remaining service period.
- When a progress rate of performance deterioration is more rapid than initial predicted progress rate in the near future.

It is necessary to consider the maintenance category and comprehensively investigation about remaining service period, life cycle cost, available budget, social impact of the bridge and difficulty of maintenance.

An appropriate remedial measure should be planed in consideration of the deterioration mechanism and degree of deterioration. When many methods and materials will be able to apply to the selected remedial measure, it is important to select most suitable method and material for the deterioration mechanism and the degree of deterioration. Moreover it should be pay attention that the remedial method is different from each deterioration mechanism even if degree of deterioration is same.

10.4.2 Types of Remedial Measure

Types of remedial measure of bridge are classified seven (7) types, as shown below and remedial measure should be selected from these seven types.

1) Intensification of Inspection

Intensification of inspection means strengthened inspection by increase in the frequency of inspection and/or the number of inspection item. The frequency and number of item of inspection should be determined from the result of assessment/judgment and the remaining service period.

2) Repair

The repair plan should be made on the basis of the target level of performance of bridge. The method and materials for repair work should be selected in consideration of the deterioration mechanism and maintenance after repair.

The purposes of repair are rehabilitation/improvement of durability and exclusion from the risk which hazards to third party in the near future. It is also important that the repair work should eliminate the deterioration factors. However, when it is difficult to eliminate the deterioration factors by the repair works, the countermeasure to control the progress of deterioration should be taken.

The basic policy of repair plan should be made by the selection of the appropriate method for deterioration mechanism, establishing the required repair level, determination of the repair policy, specifications for material and sectional dimensions and execution method. Table 10.4.1 summarizes the example of repair policies, compositions of repair works and factor to be considered for the repair level.

Table 10.4.1. Deterioration Mechanisms and Repair Plan

Deterioration Mechanism	Repair Policies	Composition of repair works	Factors to be considered for the repair level
Carbonation	- Removal of carbonated concrete - Infiltration restraint of CO_2 and water after repair	- Restoration of carbonated part - Surface protection - Re-alkalization	- Removal depth of carbonated concrete - Method of corrosion protection for reinforcement - Material of patching - Material of surface protection - Thickness of protection - Alkalinity of concrete
Salt Corrosion	- Removal of Cl^- penetrated concrete - Protection for penetration of Cl-, water and oxygen after repair	- Restoration of carbonated part - Surface protection - Desalination	- Removal depth of concrete penetrated by Cl^- - Method of corrosion protection for reinforcement - Material of patching - Material of surface protection - Thickness of protection - Reduction volume of Cl^-
Chemical attack	- Removal of deteriorated concrete - Infiltration restraint of deterioration inducing chemicals	- Restoration of deteriorated part - Surface protection	- Material of patching - Material of surface protection - Thickness of protection - Removal depth of deteriorated concrete
Alkali aggregate reaction	- Restraint of water supply - Release of internal water - Restraint of alkali supply - Restraint of cracks progress (mostly strengthening is required)	- Injection into crack - Surface protection	- Material of surface protection - Thickness of protection
Corrosion of steel member	- Restoration of corrosion part - Re-painting		- Corrosion protection of steel member
Scouring of Foundation	- Restoration of place of scouring - Protection around foundation	Protection by: -Large stones -Mat gabions -Concrete block	- Size of material and method

Repair works are composed of following works;

- Repair of damage such as crack or peeling in concrete structure and rusting or peeling of paint in steel structure.
- Repair of deteriorated part by chloride ion or carbonation etc.
- Surface coating to prevent penetration of hazardous substances.

3) Reinforcement

Reinforcement is carried out as the measure to restore or improve mechanical performances of bridge, such as load carrying capacity and stiffness of member. When the results of the assessment/judgment require to reinforce the bridge, , it is important to investigate its shape, dimensions, bar arrangement and stress conditions of the member using design drawings and specifications, and is also important to measure the actual shape, dimensions, bar arrangement and transformation in site.

After the appropriate level of reinforcement is determined in consideration of bridge importance and remaining service period, the basic policy of reinforcement plan should be made. And reinforcement method should be selected to satisfy the appropriate reinforcement level in consideration of structural condition, execution condition, durability and difficulty of maintenance after reinforcement of bridge. Moreover it should be made on the basis of the result of inspection and assessment/judgment.

The reinforcement level is a degree of restoration or improvement of mechanical performances, such as load carrying capacity and stiffness of member. This should be determined in consideration of the following:

- The result of assessment/judgment
- Deterioration mechanism(s)
- Characteristics of the structure of bridge
- Importance of the bridge
- Loading conditions
- Ease of maintenance
- Remaining service period

Major reinforcement methods are classified as shown in Table 10.4.2. When reinforcement method is selected it is necessary to consider effect on reinforcement, execution condition, cost and influence on the community/environment during execution. It is noted that ease of maintenance after reinforcement and influence on the surrounding environment should be also taken into consideration.

Table 10.4.2. Major Reinforcement Methods

Reinforcement	Exchange members	- Slab reconstruction - Exchange steel member - Exchange bearing - Exchange expansion joint
	Increase in section	- Increase thickness - Concrete jacketing
	Addition of members and/or supports	- Girder Addition - Support Addition
	Addition of reinforcement	- Steel plate bonding - FRP bonding - Steel plate jacketing - FRP jacketing - Steel plate addition to steel member
	Reinforced by prestressing	- Out cable method
	Improvement of aseismicity	- Widen the bearing base to keep Seat Length (SE) - Install Prevention System for bridge falling down
	Reconstruction or reinforce	- Increased pile and extension of footing - Reconstruction new foundation

4) Appearance Improvement

Appearance improvement means to improve landscape by, for example, coating the structure or placing concrete overlay. An appearance improvement plan and execution plan shall be made in consideration of the landscape of the area, remaining service of period of the bridge and maintenance after improvement work.

5) Improvement in Function of Bridge

Improvement in function of bridge means that the function of bridge will be improved than original condition such as increase lane or additional foot walk. Before starting improvement, target level and an execution plan should be decided.

6) Restriction of Traffic Service

Restriction of traffic service means impose on usage such as a limit on the traffic load or limit on speed of vehicle etc. The degree and method of the restriction imposed on the usage shall be determined from the result of the assessment, judgment and inspection.

7) Demolition/Removal

It should be well considered how to treat the environmental condition, safety, usage/disposal of member after demolition/removal of bridge.

10.4.3 Target level of Performance of Bridge

The target level of performance of bridge after improvement shall be determined, prior to making planes of the remedial measure for deterioration of bridge. The target level of performance of bridge is classified into three (3) levels as listed below

1) Medium level of performance between completion stage and present condition
2) Same performance at completion stage
3) Higher level than performance at completion stage

When remedial measure for deterioration of the bridge is planed, the target level of bridge performance should be determined as shown in 10.4.3

In case of serious deteriorations to hazard to third party in the near future, the appropriate remedial measure should be carried out immediately.

Table 10.4.3. Remedial Measure Classified by Target Performance Level

Performance	Target Degree of Measure		
	Performance between completion stage and present condition	Performance at completion stage	Higher than performance at completion stage
Durability and safety (Load carrying capacity, Seismic Resistance)	Repair	Rehabilitation	Reinforcement [1)]
Serviceability (Accessory)	Repair	Rehabilitation	Rehabilitation
Hazard for third party (Concrete fragment falling)	Repair	Repair	Repair
Exterior Appearance (Painting)	Repair	Rehabilitation	Improvement

Note; 1): This case require analytical support in general

In this study, the target level of performance for 10 bridges was determined as shown in Table 10.4.4. Most of target level of performance is higher than that at the stage of completion. Because, these 10 bridges were designed by old version of AASHTO, in which live load is less than latest version, and earthquake load is not considered.

However, if it were recognized that the deterioration affects the safety of third party at present or in near future, the appropriate remedial measure for the deterioration should be carried out immediately.

Table 10.4.4. Target level of Performance of Bridge

Member	Target Level
Deck Slab	To secure the enough load carrying capacity for latest live load (HS20-25%)
	To improve durability of slab asphalt pavement and to place waterproofing
Superstructure	To secure the enough load carrying capacity for HS20+25% in consideration of additional load of reinforced slab and asphalt pavement.
	To prevent fall down of girders, even in the time of earthquake.
Substructure	To secure the enough load carrying capacity for allowable capacity regulated by the seismic design standard.
Foundation	To secure the enough load carrying capacity for allowable capacity regulated by the seismic design standard.
Expansion joint	To install the un-drained type expansion joint

10.4.4 Evaluation of Rehabilitation, Reinforcement and Improvement Method

Rehabilitation, reinforcement and improvement methods that applicable for structural elements of 10 bridges are shown in Table 10.4.6. These methods are given rankings according to the three (3) grades evaluation system shown Table 10.4.5. The results of evaluation for each method are also shown in Table 10.4.6.

Table 10.4.5. Evaluation System for Select Method

Evaluation Item	Grade A	Grade B	Grade C
Cost	Lowest	Between A and C	Highest
Period of Execution	Shortest	Between A and C	Longest
Traffic Control	Not required traffic control	Required road close few hours or one side traffic control	Required road close more than one day
Ratio of Increase of Dead Load	No	Less than 10 %	More than 10%
Situation of Material Market	Market is available in Costa Rica	No market in Costa Rica but it is easy to obtain	More difficult than B
Size of Execution Yard	Execution yard can be set up with in road width	Execution yard can be set up with in ROW	Required another space near construction site
Influences on Environment	No	Little change of landform inside of ROW will be needed	Change of landform outside of ROW or noxious material will be required
Difficulty of Maintenance after Reinforcement	Present inspection method will be available	Present inspection method will be available but inspector required more detailed observation	Special technique or instrument will be required
Executive Condition	Equipment and experience are available in Costa Rica	Equipment is available but technology assistance will be required.	Equipment and experience is not available in Costa

Table 10.4.6. Evaluation Results for Each Method

	Repair / Reinforcement Method	Cost	Work Period	Traffic Control	Increase of Weight	Preparations of Material	Execution Yard	Influence on Environment	Maintenance after Reinforcement	Degree of Difficulty of Execution
Deck Slab	Injection and Filling	B	B	A	A	B	A	A	A	B
	Patching or Restore Dimension of Member	C	C	A	A	B	A	A	A	B
	Coating of Surface	A	A	A	A	A	A	A	A	B
	Reinforced by Bonded FRP Sheet	C	A	A	A	C	A	A	C	B
	Increased Thickness of Slab (Placing concrete on existing	A	B	B	B	A	A	A	A	B
	Increased Thickness of Slab (Placing concrete under existing	B	B	A	B	B	A	A	B	B
	Build Additional Stringer	B	B	A	B	A	A	A	A	A
	Reinforced by Bonded Steel Plate	B	A	A	B	B	A	A	C	B
	Replacement of Deck Slab	C	C	B	B	A	A	A	A	A
Main Girder	Injection and Filling	B	B	A	A	B	A	A	A	B
	Restore Dimension of Member	C	C	A	A	B	A	A	A	B
	Coating of Concrete Surface	A	A	A	A	A	A	A	A	B
	Reinforced by Bonded FRP Sheet	C	A	A	A	C	A	A	C	B
	Increase of Girder Height	B	B	A	B	B	A	A	B	B
	Reinforced by Bonded Steel Plate	B	A	A	B	B	A	A	C	B
	Reinfoeced by Out Cable	C	B	A	B	C	A	A	A	B
	Install Additional Main Girder	B	A	A	B	B	A	A	C	B
	Reinforced by Additional Steel Plate	A	A	A	B	A	A	A	A	A
Substructure	Injection and Filling	B	B	A	A	B	A	A	A	B
	Restore Dimension of Member	C	C	A	A	B	A	A	A	B
	Coating of Concrete Surface	A	A	A	A	A	A	A	A	B
	Roll Up Steel Plate	B	B	A	A	A	A	B	B	B
	Roll Up Corbon Fiber Sheet	C	A	A	A	C	A	B	B	B
	Reinforced Concrete Jacketing	A	C	A	B	A	A	B	A	A
	Reinforced by Prestressing	B	B	A	B	C	A	A	A	B
Fandation	Increased Pile and Extension of Footing	B	B	A	B	A	A	B	A	A
	Increased Pile and Construction of Ramen Frame Pier	B	B	A	A	A	A	B	B	A
	Protected by Large Stones	A	A	A	-	A	A	B	A	A
	Protected by Mat Gabions	B	B	A	-	A	A	B	A	A
	Protected by Concrete Block	C	C	A	-	A	A	B	A	A

10.4.5 Evaluation of Rehabilitation, Reinforcement and Improvement Method for 10 Bridges

1) Selection of the method for Rehabilitation, Reinforcement and Improvement

In order to achieve the target level of rehabilitation, reinforcement and improvement in 10 bridges, the preliminary plan was carried out based on evaluation to of regarding cost, period of execution, executive condition, difficulty of maintenance after reinforcement, influence for traffic etc. The selected methods in 10 bridges are as shown in Table 10.4.7

Table 10.4.7. Summary of Proposed Methods for 10 Bridges

Member	Bridge No.	2	3	7	12	16	17	19	20	26	29
	Bridge name / Remedial method	Rio Aranjuez	Rio Abangares	Rio Azufrado	Rio Puerto Nuevo	Rio Nuevo	Rio Chirripo	Rio Sarapiqui	Rio Sucio	Rio Chirripo	Rio Torres
Slab	Replacement	Apply	Apply								
	Bonded FRP or Steel plate				Apply			Apply		Apply	
	Increse Thickness			Apply		Apply					
	Install additional stringer	Apply	Apply								
Superstructure	Additional Member	Apply									
	PC Cable				Apply			Apply			
	Bonded FRP or Steel plate		Apply		Apply	Apply		Apply			Apply
	Install Prevention System for Bridge Falling Down	Apply	Apply		Apply	Apply		Apply		Apply	Apply
Abutment	Widen the bearing base	Apply	Apply		Apply	Apply		Apply		Apply	Apply
	Slope protection				Apply	Apply	Apply				
Pier	Widen the bearing base	Apply	Apply		Apply	Apply		Apply		Apply	Apply
	Concrete Jacketing	Apply	Apply	Apply		Apply					Apply
	Protection for rolling stone						Apply		Apply		
Foundation	Expand Fooring	Apply	Apply	Apply	Apply	Apply	Apply	Apply		Apply	Apply
	Reconstruction										
	Install Additional Pile					Apply		Apply			
Scouring	Protection of stone or mat gabion	Apply	Apply		Apply	Apply				Apply	Apply

Main points for each basic idea for rehabilitation, reinforcement and improvement method is described below. And more details of method for each bridge are as shown in tables from 10.4.10. to 10.4.19.

a) Deck Slab

- Because the deck slabs of the bridges on route No.1 are severely damaged, the replacement method should be selected.

- The deck slabs of bridges on route No.2 are not damaged so much, however according to analysis result, its load carrying capacity is less than the capacity specified latest live load (HS20+25%) and in some of bridges on route No.2, the thickness of deck slab is not satisfy the minimum thickness (17cm) specified by AASHTO. Moreover, the load carrying capacity of girder also is less than the capacity required for latest live load

(HS20+25%). Therefore, the slab shall be reinforced by Bonded FRP Sheet method or Bonded Steel Plate method to avoid increase dead load.

- In case of the brides on route No.4 and 32, the bridges were designed to HS20 load, and their carrying capacities of deck slab satisfy the required capacity level in AASHTO. Therefore, intensification of inspection shall be required by increase in the frequency of inspection of deck slab. And when it is judged that the deterioration of slab will be progressing, the deck slab shall be reinforced by a suitable method such as Bonded FRP Sheet method, Bonded Steel Plate method and/or Injection method to avoid traffic control.

- In case of Rio Torres Bridge on Route 218, the new bridge was constructed for increased traffic volume next to the old bridge. The deck slab of the new bridge used for the traffic goes to San Jose has enough capacity for latest live load. However, due to heavy traffic volume, the slab has been damaged. So during reinforcing work of a bridge, the other bridge can be used as a detour. Therefore, the method, which increases thickness of slab by placing concrete on existing slab with additional reinforcement, can be applied for this bridge.

b) Girder (Truss, Steel I Bam, RC Girder, PC I Girder)

In case of the bridges designed according to original live load such as HS15 or HS20, the load carrying capacity of girder is less than the capacity required for latest live load (HS20+25%). Therefore, the appropriate reinforcement method shown in the bellow table shall be selected to avoid increasing dead load.

Table 10.4.8. Rehabilitation Methods for Main Girder

Type of Bridge		Method for Rehabilitation, Reinforcement and Improvement
Steel Bridge	Truss	To reinforce by additional member to decrease buckling length
	I Beam	To reinforce by additional member or by prestressing with Out Cable method
RC Girder Bridge		Steel plate bonding method or FRP bonding method
RC Rigid Frame Bridge		To increase girder depth to decrease bending moment of girder in Rio Azufrado Bridge (No.7).
PC I Beam Bridge		To reinforce by FRP bonding method

c) Substructure

All of 10 bridges have enough load carrying capacity for the latest live load (HS20+25%). However, according to analysis result in some piers, the stress of reinforcement for earthquake load was bigger than the allowable capacity stress required by seismic design standard as shown below table.

Table 10.4.9. Substructures Required for Reinforcement

Bridge Name (No.)	Pier No.	Type of Pier	Direction of Lading
Rio Aranjuez Bridge (No2)	Pier (No.2)	Wall	Longitudinal
Rio Nuevo Bridge (No.16)	Pier (No.1)	Rigid Frame	Longitudinal

Rio Torres Bridge (No.29)	Pier (No.1)	Column	Longitudinal and Transversal

Therefore the Concrete jacketing method shall be selected to reinforce these piers. In case of applying steel plate jacketing method or FRP jacketing method the river flow conditions including rolling stones must be made sure.

d) Foundation

All foundations of 10 bridges have enough load carrying capacity for the latest live load (HS20+25%).

However, according to analysis result for earthquake load, the foundations in 10 bridges are not stable against bearing, toppling and sliding components. And in case of pile foundations, resisting capacities of pile foundations in Rio Nuevo Bridge (No.16) and Rio Sarapiqui Bridge (No.19) are less than the force caused by earthquake load..

Moreover, in Rio Nuevo Bridge (No.16) and in Rio Torres Bridge (No.29), large scale scouring with depth of more than 3 m was observed. The piles of piers in Rio Nuevo Bridge have protruded from the ground about 2.5m. The countermeasure for scouring are required for both bridge.

For the reinforcement of foundations of Rio Nuevo Bridge (No16), increasing of pile number and expanding footing are required.

When it is judged that above both methods are not executed because of river flow condition or executive condition, the support system, which seismic momentum force of superstructure is not affected to substructure in accordance with the phenomenon that support will be broken by earthquake load, is applied.

Regarding countermeasure for scouring in the riverbed around pier, the protection works with large stone, mat gabion or concrete block shall be applied. Material and its size shall be selected in consideration of on river flow velocity, flood discharge and condition of riverbed.

2) Others Rapier Works

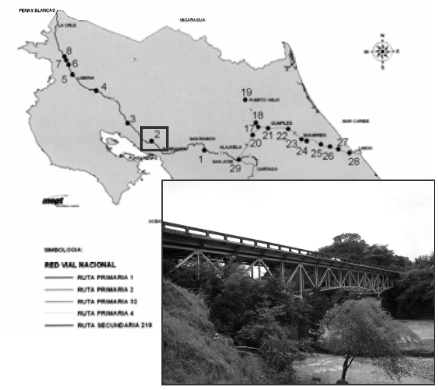
For increase durability of bridges, other repair works listed below shall be required.

i) Installation of new expansion joint with un-drain type in all of 10 bridges.
ii) Laying of asphalt pavement (5cm thickness) and water proofing on concrete slab deck for all of 10 bridges
iii) Repairing of slope protection in front of abutment
iv) Installation of Prevention System of bridge falling down and/or widening the bearing base to keep Seat Length (SE) for all of 10 bridges
v) Repainting of steel members in steel girder bridges
vi) Replacement of the bearing to new one.
vii) Repairing defectives of construction and cracks

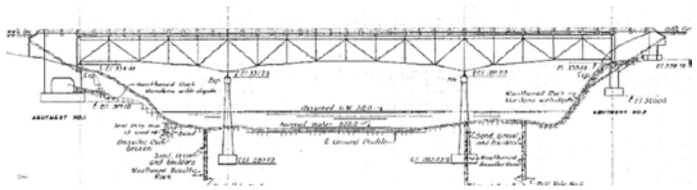
viii) Installation of protection around pier from stones carried by flood
ix) Installation of drainage pipes.

Table 10.4.10. Rehabilitation, Reinforcement and Improvement Plan for Rio Aranjuez Bridge (No.2)

Location



Side View



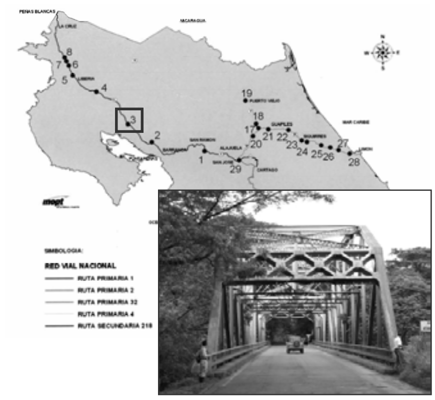
Design: 1950/11/17	**Completion: 1955**

Length: 87.78m, Design Live Load: H15-S12
Sup: Steel Truss
Sub: Wall Type Pier, Rigid Frame Abutment
Foundation: Spread Foundations

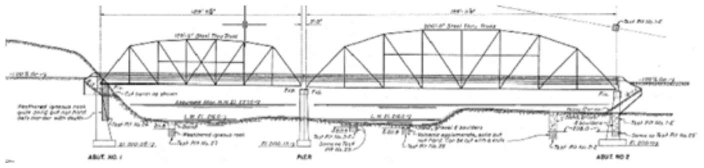
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint	Improve durability
Pavement	Laying asphalt pavement (5cm thickness) and water proofing on slab deck.	Improve durability
Main girder (Truss member)	Reinforcing with additional member to decrease buckling length of member	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Repainting of steel member	Restore to the condition completion stage
Deck Frame Deck Slab	Replacing of deck slab. Installation of additional stringer to decrease thickness of slab, if required.	Upgrade the load carrying capacity to new live load (Hs20+25%)
Abutment	Repairing of fissility and exposure of reinforcement (Remove loose concrete and corrosion of re-bar before placing concrete.)	Restore to the condition completion stage
	Widening the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Reinforcing by the concrete jacketing	Improve asismicity
	Widening the width of bearing base to keep Seat Length (S_E) for improve asismicity	
Foundation	Expanding of footing width	Improve asismicity
	Protection of riverbed by stone or mat gabion	Protection for scouring

Table 10.4.11. Rehabilitation, Reinforcement and Improvement Plan for Rio Abangares Bridge (No.3)

Location



Side View



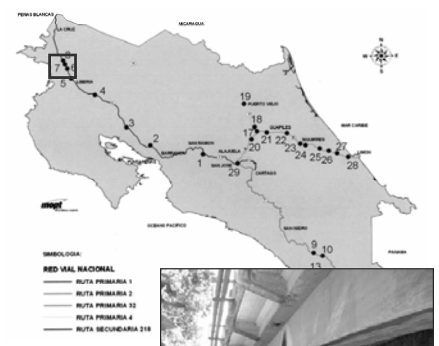
Design: 1952/03/07	**Completion: 1953**

Length: 101.34m, Design Live Load: H15-S12-44
Sup: Steel Thru Truss
Sub: Wall Type Pier, Rigid Frame Abutment
Foundation: Spread Foundations

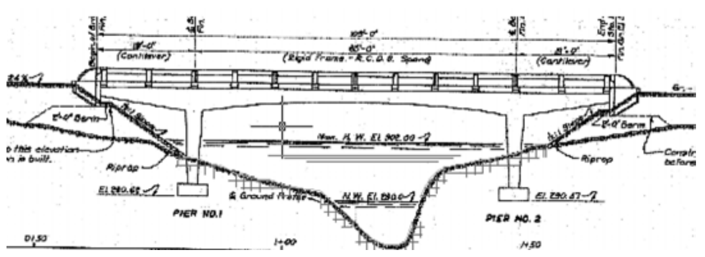
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipe r to avoid girder in wet condition by drained water.	Improve durability
Main girder (Truss member)	Reinforcement by additional member to decrease buckling length.	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Repainting of steel member	Restore to the condition completion stage
Deck Frame Deck Slab	Replacing of deck slab. And installation of additional stringer to decrease thickness of slab, if required.	Upgrade the load carrying capacity to new live load (Hs20+25%)
Abutment	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Foundation	Expanding of footing width	Improve asismicity

Table 10.4.12. Rehabilitation, Reinforcement and Improvement Plan for Rio Azufrado Bridge (No.7)

Location



Side View

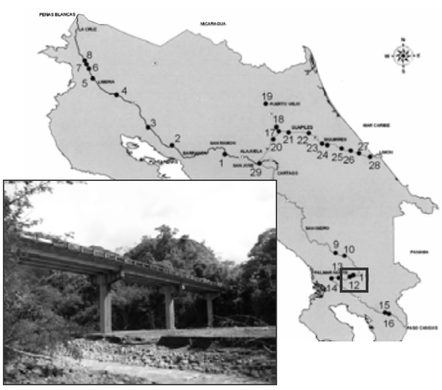
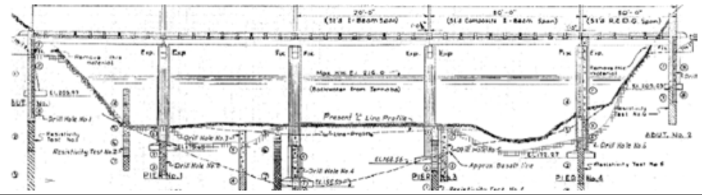


Design: 1953/09/08	**Completion: 1955**

Length: 31.39m, Design Live Load: H15-S12-44
Sup: Rigid Reinforced Concrete Frame
Sub: Rigid Frame
Foundation: Spread Foundations

Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Hand railing	Repair of damage and installation of new hand railing.	Restore to the condition completion stage.
Expansion joint	Installation of new joint.	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing on slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Deck Slab	Increasing thickness of slab using method of placing concrete on existing slab	Upgrade the load carrying capacity to new live load (Hs20+25%)
Concrete Girder	Installation of new abutment to support side span beam.	Upgrade the load carrying capacity to new live load (Hs20+25%)
	To increase frequency of inspection to observe progress of cracks.	Observe the progress of crack
Pier and Foundation	To execute Inspection periodically	Observe the deterioration

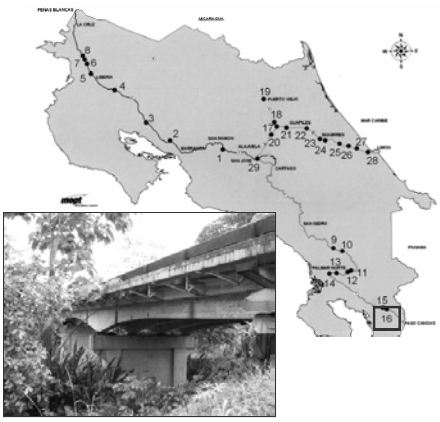
Table 10.4.13. Rehabilitation, Reinforcement and Improvement Plan for Rio Puerto Nuevo Bridge (No.12)

<table>
<tr><td rowspan="4">Location
</td><td colspan="2">Side View
</td></tr>
<tr><td>Design: 1958/01/10</td><td>Completion: 1961</td></tr>
<tr><td colspan="2">Length: 104.89m, Design Live Load: H15-S12-44
Sup: Steel Beam and Reinforced Concrete Girders
Sub: T Type Pier, Rigid Frame Abutment
Foundation: Spread Foundations</td></tr>
</table>

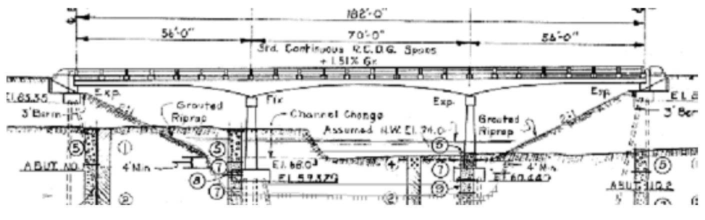
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thickness) and water proofing on slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (Steel I Beam)	Reinforcing by additional member or prestressing using Out cable method	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Installation of Unseating Prevention System	Improve asismicity
	Repainting of steel member	Restore to the condition completion stage
Main girder (RC Girder)	Reinforcing by Bonded FRP Sheet method or Bonded Steel Plate method.	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Installation of Unseating Prevention System	Improve asismicity
Deck Slab	Reinforcing by Bonded FRP Sheet method or Bonded Steel Plate method.	Upgrade the load carrying capacity to new live load (Hs20+25%)
Abutment	To repair slope protection in front of abutment	Restore to the condition completion stage
	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
	To increase the frequency of inspection to observe progress of cracks.	Observe the progress of crack
Foundation	Expanding of footing width	Improve asismicity

Table 10.4.14. Rehabilitation, Reinforcement and Improvement Plan for Rio Nuevo Bridge (No.16)

Location



Side View



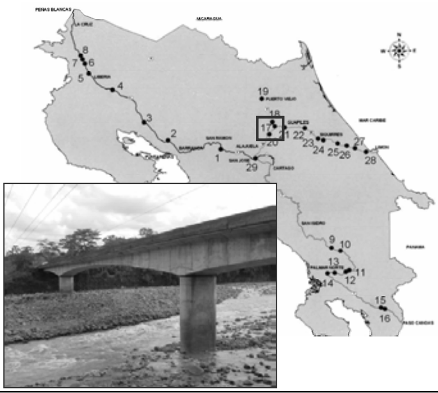
Design: 1957/12/19	**Completion: 1961**

Length: 55.47m, Design Live Load: H15-S12-44
Sup: Continuous Reinforced Concrete Girder
Sub: Rigid Frame Pier, Reversed T Type Abutment
Foundation: Pile Foundations

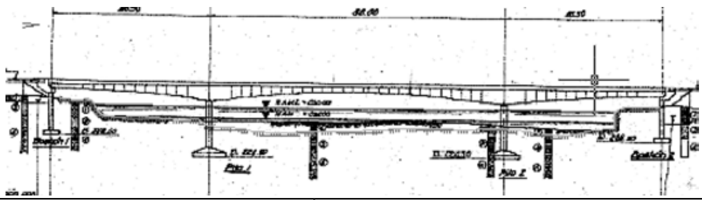
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint.	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing on slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (RC Girder)	Reinforcing by Bonded FRP Sheet method or Bonded Steel Plate method.	Upgrade the load carrying capacity to new live load (Hs20+25%)
Deck Slab	Reinforcing by Bonded FRP Sheet method or Bonded Steel Plate method.	Upgrade the load carrying capacity to new live load (Hs20+25%)
Abutment	To repair slope protection in front of abutment	Restore to the condition completion stage
	Widening the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Reinforcing by Concrete jacketing	Improve asismicity
	Widening the width of bearing base to keep Seat Length (S_E) for improve asismicity	
Foundation	Reconstruction of new foundation	Improve asismicity
	Protection of riverbed by mat gabion or concrete block	Protection for scouring

Table10.4.15. Rehabilitation, Reinforcement and Improvement Plan for Rio Chirripo Bridge (No.17)

Location



Side View



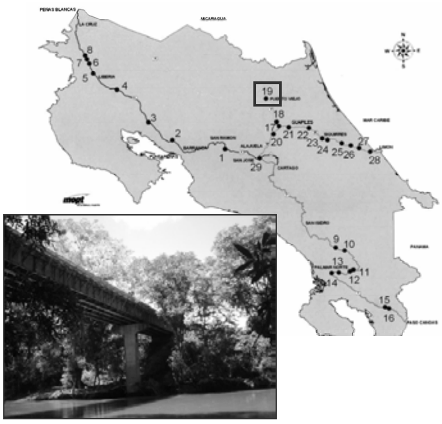
Design: 1975/03 | **Completion: 1978**

Length: 175.8m, Design Live Load: HS20-44
Sup: Concrete Box Girder
Sub: Wall Type Pier, Rigid Frame Abutment
Foundation: Spread Foundations

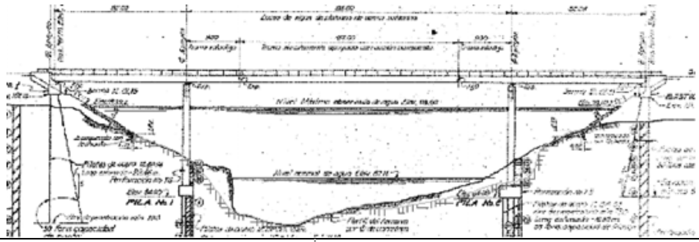
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thickness) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (PC BOX Girder)	Repairing of defective caused by construction	Restore to the condition completion stage
	To increase frequency of inspection to observe deformation and progress of deterioration.	Observe the deformation and deterioration.
Abutment	Repairing of slope protection in front of abutment	Restore to the condition completion stage
	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Repairing of fissility and exposure of reinforcement.	Restore to the condition completion stage
	Installation of protection around piers from stones carried by flood.	Improve durability
Foundation	Protected by mat gabion or concrete block	Protection for scouring

Table 10.4.16. Rehabilitation, Reinforcement and Improvement Plan for Rio Sarapiqui Bridge (No.19)

Location



Side View



Design: 1970/12 | **Completion: 1978**

Length: 100.96m, Design Live Load: HS15-44

Sup: Steel I Beam

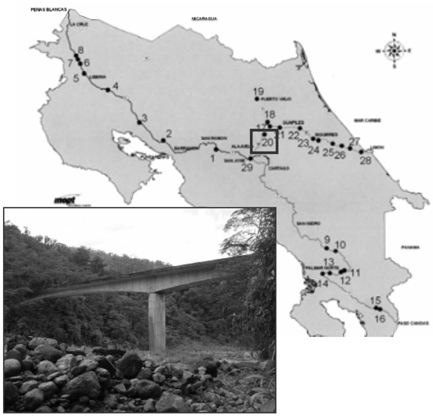
Sub: T Type Pier, Reversed T Type Abutment

Foundation: Pile Foundations

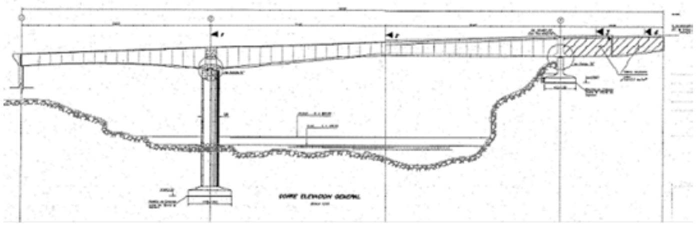
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new expansion joints at all place of joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (Steel I Beam)	Reinforcing by additional member	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Repair the part of shortage section in the girder and crossbeam by replacement of new steel plate.	Restore to the condition completion stage
	Installation of new supports to avoid the uplift of girders at both abutments.	Improve durability
	Installation of Unseating Prevention System at both abutments and at both Gerber hinges.	Improve asismicity
	Repainting of steel member	Restore to the condition completion stage
Deck Slab	Increasing of frequency of inspection to observe progress of cracks.	Observe the progress of crack
Abutment	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Foundation	Reinforcing by additional piles and expanding of footing	Improve asismicity
	To protect riverbed by mat gabion	Protection for scouring

Table 10.4.17. Rehabilitation, Reinforcement and Improvement Plan for Rio Sucio Bridge (No.20)

Location



Side View



Design: N.D. | **Completion: N.D.**

Length: 187.25m, Design Live Load: Unknown

Sup: Concrete Box Girder

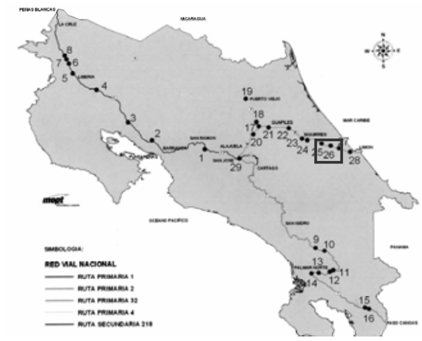
Sub: Circular Cylinder Pier, Rigid Frame Abutment

Foundation: Spread Foundations

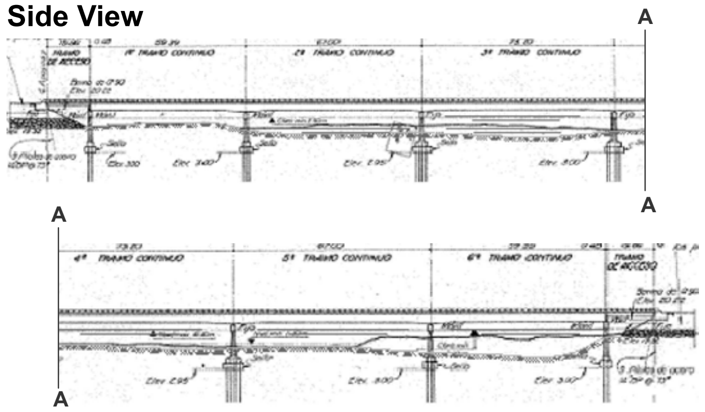
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (PC BOX Girder)	Repairing of defective of construction	Restore to the condition completion stage
	Increasing of frequency of inspection to observe deformation and progress of deterioration.	Observe the deformation and deterioration.
Abutment	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Repairing of fissility	Restore to the condition completion stage
	Installation of protection around piers from stones carried by flood.	Improve durability
Foundation	To protect riverbed by mat gabion or concrete block	Protection for scouring

Table 10.4.18. Rehabilitation, Reinforcement and Improvement Plan for Rio Chirripo Bridge (No.26)

Location



Side View



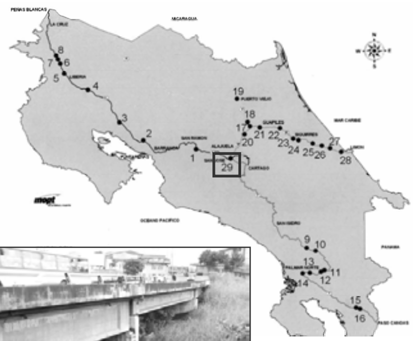
Design: 1969/07	**Completion: 1974-1978**

Length: 431.9m, Design Live Load: Unknown
Sup: Continuous Steel I Beam
Sub: Wall Type Pier, Reversed T Type Abutment
Foundation: Pile Foundations

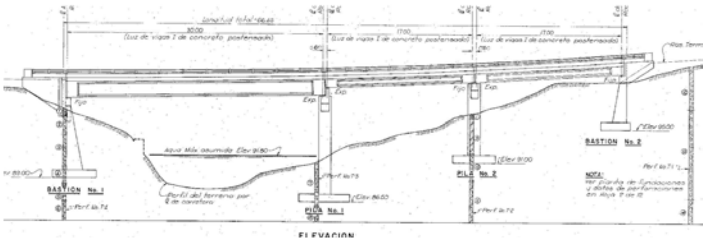
Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint at all place of joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (Steel I Beam)	Reinforcing by additional member	Upgrade the load carrying capacity to new live load (Hs20+25%)
	Return girders to original position.	Restore to the condition completion stage
	Installation of new support to avoid girders to lift up at both abutments.	
	Installation of Unseating Prevention System	Improve asismicity
	Repainting of steel member	Restore to the condition completion stage
Deck Slab	Increasing of frequency of inspection to observe progress of cracks.	Observe the progress of crack
Abutment	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Widening of the width of bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Foundation	Reinforcing by additional piles and expanding of footing	Improve asismicity
	To protect riverbed by mat gabion	Protection for scouring

Table 10.4.19. Rehabilitation, Reinforcement and Improvement Plan for Rio Torres Bridge (No.29)

Location



Side View



Design: 1979/05/ | **Completion: N.D.**

Length: 66.46m, Design Live Load: HS20-44
Sup: Concrete I Pos-tensioned Girder
Sub: T Type Pier, Rigid Frame Abutment
Foundation: Spread Foundations

Place or member	Plan for Rehabilitation, Reinforcement and Improvement	Purpose of Repair
Expansion joint	Installation of new joint at all place of joint	Improve durability
Pavement	Laying of asphalt pavement (5cm thick) and water proofing for slab.	Improve durability
Drainage	Installation of drainage pipes to avoid girder in wet condition by drained water.	Improve durability
Main girder (PC I Beam)	Reinforcing by FRP bonding method or Out cable method	Upgrade the load carrying capacity to new live load (Hs20+25%)
Deck Slab	Increasing of thickness of slab using method of placing concrete on existing slab	Upgrade the load carrying capacity to new live load (Hs20+25%)
Abutment	Widening of the width bearing base to keep Seat Length (S_E) for improve asismicity.	Improve asismicity
Pier	Reinforcing by concrete jacketing	Improve asismicity
	Widening the width bearing base to keep Seat Length (S_E) for improve asismicity	
	Repairing of Cracks by injection method	Improve durability
Foundation	Expanding of footing	Improve asismicity
	Protection of riverbed by stone or mat gabion	Protection for scouring