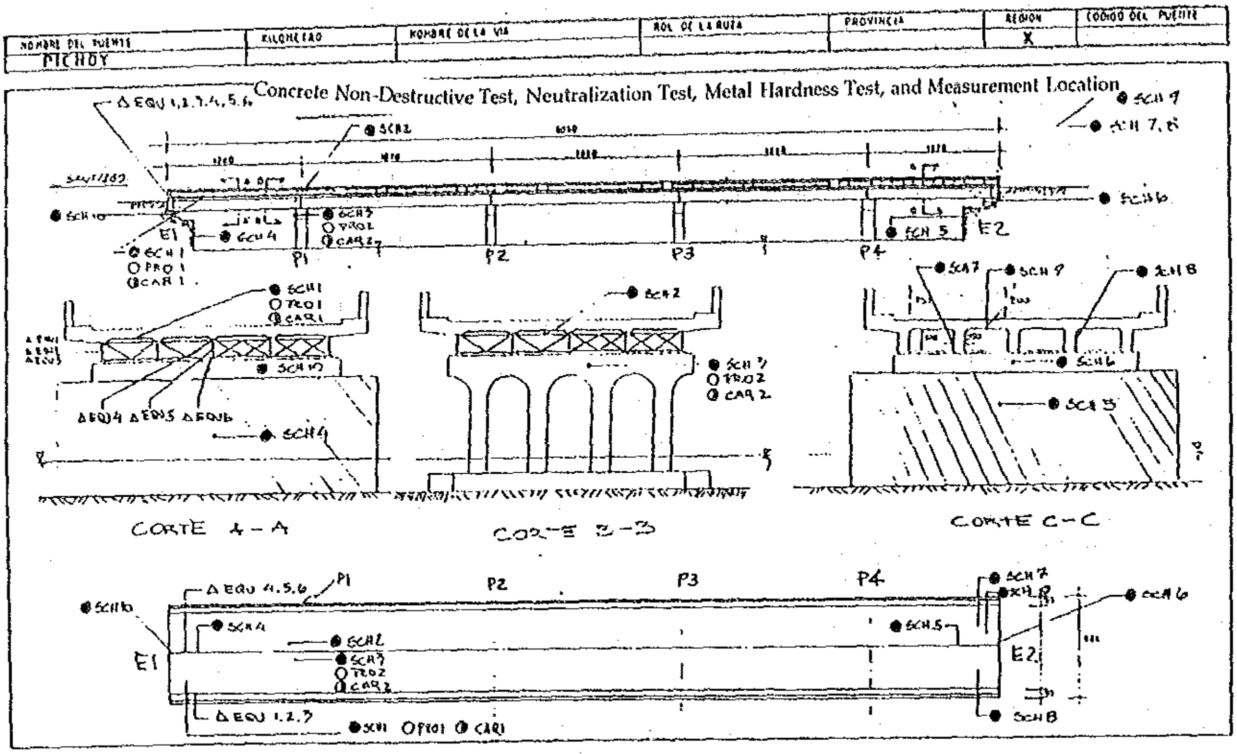


Figure 3-20 Pichoy Bridge Measurement Location

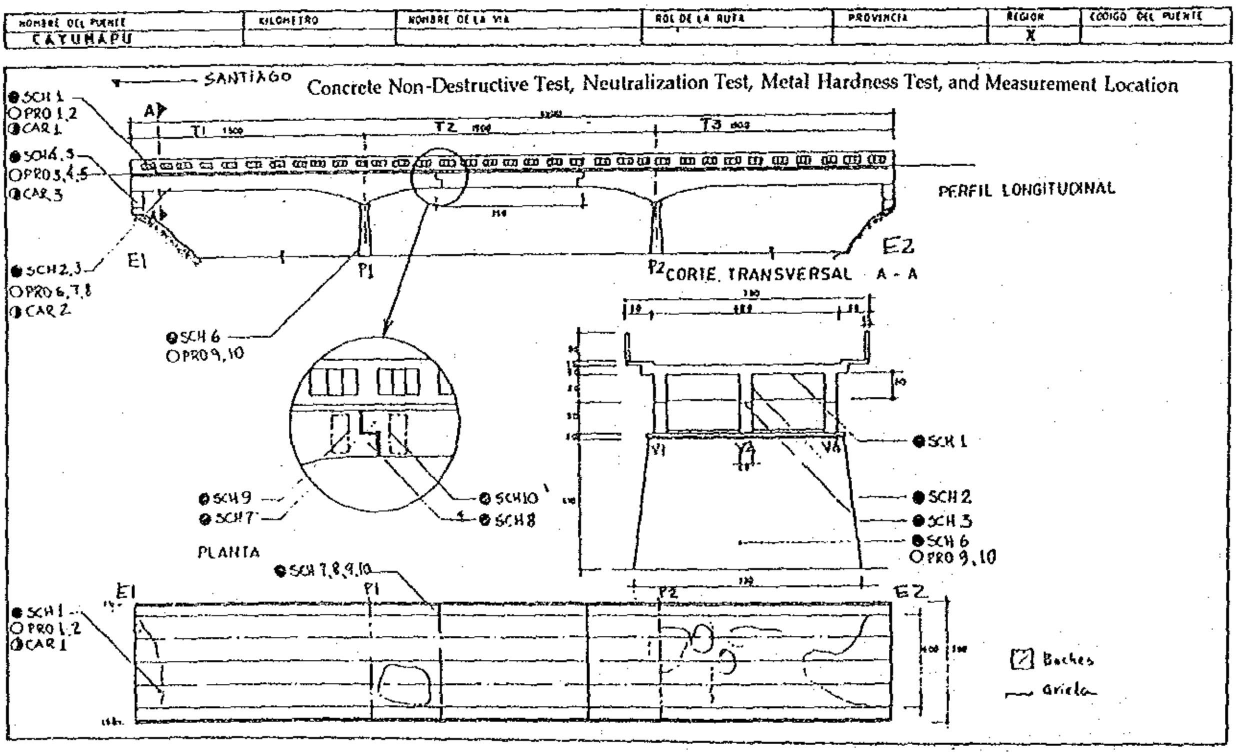


Figure 3-21 Cayumapu Bridge Measurement Location

3-1-4 Survey Results and Examination

(1) Instrumentation Measurements

1) Measurement Results

a. Concrete Strength

The concrete strength of ten bridges, as measured by the Schimdt Hammer Test, has a maximum average of 330 kg/cm2 and a minimum average of 210 kg/cm2. Maipo and Loncomilla bridges, whose upper structures consist of pressed concrete girders, are beyond the maximum of 300 kg and their concrete strength was found to be higher compared with other bridges. The concrete strength at some locations was measured as low for the Bio-Bio, Lamadillas, Pichoy, and Cayumapu bridges; however, these strengths do not present an immediate problem in terms of structure failure. Table 3-9 shows the measurement results of the Schimdt Hammer Test. These plotted results are represented by Figure 3-22 and Figure 3-23.

b. Reinforcement Inspection

Reinforcement locations were confirmed by a profometer during nondestructive reinforcement inspections. For locations where neutralization testing was performed, the diameter of the reinforcement was confirmed by drilling into the concrete at locations where reinforcement is found. Even for the Loncomilla Bridge, which has the widest interval between reinforcement (the widest reinforcement interval being 260 cm), there are no problems concerning the reinforcement interval 1. The covering thickness of concrete varies from a maximum of 78 mm to a minimum of 3 mm: These values are within the range of location and construction error and do not present any particular problems. However, we could not confirm whether the reinforcement diameter is sufficient relative to stress as there were no design calculation results. The results are shown in Table 3-10.

c. Steel Material Quality

A hardness measurement test was performed by echo tip to confirm the characteristics of the steel used for upper bridge structures. Brinell hardness of steel (SS40) used for bridges in Japan is HB=140-150. The value for the Pullally Bridge (average HB=125) and the Pichoy Bridge (average HB=128) was slightly lower in this survey.

d. Concrete Neutralization

Neutralization testing was performed at the same time as confirmation of reinforcement locations by profometer. This is performed because concrete neutralization is related to the progress of reinforcement bar rust. Concrete shows strong alkalinity (PH12) when first poured, however, concrete becomes neutralized and its alkalinity weakens with the passage of time. Reinforcement bar corrodes rapidly within concrete of less than PH9. For the Claro, Bio-Bio, Ramadillas, Pichoy, and Cayumapu bridges, the majority of locations are less than PH9 and the depth of neutralization is greater than 20 mm. This illustrates that neutralization is fairly advanced. Table 3-12 shows the measurement results.

2) Examination

The objective of this survey is to analyze the characteristics of the bridge materials. As a result of this survey, the strength of some bridge materials indicate a slight problem. However, these problems are not directly related to the safety of the bridges that were examined. Concrete neutralization for several bridges is intense, as Table 3-8 shows, due to the fact that the survey targeted bridges are old. This means that the working life of these bridges is over and that repairment measures alone will not be sufficient. It is desirable in the near future to replace the bridges which have problems as indicated by concrete neutralization testing.

Table 3-8 Survey Results by Measuring Instrumentation

Bridge Name	Concrete Strength	Reinforcement Layout	Steel Strength	Neutralization
AMOLANAS			-	Δ
PULLALLY			Δ	Δ
MAIPO			-	
CLARO			-	©
LONCOMILLA			-	Δ
BIO-BIO	Δ			©
RAMADILLAS	Δ			©
MALLECO				
PICHOY	Δ		Δ	©
CAYUMAPU	Δ		-	©

Note)
© : shows problem
Δ : shows slight problem
Blank : no problem
- : test was not performed.

Table 3-9 Schmidt Concrete Hammer Measurement Results

PUENTE	SCH-1	SCH-2	SCH-3	SCH-4	SCH-5	SCH-6	SCH-7	SCH-8	SCH-9	SCH-10
AMOLANAS	287	234	277	228	271	256	230	286	306	288
PULLALLY	261	253	244	301	300	295				
MAIPO	291	332	337	317	328	343	280	291		
CLARO	288	126	292	318						
LONCOMILLA	217	330	222	220	226	229	318	237		
BIO-BIO	262	274	249	263	262	271	215			
RAMADILLAS	261	261								
MALLECO	321	268	292	303	317	300	298	269	301	
PICHOY	274	288	258	237	205	305	317	311	247	298
CAYUMAPU	276	282	307	252	295	247	283	277	257	294

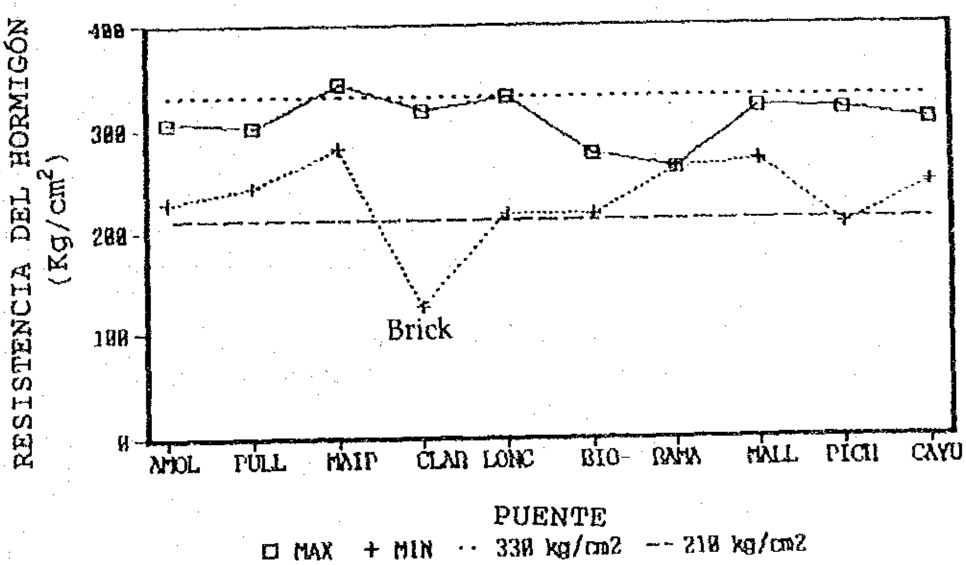
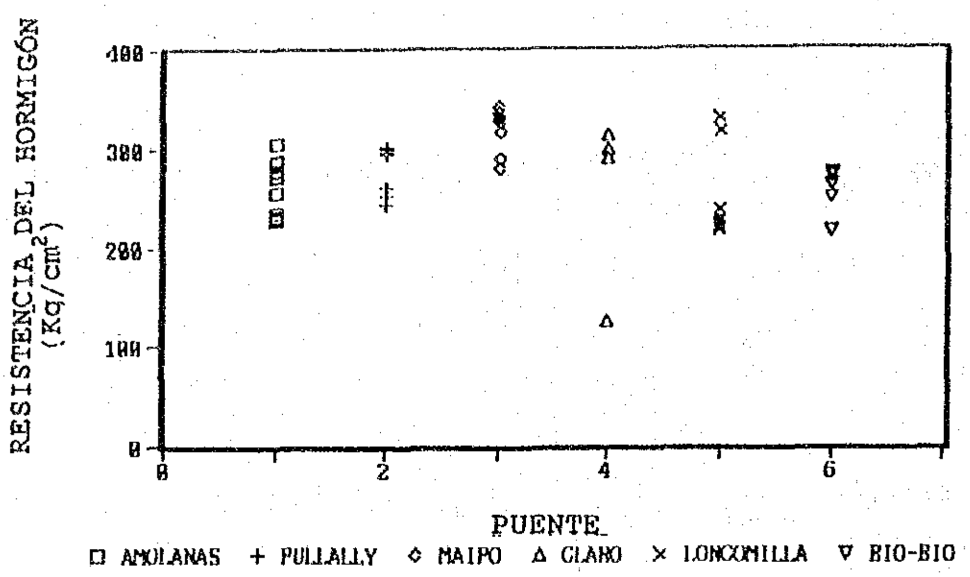


Figure 3-22 Maximum and Minimum Figures for Schmidt Concrete Hammer Measurement Results



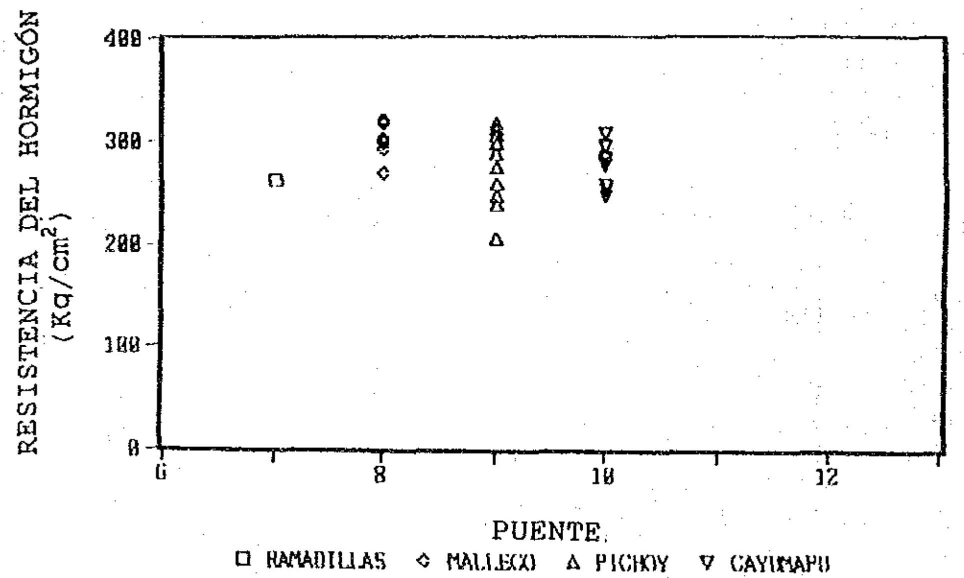


Figure 3-23 Schmidt Concrete Hammer Measurement Results

Table 3-10 Profometer Measurement Results

PROFOMETER

PUENTE	PRO-1			PRO-2			PRO-3			PRO-4			PRO-5		
	ϕ	e	r	ϕ	e	r	ϕ	e	r	ϕ	e	r	ϕ	e	r
AMOLANAS	16.5	100	32	26.5	105	17	23.5	80	61	22.8	90	8	22.8	101	35
PULLALLY	-	85	10	-	105	24	17.8	97	25	-	115	45	-	79	43
MAIPO	-	190	25	-	180	37	16.7	130	52	17.2	120	57	-	-	-
CLARO	16.1	100	12	16.4	109	8	-	-	-	-	-	-	-	-	-
LONCOMILLA	-	136	54	-	120	36	33.8	220	78	-	98	25	16	260	6
BIO-BIO	12.4	83	14	13	133	20	-	90	34	22	205	40	12.1	85	26
RAMADILLAS	12.0	102	3	10	80	47	18.6	87	45	12.6	106	25	-	-	-
MALLECO	16.5	73	67	9	90	30	-	87	39	-	91	30	-	78	27
PICHOY	12.2	98	29	24.5	130	24	-	-	-	-	-	-	-	-	-
CAYUMAPU	12.4	106	23	-	83	17	-	88	20	-	175	30	-	98	39

PUENTE	PRO-6			PRO-7			PRO-8			PRO-9			PRO-10		
	ϕ	e	r	ϕ	e	r	ϕ	e	r	ϕ	e	r	ϕ	e	r
AMOLANAS	-	101	37	-	83	68	18.4	65	11	-	-	-	-	-	-
PULLALLY	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MAIPO	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
CLARO	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
LONCOMILLA	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
BIO BIO	12.9	80	20		96	44	12	80	20	-	-	-	-	-	-
RAMADILLAS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MALLECO	-	61	55	-	63	53	-	95	49	-	80	47	-	80	47
PICHOY	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
CAYUMAPU	32	75	30	-	195	53	-	123	33	-	96	47	-	195	53

ϕ : Diametro
e : Espaciamiento
r : Recubrimiento

Table 3-11 Metal Hardness Measurement Results by Equo Tip

PUENTE	EQU-1	EQU-2	EQU-3	EQU-4	EQU-5	EQU-6	EQU-7	EQU-8	EQU-9	EQU-10
PULLALLY	371 Hb120	379 Hb123	376 Hb122	398 Hb137						
BIO-BIO	409 Hb144	460 Hb184	412 Hb146	417 Hb150	419 Hb151	499 Hb219	428 Hb158	428 Hb158	411 Hb145	438 Hb166
RAMADILLAS	383 Hb126	453 Hb178	406 Hb142	439 Hb167						
MALLECO	380 Hb124	368 Hb116	370 Hb117	406 Hb132	393 Hb132	427 Hb157	408 Hb143	447 Hb173	441 Hb168	
PICHOY	397 Hb135	392 Hb132	370 Hb117	391 Hb130	370 Hb117	402 Hb139				

Table 3-12 Neutralization Test Measurement Results

C O N K I T

PUENTE	CAR-1			CAR-2			CAR-3			CAR-4			CAR-5			CAR-6			CAR-7		
	C	PH	S	C	PH	S	C	PH	S	C	PH	S	C	PH	S	C	PH	S	C	PH	S
AMOLANAS	0.0	11.0	B	10.0	9.5	B	49.0	9.5	A	32.0	8.5	D	5.0	10.5	B	7.0	10.5	-	12.0	9.0	D
PULLALLY	11.0	9.0	-	30.0	8.5	C	25.0	9.5	-	-	-	-	-	-	-	-	-	-	-	-	-
MAIPO	5.0	9.0	-	2.0	9.0	B	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
CLARO	25.0	8.5	E	22.0	8.5	-	25.0	9.0	C	15.0	8.5	-	-	-	-	-	-	-	-	-	-
LONCOMILLA	10.0	9.5	-	5.0	9.5	-	39.0	9.5	B	-	-	-	-	-	-	-	-	-	-	-	-
BIO-BIO	13.0	8.5	D	11.0	9.0	C	36.0	8.5	B	37.0	8.5	D	-	-	-	-	-	-	-	-	-
RAMADILLAS	36.0	8.5	D	17.0	9.0	C	65.0	8.5	D	-	-	-	-	-	-	-	-	-	-	-	-
MALLECO	10.0	9.0	B	9.0	10.0	B	12.0	9.0	-	-	-	-	-	-	-	-	-	-	-	-	-
PICHOY	27.0	8.5	D	19.0	8.5	E	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
CAYUMAPU	35.0	8.5	C	24.0	8.5	D	24.0	8.5	-	-	-	-	-	-	-	-	-	-	-	-	-

C : depth of neutralization
P H : PH
S : degree of damage to reinforced bar

(2) Basic Structure and Deformation Survey

Measurement of basic bridge structures using weight equipment and the deformation survey shown below were performed at the same time as the various surveys using measuring instrumentation. The data is summarized in a separate collection of diagrams. Please refer to this information. In regard to the survey method refer to the bridge maintenance and repairment guidelines, 7-4 Survey Methods for Bridge Deformation. The problems of each bridge, as discovered from the deformation survey, are discussed in this section.

1. Deflection measurement and deformation survey of the main girders, mainly with twisted steel girders.
2. Measurement of the tilt and settlement of the substructure.
3. Concrete crack survey
4. Scouring survey

1) Survey Results

Table 3-13 shows a summary of the deformation causes of each bridge.

Table 3-13 Deformation Survey Result

Bridge Name	Major Deformation
AMOLANAS	Cracks in the pavement are obvious because of the vehicle wheel load. All of the pillars of the bridge pier on the Santiago-side span are deformed on the abutment side.
PULLALLY	Vibration is large when heavy vehicles cross. Deformation of the main girder is observed by visual observation. A difference of approximately 7 cm exists on the joints of the 3 and 4 continuous spans. Scouring of the bridge piers is advanced.
MAIPO	Scouring of the bridge piers on the Talca side (C11) is advanced. As well, settlement (approximately 10 cm) of the upper structure on the same bridge pier and lateral displacement (approximately 13 cm) are large.
CLARO	Scouring of C2 bridge pier foundation.
LONCOMILLA	Approximately 35 cm displacement is caused relative to the spans neighboring on the C5 bridge pier. At the same location, the left and right road surface height is displaced approximately 12 cm.
BIO-BIO	Scouring of the bridge piers is advanced and piles are exposed for a majority of the bridge piers. At the location of the C46 bridge pier, the left and right road surface height is displaced approximately 19 cm. Possible tilting of the same bridge piers is high. Road surface deformation is so advanced that the road is not adequate for high-speed driving.
RAMADILLAS	Road surface deformation is obvious because of settlement and tilting of the bridge piers. Deterioration of the bridge pier girder seat is at a dangerous level.
MALLECO	Deformation of the whole bridge is large when heavy vehicles cross. A displacement of approximately 8 cm is observed between the top and the base of the bridge piers. However, the cause is not clear as to whether this is a result of construction error or post-construction error.
PICHOY	Road surface deformation because of settlement and tilting of the C2 and C3 bridge piers is large. The maximum lateral displacement is 13 cm and the vertical displacement is 14 cm. Bridge piers on the Valdivia side are tilted and the bearings have come away.
CAYUMAPU	C1 bridge pier settlement (approximately 10 cm); C2 bridge pier tilt (approximately 4 degrees); E2 abutment tilt (approximately 11 degrees)

2) Examination

As a result of the deformation survey, the causes of deformation are considered as follows when the results are separated. Deformation does not occur due to any particular cause but is generated as a result of a combination of factors as shown in Table 3-14.

Table 3-14 Causes of Bridge Deformation

Bridge Name	Deformation Case			
	1	2	3	4
AMOLANAS	©			
PULLALLY	O	O	O	©
MAIPO		O		
CLARO		O		
LONCOMILLA		O	©	©
BIO-BIO			©	
RAMADILLAS	O		©	O
MALLECO	O			O
PICHOY		O	©	©
CAYUMAPU		O	©	©

1. Damage by heavy vehicle traffic.
2. Deformation caused by scouring from rivers.
3. Settlement deformation due to insufficient foundation support.
4. Deformation due to earthquakes.

© : major cause of deformation
O : secondary cause related to deformation

(3) Geological Survey

The geological surveys were conducted for the following 7 bridges;

PULLALLY Bridge
MAIPO Bridge
LONCOMILLA Bridge
BIOBIO Bridge
ANTIGUO Bridge
MALLECO Bridge
CAYUMAPU Bridge

Of the above, it is difficult to conduct the standard penetration test for MAIPO Bridge . It is because the soil at the survey site around this bridge is gravel inclusive of small stone. In this bridge, the geology is inspected by hand excavation survey. Figures 3-24 to 3-32 show the survey results.

The purpose of the survey is to obtain the rough conditions of geology around these bridges. N-value, layer and depth of bearing foundation were estimated by this survey which involved only 1 or 2 boring tests. Prior to the construction of a bridge, detailed geological surveys are necessary.

(4) Other Surveys

1) Traffic Survey

The traffic survey was conducted near BIO BIO Bridge and MALLECO Bridge. Traffic volume counted on these bridges is shown in Appendix.

2) Crack Distribution Survey

The crack distribution survey was conducted in AMOLANAS, PULLALLY, MAIPO, CLARO, RAMADILLAS, MALLECO Bridges.

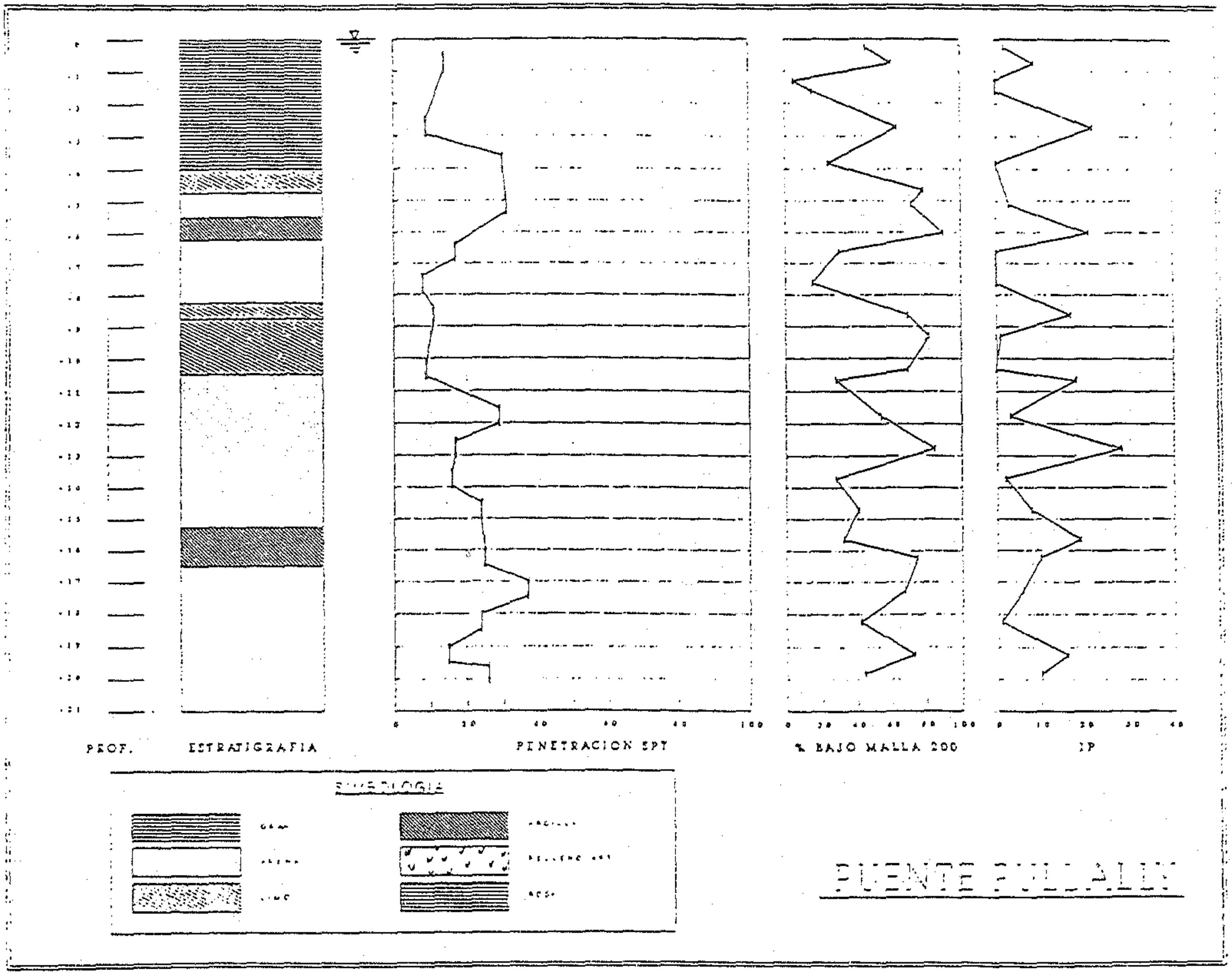


Figure 3-24 Geological Data for Pullally Bridge

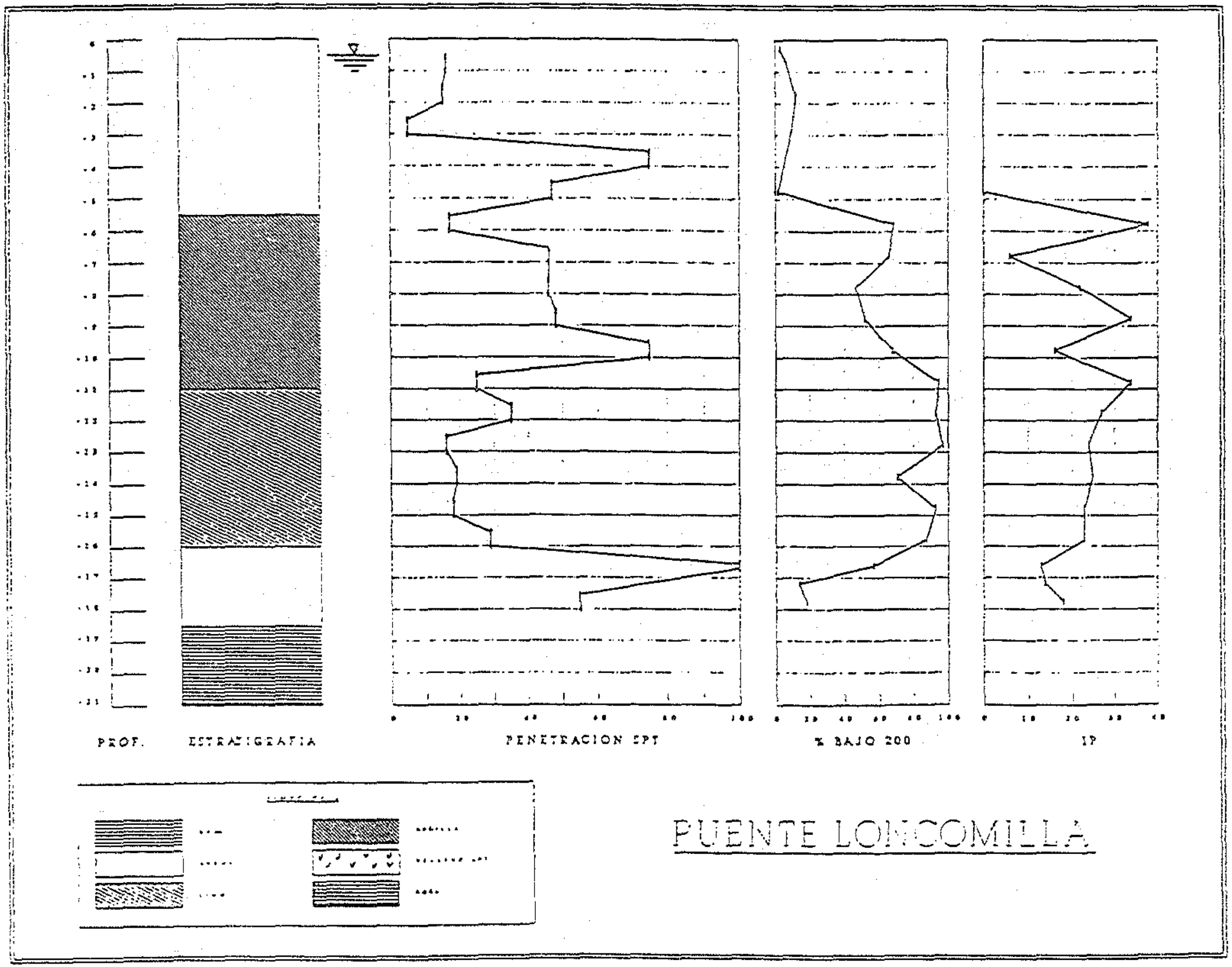


Figure 3-25 Geological Data for Loncomilla Bridge

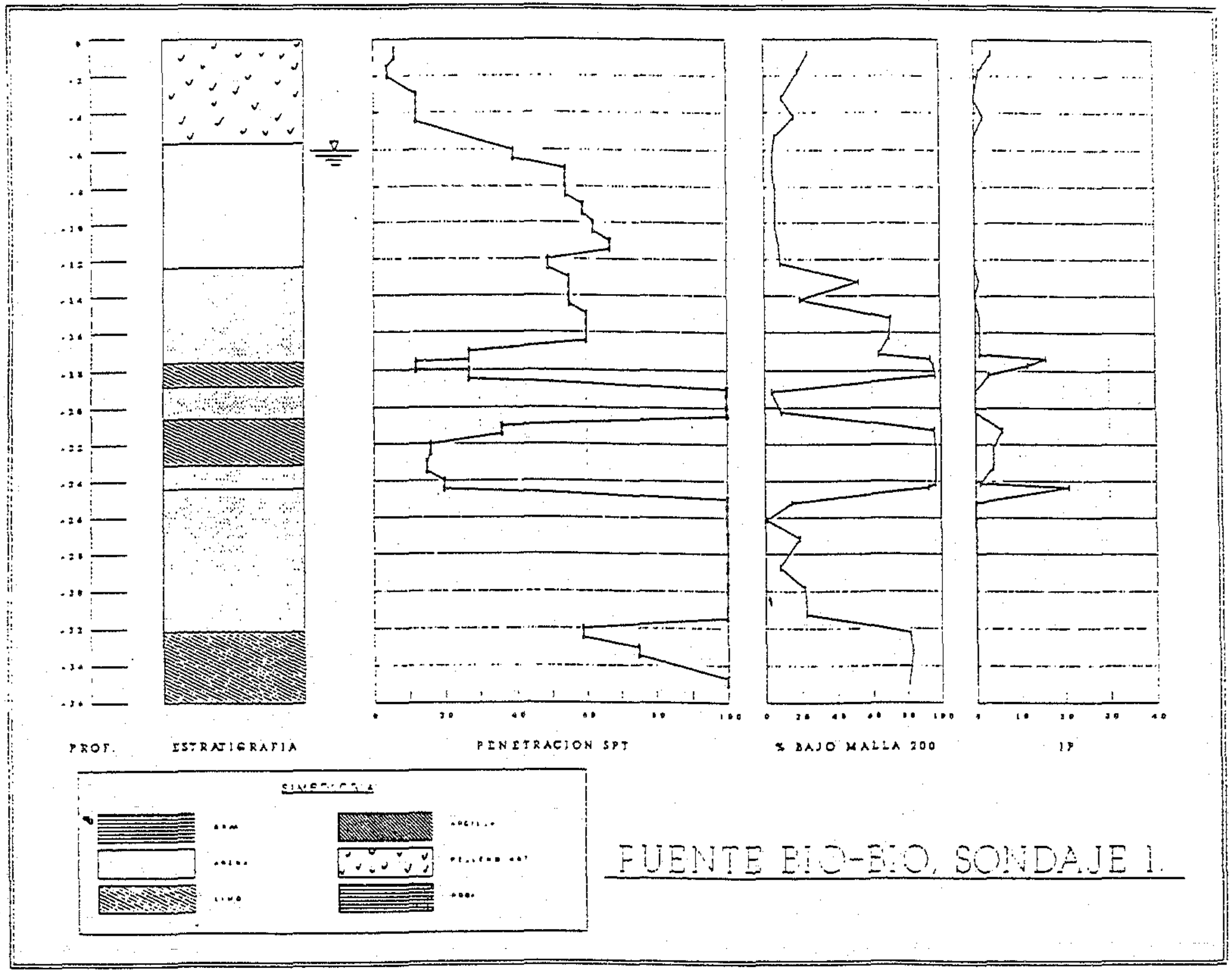


Figure 3-26 Geological Data for Bio Boi Bridge (1)

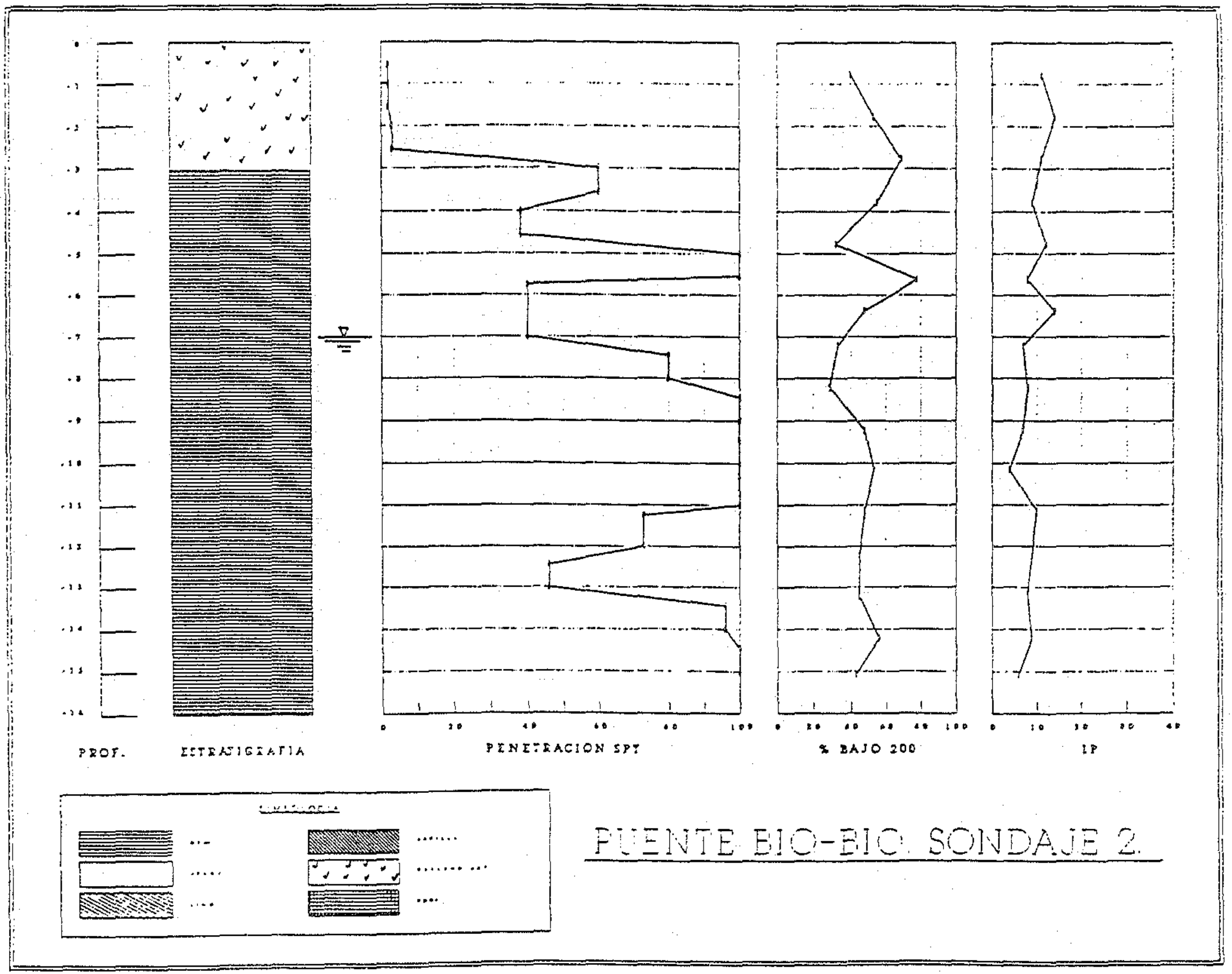


Figure 3-27 Geological Data for Bio Bio Bridge (2)

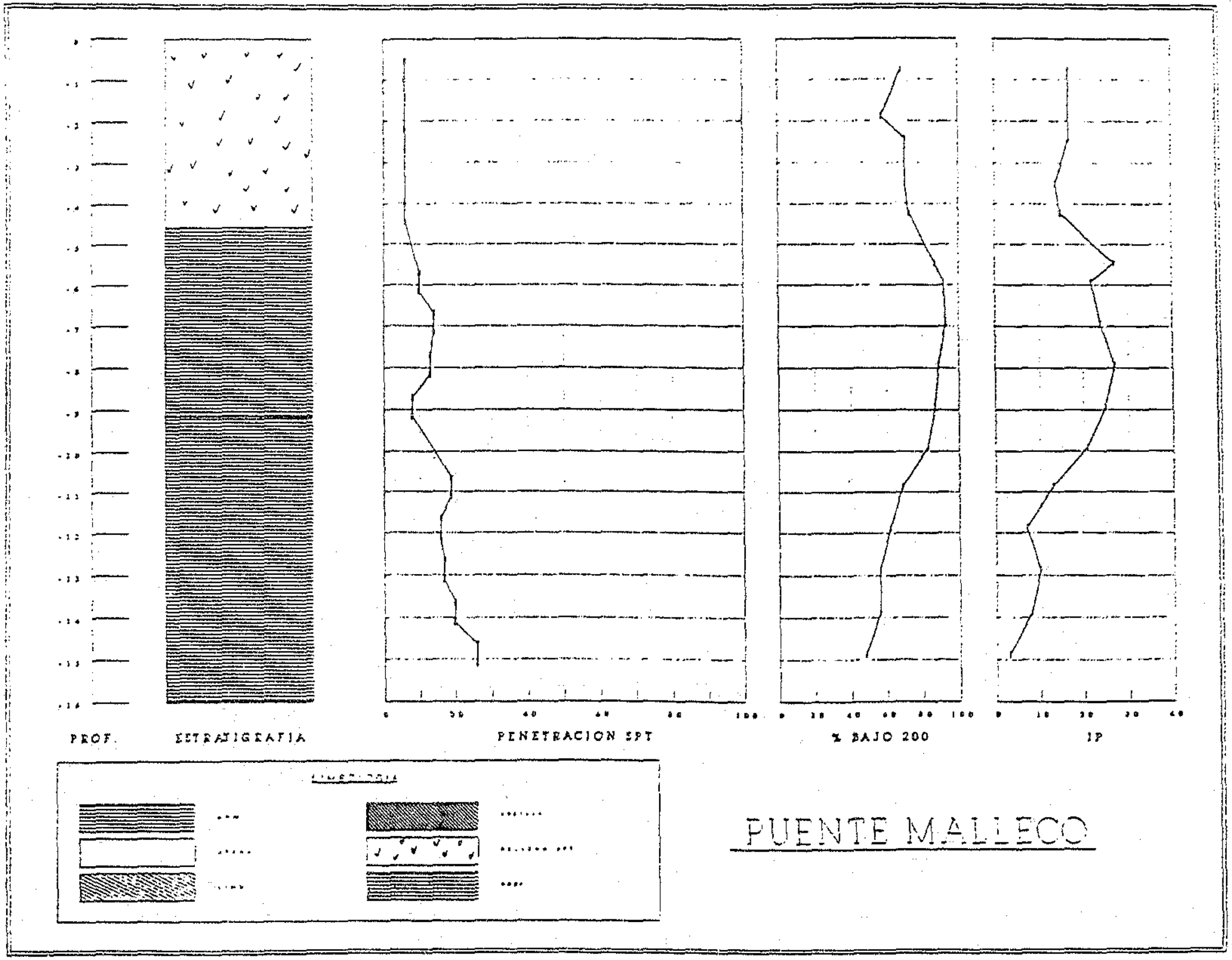


Figure 3-28 Geological Data for Malleco Bridge

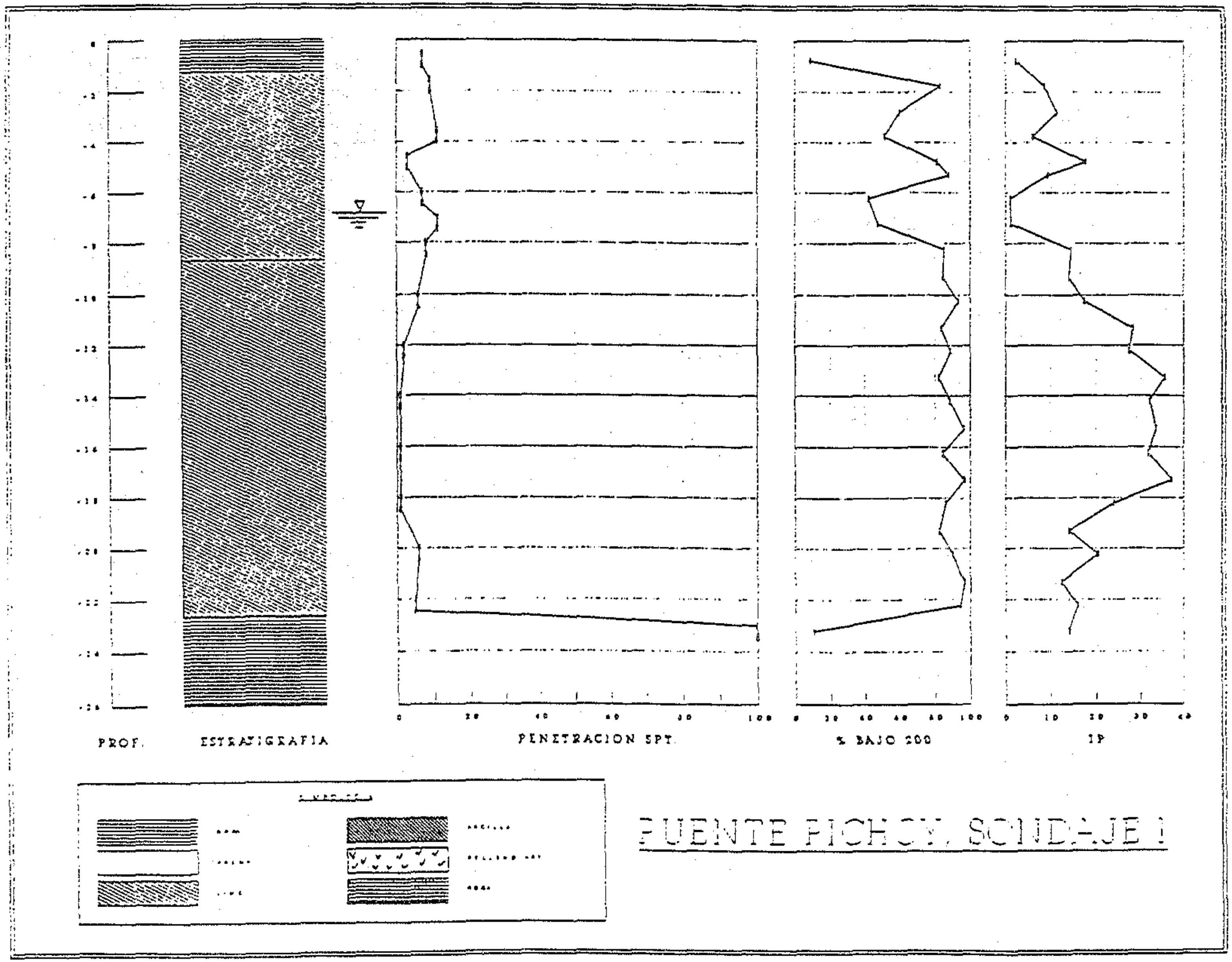


Figure 3-29 Geological Data for Pichoy Bridge (1)

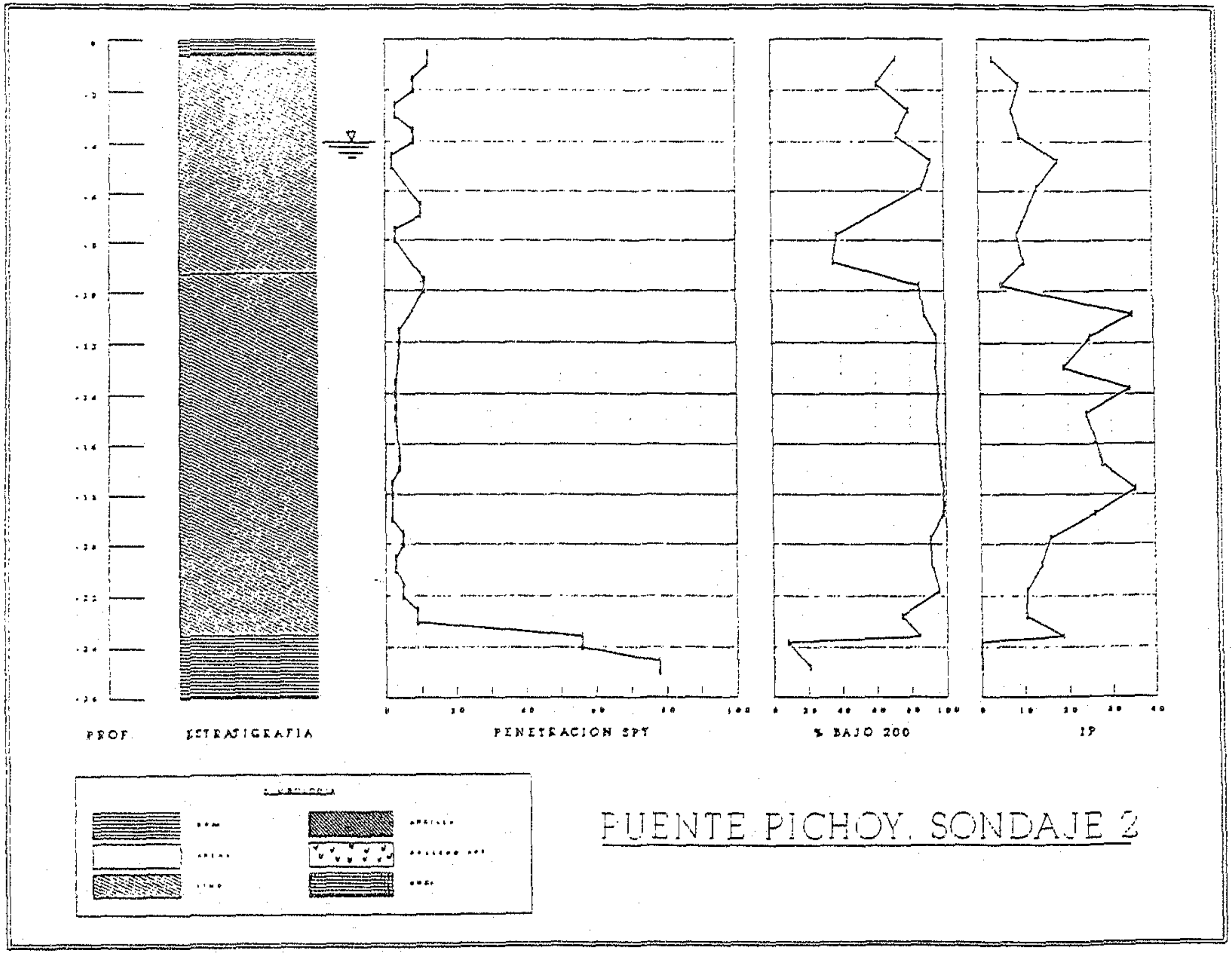


Figure 3-30 Geological Data for Pichoy Bridge (2)

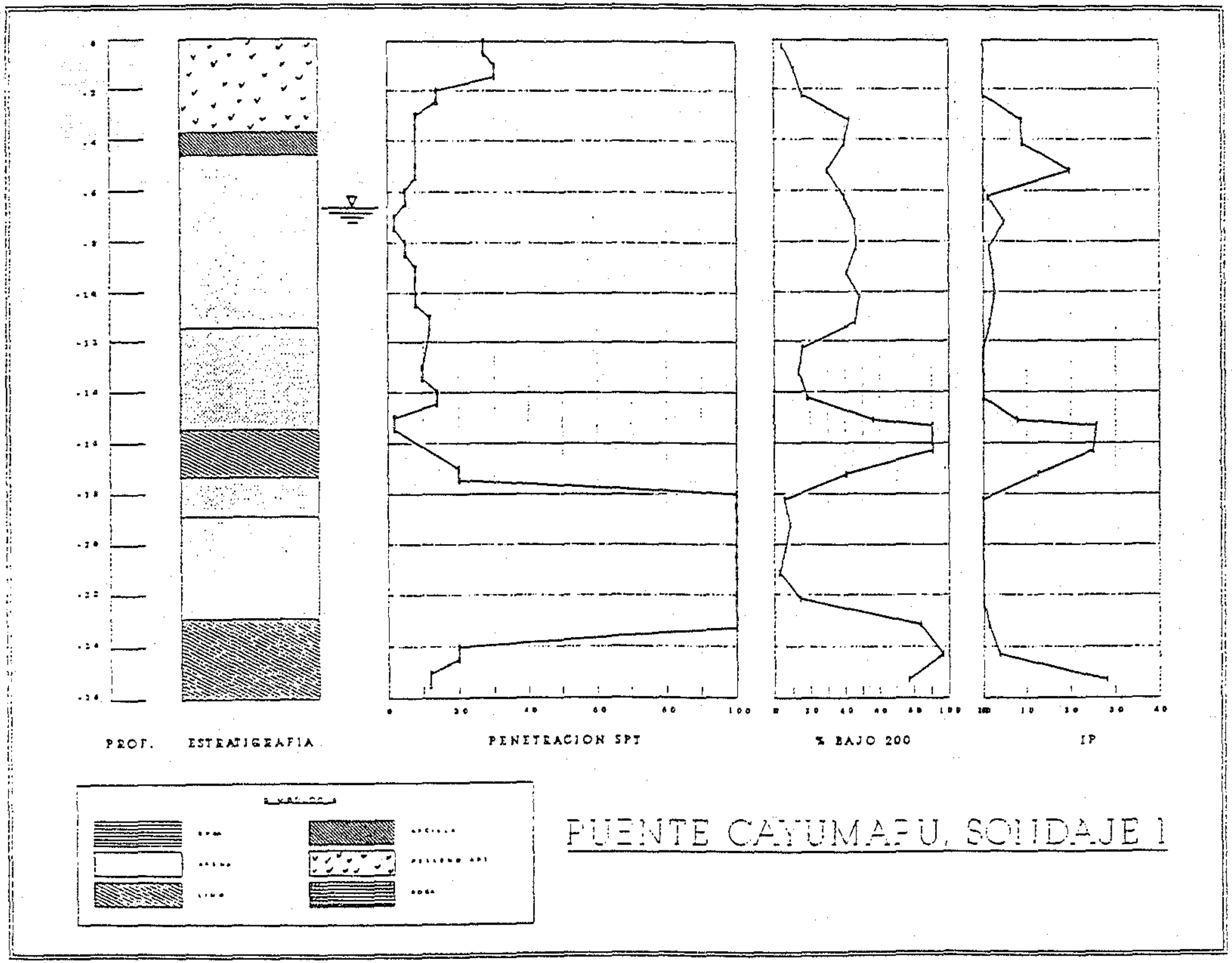


Figure 3-31 Geological Data for Cayumapu Bridge (1)

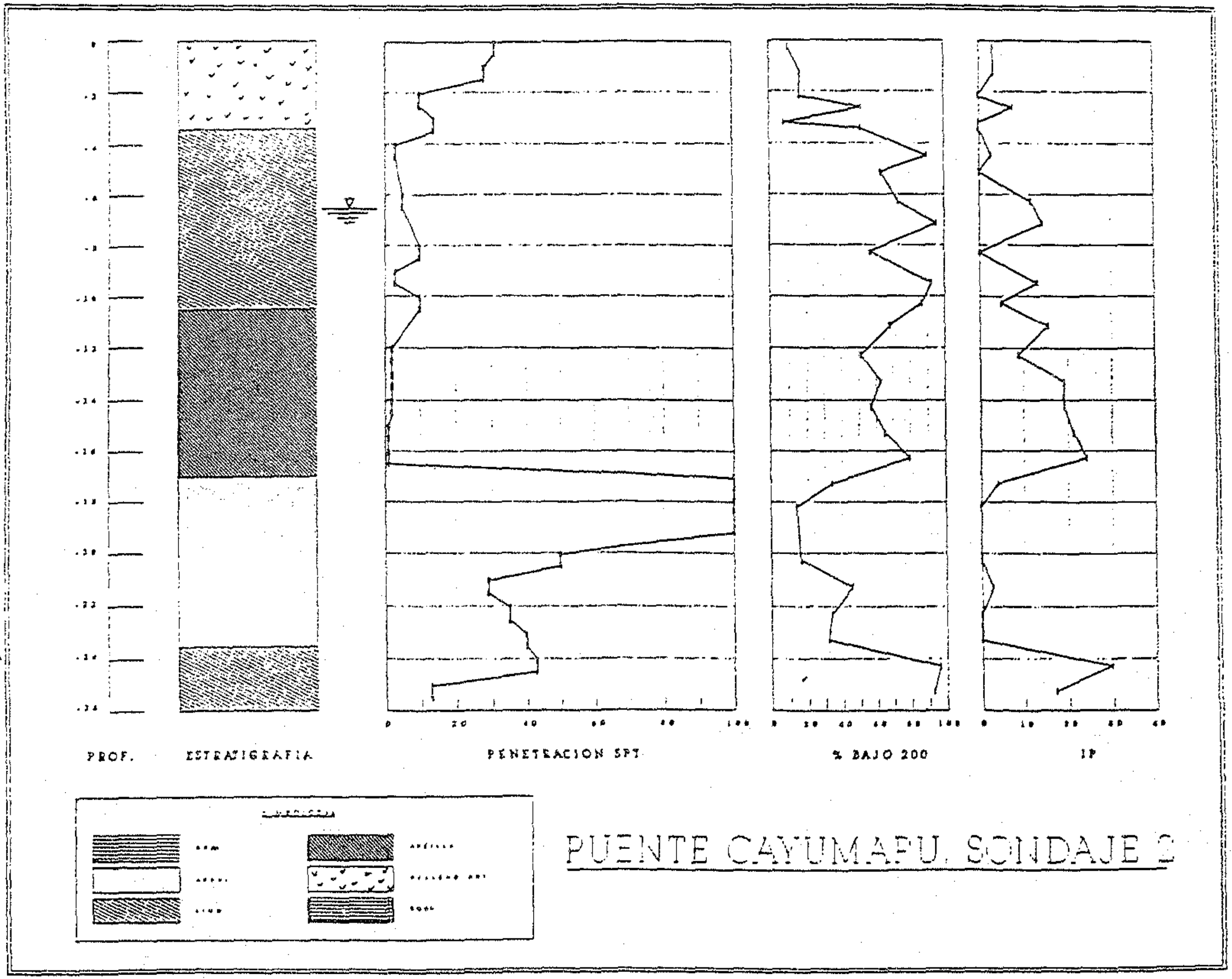


Figure 3-32 Geological Data for Cayumapu Bridge (2)

3-2 Load Test

3-2-1 Outline of Load Test

A load test and a stress frequency measurement of vehicles crossing were performed at the Peuco Bridge, located 60 kilometers south of Santiago City. Test measurements were performed using the latest instruments with the Chilean staff: the purpose being to introduce load testing to the Chilean staff. The results of the load test did not directly reflect the maintenance and repair plans which were the main purpose of this project. The principle objective was to clarify the basic characteristics of the bridge.

Peuco Bridge had 2 main composite plate girders and a relatively large slab span (3 continuous spans). All joints were welded. Based on these characteristics a load test was performed as described below.

Test Points are following:

(1) Confirmation of structural characteristics of a 2 main girder bridge.

1) deflection of the main girder and slab
2) load distribution effect by the floor beam
3) behavior of the composite girders

(2) Confirmation of fatigue characteristics

Fatigue life of the main girder at change point of cross -section and the welded part of the vertical stiffener

(3) Confirmation of the relationship between load and stress.

Measurements were performed using strain gauges attached to monitoring points on the main girders, floor beam, and vertical stiffener. Displacement gauges were also set on the main girders and slabs near the center of the span in order to act as sensors. The load test procedure is shown in Figure 3-33

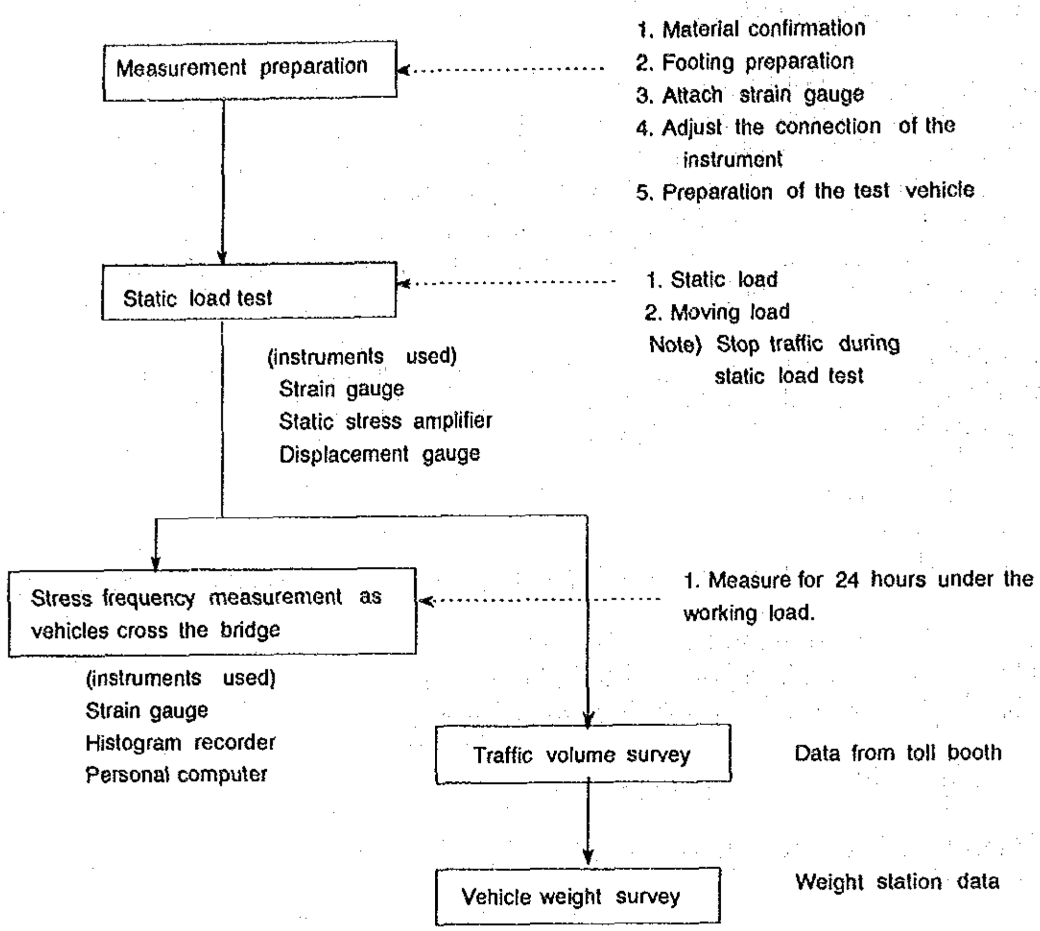


Figure 3- 33 Load Test Procedure

3-2-2 Outline of Test Targeted Bridge

Peuco Bridge, which is 62 kilometers south of Santiago City along National Route 5 (Figure 3-34), was a 3 span continuous plate girder (synthetic girder). The basic structural items were as follows.

Dimension of Peuco Bridge

Type: 3 span continuous plate girders (composite girder)
Span: 33 meters + 33 meters + 33 meters
Number of lanes: 2 lanes ongoing
Width: 8 meters (total 10.4 meters)
Number of main girders: 2 main girders
Girder intervals: 6.8 meters
Girder height: 2.0 meters
Slab thickness: 20 centimeters

The bridge possesses the following structural characteristics.

1) All joints were welded
2) The floor beams were spaced at 4 to 8 meters.
3) Crossbeams, cross -frames, and cross -stiffeners were not used.
4) Both girders ends and the middle bridge pier had inclinations of approximately 58 degrees.
5) Flange thickness was increased by welding a cover plate near the span center and middle supporting point where the cross -section force was large.
6) The floor beam at the central supporting point was secured by concrete.
7) Pre -cast slabs were used.

Cracks generated from the lower flange reached the web near the center of the span on the Santiago side and large -scale repair had been performed. Traces of breaks were also seen on the upper member of the floor beam and the diagonal members. These had been repaired by simple welding. There was evidence that the floor beam had been moved, and it was not clear whether the breaks were a result of load stress or artificial cuts in order to move the floor beams.

NOMBRE DEL PUENTE	KILOMETRO	NOMBRE DE LA VIA	ROL DE LA RUTA	PROVINCIA	REGION	CODIGO DEL PUENTE
PEUCO N°1	61,9	LONGITUDINAL SUR	RUTA 5		VI	

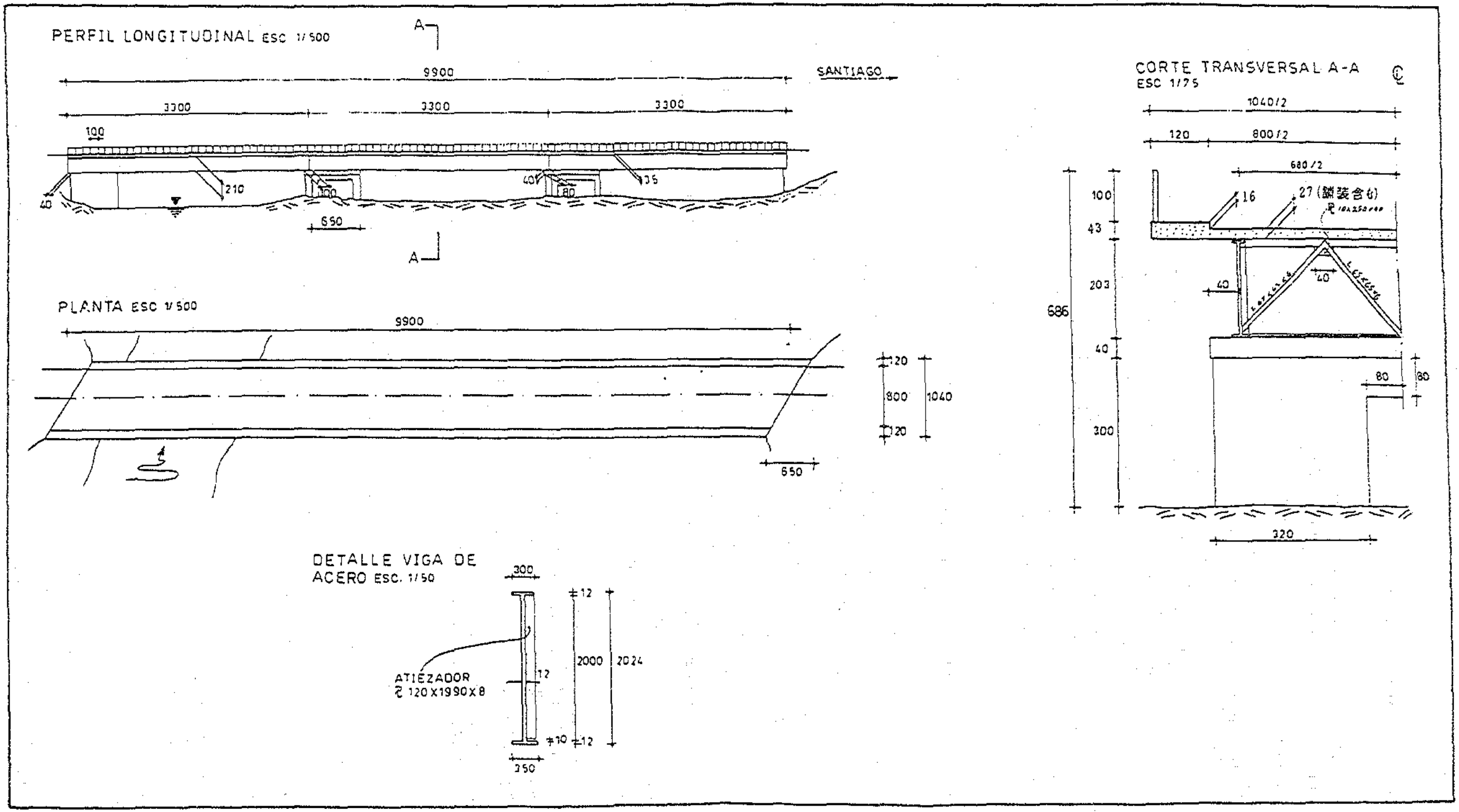


Figure 3- 34 Peuco Bridge General Drawings

3-2-3 Test Method

A. Setting Strain and Displacement Gauges

(1) Setting Location of the Strain Gauge and Displacement Gauge

In this test the strain gauge measured stress and the displacement gauge measured deflection. The main measuring points were located on the side span on the Santiago side for the following reasons.

1) In comparing the bridge's span, the maximum value of main girder stress was obtained on the side span.

2) Damage to the main girder had occurred on this span in the past, thereby confirming stress on the damaged girder.
3) There was little water in the riverbed under the girder of this span, making it suitable for establishing footings and setting the gauges.

4) The side span was convenient for directing vehicles during the load test as it was close to the end of the bridge. The locations of the strain and displacement gauges are shown in Figure 3-35. Table 3-15 shows the reasons for attaching the strain gauge. During the initial planning stage the following features (a. and b) were considered for monitoring, but these were later ignored because of the difficulties in attaching the gauges caused by the condition of the bridge.

   a. Attachment of a 3-axis gauge on the upper portion of the vertical stiffener;
   Insufficient space between the upper flange and the upper member of the floor beam.

   b. Space between the vertical stiffener and the lower flange;
   Presence of a welding bead.

Monitoring the cross-section near the center of the span, displacement gauges were set at 3 locations (on the lower flanges of the 2 main girders and at the center of the slab) in order to confirm the difference in deflection between the main girders due to the angle of skew, and between the main girders and the slab.

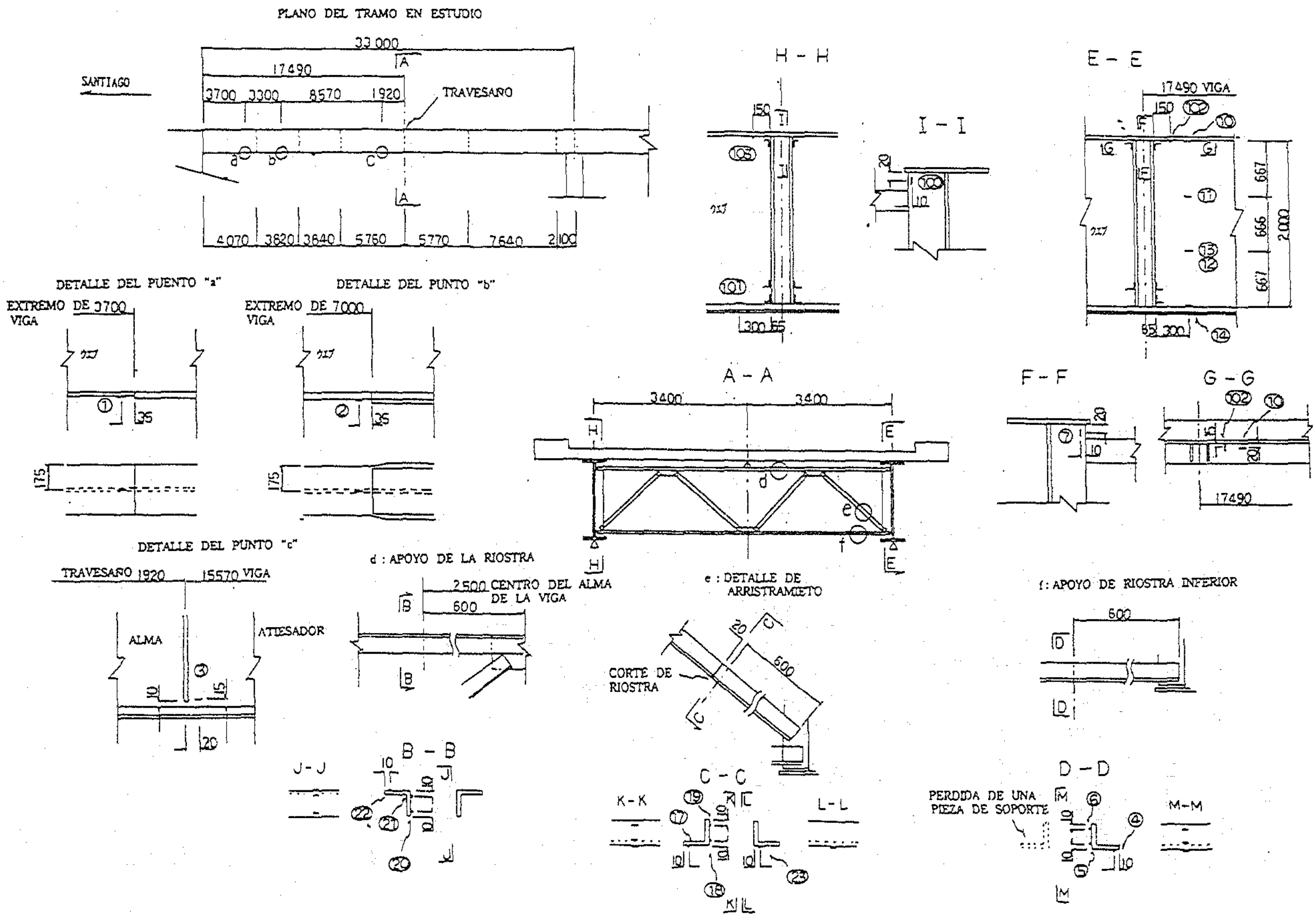


Figure 3-35 Setting Locations of the Strain Gauge and Displacement Gauge

Table 3-15 Location of the Strain Gauge

Gauge Number	Attached Location	Reasons
1～2	Main girder lower flange	Check fatigue at the butt weld joint on the cross-section change point.
3	Vertical stiffener location web	Check fatigue at the weld end of the vertical stiffener.
4～6	Lower member of the floor beam	Confirmation of strength transmission of the lower member floor beam, and a fatigue check of the welding of the attached part.
10～14	Vertical stiffener	Confirmation of stress generated on the vertical stiffener through slab deflection and a welded part fatigue check.
17～19、23	Main girder flange and web	Confirmation of main girder stress and location of the neutral axis.
20～22	Diagonal member of the floor beam	Confirmation of strength transmission of diagonal member of the floor beam.
101	Upper member of the floor beam	Confirmation of strength transmission of upper member of the floor beam and a fatigue check of the welding of the gusset part.
102、103	Lower flange	Confirmation of main girder stress.
	Upper flange	Confirmation of deflection direction stress of the upper flange slab.

Note)

1) Gauges 12 and 13 are located at the back surface of the web. This is to confirm the tranverse bending moment stress of the web.
2) Gauges 7 and 100, 14 and 101, and 102 and 103 are symmetrically located from the center of the bridge axis.
3) Gauges 102 and 103 record the deflection of the bridge axis in a right angle direction in order to confirm the upper flange stress through deflection of the slab.

(2) Attachment Method for the Strain Gauge

The method of attaching a strain gauge is shown in Figure 3-36 . The effects of temperature are compensated in this test by building a calibration gauge as shown in Figure 3-37 by using a 2 axis gauge. The effects of temperature change of the lead line were eliminated by connecting a lead line of the same length to an active gauge (gauge measures the targeted strain) and a dummy gauge (the gauge located perpendicular to the active gauge). For this bridge, strain (ε n) as a measured value was recorded at the same time both on the dummy gauge and the active gauge according to Poisson's ratio and the measured strain is included its influence of temperature. Therefore, the measured strain was corrected according to the following formula.

$$\varepsilon R = \varepsilon n / (1 + \mu)$$

where

ε R : monitored strain

μ : Poisson ratio (= 0.3)

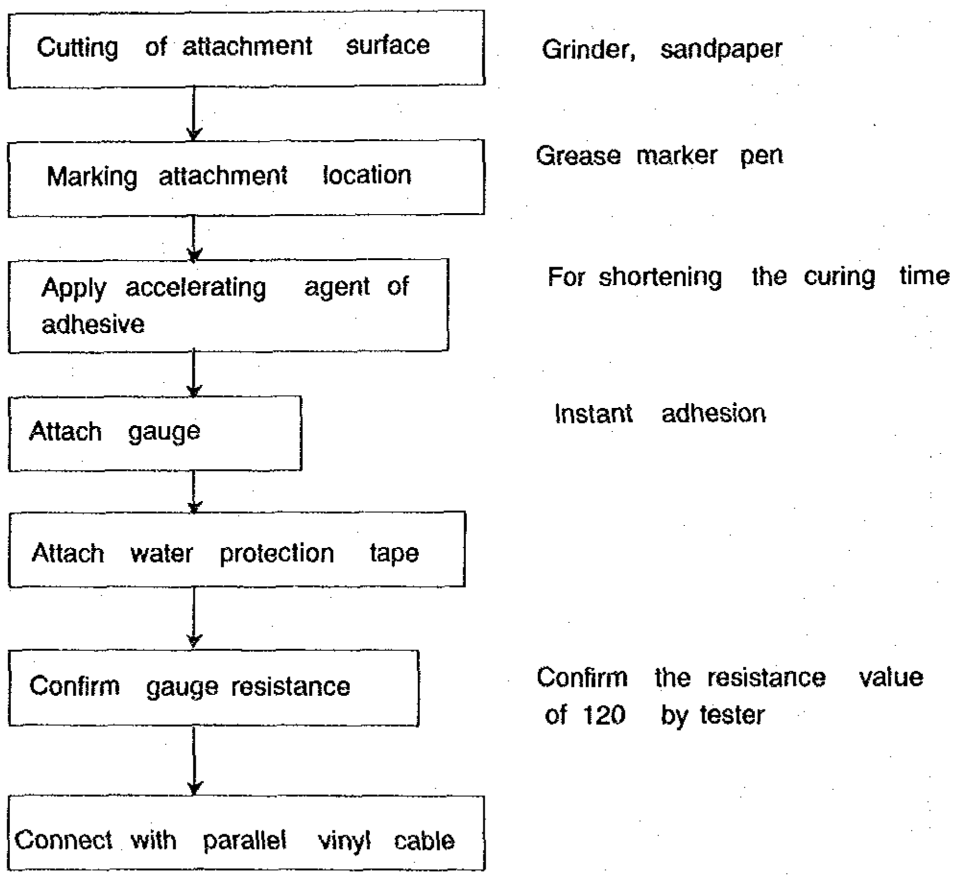
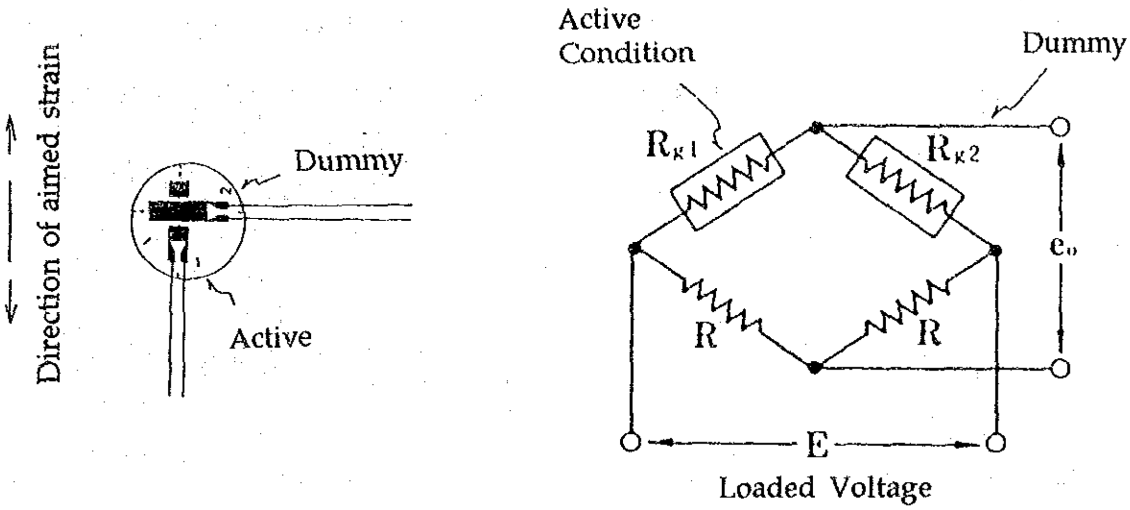


Figure 3-36 Strain Gauge Attachment Procedure



μ : Poisson ratio
Rgi,Rg2 : Registor of Gauge
Rgi : Strain : e0
Rg2 : Strain : -μe0
R : Registor

Figure 3-37 Strain Gauge and Calibration Gauge

(3) Displacement Gauge Setting Method

The displacement gauge was attached to a wooden pile, which set into the ground, and strain was measured by connecting a steel wire hanging from the main girder and slab. Figure 3-38 shows the setting of the displacement gauge.

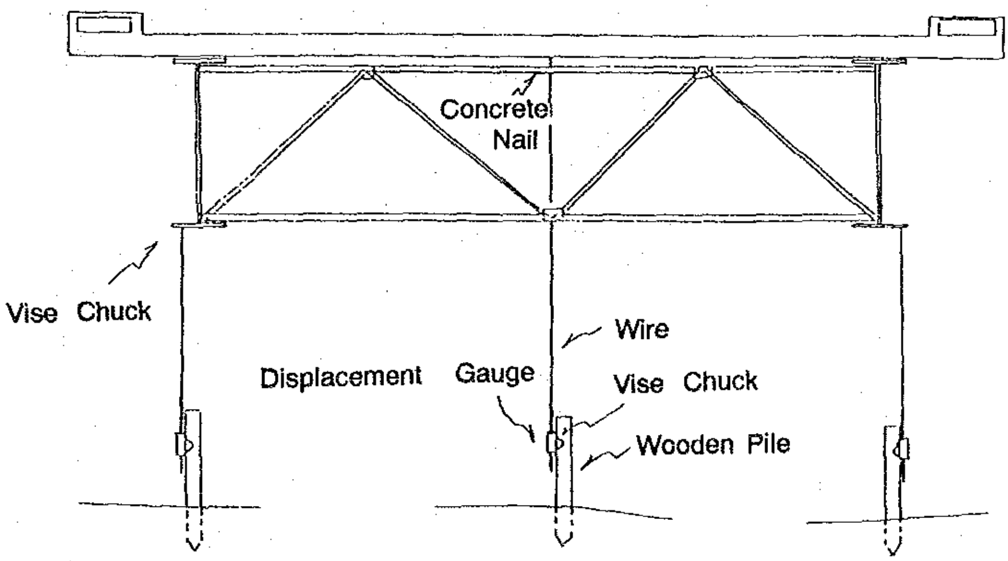


Figure 3-38 Displacement Gauge Setting

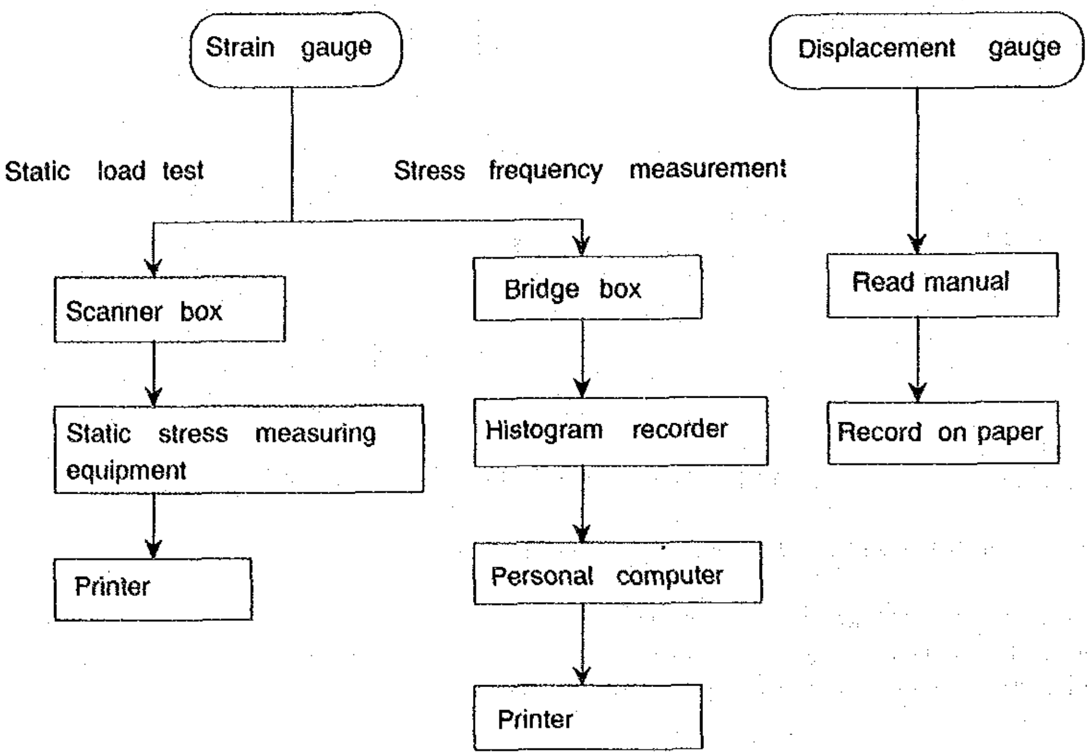
B. *Details and Method of Measurement*

Measurement was performed divided roughly into the following 2 items.

1. Measurement of the strain and deflection created by static vehicle load.

2. Stress frequency measurement during vehicles crossing.

The measurement system is shown in Figure 3-39.



Note) The strain gauge was common in both cases.
The displacement gauge did not require a transformer.

Figure 3-39 Measurement System

(1) Static load test

1) Test Vehicle

A 15 ton motor grader was used as a test vehicle. The test also placed the motor grader on a trailer to increase the weight to 36 tons. Test vehicle details are shown in the Figure 3-40.

2) Load Location and Test Method

Traffic was stopped for approximately 3 hours while the static load test was performed. Location of the test vehicle was then marked with gum tape. Load locations are shown in Figure 3- 41. Load locations were at the 1/5 point of each span, and were the locations where strain gauges were attached. Vehicle location, right angle to the bridge axis, was 1/4 of the road width which matched the center of the car axle. The static load test was performed in 2 cases.

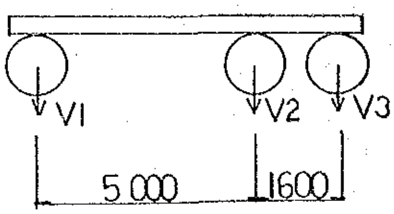
a. Monitoring the relationship between load and stress.
b. Monitoring the influence line.

As a dynamic transformer was not used in measuring, the influence line was obtained by measuring the static strain at multiple locations while moving the vehicle in the direction of the bridge axis.

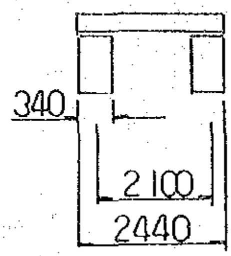
3) Results of Load Test

The stresses of each member of the bridges obtained from the load test are shown in Tables 3-26 to 3-28. The displacement of main girder and slab is also shown in Table 3-29.

Motorized Grader



Common Section for All Axles

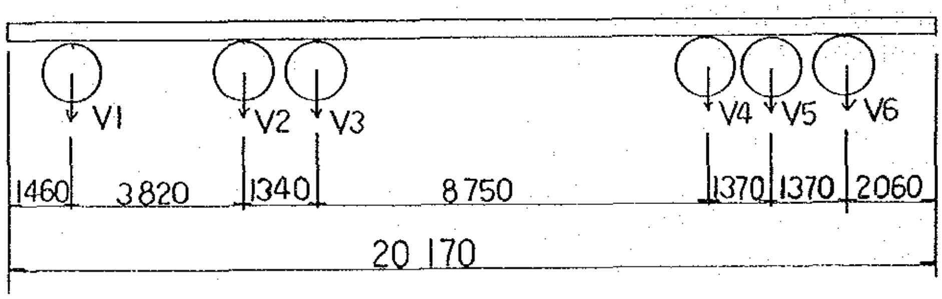


Axle Load V1= 4.65 t
V2= 5.04 t
V3= 4.99 t

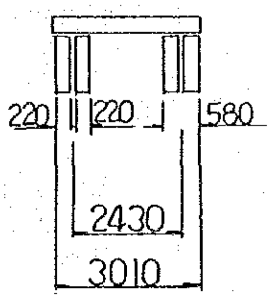
14.68 t

ﾀﾝﾃﾞﾑ軸
V2 + V3= 10.03 t

Trailer Truck



Common Section for All Axles



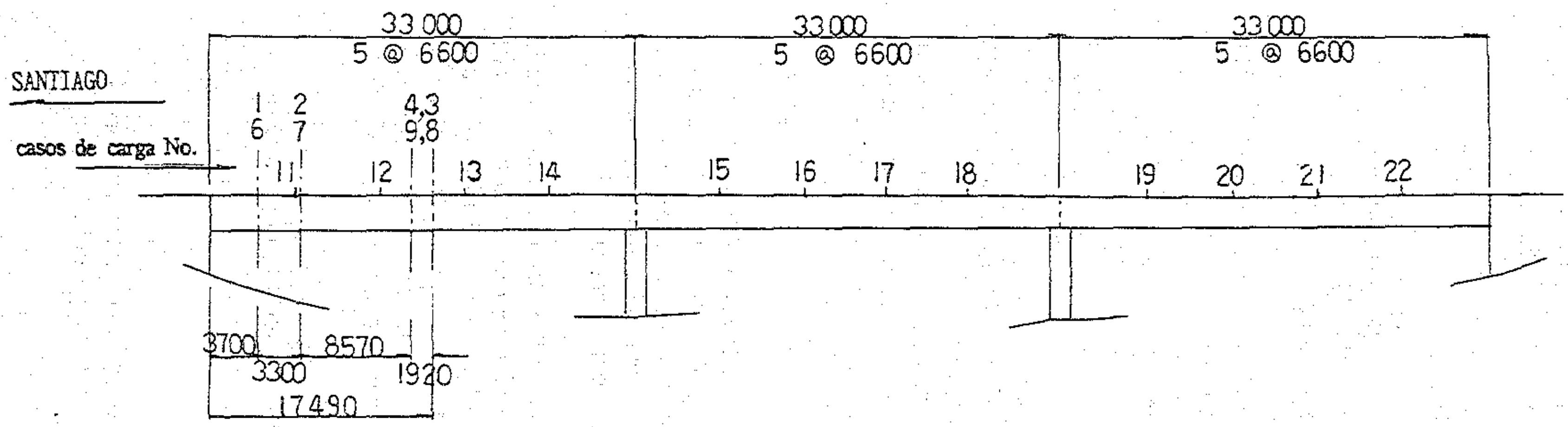
Axle Load V1= 6.45 t
V2= 6.66 t
V3= 6.71 t
V4= 4.29 t
V5= 5.46 t
V6= 6.95 t

36.52 t

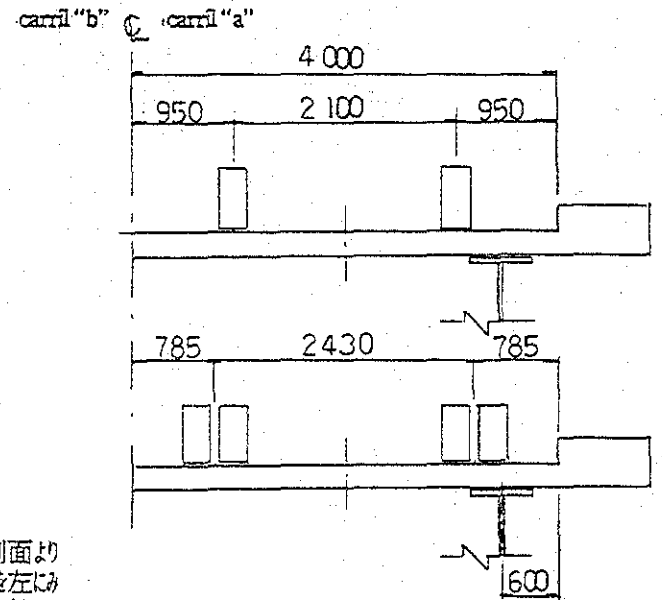
Tandem Axle
V2 + V3= 13.37 t

V4 + V5 + V6= 16.70 t

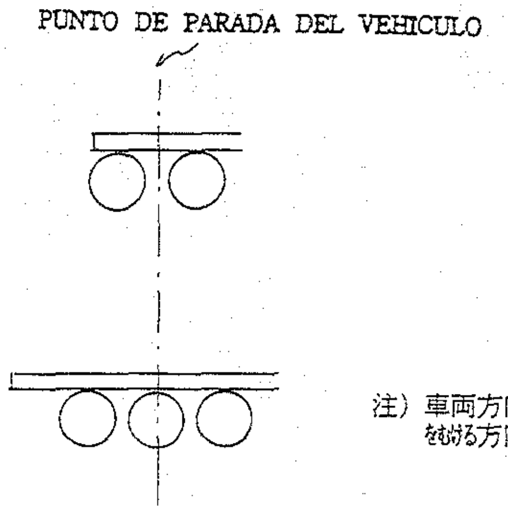
Figure 3-40 Vehicle Details for Load Test



載荷ケース
1, 6 主桁断面変化点
2, 7 主桁断面変化点
4, 9 垂直スティフナ 位置
3, 8 着目対傾構位置
注） 1～4 a車線
6～9 b車線
11～22 各径間を5等分する位置 (a車線のみ)



注） a車線は 側面よりSantiagoを左にみて手前の 車線



注） 車両方向はSantiagoに背をむける方向。

Figure 3-41 Vehicle Stop Location

(2) Stress Frequency Measurement

1) Measurement Points

Stress frequency measurements were conducted at the following 8 locations which are indices of fatigue.

Stress Frequency Measurement Location

Gauge Number	Member Location
1.	Main girder lower flange cross-section change point
2.	Main girder lower flange cross-section change point
3.	Vertical stiffener lower end of welded web part
5.	Lower member of the floor beam
7.	Near the vertical stiffener upper flange
14.	Near the main girder lower flange at the center of span
21.	Upper member of the floor beam
102.	Main frame upper flange right angle direction

2) Stress Frequency Measurement

Stress frequency measurements were performed for a 24 hour period under the weight of crossing vehicles. After the dynamically measured strain weight was processed through a histogram recorder and personal computer, the stress frequency was counted. In regard to fatigue damage, the fluctuation in stress amplitude was more important than the maximum value of the stress. The fluctuation in stress amplitude was counted by the rain flow method.

<Rain Flow Method>

The rain flow method monitors the hysteresis loop (closed loop) of the stress-strain curve and counts its strain amplitude frequency. Looking at (a) in Figure 3- 42, there is a part that never closes in the hysteresis. In the rain flow method, such a part is counted by allowing 1/2 fatigue damage to 1 range; therefore, when the same valued 1/2 cycle is counted twice it means that 1 cycle of 1 closed loop is counted.

Figure 3-42(b) shows a vertical time axis: the time change of the strain signal wave is viewed as a multiple roof structure, P1 P2 P3 P9 P10, and imagine the flow of rain drops in the order of peaks and valleys from the peak of the crest. The rain drops flow and stop by satisfying the following 3 conditions, and measure the abscissa where the rain drops flow until they stop. The size of the analyzed strain amplitude is equivalent to this measurement and it becomes an effective component in which to measure fatigue.

Figure 3-42(c) analyses the results of this method as the strain amplitude to be counted.

Conditions for Stopping the Flow of Rain Drops

Condition 1. Rain drops begin to flow in the order of numbers from the top of the ridge and continue to drop to the lower ridge until the stop conditions are satisfied.

Condition 2. The rain falling from the end of the eaves will stop when it satisfies either of the following 2 conditions.

(a) When flowing to the right, another roof appears more to the left side than the starting point of the rain drops flowing to the right. (A flow with a starting point of P1, stops at P4 because P5 appears more on the left side than P1.)

(b) When flowing to the left, another eaves appears more to the left side than the starting point of the rain drops flowing to the left.

Condition 3. The flow stops if the rain is already flowing on part of the roof. (Flow on the roof of P2 - P3 whose starting point is P2 stops when P3 drops.)

(3) Traffic Volume Survey

Data for the traffic volume survey were obtained from the tollbooth at the bridge in order to determine the characteristics of traffic during the stress frequency measurements. The purpose of the survey was to determine the number vehicles using the bridge and the mixed ratio of large vehicles, and to discuss the relationship between traffic volume and damage figures obtained by measurements.

(4) Vehicle Weigh Survey

The weight station is located 3km south of the Peco Bridge and weight regulation of large vehicles is performed. At the weight station axle load measurements were performed according to the vehicle-type classification shown in Table 3-16. In order to determine the weight of large vehicles crossing the bridge during measuring, axle load data was obtained during the several hours in which measurements were taken.

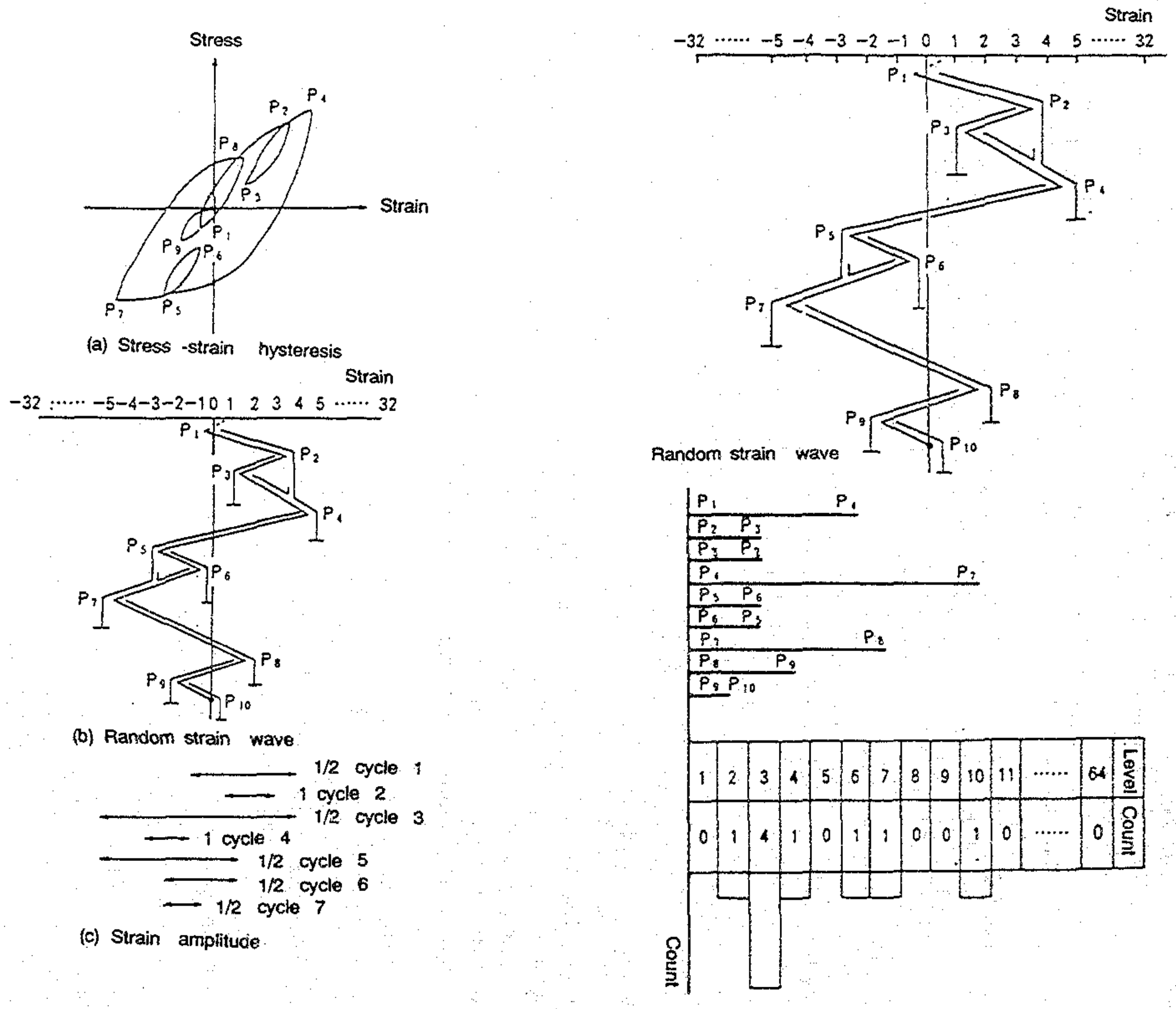
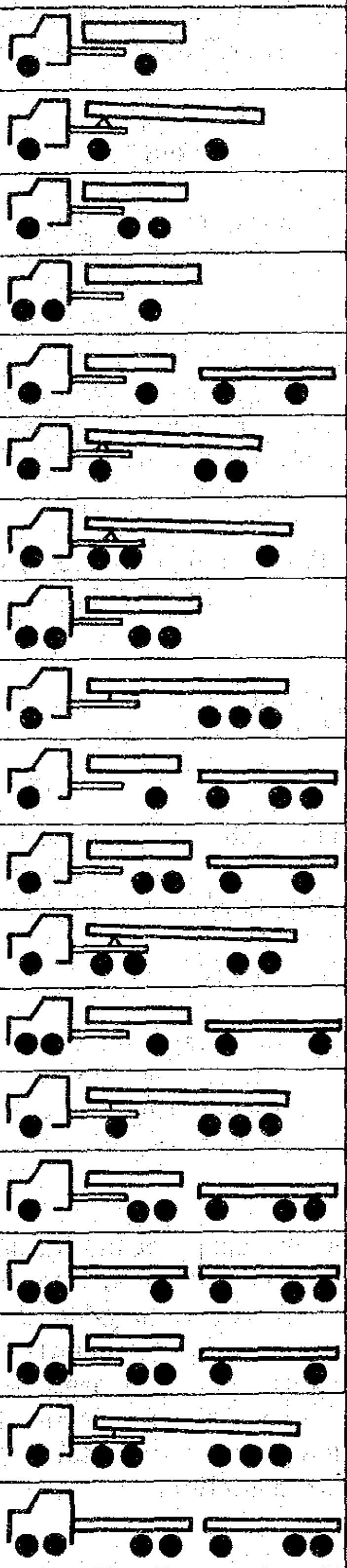


Figure 3- 42 Strain Range Count by the Rain Flow Method

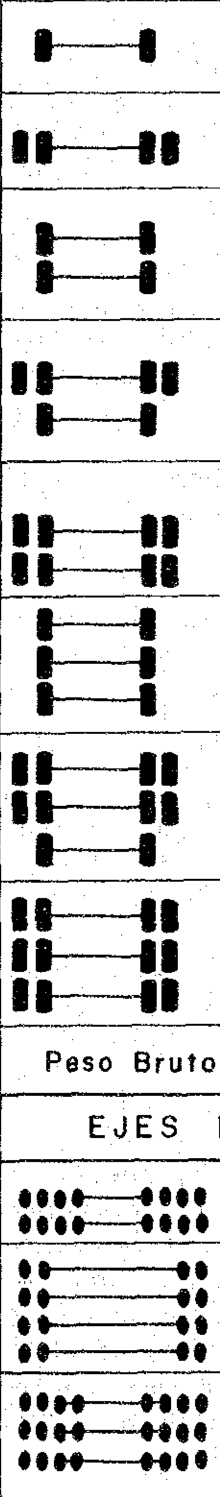
Table 3-16 Vehicle-Type Classification at the Weight Station

LIMITES DE PESO MAXIMO PERMITIDO POR EJE
EN CARRETERA SEGUN DEC. 158 DEL 29-Enero-80

TIPOS
20
30
31
34
40
41
42
45
47
51
52
53
54
57
63
65
66
69
77



EJES CONVENCIONALES			
EJES		LIMITES (ton)	TOLERANCIAS (kgs.)
	RS	7.0	500
	RD #	11.0	500
	2RS	14.0	600
	RD + RS	16.0	600
	2RD	18.0	700
	3RS	19.0	700
	2RD + RS	23.0	900
	3RD	25.0	900
Peso Bruto Total		45.0	1,000
EJES NO CONVENCIONALES			
	EDRC	22.0	0,8
	ECRD	29.0	1,0
	ETRC	30.0	1,0



3-2-4 Summary of Measurement Results

A. Static Load Test Results

Table 3- 17 shows the main girder stress. Table 3-18 shows the stress of the floor beam and stiffener. Figure 3-43 shows the stress distribution of the main girder cross-section. The following tendencies were noted as a result of testing.

1) Stress occurred mainly in the main girders. Significant stress did not occur on the upper and lower members of the floor beams, diagonal members, and vertical stiffeners.

2) Comparing the stress in the bottom flange of the main girder where the load is at each measuring point, the stress at gauges No. 1 and No. 2 at the cross-section change point was 187 kg/sq cm and 178 kg/sq cm. The stress near the general center span was 92 kg/sq cm; the stress at the cross-section change point was approximately 2 times greater.

3) Comparing of gauges No. 14 and No. 101 (lower flange), which were located symmetrically to the bridge axis, when stress was loaded on the No. 14 side, the stresses for No. 14 and No.101 were 92 kg/sq cm and 18 kg/sq cm, respectively. This means that a large load distribution on the main girder did not occur.

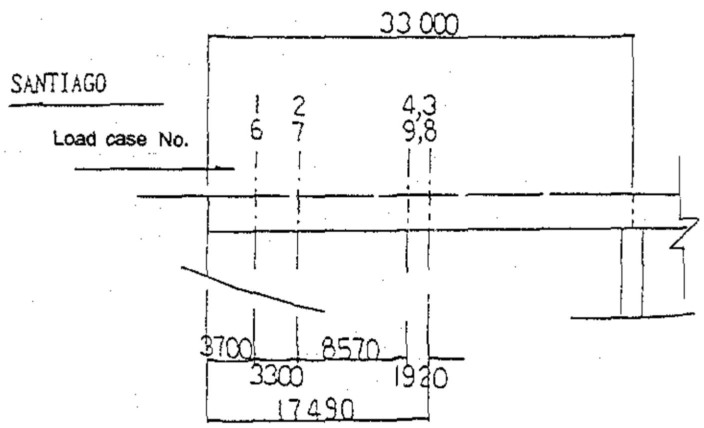
4) A relatively large amount of stress was recorded on gauge No. 3, which monitored the stress on the welding end of the vertical stiffener lower end. The maximum was 120 kg/sq cm.

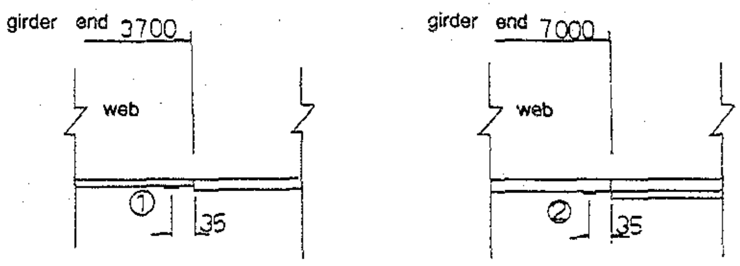
5) In regard to the floor beam member, 20 to 30kg/sq cm of stress was generated on the upper member; however, stresses on the lower member and diagonal member were almost nonexistent.

6) As Figure 3-43 shows, stress distribution on the main girder cross-section was not in a smooth triangular distribution; the neutral axis was near the upper flange and behaved as a composite girder. Comparing gauges No. 12 and No. 13 (web back surface), both showed different results based on the location of the load. It was understood that web transverse bending had occurred.

Figure 3-44 and Table 3-19 show the results of the strain measurements. The maximum value of strain was 3.0 mm for the main girder and 2.9 mm for the slab when the vehicle weight was 14.68 tons; and 5.9 mm for the main girder and 5.8 mm for the slab when the vehicle weight was 36.52 tons. The strain on the slab was relatively large.

Figure 3-45 shows the stress at the location examined and the shape of the influence line of strain. Due to the restrictions of the measuring method and the sensitivity of the instrumentation, the line's shape was not perfect. However, in general, the characteristics of the continuous span bridge from the influence line of main girder stress and strain were observed (positive on the first and third span, and negative on the second span). At the same time, based on the influence line of the lower members of the floor beams and the diagonal members, a tendency for a reversal of the positive and negative was observed near the monitored cross-section.





Note) Gauges 1 and 2 are 5mm from the welding end of cross -section change point.

Main girder stress (kg/cm²)

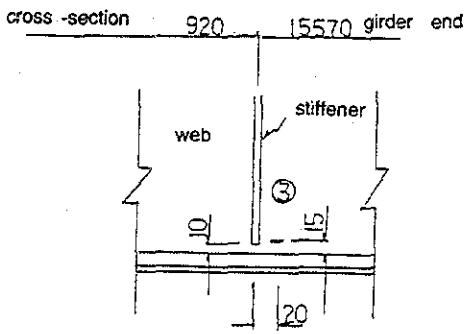
	Gauge No.	Vehicle load case No. 1	2	4	3	6	7	9	8
Lower flange	1	187 (331)	176 (328)	91 (144)	76 (118)	26 (45)	32 (53)	21 (0)	13 (-3)
	2	121 (244)	178 (338)	102 (179)	86 (142)	21 (63)	34 (86)	28 (50)	19 (42)
	14	28 (137)	52 (163)	92 (149)	91 (142)	5 (26)	11 (32)	21 (24)	18 (24)
	101	8 (50)	15 (58)	26 (61)	26 (55)	18 (136)	45 (168)	102 (186)	97 (175)
Web	3	37 (165)	73 (179)	112 (173)	99 (152)	8 (28)	18 (29)	29 (19)	24 (18)

Note) The figures within ( ) are based on the weight of the motor grader loaded on a trailer.

Vehicle weight motor grader only 14.68t
Motor grader + trailer 36.52t

Gauges 14 and 101 are on the general part lower flange near the span center. Gauge 14 is on the load side main girder and gauge 101 is on the opposite side of the main girder.

Monitoring floor beam cross -section



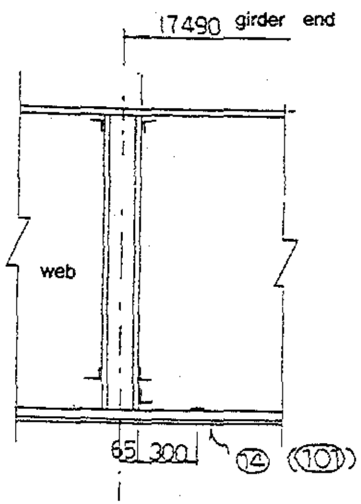
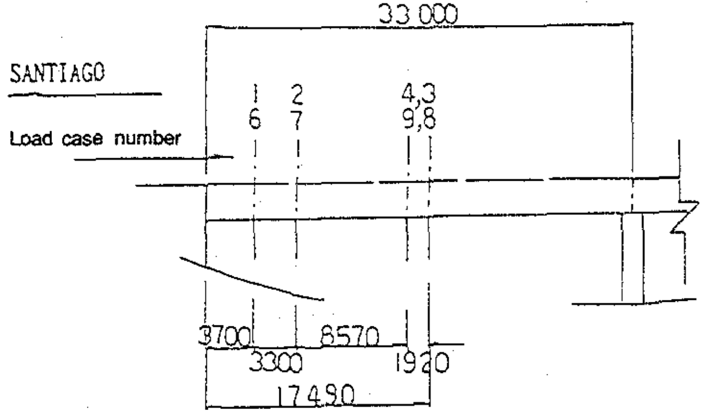


Table 3- 17 Main Girder Stress



Floor beam and stiffener stress (kg/cm²)

		Gauge number	Load case No. 3
Floor beam	Lower member	4	-5 (-11)
		5	11 (15)
		6	16 (24)
	Diagonal member	17	-5 (-13)
		18	5 (2)
		19	-2 (-5)
		23	-2 (-7)
	Upper member	20	28 (53)
		21	21 (42)
		22	29 (60)
Vertical stiffener		7	-16 (-39)

Note) The figures within ( ) are based on the weight of the motor grader loaded on a trailer.

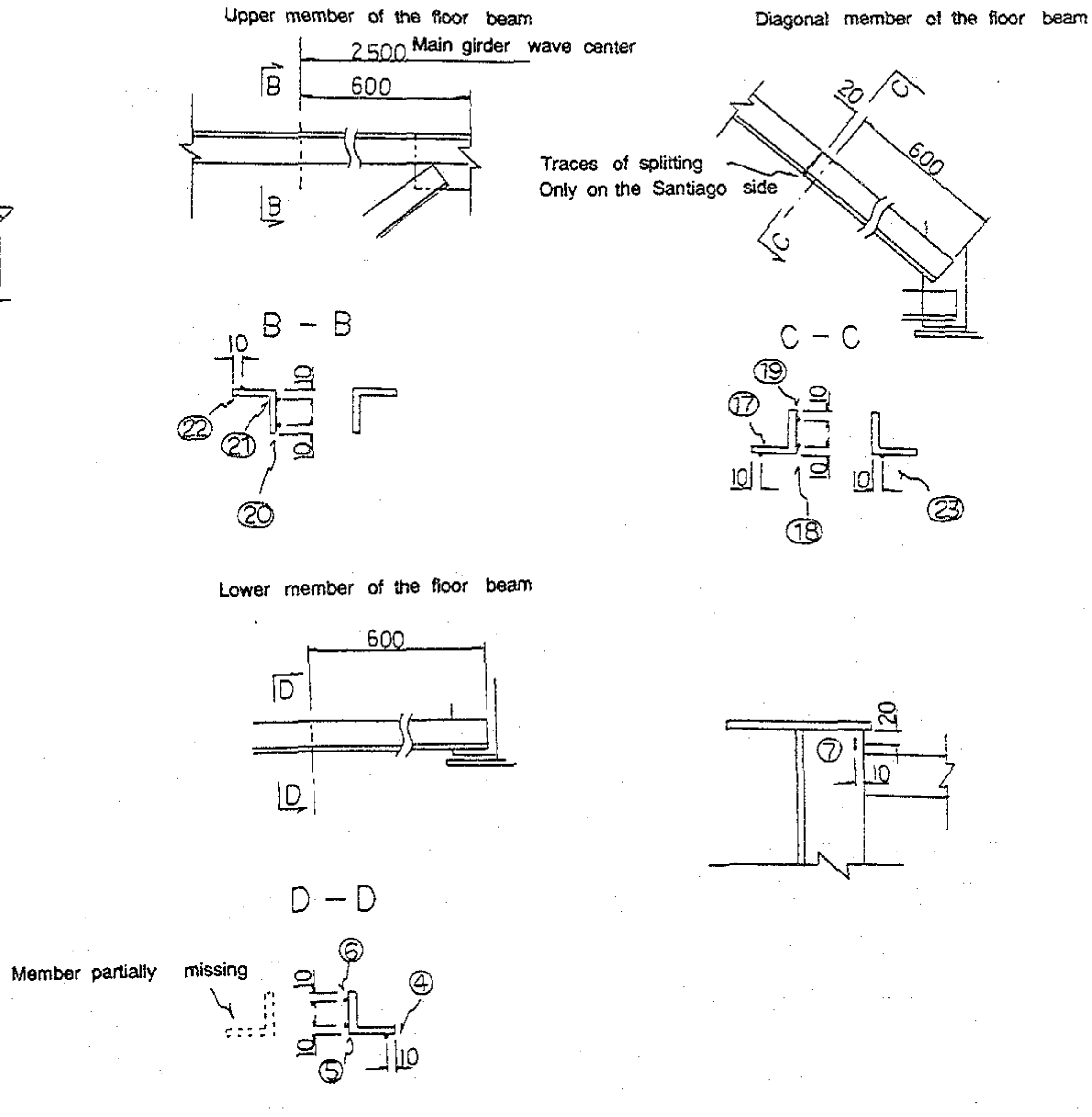
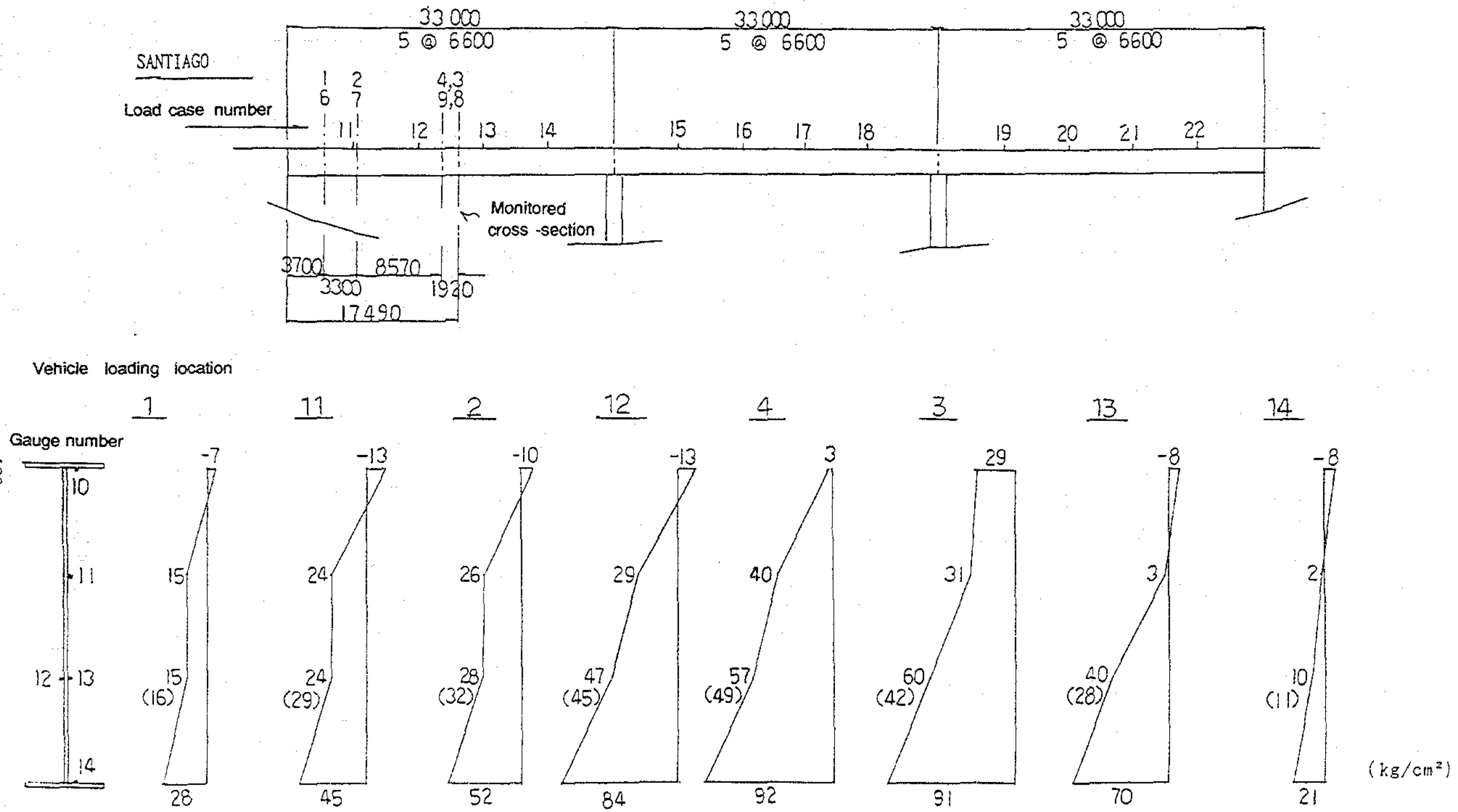


Table 3- 18 Floor Beam End Stiffener Stress



Note)
* The figures within ( ) represent the stress on the back-side.
* Positive stress is a pulling stress.

Figure 3- 43 Main Girder Stress Distribution Near the Span Center

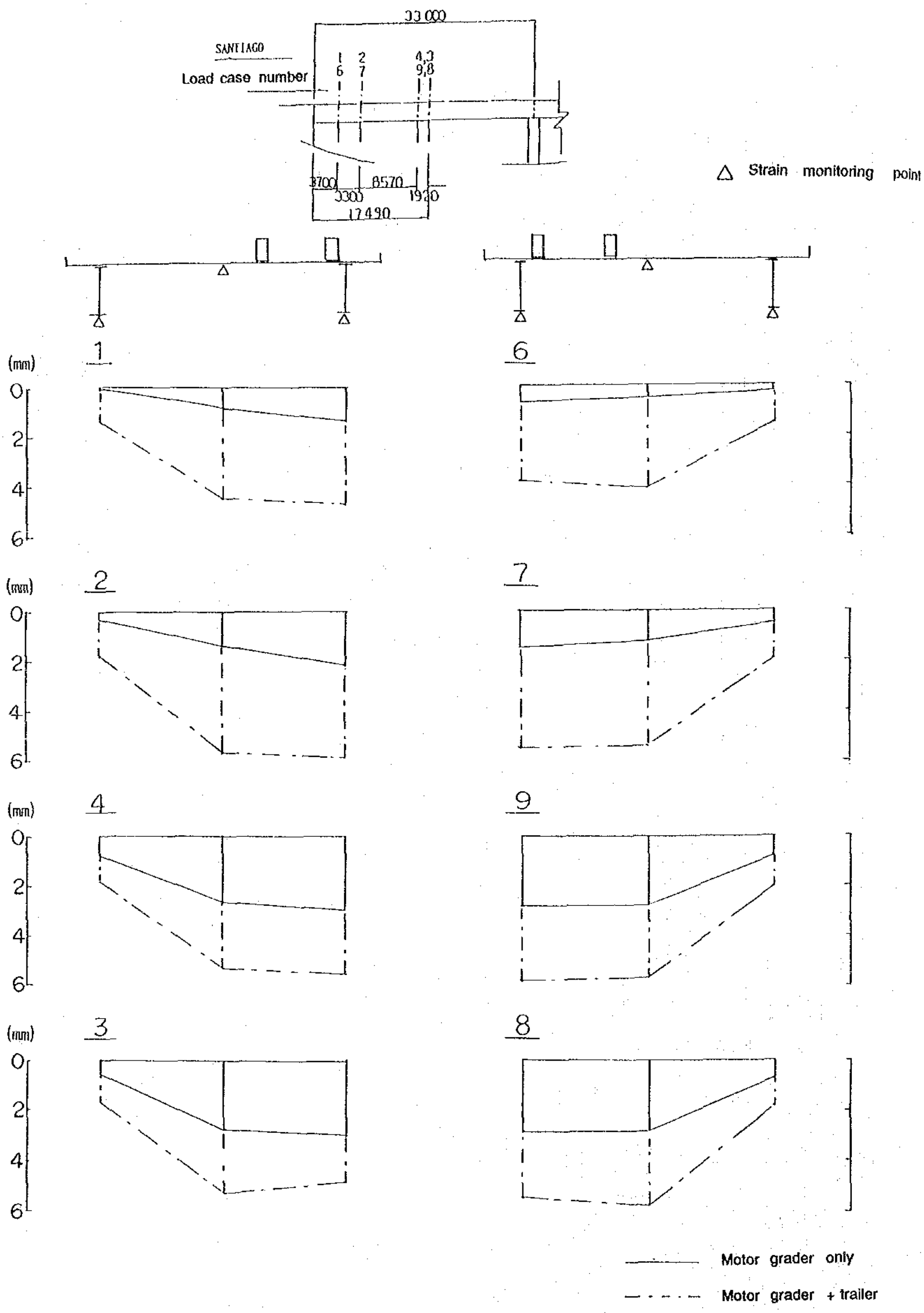


Figure 3-44 Displacement of the Main Girder and Slab

Table 3- 19 Displacement of the Main Girder and Slab based on the Static Load Test

Load case Nomber	Motor grader only			Motor grader + Trailer		
	Main girder A	Slab	Main girder B	Main girder A	Slab	Main girder A
1	0.10	0.80	1.30	1.40	4.50	4.70
2	0.30	1.40	2.15	1.75	5.65	5.80
3	0.60	2.75	3.00	1.74	5.35	4.95
4	0.74	2.65	2.95	1.89	5.36	5.55
6	0.66	0.50	0.23	3.80	4.10	1.50
7	1.50	1.18	0.44	5.50	5.42	1.98
8	2.86	2.85	0.65	5.43	5.75	1.83
9	2.78	2.78	0.75	5.85	5.65	2.00

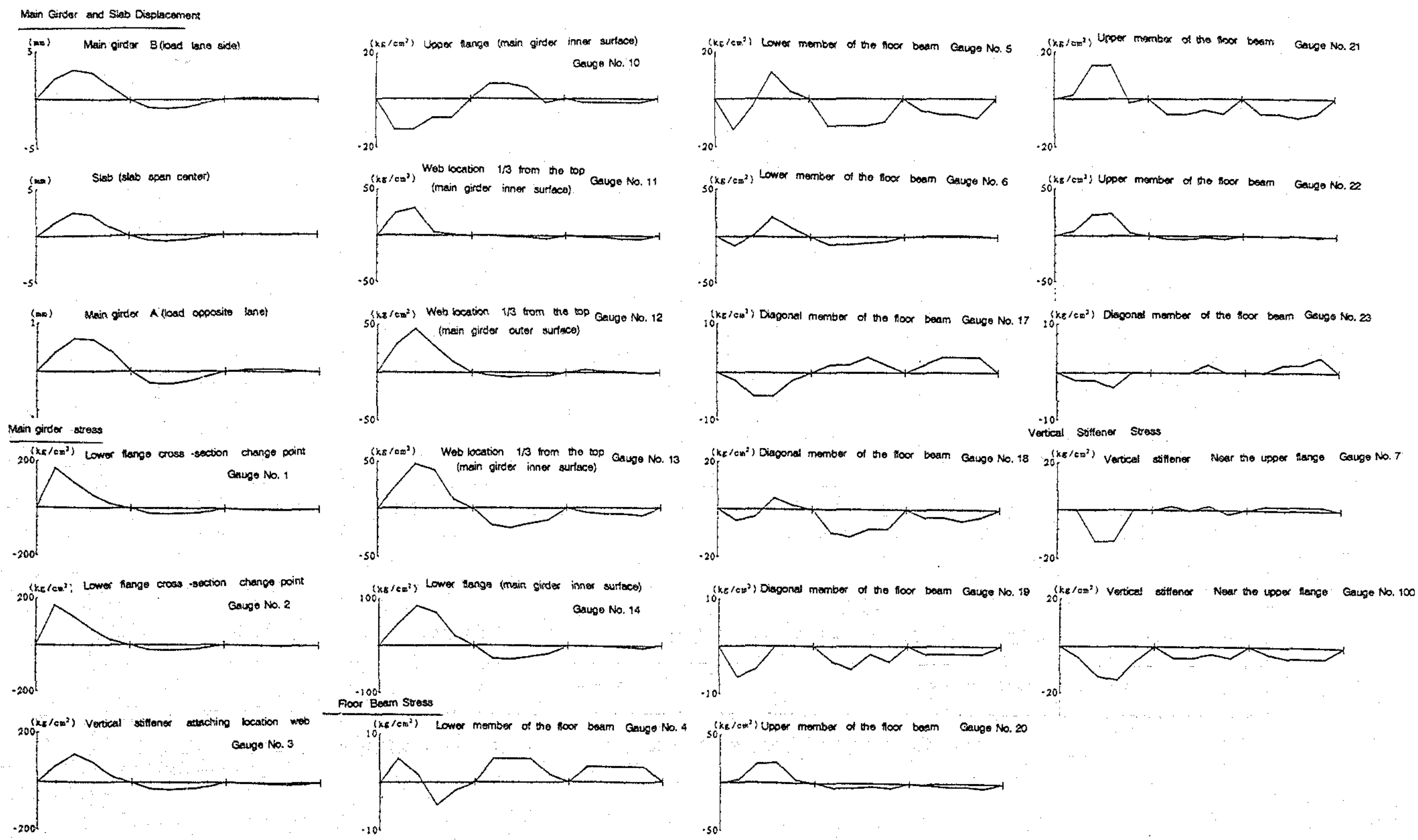


Figure 3- 45 Stress and Displacement Wave during Moving Static Load

B. Stress Frequency Measurement Results

(1) Fatigue Verification Method

A 24 hour stress frequency test was conducted using a histogram recorder while vehicles crossed the bridge. As stated earlier, fluctuation in stress amplitude was counted using the Rain Flow Method and figures were compiled by every 25.24 kg/sq cm. (The figures were not rounded-off because of restrictions in the settings for the instruments.) In the analysis the data was applied to the following formula and fatigue life was discussed.

$$D = \sum (ni / Ni) = 1$$

Fatigue damage (D), when a certain range of stress (∂ ri) interacts ni times, is given by the comparison (D=ni/Ni) with the fatigue life (Ni) of the joint when ∂ ri interacts. Stress damage is present when the load change occurs and the sum of each stress range becomes 1. N is then determined from S-N curve of the joint. The inverse amount of fatigue damage D shows the number of days until damage will occur in cases which assume the continuation of similar traffic conditions to when fatigue measurements were taken. This is also equivalent to the fatigue life. The S-N curve is given in many technical guides such as AASHTO (1983), BS5400 (1980), European Steel Structure Association (ECCS), and the Fatigue Design Guide (1985); however, we use the S-N curve shown in the Fatigue Design Guide Plan (November 1989) of the Japan Steel Structure Association. Figure 3-46 shows the S-N curve.

In this guide, the inclination of the S-N curve (m) of the joint which receives direct stress is 3. The strength grade is divided into 8 classes from A to H, according to the condition of the joint. The S-N curve shown in Figure 3-46 is the same under a certain level of stress. The stress range element less than this value is a limit value which does not contribute to fatigue damage. This is called the fatigue limit. Stress ranges less than this limit value are ignored when calculating fatigue life.

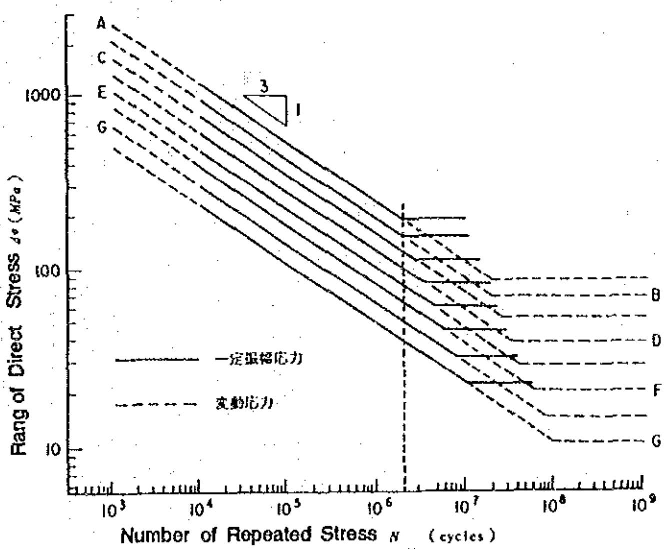


Figure 3-46 Fatigue Design Curves (S+N Curve)

Table 3- 20 shows 2 million times the basic allowable stress range, and Table 3- 21 shows the strength class for nominal stress of the monitored point. Figure 3- 48 shows the calculation procedure for fatigue life.

Table 3-20 Basic Allowable Stress Range

Class of Strength		Limit for Range of Stress (Mpa)	
Name	Basic Allowble Stress Range in 2 mil.Time	Amplitucle Stress	Fluctueive Stress
A	190	190 (2.0 x 10^6)	88 (2.0 x 10^7)
B	155	155 (2.0 x 10^6)	72 (2.0 x 10^7)
C	125	115 (2.6 x 10^6)	53 (2.6 x 10^7)
D	100	84 (3.4 x 10^6)	39 (3.4 x 10^7)
E	80	62 (4.4 x 10^6)	29 (4.4 x 10^7)
F	65	46 (5.6 x 10^6)	21 (5.6 x 10^7)
G	50	32 (7.7 x 10^6)	15 (7.7 x 10^7)
H	40	23 (1.0 x 10^7)	11 (1.0 x 10^8)

Table 3-21 Strength Class of the Monitored Point

Monitored location	Gauge number	Strength class	Remarks
Lower flange cross-section change point	1	E, F	*)
Lower flange cross-section change point	2	E, F	*)
Web on the end of the welding part of the lower vertical stiffener	3	E	3 of (d)
Lower member of the floor beam attached part	5	H	4 (2) of (f)
Near the upper flange on the vertical stiffener upper part	7	E	6 (3) of (d)
Near the lower flange span center	14	D	3 of (c)
Upper member of the floor beam attached part	21	H	6 of (e)
Upper flange and web welding part	102	E	3 of (d)

Note) The basic allowable stress range (2 million times stress)

C → 1275 kg/cm2 , D → 1020 kg/cm2 , E → 816 kg/cm2 ,

F → 663 kg/cm2 , G → 510 kg/cm2 , H → 408 kg/cm2

- The chart shows the strength class for nominal stress of the monitored point. When measured stress includes increased stress (hotspot stress) caused by the joint shape, the E-class is used. (When the end of welding is finished, the D-class is used.)
- *)→Gauges No. 1 and No. 2 are located on the butt welding part of the lower flange, however, the classifications shown in Table - does not have an identical item to this structure. As to the level of the joint, (3) of (b) was judged to be closest to this structure, and the strength class was predicted to be E or F.

<hotspot stress>

Hotspot stress is defined as stress applied to the monitored part (end of welding) by considering the structural stress turbulence. It does not include local stress concentrations due to welding.

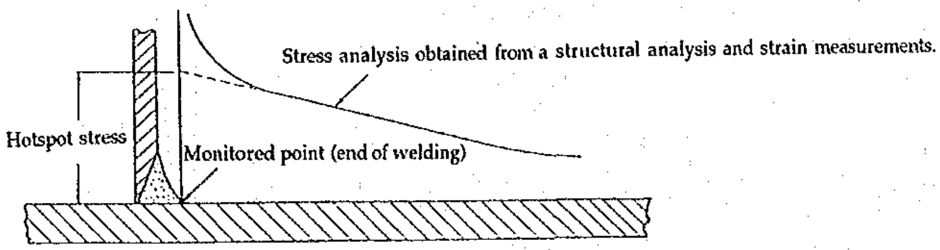


Figure 3-47 Hotspot Stress

Determination of strength class

↓

Calculation of Ni

$N_i=(\sigma_f{}^m \cdot 2.0 \times 10^6)/\sigma_{ri}{}^m$

Ni : corresponding N on the S"N curve ..

σ_f : 2 million times the basic allowable stress range

σ_{ri} : each stress range level

m : index to show the grade of the S"N curve

↓

$D= \Sigma n_i/N_i$

D: fatigue damage degree during the observation period

ni : frequency of observed ..

FL: fatigue life

↓

FL = 1/D

Figure 3- 48 Fatigue Life Calculation Procedure

(2) Stress Frequency Distribution and Fatigue Life Prediction Results

Table 3-23 shows the stress frequency distribution and Table 3-22 shows the calculation results of fatigue life. These charts show the prediction results of all class from C to H. The results take into consideration that the joint classifications shown in the design guidelines and the joint conditions of the monitored points are not identical, and the fact that some assumptions are included in the predictions.

The following points were determined based on the fatigue lives predictions.

1. Fatigue life of the main girder cross-section change point is 1 to 2 years. Severe stress for fatigue is occurring.

2. Life of the end of welding of the vertical stiffener is approximately 10 years and it is under severe stress conditions.

3. In regard to members of the floor beams, life of the welding of upper member gazette is short. However, a short life is not predicted for other points.

Table 3-22 Fatigue Life Prediction Results

(unit: year)

Monitored location	Gauge number	Strength class					
		C	D	E	F	G	H
Lower flange cross-section change plate	1	21.5	6.4	* 1.8	* 0.7	0.2	0.1
Lower flange cross-section change plate	2	25.8	7.2	* 1.9	* 0.7	0.2	0.1
Web on the end of the welding part of the lower vertical stiffener	3	219.3	37.4	* 9.4	2.2	0.7	0.3
Lower member of the floor beam attached part	5	∞	∞	∞	∞	∞	* 59.6
Near the upper flange of the vertical stiffener upper part	7	∞	∞	* ∞	∞	233.6	88.9
Near the lower flange span center	14	∞	* 89.5	27.0	5.8	1.4	0.5
Upper member of the lower beam attached part	21	∞	∞	∞	∞	270.3	* 10.9
Upper flange and web welded part	102	∞	∞	* ∞	∞	∞	∞

Note)
- When fatigue life is greater that 274 days (more than 100,000 hours) life is showed as.
- The figure marked by an asterisk is the fatigue life and this corresponds to the stress class shown in Chart - .

Table 3-23 Stress Frequency Distribution

NIVEL DE TENSIONES	RANGO DE TENSIONES (kg/cm²)	応力頻度							
		Platabanda Inferior (No.1)	Platabanda Inferior (No.2)	Atiesador, parte inferior (No.3)	Riostra Inferior (No.5)	Atiesador, parte superior (No.7)	Platabanda inferior (No.14)	Riostra superior (No.21)	Platabanda superior (No.102)
1	25.24	4	0	0	0	0	0	0	0
2	50.48	14971	14118	9416	14330	0	7091	912	571
3	75.72	12407	11961	10169	6208	0	8212	1488	409
4	100.96	6292	6175	5071	735	2	3913	796	133
5	126.20	3730	3730	2799	76	4	1995	398	10
6	151.44	2282	2207	1549	12	8	1261	71	0
7	176.68	1553	1593	1165	1	7	756	12	2
8	201.93	1128	1071	704	1	4	433	0	0
9	227.17	776	790	505	0	0	230	2	0
10	252.41	560	574	258	0	2	112	0	0
11	277.65	378	406	170	0	0	50	0	0
12	302.89	268	281	87	0	0	26	0	0
13	328.13	177	195	45	0	0	10	0	0
14	353.37	123	125	26	0	0	10	0	0
15	378.61	94	105	4	0	0	0	0	0
16	403.85	66	45	8	0	0	10	0	0
17	429.09	31	33	8	0	0	2	0	0
18	454.33	20	20	2	0	0	8	0	0
19	479.57	8	19	6	0	0	0	0	0
20	504.81	18	10	2	0	0	1	0	0
21	530.05	6	9	8	0	0	2	0	0
22	555.29	2	4	2	0	0	1	0	0
23	580.53	8	3	0	0	0	0	0	0
24	605.77	0	4	0	0	0	0	0	0
25	631.02	4	6	0	0	0	0	0	0
26	656.26	8	2	0	0	0	0	0	0
27	681.50	2	4	3	0	0	0	0	0
28	706.74	4	6	0	0	0	0	0	0
29	731.98	4	2	0	0	0	0	0	0
30	757.22	4	0	1	0	0	0	0	0
31	782.46	3	4	0	0	0	0	0	0
32	807.70	0	0	0	0	0	0	0	0
33	832.94	3	2	0	0	0	0	0	0
34	858.18	2	0	0	0	0	0	0	0
35	883.42	0	0	0	0	0	0	0	0
36	908.66	0	3	0	0	0	0	0	0
37	933.90	0	0	0	0	0	0	0	0
38	959.14	0	0	0	0	0	0	0	0
39	984.38	0	1	0	0	0	0	0	0
40	1009.63	0	0	0	0	0	0	0	0
41	1034.87	1	0	0	0	0	0	0	0
42	1060.11	0	0	0	0	0	0	0	0
43	1085.35	0	0	0	0	0	0	0	0
44	1110.59	0	0	0	0	0	0	0	0
45	1135.83	0	0	0	0	0	0	0	0
46	1161.07	1	0	0	0	0	0	0	0
47	1186.31	0	0	0	0	0	0	0	0
48	1211.55	0	0	0	0	0	0	0	0
49	1236.79	0	0	0	0	0	0	0	0
50	1262.03	0	0	0	0	0	0	0	0
51	1287.27	0	0	0	0	0	0	0	0
52	1312.51	0	0	0	0	0	0	0	0
53	1337.75	0	0	0	0	0	0	0	0
54	1362.99	0	0	0	0	0	0	0	0
55	1388.23	0	0	0	0	0	0	0	0
56	1413.47	0	0	0	0	0	0	0	0
57	1438.72	0	0	0	0	0	0	0	0
58	1463.96	0	0	0	0	0	0	0	0
59	1489.20	0	0	0	0	0	0	0	0
60	1514.44	0	0	0	0	0	0	0	0
61	1539.68	0	0	0	0	0	0	0	0
62	1564.92	0	0	0	0	0	0	0	0
63	1590.16	0	0	0	0	0	0	0	0
64	1615.40	0	0	0	0	0	0	0	0
Total Count:		44936	43601	32038	21363	27	24123	3679	1125

C. *Traffic Volume Survey Results*

Table 3- 24 shows the change in traffic volume over an 8 year period (1980 to 1988). The traffic volume survey in Chile was performed by classifying vehicles into the following 7 types in 1988. Prior to 1988, vehicle classification was not unified, for example, classifying carts pulled by animals and not distinguishing different types of trucks and trailers.

Vehicle Classification

1. Passenger car
2. Small truck
3. Twin axle truck
4. Truck with more than 3 axles
5. Semi -trailer
6. Trailer
7. Bus

The vehicles listed above from 3 to 7 are defined as large vehicles and are calculated as a mixed ratio of large vehicles. The following tendencies were observed in regard to traffic volume (See Table 3-24).

(1) Significant secular change was not observed over the 8 year period and the total traffic volume was roughly 10,000.

(2) The mixed ratio of large vehicles was large at 40% to 50%.

D. *Vehicle Weight Survey Result*

Axle load measurement data (weight station) was collected over an 8 hour period on the same day that the stress frequency measurements were taken. Table 3-25 shows the results.

Table 3-24 Weight Distribution of Vehicles Passing through the Weight Station

Weight (t)	No. of Vehicles
0 ~ 5	21
5 ~ 10	150
10 ~ 15	57
15 ~ 20	60
20 ~ 25	55
25 ~ 30	5
Total	348

Note: Maximum Figure 25.9 t

Table 3-25 Change in Traffic Volume over an Eight Year Period

		1980	1984	1986	1988
1	Autos Stations	5,020	5,286	4,334	3,454
2	Camionetas		2,318	1,294	1,329.
3	Camioness simples de 2 ejes	1,112	1,716	1,485	1,162
4	Camioness simple de mas de 2 ejes				602
5	Semi remolques	940	1,448	1,980	967
6	Remolques				1,407
7	Buses Taxibuses	723	1,189	1,132	1,253
8	Varios traction animal	188	500	279	-
24 hour total		7,983	12,457	10,504	10,174
Number of large vehicles (total of classifications 3 to 7)		2,775	4,353	4,597	5,391
Mixed ratio of large vehicles (%)		34.8	34.9	43.8	53.0

E. Welding Conditions of the Monitored Part

Figure 3-49 shows a sketch of the monitored parts. The following points were confirmed through observation of the monitored parts.

1) The No. 1 main girder cross -section change point did not have a stress grade.

2) The No. 2 main girder cross-section change point increased the flange cross-section by butt welding and a welded cover plate. The front of the cover plate was not welded.

3) The No. 7 and No. 100 stiffeners were welded with the upper flange, however, there was space between the stiffener and the upper flange.

4) Near No. 3 the space between the stiffener and lower flange was approximately 10mm. The welding line of the stiffener and web, as well as, the welding line of the web and flange were overlapping.

5) Near No. 23 the diagonal member which was connected by simple welding exhibited traces of splits, and the axes of both members were not matched.

6) On No. 103 the upper flange was not attached to the web at a right angle resulting in a space between the upper flange and the slab.

Numbers 1 to 4 were factors in influencing the level of stress and No. 3 and No. 6 provide explanations as to why significant stress was not found at No. 7, No. 100, and No. 103.

3-2-5 Summary and Recommendation

Replace of the PUECO bridge is recommendable by the following reason;

1) Fatigue life is less than two years at the cross-section change point of lower flange as shown in Table 3-22.
2) Welding at the floor beam is cut several times.
3) The main girder is broken and repaired by weldment at the side.
4) There are some problems for the design concept of two main girder bridge under construction

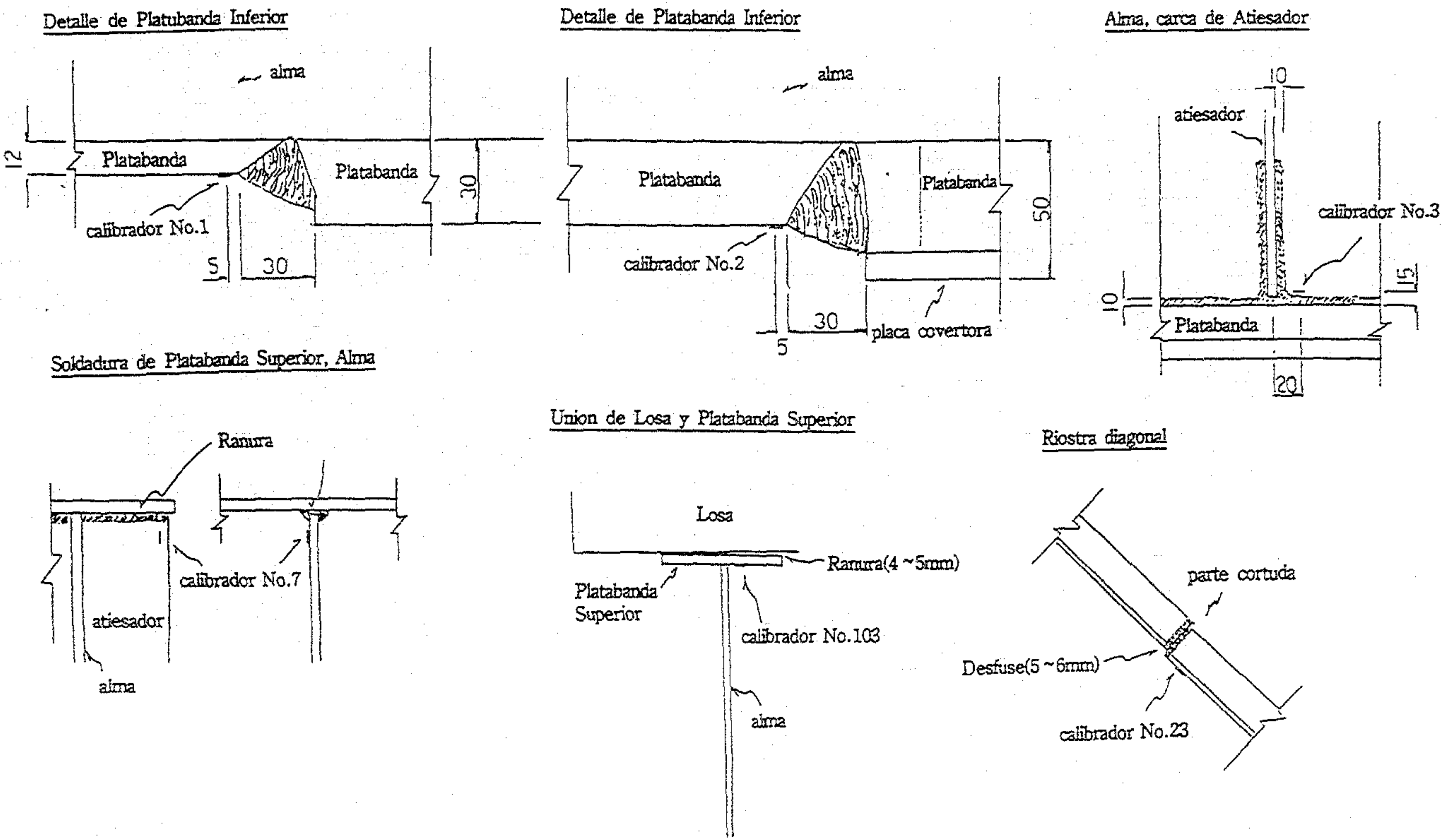


Figure 3-49 Monitoring Point Sketch

Table 3- 26 Member Stress Obtained from Static Test (1) (motor grader loaded: vehicle weight 14.68t)

(kg/cm2)

Gague number	Location	Vehicle stop location							
		①	②	③	④	⑥	⑦	⑧	⑨
1	Lower flange cross-section change point	187.4	176.1	75.9	90.5	25.8	32.3	12.9	21.0
2	Lower flange cross-section change point	121.2	177.7	85.6	101.8	21.0	33.9	19.4	27.5
3	Web on the vertical stiffener attached location	37.2	72.7	98.5	111.5	8.1	17.8	24.2	29.1
4	Lower member of the floor beam	3.2	4.8	-4.8	-1.6	0.0	1.6	-1.6	-1.6
5	Lower member of the floor beam	-6.5	-8.1	11.3	3.2	-3.2	-3.2	1.6	0.0
6	Lower member of the floor beam	-8.1	-8.1	16.2	8.1	-3.2	0.0	6.5	4.8
7	Near the upper flange on the vertical stiffener	0.0	0.0	-16.2	-14.5	0.0	-3.2	-17.8	-16.2
10	Upper flange (main girder inner surface)	-6.5	-9.7	29.1	3.2	-4.8	-1.6	11.3	8.1
11	Location 1/3 from the top of the web (main girder inner surface)	14.5	25.8	30.7	40.4	3.2	8.1	8.1	11.3
12	Location 1/3 from the top of the web (main girder outer surface)	16.2	32.3	42.0	48.5	4.8	11.3	12.9	16.2
13	Location 1/3 from the top of the web (main girder inner surface)	14.5	27.5	59.8	56.6	3.2	6.5	12.9	14.5
14	Lower flange (main girder inner surface)	27.5	51.7	90.5	92.1	4.8	11.3	17.8	21.0
17	Diagonal member of the floor beam	-1.6	-1.6	-4.8	-6.5	0.0	0.0	-4.8	-4.8
18	Diagonal member of the floor beam	-1.6	-3.2	4.8	1.6	-1.6	-3.2	-8.1	-8.1
19	Diagonal member of the floor beam	-3.2	-6.5	-1.6	-3.2	-1.6	-1.6	-4.8	-4.8
20	Upper member of the floor beam	1.6	4.8	27.5	25.8	3.2	6.5	22.6	21.0
21	Upper member of the floor beam	1.6	4.8	21.0	19.4	0.0	3.2	19.4	17.8
22	Upper member of the floor beam	1.6	6.5	29.0	27.5	3.2	6.5	27.5	27.5
23	Diagonal member of the floor beam	0.0	0.0	-1.6	-1.6	0.0	0.0	-3.2	-1.6
100	Near the upper flange of the vertical stiffener	0.0	-3.2	-14.5	-12.9	-1.6	-1.6	-17.8	-12.9
101	Lower flange (main girder inner surface)	8.1	14.5	25.8	25.8	17.8	45.2	96.9	101.8
102	Upper flange right angle direction (main girder inner surface)	1.6	4.8	-14.5	0.0	0.0	0.0	-3.2	-3.2
103	Upper flange right angle direction (main girder inner surface)	0.0	1.6	-1.6	-1.6	1.6	6.5	8.1	14.5

Table 3- 27 Member Stress Obtained from Static Test (2) (motor grader loaded + trailer: vehicle weight 36.52t)

(kg/cm2)

Gague number	Location	Vehicle stop location							
		①	②	③	④	⑥	⑦	⑧	⑨
1	Lower flange cross-section change point	331.2	327.9	117.9	143.8	45.2	53.3	-3.2	0.0
2	Lower flange cross-section change point	243.9	337.6	142.2	179.3	63.0	85.6	42.0	50.1
3	Web on the vertical stiffener attached location	164.8	179.3	151.8	172.8	27.5	29.1	17.8	19.4
4	Lower member of the floor beam	-8.1	-11.3	-11.3	-8.1	-9.7	-9.7	-8.1	-8.1
5	Lower member of the floor beam	-4.8	6.5	14.5	9.7	0.0	3.2	3.2	1.6
6	Lower member of the floor beam	1.6	14.5	24.2	19.4	6.5	11.3	12.9	11.3
7	Near the upper flange on the vertical stiffener	-42.0	-43.6	-38.8	-35.5	-33.9	-37.2	-37.2	-33.9
10	Upper flange (main girder inner surface)	-11.3	46.8	43.6	12.9	9.7	19.4	25.8	11.3
11	Location 1/3 from the top of the web (main girder inner surface)	45.2	53.3	53.3	59.8	16.2	17.8	14.5	17.8
12	Location 1/3 from the top of the web (main girder outer surface)	63.0	72.7	64.6	74.3	19.4	22.6	21.0	24.2
13	Location 1/3 from the top of the web (main girder inner surface)	77.5	101.8	93.7	90.5	22.6	30.7	30.7	32.3
14	Lower flange (main girder inner surface)	137.3	163.2	142.2	148.6	25.8	32.3	24.2	24.2
17	Diagonal member of the floor beam	-11.3	-12.9	-12.9	-12.9	-8.1	-8.1	-8.1	-6.5
18	Diagonal member of the floor beam	-4.8	1.6	1.6	1.6	-11.3	-11.3	-11.3	-12.9
19	Diagonal member of the floor beam	-8.1	-6.5	-4.8	-6.5	-6.5	-4.8	-4.8	-6.5
20	Upper member of the floor beam	43.6	53.3	53.3	48.5	35.5	45.2	45.2	38.8
21	Upper member of the floor beam	37.2	43.6	42.0	38.8	32.3	38.8	37.2	33.9
22	Upper member of the floor beam	50.1	59.8	59.8	54.9	45.2	54.9	54.9	50.1
23	Diagonal member of the floor beam	-4.8	-6.5	-6.5	-6.5	-4.8	-1.6	-3.2	-3.2
100	Near the upper flange of the vertical stiffener	-21.0	-25.8	-27.5	-24.2	-17.8	-25.8	-24.2	-17.8
101	Lower flange (main girder inner surface)	50.1	58.2	54.9	61.4	135.7	168.0	174.5	185.8
102	Upper flange right angle direction (main girder inner surface)	17.8	-11.3	-14.5	-1.6	1.6	1.6	-1.6	0.0
103	Upper flange right angle direction (main girder inner surface)	8.1	8.1	6.5	8.1	25.8	30.7	32.3	30.7

Table 3- 28 Member Stress during Static Load Test
(motor grader loaded: vehicle weight 14.68t)

Load case No.		Gauge number																		
		1	2	3	4	5	6	7	10	11	12	13	14	17	18	19	20	21	22	23	100
Side span	11	171.2	171.2	64.6	4.8	-12.9	-9.7	0	-12.9	24.2	29.1	24.2	45.2	-1.6	-4.8	-6.5	3.2	1.6	4.8	-1.6	-4.8
	12	109.8	122.8	113.1	1.6	-3.2	1.6	-12.9	-12.9	29.1	45.2	46.8	46.8	-4.8	-3.2	-4.8	19.4	14.5	22.6	-1.6	-12.9
	13	54.9	66.2	79.2	-4.8	11.3	21.0	-12.9	-8.1	3.2	27.5	40.4	40.4	-4.8	4.8	0	21.0	14.5	24.2	-3.2	-14.5
	14	16.2	21.0	22.6	-1.6	3.2	9.7	0	-8.1	1.6	11.3	9.7	9.7	-1.6	1.6	0	3.2	-1.6	3.2	0	-6.5
Center span	15	-22.6	-21.0	-27.5	4.8	-11.3	-8.1	1.6	6.5	0	-3.2	-17.8	-17.8	1.6	-9.7	-3.2	-4.8	-6.5	-3.2	0	-4.8
	16	-24.2	-22.6	-30.7	4.8	-11.3	-8.1	0	6.5	-1.6	-4.8	-21.0	-21.0	1.6	-11.3	-4.8	-4.8	-6.5	-3.2	0	-4.8
	17	-21.0	-19.4	-25.8	4.8	-11.3	-6.5	1.6	4.8	-1.6	-3.2	-16.2	-16.2	3.2	-8.1	-1.6	-3.2	-4.8	-1.6	1.6	-3.2
	18	-16.2	-12.9	-17.8	1.6	-9.7	-4.8	-1.6	-1.6	-3.2	-3.2	-12.9	-12.9	1.6	-8.1	-3.2	-4.8	-6.5	-3.2	0	-4.8
Side span	19	-4.8	0	-3.2	3.2	-4.8	1.6	1.6	-1.6	-1.6	3.2	-4.8	-4.8	1.6	-3.2	-1.6	-1.6	-6.5	0	0	-3.2
	20	-4.8	0	-4.8	3.2	-6.5	1.6	1.6	-1.6	-1.6	1.6	-6.5	-6.5	3.2	-3.2	-1.6	-3.2	-6.5	0	1.6	-4.8
	21	-4.8	-1.6	-6.5	3.2	-6.5	1.6	1.6	-1.6	-3.2	1.6	-6.5	-6.5	3.2	-4.8	-1.6	-3.2	-8.1	0	1.6	-4.8
	22	-4.8	-1.6	-8.1	3.2	-8.1	1.6	1.6	-1.6	-3.2	0	-8.1	-8.1	3.2	-3.2	-1.6	-4.8	-6.5	-1.6	3.2	-4.8

Table 3-29 Displacement of Main Girder and Slab during Static load Test (motor grade loaded: vehicle weight 14.68t)

(mm)

Load case No.	Main girder A	Slab	Main girder B
11	0.40	1.35	2.10
12	0.67	2.45	3.05
13	0.65	2.20	2.75
14	0.40	0.85	1.30
15	-0.24	-0.54	-0.84
16	-0.26	-0.65	-0.95
17	-0.20	-0.50	-0.78
18	-0.10	-0.27	-0.35
19	0.04	0.05	0.20
20	0.05	0.06	0.20
21	0.05	0.06	0.20
22	0.02	0.02	0.14

Chapter 4 Bridge Repair Design and Cost Estimation

4-1 Introduction

This chapter discusses the rehabilitation design, rehabilitation work planning and the cost estimation of those bridges for which the detailed inspection was carried out. One of the objectives of this chapter was intended to present the typical rehabilitation methods to be adopted in Chile. These methods were to be worked out through classifications by structural type and by structural component where defects were identified. However, because many of the inspected bridges were found to have comparatively severe defects, the proposed rehabilitation methods are inclined to be for the bridges with high deficiencies. However, the proposed methods did contribute to justify the existing rehabilitation methods of the Ministry of Public Works.

The river bridges in Chile normally have comparatively short bridge length in relation to the river width, and many of them are found to have insufficient bridge opening. Also, some of the revetment works around the bridges are found to be unstable. Since the Study is intended for rehabilitation of the existing bridges, changes of bridge length and span distribution are not included in the plan. In this connection, although proper rehabilitation is implemented to the bridge itself and around the pier, other problems are anticipated to occur in the future by the unexpected influence from the river. Also, there may be the case for bridge replacement other than rehabilitation. In any case, river conditions will greatly affect the design criteria of bridges such as span distribution and selection of structural type, etc., and therefore, they are to be carefully considered even in the case of rehabilitation. In the following sections, important items to be considered for bridge planning and design are discussed and described.

(1) Points to be Considered in Designing Bridges over Rivers

As rivers are major obstacles to road traffic, one of the important targets in bridge engineering is to construct low cost bridges solid enough to survive floods. In thwarting river discharge, that is, in resisting flood flow, bridges are subject to the danger of collapse. In some cases it might bring about inundation, resulting in great flood damages around the bridges. The surest way to avoid these dangers is either to construct bridges which do not resist the flood flow at all or those which are solid enough to resist any flood. However, it is impossible to construct such bridges because of the high costs.

From the peculiarity of the land utilization in Chile, it is unlikely that bridge construction will cause flood damages to the surrounding areas unless they are carried out in urban areas. Therefore, only the safety of bridges against floods will be considered, and a guideline in designing hydraulic structures of bridges on the National Highway No. 5, which crosses rivers flowing through the alluvial plain, will be described. In cases that it is necessary to reduce the flood damages which the bridge might give to the surroundings, further measures are necessary in addition to this guideline. The guideline is made so that the bridges may have the minimum resistance to flood flow.

Hydraulic situations of rivers depend much on the past experiences, and cannot be determined so much accurately as design loads of bridge substructures.

Accordingly, a design guideline described here will be qualitative, as a quantitative guideline is hard to specify. When it is necessary to design or construct a bridge, the cost of the maintenance and management should be considered at the design stage from a long viewpoint. Stream flows are affected not only by the conditions at a bridge but also by the situations upstream and downstream of the bridge and also by the long passage of time. Therefore, it is always necessary carry out river the improvement as a continuing function

The guideline is described in the following 8 items.

1) Selection of the bridge location
2) Direction of the bridge
3) Design flood discharge
4) Location of abutments
5) Piers
6) Span
7) Under clearance
8) Revetments

1) Selection of a bridge location

1) The river should be narrow.
2) The river should be flowing straight. (It should not be crooked. There should be no change in the upstream and downstream sections.)
3) The flow passage section has not changed over the years.
4) The spot where the stream flow concentrates should be avoided.
5) There should be no drastic change in the longitudinal slope.
6) The place where the river diverges or joins another river should not be close.

2) Direction of the bridge

1) The river and the bridge should cross at right angles.
2) The angle of skew should be within 60∞.

3) Design flood discharge

1) The design flood discharge should be determined taking account of the flow passage section and the revetments.
2) It should be the largest recorded flood discharge or the 50-year probable flood discharge.
3) According to the River Division of the Road Bureau, the design flood discharges at the bridges on the National Highway No. 5 must be the 200-year probable flood discharges. However, from our site survey at a newly-built bridge at Maule, it was learned that the trap of the bridge had narrowed the river, and it was judged that the bridge there was not designed based on the 200-year probable flood discharge. Actually, even in Japan, bridges designed based on the 200-year probable flood discharges are limited to those over large rivers around Tokyo and Osaka.
4) Design flood discharge should take the scour into account. The flood discharge should be around the scale of the 1-year probable flood discharge.

4) Location of abutments

The place would be appropriate where the largest recorded flood or the 50-year probable flood flows naturally. In cases that it is difficult to construct at such a place, revetments should be designed taking account of repairing costs over a long time frame.

5) Piers (within the river channel)

1) The form should be a long and narrow oval and the direction of the longer side shall be the same with that of the flood flow.
2) The form should be circular at the places where the direction of the flood flow is not constant.
3) The use of pile bent piers should be prohibited, as they are likely to cause vortex flows and subsequent extraordinary scours in the vicinity.
4) The ratio of the total width of the piers to the width of the river should be determined so as not to bring about the conspicuous rise in water level. (8% at the maximum)
5) The embedment depth should be the longer length between the two: 2 m or 80% of the water depth of the 1-year discharge. As the low water channel is not fixed but changes with the passage of time, embedment should be made deep not only in the present low water channel but also in the places which might become low water channels in the future.

6) Span

1) This should be more than 20 m. (Rivers less than 20 m wide are excluded.)
2) The span should be determined so that the total width of piers (the sum total of the widths of piers) may be less than 5% of the width of the river in normal conditions.

7) Under clearance

1) It should be higher than the crest of the approach roadway.
2) It should be higher than the crest of the levee.
3) It should have a height that assures the cross-section that allows the discharge of the largest recorded flood or the 50-year probable flood.

8) Revetment

1) The revetment length should be 10 m or half of the span measuring from the upper and lower stream ends at the abutments and piers.
2) The revetment height should be determined so as to allow the larger discharge, the one at the time of the highest recorded water level or the one at the time of the 50-year probable flood.

The above description is a guideline in designing a new bridge or a reconstructed bridge. In repairing the existing bridges, it is desirable to implement effective measures which are as close to the above described guideline as possible.

(3) Repair Works for Scours

The only enduring countermeasure against scouring is to increase embedment depth. As urgent and temporary methods, the following measures are available:

1) Construction of the training levee around the bridge.(Refer to Figure 4-1)

(i) It changes the stream direction and reduces the scours of the abutments and piers.
(ii) It protects the access road embankments by deflecting the stream parallel to them.

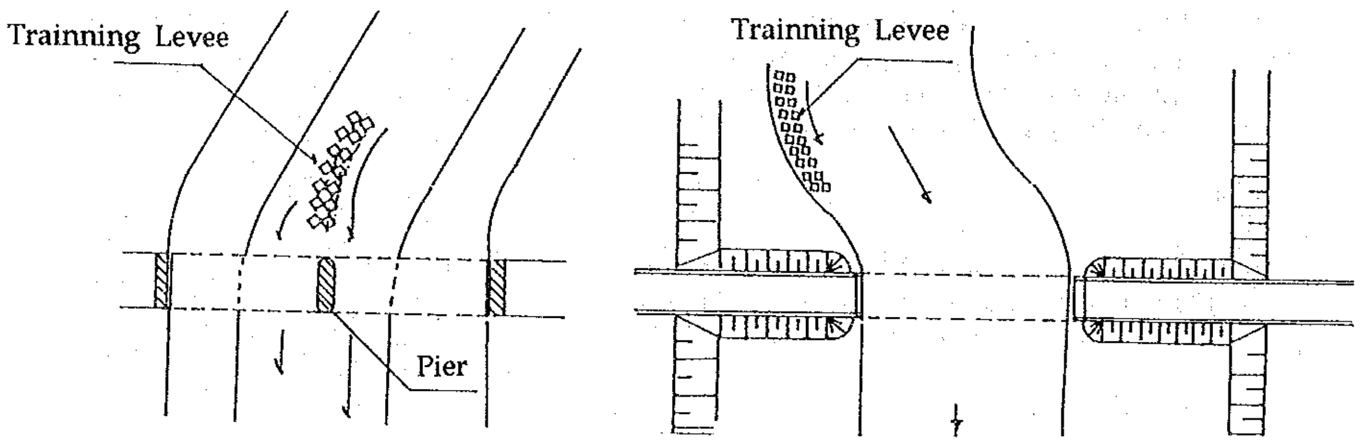


Figure 4-1 Training Levee

2) Embedding the existing protecting works deeper than the scour depth (refer to Figure 4-2).

(i) Large stones or artificial blocks are buried from the level deeper than the scour depth to the level of the present riverbed.
(ii) It is protected by covering concrete. (Either tetrapod or cross-shaped concrete blocks will do)

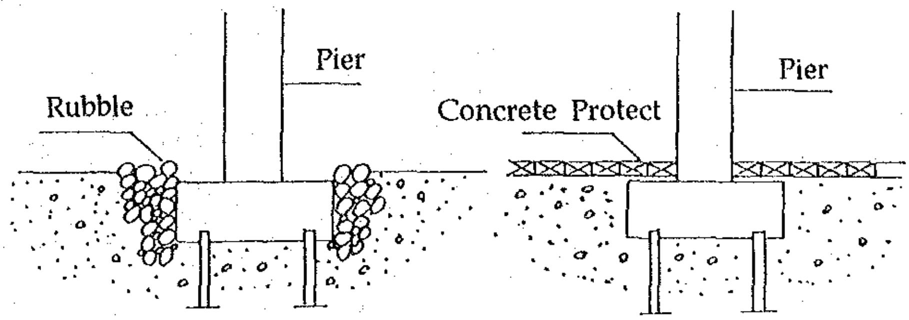


Figure 4-2 Protecting Works for the Scour

3) Construction of band works (i.e. small-scale groundsill) to prevent the scour of the riverbed. (It aims to prevent the riverbed from being washed away.) (Refer to Figure 4-3)

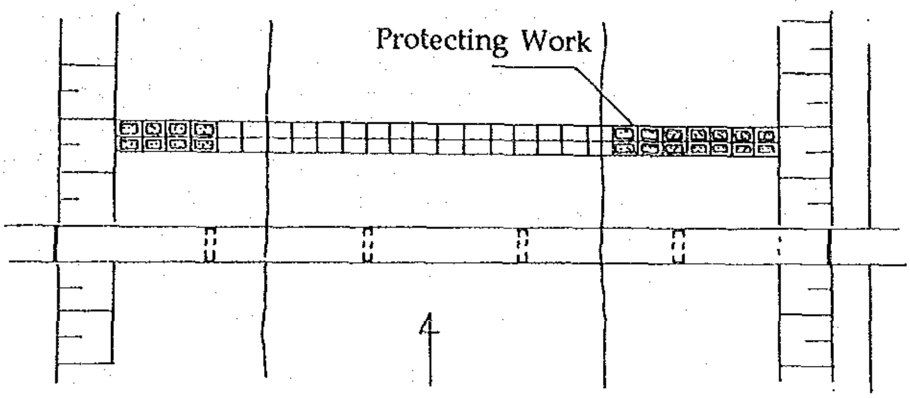


Figure 4-3 Protecting Work for the Scour of Riverbed

4) The enduring countermeasures against the scouring of the abutments and piers are the following 2 methods: (Refer to Figure 4-4)

(i) The underpinning of the scoured part of the existing abutments and piers by driving diaphragm walls or piles. (increase in the embedment)

(ii) The increase of the flow passage section by extending the bridge length. (measure to reduce the scour depth)

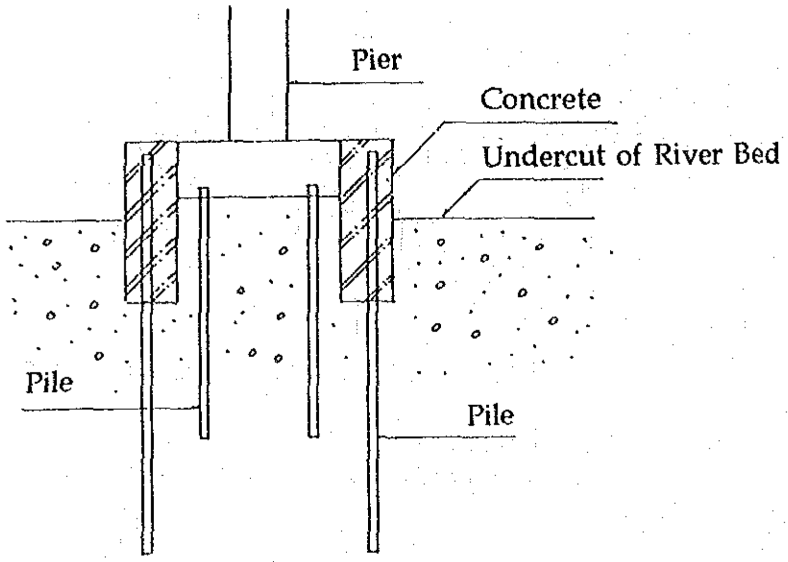


Figure 4-4 Increase of the Embedment for Foundation

There are various urgent and temporary methods as described before. In cases that those methods are adopted, it is necessary to carry out surveys every year on the scours at abutments and piers and also on the impact on other abutments and piers which were not the objects of the repair works.

When a training jetty is made to protect the scour of abutments and piers, the part where the stream power is concentrated may strike other abutments and piers, bringing about a scour in some cases. When band works are constructed crossing a river, the scour may occur at the downstream side of the band works, and it will cost much for the maintenance. The water level at the upstream face of the band works will rise, and the countermeasures against it are indispensable. In cases where the river is wide, the costs will be very high.

4-2 Bridge Repair Design

4-2-1 Basic Policies

The aim of this chapter is to present proposals for the methods to be used in the repair work of the ten bridges selected as a result of the First Field Survey, conducted between November and December 1992. The ten bridges were selected for the following reasons.

1) The bridges required urgent repair as the degree of damage was severe.
2) The type of damage observed on them was typical for bridges in Chile, as well as satisfying condition 1) above.
3) They were bridges that were of socioeconomic importance.

The ten bridges selected are therefore generally in unsatisfactory conditions and some of them will in fact require replacement with new bridges. Under the present project, however, replacement and upgrading of the bridges will not be considered. Proposals will be made here for methods of reinforcing the bridges so as to prevent collapse of the bridges that are in particularly dangerous conditions, and for methods of conducting repair work with a view to maintaining the existing bridges functional and removing the dangers.

Single reinforcement/repair methods are proposed and drawn up in principle for each bridge, but the proposals here do not go as far as the calculation of the details such as the thickness of the members and the quantities of reinforcement steel. Possible amelioration methods include a wide variety of choices from maintenance and repair to reinforcement and replacement and further studies will be required in many cases before the final decision is reached. Furthermore, the design for the repair work should deal not only with individual items of damage but should be based on an accurate assessment of the overall condition of each bridge. Various factors will therefore be taken into account in the design of the repair work, not only the conditions of the bridges themselves but factors relating to the topographical, geological, seismological and sociological conditions of the area where the bridges are located, as well their relation to the rivers which they cross and roads with which they are connected.

4-2-2 Results of Detailed Surveys and Conditions Relating to Selection of Repair Methods

(1) Results of Detailed Surveys

The detailed surveys were conducted using the instruments described in Chapter 3 and include tests on concrete strength, carbonation of concrete and hardness of the steel members, as well as measurement of the sectional dimensions of the bridges and their deformation. Soil surveys were also conducted around some of the bridges in question. The results of these surveys provide the basis for the selection of the repair methods. The items of the damage as observed in these surveys and visual inspection may be classified as follows.

(a) Appropriateness of the relationship between the road alignment and bridge position
(b) Topographical and geological problems at the bridge (landslides, soft ground, liquefaction)

(c) Problems relating to the river
(d) Traffic volume and bridge width
(e) Aging of the bridge
(f) Damage to structures
(g) Problems arising from the design and construction of the bridge
(h) Seismic structure of the bridge
(i) Deformation of the bridge
(j) Conditions relating to detour routes

The items of damage observed on each bridge may be summarized as shown in Table 4-1.

Table 4-1 Damage and Causes

Name of Bridge	A	B	C	D	E	F	G	H	I	J	Bridge Type
Amolanas	*	*		*	*	*	*	*	*	*	Reinforced concrete arch
Pullally		*	*				*			*	Steel-plate girder
Maipo			*				*	*		*	Prestressed concrete (post-tension)
Claro			*	*	*					*	Brick arch
Loncomilla			*			*			*	*	Prestressed concrete (pre-tension)
Bio Bio			*	*	*						Steel-plate girder
Ramadillas		*	*	*		*			*	*	Steel-plate girder
Malleco	*	*				*	*	*	*	*	Steel-plate girder
Pichoy		*			*	*			*		Steel-plate girder, T-shaped reinforced concrete girder
Cayumapu		*			*	*			*	*	T-shaped reinforced concrete girder

Note: Asterisks (*) show the applicable items of damage on each bridge.

(2) Conditions Relating to Repair Design

In addition to the examination of the damage and its causes on each bridge, certain conditions must be established for each item of damage in selecting the methods to be used in the repair work. The following conditions are used as the criteria in the design of the repair work.

1) Conditions for Road Alignment

Are the horizontal curve and longitudinal gradient on the bridge and in the sections immediately on either side appropriate for the design speed of the road? (Amolanas and Malleco Bridges)

2) Topographical and Geological Conditions

a. Landslides have been observed on the Temuco side of Malleco Bridge, and a drainage well (diameter: 25 m, depth: approx. 30) and approximately 50 landslide prevention piles (diameter: 1.0 m) have been constructed. The pier of this bridge, with a height of 77 m, penetrates only 6 m into the ground, a fact which gives rise to doubt as to whether account was taken of the location of the bridge in a landslide-risk area at the time of its construction. Geological surveys will need to be carried out to decide whether to implement landslide prevention works for protection of the bridge or to relocate it altogether.

b. On loose sandy ground liquefaction at times of earthquakes may result in the lowering of the bearing capacity. Investigations need to be carried out in such cases on the requirement for foundation works.

c. Where the penetration of the piles is inadequate on soft ground, this makes them prone to settlement and lateral movement. These piles should be reinforced with long additional piles or, where the conditions are not satisfactory, the substructure as a whole should be replaced. (Ramadillas, Pichoy and Cayumapu Bridges)

d. Foundation works are of importance where the abutments have been constructed on fill soil (Amolanas Bridge) or where there is a tall embankment nearby. Geological surveys should be carried out in such cases for investigation of the measures to be taken in such cases.

(3) Conditions Relating to Rivers

a. Where the bridge is too short in relation to the width of the river, the side spans will need to be lengthened in future. (Pichoy and Cayumapu Bridges)

b. Where the obstruction by the bridge in the cross-section of the river is large, effective spans will need to be enlarged in future. (Bio Bio Bridge)

c. Bridges whose clearance below the girders is inadequate in relation to the high water level should be replaced.

d. Where the bridge piers are not in line with the flow of water, the piers are subjected to high water pressure and this may also cause scouring. Construction of groynes and reinforcement of the foundation will be required in such cases. (Pullally, Maipo, Claro, Loncomilla, Bio Bio, Ramadillas and Cayumapu Bridges)

(4) Conditions Relating to Traffic Volume

Conditions relating to traffic volume include the bridge width, number of traffic lanes, and the problems in the structures for supporting the live load. These are summarized in Table 4-2.

Table 4-2 Traffic Volumes around Bridges to be Repaired

1988

Name of Bridge	Traffic Volume			Investigation Items	
	Small to Medium-Sized Vehicles	Large Vehicles	Total	Width	Live Load and Supporting Mechanism
Amolanas	1350	651	2001	*	(substructure)
Pullally	2235	1082	3317		(superstructure)
Maipo	11294	7400	18694		
Claro	4332	2867	7199	*	
Loncomilla	1199	684	1883		
Bio Bio	11700	2145	13845	*	(superstructure)
Ramadillas	1712	1501	3213		(substructure)
Malleco	2300	1444	3744		(superstructure)
Pichoy	1398	484	1882		(substructure)
Cayumapu	1398	484	1882		

Note: Those bridges marked with asterisks (*) in the table above do not have adequate widths and require widening or addition of new lanes. In the right-hand column, "superstructure" means that the bridge in question has problems supporting the live load for reasons arising out of the superstructure, such as damage to the floor slabs, while those marked "substructure" have problems such as the settlement of the substructure.

(5) Conditions Relating to Aging of Bridges

Of the bridges in question, Malleco and Maipo Bridges are relatively new. Serious symptoms of aging are observed on Amolanas, Claro and Bio Bio Bridges. These three should ideally be replaced. For reference, unsatisfactory results were also obtained in the carbonation test on Ramadillas, Pichoy and Cayumapu Bridges. Radical measures are required on Amolanas, Pullally and Bio Bio Bridges, where aging is observed in addition to the damage to the floor slabs, in view of the problems regarding the supporting mechanism for the floor slabs. On Ramadillas and Pichoy Bridges, where the widths of the bridge pier crowns are inadequate, the concrete has undergone carbonation and the sectional thickness is inadequate. It is thought best to carry out partial placement of new concrete on these two bridges.

(6) Conditions Relating to Damage to Structures

Considerations will be restricted here to damage to various components of the bridges. The positions where damage is observed are listed and given as conditions for devising countermeasures. Cases of slight damages have been omitted. Those cases listed below are also judged to require repair or reinforcement. (Refer to Table 4-3)

Table 4-3 Damage to Structures and Countermeasures

Name of Bridge	Floor Slabs	Main Girder	Cross Beam	Abutment	Pier	Foundation	Measures Required on
Amolanas	*	*		*	*		Floor slabs, pier
Pullally	*	*	*		*	*	Floor slabs, main girder
Maipo			*		*	*	Pier crown, foundation
Claro		*				*	Foundation, arch
Loncomilla				*	*	*	Repair of pier, foundation
Bio Bio	*	*	*	*	*	*	Floor slabs, fall-off prevention
Ramadillas		*	*		*	*	Pier crown, foundation
Malleco		*	*		*	*	Super- and substructures
Pichoy		*	*	*	*	*	Pier crown, foundation
Cayumapu		*		*	*	*	Gerber girder, substructure

Note: Asterisks (*) show the existence of damage in the relevant positions.

(7) Problems Arising out of Design and Construction of Bridges

Deficiencies that easily arise at the time of design include inadequate studies and inadequacy of the design standards used. Unsatisfactory execution, lack of satisfactory construction technology (machinery) and use of cheap materials and equipment for economic reasons will mean that the bridge thus constructed will suffer various problems on a semi-permanent basis. The problems arising out of such factors tend to run deep and usually require radical measures for amelioration. (Refer to Table 4-4)

Table 4-4 Problems Arising out of Design and Construction

Name of Bridge	Soil Surveys	Design Standards	Unsatisfactory Execution	Construction Technology	Economy	Remarks
Amolanas		*	*			Seismic design, floor slab camber
Pullally	*	*				Inadequate main girder section
Maipo		*				Seismic design
Claro						
Loncomilla	*			*		Inadequate pile penetration length
Bio Bio	*	*		*	*	Inadequate span
Ramadillas	*	*	*			Inadequate pile penetration length and pier crown width
Malleco	*	*				Landslide, seismic design
Pichoy	*	*				Inadequate pile penetration length and pier crown width
Cayumapu	*					Inadequate pile penetration length, Gerber girder

Note: Asterisks indicate problems arising out of design and execution.

(8) Conditions Relating to Seismic Design of Bridges

To determine the need for countermeasures, considerations are made on the overall structure of the bridge, as well as on the details of the structure, in relation to adequate strengths against earthquakes, and as to whether the bridges are located on ground easily affected by earthquakes. (See Table 4-5.)

(9) Conditions Relating to Deformation of Bridges

Deformation was observed on a number of bridges and in a number of cases the lack of bearing capacity was seen to have resulted in settlement and inclination, as well as deflection and torsion on the superstructure and tall bridge piers (see Table 4-6). The causes of such cases of deformation must be studied and appropriate measures taken accordingly.

Table 4-5 Parts Presenting Problems in Seismic Structure

Name of Bridge	Overall Structure	Details of Structure	Ground	Parts Presenting Problems
Amolanas	*	*		Column structure, girder support, longitudinal gradient
Pullally			*	Foundation
Maipo	*	*		Cross beams, independent piers
Claro				
Loncomilla				
Bio Bio		*	*	Girder support
Ramadillas		*	*	Girder support, soft ground
Malleco	*	*	*	Column structure, independent piers
Pichoy		*	*	Girder support, soft ground
Cayumapu		*	*	Girder support, soft ground

Table 4-6 Deformation of Bridge

Name of Bridge	Bridge Face			Superstructure		Substructure		
	Undulation	Settlement	Lateral Movement	Torsion	Deflexion	Settlement	Inclination	Deflexion
Amolanas	*	*	*		*		*	*
Pullally			*	*	*			
Maipo		*				*		
Claro								
Loncomilla		*	*				*	
Bio Bio	*	*	*					
Ramadillas	*	*	*	*	*	*		
Malleco				*	*	*	*	*
Pichoy	*	*	*			*		
Cayumapu							*	

(10) Detour Route Conditions

When a bridge faces major repairs, the execution methods will differ depending on whether or not the bridge can be closed to traffic. Even for work on the substructure, it may be easier to carry out the work with the superstructure removed, but if the traffic cannot be diverted this is not possible. The conditions relating to the detour routes are summarized in Table 4-7, and the sketch for the detour is shown in Figure 4-5.

Table 4-7 Detour Routes near Bridges

Name of Bridge	① Distance	② Town	③ Distance	④ Town	⑤ Distance	⑥ Town	Detour Route
Amolanas	197	Ovalle	45	Carelabaja	66	Socos	long
Pullally	53	Laligua	5	Placilla	8	Longotoma	long
Maipo	36	Caleradetango	6	Buin	31	Nos	long
Claro	42	Molina	2	Camarico	15	Molina	long
Loncomilla	96	Villaalegre	59		1	Sanjavier	long
Bio Bio	14	Concepcion	4	Concepcion	6	Concepcion	short
Ramadillas	55	Arauco	27	Curanilahue	6	Carampanque	long
Malleco	131	Angol	36	Victoria	1	Collipulli	long
Pichoy	16		4		5	Pelchiquin	short
Cayumapu	73		15	Antilhue	10	Pelchiquin	long

Where the detour routes have been judged to be "long" in the table above, the repair methods must be devised on the assumption that the bridge cannot be closed to traffic.

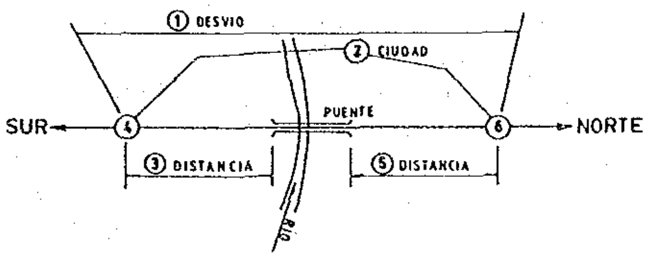


Figure 4-5 Notion of Detour

4-2-3 Outline Repair Design of Ten Bridges

(1) Repair Methods of Bridges as Permanent Structures

Investigations are conducted on the results of the detailed surveys and the repair methods are selected in accordance with the policies in drawing up the design for the repair work on the ten bridges. Although this does not fall within the scope of the present project, proposals are given below for the ideal amelioration methods for each of the bridges from the long-term point of view. Table 4-8 shows a summary of repair methods for bridges as permanent structures.

Table 4-8 Repair Methods for Bridges as Permanent Structures

Name of Bridge	Principal Defects	Repair Method
Amolanas	Seismic structure	Replacement
Pullally	Bearing capacity of superstructure	Replacement
Maipo	Seismic structure	Reinforcement of cross beams and piers
Claro	Aging, scouring	Reinforcement
Loncomilla	Scouring	Replacement
Bio Bio	Traffic volume, river obstruction, aging	Replacement
Ramadillas	Scouring, uneven settlement	Replacement
Malleco	Seismic structure, landslide zone	Replacement
Pichoy	Aging, uneven settlement, earthquake damage (girder movement)	Replacement
Cayumapu	Aging (Gerber girder), inclination of substructure (soft ground)	Replacement

(2) Methodology of Rehabilitation for the Detailed Investigation Bridges

1) Amolanas Bridge

This is a reinforced concrete arch bridge constructed around 1945. The cross-sections of the column foundations in the side spans have been reinforced and repair work has been carried out also on the front of the abutment.

(a) Superstructure

Fatigue cracks have developed on the surface of the floor slabs due to aging and the effects of the wheel loads, requiring repair work. The conditions are particularly bad in the side span at the Santiago end. For complete repair, ideally, either concrete should be newly placed over the whole bridge (Alternative No. 1), or the present reinforced concrete slabs should be replaced by different types of slabs, such as precast concrete slabs (Alternative No. 2). The present floor slabs, however, are 50 cm thick and their load carrying capacity is thought to be adequate. A further repair method is to inject resin into the cracks (Alternative No. 3). While this will be of little effect in raising the load carrying capacity of the floor slabs, it will have the effect of preventing the deterioration of concrete and reinforcement steel. As to the arch members, cracks are visible and their aging is undeniable. Furthermore, the member sections are too small and repair of these members will entail dangerous work high above the ground. From this point of view, the whole superstructure should ideally be replaced.

(b) Substructure

Although carbonation of concrete, exposure of reinforcement steel and cracking were observed on the piers in the arch section, there were no major abnormalities overall. The floor slabs were reported to have undergone deflection since the time of construction (no camber). Perhaps because there is displacement also of the pier crowns, the deflection is today visible by sight. The detailed survey revealed the crowns of the piers to have been displaced uniformly towards the abutments and towards the downstream side in the direction orthogonal to the bridge axis. A possible reason for this is that the cross-sections of the columns are too small in relation to the superstructure load. Perhaps it is because of this that the floor slabs are tied together with reinforcement bars on the bridge at present, but it would be more effective to provide a structure less prone to displacement at the pier crowns. There is a need, in other words, to implement cross-sectional reinforcement on the columns and to either construct beams at the pier crowns to create a rigid-frame structure or to create a wall structure as has been done in the case of Pier No. 1. (Refer to Figure 4-6)

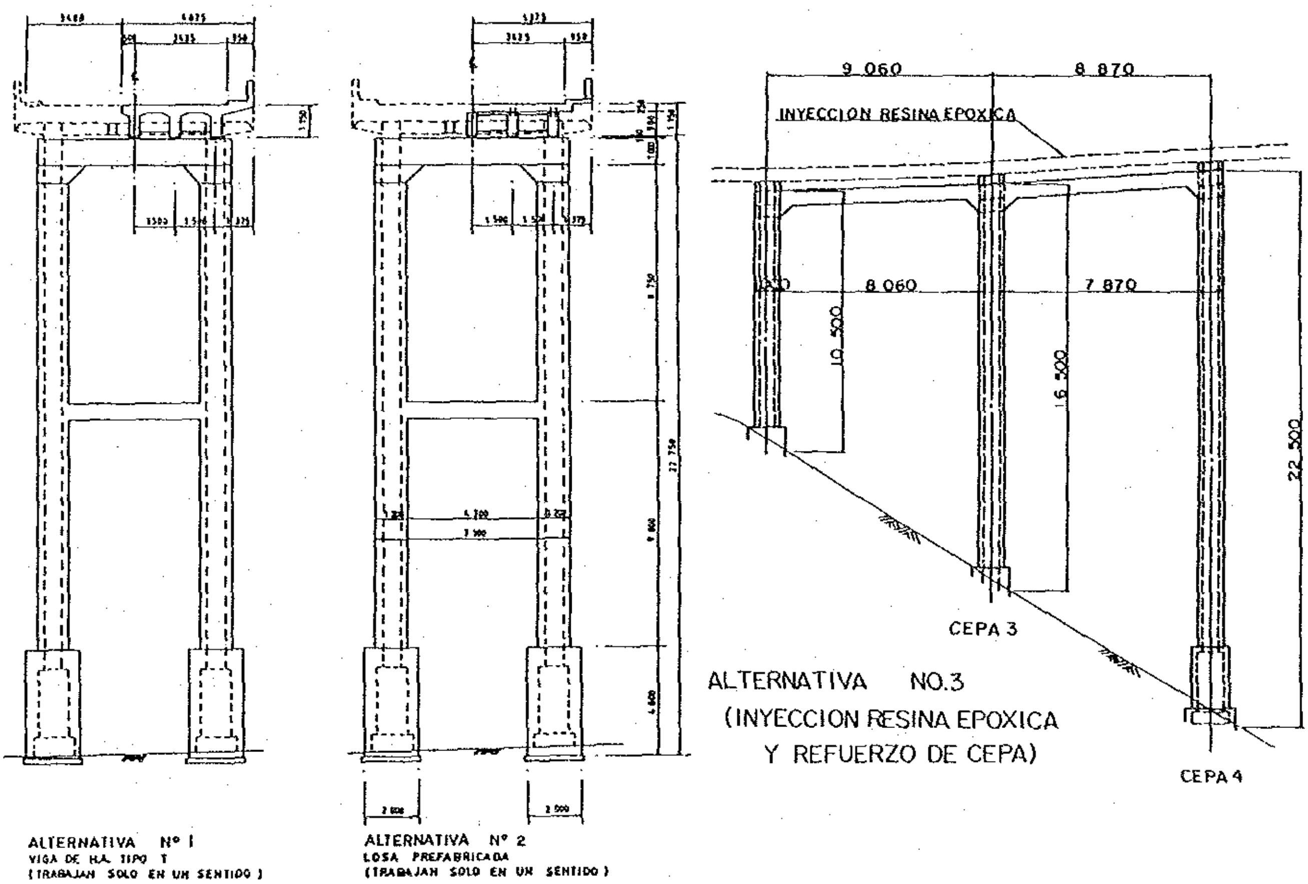


Figure 4-6 Repair of Amolanas

2) Pullally Bridge

(a) Superstructure

Failure has occurred in the past on this bridge around Pier No. 4 and the positions of the main girders were exchanged at the time of reconstruction. Deformations are observed on the webs and flanges of the main girders, either because of the effects of the reconstruction or because of the lack of adequate cross-section (girder depth to span ratio: 1/25). There is heavy vibration on the bridge deck and cracking has progressed to a significant level. While this is due mainly to the wheel load, there are problems also in the supporting mechanism for the floor slabs, including the lack of adequate main girder cross-section, inadequate load distribution on the cross beams and inadequate integration of the girders and the floor slabs. The cracks on the floor slabs seem to have originated as minute cracks in the tensile parts of the concrete and to have gradually developed under the load of passing vehicles. Mere reinforcement of the floor slabs will result in the regeneration of the cracks and development of potholes in a few years' time. In the long term, therefore, new concrete should be placed for all the floor slabs, reinforcing the main girders and the floor system and ensuring integration of the slabs and the main girders (Alternative No. 1), or the present floor slabs should be replaced by other types of slabs, such as the following, to strengthen the floor slabs themselves.

a) steel slabs
b) I-shaped steel grid slabs (Alternative No. 2)
c) steel panel slabs
d) precast concrete slabs

The use of these types of slabs will be more costly than replacement of the concrete but the slabs themselves will be lighter and will involve a shorter construction period. Methods of repairing the damage on the slabs themselves include the following.

a) attachment of steel plates (Alternative No. 3)
b) attachment of FRP rods
c) mortar spraying
d) resin injection

These methods are not intended to provide higher functions than at present, but are aimed merely at the restoration to conditions close to the original design or simply to prevent further lowering of the functions than at present. They should be understood as repair rather than reinforcement work. A fourth alternative addresses the weakness in the flooring structure among the causes of the damage, by construction of additional stringers and cross beams to reinforce the supporting mechanism for the floor slabs. Under this alternative, the cracks will be repaired by resin injection and should the conditions be found to have deteriorated again after several years the slab concrete will be replaced. This method, however, requires that the main girders have adequate bearing capacity and is therefore not suitable for this bridge.

Methods of reinforcing the main girders include the construction of additional main girders, but this being a continuous skew bridge, the analysis of the conditions before and after the addition will be complicated. Replacement of all the main girders increasing in the number of main girders and replacement of the supporting mechanism will be a difficult task. Methods in which the existing main girders are retained include introduction of prestress and enlarging the cantilever on the bridge piers to reduce the effective spans but the small size of the bridge renders these unrealistic. (See next page for the four alternatives discussed above.) (Refer to Figure 4-7)

(b) Substructure

Problems on the substructure include scouring by the river and problems in the foundation. There is sharp bend in the river just upstream of the bridge and the direction of the flow is not totally in agreement with the direction of the bridge piers. Construction of groynes should be considered to rectify this. For the time being, flood protection should be provided for the foundations. Because the bearing stratum is deep, there is a possibility that the piles used at present are friction piles. If the bearing capacity is inadequate, the piles may be subject to settlement. Regular observation should be carried out in future and should there be a tendency for settlement, measures should be taken, for example, by driving additional piles. The crown of Pier No. 4 has a large skew angle and failure has occurred here in the past. The pier should be widened together with the abutment. (Refer to Figure 4-8)

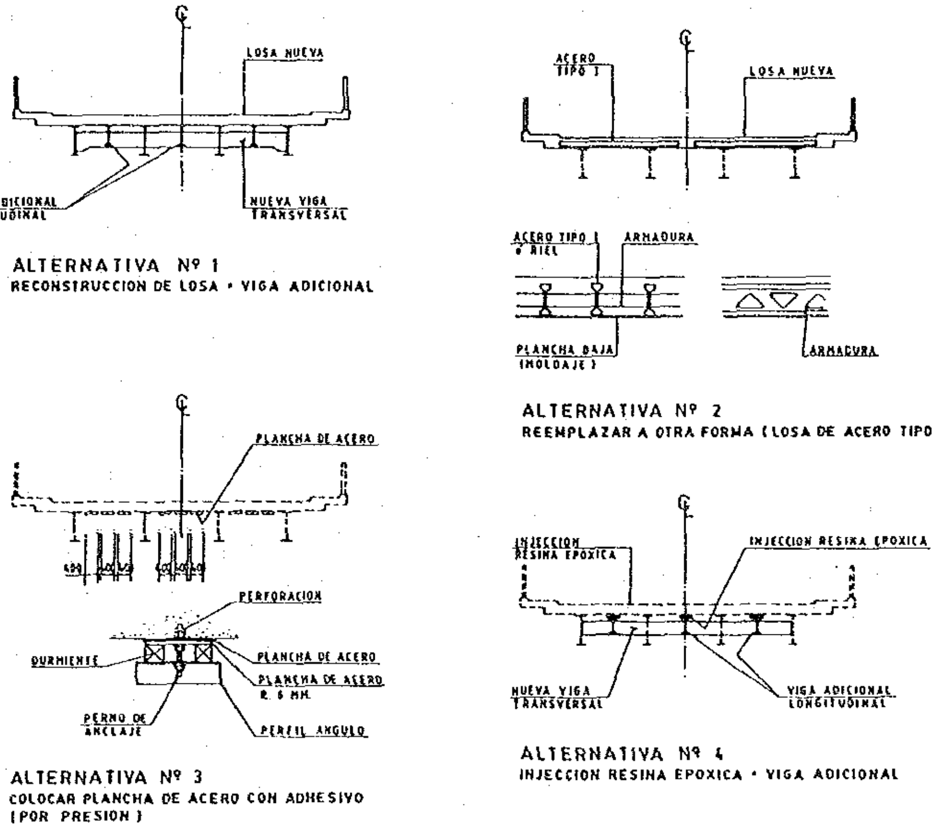


Figure 4-7 Repair of Superstructure for Pullally

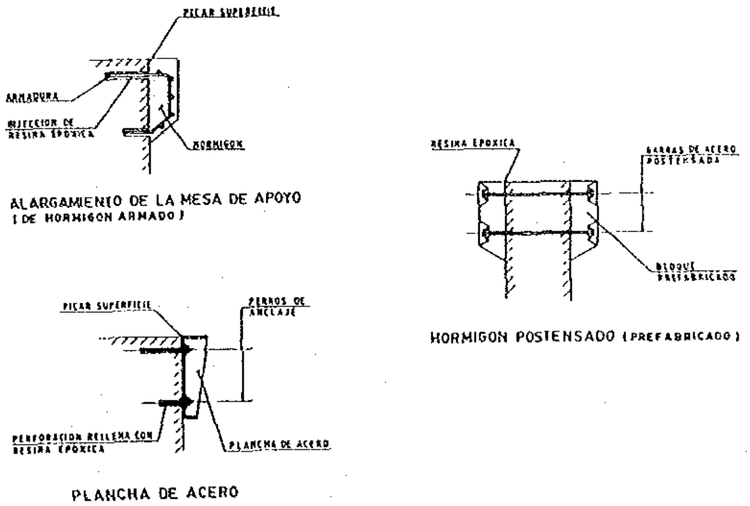


Figure 4-8 Repair of Substructure of Pullally

3) Maipo Bridge

I-shaped prestressed concrete girders and precast concrete floor slabs are used in the superstructure. The superstructure load is transmitted from the main girders to the end cross beams and thence to the caps of the bridge piers. The cross beams are 44 cm thick. The mechanism of the load transmission to the piers, the present stress conditions and the seismic resistance are unknown. No damage was observed in the field survey.

Each pier consists of two independent single shafts. The field survey results reveal tendencies for settlement or inclination on the upstream side of Pier No. 7 and on the downstream side of Pier No. 11. The length of the piles is estimated at 12 m. Although the foundation ground is satisfactory with a mixture of boulders, a lack of horizontal resistance force resulting from inadequate penetration of the piles is suspected. Thus, inadequacies in the structure of the end cross beams, and in the independent piers and their penetration have been pointed out from the point of view of seismic design. Possible countermeasures include a method that ensures that the load is supported even in the event of the rupture of the cross beams, a method in which the main girders are supported by other structures and a method simply providing a fall-off prevention mechanism (Alternative No. 3).

Possibilities on the substructure include retention of the present structural system (Alternative No. 1) and improvements on the structural system (Alternative No. 2), both of which involve large-scale work and are not economic. A third alternative is to prevent scouring by construction of such structures as gabions. The area around the bridge seems to be used for collection of boulders and gravel. There are cases in other countries where boulder and gravel collection has aided scouring of the riverbed, resulting in the end in fall-off of bridges. The riverbed variation should be studied carefully and the boulder and gravel collection implemented according to established plans. (Refer to Figure 4-9)

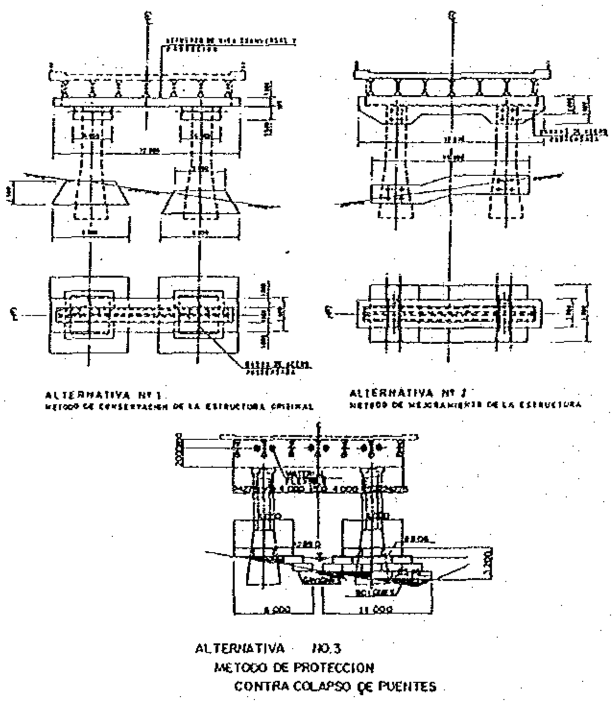


Figure 4-9 Repair of Maipo

4) Claro Bridge

This is a seven-span continuous brick arch bridge constructed around 1870 and, as a historic monument in Chile, a different approach is required in planning its repair from the other nine bridges. The purpose of the repair is to retain the bridge in its present form and to ensure its long-term preservation. At the same time, the bridge is located on Route 5 and the conditions surrounding it has become increasingly severe with the increases in the size and volume of the traffic, as well as the speed (impact) at which they travel over the bridge. If the bridge is to be preserved merely as a historic monument, there are precedents where the load only of the tourists has been considered as the live load. Bridges, unlike other architectural structures, are subject to continuous action of live loads and the required safety factor cannot in theory be achieved unless the resistance force of the members is made greater than the live load (which is larger than at the time of its original design). For this purpose, the arch section has to be reinforced with materials with a certain amount of rigidity. A certain amount alteration is inevitable either in the external appearance or internal structure of the bridge. Possibilities include construction of reinforced concrete arches overlapping with the original brick arches (Alternative No. 2) and construction of piers and beams on the original arches to relieve them from the live load (Alternative No. 3). Utmost care will be required so as not to cause damage to the original arches in the implementation of these alternatives and this will not be easy. Another possibility is to preserve the arches by injection of chemicals, but for this a detailed study of the internal structure is required to ensure if this method will be effective. A further alternative is to reconstruct the arches in the original form renewing the bricks and other materials.

There is also the possibility of carrying out repair work with the aim merely of maintaining the status quo without increasing the safety factor and the bearing capacity (Alternative No. 1). Discussions are made below on this alternative.

(a) Underside of Arches

There are indications that the lower faces of the arches have been repaired in the past with mortar but this mortar has fallen off. This part of the bridge is relatively free of restrictions from the point of view of aesthetics, and the materials used for repair (e.g. mortar) need to be made self-supporting to prevent their fall-off. Possibilities include the use of:

a) H-shaped steel
b) reinforced concrete
c) spray concrete

The support points for the arches, however, need to be widened slightly as there is insufficient space for supporting the reinforcement materials. (Refer to Figure 4-10)

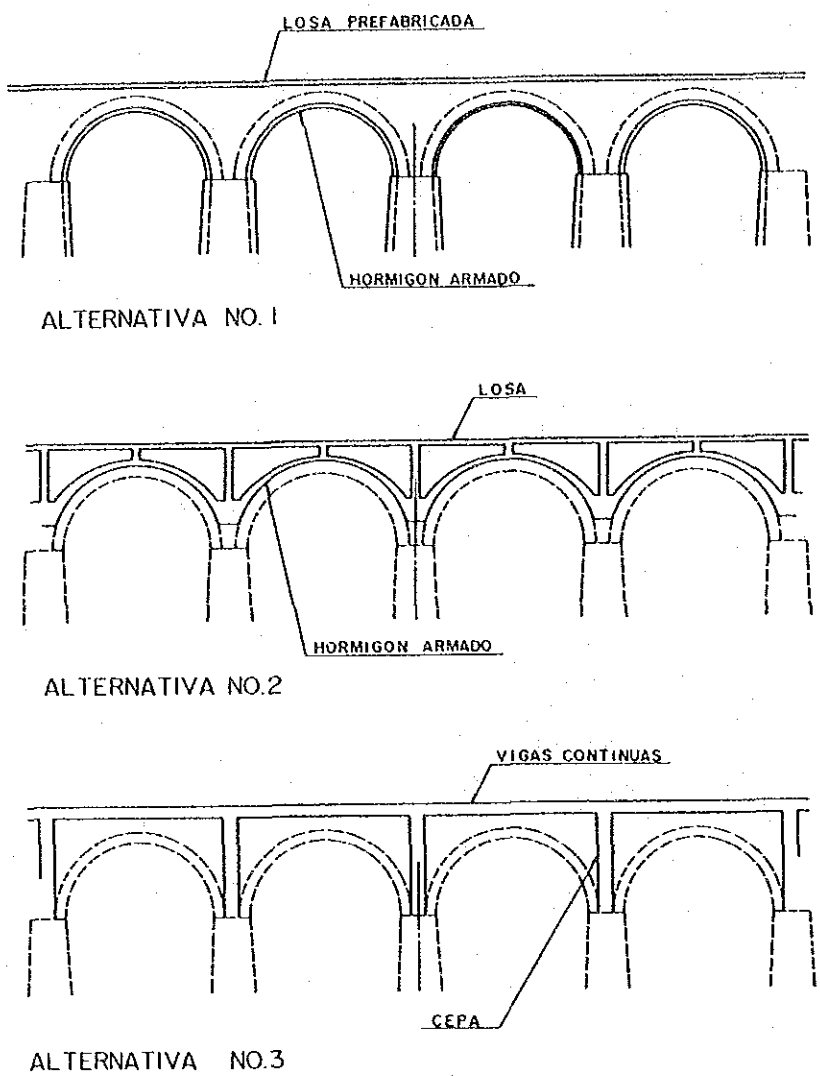


Figure 4-10 Repair of Arch for Claro

(b) Wall Faces

Stone panels have been attached to the brick face on the vertical walls for decoration, but these panels have fallen off in some places. The problem arises here of how to attach these stone panels to the aging bricks. Possibilities include insertion of a large number of anchor bars into the brick face or use of powerful resin adhesives. (Refer to Figure 4-11)

Some form of reinforcement work is desirable also on the sides of the arches in view of the increase in the wheel load and the progress of aging. The best solution is to connect the two sides with tie rods, but this is costly. A simpler method is to install new floor slabs to spread the wheel load. In this case, cutoff works will be required to prevent rainwater percolating into the arches. Since the new floor slabs will protrude further than the main body of the arches it will be easier to construct them with precast concrete slabs.

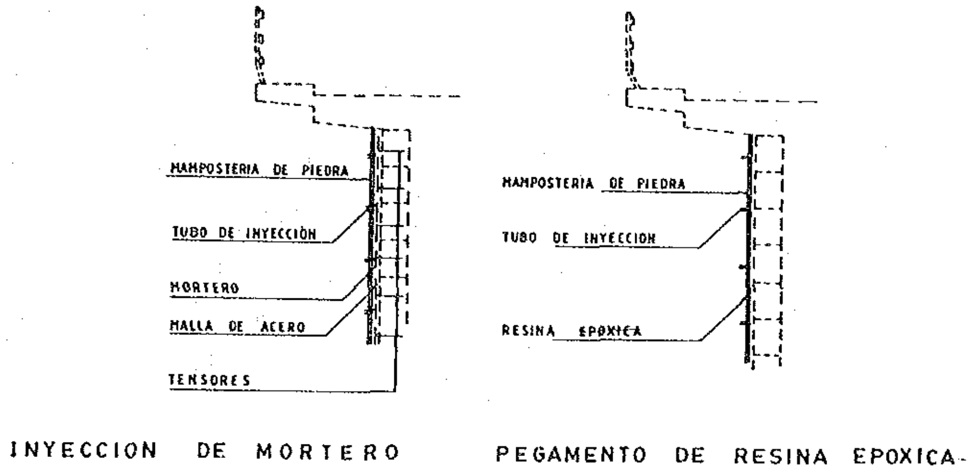


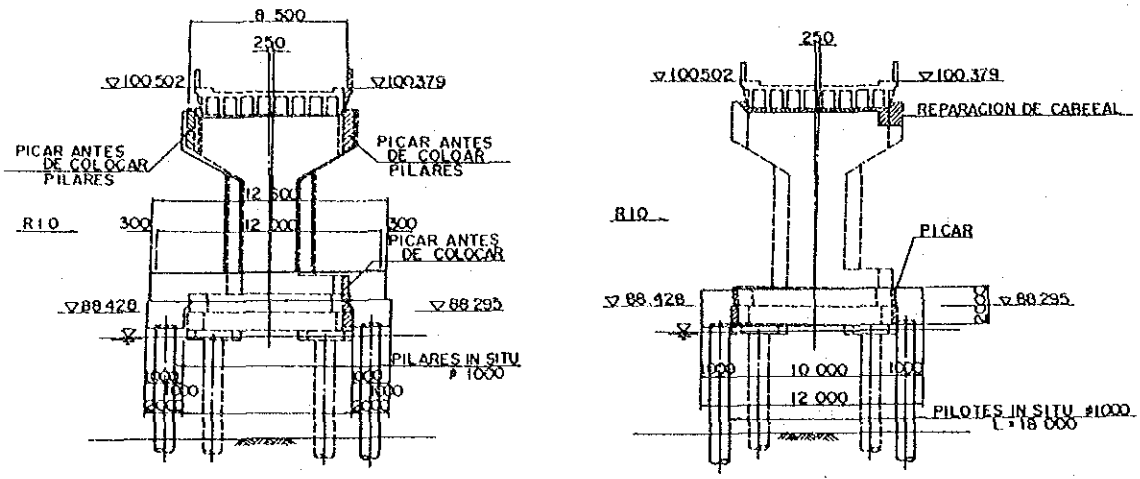
Figure 4-11 Repair of Side Wall for Claro

(c) Riverbed

Since settlement will cause irreversible damage to an arch bridge, the surroundings of the pier foundations must be protected from scouring. Groynes and small dams have been constructed in the past with steel piles in the water, but these have been destroyed. This is due to the fast flow of water at this point and the steep gradient of the rocks which makes the riverbed prone to scouring. To prevent scouring, a smooth flow has to be created by eliminating the steps and unevenness on the riverbed. In concrete terms, this means leveling the riverbed, constructing ripraps with boulder stones and laying gabions over them throughout the width of the riverbed.

5) Loncomilla Bridge

This is a pre-tension precast concrete bridge constructed around 1955. There are very few examples of this type of bridge in Chile constructed subsequently. Besides the damage to the railing, unsatisfactory execution of the fill concrete between the main girders is observed on the superstructure. The problem lies in the fact that Pier No. 5 has undergone a major tilting and a horizontal displacement of 25 cm and a vertical displacement of 12 cm towards the downstream side have occurred in the direction orthogonal to the bridge axis. The water depth, which was 6 m at the time of the survey, rises 6 to 7 m above this during floods and there is difference of nearly 10 m between the high and low water levels. While the foundation for this pier consists of 2 cast-in-place piles, other piers seem to be supported on rail-piles. The progress of scouring has resulted in the exposure of a significant length of the foundation piles and the lack of bearing capacity resulting from the buoyancy during floods is thought to be the cause of the tilting. Possible measures include the construction of a new pier (Alternative No. 1) and construction of additional piles to reinforce the foundation (Alternative No. 2). The lack of detour routes means that the bridge cannot be closed to traffic, such additional cast-in-place piles would be placed outside the bridge. The other 5 piers may also be subject to damage in future depending on the progress of scouring and it would be best to either reinforce them through construction of additional piles or to provide scouring-prevention works. (Refer to Figure 4-12)



ALTERNATIVA NO.1
CONSTRUCCION DE NUEVA CEPA

ALTERNATIVA NO.2
REFUERZO DE CEPA CON PILOTES ADICIONALES

Figure 4-12 Repair of Substructure for Loncomilla

6) Bio Bio Antiguo Bridge

The construction of the wooden bridge began around 1930 and the bridge has undergone repair on several occasions. Piers 12 and 13 (Concepción side) were washed away as a result of scouring in 1965. The river flows fast near both banks and the soil consists of fine sand, making the riverbed prone to scouring. The 90 bridge piers constructed at 15 m spans cause a major obstruction to the flow of water. The piers on the San Pedro side (C68 to C103) are each supported by two cast-in-place piles, while the rest of them (C15 to C67) have been reinforced with 12 steel pipe piles 30 cm in diameter and 12 m in length. The progress of aging on these piers is undeniable with cracks, exposure of reinforcement bars, leakage and growth of moss observed on them. Cracking of floor slabs and corrosion of steel girders are observed on the superstructure and the heavy vibration indicates that there is little allowance in the load carrying capacity. The traffic volume has reached the limit and the passage of large vehicles is prohibited.

The possible alternatives here are to construct a completely new bridge with an increased number of traffic lanes, or to construct a new bridge to take the extra traffic and to reinforce and keep the existing bridge in use. The latter alternative, however, will not provide a satisfactory solution if the bridge is to continue in use for a long time by traffic including heavy vehicles. In the superstructure, the girders will need to be increased and the floor slabs widened and replaced to address the problem of inadequate bearing capacity, while the substructure too will have to be reinforced to take the increased load of the superstructure. Even then the problem of the obstruction of the river will not be solved. It is judged best to construct a completely new bridge. A proposal is made below for the measures to be taken to maintain the existing bridge in function (to prevent its collapse) until the completion of the new bridge. For the substructure, because of the high costs of the repair work involved, regular inspections should be carried out for signs of settlement and inclination and measures taken according to need. Partial replacement of the floor slabs should be carried out in parts subject to severe damage, while the less serious damage should be remedied by such means as resin injection. Measures such as the following will need to be undertaken to prevent collapse of the bridge. (Refer to Figure 4-13)

a) attachment of movement control devices
b) widening of pier crowns
c) attachment of fall-off prevention devices

Some of these methods are illustrated below.

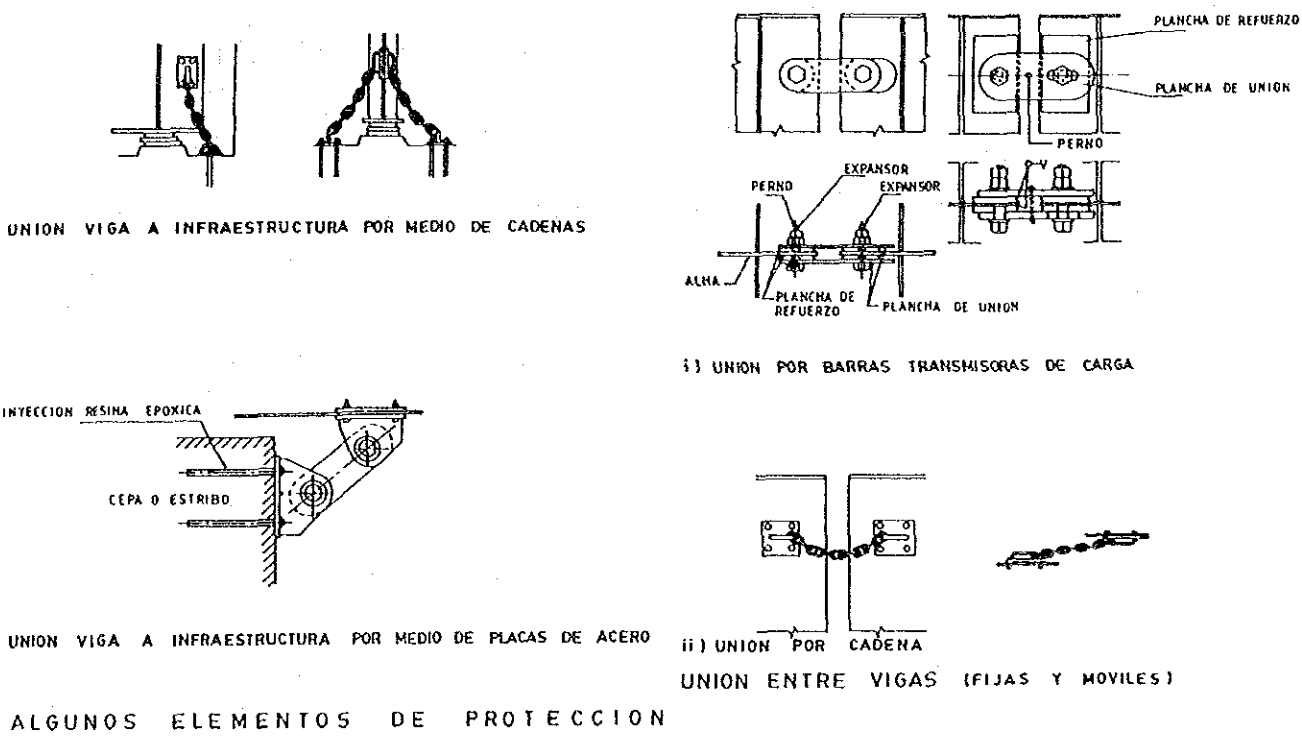


Figure 4-13 Counter Measure for the Damage by Earthquake

7) Ramadillas Bridge

This is a 15 m-span steel girder bridge. Although the traffic volume itself is not so large, the passage of large vehicles loaded with timber makes the bridge subject to large impact, and this seems to be the cause of the large cracks observed at the caps of the piers. The main girders seem to have been constructed without camber and have uniformly undergone deflection as a result. Irregular settlement and steps are observed on the bridge face resulting in undulation and uneven settlement. This is thought to indicate that either the bridge is located on soft ground or that the piles do not reach down to the bearing stratum. Either new piers (Alternative No. 1) or additional piles (Alternative No. 2) will have to be constructed. If additional piles are to be constructed, either islands will have to be created and piles driven with machines that can be used where there is little height allowance during the dry season, or holes will have to be constructed in the floor slabs for the construction of the piles. The pier caps are too narrow and the concrete has undergone carbonation. Here, walls will be constructed upwards from the floor slabs to surround the hole. (Refer to Figure 4-14)

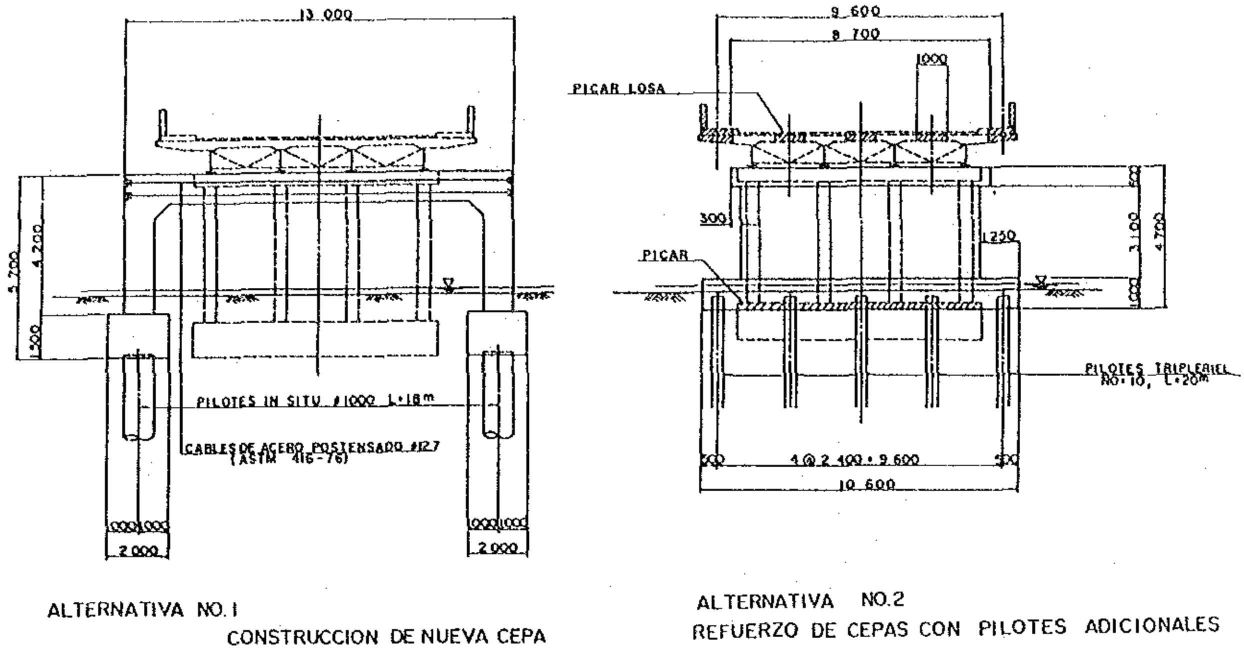


Figure 4-14 Repair of Substructure for Ramadillas

8) Malleco Bridge

(a) Survey Results

This is a 1-span simple and 8-span continuous bridge with three main steel girders constructed in 1972. Abnormalities were observed in the main girders the year after its completion and this was rectified by attachment of vertical reinforcement materials in certain parts. In the field survey, the deformation of the bridge face, main girders and bridge piers was measured with transits and levels The measurement revealed irregular displacement on the main girders and piers. Although the values for the displacement includes the measurement error as well as the displacement that has occurred during and after construction, the displacement observed differs from the type of displacement expected theoretically and is abnormal. The displacement of the main girders arises from the structure of the piers in which the substructure is more prone to displacement than the superstructure. The interaction of the super- and substructure loads has resulted today in a dangerous balance in which the main girders follow the displacement of the pier crowns.

(b) Analysis Results

The biggest problem concerning this bridge is its stability: Vibration in lateral and vertical directions is significant when large vehicles cross, and we were slightly anxious about its stability during our inspection. Details of the bridge structure include several unique design features which would never be permitted by Japanese construction standards, such as, a lack of bearing on the central of the three main girders. Therefore, we performed a three-dimensional stereo analysis and examined the stability of the bridge. Our analysis included an examination of the original bridge and the bridge after attaching diagonal members to the bridge piers, as shown in Figure 4-15, in order to increase the lateral rigidity. As a result, we discovered the following:

1. The natural period of the present structure is long at approximately eight seconds, therefore, the inertial force exerted during earthquakes is small.

2. The natural period of the structure enhanced by diagonal members is shorter and the inertial force is greater; as well, rigidity of the enhanced piers increases compared with piers which lack additional support. As a result, the horizontal force at the top of the bridge piers increases, which also increases the overturning moment at the base of the bridge piers.

3. As the present structure is flexible, the lateral deformation has become significant; however, despite their deformation the girders still maintain their proof stress.

From the above discussion it is obvious that enhancement of the bridge piers with trusses increases the problems of the bridge. The structure of the Malaeco Bridge as a whole has retained its stability, and partial enhancement of the piers will make the bridge dangerous by changing the location of the stress points along the structure. Based on these observations, when enhancing the substructure of the bridge, the bridge pier structure should possess sufficient resistance against the overturning moment exerted during earthquakes, which is increased because of greater pier rigidity.

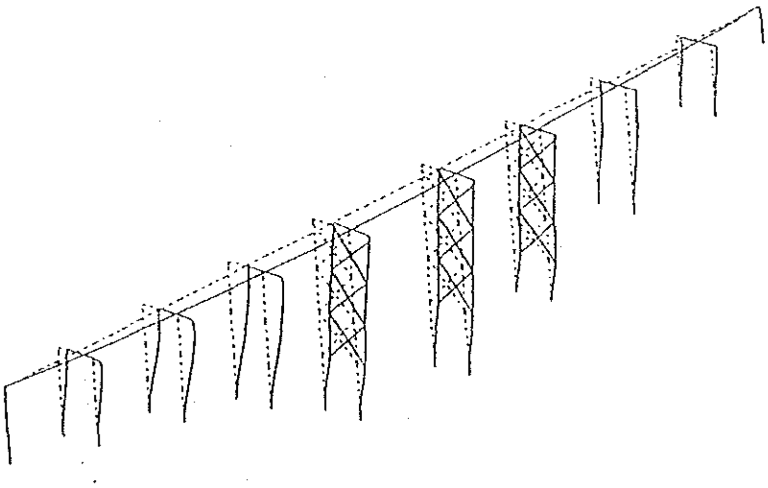
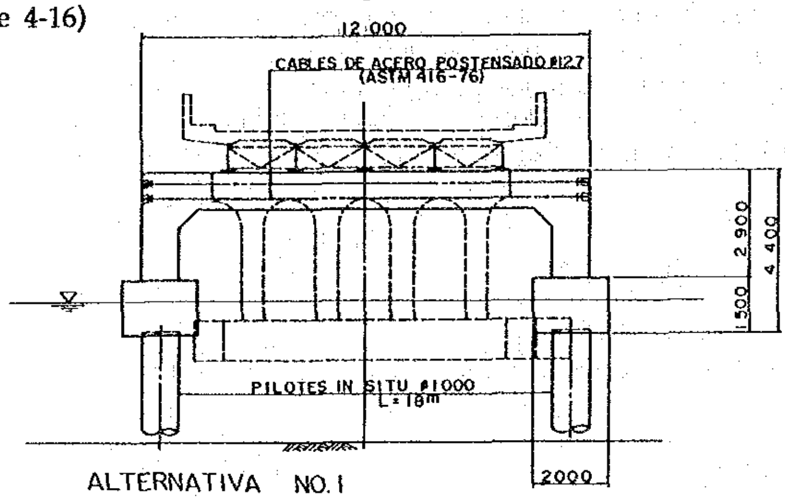


Figure 4-15 Frequency Mode for A Plan of Enhancement of Pier for MALLECO Bridge

9) Pichoy Bridge

T-shaped reinforced concrete girders and steel girders are used in combination on this bridge. The two spans on the Santiago side have already been replaced once. The support for the abutment girder on the Valdivia side has undergone major deformation in an earthquake and there is little allowance in the pier cap widths of the other piers. The pavement at the back of the abutments, too, has undergone major settlement (due to earthquakes, according to the hearing survey from related engineers), indicating that lateral movement may be occurring in the ground. The cause of the damage lies in the inadequate bearing capacity of the foundation piles and reinforcement of the foundation work cannot be avoided. This may take the form of construction of piles outside the bridge and construction of new piers (Alternative No. 1), or drilling holes in the floor slabs to drive in the piles from the bridge deck (Alternative No. 2). The lack of cap width on the piers can be dealt with by widening in the T-shaped reinforced concrete girder section and attachment of fall-off prevention devices in the steel girder section. (Refer to Figure 4-16)



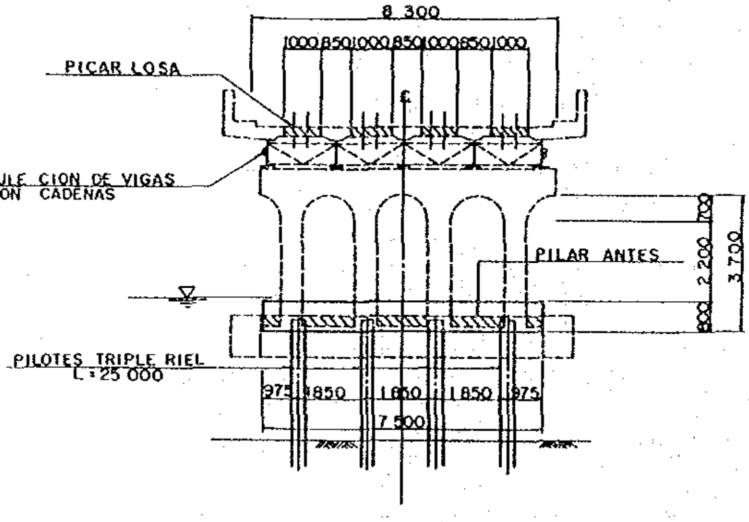
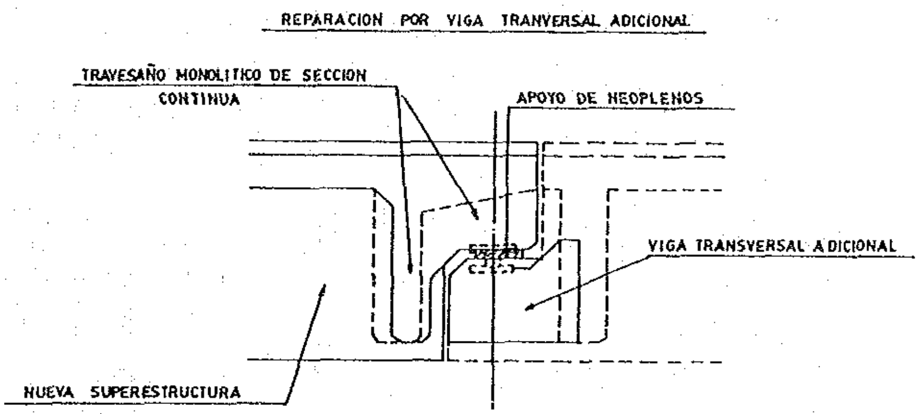


Figure 4-16 Repair of Substructure for Pichoy

10) Cayumapu Bridge

This is an irregular T-shaped reinforced concrete girder bridge with Gerber hinges in the middle. Leakage through the hinges have resulted in rusting of the shoes and the Gerber girders do not have adequate cross-sections against shear forces. Reinforcement work will be carried out either by insertion of additional cross beams or steel plates as shown below. (Refer to Figure 4-17)

Abutment E2 has undergone an inclination of 11° at its crown, perhaps because of the lateral movement, and is irreparable. Pier P2 has also undergone a major inclination along the bridge axis and its safety cannot be guaranteed. This is thought to be due to the softness of the foundation ground and lack of bearing capacity in the piles. Since the abutment and the pier have lost their functions as parts of the substructure, it seems best to replace them altogether rather than repair them.



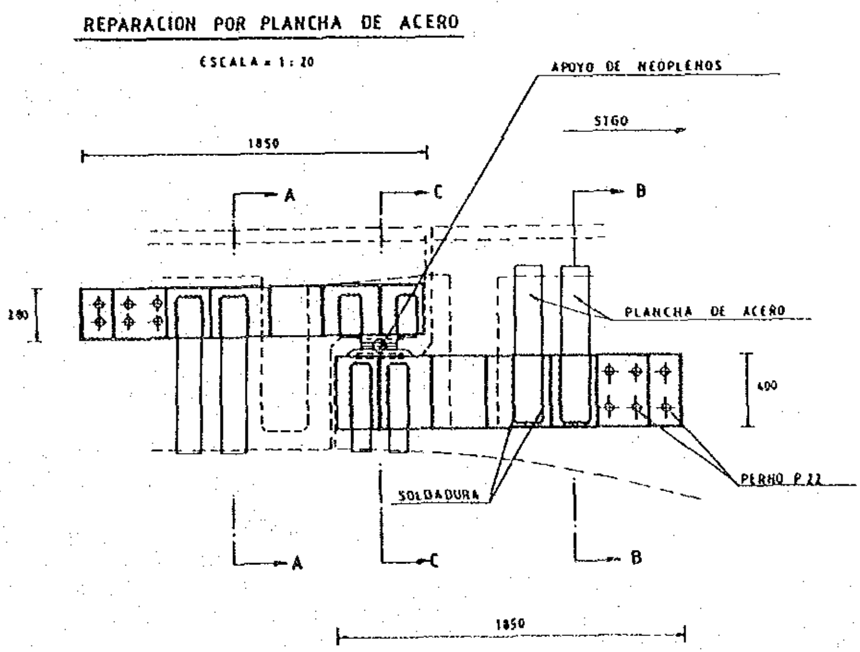


Figure 4-17 Standard Repair Method for Gerber Hinge

(3) Summary of Outline Repair Design

The repair and reinforcement methods discussed in the preceding sections (2) to (11) are summarized below.

Table 4-9 Repair and Reinforcement Methods for the Ten Bridges

Repair/Reinforcement Method		1	2	3	4	5	6	7	8	9	10
Floor Slabs	Placement in situ	#	#		#						
	Precast concrete slabs	#	#		*						
	Steel slabs		#								
	I-shaped steel grid slabs		#								
	Steel panel slabs		#								
	Attachment of steel plates		*				#				
	Attachment of FRP rods		#				#				
	Mortar spraying				#						
	Resin injection	*	*	#			*	#			
	Patching						*				
Floor System	Replacement of main girders	#	#								
	Overlapping beams				#						
	Addition of main girders		#				#	#	#	#	
	Addition of cross beams		#				#				
	Reinforcement of main girders	#	#		*				#	#	
	Span reduction		#								
	Introduction of prestress		#						#		
	Reinforcement of cross beams		#	#			#	#	#	#	#
	Repair of Gerber girders										*
Substructure	Widening of shoe seat	*	*	#			*	*		*	*
	Reinforcement/repair of piers	*		#	*	*	#	*	#	#	#
	Reinforcement of foundation		#		*	*		*		*	#
	Construction of new substructure	#				#		#	*	#	*
Others	Expansion joints	*		*		*	#	*		*	*
	Shoe					*	#	*	*	*	*
	Fall-off prevention	#	#	*			*	#		*	
	Painting		*				#	*	*	*	
	Railing				*	*	#				
	Approach slab	*								*	*
	Pavement	*	#		*					*	*
	Scouring prevention		*	*	*	*	#	#		#	*

Note) *: method adopted in outline repair design
#: method considered as alternative in outline repair design (excluding replacement of bridge)
Bridge names: 1 (Amolanas), 2 (Pullally), 3 (Maipo), 4 (Claro), 5 (Loncomilla), 6 (Bio Bio), 7 (Ramadillas), 8 (Malleco), 9 (Pichoy). 10 (Cayumapu)

4-3 Execution Plan

4-3-1 Outline of Works

The repair and reinforcement measures to be taken on each bridge may be summarized as follows.

1) Amolanas Bridge

• Pier cross-section reinforcement in side spans	x 5
• Slab repair by resin injection	1863 m2
• Widening of girder supports	x 2
• Approach slabs	x 2
• Expansion joints	x 2

2) Pullally Bridge

• Slab repair by attachment of steel plates and resin injection	1171 m2
• Scouring prevention with gabions	x 6
• Widening of girder supports	x 2
• Painting	m2

3) Maipo Bridge

• Fall-off prevention	x 13
• Scouring prevention with gabions	x 6
• Expansion joints	x 15

4) Claro Bridge

• Concrete arch repair and masonry work	118 m
• Precast concrete slabs	1062 m2
• Scouring prevention with ripraps and gabions	816 m2

5) Loncomilla Bridge

• Expansion joints	x 3
• Pier reinforcement by construction of additional piles	x 3
• Pier crown repair	x 1
• Scouring prevention with gabions and ripraps	x 4
• Shoe	x 1

6) Bio Bio Bridge

• Slab repair by resin injection (C8 to E2)	8667.5 m2
• Partial patching of slabs (1%)	87 m2
• Fall-off prevention (C8 to E2)	x 95
• Painting	m2
• Widening of girder support	x 1

7) Ramadillas Bridge

• Foundation reinforcement by construction of additional piles	x 13
• Pier repair accompanying shoe seat widening	x 13
• Expansion joints	x 15
• Painting	m2

8) Malleco Bridge

• Construction of new steel piers	x 8

- Shoes x 8
- Painting m2

9) Pichoy Bridge
- Foundation reinforcement by construction of additional piles x 4
- Fall-off prevention x 4
- Shoe seat widening x 2
- Approach slabs x 2
- Expansion joints x 6

10) Cayumapu Bridge
- Construction of new substructure x 2
- Repair of Gerber section x 2
- Approach slabs x 2
- Expansion joints x 3

See Section 4-3 for the cost estimates for the above and Annex for the drawings of the work listed.

4-3-2 Repairing Methods

Brief discussions are made below on the principal repair and reinforcement methods investigated in Section 4-2-3 "Outline Repair Design of Ten Bridges."

(1) Resin Injection (Bio Bio Bridge)

Resin injection is used, for example, in repairing floor slabs with the aim of minimizing the displacement due to cracking, corrosion of steel bars after the occurrence of cracking and preventing further development of the cracks.

1) Preparatory Work

- The cracks are cleaned by blowing air into them with compressors.
- If the cracks are filled with free lime, this is removed with wire brushes and grinders.
- Honeycombs and hollow and carbonated concrete are removed and the space filled with putty epoxy resin.

2) Pipe Attachment

- For injection, the centre of the crack is aligned with the centre of the pipe and a pipe with metal washers is positioned as shown below.
- If two cracks intersect, it is best to attach the pipe at the intersection.
- The cracked section is then sealed in a strip over a width of around 50 mm and depth of 5 mm as shown below with putty epoxy resin. (Refer to Figure 4-18, 4-19 and 4-20)

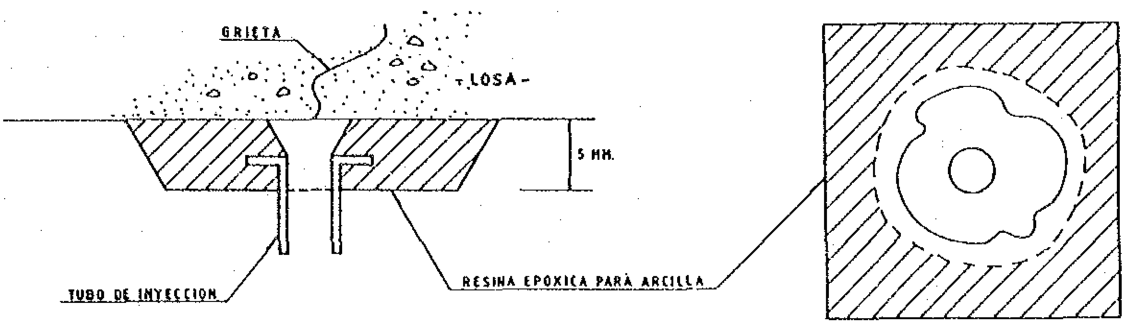


Figure 4-18 Pipe Setting Method for Injection

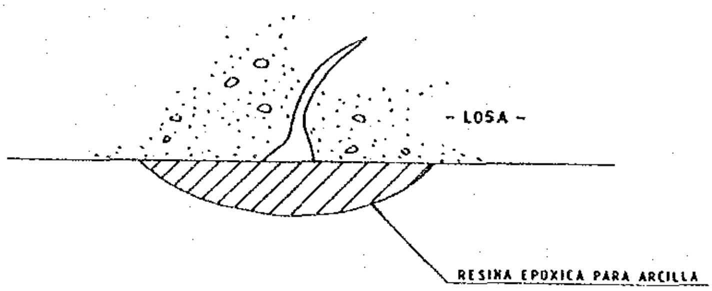


Figure 4-19 Sealing for Crack

3) Resin Mixing

The resin is measured out for mixing and mixing of excess resin is avoided. With a low-speed mixer the standard mixing time for 10 kg of resin would be over 2 minutes. If the amount of resin to be mixed is small and it is mixed manually, three times the amount of time will be required. As mechanical mixing results in mixture of a large amount of air bubbles in the resin, the air is removed before application.

4) Resin Injection

The external temperature should be between 10 and 30°C and the temperature at the concrete surface above 10°C at the time of injection. When injecting the resin with a pump, extrusion of the resin from the adjacent pipe is confirmed and the pipe is plugged before moving to the next pipe.

5) Finish and Inspection

After the injected resin has hardened, the injection pipes are cut off and the sealing materials are smoothened with grinders for the finish. Before this finish work, it is confirmed that the cracks are properly filled with resin while the pipes are attached.