CHAPTER 11 Design for Rehabilitation, Reinforcement and Improvement of 10 Selected bridges

11.1 Deck Slab

The concrete deck slab is the part, where the most damages are observed in the bridge structure, as the deck slab supports the wheel load of the vehicles directly. The damages observed on the concrete deck slab are the crack, water leaking, peeled off and subsidence of concrete. These damages are occurred by the repetition of act from wheel load. The crack in one direction develops to the mesh of crack and finally some of part of concrete will be peeled off. Therefore, the damages shall be repaired in the early stage to avoid development of damage. The reinforcement is required for the bridges, which are designed for small live load.

The design live loads of the objective 10 bridges are HS15 or HS20, which depend on the constructed year of the bridge. The design live load HS20 +25% is used in Costa Rica at now corresponding to the increas of vehicles weight. Therefore, the load capacity of the deck slab is not enough for most of 10 bridges except No.17 Chirripo Bridge and No.20 Sucio Bridge. As these two bridges are the PC box girder type, and the prestress structure is also used for their deck slab. The RC structure type of deck slab is used for the other bridges, and the stress of the reinforcement bar in the deck exceed its allowable stress for the increased live load HS20 + 25%.

Therefore, the reinforcement for the deck slab is required for 8 bridges, even for the bridges, which any serious damaged were not observed in the detailed inspection.

11.1.1 Reinforce Method for Deck Slab

Several kinds of methods are combined and applied for repair and reinforcement of concrete deck slab according to degree of damage. The remedial measures for the deck slab of bridge structure are classified by their purpose and method as shown in Table 11.1.1.

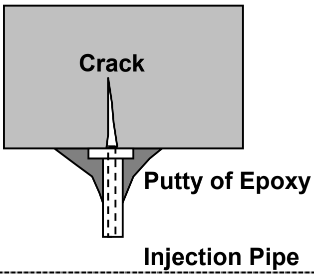
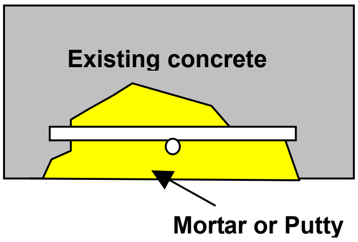
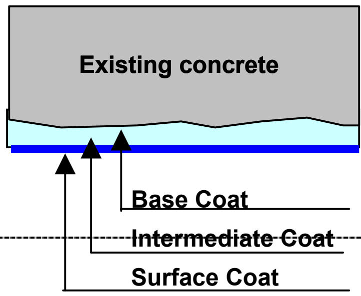
Table 11.1.1. Remedial Methods for Deck Slab

Purpose	Remedial Measure	Methods
Repair or Rehabilitation	Repair of Crack	Injection of Resin Mortal
		Filling of Resin Mortal
	Restore Structural Section	Restore Structural Section
	Concrete Surface Cover	Coating of Surface
Reinforcement	Increase Dimension	Slab Thickness Increase on Top of Deck
		Slab Thickness Increase on Bottom of Deck
	Additional Support	Additional Stringer Installation
	Reinforce by Additional Material	Steel Plate Bonding
		FRP Bonding (Carbon Fiber, other)
	Replacement	RC Deck Slab
		Precast RC Deck Slab
		Precast PC Deck Slab
		Semi-Precast RC Deck Slab
		Steel Deck
		Open Grating Deck

1) Repair Method of Concrete Deck Slab

Table 11.1.2 shows the principal examples of the repair method for damages on concrete deck slab, such as crack, isolated lime, and peeling off. These repair works are also required for reinforcement where the some damages are observed.

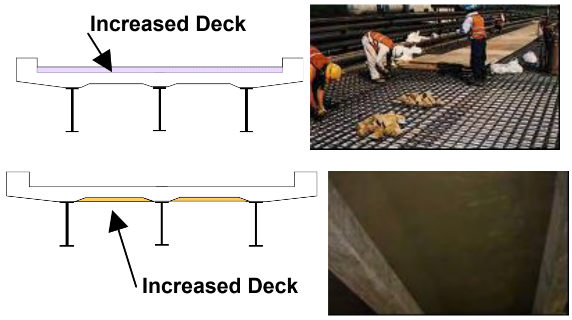
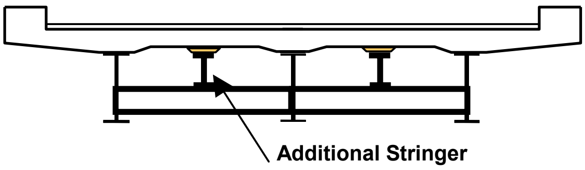
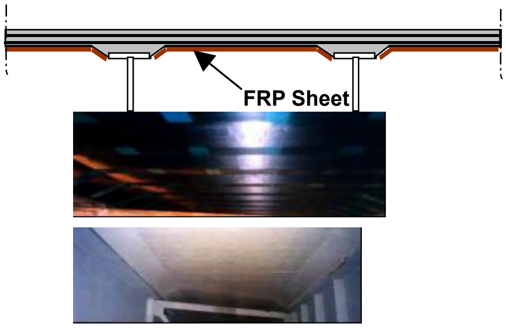
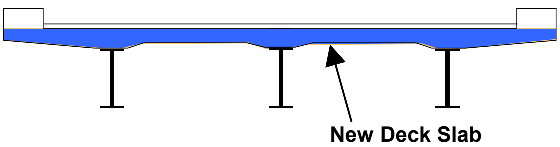
Table 11.1.2. Repair Methods for Deck Slab

Repair Method	Figures and Photos	Description of Method
Repair of Crack -Injection of Resin Mortal -Filling of Resin Mortal	 	Cut concrete surface in V or U shape along cracks and fill resin mortal to prevent the penetration of water, salt, or carbon dioxide. Filling materials shall have quality of penetration into crack and flexibility to adjust the crack.
Restore Structural Section		Remove damaged parts and restore dimension of structure. Materials applied to deck slab shall have some adhesive strength to original concrete and not to oxygenate reinforcement bar in the concrete.
Concrete Surface Cover -Coating of Surface		Coating on the surface of concrete to prevent the penetration of water, salt, or carbon dioxide. Materials for coating shall have quality of water-tightness and weatherproofing.

2) Reinforcement Method

Table 11.1.3 shows the principal examples of reinforcement method for concrete deck slab. As some of the reinforcement methods increase the weight of deck slab, the method shall be selected taking account of the effects for the influence to superstructure and substructure.

Table 11.1.3. Reinforce Methods for Deck Slab

Reinforce Method	Figures and Photos	Description of Method
Increase Dimension - Slab Thickness Increase on Top of Deck - Slab Thickness Increase on Bottom of Deck		By increasing stiffness of deck slab, to reduce the stress of existing reinforcement bar and concrete. Dead load of deck is increased Traffic suspension is necessary for Top Side Method.
Additional Support -Additional Stringer Installation		By reducing the slab span length to reduce moment of deck. Effective for steel bridge.
Reinforce by Additional Material -Steel Plate Bonding -FRP Bonding		By distributing stress to additional material, to reduce the stress of reinforcement bar. Steel plate cannot be applied on top of slab, because of the problem about the stability of adhesion of the asphalt paving on the steel plate. Therefore, it is not effective for negative moment on girder.
Replacement -RC Deck -Precast RC Deck -Precast PC Deck -Semi-Precast RC Deck -Steel Deck -Open Grating Deck	 	Effective for large damaged deck, which cannot be reinforced. Long traffic suspension is necessary. Construction period can be reduced by precast method. PC deck can reduced dead load Steel and Grating deck can reduce self weight but costly

11.1.2 Selection of Reinforcement Method for Deck Slab

The selection of repair or reinforce for the damaged deck slab depends on the load capacity of the existing deck slab as shown in Figure 11.1.1. It is the important to avoid increasing the self-weight of deck slab by reinforcement, because the superstructures and substructures are also required to be reinforce for the increased live load HS20 + 25%.

As the thickness increase method, which increases the weight of deck slab, is not advantage to the superstructure. However, this method has an advantage for the RC deck bridge, because the increased deck can be taken as a part of the flange of RC girder, and the stress of main girder due to the live load can be reduced.

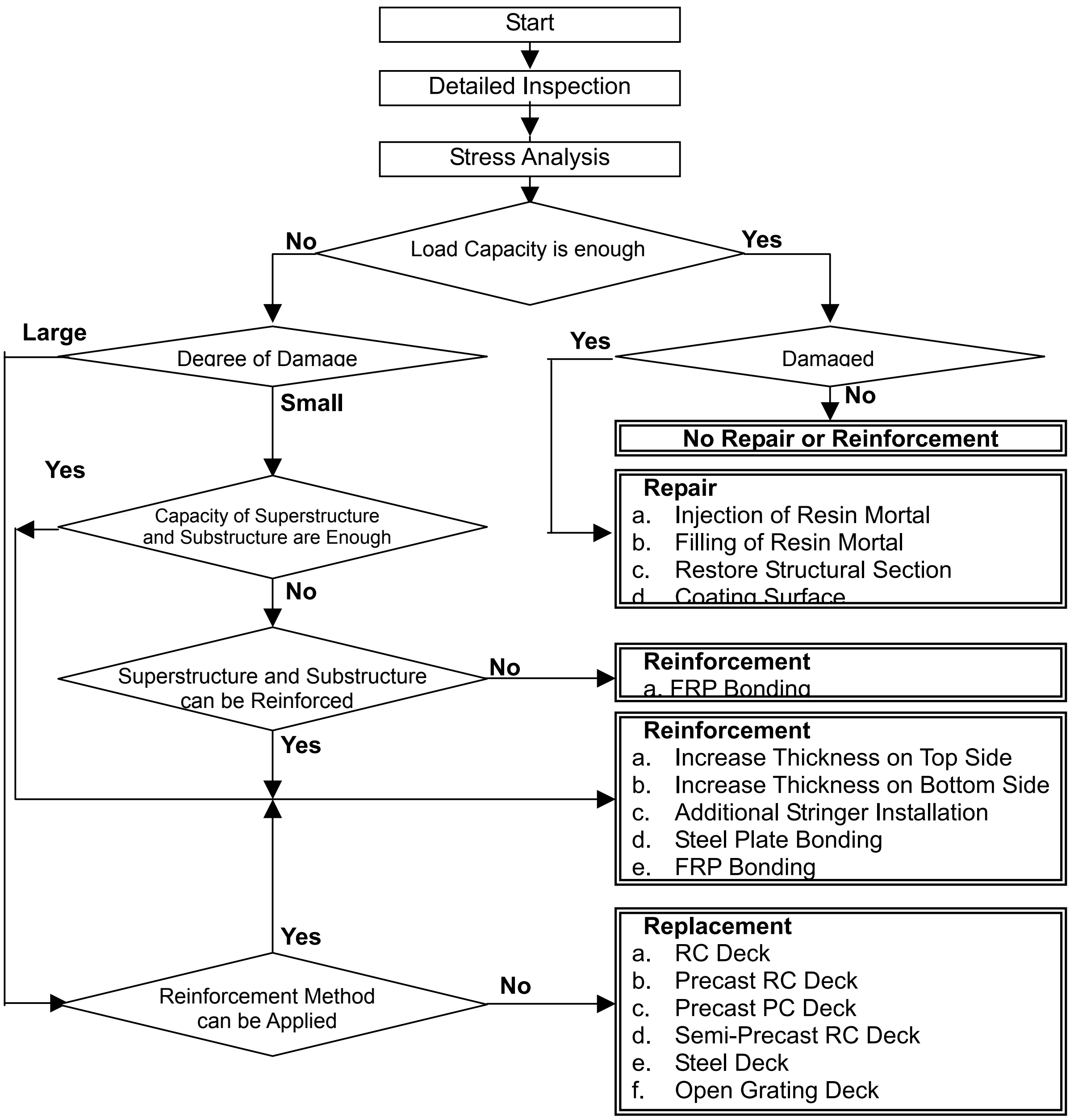


Figure 11.1.1. Selection Flow for Repair or Reinforcement for Deck Slab

The applicable method for several conditions are as shown in Table 11.1.4.

The design value of the negative moment of deck slab is same as that of the positive moment in standard of AASHTO. All objective bridges except the PC box bridge do not have the hunch of deck slab on the top of the girder. The deck slab shall be reinforced for both positive and negative moments. The steel plate bonding on the upper side of deck slab cannot be applied, because the pavement cannot be laid on the steel surface. The deck slab increase on bottom side of the deck is not effective for the negative moment on the girder.

Therefore, the applicable reinforcement methods for the deck slab of the RC and PC bridges are only the slab thickness increase on top side method and the FRP bonding method, and for the steel bridge the stringer addition method is applicable adding these two methods.

Table 11.1.4. Comparison of Applicable Method

	Slab Thickness Increase		Bonding		Stringer Addition
	Top Side	Bottom Side	FRP	Steel Plate	
RC Bridge	OK	OK	OK	OK	NO
PC Bridge	OK	OK	OK	OK	NO
Steel Bridge	OK	OK	OK	OK	OK
Increased Weight	Large	Large	NO	Small	Small
Negative Moment	OK	NO	OK	NO	OK
Traffic Suspension	NO	OK	Applicable	Applicable	OK
Capacity of Girder	Applicable	NO	OK	OK	OK
Cost	Low	Low	Medium	Medium	High

1) Truss Bridge

The truss type bridges existed in the objective 10 bridges are No.2 Aranjues Bridge and No.3 Abangares Bridge which locate on Route 1. The damages of deck slab on these two truss bridges are most seriously compared with the other types of the bridge. Especially, the deck slab of the Abangares has a large hole near the abutment, and one lane of the carriageway is closed for traffic. As the design live load for these bridges is HS15, the stress of the reinforcement bars for the increased live load exceeds the allowable stress as same as other bridge.

However, the main reasons causing the damages on the deck slab is the damage of the stringer of the deck support system, which supports the deck slab directly. The crack and broken bolts, which were caused by the fatigue of the steel due to the repeated heavy truckload, are observed on the stringer of the Abangares Bridge. These damages already developed and cannot be reinforced and it is necessary for both bridges to improve the deck support system for keeping the deck slab from damage. Therefore, the deck slab replacement is required for these bridges and the precast PC slab is most suitable to reduce the self-weight of the deck slab and to reduce the influence to traffic as much as possible.

Figure 11.1.2. Damages on Deck Slab of Truss Bridge

2) Steel I Beam Bridge

The steel I beam type bridges existed in the objective 10 bridges are No.12 Puerto Nuevo on Route 2, No.19 Sarapiqui Bridge on Route 4 and No.26 Chirripo Bridge on Route 32. The deign live loads are HS20 for the Chirripo bridge and HS15 for the other bridge. The two directional cracks are observed for all three bridges. Regardless of the difference of the design load, the degrees of damage are almost same for three bridges and can be reinforced. The reason seems that the traffic volumes of the bridges on Route No.2 and No.3 are not heavy comparing with the bridge on Route 32.

All three bridges cannot be suspended the traffic so long time. Therefore, the FRP bonding method is most suitable for the reinforcement of deck slab for these bridges.

Figure 11.1.3. Crack on Deck Slab of No.19 Sarapiqui Bridge

3) RC Deck Bridge

No.7 Azufrado Bridge on Route 1, the end span of No.12 Puerto Nuevo Bridge on Route 2, and No.16 Nuevo Bridge on Route 2 are the RC deck type bridge in the 10 bridges. All bridges were designed for the live load HS15, and the deck slabs are required to be reinforced for increased live load. Two directional crack are observed on the deck of these bridge.

For RC bridge the applicable methods are only two methods as shown in Table 11.2.4. All girders of three bridges don not have enough load capacity for the increased live load as same as deck slab and it is better to avoid increasing the dead weight of deck. However, the increased deck slab can be effective as the part of the flange of RC beam. Especially, for the continuous bride (No.16 Nuevo Bridge) and the rigid frame bride (No.7 Azufrado Bridge), the increased deck section is more effective for the negative moment at support.

The Puerto Nuevo Bridge cannot suspend the traffic, because no detour can be maid. Therefore, the slab thickness increase on upper side method is selected for the Azufrado Bridge and Nuevo Bridge, and the FRP bonding method is selected for the Puerto Nuevo Bridge.

4) PC I Beam Bridge

Only No.29 Torres Bridge located next to San Jose on Route 218 is the PC I beam type bridge in the 10 bridges. The stress of reinforcement bar of No. 29 Torres Bridge is less than the allowable stress for the increased live load, as this bridge was designed for the HS20 live load. However, the two-dimensional cracks are observed on many parts of the deck slab due to a heavy traffic volume of 29,500 per day. It seems that many over loaded vehicles are actually passing through this bridge and the deck slab was damaged by the punching due to the heavy wheel load.

Therefore, It was judged that only repairing by the resin injection into the cracks is not enough to maintain the deck slab to keep in good condition for long time and the reinforcement by putting the fiber concrete on the deck slab is required for this bridge.

5) PC Box Girder Bridge

PC box girder bridges are No.17 Chirripo Bridge on Route 4 and No.20 Sucio Bridge on Route 32. Both bridges are designed for the HS20 live load and the prestress type deck slab has enough load capacity for the increased live load. The seriously damages on the deck is not observed on the bridges. Therefore, no reinforcement is required for the deck slab of these bridges.

11.1.3 Methodology of Design for Slab Reinforced by FRP Sheet

The methodology of the design for reinforcing for deck slab by the FRP bonding is to reduce the stress of the reinforcement bars inside of the existing deck slab by distribution of stress to the bonded carbon fiber sheets. The carbon fiber sheets can only resist for the forces due to the live and dead loads after bonding. The force due to the dead load before bonding the fiber sheet has to be resisted by the reinforcement bars.

Many kinds of carbon fiber sheet materials are produced for the strength and the elastic modulus. The characteristics of the typical carbon fiber sheet are shown in Table 11.1.5.

Table 11.1.5. Characteristics of Typical Carbon Fiber Sheet

Strength	Large							Small
Elasticity	Small							Large
Unit Weight	g/m^2	200	300	200	300	300	300	300
Thickness	mm	0.111	0.167	0.111	0.167	0.165	0.143	0.143
Ultimate Stress	N/mm^2 Kgf/cm^2	3,400 35,000	3,400 35,000	2,900 30,000	2,900 30,000	2,900 30,000	1,900 20,000	1,900 20,000
Elastic Modulus	N/mm^2 Kgf/cm^2	$2.3x10^5$ $2.35x10^6$	$2.3x10^5$ $2.35x10^6$	$2.3x10^5$ $2.35x10^6$	$2.3x10^5$ $2.35x10^6$	$3.9x10^5$ $4.00x10^6$	$5.4x10^5$ $5.50x10^6$	$6.4x10^5$ $6.5x10^6$

The strength of fiber sheets is about 7 to 8 times stronger than that of the steel material as shown in Figure 11.1.4.

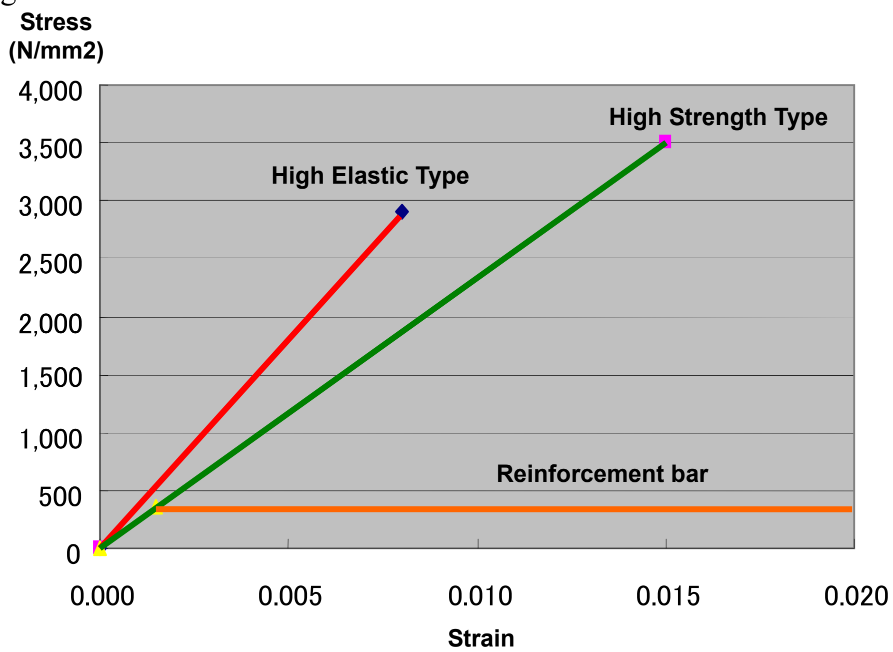


Figure 11.1.4. Stress/Strain Curve for Carbon Fiber Sheet

1) Design Flow for FRP Bonding

The design flow for the member which reinforced by the FRP bonding is as shown in Figure 11.1.5.

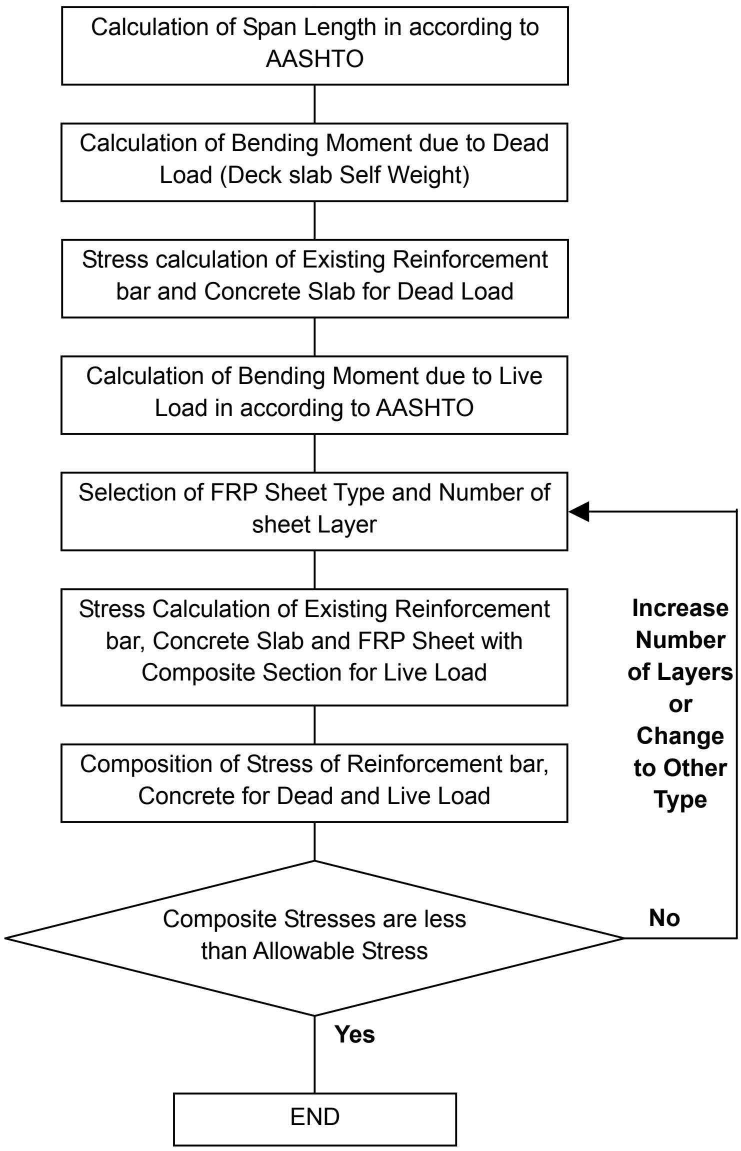


Figure 11.1.5. Design Flow for FRP Bonding on Deck Slab

The stress of the reinforcement bar in the existing slab desk, which is composite of the stresses due to both dead and live loads, shall be less than the allowable stress. As the carbon fiber sheet has a large value of allowable stress, which is one third of the specified strength, the stress of fiber is normally smaller than its allowable stress. The required number of fiber sheet layers will be fixed by the stress of the reinforcement bar.

2) Stress Check Method for FRP Bonding

The stress check method for the FRP Bonding slab is as below.

1. Bending Moment **Md** and Stress of Reinforcement bar and Concrete due to **Dead Load**

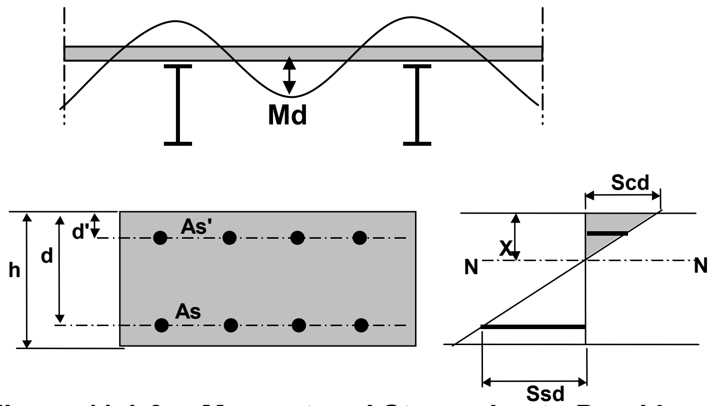


Figure 11.1.6. Moment and Stress due to Dead Load

Stress of Concrete due to Dead Load **Scd**: = Md x X / I
Stress of R-Bar due to Dead Load **Ssd**: = Md x n (d-X)/ I
Where,

- Md : Bending Moment due to Dead Load
- X: Distance between Neutral
- n: Modulus Ration between Concrete and Reinforcement bar
- d: Distance between Tension Bar and Compression Side of Concrete Edge
- I: Moment of Inertia for RC Slab Section

2. Bending Moment **ML** and Stress of Reinforcement bar, Concrete and FRP Sheet due Live Load

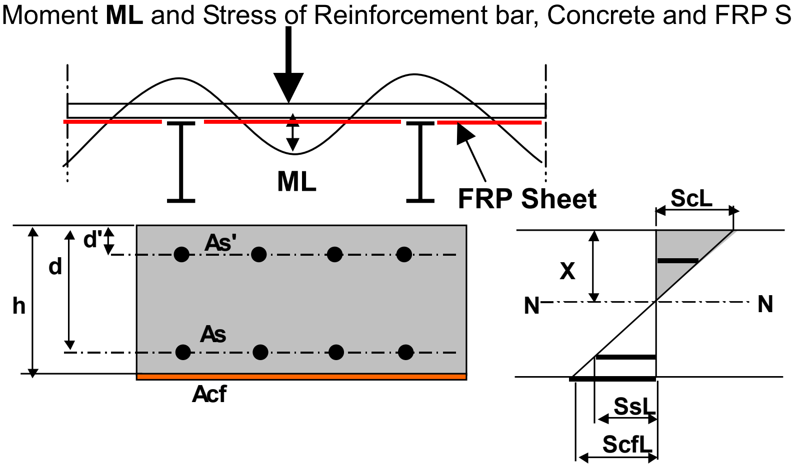


Figure 11.1.7. Moment and Stress due to Live Load

Stress of Concrete due to Live Load **ScL**: = ML x X / Ip
Stress of R-Bar due to Live Load **SsL**: = ML x n (d -X)/ Ip
Stress of FRP due to Live Load **ScfL**: = ML x nsp (h -X)/ Ip

Where,

- ML : Bending Moment due to Live Load
- nsp: Modulus Ration between Concrete and FRP
- Ip: Moment of Inertia for RC Slab Section with FRP

3. Stress of Reinforcement bar, Concrete and FRP Sheet due to Dead and Live Loads

All stresses of Concrete, Reinforcement bar and FRP Sheet shall be less than their allowable one respectively.

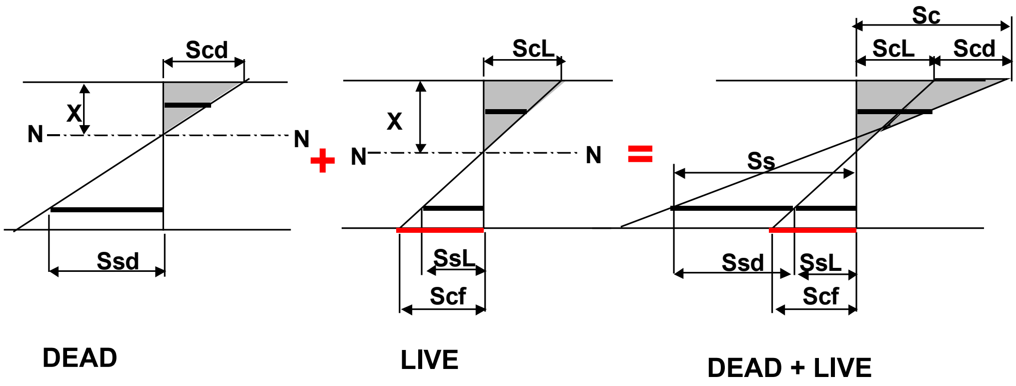


Figure 11.1.8. Stress due to Dead and Live Loads

Stress of Concrete due to Dead + Live Loads	**Sc** = **Scd** + **ScL** < **Sca**
Stress of R-Bar due to Dead + Live Loads	**Ss** = **Ssd** +**SsL** < **Ssa**
Stress of FRP due to Dead + Live Loads	**Scf= ScfL** **< Scfa**

Where,

Sca : Allowable Stress of Concrete
Ssa: Allowable Stress of Reinforcement bar
Scfa: Allowable Stress of FRP Sheet

11.1.4 Methodology of Design for Slab Reinforced by Slab thickness increase

The methodology of the design for reinforcing deck slab by the slab thickness increase method is to reduce the stress of the reinforcement bars inside of the existing deck slab by increasing the cross section of the deck slab. The original section has to resist for the forces due to the dead load including the additional dead load of the increased section. The increased section can only resist for the force due to the live load and the dead load of the pavement.

The stresses of reinforcement bars and concrete in the original section, which are occurred due to the dead load, remain even after the slab thickness is increased. Therefore, the stresses of original section can be reduced for only the live load as same as the FRP bonding method.

1) Design Flow for Slab Thickness Increase

The design flow for the slab thickness increase method is as shown in Figure 11.1.9. The stress of the reinforcement bar in the original slab desk, which is composite of the stresses due to both dead and live loads, shall be less than the allowable stress.

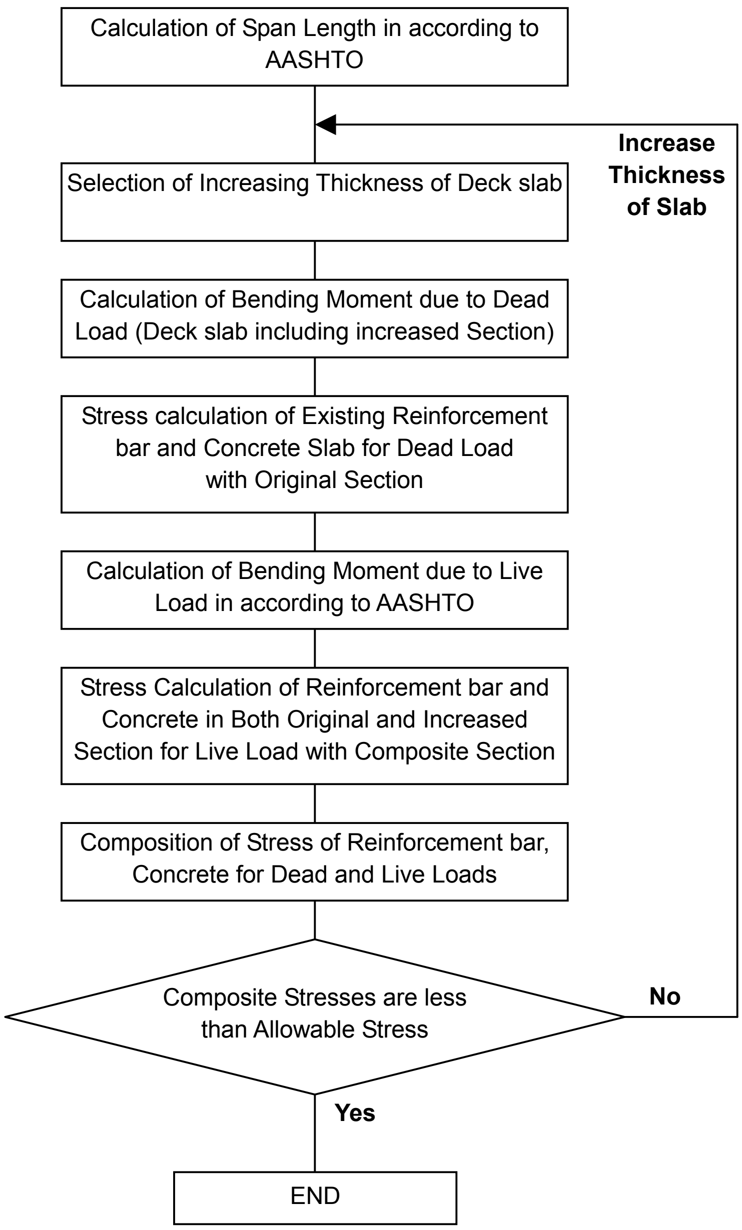


Figure 11.1.9. Design Flow for Slab Thickness Increase

2) Stress Check Method for Slab Thickness Increase

The stress check method for the slab thickness increase is as below.

1. Bending Moment **Md** and Stress of Reinforcement bar and Concrete due to **Dead Load** including the weight of increased slab weight.

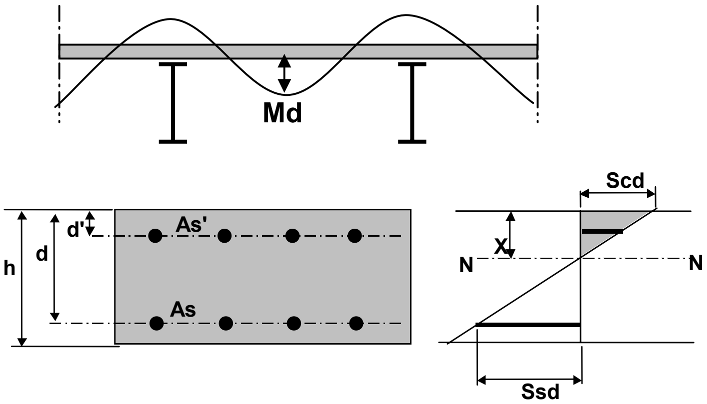


Figure 11.1.10. Moment and Stress due to Dead Load

Scd : Stress of Concrete due to Dead Load
Ssd : Stress of R-Bar due to Dead Load

2. Bending moment **ML** and stress of reinforcement bar and concrete due Live Load for Increased section.

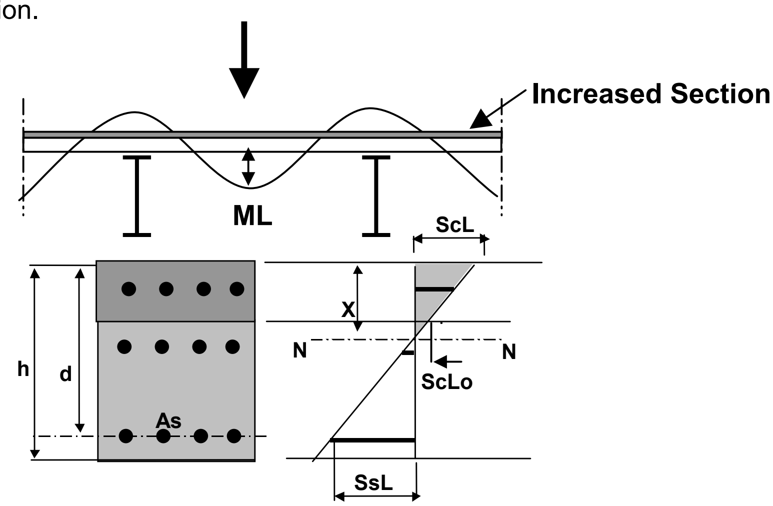


Figure 11.1.11. Moment and Stress due to Live Load

ScL : Stress of concrete at new surface due to live load
ScLo : Stress of concrete at original surface due to live load
SsL : Stress of R-bar due to live load

3. Composition of Stress of Reinforcement Bar and Concrete due to Dead and Live Loads

All Composite stresses of concrete and reinforcement bar and FRP sheet shall be less than their allowable one respectively.

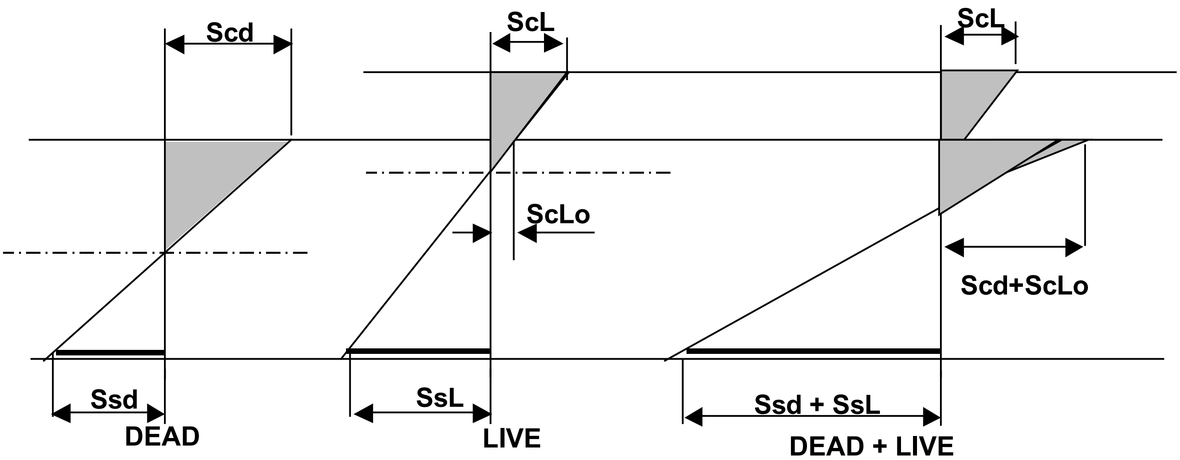


Figure 11.1.12. Stress due to Dead and Live Loads

Stress of Concrete at New Concrete Surface	**Scn** = **ScL** < **Sca**
Stress of Concrete at Original Concrete Surface	**Sco** = **Scd** + **ScLo** < **Sca**
Stress of R-Bar in Original Section	**Ss** = **Ssd** +**SsL** < **Ssa**

11.1.5 Existing Condition and Condition after Reinforcement

7 bridges except No.17 Chirripo Bridge, No.20 Sucio Bridge, and No.29 Torres are required reinforcement or replacement of the deck slab for the increased live load HS20 +25% and increased dead load of the asphalt pavement (5cm).

The reinforce methods and the stress conditions of both before and after reinforcement for 7 bridges are as shown in Table 11.1.6.

Table 11.1.6. Comparison of Slab Conditions

Bridge No.	Bridge Name	Existing Conditions for Increased Load				Conditions after Reinforcement						
		Slab Thickness (Mm)	Re-Bar area (cm^2/m)	Stress (N/mm^2)		Reinforce Method	Slab Thickness (mm)	Re-Bar area (cm^2/m	FRP Layers	Stress (N/mm^2)		
				Concrete (7.40))	Re-Bar (138)					Concrete (7.40)	Re-Bar (138)	FRP (633)
2	Aranjues	17.78	11.13	7.95	224	Replace-(PC Slab)	16.0	-	-	-	-	-
3	Abangares	16.51	11.13	6.14	173	Replace (PC Slab)	16.0	-	-	-	-	-
7	Azufrado	16.51	11.13	8.18	203	Increase	25.51	11.13	-	2.77	93	-
12	Puerto Nuevo (Steel Bridge)	17.78	11.13	7.50	211	FRP Bonding	17.78	11.13	1	6.04	136	478
	Puerto Nuevo (RC Bridge	16.51	11.13	7.73	203	FRP Bonding	16.51	11.13	1	6.11	127	476
16	Nuevo	16.51	11.13	8.18	203	Increase	25.51	11.13	-	2.77	93	-
19	Sarapiqui	17.0	11.13	5.93	163	FRP Bonding	17.0	11.13	1	4.90	114	390
26	Chirripo	17.0	14.49	6.76	136	FRP Bonding	17.0	14.49	1	5.85	121	426

11.2 Deck Support System

11.2.1 Method for Reinforcing Deck Support System

Several kinds of methods are applied for reinforcement of the deck support system depending on required capacity by reinforcement. The reinforcement methods for the deck support system are classified by their purpose as shown in Table 11.2.1.

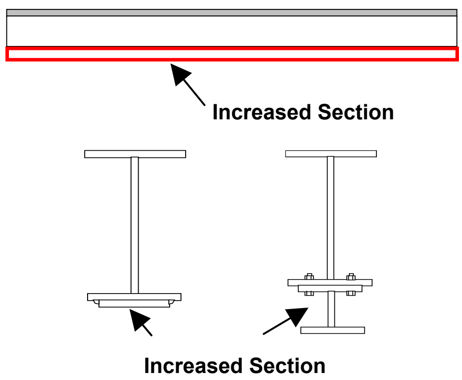
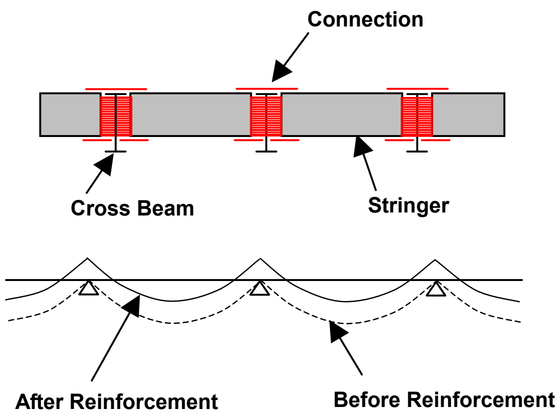
Table 11.2.1. Reinforcement Method for Deck support system

Purpose	Remedial Measure	Methods	Stringer	Cross Beam
Reduce Stress of Section	Increase Dimension	Steel Plate Thickness Increasing (Cover Plate)	App.	App.
		Member Section Increasing	App.	App.
Reduce Force of Section	Introducing of Pre-Stress	Change simple beams into Continuous	App.	-

App. : Applicable

Table 11.2.2 shows the principal examples of reinforcement method for deck support system.

Table 11.2.2. Reinforcement Method for Deck Support System

Reinforce Method	Figures	Description of Method
Increase Dimension - Steel Plate Thickness Increase - Member Section Increase		By increasing Inertia of section, to reduce the stress of original member. The consideration is especially required for the joints newly introduced, not to interfere with the existing members and materials.
Retrofit structural system -Change the load carrying system		Simple beams are connected together as a continuous beam. The forces (due to dead, live, etc.) can be decreased.

11.2.2 Selection of Reinforcement Method for Deck Support System

The truss type bridges are No.2 Aranjues Bridge and No.3 Abangares Bridge, which locate on Route 1. Only these bridges have the deck support system in ten bridges. As the design live load for these bridges is HS15, the stress of deck support system exceeds the allowable stress for the increased live load.

1) Stringer

The No.2 Aranjues Bridge and the No.3 Abangares Bridge have the same type of the stringer. The cracks on a connection plate and damages of a bolt are observed during the detailed inspection. These damages are basically caused by a fatigue of steel material.

The continuation method to change simple beam into continuous beam is adopted as the reinforcement method of the stringer. The reasons for adoption are shown below.

- Original condition is simple girder structure.
- Continuation of girder structure has high fatigue durability.
- The increase in a steel section is unnecessary, and there is no increase of dead load.

2) Cross Beam

The No.2 Aranjues Bridge and the No.3 Abangares Bridge have the same type of cross beam, too. The member section increase is adopted as the reinforcement method for the cross beam. The reasons for adoption are shown below.

- The increased stiffness of a girder decrease the secondary stress leading fatigue.
- The space under the floor beam is enough clearance.

11.2.3 Methodology of Design for Reinforcing Deck Support System

1) Stringer

(1) Design Flow for Stringer

The design flow for the reinforcing stringer by change simple beam into continuous method is shown in Figure 11.2.1.

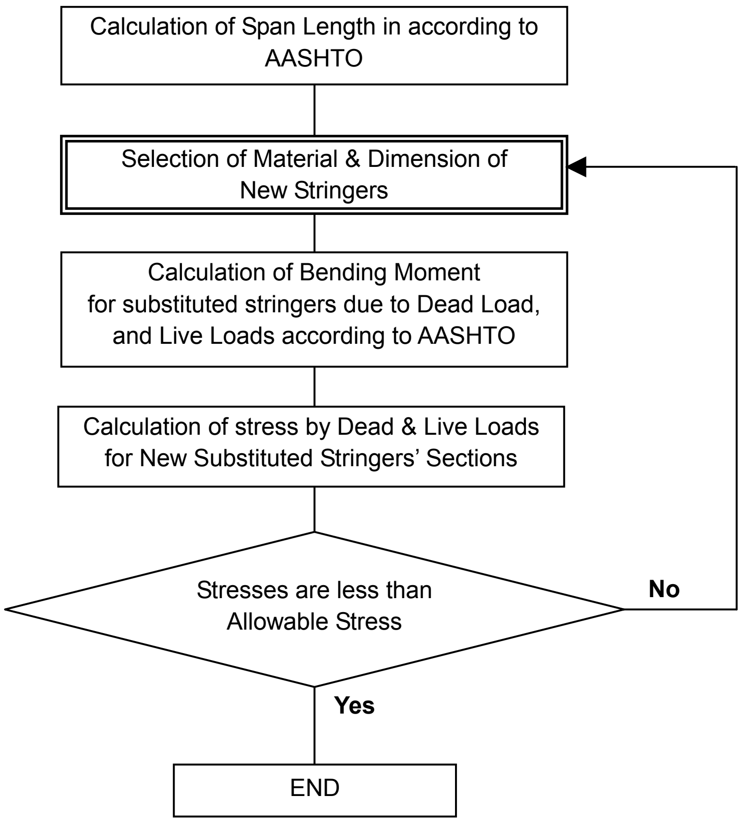


Figure 11.2.1. Design Flow for stringer by Beam Continuation Method

(2) Stress Check Method for Beam Continuation Method

1. Static anarisys for new structure
   For reinforcement the deck support system, the stringers are connecting together with fixing to the crossbeam. As shown in the figure below, the multiple simple beams are changed into a whole continuous beam. The static anarisys for this restricted frame is to be performed.

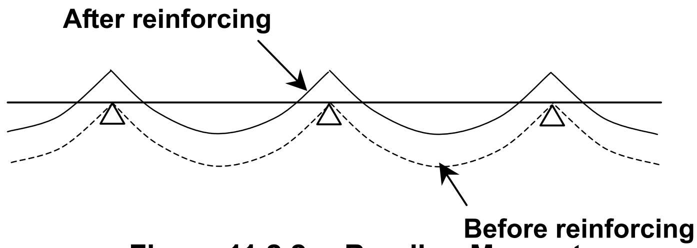


Figure 11.2.2. Bending Moment

2. Check stress for new additional members
   The procedure of the stress check is to be performed with the response as the result of the static anarisys. This procedure is almost in the same way of the newly designed structure. A certain difference is that the other existing members' materials and dimensions are already fixed.

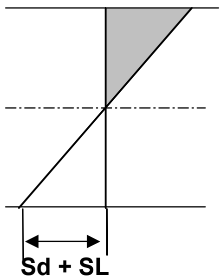


Figure 11.2.3. Stress due to Dead Load and Live Loads

The stress of new stringer due to the dead and live loads is

S = Sd + SL < Sa

Where, Sd : Bending stress by dead load
SL : Bending stress by live load
Sa : Allowable stress of existing member

2) Cross Beam

(1) Design Flow for Cross Beam

The design flow for the reinforcement of cross beam by the method of member section increase is as shown in Figure 11.2.4.

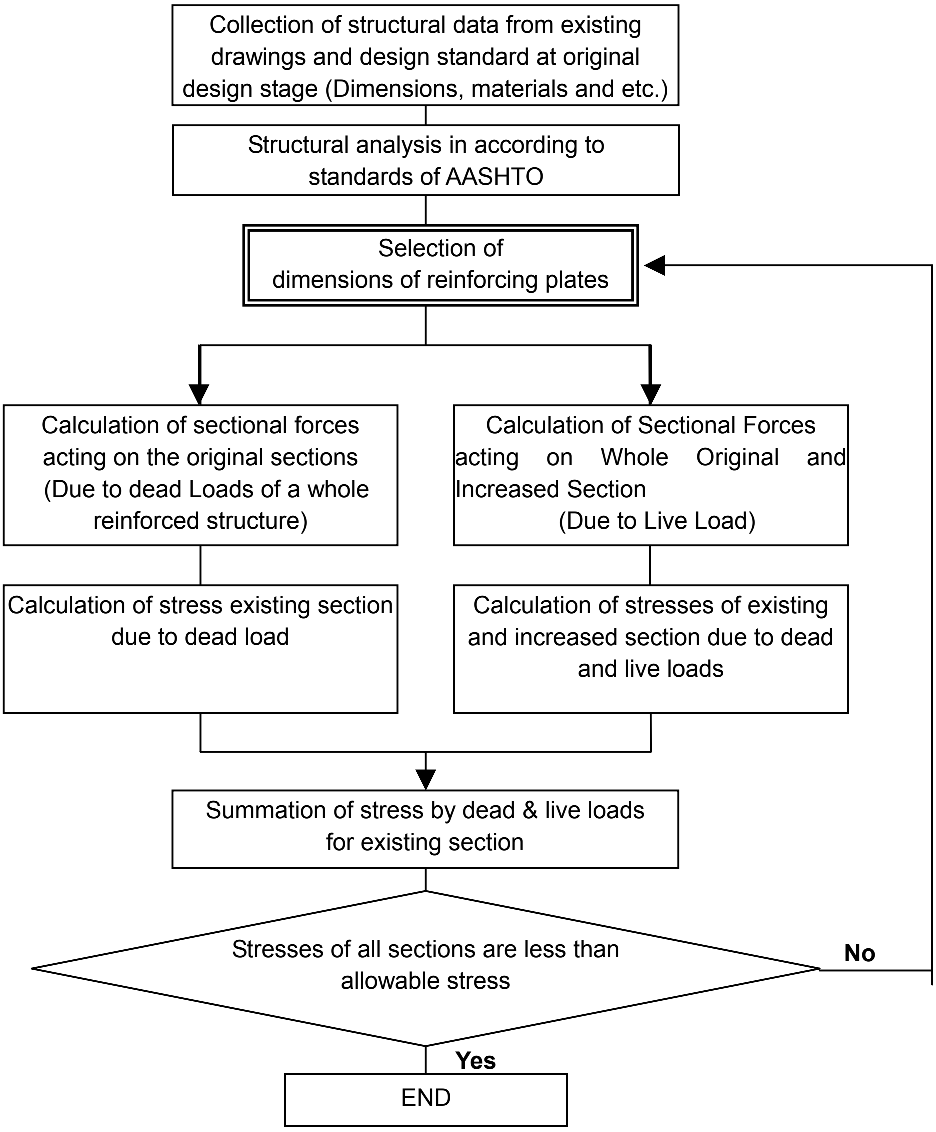


Figure 11.2.4. Design Flow for Cross Beam by Member Section Increase

(2) Stress Check Method for Member Section Increase

New H-beam is added downside of the existing cross beam or stringer for the reinforcement. The stresses due to dead and live loads are considered separately as follows:

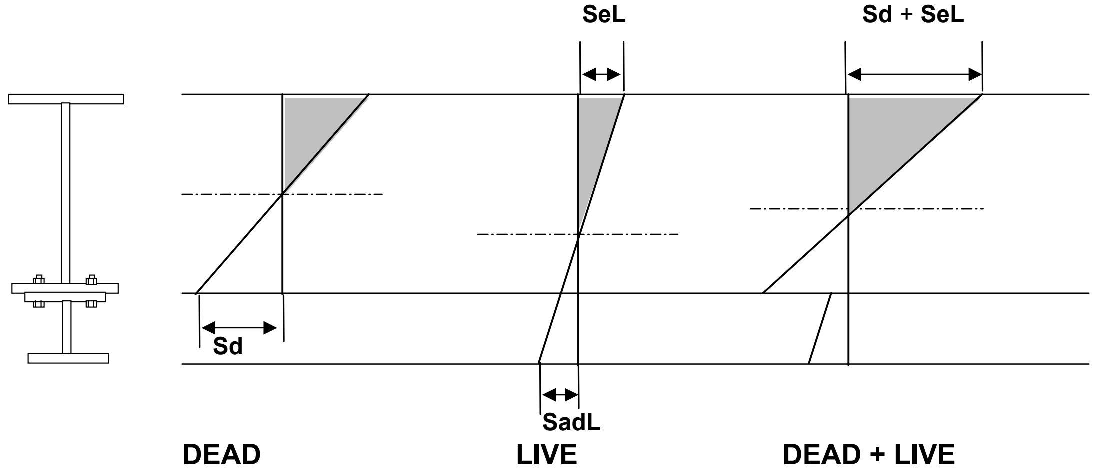


Figure 11.2.5. Stress due to Dead Load and Live Loads

Stress of existing section due to dead and live loads is

S = Sd + SeL < Se-a

Stress of added H-section due to live load is

S = SadL < Sad-a

Where, Sd : Bending stress by dead load
SeL, SadL : Bending stress by live load
Se-a : Allowable stress of existing member
Sad-a : Allowable stress of additional H-beam

11.2.4 Existing Condition and Condition after Reinforcement

The stresses before and after reinforcement are shown in the following tables.

1) Stringer

The stringers of No.2 Aranjuez Bridge and No.3 Abangares Bridge do not have enough load capacity for the increased live load. The stringers of both bridges are reinforced by the continuation method of simple beam, and it was confirmed that they have enough load capacity for the live load HS20+25%.

Table 11.2.3. Comparison of Stringer

Bridge No.	Bridge Name	Existing Conditions			Conditions after Reinforcement		Allowable Stress (N/mm^2)
		Stress Over Position		Stress (N/mm^2)	Reinforcement Method	Stress (N/mm^2)	
2	Aranjuez	Center of Span	In.	143	Continuation	116	125
			Ex.	131		86	125
		At Support	In.	-		64	125
			Ex.	-		74	125
3	Abangares (129ft)	Center of Span	In.	166		125	125
			Ex.	166		114	125
		At Support	In.	-		65	125
			Ex.	-		79	125
	Abangares (200ft)	Center of Span	In.	174		125	125
			Ex.	174		100	125
		At Support	In.	-		63	125
			Ex.	-		84	125

Note: In.; Internal Beam, Ex.; External Beam

2) Cross Beam

The existing cross beams of both No.2 Aranjuez Bridge and No.3 Abangares Bridge do not have enough load capacity for the increased live load. To increase the load capacity of cross beams for the live load HS20+25%, the member section increase with H-shaped steel was applied for them. The types of H shaped steels used for the member section increase are as bellows for on each bridge.

No.2 Aranjuez: W12x65
No.3 Abangares(129ft): W14x74
No.3 Abangares(200ft): W14x90

Table 11.2.4. Comparison of Cross Beam

Bridge No.	Bridge Name	Existing Conditions		Conditions after Reinforcement		Allowable Stress (N/mm^2)
		Stress Over Position	Stress (N/mm^2)	Reinforcement Method	Stress (N/mm^2)	
2	Aranjuez	Center of Span	141	Member Section Increasing	110	125
3	Abangares (129ft)		141		118	125
	Abangares (200ft)		160		125	125

11.3 Main Girder

11.3.1 Method for Reinforcement for Main Girder

Several kinds of method are applied for the reinforcement for the bridge main girders depending on the bridge type and required load capacity by reinforcement. The reinforcement methods for the bridge main girder are classified by their purposes as shown in Table 11.3.1.

Table 11.3.1. Reinforcement Method for Main Girder

<table>
<tr><th>Purpose</th><th>Remedial Measure</th><th>Methods</th><th>RC Bridge</th><th>PC Bridge</th><th>Steel Bridge</th></tr>
<tr><td rowspan="5">Reduce Stress of Section</td><td rowspan="3">Increase Dimension</td><td>Girder Height Increase</td><td>App.</td><td>-</td><td>-</td></tr>
<tr><td>Steel Plate Thickness Increase (Cover Plate)</td><td>-</td><td>-</td><td>App</td></tr>
<tr><td>Member Section Increase</td><td>-</td><td>-</td><td>App</td></tr>
<tr><td rowspan="2">Reinforce by Additional Material</td><td>Steel Plate Bonding</td><td>App.</td><td>App.</td><td>-</td></tr>
<tr><td>FRP Bonding (Carbon Fiber, other)</td><td>App.</td><td>App</td><td>-</td></tr>
<tr><td rowspan="3">Reduce Force of Section</td><td>Built Additional Member</td><td>Member Addition</td><td>App.</td><td>-</td><td>App</td></tr>
<tr><td>Introducing of Pre-Stress</td><td>External cable Addition</td><td>App</td><td>App.</td><td>App.</td></tr>
<tr><td>Additional Support</td><td>Support Addition</td><td>App</td><td>App.</td><td>App.</td></tr>
</table>

Note: App. ; Applicable

Table 11.3.2, 11.3.3, 11.3.4, and 1.3.5 show the principal examples of reinforcement method for RC deck bridge, PC I beam bridge, PC box girder bridge and steel I beam bridge, respectively.

Table 11.3.2. Reinforce Methods for RC Deck Bridge

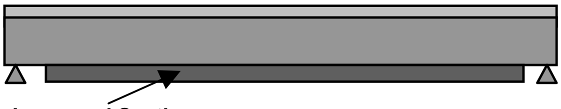
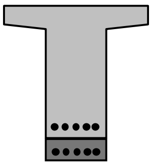
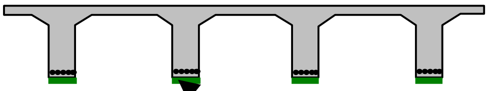
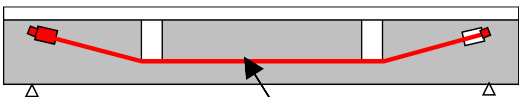
Reinforcement Method	Figures and Photos	Description of Method
Increase Dimension - Girder Height Increase	  	By increasing stiffness of RC girder, to reduce the stress of existing reinforcement bar and concrete. Dead load of girder is increased
Reinforce by Additional Material -Steel Plate Bonding -FRP Bonding	  	By distributing stress to additional material, to reduce the stress of existing reinforcement bar. Increasing of weight by this method is small and the weight of FRP sheets are negligible. Bonding work of FRP is easer then that of steel plate.
Introducing of Pre-Stress -External cable Addition	 	Effective for reduce the stress of reinforcement bar, but the stress of concrete will be increased. Traffic suspension is not necessary. Most Costly.

Table 11.3.3. Reinforce Methods for PC I Beam Bridge

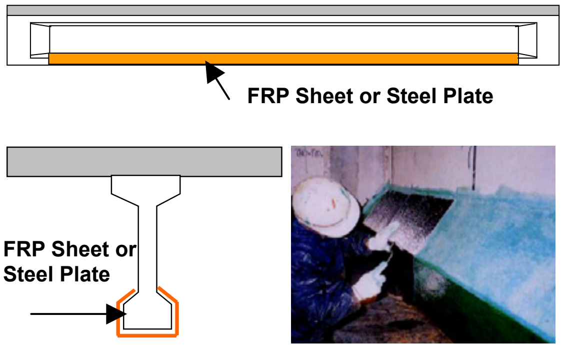
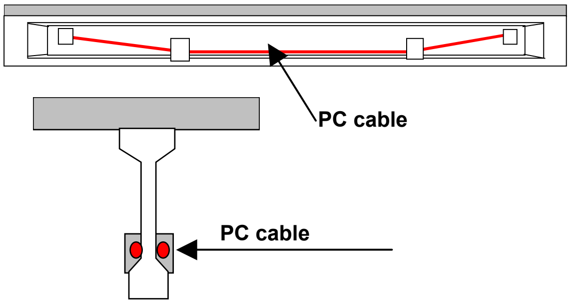
Reinforcement Method	Figures	Description of Method
Reinforce by Additional Material -Steel Plate Bonding -FRP Bonding		By distributing stress to additional material, to reduce the stress of reinforcement bar. Increasing of weight by this method is small and the weight of FRP sheets are negligible. Bonding work of FRP is easer then that of steel plate.
Introducing of Pre-Stress -External cable Addition		Effective to reduce the tensile stress of concrete, but the completion stress of concrete increase. Traffic suspension is not necessary. Costly.

Table 11.3.4. Reinforce Methods for PC Box Bridge

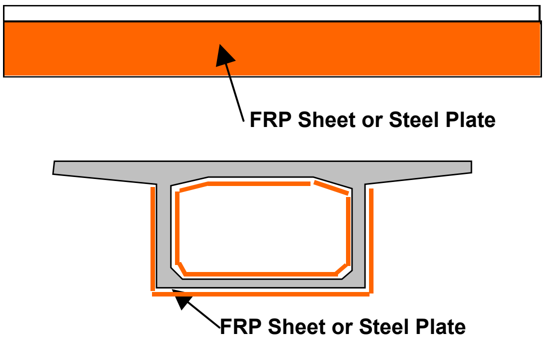
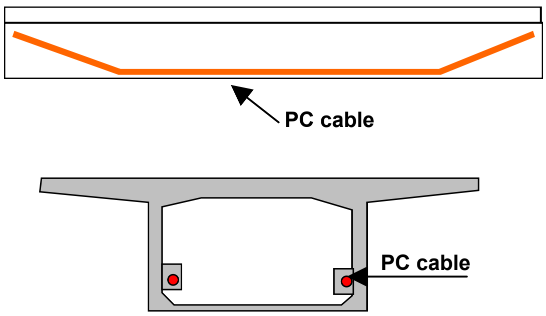
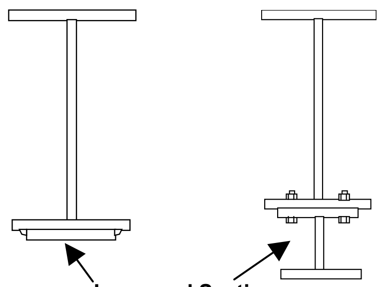
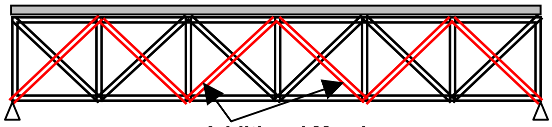
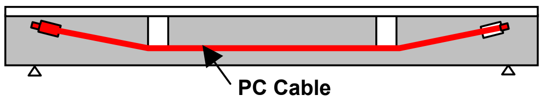
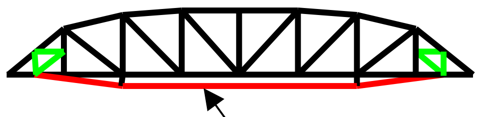
Reinforcement Method	Figures	Description of Method
Reinforce by Additional Material -Steel Plate Bonding -FRP Bonding		By distributing stress to additional material, to reduce the stress of reinforcement bar. Increasing of weight by this method is small and the weight of FRP sheets are negligible. Bonding work of FRP is easer then that of steel plate. Both methods are used to increase the load capacity for the shear force and reinforce the top deck slab.
Introducing of Pre-Stress -External cable Addition		Most effective and common method for PC Box Girder to reduce the tensile stress of concrete, but the compression stress increase. Traffic suspension is not necessary. Costly.

Table 11.3.5. Reinforce Methods for Steel Bridge

Reinforce Method	Figures and Photos	Description of Method
Increase Dimension - Steel Plate Thickness Increase - Member Section Increase	 	By increasing inertia of section, to reduce the stress of original section. For connection parts, special arrangement required to avoid bolts Dead load of girder increases
Built Additional Member -Member Addition	 	By distributing stress to additional members, to reduce the stress of original member.
Introducing of Pre-Stress -External cable Addition	  	Effective for reducing the stress of member, but reinforcement is required at anchorage of cable. Most Costly.

11.3.2 Selection of Reinforcement Method for Main Girder

1) Truss Bridge

The truss type bridges are No.2 Aranjues Bridge and No.3 Abangares Bridge, which located on Route 1. As the design live load for these bridges was HS15, the stress of some parts of member due to the increased live load exceeds the allowable stress.

(1) No.2 Rio Aranjues Bridge

For the increased live load, the bottom chord, vertical member, and diagonal member near the support on the piers exceed the allowable stress. To increase the section of member is impossible due to their shape and connecting conditions. It is also impossible to introduce the prestress to the continuous truss type bridge.

Therefore, only the member addition method can be applied for this bridge. However, by addition of member for reinforcing the over stressed member, the members, which are commented by the additional member, increase their stresses and finally whole length of the bridge is required to be reinforced by the member addition as shown in Figure 11.3.1.

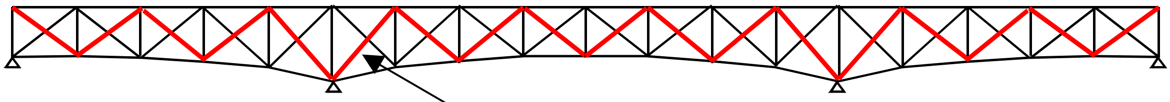


Figure 11.3.1. Additional Member for Reinforcement of Aranjues Bridge

(2) No.3 Rio Abangares Bridge

The stress of the bottom chord member of the 129 ft long truss bridge exceed the stress for HS20 +25% live load. The member of 200 ft long truss bridge does not need to be reinforced. These over stressed members of 129ft long truss bridge are can be reinforced by the steel plate fixing.

Therefore, the steel plate fixing method was selected as a most simple method for this bridge.

2) Steel I Beam Bridge

The steel I beam type bridges existed in the objective 10 bridges are No.12 Puerto Nuevo on Route 2, No.19 Sarapiqui Bridge on Route 4 and No.26 Chirripo Bridge on Route 32.

The deign live load is HS20 for the Chirripo bridge and HS15 for the other bridge. The two bridges designed for HS15, do not have enough capacity for the increased live load. Only the Chirripo Bridge has an enough load capacity for the live load.

(1) No.12 Rio Puerto Nuevo Bridge

The Puerto Nuevo Bridge consists of 3 simple spans steel I beam, 1 simple span composite steel I beam, and 1 simple span RC I beam. The required reinforcement of girder exceeds the level, which can be reinforced by increasing member section. Therefore, the external cable method was selected for reinforcing the main beam.

(2) No.19 Rio Sarapiqui Bridge

The Sarapiqui Bridge is 3 spans Gerber beam type bridge. As same reason as the Puerto Nuevo Bridge, the girders cannot be reinforced by increasing of member section. The external cable method was selected for this bridge too.

(3) No.26 Rio Chirripo Bridge

The main girder of the Chirripo Bridge has enough load capacity for the increased live load, any reinforcements are not required for the main girder.

3) RC Deck Bridge

The No.7 Rio Azufrado Bridge, the fifth span (between P4 pier and A2 abutment) of the No.

12 Rio Puerto Nuevo Bridge, and No.16 Rio Nuevo Bridge are the RC deck type bridge. They are constructed more than 35 years ago and designed for the HS15 live load. The stress of reinforcement bar in the main gilder exceeds its allowable stress for the increased live load HS20 + 25%. And the reinforcement for the shear stress also required.

(1) No.7 Rio Azufrado Bridge

This bridge type is 3 spans continuous rigid frame RC bridge. The load capacity of the main girder does not satisfy for the increased live load both the positive moment at span center and the negative moment at support. The deck slab of this bridge is also required to be reinforced for the increased live load. The reinforcement method of the main girders was considered taking account of the reinforcement for the deck slab.

The slab thickness increase method on topside and the FRP bonding method are applicable for the deck slab of this bridge as shown in Table 11.3.6. The slab thickness increase method is selected from the effectiveness as a part of main girder and cost.

Table 11.3.6. Comparison of Reinforcement for Deck slab of No.7 Azufrado Bridge

Reinforcement Method	Slab Thickness Increase		Bonding		New Girder Addition
	Bottom Side	Top Side	FRP	Steel Plate	
For Negative Moment	No	OK	OK	No	OK
Usage as girder section	OK	OK Effective	No	No	No Not applicable as rigid frame
Increase of dead load	Yes Large	Yes Large	No	Yes Small	Yes Medium
Traffic Suspension	No	Yes	Yes	No	No
Cost	Low	Low	High	High	Medium
Evaluation	NO	Good	Fair	No	No

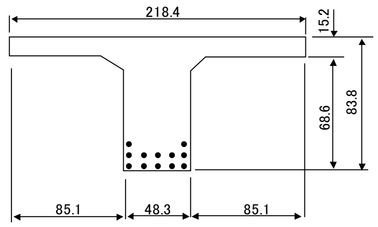
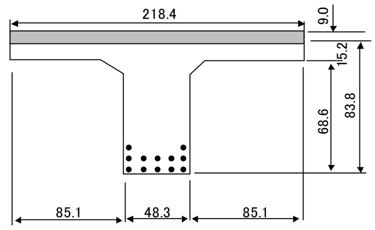
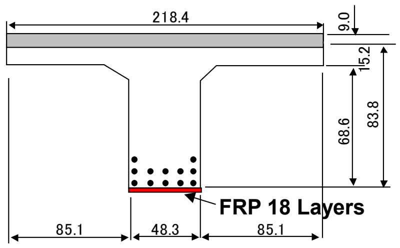
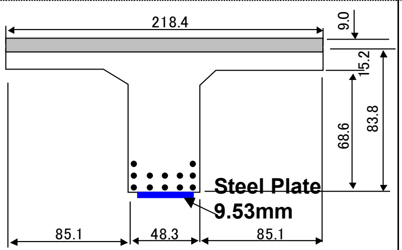
The applicable reinforcement methods for the RC main girder was considered taking account of the increased deck slab, which is effective as a flange of girder, for both the positive moment and negative moment as shown in Table 11.3.7.

Table 11.3.7. Preliminary Comparison of Reinforcement for Main Girder of No.7 Azufrado Bridge

Reinforcement Method	For Positive Moment	For Negative Moment	Remarks
Section Increase on bottom side	OK	OK	Dead load Increase
FRP Bonding	OK	No	Can not use at rigid area with pier
Steel Plate Bonding	OK	No	Can not use at rigid area with pier
External cable Addition	Applicable	Applicable	Costly

For the positive moment area, four types of reinforcement method can be applied as listed above table. However, as the section increase method increases the dead load more and increases both positive and negative moments. And the external cable addition is costly. The FRP bonding method and the steel bonding method are compared as shown in Table 11.3.8.

Table 11.3.8. Comparison of Reinforcement for Main Girder of No.7 Azufrado Bridge

Reinforcement Method	Section	Size of Bonding Material	Stress for HS20 +25% (N/mm^2)			
			Concrete (7.4)	Re- Bar (138)	FRP Sheet (633)	Steel Plate (141)
Original Section		-	5.7	227	-	-
Slab Thickness Increase (9 cm)		-	3.6	204	-	-
Slab Thickness Increase (9 cm) + FRP Bonding		**18 Layer** x 484 mm	3.0	136	277	-
Slab Thickness Increase (9cm) + Steel Plate Bonding		**3"/8 (9.53mm)** x 400 mm	3.0	134	-	90

The 18 layers of FRP sheets are required for the FRP bonding method. However, normally the maximum number of layers is 10 and there are no experiences used 18 layers. It may happen problems about the bonging of sheets in future. Therefore, The steel plate bonging method shall be selected as the reinforcement method for both bending moment and shear force.

For the negative moment at the support, the increase girder height on bottom side is only one method for reinforcement. The height of 30 cm is required to satisfy the allowable stress at the support.

(2) No.12 Rio Puerto Nuevo Bridge

The Rio Puerto Nuevo Bridge consist of 4 simple spans of steel I girder and one simple RC girder. The load capacity of the main girder does not satisfy for both bending moment and shear force due to the increased live load. The deck slab of this bridge is also required to be reinforced for the increased live load. The FRP bonging method was selected for the deck slab to meet the method for the steel girder section.

The increase girder height method is not applicable, because enforcing capacity for bending moment becomes large-scale and not effective for shear force. The external cable addition method is not easy for the construction either and costly. Therefore, the FRP bonding method and the steel plate bonding method were compared as shown in Table 11.3.9

Table 11.3.9. Comparison of Reinforcement for Main Girder of No.12 Puerto Nuevo Bridge

Reinforcement Method	Size of Bonding Material	Stress for HS20 +25% (N/mm^2)				Evaluation
		Concrete (7.4)	Re- Bar (138)	FRP Sheet (633)	Steel Plate (141)	
Original Section	-	4.3	155	-	-	-
FRP Bonding	**16 Layer** x 484 mm	4.5	138	240	-	NO (More than 10 Layer)
Steel Plate Bonding	**3”/8 (9.53mm)** x 400 mm	4.4	136	-	75	OK

The 16 layers of FRP sheet cannot be applicable. Therefore, the steel Plate Bonding method was selected for this bridge.

(3) No.16 Rio Nuevo Bridge

This bridge type is 3 spans continuous RC bridge as similar as the No.7 Azufrado Bridge. The load capacity of the main girder does not satisfy for the bending moment (both the positive moment at center of spans and the negative moment at support) and shear force due to the increased live load. The deck slab of this bridge is also required to be reinforced for the increased live load. The reinforcement method was considered for the deck slab and main girder together.

The deck slab increase method is applied as same reasons as the Azufrado Bridge. By increasing of deck slab, the stresses both at the center of center span and at support satisfy the allowable stress. However, the stress near the center of side span does not satisfy the allowable stress. The member section increase method is not effective for the reinforcement of shear stress. As the required number of FRP layers is 6, the FRP bonding method can be applied and it is easier than the steel bonging for the bonging works. Therefore, the FRP bonding method was selected for this bridge as shown in Table 11.3.10.

Table 11.3.10. Comparison of Reinforcement for Main Girder of No.16 Nuevo Bridge

Reinforcement Method	Size of Bonding Material	Stress for HS20 +25% (N/mm^2)				Evaluation
		Concrete (7.4)	Re- Bar (138)	FRP Sheet (633)	Steel Plate (141)	
Original Section	-	5.0	156	-	-	-
FRP Bonding	**6 Layer** x 484 mm	3.2	136	283	-	Good
Steel Plate Bonding	**3”/8 (9.53mm)** x 400 mm	3.1	129	-	85	Fair

4) PC I Beam Bridge

The Rio Torres consists of two kinds of span length of the PC simple I Beam. The long bridge with 30 m span is over in the bending stress of girder for the increased live load HS20 +25%, on the other hand, the short bridge with 17m span is not over in the bending stress. The shear stress of the girder exceeds the allowable stress for both bridges. The slabs of both bridges have enough load capacity for the increased live load.

The actual arrangement of PC cables in the girder are not shown in the drawings, the line of required position of the center of gravity for cables is only shown. This means that it is very dangerous to hole the girder for reinforcing, especially on the bottom side girder near center of the beam. In taking account with this situation, for the reinforcement methods for this PC I girder, only the FRP bonding and the external cable addition methods are can be applicable as shown in Table 11.3.11.

Table 11.3.11. Comparison of Reinforcement Method for PC I Beam

Reinforcement Method	Effectiveness	Risk for damage to PC Cables	Cost	Applicable
Increase Section	No	Large	Low	No
Steel Plate Bonding	Yes	Large	Medium	No
FRP Bonding	Yes	Small	Medium	Yes
External cable Addition	Yes	Medium	High	Fair

As the results of stress calculation, 4 layers of FRP sheet are required to satisfy the allowable stress of the reinforcement bar in the tensional area as shown in the Table 11.3.12. Therefore, the FRP bonding method was selected for this bridge.

Table 11.3.12. Result of Stress by FRP Bonding Method for Main Beam of No. 29 Torres Bridge

Reinforcement Method	Size of Bonding Material	Stress for HS20 +25% (N/mm^2)				Evaluation
		Concrete		Re- Bar (138)	FRP Sheet (633)	
		Completion (7.4)	Tension (-2.9)			
Original Section	-	5.0	-3.7	-	-	-
FRP Bonding	**4 Layer** x 484 mm	3.2	-	118	379	OK

5) PC Box Girder Bridge

(1) Stress of PC Box Girders

As the results of structural analysis, both of No. 17 Chirripo Bridge and No.20 Sucio Bridge have the load capacity for increased live load HS20 + 25% as shown in Table 11.3.13. No reinforcement is required for these bridges.

Table 11.3.13. Stress for PC Box Girders of No.17 Chirripo Bridge and No.20 Sucio Bridge

	Concrete Stress for HS20 +25% (N/mm²)							
	At Center of Span				At Support			
	Top		Bottom		Top		Bottom	
	Max	Min	Max	Min	Max	Min	Max	Min
No.17 Chirripo Bridge	3.6	-0.3	9.0	3.0	11.8	10.0	8.0	5.9
No.20 Sucio Bridge	4.1	-0.5	9.3	2.4	12.1	9.8	7.0	5.7
Allowable Stress	Completion		13.7					
	Tension		- 2.9					

(2) Deflection of PC Box Girders

The abnormal deflections are observed both No.17 Chirripo Bridge and No.20 Sucio Bridge. The maximum deflections at the central span comparing with the height at middle support are about 20 cm and 27 cm on the Chirripo Bridge and the Sucio Bridge respectively.

Figure 11.3.2 shows the comparison between the present elevation of bridge deck and its planned elevation in the original drawing existed for No.20 Chirripo Bridge. The deference between the actual level and planned level at the center of central span is about 30cm and even the present heights at the piers of P1 and P2, where the height is normally never changed, are not same as their planned heights. It is considered that the bridge was not constructed as the planned elevation in the existing drawing. However, this is an assumption because of no as-built drawing or construction records at present.

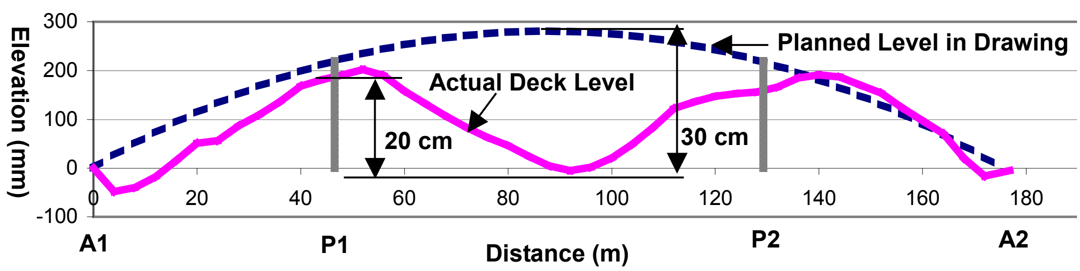


Figure 11.3.2. Deflection of No.17 Chirripo Bridge

Figure 11.3.3 shows the comparison between the present elevation of bridge deck and its planned elevation in the original drawing existed for No.20 Sucio Bridge. The figure is drawn with assuming as the deck is a level, because the planned vertical alignment of the bridge is located in the steep incline section. The deference of height between at the pier P1 and at the center of central span is about 27cm and even the present height at the pier of P1 is not same as it planned heights.

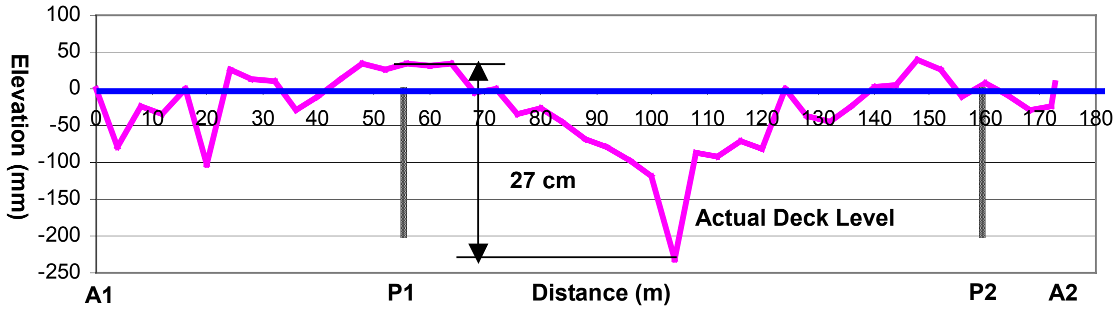


Figure 11.3.3. Deflection of No.20 Sucio Bridge

i) Causes for Deflection

The prestressed concrete girder is deformed by several forces both during construction period and after completion. Normally, these deformations are considered for setting camber during construction works. Only the deflection due to the stress loss of PC cables is occurred after the completion. The normal stress loss of PC cables due to creep, elastic shrinkage of concrete and relaxation of PC cable, is considered about 13 % of the installed stress as described in AASHTO. The deflection due to this prestress loss is only 20 mm for Chirripo Bridge and Sucio Bridge. Table 11.3.14 shows the deflections due to each force calculated by structural analysis for two bridges.

Table 11.3.14. Deflections at Center due to Several Forces

Forces		Deflection (mm)	
		No.17 Chirripo Bridge	No.20 Sucio Bridge
Dead load of main girder		-166	-248
Prestress of cantilever cable		+ 99	+119
Prestress of continuous cable		+61	+68
Superimposed dead load		-29	-27
Stress loss due to creep and others	**During Cantilever Construction**	**-12**	**-22**
	After connection of Center	**- 8**	**-8**
Total	With loss during Construction	-43	-118
	Without loss during Construction	-55	-96

The comparisons between actual defection and defection due to stress loss are shown in Figure 11.3.4 and 11.3.5 for Chirripo Bridge and Sucio Bridge, respectively. The deflection due to the stress loss is very small compared with the actual deflection. Therefore, it is considered that this abnormal deflection is caused not only by the stress loss of PC cables.

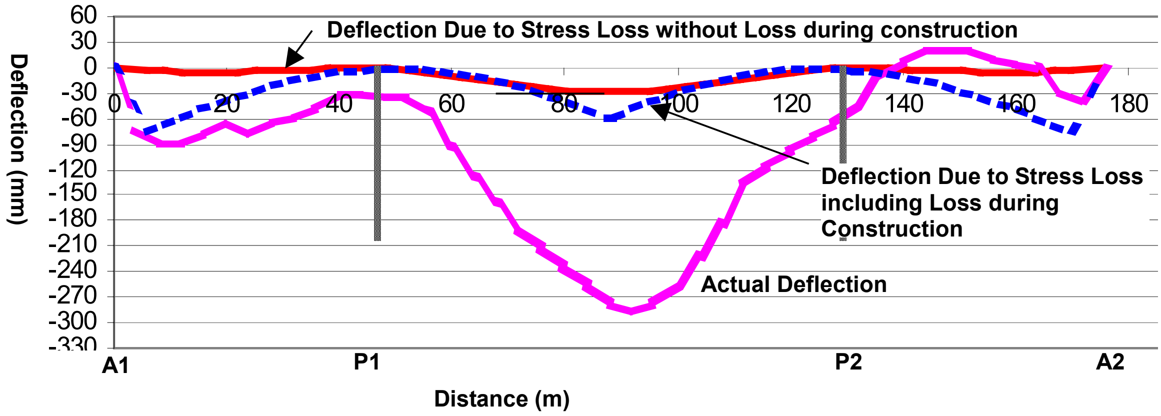


Figure 11.3.4. Comparison of Deflection for No.20 Sucio Bridge

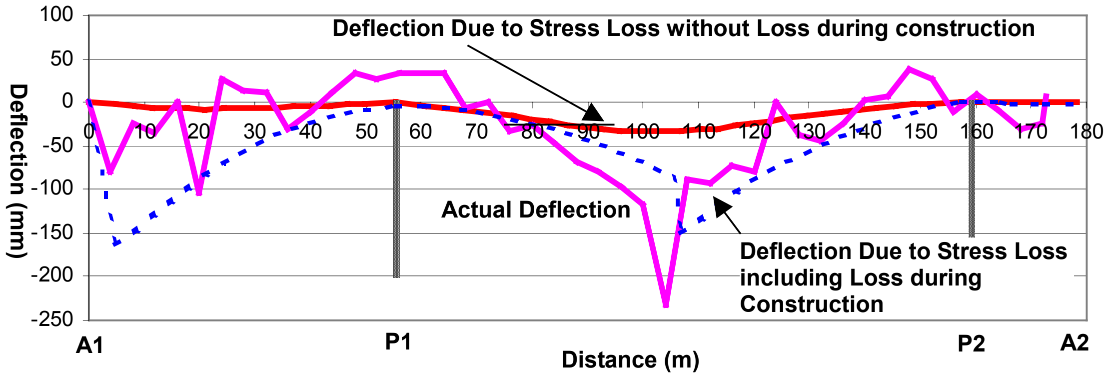


Figure 11.3.5. Comparison of Deflection for No.20 Sucio Bridge

The other potential causes of deflection for the prestressed structure are as follows;

- Lack of concrete box girder strength
- Decrease in the elastic modulus of concrete box girders
- Lack of the introduced tension of PC cables
- Abnormal progress in concrete creep
- Insufficient quality control during construction
- Increased dead weight due to asphalt pavement

ii) Investigation of Bridge Condition

The results of investigations into the above-mentioned potential causes of deflection are described below.

(a) According to the detailed inspection of the two bridges conducted by th the following can be stated;

- There is no abnormal cracking.
- There is no abnormal carbonation.
- There is no abnormal vibration due to vehicles passing.

(b) According to a concrete core boring test for No.20 Rio Sucio Bridge and a Schmidt Hammer test for two bridges, the following can be stated;

- The concrete strength of the bridge bodies satisfies required levels.
- The elastic modulus of boring core specimens satisfies required levels.

(c) According to the structural analysis and loading test for No.17 Rio Chirripo Bridge, the following can be stated;

- Required stiffness was confirmed and is the same as the drawings, which is understood in accordance with good coincidence of the results of structural analysis and loading test.

iii) Counter Measures

The above analyses indicate that the both bridges satisfy the necessary structural requirements, except for the deflections observed at the central spans, and will therefore be able to serve traffic in safely.

It is thought that the observed deflections are due to quality control during construction. However, this is an assumption as there are no original construction plans and construction records available. Note that the load capacity of the bridges for HS20+25% was checked in the structural analysis using a model based on original design drawings.

Given the preceding, It is considered that the bridges can serve the public safely in the future. However, it is recommend that the deflections of these bridges be measured for change at least once every year.

11.3.3 Methodology of Design for Reinforcing Main Girder

1) Truss Bridge

(1) Design Flow for Truss Bridge

a) Diagonal Member Addition

The design flow for the steel truss bridge, which is reinforced by the diagonal member addition methods, is as shown in Figure 11.3.6. The basic methodology of the reinforce design is to reduce the stress of original member and to increase load capacity.

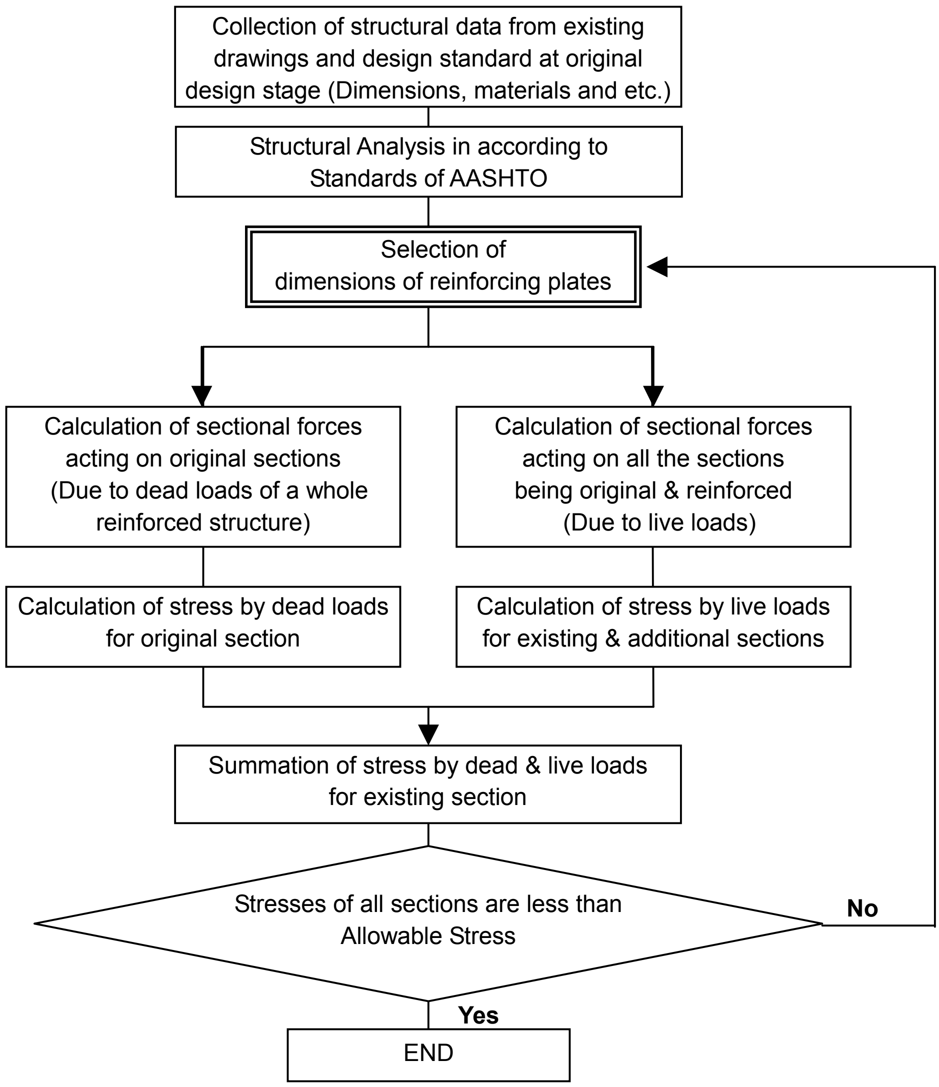


Figure 11.3.6. Design Flow for Steel Truss Bridge Reinforced by Diagonal Member Addition

b) Member Section Increase

The design flow for the steel truss bridge, which is reinforced by the member section increase methods, is as shown in Figure 11.3.7. The basic methodology of the reinforce design is that the tensile stress in the section shall be resisted by the existing steel in side the tensile area of the girder and the increased section.

The stress of the original girder section, which is composite of the stresses due to both dead load and live loads, shall be less than the allowable stress.

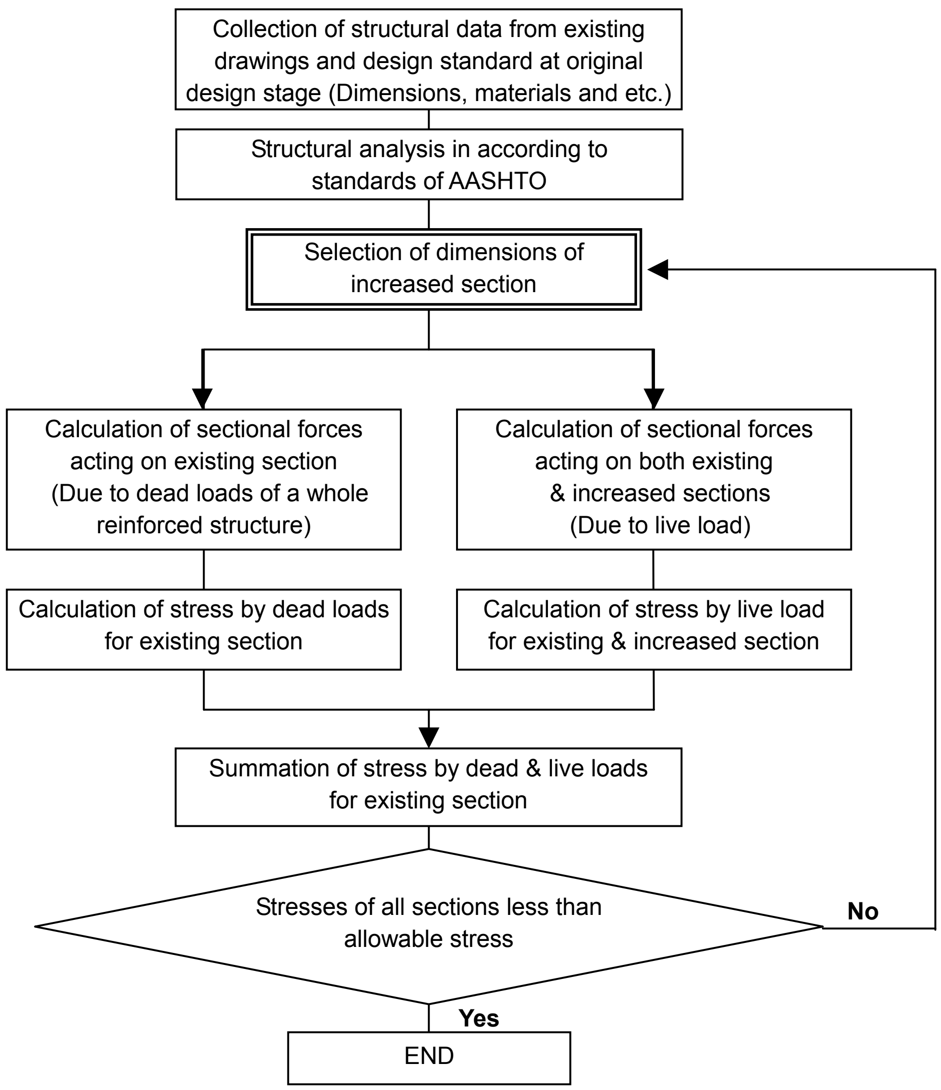


Figure 11.3.7. Design Flow for Steel Truss Bridge Reinforced by Member Section Increase

(2) Stress Check Method for Member Addition or Member Section Increase

a) Member Addition

The existing members can resist to both dead and live loads and the additional member can resist only to live load, as follows:

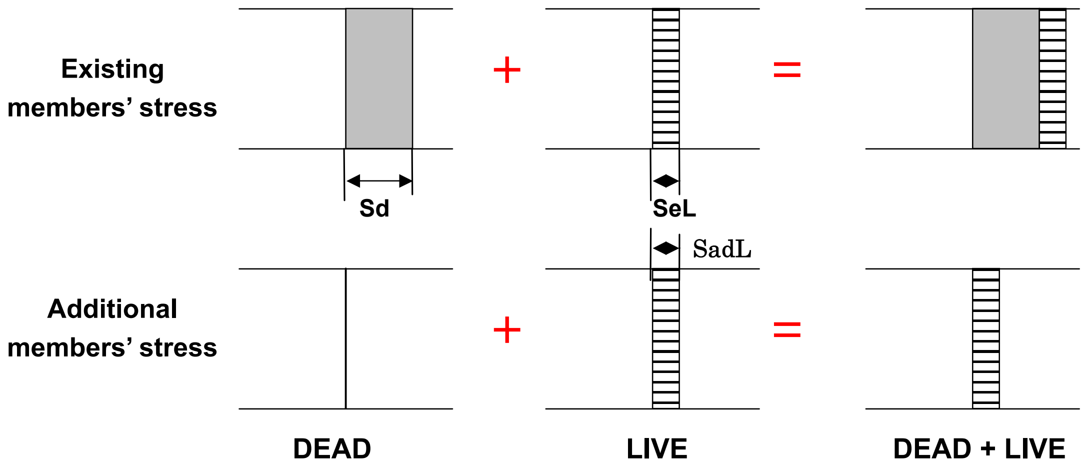


Figure 11.3.8. Stress due to Dead Load and Live Loads

Stress of existing member due to dead + live loads

Se = Sd + SeL < Se-a

Stress of additional member due to live load

Sad = SadL < Sad-a

Where, Sd : Stress by dead loads' axial force
SeL, SadL : Stress by live loads' axial force
Se-a : Allowable stress of existing member
Sad-a : Allowable stress of additional bracing

b) Member Section Increase

In the method of member section increase, The stress due to the dead loads and the stress due to live load shall be considered separately. Dead loads of a whole structure, which includes both existing and additional sections, act on only original section, while live load acts on increased section. The stress checking for original sections is mainly performed as follows:

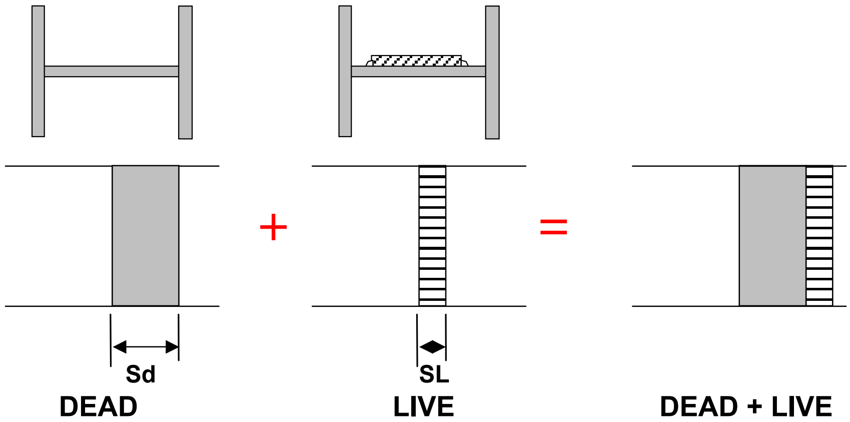


Figure 11.3.9. Stress due to Dead Load and Live Loads

Stress of existing section due to dead + live loads

S = Sd + SL < Sa

Where, Sd : Stress by dead loads' axial force
SL : Stress by live loads' axial force
Sa : Allowable stress of existing material

2) Steel I Beam Bridge (Rio Puerto Nuevo (No.12), Rio Sarapiqui (No.19) and Rio Chirripo (No.26))

(1) Design Flow for Steel I Beam Bridge

The design flow for the steel I beam bridge, which is reinforced by the external cable addition methods, is as shown in Figure 11.3.10. The prestress introduced by the external cable acts as a dead load on the girder section. The stress of the steel of the original girder section shall be less than the allowable stress for the dead load, live load and the prestress force.

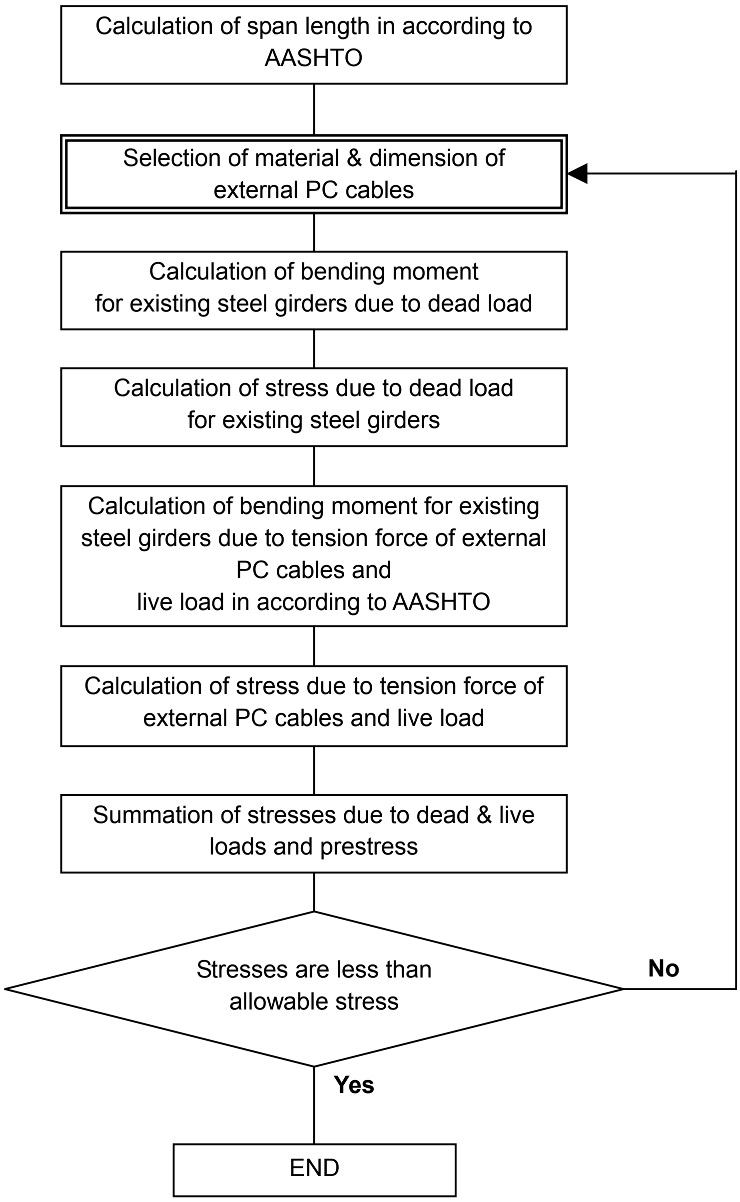


Figure 11.3.10. Design Flow for Steel I Beam Bridge Reinforced by External Cable Addition

(2) Stress Check Method for Reinforcement by External Cable Addition

External PC-cables make existing steel girders consistently resistant to dead and live loads. In addition to the axial force, the eccentric bending moment occurs to girder section by pre-stressing. Consequently, the stress checking method is as follows.

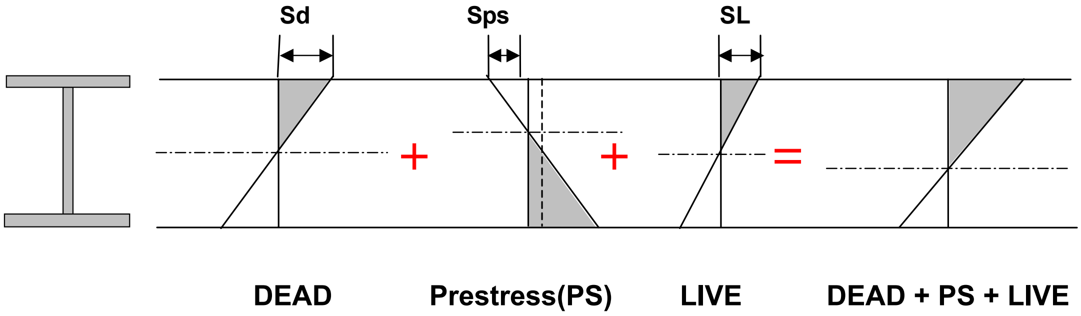


Figure 11.3.11. Stress due to Dead Load, Live Load and Prestress

Stress of steel I-girder due to dead + live loads

S = Sd + Sps + SL < Sa

Where, Sd : Bending stress by dead loads
Sps : Stress by axial prestress & its eccentric moment
SL : Bending stress by live loads
Sa : Allowable stress of steel girder

3) RC Deck Bridge (Rio Azufrado (No.7), Rio Puerto Nuevo (No.12), and Rio Nuevo (No.16))

The selected reinforcement methods for three RC deck bridges, which are No.7 Azufrado Bridge, the Puerto Nuevo Bridge, and the Nuevo Bridge, are deferent. However, the methodologies of the design for reinforcing of the RC deck bridge are almost same. It is the basic point for the design of the reinforcing the RC bridges, to check the stress of the existing reinforcement bar and concrete of original girder section. The required sections of reinforcement or material are normally controlled by the stresses of existing reinforcement bar and concrete of the original girder section.

(1) Design Flow for RC Deck Slab

The design flow for the reinforcing RC deck bridge by the FRP bonding or the Steel Bonding methods is shown in Figure 11.3.12.

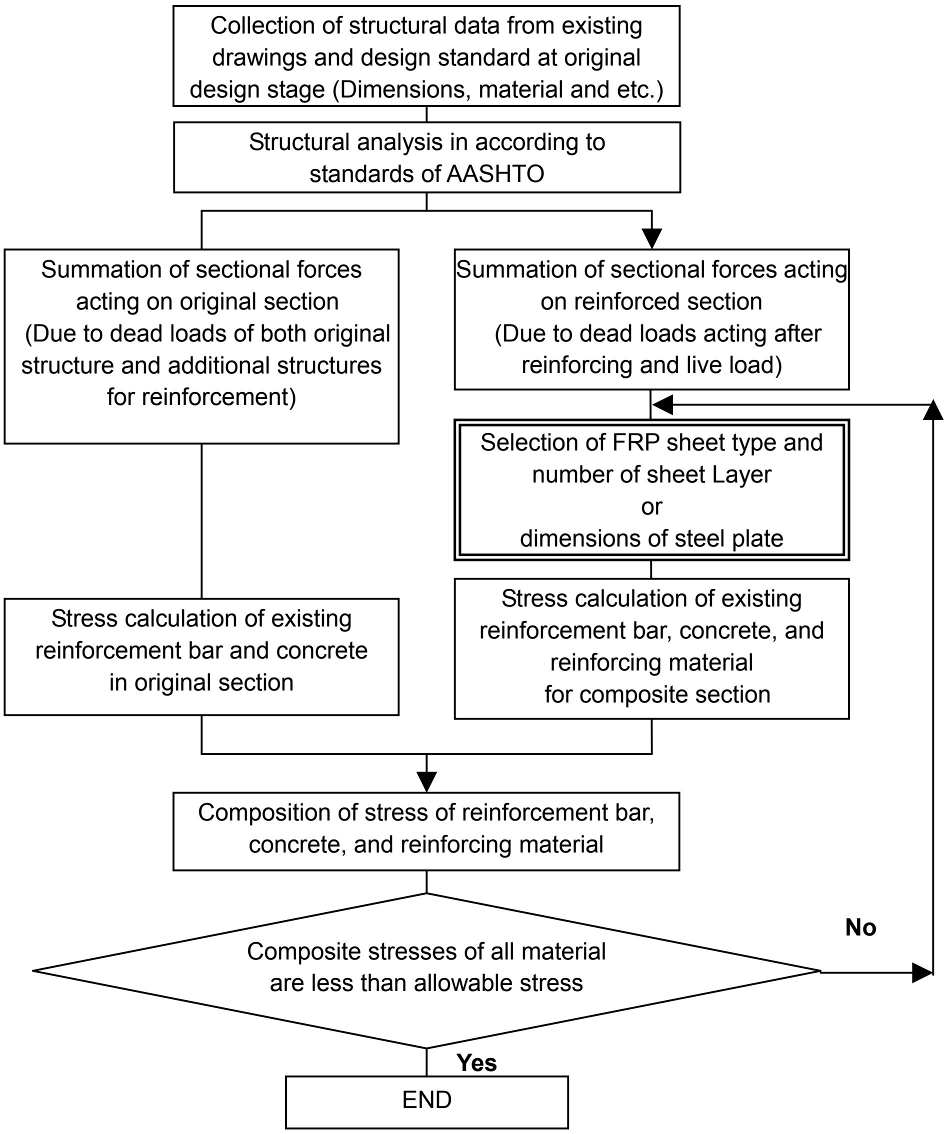


Figure 11.3.12. Design Flow for RC Girder Reinforced by FRP Bonding and Steel Plate Bonding

When the main girder is reinforced by the member section increase method , the structural analysis is required for both original section and increased section as shown in the design flow of Table 11.3.13.

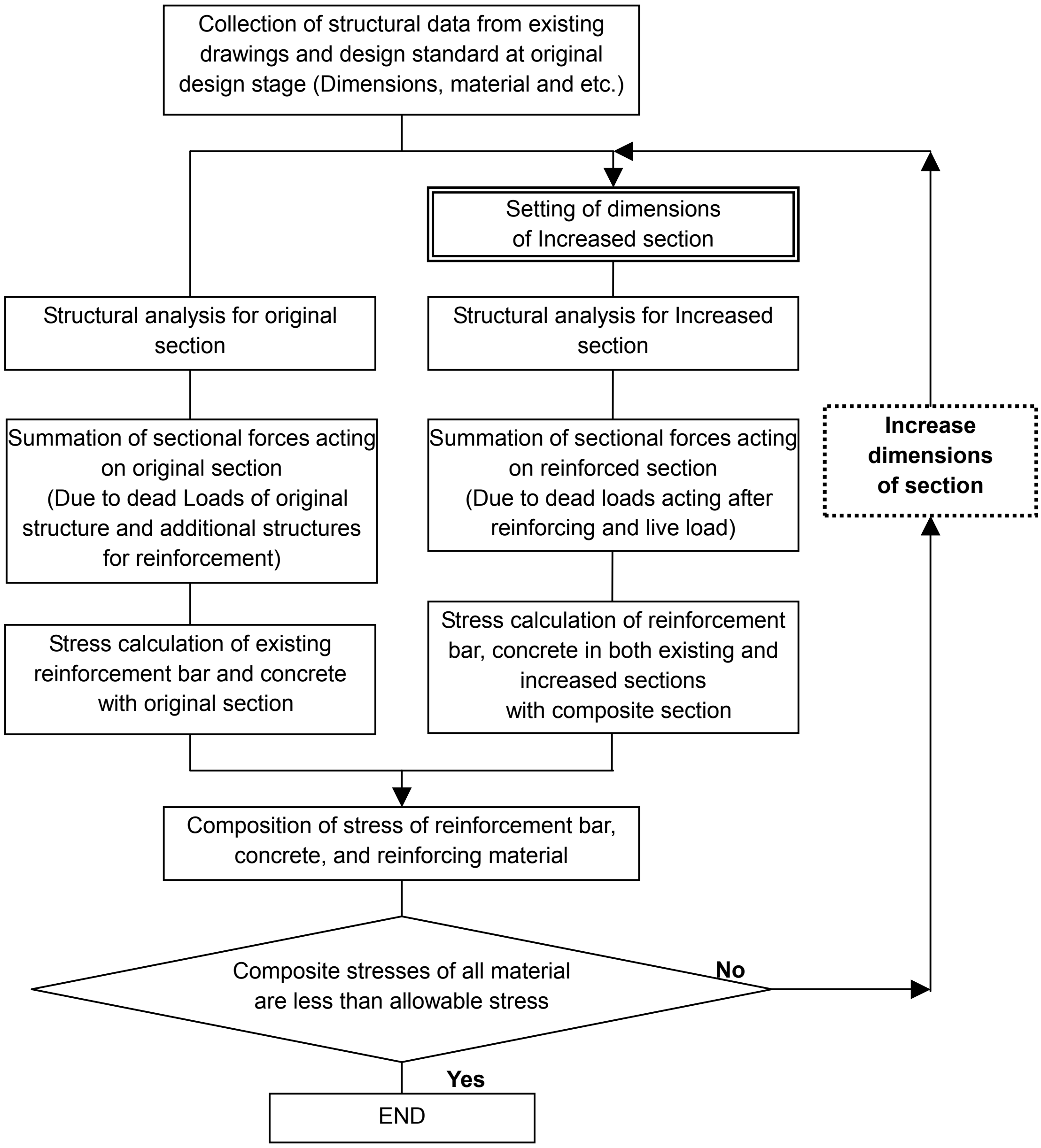


Figure 11.3.13. Design Flow for RC Girder Reinforced by Section Increase

When the deck slab is already reinforced by the member section increase and the main girder is required reinforcing by FRP or steel plate bonding such as the No.7 Azufrado Bridge and No.16 Nuevo Bridge, the structural analysis is required for both original section and increased section for deck slab as shown in the design flow of Figure 11.3.14.

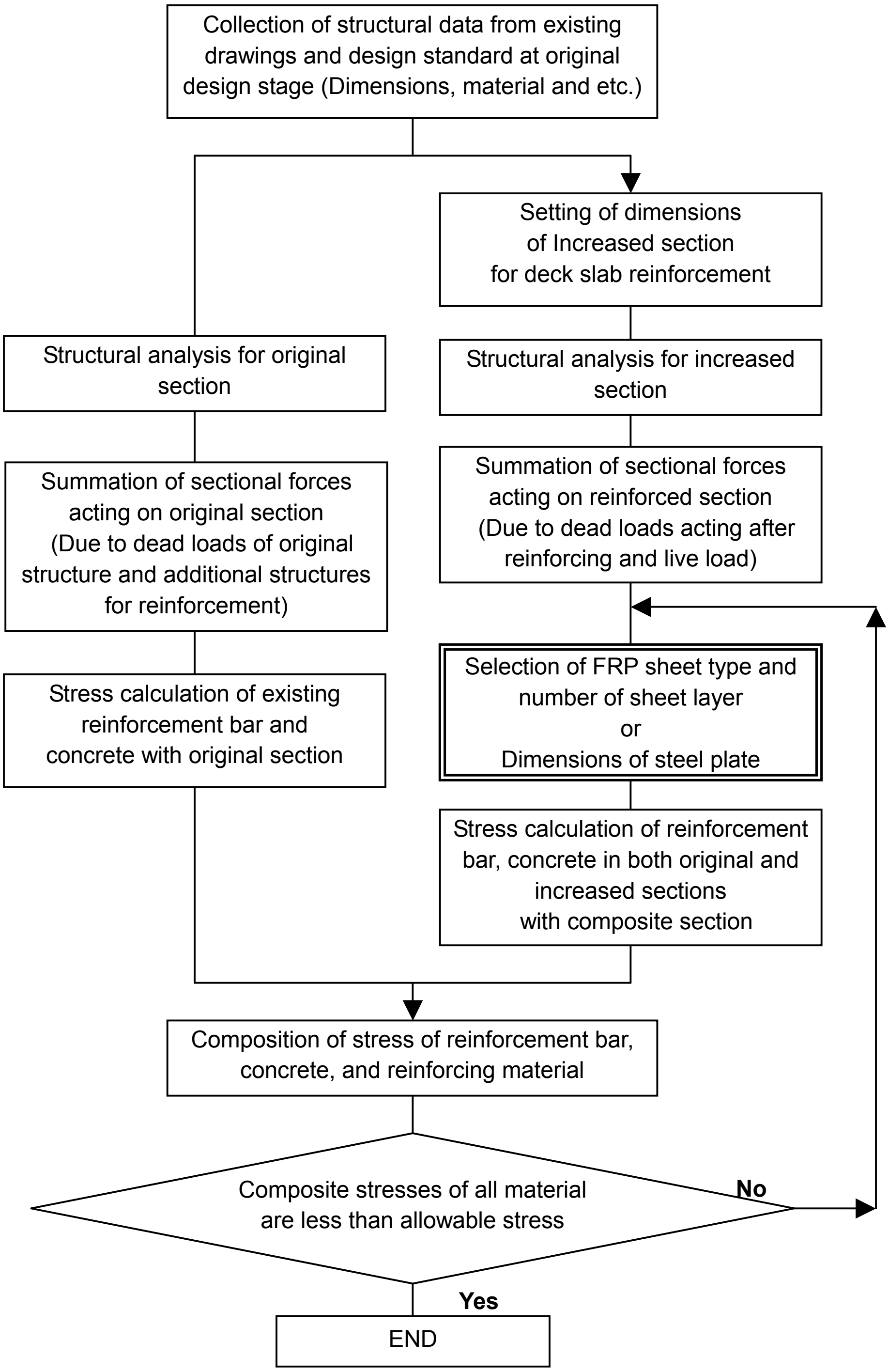


Figure 11.3.14. Design Flow for RC Girder, which Required Deck Slab Increase, by FRP or Steel Plate Bonding

(2) Stress Check Method for FRP Bonding or Steel Plate Bonding

The stress check method of the reinforced main girder by the FRP or steel plate bonding is as below. After composite the stresses of reinforcement bar, concrete, and FRP sheet or steel plate shall be smaller than their allowable stresses respectively.

1. Stress calculation of reinforcement bar and concrete existed in the original section for the bending moment **Md** due to the dead load including additional dead load for reinforcement.

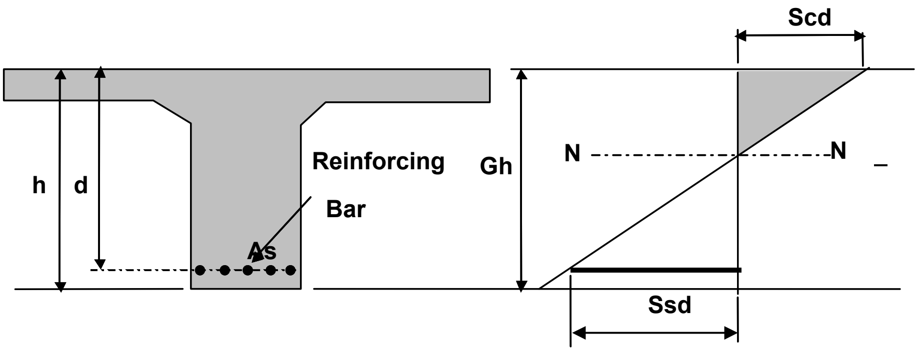


Figure 11.3.15. Stress due to Dead Load

Scd: Stress of Concrete due to Dead Load
Ssd: Stress of R-Bar due to Dead Load

2. Stress calculation of reinforcement bar, concrete, and FRP sheet or steel plate in the reinforced section for the bending moment **ML** due to the live load including dead load acting after reinforcement

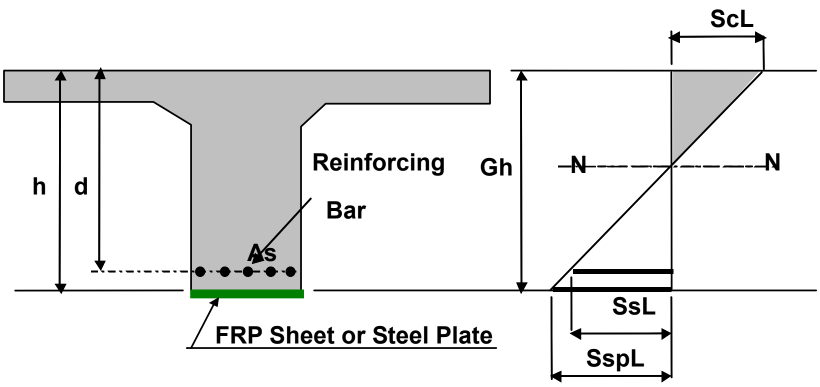


Figure 11.3.16. Stress due to Live Load

ScL: Stress of Concrete due to Live Load
SsL: Stress of R-Bar due to Live Load
SspL: Stress of FRP or Steel Plate due to Live Load

3. Stress of reinforcement bar, concrete, and FRP or steel plate due to dead and live loads

All stresses of concrete, reinforcement bar and FRP sheet shall be less than their allowable one respectively.

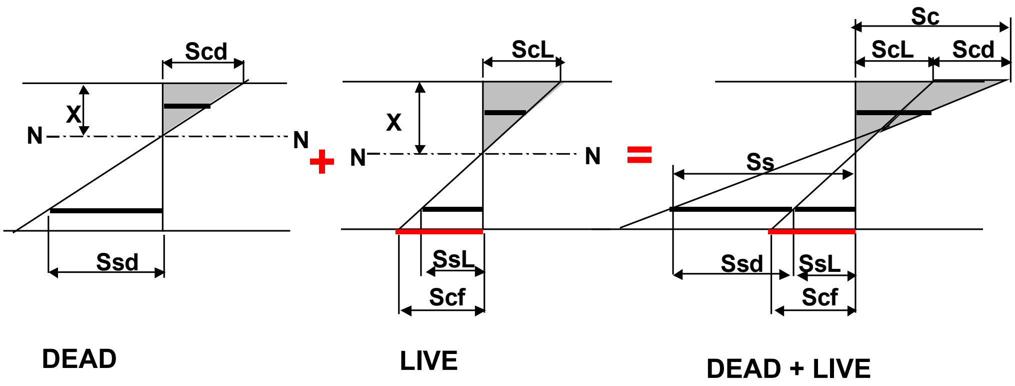


Figure 11.3.17. Composite Stress due to Dead and Live Loads

Sc = Scd + ScL < Sca
Ss = Ssd +SsL < Ssa
Scf = ScfL < Scfa

Where, Sca : Allowable Stress of Concrete
Ssa: Allowable Stress of Reinforcement bar
Scfa: Allowable Stress of FRP Sheet

(3) Reinforcing Area

The area where the registration capacity of the original section is smaller than the acting force has been refocused.

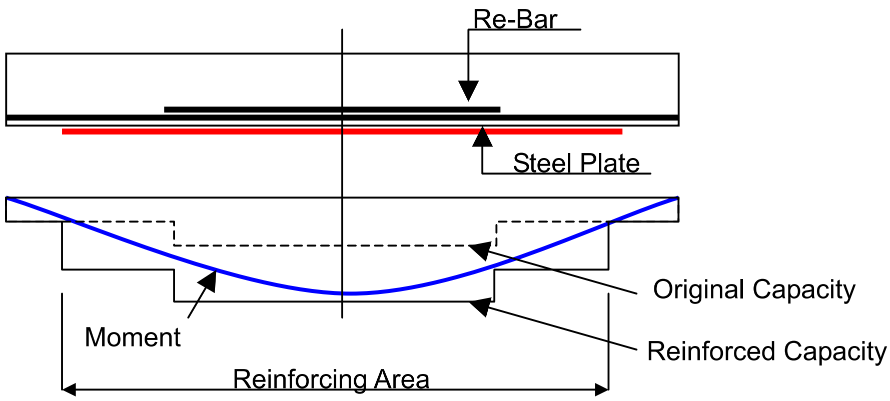


Figure 11.3.18. Reinforcing Area of Girder

4) PC I Beam Bridge (Rio Torres (No.29))

The FRP bonding was selected as the reinforcement methods for No.29 Rio Torres Bridge, which is the PC I beam type bridge. The basic point for the design of the reinforcing the PC I beam bridges is that both the reinforcement bars and the FRP sheet resist to the total tension force in the tension stress area of the section. And it is assumed that the allowable tensile stress of FRP sheet is controlled by the allowable stress of the reinforcement bar and their ratio of the elastic modulus, because both strains of the reinforcement bar and the FRP sheet are same at the tensile stress area.

(1) Design Flow for PC I Beam Bridge

The design flow for the reinforcing PC I beam bridge by the FRP bonding is shown in Figure 11.3.19.

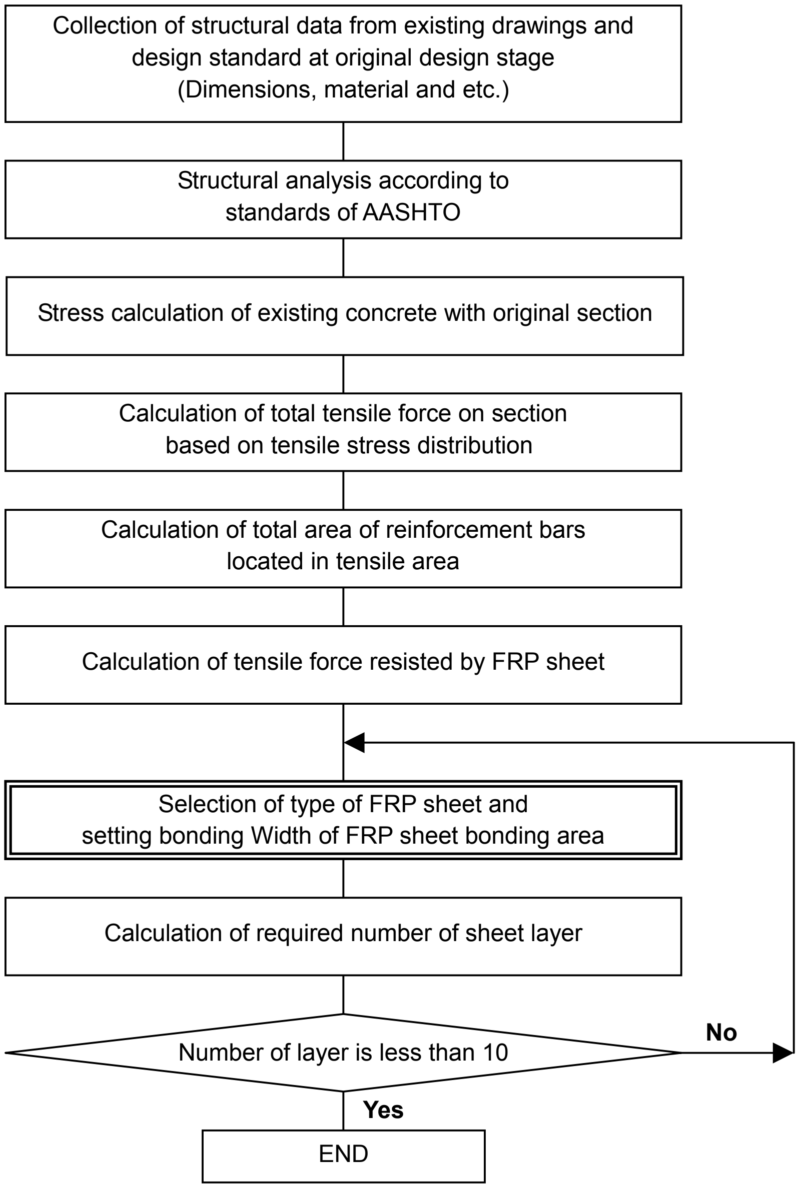


Figure 11.3.19. Design Flow for PC I Beam Reinforced by FRP Bonding

(2) Design Method of FRP Bonding for PC I Beam

The design method for reinforcing the PC I beam by the FRP bonding is as below. The required number of FRP sheet layers shall be less than 10.

1. Stress of the concrete of the original section shall be calculated for all loads including the prestress. The total tensile force **Tc** of the section can be calculated based on the tensile stress area of the section.

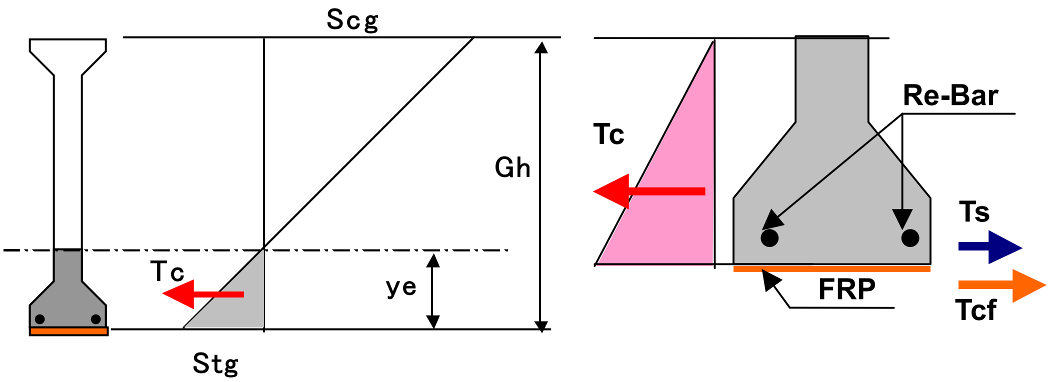


Figure 11.3.20. Image of Tensile Stress of PC I

2. The total tensile force **Tc** acting on concrete section is resisted by the existing reinforcement bars and the bonded FRP sheet.

Tc = Ts + Tcf = Ss x As + Scf x Acf = Ss x As + Ss (Ecf / Es) x Acf

Where, Tc: Total tensional stress acting on concrete section
Ts: Tensional force resisted by existing reinforcement bar in tensional area
Tcf: Tensional force resisted by bonded FRP
Ss: Stress of Re-bar
Scf: Stress of FRP
As: Sectional area of Re-Bar
Acf: Sectional area of FRP
Es: Elastic modulus of Re-Bar
Ecf: Elastic modulus of FRP (kN/mm^2)

3. The stress of the reinforcement bar shall be less than the allowable stress Ssa. Therefore, the required area of the FRP sheet and required Number of layer can be calculated as below.

$$\mathbf{Acf} = \frac{\mathbf{Tc / Ssa - As}}{\mathbf{Ecf / Es}}$$

$$\mathbf{Ncf = Acf / (tcf \times Wcf)}$$

Where, Ssa: Allowable stress of reinforcement bar
Ncf: Required number of FRP sheet layer
tcf: Thickness of FRP sheet resisted by existing reinforcement bar in tensional area
Wcf: Bonding width of FRP sheet

11.3.4 Existing Condition and Condition after Reinforcement

For the increased live load HS20 +25%, the main girders or structures of 7 bridges shall be reinforced excluding No.17 Chirripo Bridge, No.20 Sucio Bridge, and No26 Chirripo Bridge.

The reinforcement methods for each bridge are summarized for each bridge type as the following tables.

1) Truss Bridge

The axial force of the original member decreases by addition of member, and the No.2 Aranjuez Bridge shall have enough load capacity.

It is necessary for the No.3 Abangares Bridge (129ft) to be reinforced by steel cover plate for the area of about 18m length at the span center of the lower chord. The plates used for reinforcement are 180mm in width, and 5/8in. in thickness. On the other hand, No.3 Abangares Bridge (200ft) is unnecessary to be reinforced.

Table 11.3.15. Comparison of Main Structure Conditions for Truss Bridges

Bridge No.	Bridge Name	Existing Conditions		Conditions after Reinforcement		Allowable Stress (N/mm^2)
		Stress Over Position	Stress (N/mm^2)	Reinforcement Method	Stress (N/mm^2)	
2	Aranjuez	Center of Main Span	86	Additional member	55	63
		At Support	130	Additional member	94	100
3	Abangares (129ft)	Center of Span	144	Steel Cover Plate	124	125
	Abangares (200ft)	Center of Span	70	-	-	125

2) Steel I Beam Bridge

The bending moment of the original member decreases by external cable additional, and the No.12 Puerto Nuevo Bridge and No.19 Sarapiqui Bridge shall have enough carrying capacity.

The No.26 Chirripo Bridge has enough carrying capacity in the original condition.

Table 11.3.16. Comparison of Main Structure Conditions for Steel I Beam Bridges

Bridge No.	Bridge Name	Existing Conditions		Conditions after Reinforcement		Allowable Stress (N/mm^2)
		Stress Over Position	Stress (N/mm^2)	Reinforcement Method	Stress (N/mm^2)	
12	Puerto Nuevo (70ft)	Center of Span	143	External cable Addition	123	125
	Puerto Nuevo (80ft)	Center of Span	149	External cable Addition	119	125
19	Sarapiqui	Center of Span	191	External cable Addition	126	130
		At Support	136	External cable Addition	126	130
26	Chirripo	Center of Span	130	-	-	130
		At Support	104	-	-	180

3) RC I Girder Bridge

The RC main girders of No.7 Azufrado Bridge, No.12 Puerto Nuevo Bridge, and No.16 Nuevo Bridge do not have enough load capacity for the increased live load.

On Azufrado Bridge, the stresses at center of the main span and at support exceed the allowable stress even after increasing the thickness of deck slab, which can be taken as the part of main girder section. The center of main span was reinforced by the bonding of the steel plate (3”/8 thickness, 400 mm width) for length of 7.2m area and the section at support was reinforced by the increasing the girder depth of 30 cm on the bottom side.

The simple beam of No.12 Puerto Bridge does not have enough load capacity for the almost whole section of the beam. The beam was reinforced by bonding of steel plate (3”/8 thickness, 400 mm width) for length of 11.6 m of the total beam length of 15.24 m.

The girders of both the Azufrado Bridge and the Puerto Nuevo Bridge were reinforced by the bonding of steel plate with the thickness of 1/4” for increased shear force.

The deck slab of No.16 Puerto Bride was also reinforced by increasing the slab thickness. The section near center of side span does not have enough load capacity even after the increasing the thickness of deck slab. The sections of main girder in the both side spans were reinforced by FRP bonding method with the 6 layers of FRP sheet (483 mm width) for 3.70m long.

And the girders were also reinforced by the FRP bonding with 1 layer for increased shear force.

Table 11.3.17. Comparison of Main Structure Conditions for RC Deck Bridges

Bridge No.	Bridge Name	Existing Conditions			Conditions after Reinforcement				
		Stress Over Position	Stress (N/mm^2)		Reinforcement Method	Stress (N/mm^2)			
			Concrete (7.40))	Re-Bar (138)		Concrete (7.40)	Re-Bar (138)	FRP (633)	Steel Pate (141)
7	Azufrado	Center of Main Span	5.7	227	Steel Plate Bonding	3.0	134	-	90
		At Support	8.2	173	Increase	6.5	137	-	-
12	Puerto Nuevo (RC Bridge	Center of Span	4.3	155	Steel Plate Bonding	4.4	136	-	75
16	Nuevo	Center of Side Spans	5.0	156	FRP Sheet Bonding	3.2	136	263	-

4) PC I Beam Bridge

The PC I beam of No.29 Torres Bridge consists 3 simple beam with two types of span lengths of 30m and 17 m. The 17 m beam has enough carrying capacity for the increased live load, the 30 m doe not have the capacity. The tensional stresses at center of the 30m beam exceed the allowable tensional stress of the prestressed concrete. The center of main span was reinforced by the bonding of the 6 layers of FRP sheet around the lower flange for length of 13.0 m.

And the girders were also reinforced by the bonding of FRP sheet with 1 layer for increased shear force.

Table 11.3.18. Comparison of Main Structure Conditions for PC I Beam Bridges

Bridge No.	Bridge Name		Existing Conditions			Conditions after Reinforcement			
			Stress Over Position	Stress (N/mm^2)		Reinforcement Method	Stress (N/mm^2)		
				Concrete			Concrete (7.40)	Re-Bar (138)	FRP (633)
				Compression (12.3)	Tension (-2.9)				
29	Torres	30 m	Center of Main Span	11.4	-3.7	FRP Sheet Bonding	11.4	118	379
		17 m	Non	8.6	-1.8	-	-	-	-

11.4 Accessory

The accessories, which were taken into account into the rehabilitation or improvement for objective 10 bridges, are as following items:

1) Asphalt Pavement
2) Waterproofing of deck slab
3) Expansion Joint
4) Bearing Support
5) Railing

These accessories are very important for keep the performance of bridge structure and traffic safety. The purposes of these structures are as shown in Table 11.4.1.

Table 11.4.1. Purpose of Accessories

Kind of Accessory	Purpose
Asphalt Pavement	Asphalt pavement is laid on the concrete deck slab, to provide a smooth riding surface and the comfortable driving condition, to protect the deck slab from the damages due to traffic, weathering and chemical action.
Waterproofing of deck slab	Waterproofing is laid between the asphalt pavement the concrete deck slab, to improve the water tightness and protect the concrete deck slab from the damage, to protect the FRP sheet used for the reinforcement for the deck slab
Expansion Joint	The expansion joints covers the pavement gaps between girder and abutment to provide a smooth transition from the approach roadway to the bridge and, to be able to protect the supports and girders under the deck slab from water leaking through the gap of pavement by using the un-drain type of joints.
Bearing Support	Bearing support is provided to support the all dead weight of superstructure and live load and transfer the these load to the substructures
Railing	Railing is set on the both side edges of deck slab to prevent

1) Asphalt Pavement

An asphalt pavement is common paving material for road in both earthwork section and bridge section except special area such as near the tollgate The most of bridges in Costa Rica have not been paved by the asphalt concrete, and the thickness of 1/2" of top of the concrete slab deck is increased as a concrete wearing course in the design. However, the surfaces of concrete deck slab have been damaged/deteriorated by heavy traffic load or a severe weather condition. And the cracks of the concrete don't stop in the wearing layer and extend into the layer designed as the effective thickness of deck slab. . Some bridges have been paved by the asphalt concrete on the top of slab deck. But, these asphalt pavements are damaged and the surface conditions are very bad by the lack of maintenance.

Therefore, it was proposed to pave the slab deck of existing bridges by asphalt concrete with thickness 50mm, even the bridge has the concrete wearing layer as the part of concrete slab deck. For the bridges already paved, the asphalt pavement shall be replaced shall be repaved with a waterproofing to protect the slab deck.

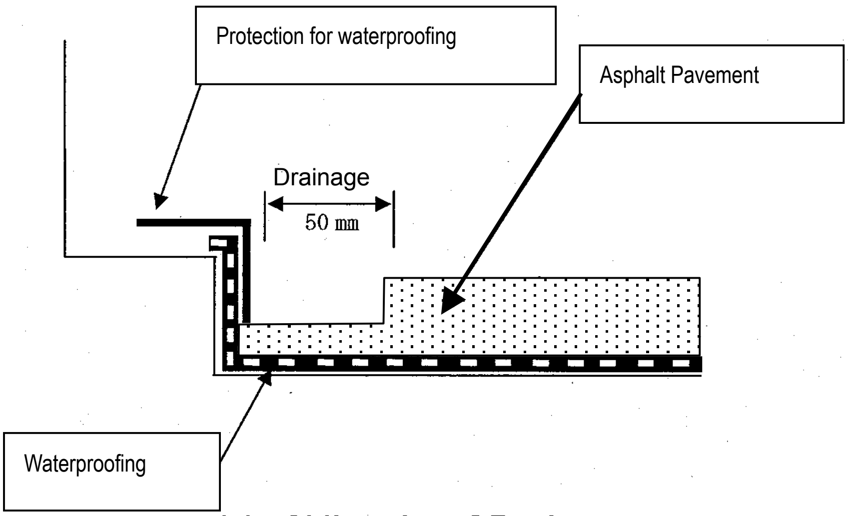
2) Waterproofing of Deck Slab

Even if the concrete slab deck is paved by the asphalt concrete, there is a possibility of that the deck slab is deteriorated such as corrosion of re-bar or deterioration of concrete by penetrated water from the surface of pavement. Specially the deck slab around the intermediate support in the continuous bridge, where is tension part for the longitudinal direction of the bridge, and the connection parts of the precast concrete slab are the weak points for the water leaking. Therefore, it was proposed to execute the waterproofing on the deck slab to protect the deck

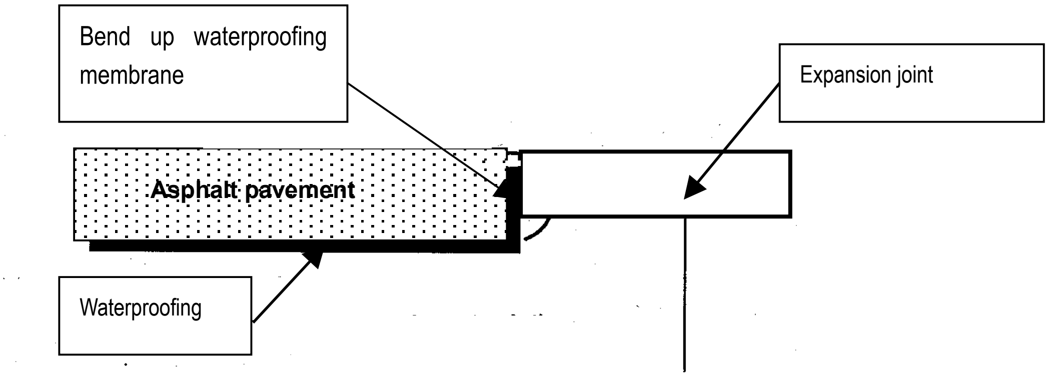
There are two types of bridge deck waterproofing membrane systems as follow

Self-adhering membrane	:	A high strength polyester reinforced membrane with a rubber/bitumen compound, which is cold applied.
Liquid waterproofing membrane	:	A two-component compound, which is simply mixed on site to produce a viscous seamless rubber/bitumen liquid that cures to an elastomeric waterproof membrane.

The waterproofing membrane shall be bended up at the edge and the drainage shall be provided at the edge of carriageway as shown in Figure 11.4.1.



(a) At the edge of Carriageway



(b) At the Expansion Joint

Figure 11.4.1. The detail of waterproofing

3) Expansion Joint

On some bridges, there is no expansion joint and the edge of concrete deck is protracted by the steel member. And most of the expansion joints are broken already and the asphalt concrete is laid on the joint as shown in Figure 11.4.2. The water leaking through these joints causes the deterioration of the supports and girders under the joint.

It was proposed that all expansion joints including the protections of the concrete deck shall be replaced the new joint, which can protect the water leaking through the joint.

(a) Broken Expansion Joint **(b) In the case of no expansion Joint**

Figure 11.4.2. Condition of Expansion Joint

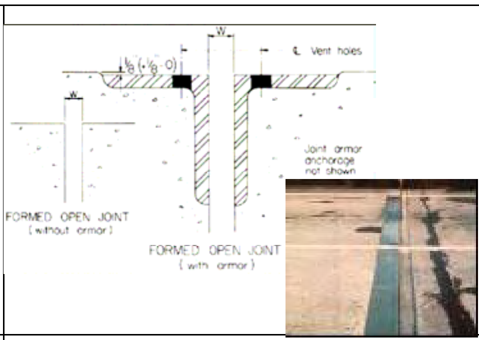
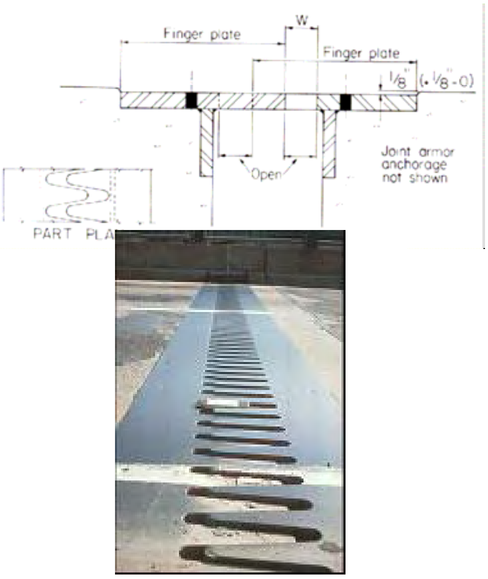
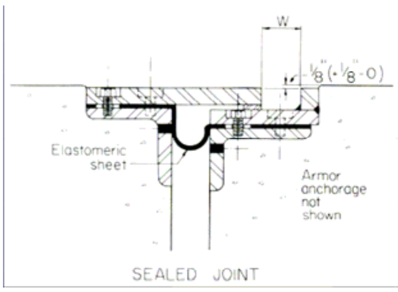
(1) Type of Expansion Joint

There are many types of the expansion joint for the required expansion width. They can be classified into two types based on the drainage system of the joint.

a) Open Joint

The open joints allow the water and the debris from deck surface to pass through the joint. The examples of the open joints are as shown in Table 11.4.2.

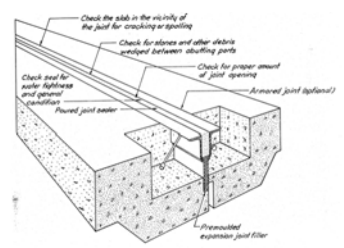
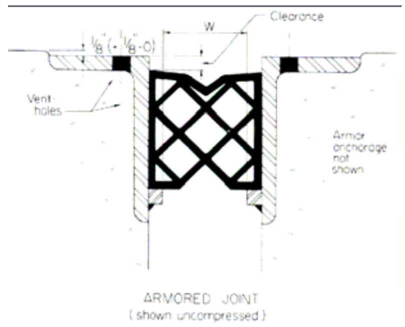
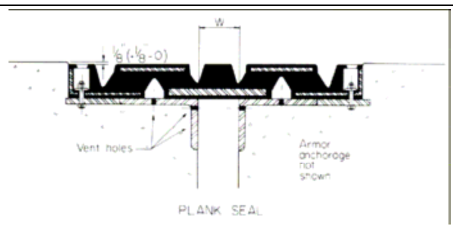
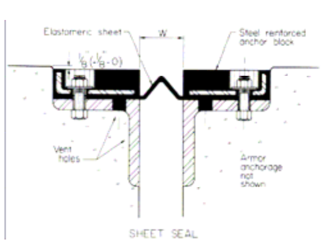
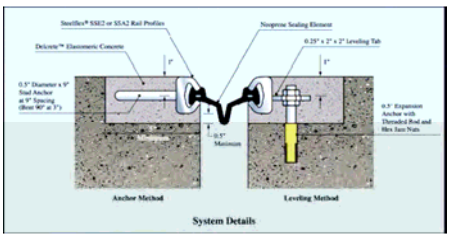
Table 11.4.2. Open Joint (Drain Type)

Formed joints		- Used at fixed supported points with a small gap of slab deck - Used for un-paved concrete deck - the edge of concrete of deck slab and parapet wall are reinforced with the steel angles.
Finger plate joints		- A finger plate joint, also known as a tooth plate joint or a tooth dam, consists of two steel plates with interlocking fingers. These joints are usually found on longer span bridges where large expansion is required. - Used to accommodate movement from 100 to over 600 mm - A troughs system is sometimes set under open finger plate to protect water leaking through the joint. - There are two types of finger plate joints, a cantilever finger plate joints and a supported finger plate joints. The cantilever type is used for short expansion width and the supported type ids for long expansion. - Some types of finger plate joints are segmental, and to be allow the replacement of the parts.
Sliding plate joint		- A sliding plate joint is composed of two plates sliding on top of each other. - The sliding plate joint is usually not watertight. In an attempt to seal the joint, an elastomeric sheet is sometimes used for trough that serves to carry water away to the sides of the. - This sheet is attached between the plates and the joint armoring. The sliding plate joint can accommodate a maximum movement of approximately 100 mm (4 inches).

b) Closed Joints

Closed joints are designed to avoid the water and the debris from the deck surface to pass through them and to protect the girder and substructure under the joint from the deteriorations due to water and debris. The typical examples of the closed joints are shown in Table 11.4.3.

Table 11.4.3. Closed Joint (Un-drain Type)

Poured joint seal		A poured joint seal is made up of two materials: a base and a poured sealant. The base consists of a preformed expansion joint filler. The top of this material is 25 to 50 mm from the top of the deck. The remaining joint space consists of the poured sealant that is separated from the base by a backer rod or a bond breaker. Since the poured joint seal can only accommodate a movement of about 6 mm, it is usually found on short span structures.
Compression seal		- A compression seal consists of neoprene formed in a rectangular shape with a honeycomb cross section (see Figure). The honeycomb design allows the compression seal to fully recover after being distorted during bridge expansion and contraction. - Compression seals can have steel angle armoring on the deck and parapet wall - These seals come in a variety of sizes and are often classified by their maximum movement capacity. - A large compression seal can accommodate a maximum movement of approximately 50 mm.
Prefabricated elastomeric seal (Plank seal)		- A plank seal consists of steel reinforced neoprene that supports vehicular wheel loads over the joint. This type of seal is bolted to the deck and is capable accommodating movement ranges from 50 to 330 mm.
Prefabricated elastomeric seal (Sheet Seal)		- A sheet seal consists of two blocks of steel reinforced neoprene. A thin sheet of neoprene spans the joint and connects the two blocks. This joint can accommodate a maximum movement of approximately 100 mm.
Prefabricated elastomeric seal (Strip Seal)		- A strip seal consists of two slotted steel anchorages cast into the deck and parapet wall. A neoprene seal fits into the grooves to span the joint. This joint can accommodate a maximum movement of approximately 100 mm

(2) Installation

For the existing decks which is not paved the asphalt pavement, there will be a 5cm gap between the surface of new pavement and the existing expansion joint. The expansion joint shall be replaced at 5 cm higher position with the cast concrete.

The example of the setting layout of the expansion joint for simple steel girder bridge or PC I beam bridge is shown in Figure 11.4.3.

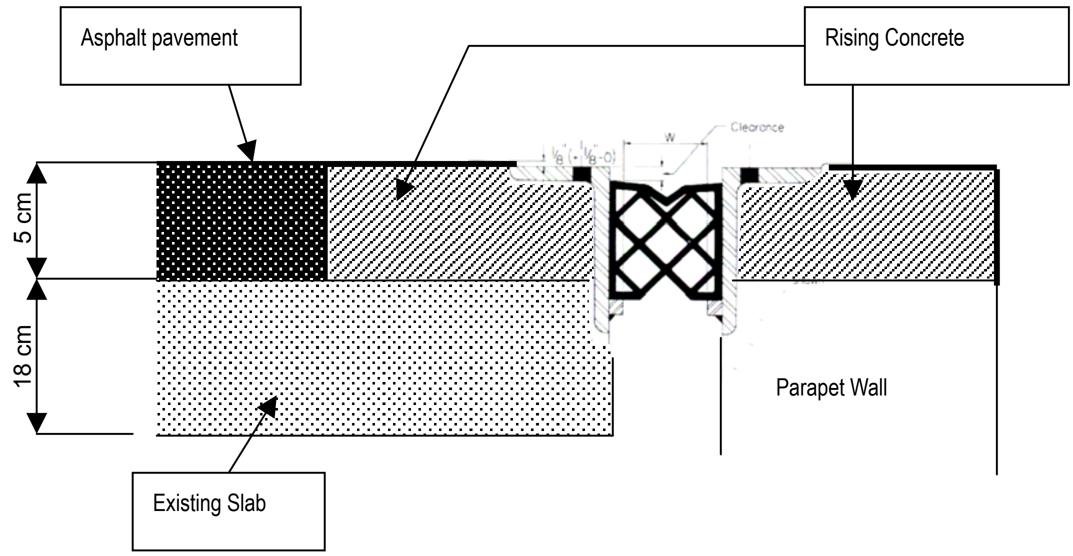


Figure 11.4.3. Example of Method for Fixing New Expansion Joint

11.5 Prevention System for Bridge Collapse

11.5.1 Seismic Performance

The privation system for the bridge collapse shall be selected for taking account of the seismic performance level, which will be decided based on the importance of bridge. There are two levels for the target performance of the seismic design. The basic level of seismic performance is a safety for the seismic force, and the second level is a kind of the serviceability after an earthquake happened. This serviceability performance is that the bridge can be open traffic for evacuation, rescue operation and transportation route for medical services and to send relief to the victims.

The level of seismic performance is defined as shown in Table 11.5.1.

Table 11.5.1. Seismic Performance Level

Seismic Performance-1	Safety performance of the bridge structure against earthquake load, which is evaluated as the load capacity and stability for the seismic force.
Seismic Performance-2	Serviceability performance to keep same performance for traffic as before an earthquake. To avoid interruption of traffic, even when an unexpectedly large displacement occurs between the superstructure and the substructure, To prevent the girder collapse or transforming and to prevent a settlement of the surface of bridge

The all objective 10 bridges locate on the most important routes in Costa Rica. It is required to keep a route even after earthquake, and the prevention system for bridge collapse for the bridges were designed to satisfy this performance-2

11.5.2 Basic concept of Prevention System for Bridge Collapse

1) Components of Prevention System for Bridge Collapse

The prevention system consists of the beam seat length at the support, the girder connection, the excessive displacement limiting structure, and the structures for preventing the superstructure from settling as shown in Table 11.5.2.

Table 11.5.2. Component of Prevention System for Bridge Collapse

Component	Explanation
Seating Length of girder	By keeping an enough length from the end of girder to the edge of substructure, to prevent the girder collapse,
Girder Connection	To prevent the girder collapse beyond the seat length, A complementary system to seat length and a fail-safe structure To work before a girder moves beyond the seat length.
Limiting Excessive Displacement	To prevent a large displacement of girder, after the bearing support is damaged
Prevention of Girder Settlement	To prevent girder settlement, after the bearing support is damaged. To avoid the difference in level of carriageway

(1) 2) Design for Prevention System

The components of the prevention system for bridge collapse were selected taking account of the structural conditions such as a type of bridges and support condition. The key points for design on the Prevention System are as shown below.

- Consideration of the working sequence, the structural characteristics and allowable movement capacity of each prevention structure.
- The relationship of capacity for moving between each prevention structure is as below.

  Bearing < Structures Limiting Excessive Displacement
  < Girder Connection < 75% of Seating Length of girder

- The moving capacity of each prevention structure should be same in the same bearing line.
- Fabrication errors can be adjusted on site.
- Differences in scale and type between the adjoined girders. The girder connection system shall not applied in the cases of that the difference of the reaction forces between adjoined girders is more than 2 times, or the difference of the natural period in design vibration unit is more than 1.5 times.

11.5.3 Methodology of Design for Prevention Systems for Bridge Collapse

1) Seating Length of girder

(1) Required Seat length

The seat length SE of girder at the support is defined as shown in Figure 11.5.1. And the required length is calculated according to the formulas below, which is provided in “Japanese Standard for Road Bridge No. V”.,

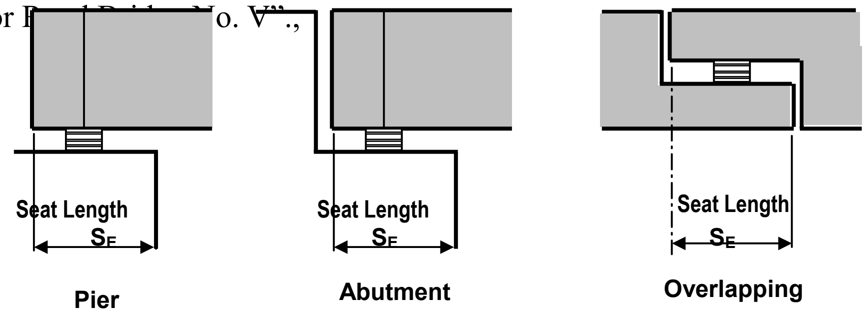


Figure 11.5.1. Seat Length of Girder

$$S_E = 0.7 + 0.005L$$

where

S_E : Seat length（m）
L : Distance between substructures affecting the seat length of girder （m）

The length L is the distance from the girder fixed point to object point as shown in Figure

11.5.2.

When the acting direction of the earth pressure working on the substructure is not same as the longitudinal direction of the bridge axis, such as the skewed bridge or the curved bridge, the seat length of a girder shall be measured in direction that give a minimum distance between the end of the girder to the edge of the substructure as shown Figure 11.5.3.

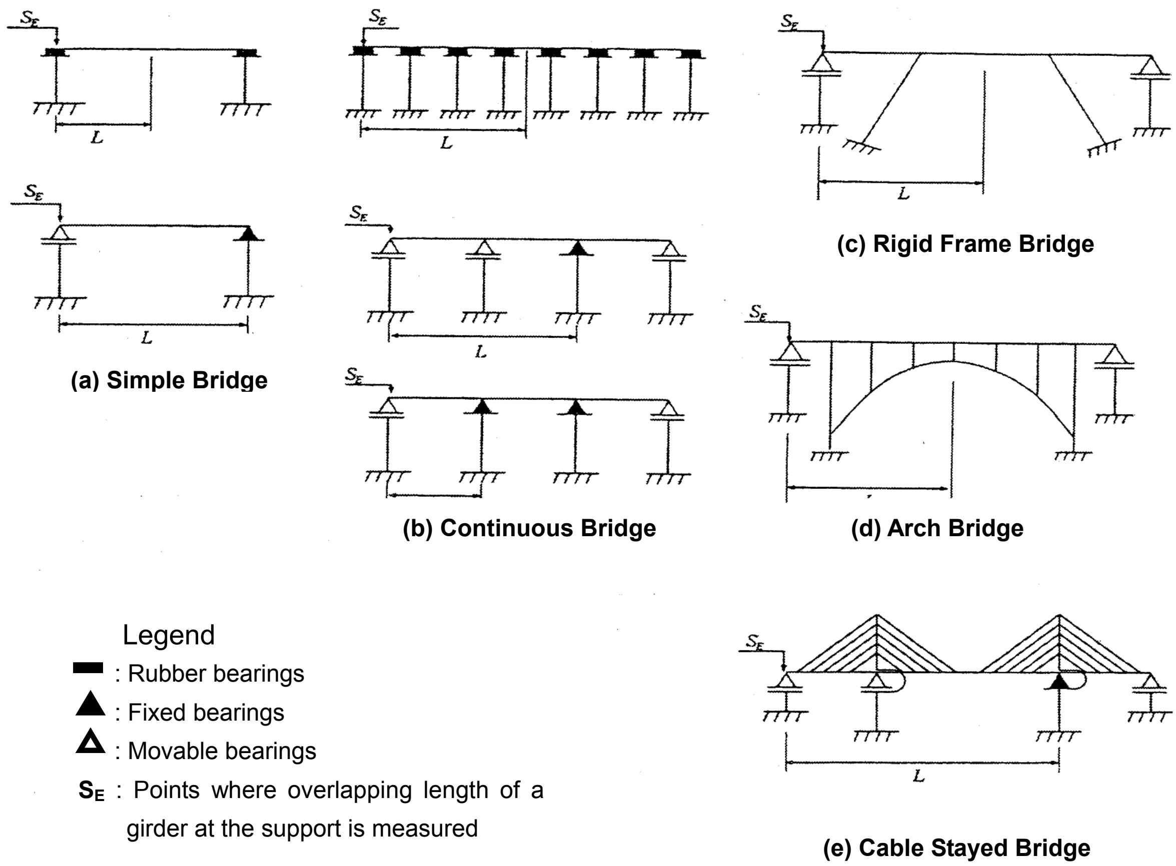


Figure 11.5.2. Distance to Calculate the Seat Length

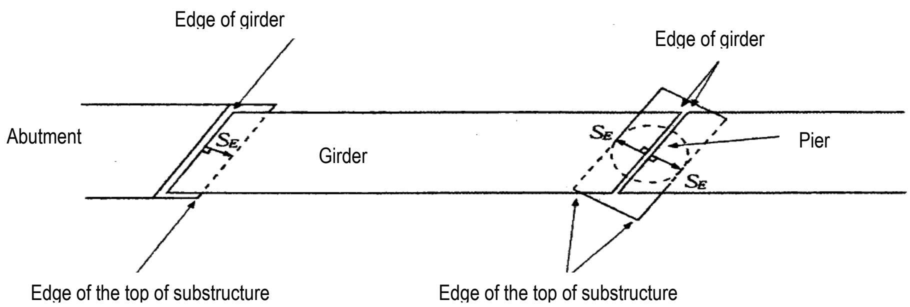


Figure 11.5.3. Seat Length for Skewed Bridge

(2) Structure for Widening Seat Length

There are two types of structures for widening the seat length of the existing structure. One is a steel bracket and the other is a concrete bracket. The steel bracket shall be prefabricated in a factory and fixed on site. The fixing work is not easy to adjust on the existing concrete surface of the substructure. The steel type is more costly than the concrete one. The concrete type is more easy to adjust and can be made together with other required prevention structures.

The concrete type was applied for widening the seat length for all bridges in this project.

The dimensions of the concrete bracket were determined based on the required width for seat length and the size of bearing support, according to the Japanese standards as shown in Figures 11.5.4. and 11.5.5. For the design of the bracket, the design load Rd is taken as 1.5 times of the dead load acting on the bearing support.

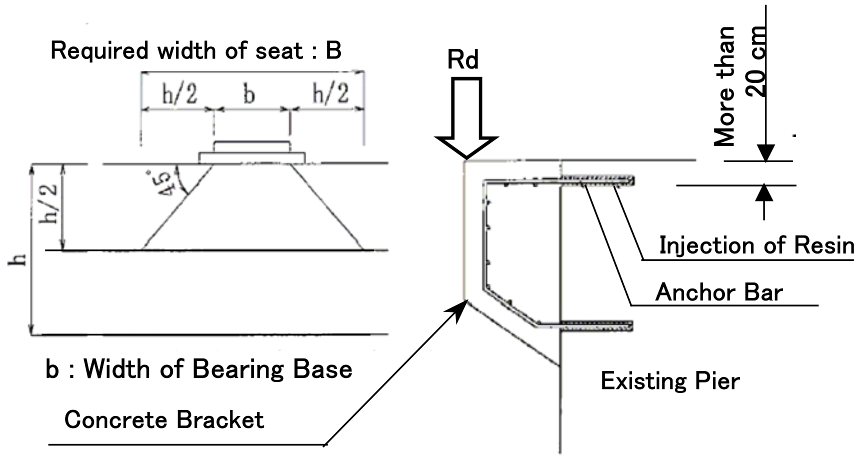


Figure 11.5.4. Required Dimensions for Concrete Bracket

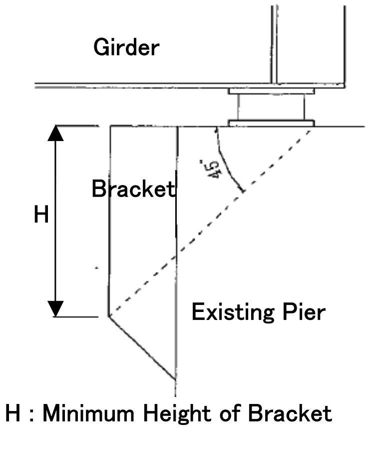


Figure 11.5.5. Minimum Height of Bracket

2) Girder Connection

The girder connection system is set to prevent a collapse of girders directly. It shall work after both the bearing support and the structures limiting excessive displacement are broken by earthquake. The connection system shall not disturb the moments and rotation of the bearing and the structures limiting excessive displacement. The girder connection system can be divided into three types as below.

- Protuberant structure set on both superstructure and substructure
- Structure connecting the girder and the substructure
- Structure connecting two girders

a) The details of each type are as shown in Table 11.5.3 to 11.5.5.

Table 11.5.3. Protuberant Structure

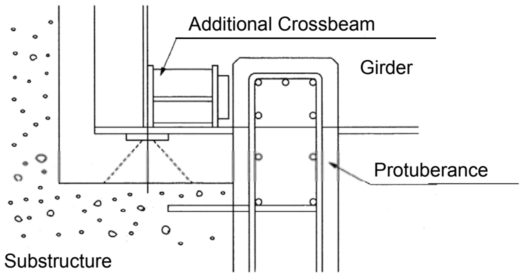
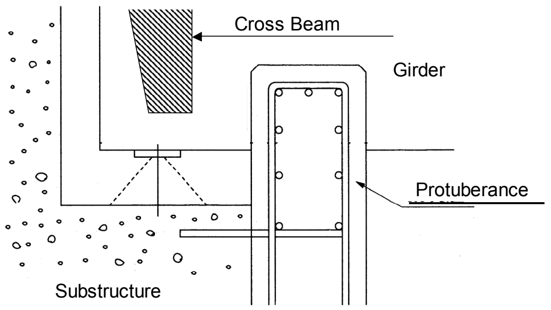
Outline of structure	Feature
 a) In the Case of Steel Bridge  b) In the Case of Concrete Bridge	- A protuberance of the substructure side is a concrete structure and fixed by anchor bars. -For the superstructure side, an additional crossbeam is installed and fixed with H.T.B or welding. -Where substructure has an enough space on the top, a protuberance structure is installed on the top. -A protuberance can be unified with the concrete bracket for widening of seat length

Table 11.5.4. Structure Connecting Girder and Substructure

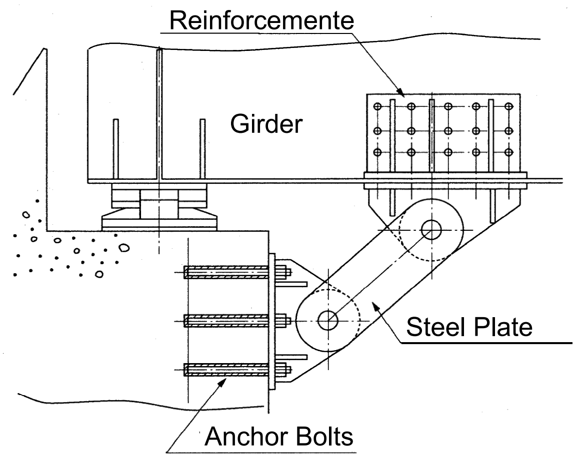
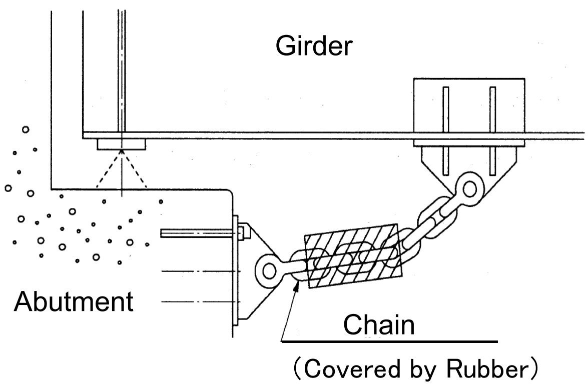
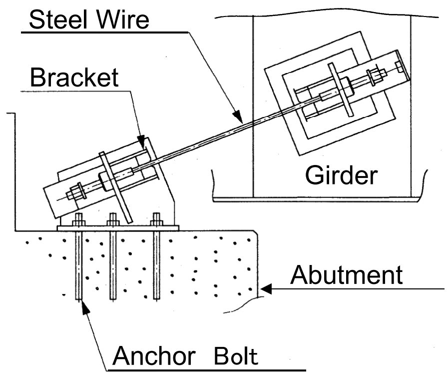
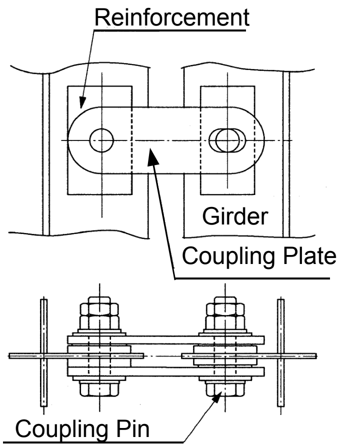
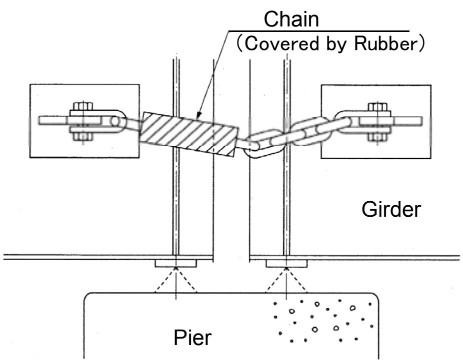
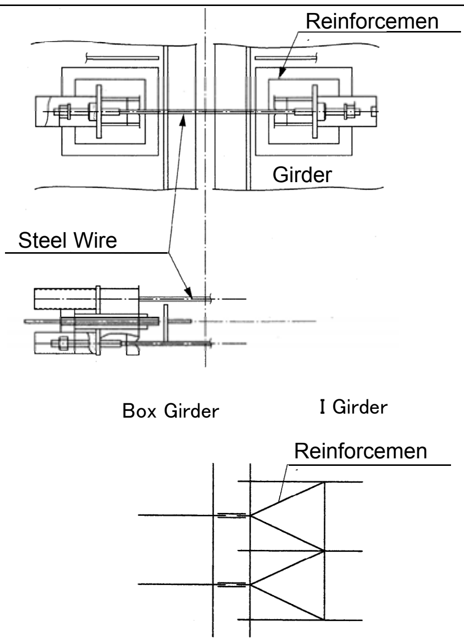
Outline of structure	Feature
	-Not applied for large movement of superstructure -Cannot resist the movement of transversal direction -Cannot absorb the impact force of earthquake. - Reinforcing of the girder is need. -Applicable for structures with no obstacle near the end crossbeam. -A bracket for substructure is fixed by anchor bolt and a bracket for superstructure is fixed by H.T.B. -Since anchor bolt is horizontal direction, making hole work is not so easy.
	-Applicable for large movement of superstructure -Applicable for the substructure with enough space on top . -Applicable for the structure with obstacle near the end crossbeam. -Can absorb the impact of earthquake force. -Reinforcing of the girder is need. -Available for movement in any direction -Error of manufacture can be assimilated in the stage of construction. -If necessary the chain shall be covered by rubber.
	-Applicable for large movement superstructure . -When there are some obstacles around end crossbeam, it can be installed. -Can absorb the impact of earthquake force. - Available for movement in any direction -Reinforcing of web plate of girder is required. -Rust proof treatment for steel wire shall be done -Adjustment for length of steel wire shall be needed -Error of manufacture can be assimilated in the stage of construction.

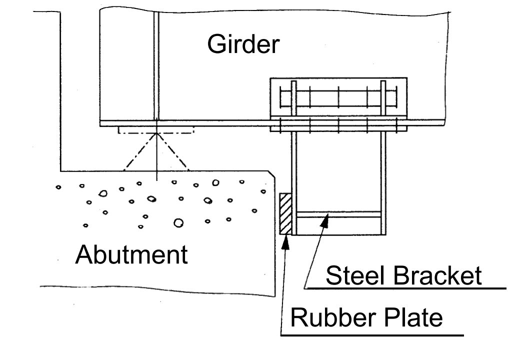
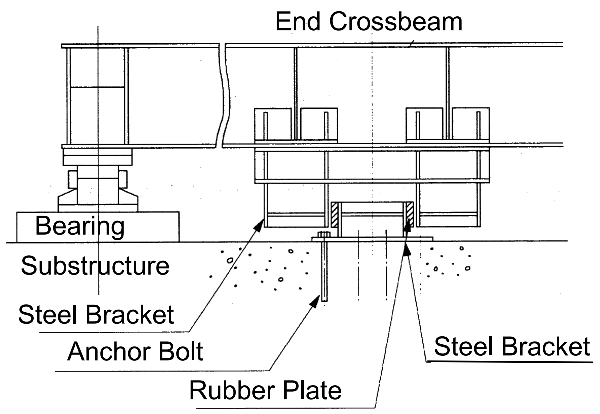
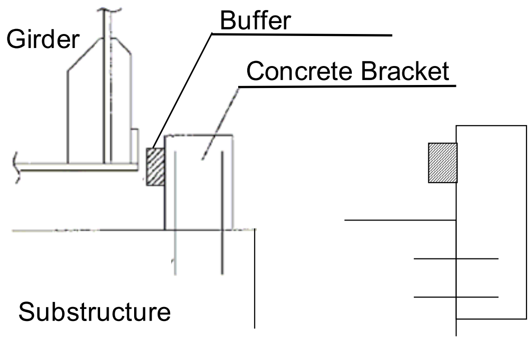
Table 11.5.5. Structure Connecting Two Girders

Outline of structure	Feature
 	-Applicable fro only small movement of -Applicable for the substructure with enough space on top. - Cannot resist the movement of transversal direction -This type one may restrict the deformation by temperature or live load such as movement or rotation of bearing. -Cannot absorb the impact by earthquake. -Reinforcing of web plate of girder is required. -Accuracy management will be required since it is difficult to assimilate the error of manufacture in the stage of construction.
 	-Applicable for large movement of superstructure -This type structure is available in the case that there is not enough space top of the substructure. -Applicable for the substructure with enough space on top . -Applicable for the structure with obstacle near the end crossbeam. -Can absorb the impact of earthquake force. -Reinforcing of the girder is need. -Available for movement in any direction -Error of manufacture can be assimilated in the stage of construction. -If necessary the chain shall be covered by rubber
 	-Applicable for large movement of superstructure . -When there are some obstacles around end crossbeam, it can be installed. -Can absorb the impact of earthquake force. - Available for movement in any direction -Reinforcing of web plate of girder is required. -Rust proof treatment for steel wire shall be done -Adjustment for length of steel wire shall be needed -Error of manufacture can be assimilated in the stage of construction. -Cannot connect the girders, which axis of girder is not same position -Reinforcing web plate of superstructure is need.- -Error of manufacture can be assimilated in the stage of construction. -Checking the position of steel wire or penetrated hole shall be required to avoid interference with anther member

3) Structures Limiting Excessive Displacement

Details of the structures limiting excessive displacement are shown in Table 11.5.6.

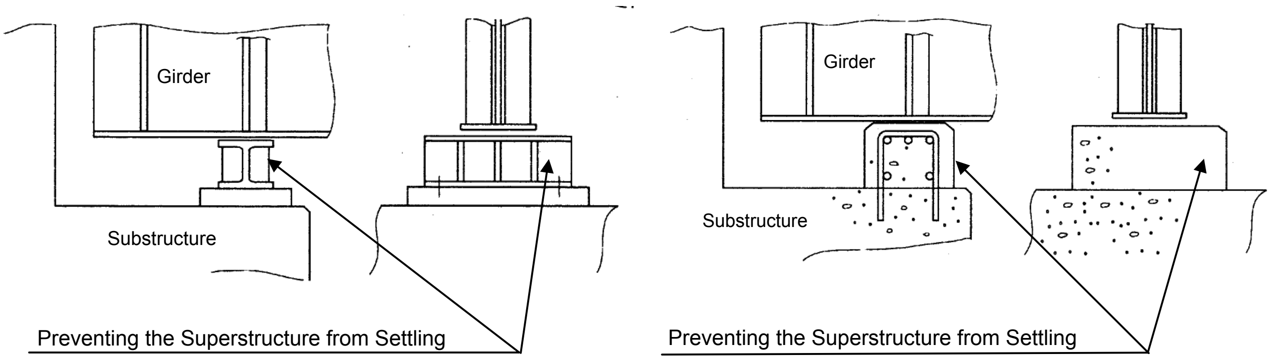
Table 11.5.6. Structure limiting Excessive Displacement

Outline of structure	Feature
	-To limit the movement of longitudinal direction to bridge axis. -In order to absorb the impact of seismic force, rubber plate is fixed on the bracket. - The bracket is fixed with H.T.B. on the girder. - Reinforcement of the girder is required.
	-To limit the movement of transversal direction. -In order to absorb the impact of seismic force, rubber plate is fixed on the bracket. -Not applicable where the obstacles exist around the end crossbeam - Required to check the position of the bearings. - The bracket is fixed with H.T.B. on the girder, and the stopper is set on the substructure with anchor bolts. - The end crossbeam shall be reinforced.
	-Concrete protuberance type stopper set on the substructure with anchor bars. -To be set on the side face of substructure, the top of substructures does not enough -Can be set between the internal girders -Any structures are not required fro the girders (Easy to construction and cheapest)

4) Structures for Preventing Superstructure Settlement

The structure for preventing superstructure settlement keeps the differences in level on surface of bridge in minimum level, which vehicles can pass thorough the bridge after an earthquake. The destruction of the tall bearing supports by earthquake is one of the causes occurring the difference in surface. Therefore, this structure for prevention is effective for the bridges having tall bearing. If the difference in level is less than 10 cm, it may not be fatal obstacle to the movement of emergency vehicles. Therefore, this structure supports the superstructure temporarily and shall be designed to have an enough capacity to support the load of superstructure.

Figure 11.5.6 shows the examples of the structure for preventing superstructure settlement.



a) In the case of using steel H-beam for structure

b) In the case of casting concrete for structure

Figure 11.5.6. Required Dimensions for Concrete Bracket

5) Reinforcement for Hinge of Cantilever Bridge

The hinge of cantilever bridge are normally reinforced by the following methods.

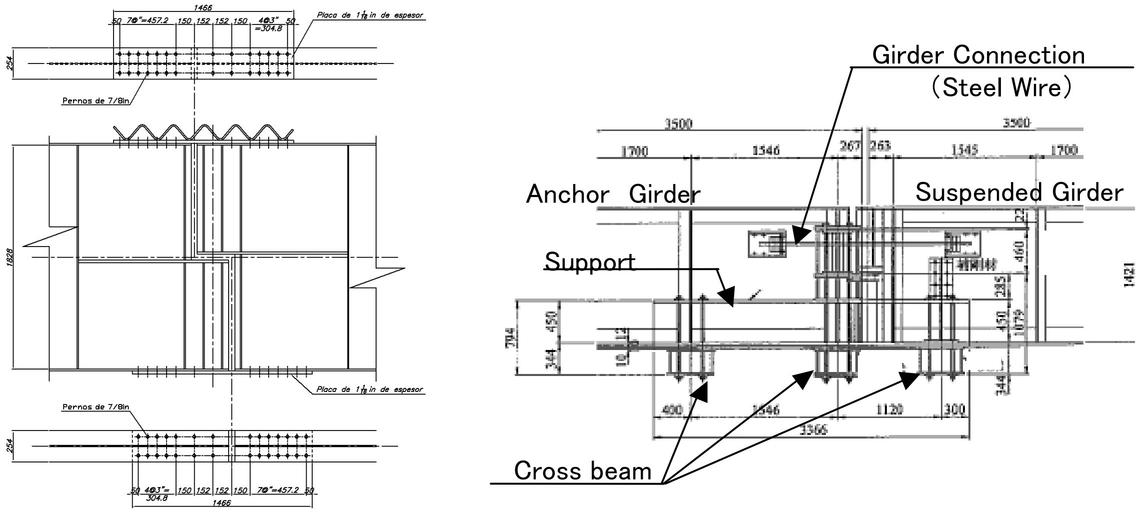
- Continuation of girders
- Widening of seat length
- Connection of girders

Figure 11.5.7. shows the examples of reinforcement for the hinge of cantilever bridge. These methods are compared as shown in Table 11.5.7. The Method of the continuation of girder is the most applicable method, however they shell be evaluated based on the characteristics of the target bridge.

Table 11.5.7. Reinforcement of Hinge of Cantilever Bridge

	Continuation of Girder	Widening of Seat Length	Connection of Girders
Performance of seismic	A	B	C
Trafficability	A	B	B
Improvement in load capacity	A	B	C
Traffic control during construction	C	B	B
Maintenance after Reinforcement	A	B	C
Degree of Difficulty of Execution	A	C	B

A: Good, B: fair, C : Bad



(a) Continuation of Girder **(b) Widening Seat Length**

Figure 11.5.7. Reinforcement for Hinge of Cantilever Bridge

11.5.4 Existing Conditions and Required Measures

No. 7 Azfurado Bridge is a rigid frame RC deck slab bridge and dose not have abutment, so it is not required to estimate the prevention system. 9 bridges except no.7 Azfurado Bridge were studied on the prevention system for bridge collapse and the existing condition and necessary measure on each bridge are described below.

Both No.16 Nuevo Bridge and No.19 Sarapiqui Bridge were changed their support conditions to improve the seismic performance. The prevention systems for bridge collapse for these two bridge were evaluated for improved structure.

1) Seating Length of Girder

The existing condition of seat length in each bridge and necessary seat length of girder are as shown in Table 11.5.8. Only No.17 Chirripo Bridge and No.20 Sucio Bridge satisfy the required seat length, and other 7 bridges do not satisfy the required seat length.

The No.19 Sarapiqui Bridge is cantilever bridge with 2 hinges between P1 and P2 at present. To improve seismic performance, 2 series girders will be connected together at hinge points. Therefore the prevention system for bridge collapse for Sarapiqui Bridge was designed for improved 3 span continuous bridge. .

Table 11.5.8. Existing Condition and Required Seat Length

Route No.	Bridge No.	Bridge Name	Type of Bridge	Total Length (m)	Member	Suport Condition	Existing Condition: Width (m) L	T	Hight (m)	Angle (deg)	Span Length	Seat Length: Required (m)	Existing (m)	Lack Length(m)	Addition (m)
1	2	Rio Aranjues	3 Spans Coninuous Truss Bridge	87.782	A1	M	0.762	9.246	9.373	0	63.4	1.01699	0.6096	0.40739	0.41
					P1	M	1.067	6.858	14.63	0	–	–		–	–
					P2	F	1.067	6.858	14.63	0	–	–		–	–
					A2	M	0.762	9.246	10.72	0	24.38	0.82192	0.6096	0.21232	0.3
	3	Rio Abangares	Single Span Truss Bridge	39.319	A1	F	0.838	10.52	13.41	0	39.32	0.8966	0.5842	0.3124	0.32
					P1	M	1.524	10.21	11.13	0	39.32	0.8966	0.7112	0.1854	0.3
				60.96	P1	M				0	60.96	1.0048	0.7112	0.2936	0.3
					A2	F	0.838	10.26	12.34	0	60.96	1.0048	0.5842	0.4206	0.43
	7	Rio Azfurado	3 Spans Riged Frame RC Deck Slab Bridge	31.394	P1		–	–	–	–	–	–	–	–	–
					P2		–	–	–	–	–	–	–	–	–
2	12	Rio Puerto Nuevo	Single Span Non Composite Steel I Beam	21.336	A1	F	0.457	9.144	9.154	5.028	–	–	–	–	–
					P1	M	0.914	7.62	16.61	5.028	21.34	0.80668	0.28734	0.51934	0.52
			Single Span Non Composite Steel I Beam	21.336	P1	M				0	21.34	0.80668	0.28734	0.51934	0.52
					P2	F	0.914	7.62	17.53	0	–	–	–	–	–
			Single Span Non Composite Steel I Beam	21.336	P2	F				0	–	–	–	–	–
					P3	M	0.914	7.62	19.05	0	21.34	0.80668	0.28734	0.51934	0.52
			Single Span Composit Steel I Beam	24.384	P3	M				0	24.38	0.82192	0.28734	0.53458	0.54
					P4	F	0.914	7.62	18.14	0	–	–	–	–	–
			Single Span RC Deck Slab Bridge	15.24	P4	M				0	15.24	0.7762	0.31274	0.46346	0.47
					A2	F	0.533	9.042	7.01	0	–	–	–	–	–
	16	Rio Nuevo	3 Spans Coninuous RC Deck Slab Bridge	55.474	A1	M	0.533	9.042	2.613	0	17.07	0.78534	0.4572	0.32814	0.33
					P1	F	–	7.925	8.153	0	–	–	–	–	–
					P2	M(F)	–	7.925	8.153	0	–	–	–	–	–
					A2	M	0.533	9.042	2.613	0	17.07	0.78534	0.4572	0.32814	0.33
4	17	Rio Chirripo	3 Spans PC Box Girder Bridge	175.8	A1	M	1.75	10.9	16.13	0	46.5	0.9325	1.7	−0.7675	–
					P1	Stopper	200	520	15.79	0	–	–		–	–
					P2	Stopper	200	520	15.79	0	–	–		–	–
					A2	M	1.75	10.9	16.13	0	46.5	0.9325	1.7	−0.7675	–
	19	Rio Sarapiqu	3 Spans Steel I Beam Gerbaer Bridge	99.56	A1	M(F)	1.35	8.1	3.34	0	99.56	1.1978	1.1	0.0978	0.3
					P1	M	0.9	8	17.9	0	–	–	–	–	–
					H1	Cont(M)	0.254		0	0	–	–	–	–	–
					H2	Cont(F)	0.254		0	0	–	–	–	–	–
					P2	M	0.9	8	17.9	0	–	–	–	–	–
					A2	F	1.35	8.1	3.34	0	–	–	–	–	–
32	20	Rio Sucio	3 Spans PC Box Girder Bridge	187.25	A1	M	1.5	10.6	20.45	0	55.25	0.97625	1.4	−0.4238	–
					P1	Stoper	5.2	5.2	30.99	0	–	–	–	–	–
					P2	H	1.5	5.2	7.18	0	–	–	–	–	–
	26	Rio Chirripo	6 Spans Continuous Steel I Beam Bridge	399.18	P1	M	0.82	8.9	16.37	0	126.4	1.33195	0.7	0.63195	0.64
					P2	M	1.5	8.9	15.95	0	–	–	–	–	–
					P3	F	1.5	8.9	16.19	0	–	–	–	–	–
					P4	F	1.5	8.9	16.19	0	–	–	–	–	–
					P5	F	1.5	8.9	16.19	0	–	–	–	–	–
					P6	M	1.5	8.9	15.95	0	–	–	–	–	–
					P7	M	0.82	8.9	16.37	0	126.4	1.33195	0.7	0.63195	0.64
			Single Span Non Composite Steel I Beam	15.86	P7	M	0.3	8.1		0	15.86	0.7793	0.3	0.4793	0.48
					A2	F	0.53	10.32	2.99	0	15.86	0.7793	0.43	0.3493	0.35
218	29	Rio Toress	Single Span PC I Girder Bridge	30	A1	F	0.55	11.68	12.63	0.278	30	0.85	0.515	0.335	0.34
					P1	M	1.1	11.1	13.02	0.278	30	0.85	0.515	0.335	0.34
			Single Span PC I Girder Bridge	17	P1	M				0.328	17	0.785	0.45	0.335	0.34
					P2	F	1.1	11.1	9.06	0.328	17	0.785	0.45	0.335	0.34
			Single Span PC I Girder Bridge	17	P2	M			9.06	6.186	17	0.785	0.45	0.335	0.34
					A2	F	0.6	11.87	7.67	6.186	17	0.785	0.45	0.335	0.34

2) Girder Connection

The types of the girder connection selected for the bridges are as shown in Table 11.5.9. and 11.5.10. The chain type connection was applied for the steel bridges and the concrete protuberance type was applied for the concrete bridges.

Table 11.5.9. Selected Girder Connection

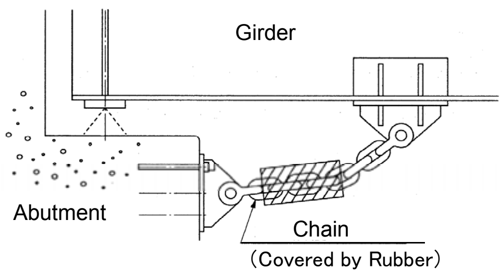
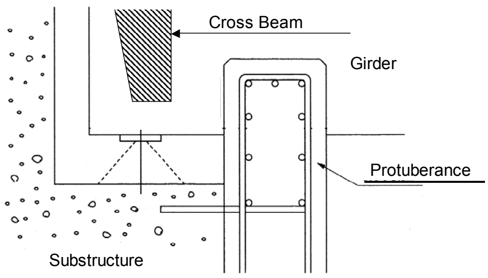
Type of Bridge	Type of Girder Connection	Reason for selection
Steel Bridge	The Chain type structure which is connecting the superstructure to the substructure 	- There is not enough space to install bracket in top of substructure -When a protuberance form structure is employed reinforcement of end cross beam will be required. -The construction and maintenance is easiest in 3 types of structure for connecting the superstructure to the substructure -This type structure can be applied any type of bridge -If necessary the chain shall be covered by rubber.
Concrete Bridge	A protuberance form structure which is made of casting concrete in site 	-This type structure is cheapest and easy to construction and maintenance. -An end cross- beam of concrete bridge has enough capacity against earthquake loadings. -When the bracket is installed to concrete girder, re-bar layout of girder must inspect before install.

Table 11.5.10. Type of Unseating Prevention Structure

Bridge No.	Bridge Name	Type of Bridge	Total Length (m)	Member	Suport Condition	Angle (deg)	0.75SE	Unseating Prevention structure	Limiting Excessive displacement	
									Longitudinal	Transversal
2	Rio Aranjues	3 Spans Coninuous Truss Bridge	87.782	A1	M	0	0.77	N/A	N/A	Concrete Block(1.0*0.25*0.66) inside of girder
				P1	M	0	–	N/A	N/A	Concrete Block(1.0*0.6*0.75) inside of girder
				P2	F	0	–	N/A	N/A	Concrete Block(1.0*0.6*0.75) inside of girder
				A2	M	0	0.62	N/A	N/A	Concrete Block(1.0*0.35*0.66) inside of girder
3	Rio Abangares	Single Span Truss Bridge	39.319	A1	F	0	0.68	Chain type	N/A	Concrete Block(0.4*1.5*1.65) out side of girder
				P1	M	0	0.68		N/A	Concrete Block(0.4*0.7*2.1) inside of girder
			60.96	P1	M	0	0.76		N/A	
				A2	F	0	0.76		N/A	Concrete Block(0.4*1.5*1.8) out side of girder
7	Rio Azfurado	3 Spans Riged Frame RC Deck Slab Bridge	31.394	P1		–	–	N/A	N/A	N/A
				P2		–	–	N/A	N/A	N/A
12	Rio Puerto Nuevo	Single Span Non Composite Steel I Beam	21.336	A1	F	5.028	–	Chain type	N/A	Concrete Block(0.6*0.3*1.0) out side of girder
				P1	M	5.028	0.61		N/A	Concrete Block(0.9*0.3*2.0) between 2nd and 3rd girder
		Single Span Non Composite Steel I Beam	21.336	P1	M	0	0.61		N/A	
				P2	F	0	–		N/A	
		Single Span Non Composite Steel I Beam	21.336	P2	F	0	–		N/A	
				P3	M	0	0.61		N/A	
		Single Span Composit Steel I Beam	24.384	P3	M	0	0.62		N/A	
				P4	F	0	–		N/A	
		Single Span RC Deck Slab Bridge	15.24	P4	M	0	0.59	Concrete block (1.7*1.0*0.3)*2	N/A	
				A2	F	0	–		N/A	Concrete Block(0.6*0.3*1.0) out side of girder
16	Rio Nuevo	3 Spans Coninuous RC Deck Slab Bridge	55.474	A1	M	0	0.59	Concrete block (1.7*1.0*0.3)*2	Concrete block	Concrete Block(0.9*0.4*0.826) out side of girder
				P1	F	0	–	–	–	N/A
				P2	M(F)	0	–	–	–	N/A
				A2	M	0	0.59	Concrete block (1.7*1.0*0.3)*2	N/A	Concrete Block(0.9*0.4*0.826) out side of girder
17	Rio Chirripo	3 Spans PC Box Girder Bridge	175.8	A1	M	0	0.7	N/A	N/A	Already intalled
				P1	Stopper	0	–	N/A	N/A	N/A
				P2	Stopper	0	–	N/A	N/A	N/A
				A2	M	0	0.7	N/A	N/A	Already intalled
19	Rio Sarapiqu	3 Spans Steel I Beam Gerbaer Bridge	99.56	A1	M(F)	0	0.9	Chain type	N/A	Concrete Block(1.0*0.45*1.6) out side of girder
				P1	M	0	–	N/A	N/A	N/A
				H1	Cont(M)	0	–	–	N/A	–
				H2	Cont(F)	0	–	–	N/A	–
				P2	M	0	–	N/A	N/A	N/A
				A2	F	0	–	Chain type	N/A	Concrete Block(0.9*0.4*0.826) out side of girder
20	Rio Sucio	3 Spans PC Box Girder Bridge	187.25	A1	M	0	0.74	N/A	N/A	Already intalled
				P1	Stoper	0	–	N/A	N/A	–
				P2	H	0	–	N/A	N/A	–
26	Rio Chirripo	6 Spans Continuous Steel I Beam Bridge	399.18	P1	M	0	1	Chain type	N/A	Concrete Block(1.35*0.6*1.47) in side of 1st and 4th girder
				P2	M	0	–	–	N/A	–
				P3	F	0	–	–	N/A	–
				P4	F	0	–	–	N/A	–
				P5	F	0	–	–	N/A	–
				P6	M	0	–	–	N/A	–
				P7	M	0	1	Chain type	N/A	Concrete Block(1.35*0.6*1.47) in side of 1st and 4th girder
		Single Span Non Composite Steel I Beam	15.86	P7	M	0	0.59	Chain type	N/A	Concrete Block(0.3*1.5*0.65) outside of 1st and 4th girder
				A2	F	0	0.59	Chain type	N/A	
29	Rio Toress	Single Span PC I Girder Bridge	30	A1	F	0.278	0.64	Concrete block (1.7*1.0*0.3)*2	N/A	Already intalled
				P1	M	0.278	0.64		N/A	
		Single Span PC I Girder Bridge	17	P1	M	0.328	0.59		N/A	
				P2	F	0.328	0.59		N/A	
		Single Span PC I Girder Bridge	17	P2	M	6.186	0.59		N/A	
				A2	F	6.186	0.59		N/A	

3) Structures Limiting Excessive Displacement

The concrete protuberance type was selected for the structure limiting excessive displacement both longitudinal and transversal directions, because of its cost and the advantages for the construction work together with the concrete bracket for widening of seat length.

4) Prevention Structure for Superstructure Settlement

This structure was installed on P1 and P2 of No. 2 Aranjues Bridge, on P1 and A2 of No.3 Abangares Bridge and on P1 and P7 of No.26 Chirripo Bridge, which have tall bearings. Concrete type structure is selected by reasons of cost and the easiness of construction and maintenance.

As No. 3 Aranjues Bridge is a 3 span continuous bridge, generally it is not required this kind of structure at the movable support pier. However this bridge is variable depth deck truss, so if girder fall down from bearing, the lower chord member may have a fatal damage. Therefore, it is preferable to install this structure for all substructures even for P2 pier.

5) Reinforcement for a Hinge of Cantilever Bridge

No. 19 Sarapiqui Bridge has the hinges of cantilever bridge. To improve seismic performance, the adjacent girders were connected together at the hinge.

11.6 Substructure

The substructure consists with the transversal beam and the pier. Among 10 bridges, T-shape pier exists in 4 bridges, No.12 Rio Puerto Nuevo Bridge, No.19 Sarapiqui Bridge, No.26 Chirripo Bridge and No.29 Tress Bridge

A transversal beam shall be reinforced to withstand dead load and new live load (HS20+25%). On the other hand the reinforcement of pier shall be designed against not only dead load and the new live load (HS20+25%) but also earthquake loadings. The earthquake load may be the most critical load case to the pier.

11.6.1 Method for Reinforcing Substructure

Table 11.6.1 shows the principal examples of reinforcement method for transverse beam. The reinforcement methods for piers are shown Table 11.6.2. As some of the reinforcement methods increase the weight of substructure, the method shall be selected taking account of the effects for the influence on the foundation. The repair works are required to execute together for damaged parts of the structure.

Table 11.6.1. Reinforcement Methods for Transversal Beam

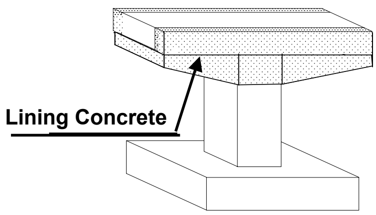
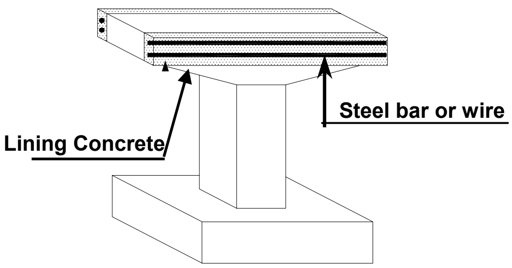
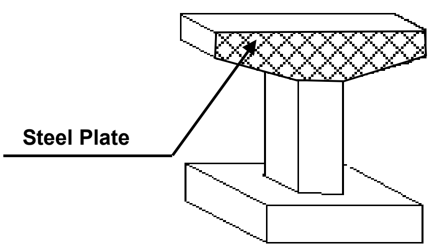
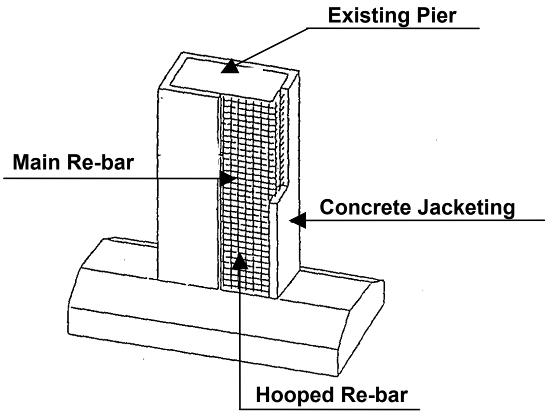
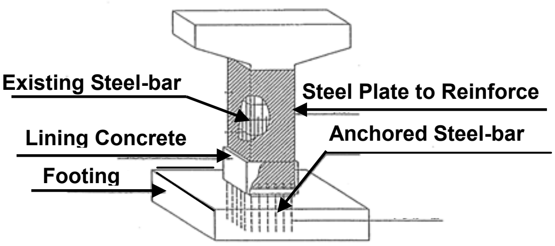
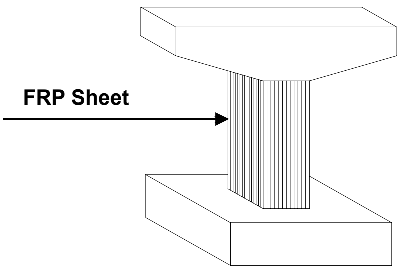
Reinforce Method	Figures	Description of Method
Increase Dimension - Height and/or width is increased by lining concrete	 	By increasing dimension of beam the resisting moment and shear resistance is increased. Dead load of beam is increased Construction is easy
Introducing of Pre-Stress -PC bar or wire is arranged out side of existing transversal beam.	 	By introducing pre-stress into the beam, the resisting moment is increased. The crack of the concrete will be closed or decreased its width. For the resisting shear strength this method is not so much effective. The careful execution management for PC work is required
Reinforce by Additional Material -Steel Plate Bonding	 	By bonding steel plate the shear resistance is increased. The resisting moment cannot be increased since steel plate can be bonded only side face of beam.

Table 11.6.2. Reinforce Methods for Pier

Reinforce Method	Figures	Description of Method
Increase Dimension - Concrete Jacketing		By concrete jacketing on the existing concrete, the stiffness of pier is increased. Dead load of substructure is increased Attention is necessary for an influence to the river flow section, because of increased section. Generally it is economical comparing with other method. Control of maintenance is easy
Reinforce by Additional Material -Rolling Up Steel Plate -Rolling Up FRP Sheet	 	By Rolling on the existing pier with additional Material, the resisting capacities for moment and shear force are increased. Additional material is bonded by non-shrinking mortar or epoxy resin. After bonding additional material, it is difficult to inspect cracks or deterioration of existing concrete. The FRP sheet is flammable material and apt to be deteriorated by ultraviolet rays or ozone Car accident or rolling stone may damage the FRP sheet.

11.6.2 Selection of Reinforcement Method for Substructure

The reinforcing work for transversal beam and pier will be carried out in a narrow space under superstructure. The almost piers locate in the river flow. Therefore, it is important to consider the condition of construction, maintenance, structural characteristic of method and the cost.

1) Transversal Beam

Among the 10 bridges, No.12 Puerto Nuevo Bridge, No.19 Sarapiqui Bridge, No.26 Chirripo Bridge and No.29 Torres Bridge have T- shape pier. Table 11.6.3 shows the comparison of method of reinforcement for the transversal beam, with the consideration the site conditions and the load capacity of existing condition. The section increase method was selected by the reason below.

- This method is most economical
- Its construction is easier than other method
- It is possible to inspect concrete condition after reinforcement.
- Above 4 bridges can be reinforced by this method and widening size is same as necessary seat length.

Table 11.6.3. Comparison of Method of Reinforcement Method for Beam

Item \ Method		(a) Section Increase	(b) Introducing of Pre-Stress	(c) Reinforce by Additional Material
Construction		Easy	Required careful execution management for PC work	Not so easy (Steel plate is Heavy)
Maintenance		Easy	Easy	Cannot inspect concrete condition because steel plate cover surface of concrete
Cost		Cheapest	Higher than (a)	Higher than (a)
Period		Short	Longer than (a)	Longer than (a)
Structural Characteristic	Moment	Applicable	Applicable	Not Applicable
	Shear	Applicable	Not Applicable	Applicable
Protection		Not required	Not required	Corrosion protection is necessary
Evaluation		**Applicable and Best**	**Not Applicable**	**Not Applicable**

2) Pier

Among 10 bridges, the cylinder type pier exisst in No.12 Puerto Nuevo Bridge (D=1.8m), No.20 Sucio Bridge (D=5.2m), and No.29 Torres Bridge (D=1.5m). The piers of the other bridge are wall type or ellipse with longitudinal thickness of around 2 m.

Table 11.6.4 shows comparison of method of reinforcement for pier, in consideration of existing site conditions and the load capacity. The concrete jacketing method is selected by the reasons below.

- Concrete jacketing is most economical methods
- Among 10 bridges, the highest blocking ratio of the river flow area is 7% in Rio Puerto Nuevo, in other bridges it is around 5%. Although pier width will be increased 50cm by concrete jacketing, the increased blocking ratio is about 1% to 2%, Therefore, this method will not affect river flow.
- This method is same as normal concrete work, so its construction is easier

than other methods.
- It is possible to inspect the concrete condition after reinforcement.

Table 11.6.4. Comparison of Method of Reinforcement Method for Pier

Item \ Method	Concrete Jacketing	Rolling Up Steel Plate	Rolling Up FRP Sheet
Construction	Easy (General concrete structure work)	Not so easy (Steel plate is Heavy)	Easy (light material)
Maintenance	Easy (General concrete structure work)	Cannot inspect concrete condition Because steel plate cover surface of concrete	
Cost	Cheapest	High	Highest
Period	Long	Middle	Short
Influence on river flow	Increasing width is 50cm, it not so much affect on river flow(the blocking ratio is about 6% to 9%)	Nothing	Nothing
Protection	Not required	Corrosion protection is necessary	Protection for ultraviolet rays or ozone Protection for rolling stone or river flow
Evaluation	**Applicable and Best**	**Applicable**	**Not applicable**

11.6.3 Methodology of Design for Reinforcing of Beam and Pier

The section increasing method was selected for the reinforcement of both the transversal beam and the pier. The Load Factor Design Method in according to AASHTO was applied to design for the reinforcement of substructure.

The load capacity for the beams can be checked by composition of both ratio of resistance at original dimension to working force due to dead load, and ratio of resistance at increased section to working force due to increased weight and new Live Load (HS20+25%) or the seismic load.

$$\frac{Muo}{\phi Mo} + \frac{Mua + Mue}{\phi Mm} \leq 1.0$$

$$\frac{Vuo}{\phi Vo} + \frac{Vua + Vue}{\phi Vm} \leq 1.0$$

where

:

Muo (Vuo,Nuo)	:	Bending Moment (Shear force, Axial force) caused by original dead load
Mua (Vua,Nua)	:	Bending Moment (Shear force) caused by additional dead load
Mue (Vue,Nue)	:	Bending Moment (Shear force) caused by earthquake loadings or new live load
ϕMo (ϕVo, ϕNo)	:	Design Moment (shear force) strength for original section
ϕMm(ϕVm, ϕNm)	:	Design Moment (shear force) strength for modified section (Original section + Reinforced section)

11.6.4 Existing Condition and Condition after Reinforcement

1) Transversal Beam

The 4 bridges, which are No.12 Puerto Nuevo Bridge, No.19 Sarapiqui Bridge, No.26 Chirripo Bridge and No.29 Torres Bridge, have transversal beam. Table 11.6.5 shows the results of check of the load capacity for the increased live load (HS20+25%). The 3 bridges except Sarapiqui Bridge, are required to reinforce for the transversal beam for the increased live load.

For the Puerto Nuevo Bridge and the Torres Bridge, the load capacity for the bending moment is critical, and these bridges also required widen the beam width to keep seat length of girder at support. On the other hand, the shear force is critical in the Chirripo Bridge, it can be reinforced by increasing width of beam with the stirrup (re-bar size #6, 14cm pich).

Table 11.6.6 shows the conditions after reinforcement by the method of section increase.

Table 11.6.5. The Load Carrying Capacity of transversal Beam

Bridge Name		Member	Original Condition											
			Size		Dead		Live Load		Dead+Live		φMo	φVo	φMo/Muo	φVo/Vuo
			B	H	Mud	Vud	MuL	VuL	Muo	Vuo				
12	Rio Puerto Nuevo	P2	91.44	182.88	175.23	39.65	353.77	80.05	529.00	119.70	464.8	198.5	0.88	1.66
		P3	91.44	182.88	193.90	43.88	357.18	80.82	551.08	124.70	464.8	198.5	0.84	1.59
		P4	91.44	182.88	235.55	53.30	346.12	78.32	581.67	131.62	464.8	198.6	0.80	1.51
19	Rio Sarapiqui	P1	90.00	180.00	172.17	85.02	198.13	97.84	370.30	182.86	409.0	185.3	1.10	1.01
		P2	90.00	180.00	172.17	85.02	198.13	97.84	370.30	182.86	409.0	185.3	1.10	1.01
26	Rio Chirripo	P4	150.00	244.00	405.60	208.00	383.00	160.87	788.60	368.87	825.3	286.4	1.05	0.78
29	Rio Torres	P1	110.00	220.00	490.50	141.70	723.34	160.74	1213.8	302.44	1067.4	432.1	0.88	1.43
		P2	110.00	200.00	318.00	91.87	692.75	153.94	1010.75	245.81	713.8	247.2	0.71	1.01

Table 11.6.6. The Load Carrying Capacity of transversal Beam after Reinforcing

			Required Size		Aditional Section		Evaluation					
							(1) Original Condition		(2) After Reinforce		(1)+(2)	
			B	H	φMA	φVA	Mud/φMo	Vuo/φVo	MuL/φMA	VuL/φVA		
12	Rio Puerto Nuevo	P2	195.44	182.88	638.44	404.46	0.38	0.20	0.55	0.20	0.93	0.40
		P3	195.44	182.88	638.44	404.56	0.42	0.22	0.56	0.20	0.98	0.42
		P4	195.44	182.88	638.44	404.67	0.51	0.27	0.54	0.19	1.05	0.46
26	Rio Chirripo	P4	210.00	244.00	1007.01	688.86	0.49	0.73	0.38	0.23	0.87	0.96
29	Rio Torres	P1	178.00	220.00	1563.54	517.25	0.46	0.33	0.46	0.31	0.92	0.64
		P2	178.00	200.00	1159.68	441.80	0.45	0.37	0.60	0.35	1.04	0.72

2) Pier

The existing conditions of pier are shown in Table11.6.7. From this table, 2 piers, which are

P2 pier of No.2 Aranjues Bridge and P1 pier of No.3 Abangares Bridge, have not enough capacity against the seismic load. Therefore, these substructures were reinforced by the method of concrete jacketing and their width of longitudinal direction of the pier has increased with 50 cm (25cm increased each side).

The conditions of piers after reinforcement are shown in Table 11.6.8.

Table 11.6.7(a). The Load Carrying Capacity of Pier at Existing Condition

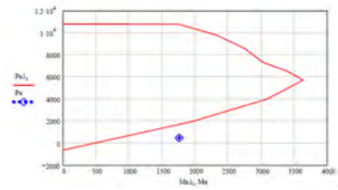
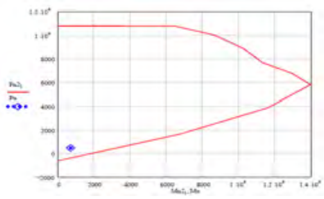
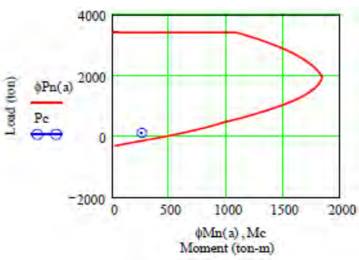
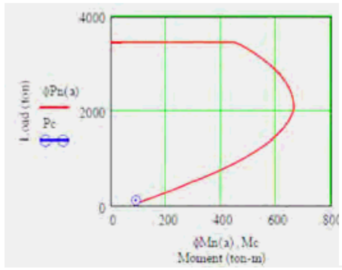
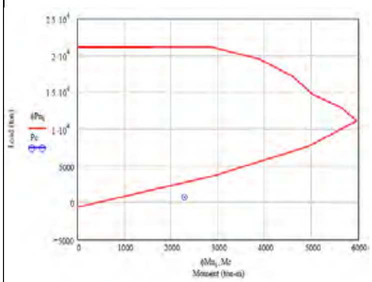
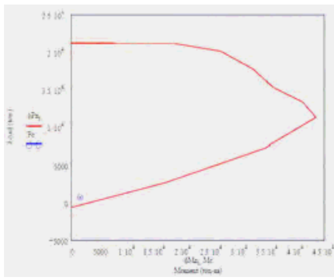
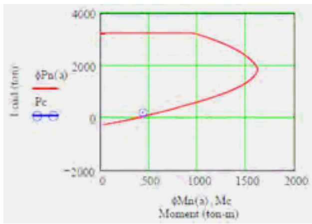
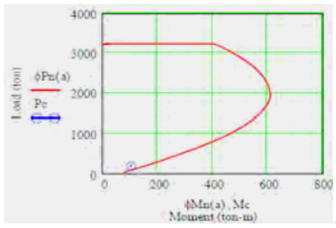
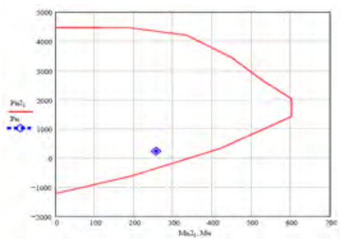
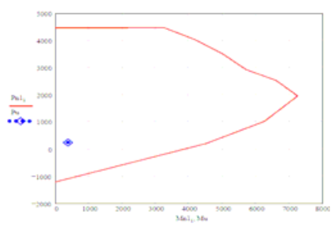
Bridge Name (No.)		Type of pier	Longitudinal	Transversal	Evalu-ation
Rio Aranjues (No.2)	P2	Wall			Not Ok
Rio Abangares (No.3)	A1	Rigid Frame			OK
	P1	Wall			Not OK
	A2	Rigid Frame			OK
Rio Azufrado (No.7)	P1 P2	Wall			OK

Table 11.6.7 (b). The Load Carrying Capacity of Pier at Existing Condition

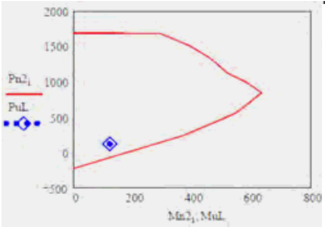
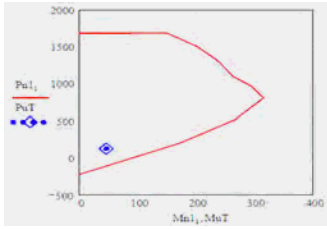
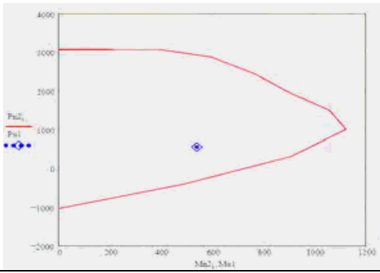
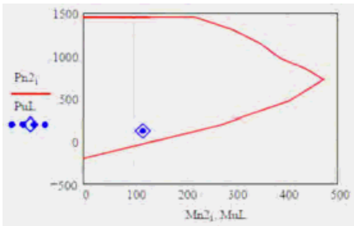
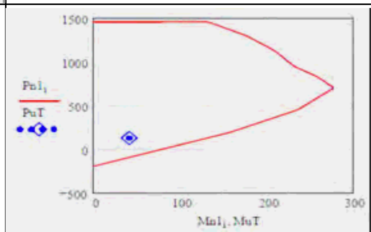
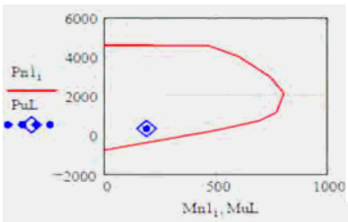
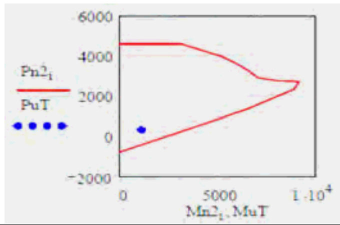
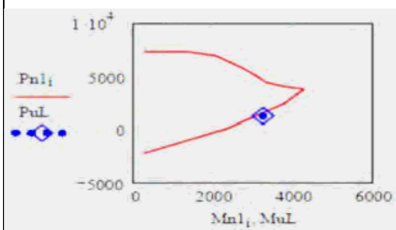
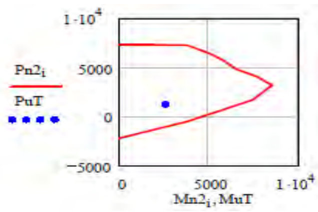
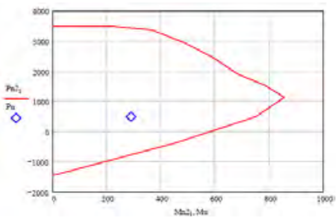
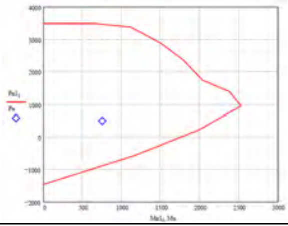
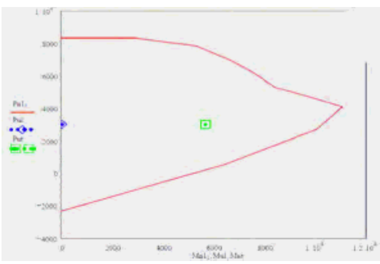
Bridge Name (No.)		Type of pier	Longitudinal	Transversal	Evaluation
Rio Puerto Nuevo (No.12)	A1	Rigid Frame			OK
	P1 To P4	Cylinder		----	OK
	A2	Rigid Frame			OK
Rio Nuevo (No.16)	P1 P2	Wall			OK
Rio Chirripo (No.17)	P1 P2	Ellipse			OK
Rio Sarapiqui (No.19)	P1 P2	Ellipse			Ok
Rio Sucio (No.20)	P1 P2	Cylinder		-----	OK

Table 11.6.7 (C). The Load Carrying Capacity of Pier at Existing Condition

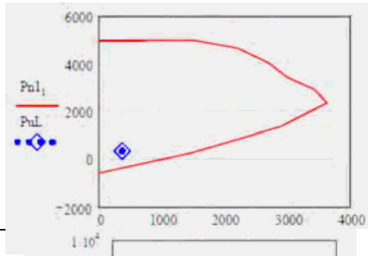
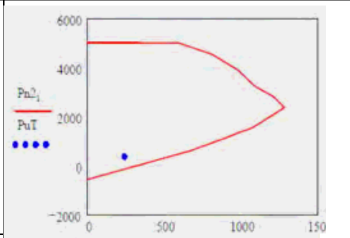
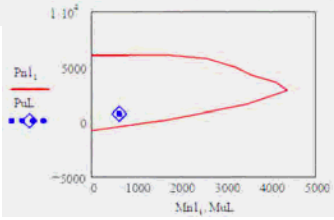
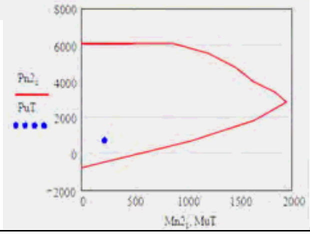
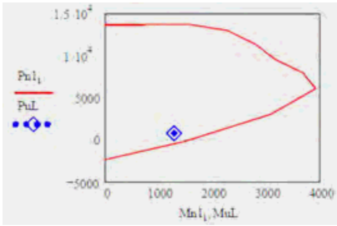
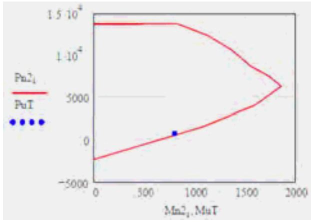
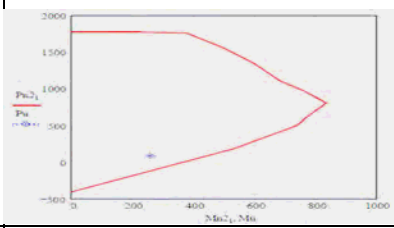
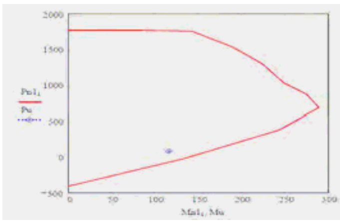
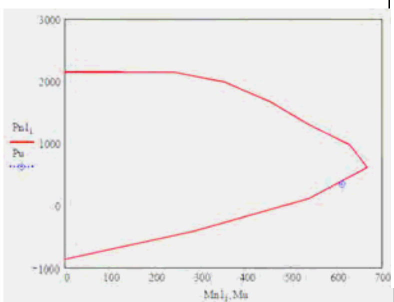
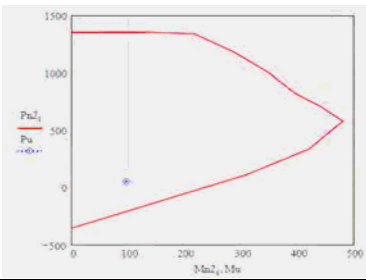
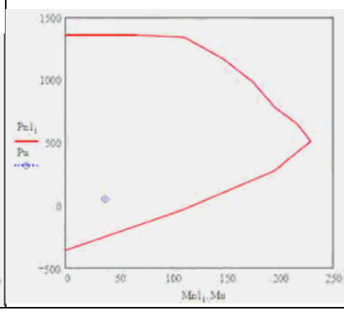
Bridge Name (No.)		Type of pier	Longitudinal	Transversal	Evaluation
Rio Chirripo (No.26)	P1 P7	Ellipse			Ok
	P2 P6	Ellipse			Ok
	P3 P4 P5	Ellipse			Ok
Rio Toress (No.29)	A1	Rigid Frame			Ok
	P1	Cylinder		----------	Ok
	A2	Rigid Frame			Ok

Table 11.6.8 . The Load Carrying Capacity of Pier after reinfoce

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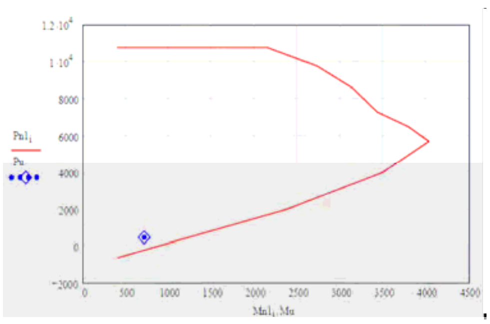
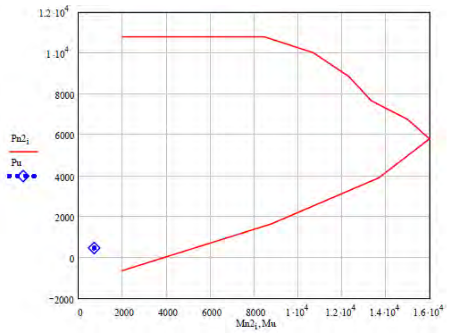
<tr><td>Rio Aranjues (No.2)</td><td>P2</td><td>
</td><td>
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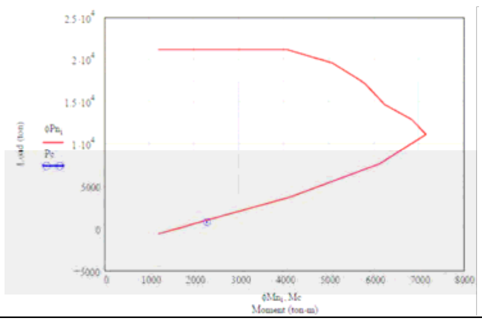
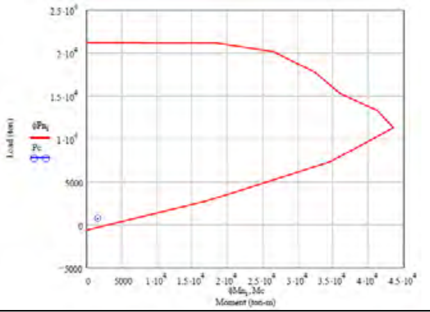
<tr><td>Rio Abangares (No.3)</td><td>P1</td><td>
</td><td>
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