CENTER FOR INFRASTRUCTURE ENGINEERING STUDIES

PRACTICAL ISSUES RELATED TO THE STRUCTURAL PRESERVATION OF EXISTING BRIDGE STRUCTURES

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ABSTRACT

In this paper two design examples are provided that describe how different techniques were used to accomplish the structural preservation and/or maintenance of existing bridge structures in the State of Missouri.

One example describes repair of a bridge structure that suffered damage to one exterior precast/prestressed concrete girder due to vehicular impact. From the impact, two prestressing steel tendons were fractured, resulting in approximate 10% decrease in the moment capacity. To restore the structural capacity of the girder, carbon FRP laminates were used, and re-strengthening was accomplished in one single day without traffic interruption. This case study demonstrated that FRP bonded reinforcement was an effective repair technique for this bridge.

The second example describes a technique that was used for the retrofit of a bridge structure that showed significant spalling of the cover concrete and wide-open cracks in the main span bridge piers. The existing piers for the main span consist of circular boundary elements connected by shear wall panels. No transverse reinforcement was provided in the boundary elements and due to the state of deterioration of the piers it was proposed to retrofit the boundary elements with external prestressing in the diaphragm region and FRP wrapping along the entire length of the boundary elements. This retrofit technique will inhibit further development of cracks while ensuring the best possible retrofit technique for this particular bridge.

REPAIR OF A BRIDGE STRUCTURE AFTER IMPACT LOADING DAMAGE

Bridge A10062, located at the interchange of the Interstate 44 and Interstate 270, with a roadway clearance of 4.47m in St. Louis County, Missouri, was damaged by an overheight truck in one exterior prestressed concrete girder. Removal of the loose concrete showed that two prestressing tendons were fractured due to the impact (see **Figure 1**).

Repair methods that deal with the repair of vehicular impact to bridge girders are rare. Techniques such as external post-tensioning and internal strand splices were found to be delicate to the repetitive loading nature of highway. Researchers have indicated that these methods can only provide durable repairs in order to extend the usable life of the structure rather than to restore ultimate strength to a damaged member *(1,2)*. Therefore it was decided to use carbon fiber reinforced polymer (CFRP) laminates to restore the original structural capacity of the girder due to its many advantages compared to traditional repair techniques. To name some, for the specific case of bridges strengthening, FRP laminates are easy to install, the length of the repair work is shortened, and minimum traffic disruption is caused.

Analysis of Damaged Bridge

The cross section of the damaged girder and prestressing details are shown in **Figure 2**. Girder flexural reinforcement consisted of 20 low relaxation prestressing strands with an ultimate strength of 1862MPa. Properties of material used in the girder are shown in **Table 1**. It was assumed that a portion of the deck with the dimensions of 21.6cm thick $\times$ 122cm wide forms composite action with the girder as shown in **Figure 2**.

The nominal moment capacity of the girder was determined by the ACI *(3)* rectangular stress block approach. The stress in the tendons should be determined from an appropriate equation for the stress-strain relationship of the particular prestressing steel. The PCI *(4)* Handbook gives the following equations for Grade 270 tendons:

$$f_{ps} = e_{ps} E_{ps} \qquad \text{for } e_{ps} \leq 0.008$$

$$f_{ps} = f_{pu} \frac{0.52}{(e_p - 0.0065)} - 14 \qquad \text{for } e_{ps} > 0.008 \quad (MPa) \tag{1}$$

Where: f_{ps} is the prestressing steel stress, e_{ps} is the prestressing steel strain, E_{ps} is the prestressing steel Young's Modulus, and f_{pu} is the prestressing steel ultimate strength.

The computed factored moment capacity before damage was $f M_{n(original)}$= 2,841kN-m. As a result of the imposed damage, the capacity of the existing girder was reduced to $f M_{n(damaged)}$= 2,556kN-m. Thus, strengthening had to restore a loss of about 285kN-m of moment capacity at ultimate.

Design of Repair Technique

After the concrete was repaired, a unidirectional CFRP composite system was used to restore the loss of flexural capacity. The suggested system was MBrace CF130 *(5)* with the following properties shown in **Table 2**.

The increased capacity of the strengthened girder generated by CFRP laminates can be computed as:

$$M_{n(FRP)} = t_f \, w_f \, f_f \, (h - c) \tag{2}$$

Where: $M_{n(damaged)}$ is the remaining capacity of girder after damage, t_f is the thickness of one ply of fiber sheet, w_f is the total width of the FRP laminate, f_f is the stress level developed in the FRP, h is the total height of the section and depth to the FRP flexural reinforcement, and c is the depth to neutral axis

The capacity of the repaired girder after the FRP reinforcement strengthening was estimated as:

$$f\, M_{n(repaired)} = f \, (M_{n(damaged)} + M_{n(FRP)}) \tag{3}$$

The flexural strengthening consisted of applying two 45.7cm wide CFRP sheets with the lengths of 285cm and 325cm sequentially to the area of damage (see **Figure 3**). Sixteen strips spaced at 10.2cm were then U-wrapped to both sides of the girder on the top of previous installation, as depicted as in **Figure 4**.

After repair, the capacity of the girder was restored to $f M_{n(repaired)}$= 3,035kN-m, which is 7 % larger than the original capacity.

Repair Implementation

A maintenance crew from the Missouri Department of Transportation conducted the repair with assistance of research personnel from the University of Missouri-Rolla. Before carrying out the CFRP laminates installation, the concrete section of the girder was restored with a mortar consisting of rapid setting patching cement (Conpatch V/O).

The sequential installation details of the CFRP sheets are as follows:

Surface Preparation. The bottom edges of the girder were rounded for proper wrapping (see **Figure 5**). Next, the concrete surface was sandblasted approximately 1.5cm until the aggregate was exposed (see **Figure 6**) and the surface of the concrete was free of loose and unsound materials.

Application of the Primer. A layer of epoxy-based primer was applied to the prepared concrete surface using a short nap roller to penetrate the concrete pores and to provide an improved substrate for the saturating resin (see **Figure 7**).

Application of the Putty. After the primer became tack-free, thin layer of putty was applied using a trowel to level the concrete surface and to patch the small holes (see **Figure 8**).

Application the first layer of the Saturant Resin. Then the first layer of saturant was rolled on the putty using a medium nap roller. The functions of the saturant are: to impregnate the dry fibers, to maintain the fibers in their intended orientation, to distribute stress to the fibers, and to protect the fibers from abrasion and environmental effects (see **Figure 9**).

Application of Fiber Sheets. After each fiber sheet was measured and pre-cut, they were placed on the concrete surface and gently pressed into the saturant. Prior to removing the backing paper, a trowel was used to remove any air void. After the backing paper was removed, a ribbed roller was rolled in the fiber direction to facilitate impregnation by separating the fibers (see **Figure 10** and **Figure 11**).

Application of the Second Layer of Saturant. The sheet was then installed and the excessive resin was removed (see **Figure 12** and **Figure 13**).

RETROFIT OF EXISTING BRIDGE PIERS USING EXTERNAL PRESTRESSING AND GFRP WRAPPING

For the second example, the objective of the research was to provide design and construction guidelines for the application of external circumferential prestressing and Glass Fiber Reinforced Polymer (GFRP) wrapping as a measure for retrofitting of the existing piers of the Bridge over the Gasconade River, in Pulaski County, Missouri.

This bridge consists of one 61.0m main truss span, two 30.5m truss span, and two 10.70m I-girder beam approach spans, as shown in **Figure 14**. The bridge piers that support the 61.0m main span consist of circular-tapered boundary elements connected by shear wall panels (see **Figure 15**). These boundary elements were retrofitted according to the retrofit technique described in this paper. The superstructure is connected at these bridge piers by means of steel rockers mounted at the boundary elements, (see **Figure 15**).

Deterioration of the boundary elements is depicted in **Figure 16**. Weathering has definitely played a strong role in the state of deterioration of the boundary elements, but it is also judged that this process was accelerated by the lack of transverse reinforcement in combination with highly concentrated loads over the boundary elements. Because no transverse reinforcement was provided in the boundary elements there is a lack of resistance to: (i) prevent buckling of the longitudinal reinforcement once the cover concrete begins to spall *(6)*, and (ii) provide shear transfer of the superstructure service loads to the foundation level *(6)*.

In order to prevent buckling of the longitudinal reinforcement and increase the shear capacity of the boundary elements, which would lead to sudden failure of the bridge piers, it was proposed to retrofit the main span bridge piers by applying external prestressing over GFRP wrapping. Because of the poor quality of concrete in the diaphragm regions, the main advantage of applying first GFRP wrapping directly onto the concrete surface was to cause no further damage to the cover concrete, as the strands are post–tensioned. In addition, by applying the GFRP wrapping will reduce the prestressing curvature friction coefficient; thus, increasing the effectiveness of the strands and reduce the number of required strands.

This retrofit technique, in addition to providing active confinement, will also reduce cracks width and inhibit further deterioration of the boundary elements. Design considerations developed under this proposal will lead to a retrofit technique that is simple to implement while ensuring the best possible retrofit technique for this particular bridge.

Design of the External Circumferential Prestressing

Retrofitting of the main span bridge piers was accomplished by using a combination of external circumferential prestressing and GFRP wrapping. The GRFP wrapping was applied first over the concrete surface, and than external prestressing in the form of hoop mono-strands were tensioned over the GFRP wrapping.

General Description of the Bridge Piers

In the main span piers, the circular boundary elements at the deck level are 1.45m in diameter and increase to 2.21m at the foundation level (see **Figure 17**). The connecting shear wall panels terminate 3.05m below the 0.46m thick pier cap.

Reinforcement of these boundary elements consists of 8-#8 (D25) longitudinal bars for an average longitudinal reinforcement ratio of approximately 0.25%. No transverse reinforcement is shown on the existing plans and preliminary investigation of the bridge piers showed also the lack of transverse reinforcement. The connecting shear wall panels are 0.38m with #8 (D25) horizontal bars at 0.61m on centers, and #4 (D13) vertical bars at 0.61m on centers, (see **Figure 18**).

Design Criteria

Design of the external circumferential prestressing was accomplished by assuming that a compression strut $\boldsymbol{C}$ develops at an angle of approximately 30^{o}, as shown in **Figure 19**. Based on this angle and referring to **Figure 19** the following expression may be used to estimate the tension force $\boldsymbol{T}$ required to sustain the formation of the compression strut $\boldsymbol{C}$:

$$\boldsymbol{T = W_{D+L}\ tan30^{o}} \tag{4}$$

where $\boldsymbol{W_{D+L}}$ = 4,735kN is the superstructure service loads, which corresponds to an axial load ratio of 10% assuming a concrete compression strength of 35MPa. Live loads were obtained from AASHTO design specifications for a truck HS20-44 *(7)*. Service loads are shown in Error! Reference source not found.. This tension force $\boldsymbol{T}$ will be resisted by prestressing strands placed externally around the boundary elements. The total allowable tension force that can be developed by the prestressing strands is computed as:

$$\boldsymbol{T = n_p\ A_{ps}\ f_{ps}} \tag{5}$$

where $\boldsymbol{n_p}$ is the total number of strands crossing strut $\boldsymbol{C}$ and $\boldsymbol{f_{ps}}$ is the prestressing stress per strand after losses. To compute n_p the following expression can be used:

$$\boldsymbol{n_p = \frac{D}{s\ \tan 30^{o}}} \tag{6}$$

where $\boldsymbol{D}$ **=** 1.45m is assumed the diameter of the boundary elements at the pier cap level, and $\boldsymbol{s}$ is the strand hoop spacing. The prestressing stress at jacking was computed not to exceed $\boldsymbol{0.80f_{pu}}$. Since the stressing operation is self-reacting, friction losses were considered for only half-circle assuming a curvature friction coefficient of 0.05 (i.e. 85% losses – strands will be tensioned over the GFRP wrapping), and assuming other losses not to exceed 10% the maximum design stress was $\boldsymbol{0.60f_{pu}}$ = 1,120kN, considering an ultimate tensile strength $\boldsymbol{f_{pu}}$ = 1,860kN. Thus, the total allowable tension force is:

$$\boldsymbol{T = \frac{D}{s\ \tan 30^o}\ A_{ps}\ 0.53 f_{pu}} \quad \textbf{(7)}$$

Combining **EQ's (1)** and **(4)** one obtains the expression:

$$\boldsymbol{\frac{A_{ps}}{s} = \frac{W_{D+L}}{2\ D\ 0.60 f_{pu}} (\tan 30^o)^2} \quad \textbf{(8)}$$

For ½" diameter strands ($\boldsymbol{A_{ps}}$ = 99mm^2) the required spacing is 203mm, computed from:

$$\boldsymbol{s = \frac{99\ x\ 2\ x\ 1450\ x\ 0.60\ x\ 1{,}860}{4{,}735\ x\ (0.58)^2} = 203mm} \quad \textbf{(9)}$$

Construction Guidelines

Construction guidelines for the retrofit technique described under this section are specified in this section.

Surface Preparation

The first steps in the construction process involved patching of the boundary elements concrete surface to a smooth finish *(8, 9)*, followed by drilling of 16mm diameter holes along the shear wall panels for installation of the fiber wrapping and strands. In order to avoid damage to the shear wall panel reinforcement a 76mm strip of cover concrete along these panels on one face only were removed to expose the reinforcement. Location of the 16mm diameter holes is shown in **Figure 20**.

GFRP wrapping

After preparing the surface the boundary elements were wrapped with GFRP *(8, 9)*, according to the specifications described in example one.

External Prestressing Operation

After wrapping was accomplished, the strands were stressed to approximately $\boldsymbol{0.80f_{pu}}$ *(10)*. This operation was conducted under the direct supervision of the UMR and SPS design teams.

CONCLUSIONS

Reseachers worldwide have shown that strengthening of civil structures using externally bonded FRP sheets is applicable to many types of RC structures. Applications of FRP to strengthening of columns, beams, slabs, walls, chimneys, tunnels, and silos have been successfully achieved. The uses of external FRP reinforcement have been successfully used in flexural strengthening, improving the confinement and ductility of compression members, and shear strengthening. These studies led to the successful retrofit techniques developed for these two case studies.

Steel reinforced concrete has been widely used for a variety of civil engineering application, and throughout the world, steel is the most commonly used construction material. Traditional techniques used to repair precast concrete structures are expensive and time consuming. Structural retrofit work has come to the forefront of industry practice in response to the problem of aging of infrastructures worldwide. Repairs and maintenance of bridge structures, such as those discussed in this paper, have demonstrated the positive benefits of using FRP technology for damage repair and/or maintenance.

Although, the techniques described in this paper are efficient options for the repair/retrofit of bridge structures, successful implementation of these techniques will depend on the engineer's material and structural knowledge. For example, considerations regarding delamination of FRP sheets that may significantly affects the strength of the member in the controlling zones have to be taken. In addition, existing RC beams need to be additionally strengthen to insure that the shear strength equals or exceeds its flexural strength at all points along the beam span, since deficiencies in the structure to resist shear forces are proven to lead to catastrophic failures. Deficiencies in the shear capacity of the existing RC beam may occur due to a variety of factors such as; reduction in or total loss of shear reinforcement due to corrosion, and changing the function of a structure from a lower to a higher service load. In these situations, it has been shown that externally bonded FRP sheets may be used to increase the shear capacity of RC members. However, few studies have specifically addressed shear strengthening, and design algorithms for computing the shear contribution of FRP sheets are not yet clear because of the complicated problems of bond, the behavior of crack propagation, and the wide variety of possible FRP shear reinforcement configurations.

These case studies demonstrated that FRP bonded reinforcement can be an effective repair technique, and its applications can be further expanded in combination with other retrofit techniques. A sharp increase in FRP application is forecasted, if the present trend in growing availability of material and design information is to continue.

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Pedro Franco Silva, Pei-Chang Huang and Antonio Nanni " Practical Issues Related to the Structural Preservation of Existing Bridge Structures".

LIST OF TABLES

Table 1. Material Properties

Table 2. Design Service Loads per Boundary Element

Table 3. Properties of High Tensile Carbon

LIST OF FIGURES

Figure 1. Fractured Tendons after the Removal of the Damaged Concrete

Figure 2. Girder Dimensions and Prestressing Details

Figure 3. Longitudinal Installation of Two Plys CFRP Laminates

Figure 4. CFRP Strips U-Wrapped to the Sides of the Member to be Repaired

Figure 5. Surface Preparation – Rounding of Corners

Figure 6. Surface Preparation – Sandblasting of Surface Concrete

Figure 7. Application of Primer

Figure 8. Application of Putty

Figure 9. Application of first layer of Saturant

Figure 10. Manual lay-up of CFRP sheets

Figure 11. Rolling of CFRP laminate to eliminate entrapped air

Figure 12. Application of second layer of Saturant after placing the CFRP sheet

Figure 13. Strips lay-up

Figure 14. Bridge Longitudinal Profile

Figure 15. Main Span Bridge Pier

Figure 16. Deterioration of Circular Boundary Elements

Figure 17. Partial Side View - Main Span Bridge Pier

Figure 18. Boundary Elements X-Sections

Figure 19. Design Criteria – Strut-and-Tie Model

Figure 20. Retrofit Criteria

Table 1.

Prestressing Tendons	Strand Type	Low Relaxation
	Strand Tensile Strength (MPa)	1,862
	Nominal Diameter (mm)	12.7
	Strand Area (mm^2)	98.71
	Modulus of Elasticity, E_{ps} (GPa)	19.31
Concrete	Existing Concrete Deck, f_c' (MPa)	34.5

Table 2.

Ultimate Strength (MPa), f_{pu}	4,275
Design Strength (MPa), f_{fe}	3,792
Tensile Modulus (GPa), E_f	22.8
Thickness (mm), t_f	0.165
Ultimate Strain (mm/mm), ε_{fu}	0.0167

Table 3.

Loading Category		Tributary Support Reaction (kN)
Dead Load	Tributary 197mm x 7.62m Concrete Deck	1,670
	Tributary 61m + 30.5m Steel Trusses	780
	1.70kN/m^2 for Future Asphalt Resurfacing	670
Live Load	Maximum Live Load Value [1]	1,320 [2]
	Impact Loading	295

[1] Values shown are for two lanes of traffic.
[2] Standard Lane Loading Governs Design.

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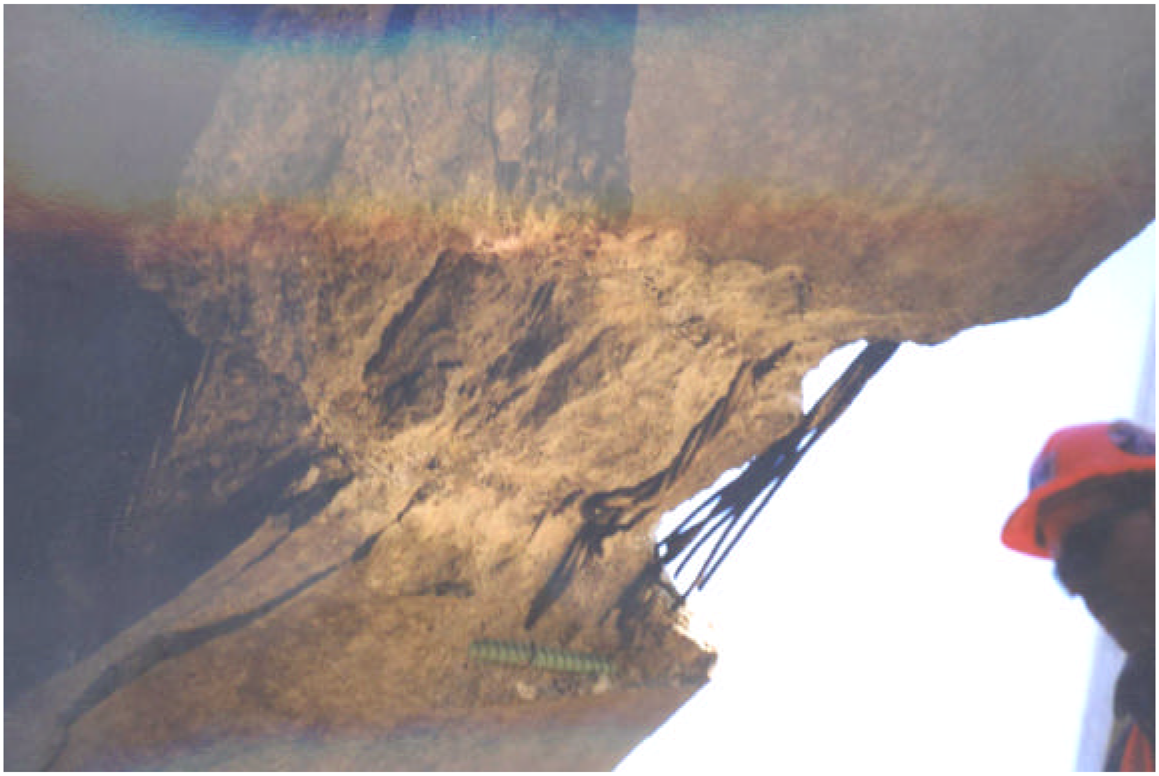

Figure 1.

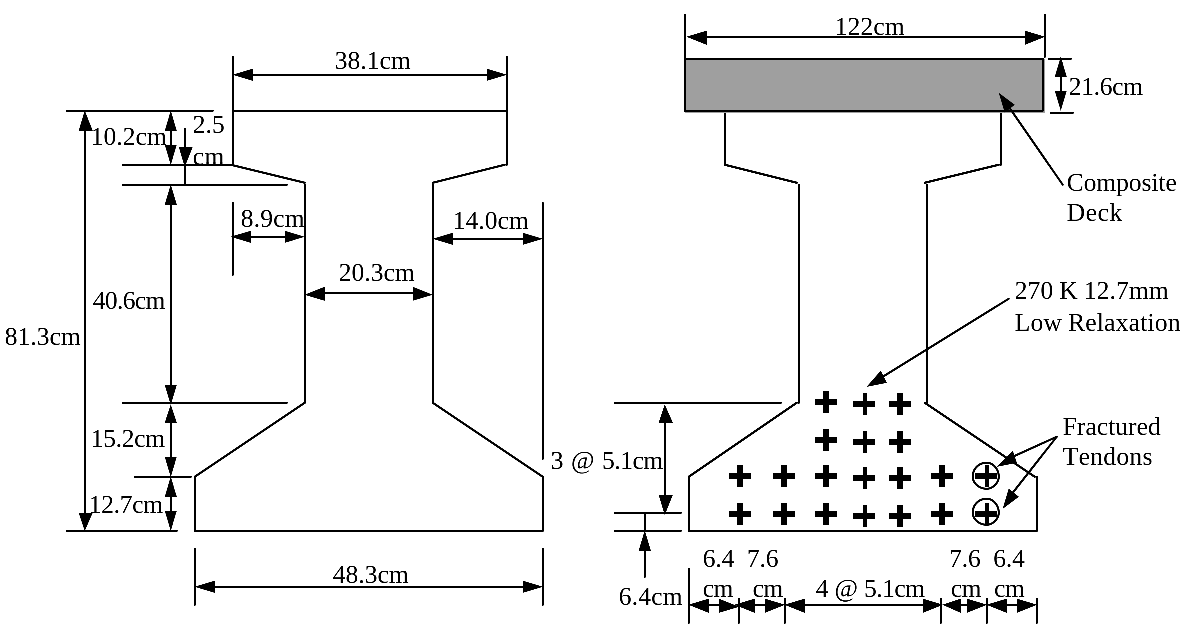

Figure 2.

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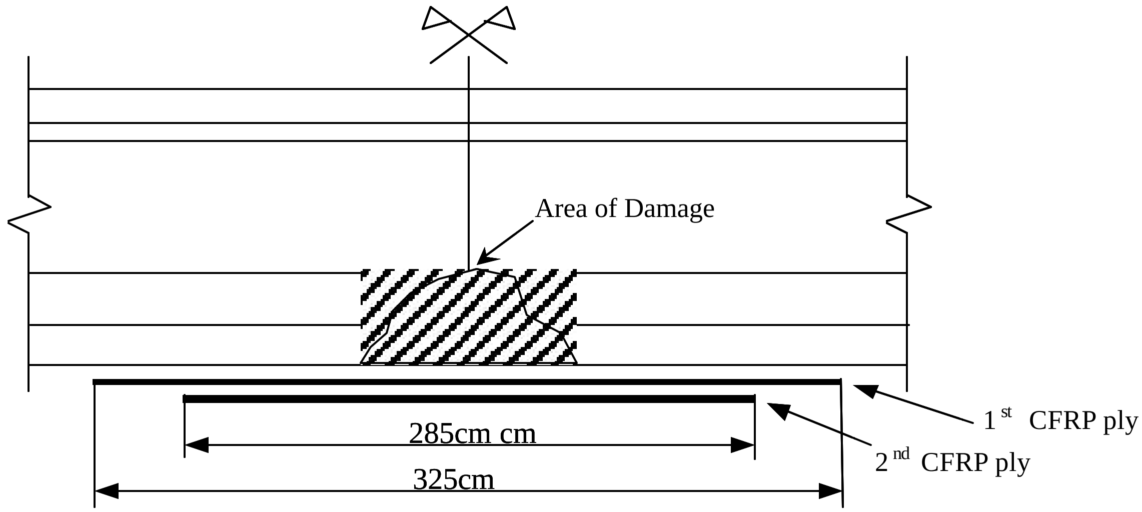
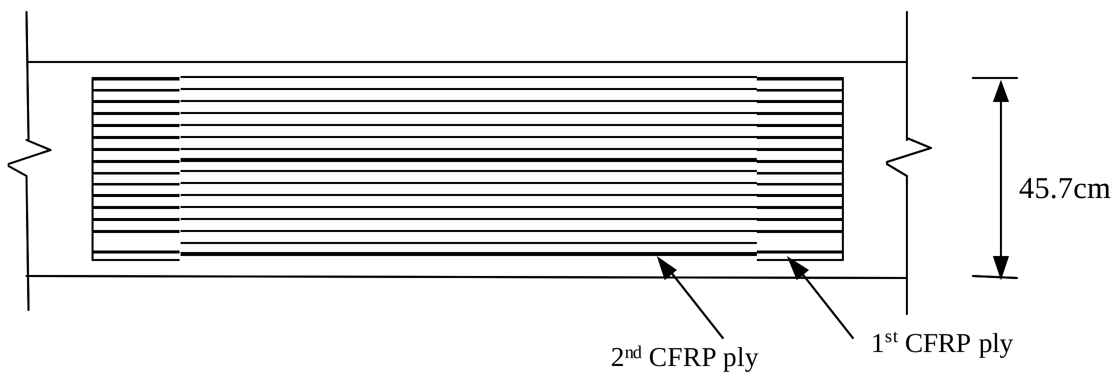
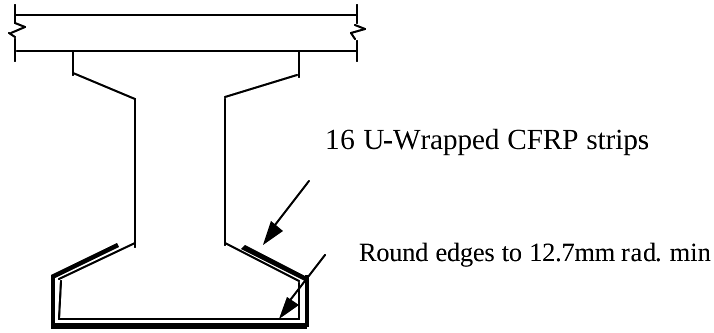


a) Side View

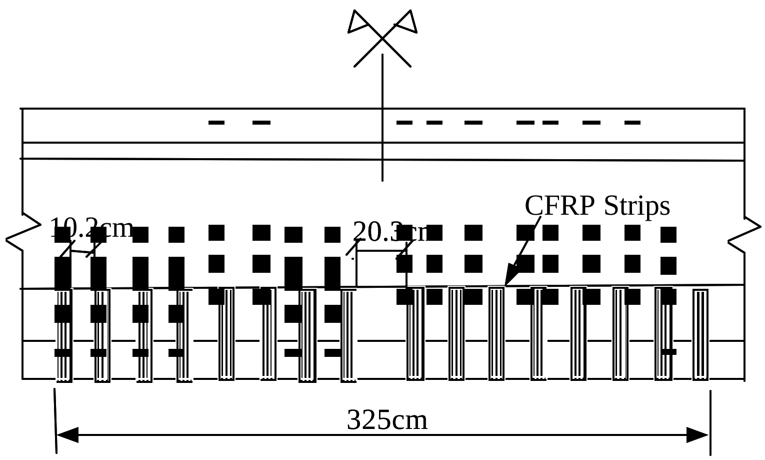


b) Bottom View

Figure 3.



a) Cross Section



b) Side View

Figure 4.

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Figure 5.

Figure 6.

Pedro Franco Silva, Pei-Chang Huang and Antonio Nanni " Practical Issues Related to the Structural Preservation of Existing Bridge Structures".

Figure 7.

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Pedro Franco Silva, Pei-Chang Huang and Antonio Nanni " Practical Issues Related to the Structural Preservation of Existing Bridge Structures".

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Figure 12.

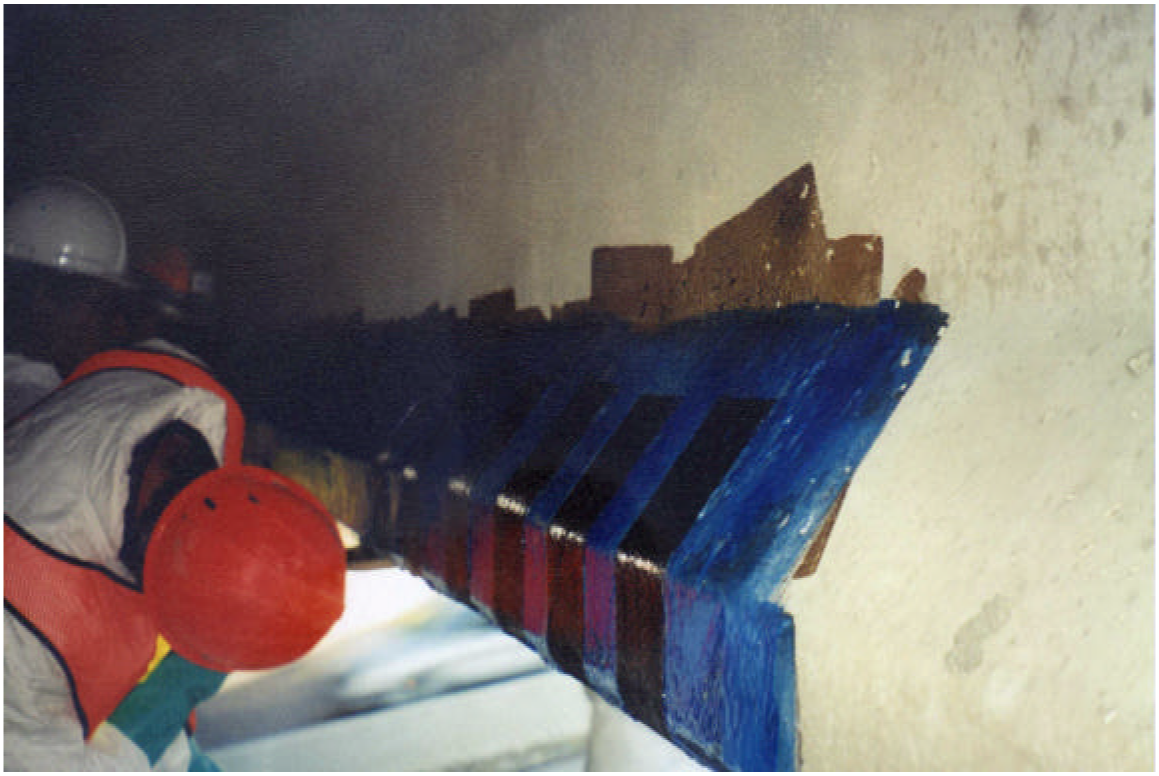

Figure 13.

Figure 14.

Pedro Franco Silva, Pei-Chang Huang and Antonio Nanni " Practical Issues Related to the Structural Preservation of Existing Bridge Structures".

Figure 15.

Figure 16.

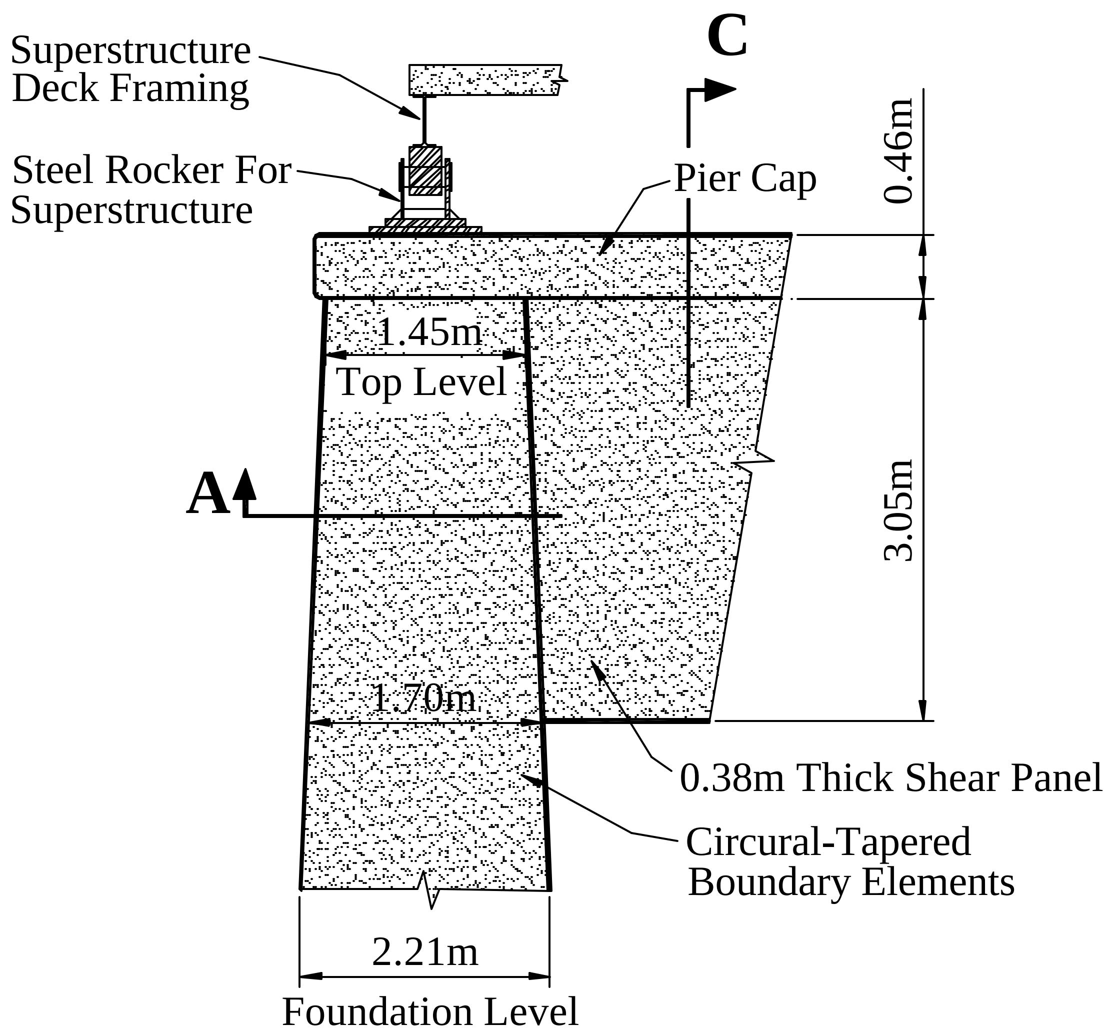
Superstructure
Deck Framing
Steel Rocker For
Superstructure
C
Pier Cap
0.46m
1.45m
Top Level
A
3.05m
1.70m
0.38m Thick Shear Panel
Circural-Tapered
Boundary Elements
2.21m
Foundation Level

Figure 17.

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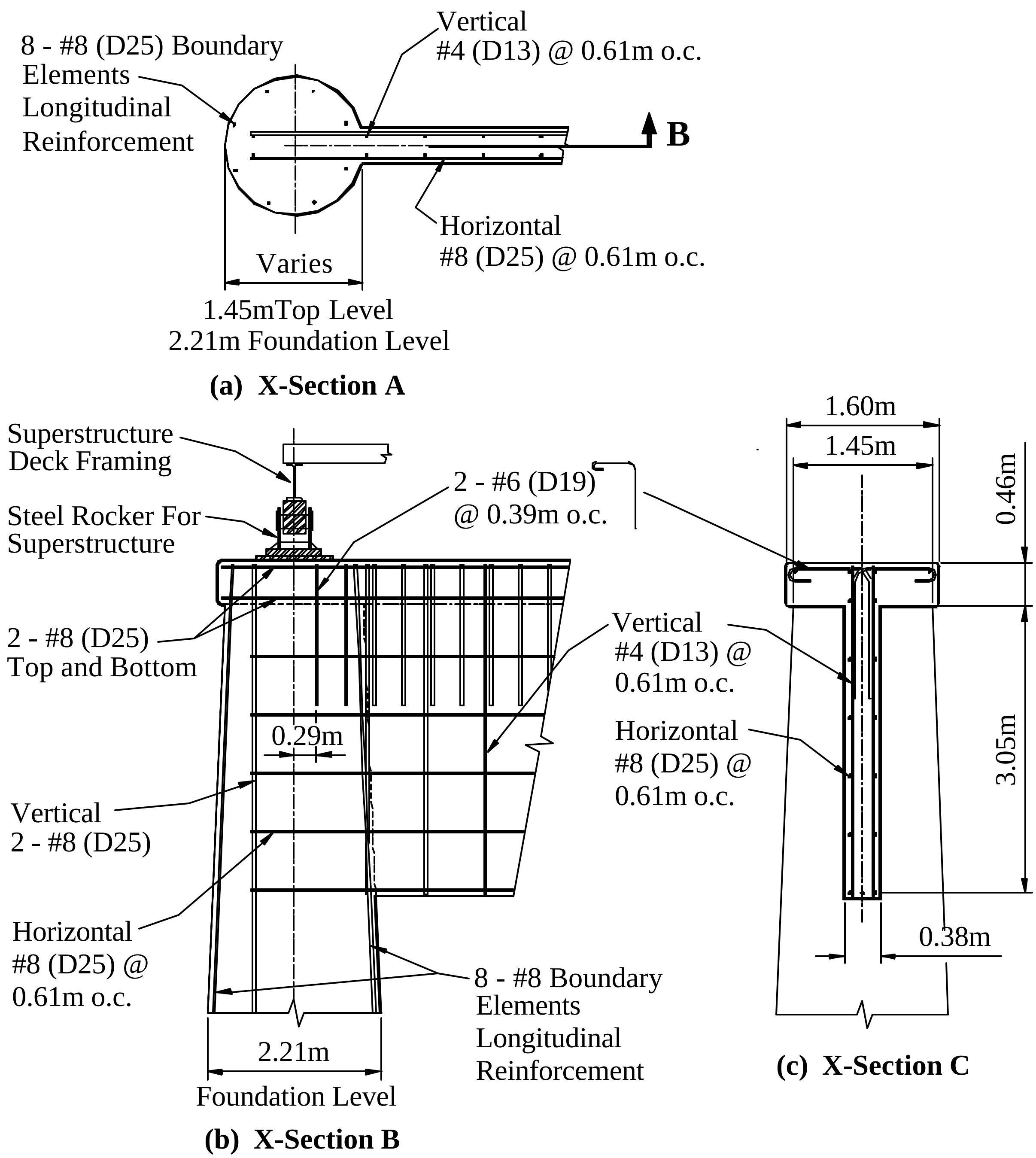

Figure 18.

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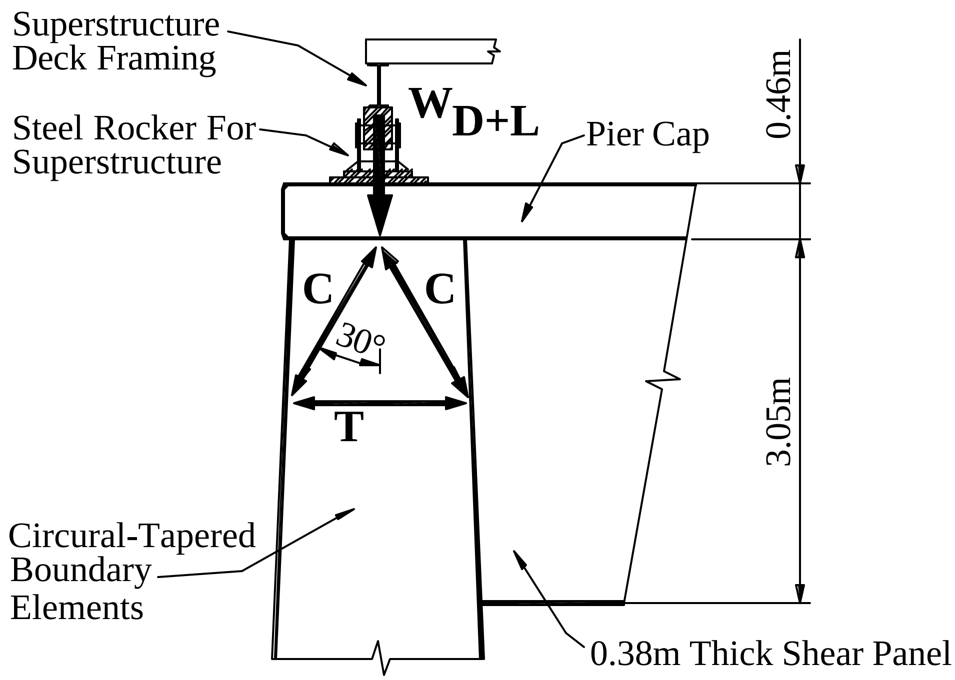

Figure 19.

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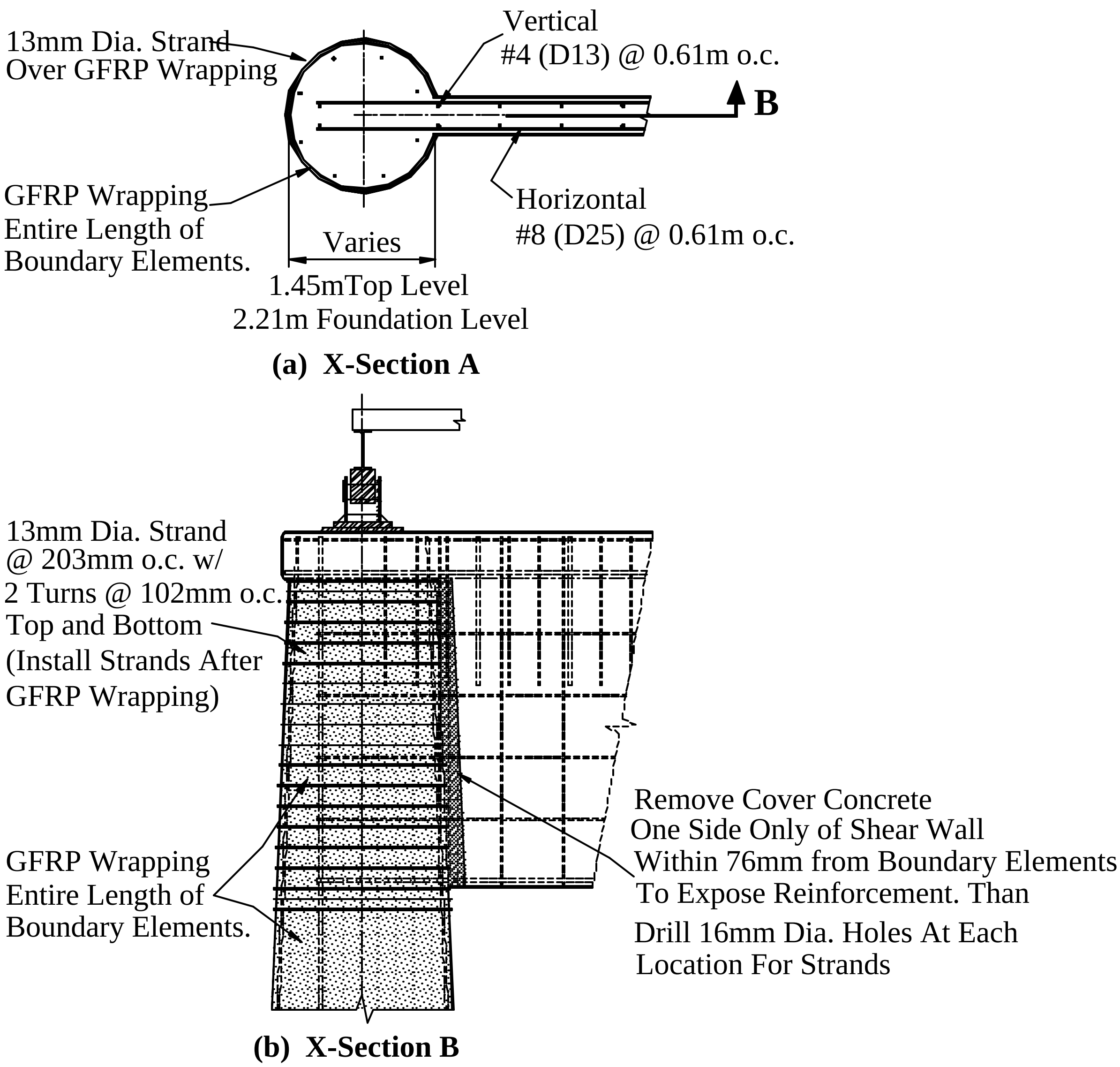

Figure 20.