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ASSESSMENT OF REHABILITATION METHODOLOGIES FOR DAMAGED CONCRETE BRIDGE PIER

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Abstract: A nonlinear finite element numerical study was conducted to simulate the current damage in a bridge pier and to establish the most effective rehabilitation strategy by means of Fiber Reinforced Polymer (FRP) sheets. The bridge pier model predicted splitting cracks and dilation due to the axial load imposed by the superstructure, in addition to yielding of the shear reinforcement. The simulated response was in close agreement with that observed. Thereafter, the model in its damaged state was subjected to a pushover analysis to simulate the effects of an earthquake. A substantial reduction of both lateral load and displacement capacities was predicted. The widening of the longitudinal cracks and yielding of the shear reinforcement prevented the pier from behaving as a single structural element, consequently, the stiffness and ductility of the pier were reduced in comparison to the design values. The rehabilitation strategy consisted of applying grout injection to seal cracking; followed by the application of a single layer of FRP jacketing. The rehabilitated model was subjected to lateral loading to determine improvements in seismic response. The analysis demonstrated an enhanced behavior, including an increase in the deformation capacity. The predicted brittle failure of the damaged pier was suppressed and the lateral load resistance was restored to its original capacity.

1 INTRODUCTION

Developing countries are faced with a lack of resources to properly monitor their built infrastructure, jeopardizing the preservation of public structures. For this reason, many existing structures are not in their optimal operating condition. One such example is the Tárcoles River Bridge in Costa Rica that was not properly maintained during its 50 years of service. Moreover, the structural design of this bridge was based on old standards and construction practices, and based on current design codes, the structure does not have the capacity to sustain expected loads. This has resulted in structural damage of one of the piers of the bridge. Therefore, rehabilitation is now necessary.

The research presented herein is based on nonlinear finite element modelling to simulate the current damage in the bridge pier followed by establishing the most effective rehabilitation strategy by means of Fiber Reinforced Polymer (FRP) sheets.

2. BRIDGE PIER DETAILS

The bridge pier was constructed with concrete with a compressive strength of 34 MPa and consists of an oval cross-section. The nominal dimensions (Figure 1a) are: 8.85 m high, 5.20 m wide, and a variable thickness due to the subdivisions of the pier along its height. From the base up to 7.85 m in height, the pier has a hollow cross-section (Figure 1c), while the upper portion is composed of a solid section (Figure 1d).

This subdivision was included to distribute the axial compressive forces imposed on the pier from the superstructure. The original design incorporated a shear key at the top of the pier to transmit the inertia forces from the superstructure during a seismic event, along with 4 neoprene pads distributed on the surface of the pier (Figure 1b), whose function is to transmit the gravity loads from the superstructure.

The pier contains typical reinforcing steel and post-tensioning steel. In total, twelve post-tensioning mechanisms were used in the construction of the pier. The presence of post-tensioning within the top 3 m of the pier (Figure 1a) controls the behavior of the pier when subjected to seismic loading. The post-tensioning is intended to reduce the amount of conventional steel reinforcement and optimize the dimensions of the pier. The type of each post-tensioning mechanism is referred to as T13 (13 strands of 38 mm diameter set inside a 140 mm-diameter sleeve). The yield strength of the tendons is 1570 MPa, and the yield strength of the deformed steel is 420 MPa.

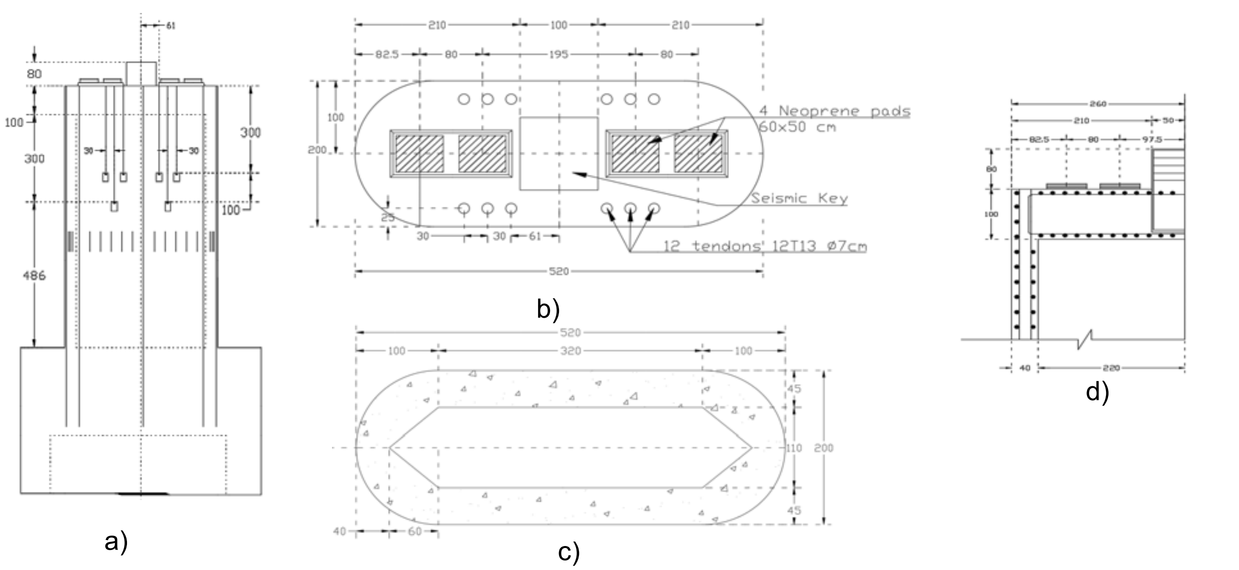


Figure 1. Typical pier details: a) Elevation view, b) Top view, c) Hollow section, d) Solid Section (Note: dimensions in cm) (MOPT, 2013)

It has been postulated that the damage experienced in the bridge pier is that result of failed neoprene pads, such that the gravity load of the superstructure is solely being transmitted into the pier through the single shear key.

3. RESEARCH METHODOLOGY

The nonlinear finite element software VecTor2 was used to develop two models to represent the original design and actual conditions of the pier, with the objective of understating their behavior when subjected only to the axial load imposed by the superstructure and to simulate the damage that currently exists (severe vertical cracking along the height) prior to the rehabilitation strategy. The construction of both models is similar including: the dimensions of the elements, the discretization used to generate the mesh that simulates the concrete with "smeared" reinforcement and the discrete truss elements to represent the post-tensioning steel. The main difference lies in the transmission of gravity loads between the superstructure and the pier. In the design condition model, the pier receives the load through the 4 neoprene pads. Conversely, in the actual condition model, the damage that the pier has experienced was generated from the assumption that the gravity load is transmitted directly through the shear key as a concentrated axial load.

A third model was generated, similar to the two previous models, with the difference that the model retained the damage in the pier prior to the rehabilitation incorporating grout injection and FRP jacketing. In addition to this, a bond-slip model based on the energy fracture method was implemented to simulate the contact behavior between the FRP and the concrete of the bridge pier. Figure 2 provides schematics of the various states of the bridge pier.

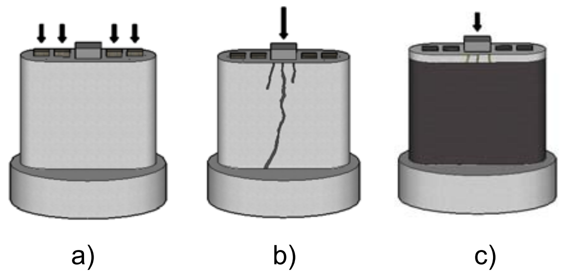


Figure 2. States of the structure: a) Design condition, b) Actual condition, c) Rehabilitated condition

The models in their three states: design condition, actual condition, and rehabilitated condition, were subjected to lateral loads to simulate the effects of seismic activity to understand the effectiveness of the rehabilitation. In the analyses, the lateral load was imposed on the shear key and the application of the axial load varied according to Figure 2.

4. FINITE ELEMENT BASES

4.1 Conceptual bases

VecTor2, a two-dimensional nonlinear finite element program was used for the numerical analyses. The program uses the Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986) and the Disturbed Stress Field Model (DSFM) (Vecchio, 2000). The program is based on a smeared, rotating crack model for reinforced concrete, in which cracked concrete is represented as an orthotropic material.

4.2 Material Constitutive Models and Finite Elements

VecTor2 contains a library of constitutive models for smeared reinforced concrete, discrete reinforcement and bond materials. The default constitutive models were used in this investigation; these are appropriate to represent second-order mechanisms in concrete behavior, including compression softening, tension stiffening and softening, and slip on the shear crack surface, to name a few.

A typical ductile response was utilized for steel reinforcement that includes a linear elastic region, a yield region and a strain hardening zone, while the FRP reinforcement is assumed to be linear elastic with brittle fracture in tension. The details of the constitutive models are available elsewhere (Wong and Vecchio, 2002). To simulate the effect of bond between external FRP sheets and concrete, link elements connecting the FRP to the concrete were used. The bond-slip behavior follows a bilinear response. The parameters in the model include the fracture energy of the concrete (G_f =0.40 MPa), the concrete cylinder compressive strength (f'_c=34 MPa), maximum bond stress (U_{max}=4.17 MPa) and corresponding slip (S_{max}=0.04mm) defining the linear ascending elastic response. The linear descending post-peak region is defined by the slip corresponding to zero bond stress (S_{ult}=0.19 mm). The model and the calculation of these values were based on the fracture energy method (Sato and Vecchio 2003)

5. FINITE ELEMENT MODELLING AND ANALYSIS METHODOLOGY

5.1 Modelling of Original Pier

A total of 1500, 4-noded, plane stress rectangular elements were used to model the bridge pier, divided between the hollow section, solid section and seismic key, as shown in Figure 3. The elements were generated with dimensions of 200 mm in the horizontal direction and 195 mm in the vertical direction, maintaining an optimal aspect ratio of 3:2.

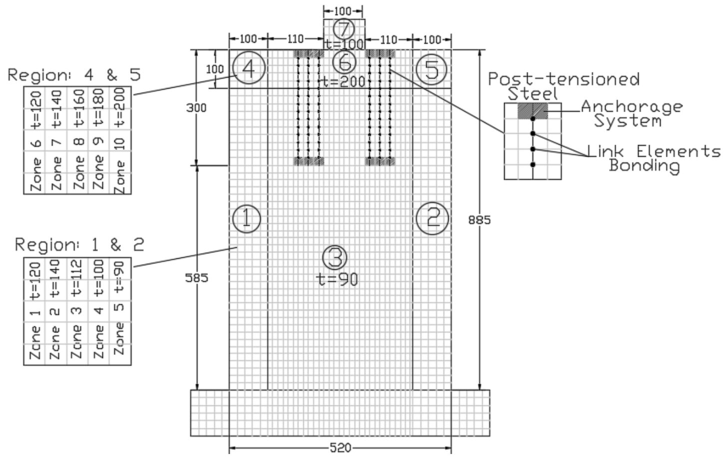


Figure 3. Finite element model of the pre-rehabilitated pier

The typical reinforcing steel was modeled as smeared within the concrete rectangular elements. The deformed steel was considered perfectly bonded to the concrete. A total of 13 different materials types (simulating reinforced concrete) were used in the model in its pre-rehabilitated condition, each with different thicknesses, and reinforcement ratios (Table 1).

Table 1. Thickness and reinforcement ratio for each zone of the FE model

Region	Zone	Thickness (cm)	Reinforcement Ratio (%)		
			Vertical	Horizontal	Out of plane
1 & 2	1	120	2.37	0.08	0.26
	2	140	2.37	0.08	0.26
	3	112	2.37	0.08	0.26
	4	100	2.37	0.08	0.26
	5	90	2.37	0.08	0.26
3	-	90	1.1	0.11	0.07
4 & 5	6	120	0.141	0.16	0.2
	7	140	0.141	0.16	0.2
	8	160	0.141	0.16	0.2
	9	180	0.141	0.16	0.2
	10	200	0.141	0.16	0.2
6	-	200	0.141	0.16	0.2
7	-	100	15	15	15

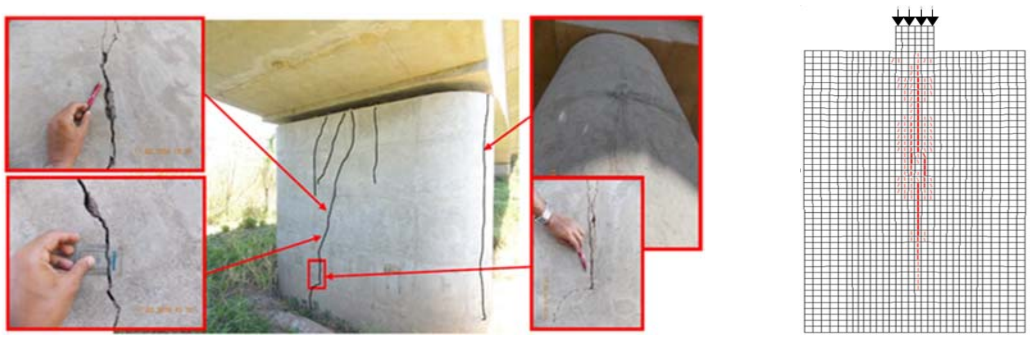
The post-tensioned tendons were represented by two-node truss bar elements with uniform cross-section, as shown in Figure 3. A total of 90 truss-bar elements were used in this model. The tendons are assumed to be satisfactorily simulated by smooth bars since they are placed inside sleeves fully debonded from the concrete and only joined to the concrete at the ends. The anchorage region of the post-tensioning was assigned a concrete material with high strength and stiffness properties. This strategy prevented localized crushing of the concrete elements at the end regions of the tendons.

The tendons are assumed perfectly bonded at the ends, while link elements were used between the ends to decouple the tendons from the concrete. The pre-stress assigned to the tendons incorporated a reduction of 20% for losses encountered during installation. The final pre-stress force applied to each tendon was approximately 200 kN. The base of the pier was assumed fixed.

5.2 Simulation of Pier Damage

In its design condition, the pier was subjected to a total of 17 230 kN of axial load (live and dead load) from the superstructure through four neoprene pads, which were responsible for distributing the load over the pier. However, due mainly to the lack of maintenance, the pads failed, altering the transmission of axial load through the seismic key resulting in a concentrated axial load at the center of the pier.

To simulate the altered load path, the total axial load was imposed on the shear key as shown in Figure 4. The figure provides a comparison between the actual condition of the pier and that predicted by the model. The model satisfactorily captured the actual behavior. The maximum crack opening reported in the inspection report (LANAMME 2014) was 7 mm, while VecTor2 calculated a maximum crack width of 6.5 mm. As a result of the cracking, the shear reinforcement yielded.



a) b)

Figure 4. Cracking patterns: a) Real structure (eBridge, 2016), b) Model structure

5.3 Pushover Analyses of Pre-Rehabilitated Model

One of the main objectives of this study was to investigate the behavior of the structure in its actual condition when subjected to lateral load (seismic load) to assess the capacity of the bridge if it remains in its damaged state. For this, pushover analyses were conducted on three models of the pier.

The design condition model was subjected to a lateral load at the shear key with the gravity loads transmitted equally across the neoprene pads. In the actual condition model, gravity loads were first applied on the shear key until damage was present (Figure 4b) and thereafter, the pier was subjected to a lateral load at the shear key. A complementary analysis was performed where the bridge pier was subjected only to lateral load. Figure 5 illustrates the predicted lateral load-displacement response for each model, and Figure 6 provides a schematic of the cracking patterns.

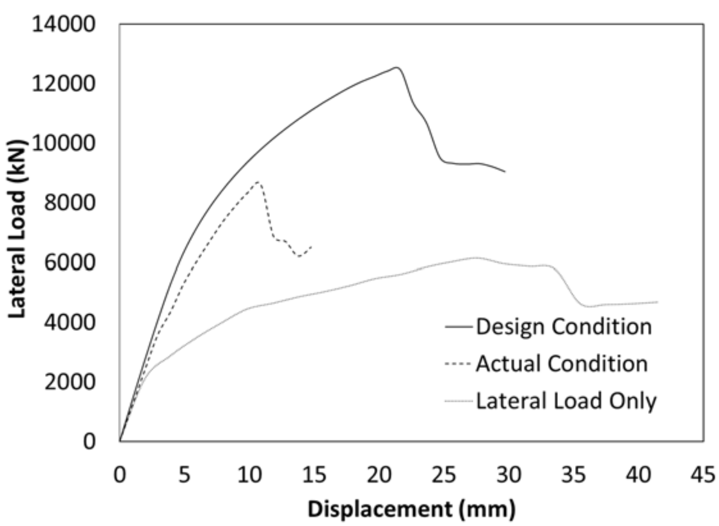
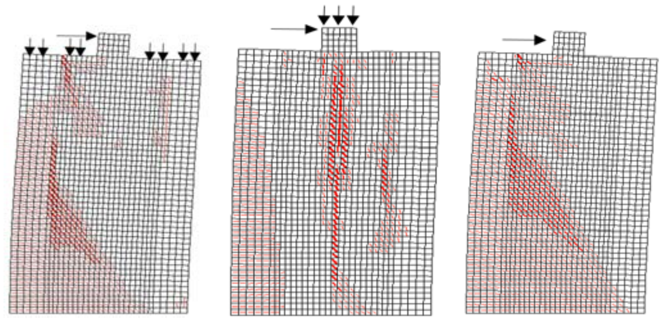


Figure 5. Bridge pier capacity curves.



a) b) c)

Figure 6. Cracking pattern and deformation at the peak load for: a) Design condition, b) Actual condition, c) Lateral load only.

Figures 5 and 6c provide the numerical analysis results on the pier when subjected only to lateral loading. In comparison to the design condition of the pier (Figure 5 and Figure 6a), this model provides an improvement in lateral deformation capacity, but a reduction in lateral load capacity. The axial load that imposed by the superstructure causes a significant increase in lateral load capacity with a notable decrease in displacement capacity.

Figures 5 illustrates the difference in behavior of the structure between the design and actual conditions. A substantial reduction of both the lateral load and displacement capacities arises due to the damage imposed by the concentrated axial load. The initial cracking and yielding of the shear reinforcement (Figure 4b) causes the pier to behave as two separate structural elements, causing a decrease in stiffness, ductility, and lateral load capacity. The peak lateral load in the design condition of the pier was 12 470 kN while for the actual condition it is 8 620 kN, thus the initial damage reduces the lateral load capacity approximately by 30%. The displacement corresponding to the peak lateral load of the design condition model was 21.6 mm, while the actual condition model experienced only 10.9 mm of lateral displacement, a reduction in displacement capacity of approximately 50%.

Figure 6a illustrates the cracking and deformation pattern at the peak lateral load for the pier based on its design condition. The proper transmission of the axial load to the pier promoted wider spread cracking in comparison to the actual condition (Figure 7b).

Given the inferior performance of the bridge pier due to the damage imposed by the gravity loading, it has become necessary to investigate rehabilitation strategies with the objective of returning the bridge pier to its design lateral load and displacement capacities.

5.4 Modelling of Rehabilitated Pier

VecTor2 has the capability to simulate the chronology of construction and loading through engaging and disengaging of the elements. The engaged elements represent portions of the structure that are present, contributing to strength and stiffness. Conversely, disengaged elements represent sections of the structure that do not contribute to strength and stiffness but experience the same deformations as the elements that are engaged and occupying the same position in the model. The disengaged elements are typically used to represent a repair or retrofit material that will be implemented after the structure has experienced loading and deformation. The disengaged elements are activated to simulate the instant that they are intended to contribute to the strength and stiffness. The previously engage elements occupying the same space are deactivated.

The rehabilitated model considered the application of grout injection to seal the cracks and then jacketing with FRP sheets. Note that the rehabilitation methodology was applied to the model of the pier that reflects its actual condition (cracking and yielding of the shear reinforcement).

To simulate the grout injection in VecTor2, the elements that experienced significant cracking due to the applied gravity loads in the actual condition model (Figure 6 b) were replaced by a concrete material with similar properties to those of the design condition of the pier. Figure 7 shows the finite element model of the rehabilitated pier. The vertical shaded strip represents the portions of the pier to which new properties were assigned, representing the cracked concrete that was repaired.

Truss bar elements were used to model the FRP sheets as shown in Figure 7. A total of 1400 elements were assigned to simulate this material. To capture the bond between the concrete and the FRP, a bilinear bond-slip response was used to simulate the elastic behavior of the FRP and the interaction between these two materials, capturing the brittle failure due to the debonding of the FRP. Link elements were introduced between the FRP truss elements and the concrete elements to determine the slip between these two materials.

The FRP utilized in the model has a laminate thickness of 1.0 mm with an ultimate tensile strength in primary fiber direction of 834 MPa, an elongation of 1.0% and a tensile modulus of 82 GPa. The grout injection employed has a compressive strength of 34 MPa.

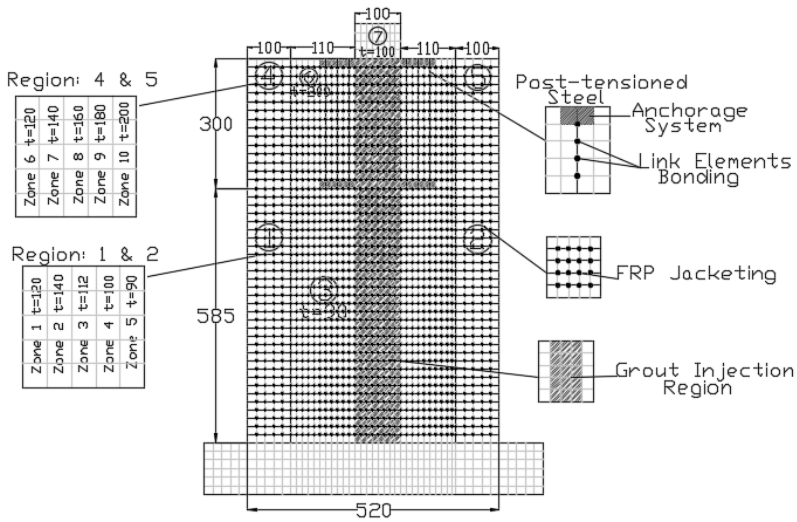


Figure 7**.** Finite element model of the rehabilitated pier

5.5 Pushover Analyses of Rehabilitated Model

The rehabilitated model was subjected to a series of pushover analyses to observe the behavior of the methodology and compare it with the design and actual conditions of the pier. Note that the replacement of the neoprene pads was not considered in the strategies investigated. The objective herein was to determine a strategy that was quick and non-invasive. Therefore, the analyses considered the axial load from the superstructure to be transferred as a concentrated load on the shear key.

Figure 8 provides the lateral load-displacement responses of the strategies, including retrofitting with grout injection only, jacketing with FRP assuming perfect bonding, jacketing with 1 and 2 layers with bond consideration, and grout injection combined with jacketing with 1 FRP layer.

Figure 9 provides the predicted cracking patterns and deformations of the pier at the peak lateral load capacity for the various rehabilitation strategies. Table 2 provides a summary of the key behavioural parameters predicted by the numerical models.

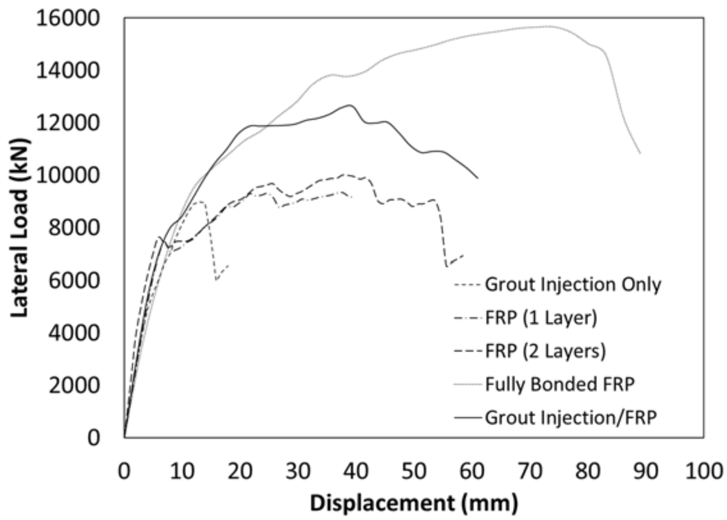


Figure 8. Rehabilitated pier capacity curves

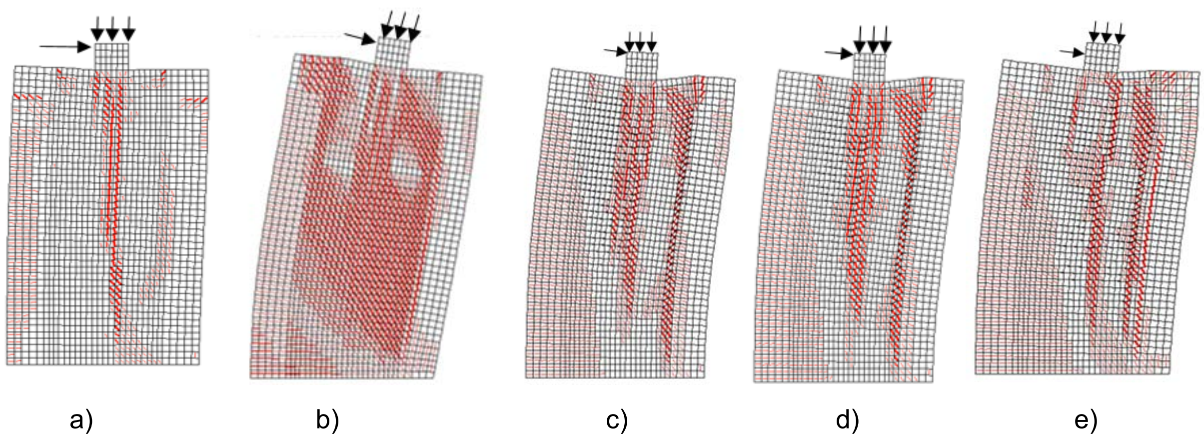


Figure 9. Cracking pattern and deformation at the peak load for rehabilitated pier with: a) Grout injection, b) Fully bonded FRP, c) FRP (1 layer), d) FRP (2 layers), e) FRP with Grout injection

Table 2. Summary of Behavioral Responses

Model	Global Yielding		Peak		Ultimate		Ductility Capacity
	Displacement (mm)	Load (kN)	Displacement (mm)	Load (kN)	Displacement (mm)	Load (kN)	
Design Condition	13.1	10620	21.6	12470	24.3	9980	1.9
Actual Condition	8.8	7870	10.9	8620	11.8	6890	1.4
Lateral Load Only	17.4	5200	27.6	6150	35.0	4920	2.0
Grout Injection Only	10.0	8300	12.9	8970	15.0	7180	1.5
Fully Bonded FRP	32.0	13340	74.1	15650	85.7	12520	2.7
FRP (1 Layer)	11.3	7500	37.7	9330	45.0	7460	4.0
FRP (2 Layers)	15.3	8120	38.7	10010	55.0	8000	3.6
Grout Injection / FRP	17.0	10820	39.0	12620	60.0	10100	3.5

The first rehabilitation strategy considered only grout injection to seal the cracks present in the pier. Figures 8 and 9a provide the analysis results, which predicted a very similar behavior to the actual condition model as the transmission of the axial load did not change. Sealing the cracks did not restore the pier to its design condition, as the shear reinforcement that had yielded was not addressed in this strategy.

The second strategy involved only jacketing of the pier with a single layer of FRP, assuming perfect adhesion to the concrete. The load-deformation response in Figure 8 demonstrates a substantial increase in both the peak strength and ductility. Figure 9b shows the cracking pattern and deformation that the pier experienced prior to failure. This behavior would be ideal for the pier. However, the assumption of perfect bond may not be realistic.

The third methodology was based on a single layer of FRP wrapping, but with consideration to the bond with the concrete elements. The numerical results are shown in Figures 8 and 9c. A significant improvement in the deformation and lateral load capacities were obtained in comparison to the predicted actual condition of the pier (Figure 5 and 6b). The displacement corresponding to the peak lateral load capacity of 9 330 kN was 37.7 mm, while in the actual condition the displacement was 10.9 mm at a peak load of 8 620 kN. Relative to the predicted design condition, this strategy provided an increase in the displacement but at a lower peak lateral load.

A complementary rehabilitation was considered that assumed two layers of FRP wrapping around the bridge pier. The results are illustrated in Figures 8 and 9d. A relatively minor increase in ductility and stiffness is provided by this model in comparison to the model with only one layer of FRP, which establishes that the increase in the displacement capacity and lateral load capacity when using FRP jacketing is not linear when increasing the number of layers for this specific bridge pier.

A final strategy was developed to consider a combination of grout injection to seal the initial cracking followed by a single layer of FRP wrapping. The cracking pattern and deformation predicted by the analysis is illustrated in Figure 9e. The fact that the cracks have been sealed and that the pier is confined with FRP sheets results in an improved behavior. The pier sustains a lateral load similar to the design condition, but with the capacity to experience greater displacements.

Figure 10 provides a comparison between capacity curves for the design condition, actual damaged condition and rehabilitated condition consisting of grout injection and FRP wrapping of the pier. It is observed that this methodology improves the behavior of the bridge pier and restores the original lateral load capacity and improves the displacement capacity when subjected to lateral loading. This method proved to be effective as a local repair strategy, where a significant increase in ductility is achieved, thus eliminating a brittle failure mechanism.

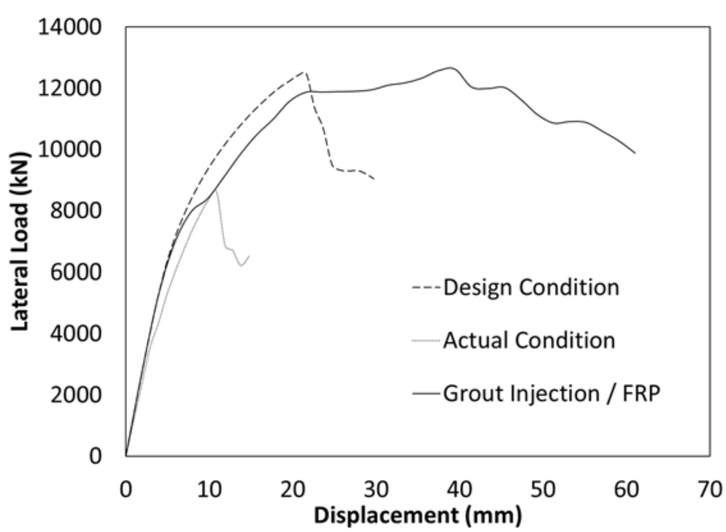


Figure 10. Comparison of capacity curves of the pier

6. CONCLUSIONS

Nonlinear finite element analyses were conducted using VecTor2 to establish a rehabilitation strategy for a damaged bridge pier. VecTor2 was selected as the numerical tool due to its capability to retain damage while investigating various rehabilitation options.

The study demonstrated that the structural cracking currently present in the bridge pier was generated as a result of failure of the neoprene pads, which functioned to distribute the gravity loading from the superstructure into the pier. As a result of this failure, the gravity load was transferred solely through the shear key giving rise to a concentrated load on the pier. This resulted in significantly splitting cracks in the pier and yielding of the shear reinforcement. In the current damaged state, the bridge pier experiences a 30% reduction in the lateral load capacity and a 50% reduction in its displacement capacity at the peak lateral load in comparison to the design conditions.

A number of rehabilitation options were considered including: grout injection only, jacketing with FRP assuming perfect bonding, jacketing with 1 and 2 layers with bond consideration, and grout injection combined with jacketing with 1 FRP layer. The analyses demonstrated that injecting grout to seal the splitting cracks followed by a single layer of FRP wrapping, was sufficient to restore the initial stiffness and lateral load capacity. In addition, the bridge pier experienced an increase in ductility and eliminated brittle failure mechanism. The application of two layers of FRP wrapping did not provide any appreciable improvement in overall behavior for this bridge pier.

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