Advanced Nonlinear Finite Element Modelling of Reinforced Concrete Bridge Piers

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ABSTRACT: Severe cracking was observed on the flexural tension face of a reinforced concrete hammerhead bridge pier cap. Determining the cause and severity of the damage was complicated by the geometry of the piers, which influenced the strut-and-tie mechanism. A tapered end cap raised questions about the development of the tension reinforcement in the chord, while a hexagonal pier column meant that the effective area of the compressive strut was uncertain. Depending on the assumptions made, widely different estimates of the ultimate capacity were reached by traditional calculation methods. Therefore, to accurately assess the safety of the bridge piers, a series of analyses were performed using VecTor2 – a nonlinear finite element analysis program (NLFEA) developed at the University of Toronto. A parametric study was conducted by varying the effective width of the piers and the amount of reinforcement engaged, and the factored strength of the pier was obtained using a novel procedure. This paper concludes that using 2-D NLFEA in a discerning manner allows engineers to understand complex 3-D situations.

1 INTRODUCTION

1.1 *Background*

Despite the development of simple and powerful tools to assess the strength of reinforced concrete structures, hidden complexity often makes determining the strength of even a simple structure a nontrivial task. Consider the following structure shown in Figure 1, which is a typical pier supporting a multi-span bridge in Canada. A recent inspection noted the presence of several vertical, and some diagonal, cracks, and it was concluded that strength of the pier needed to be determined to evaluate the overall safety of the structure. At a first glance, the structure appears to be a straightforward cantilever, suitable for a 2-D analysis using a strut-and-tie model. Looking more closely, the geometry of the pier and its bearing pads as well as its reinforcement detailing complicate matters considerably across the width. The width of the bearing pad suggests that shear lag effects may be significant - combined with the taper on the hammerhead and supporting column, it is not clear how much of the concrete section is effectively utilized. It is also not clear if all of the flexural reinforcement can be utilized, due to the anchorage detailing of the various layers of steel, as well as the aforementioned shear lag effects. When considering all of these factors, using simple code-based procedures is difficult.

Nonlinear finite element analysis (NLFEA) is a powerful tool for understanding overall structural behaviour given various geometric and material complexities. Specifically in the case of reinforced concrete, NLFEA allows engineers to consider effects such as post-cracking concrete tensile stresses, compression softening, bar development and crack slip among other things.

While NLFEA is appropriate for predicting the actual strength of a structure, it may sometimes be necessary to verify that a structure has adequate capacity within a Load and Resistance Factor Design (LRFD) framework. For example, this may be needed to check if the original structure was properly designed to carry the anticipated factored loads. However, it is problematic trying to assign a representative resistance factor to the predicted strength from an NLFEA output, since the associated resistance factors for concrete and steel are not the same, and guidelines for doing this currently do not exist.

Using the hammerhead pier structure as a case study, this paper describes a procedure for simplify-

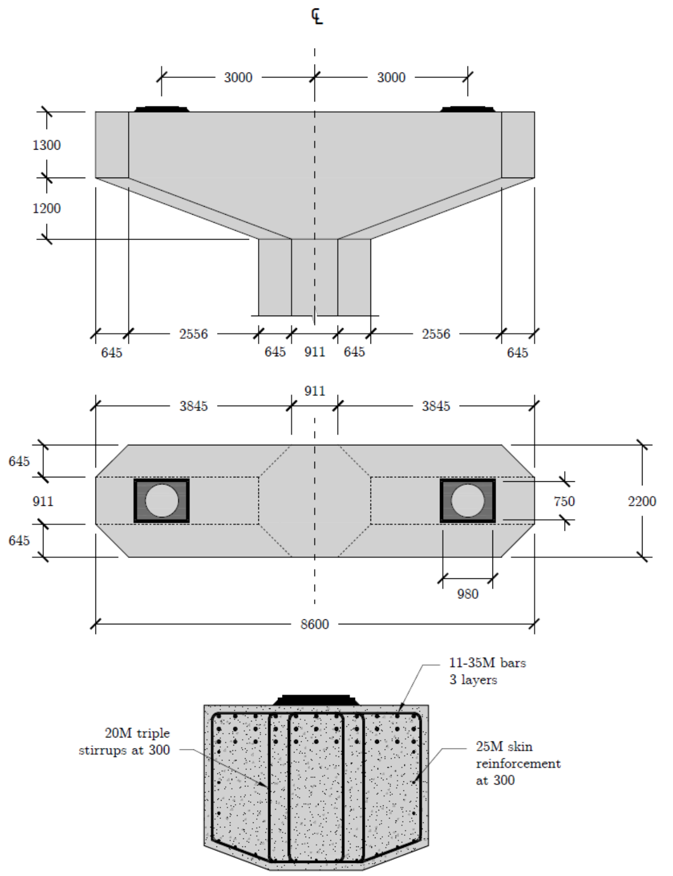

Figure 1: Elevation (top),Plan (middle) and Cross Section (bottom) of full hammerhead pier

ing the complex geometry and evaluating the strength of these piers using VecTor2, a 2-D NLFEA program developed at the University of Toronto, which uses the Disturbed Stress Field Model (DSFM) as its theoretical foundation. A method is then described to obtain the factored strength of the pier for evaluation using an LRFD framework.

1.2 *Description of Nonlinear Finite Element Analysis Methodology*

An analysis of the bridge pier was performed using a 2-D VecTor2, a 2-D finite element analysis program which is based on the DSFM. The DSFM, an advanced reinforced concrete material model based on the Modified Compression Field Theory (Vecchio 1986), is a smeared rotating-crack model which is formulated to consider tension stiffening, compression softening, and crack slip (Vecchio 2000). As the DSFM is capable of predicting the complete load-deformation behaviour of reinforced concrete elements, the full response of the hammerhead piers can be obtained from a VecTor2 model. Equally useful is the ability to calculate important parameters at any load level, such as the stresses and strains in the concrete and steel, crack patterns, crack widths and crack slips. Various other numerical models are implemented to complement the DSFM and allow for additional effects to be considered, such as tension softening, bond slip amongst others. VecTor2 uses 4-node rectangular elements for reinforced concrete materials, and 2-node truss elements for discrete steel reinforcement.

While a 3-D nonlinear finite element analysis based on the DSFM is possible to perform, it is a challenging task because a compatible preprocessor and postprocessor do not currently exist. In addition, the prohibitively large computational time needed to perform a full 3-D analysis currently makes nonlinear 3-D analysis of such a large structure impractical. Thus, a 2-D approach was used instead. To account for the aforementioned 3-D effects, a parametric study was performed to understand the sensitivity of the structural behaviour to varying model parameters. The key variables investigated were the effective section width and effective quantity of longitudinal reinforcement.

2 MODELLING PARAMETERS

The following subsections will outline the input parameters for the models created for this study. Conservative assumptions were made if precise information was not available.

2.1 *Model Mesh*

Due to symmetry, only half of the pier cap was modelled, using the centreline of the column as the vertical boundary. Figure 2 shows the breakdown of the mesh used in the analysis. To realistically represent the flow of forces in the structure, a portion of the pier column was also modelled (1100 x 1000 mm stub), with the horizontal boundary representing the base of the stub. Rollers permitting vertical displacements were placed along the vertical boundary to allow for the development of moment along this face, and pins along horizontal boundary to represent the transfer of the applied shear force into the column. A summary of these boundary conditions is shown in Figure 3.

The VecTor2 finite element model consists of 5 main regions:

Region A: The column base, which is further subdivided to represent the varying width as a result of the column taper. These regions remain the same throughout all the models, since the column itself is not being investigated. The modelling assumptions for the column taper are discussed further in Section 3.1.

Region B: The connection point of the cantilever, which is further subdivided to allow for the out-of-plane width of the cantilever to be adjusted. A key part of this study (Section 4.2) is investigating how the effective width of the cantilever influences the capacity of the pier.

Region C: The cantilever portion of the pier cap with a constant thickness throughout. This region

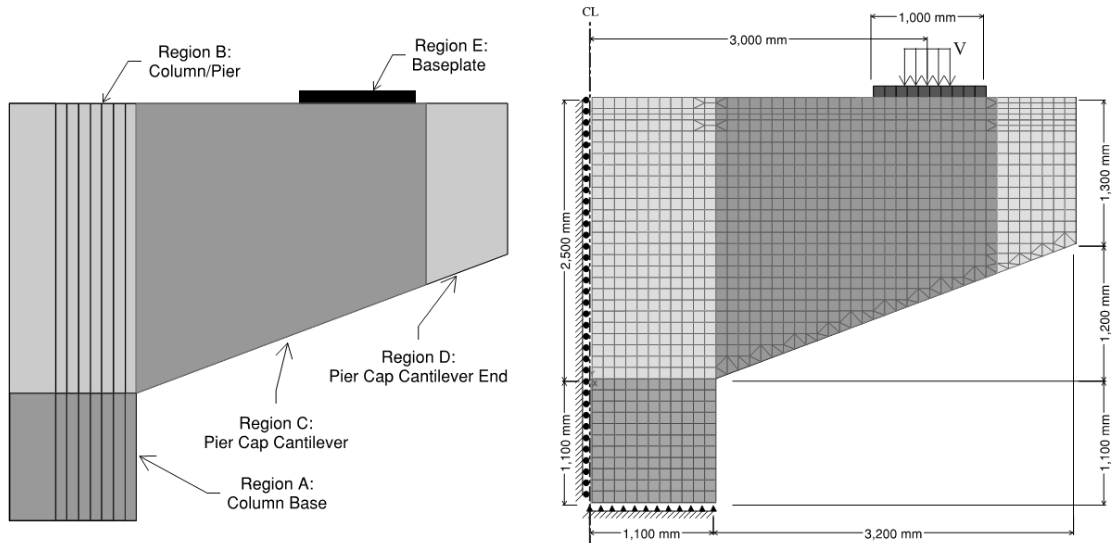

Figure 2: Finite element model representative of the pier (left = the distinct regions, right = actual element mesh)

is also be adjusted to account for the effective width of the pier (Section 4.2).

Region D: The end portion of the pier cap cantilever with a constant thickness of 911 mm throughout, which is based on the thickness of the column at its connection to the cantilever. The assumptions behind this decision are discussed further in Section 3.3.

Region E: The baseplate spreading the applied load into the structure, with dimensions based on the assumptions discussed in Section 3.4.

Since VecTor2 makes use of relatively simple linear isometric elements, the element size was chosen to be 100 mm x 100 mm (XY-plane) in order to get a fine enough strain profile describing the structural behaviour through the depth of the pier. The irregular geometry of the column was modelled using elements of varying individual thickness (into the page, Z-direction).

2.2 *Material Properties*

The material properties were chosen according to the to design specifications, with $f'_c = 35$ MPa and $f_y = 400$ MPa. The Hognestad parabola was used for the model pre-peak stress-strain behaviour of the concrete, and a bilinear model was used for the steel.

2.3 *Steel Reinforcement*

The main flexural tension reinforcement was modelled as three layers of discrete elements across the top of the pier cap. To investigate the effect of varying levels of effective reinforcement on the capacity of the pier cap, the quantity of steel for each layer ranged from 7000 $\frac{mm^2}{layer}$ (7 – 35M) to 11000 $\frac{mm^2}{layer}$ (11 – 35M).

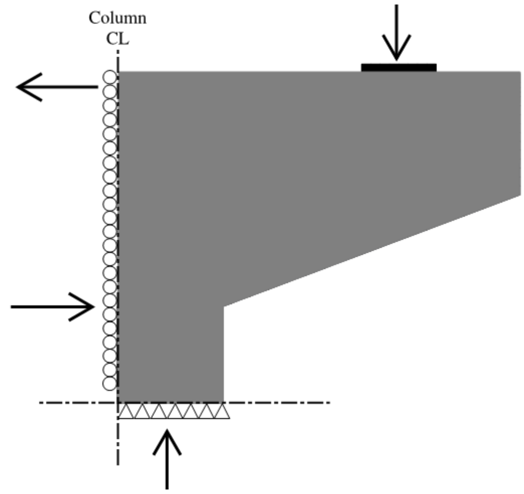

Figure 3: Boundary conditions assumed in the model and the approximate structural load path in the pier

Skin reinforcement and stirrups were distributed over the pier by averaging the steel over the concrete material in the region.

It was found that including the beneficial effects of the compression reinforcement did not lead to significant changes in the load-deformation behaviour of the piers. Thus, the analyses were performed without including discrete compression reinforcement in the finite element model.

2.4 *Applied Loading*

The bridge loading on the pier was distributed over a 500 mm width onto a 1000 mm wide base plate representative of the bearing details discussed in Section 3.4. In the analysis, the total loading on the pier was incremented by 200 kN per load stage until ultimate failure.

3 IMPORTANT FINITE MODELLING ASSUMPTIONS

To account for the complexities caused by the geometry of the hammerhead piers, the following modifications were made to the 2-D model in order to capture the effects of the real 3-D structure.

3.1 *Pier Column Taper*

As shown in Figure 4, the tapered edged of the pier column were represented as discrete steps with a width of 100 mm (to match the typical element mesh size). Each step represented a 200 mm decrease in the width, except for the final step that was set to 911 mm to match the real minimum end width.

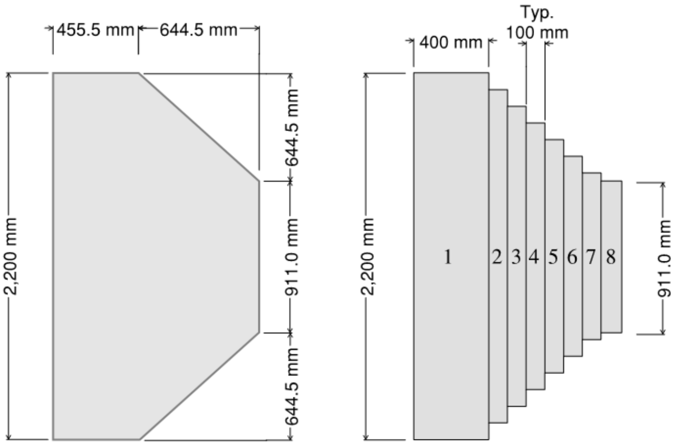

Figure 4: Cross-section of the pier half-column (left = as-built, right = model approximation)

The reinforcement in the pier column was represented as smeared throughout the slices, based on the resulting ρ_s from the as-built reinforcement as shown in Figure 5.

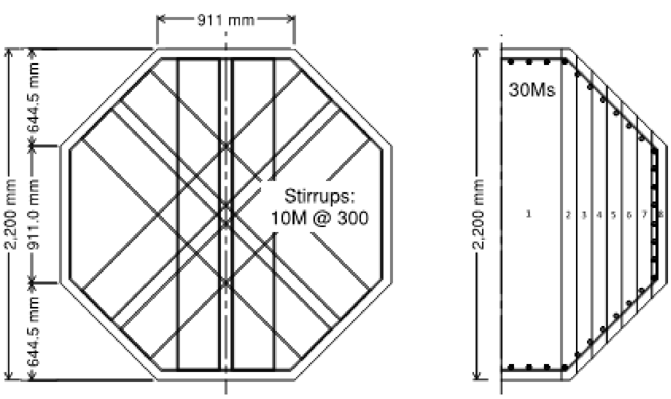

Figure 5: Reinforcement in pier column

3.2 *Pier Cap Chamfers*

As shown Figure 6, The cross-section of the pier cap cantilever was simplified to a rectangle, omitting the slight chamfer at the bottom edges and cap ends. The structural impact of this simplification was deemed minimal with respect to other parameters, such as out-of-plane effective width of the compressive strut or effective quantity of longitudinal steel.

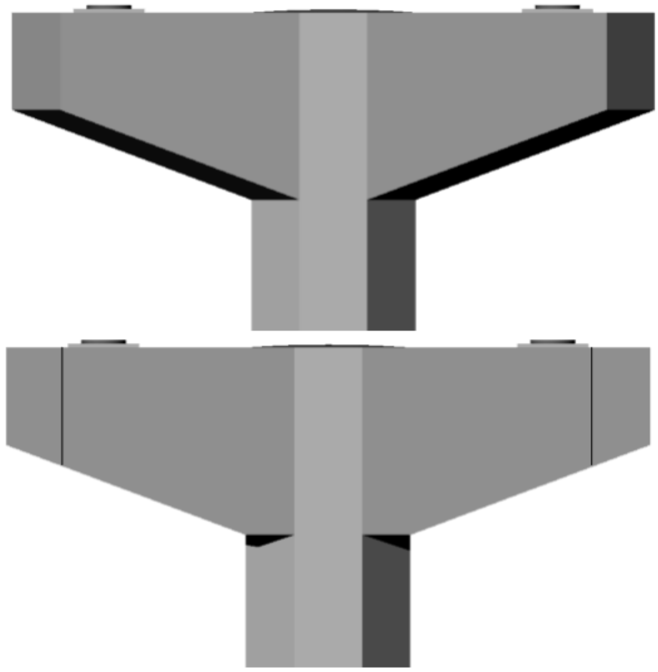

Figure 6: As-built (top) and simplified (bottom) pier cap profile. Note the uniform hammerhead thickness.

3.3 *Pier Cap Ends*

In all the models the tapered portion at the end of the pier cap was omitted leaving only the minimum 911 mm constant width section (Figure 7). This is a conservative but valid assumption since the region is not critical for the load carrying S&T mechanism, while the effect that this region has on the anchorage of the bars is handled explicitly when considering the development lengths of the reinforcement bars (Section 3.5. The variable W_{eff} refers to the effective width of the pier cap set as a parameter in the various models.

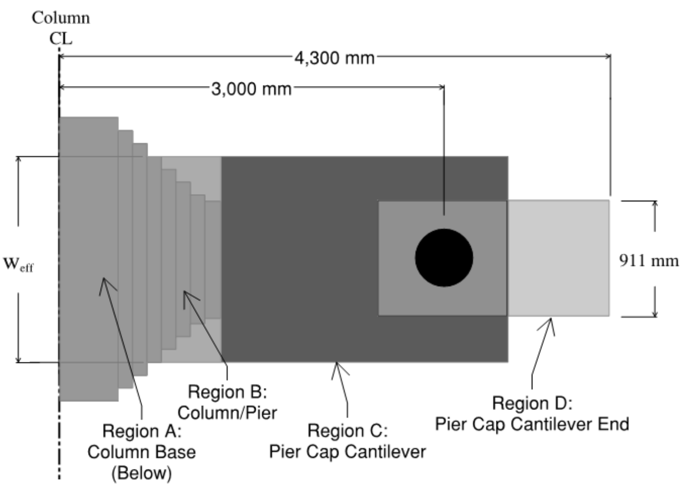

Figure 7: Plan view of pier cap showing simplified end portion

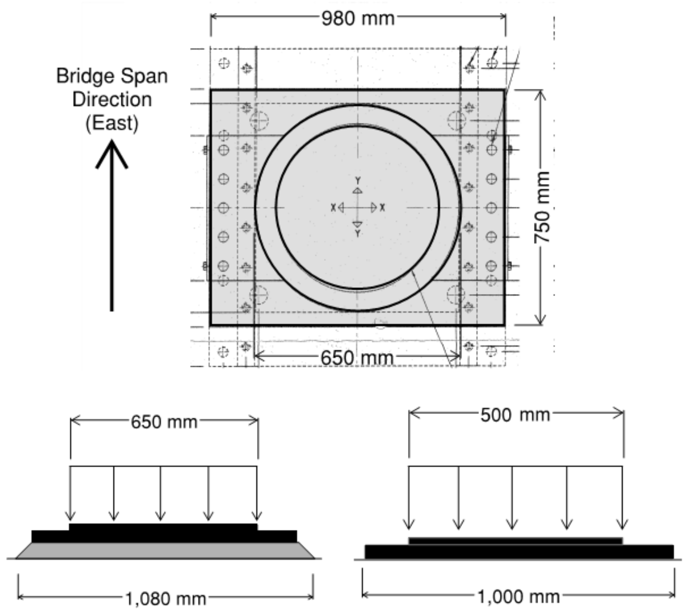

Figure 8: Plan view of pot bearing (top), with the actual (bottom left) and modelled (bottom right) loaded areas

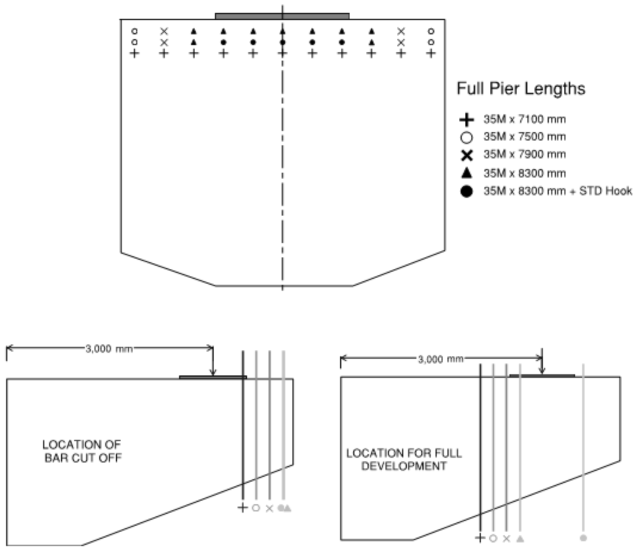

Figure 9: Cross-section of pier cap showing the 35M bars (top), where the cutoff points (bottom left) and full development points (bottom right) are shown for each type of bar

3.4 *Applied Load Bearing Area*

The pot bearing base plate is listed as 980 x 750 mm, with the bearing area further increased by 50 mm of grout at 45 degrees between the bottom of the plate and the top of the pier cap. The load is transferred through the pot plate, which has a diameter of 650 mm. As a conservative simplification, the base plate is modelled as stiff 1000 mm x 850 mm area, with the load distributed over a 500 mm width (Figure 8).

3.5 *Reinforcement Bar Anchorage*

The longitudinal reinforcement in the pier cap was made up of mostly straight deformed bars, only the middle five bars had standard hooks at the end. While the taper at the pier ends was not modelled explicitly, its effect on the development of individual longitudinal bars was accounted for using the methodology described in this section. For example, since the outer bars were cut off closer to the column centreline due to the taper, their full development also occurred closer to the column and past the center of the applied load (Figure 9) - ideally all the bars in an S&T chord should be detailed so that full development is achieved before this point.

The axial force capacity of the top chord, accounting for incomplete anchorage, was then determined along the length of the cantilever. This capacity was compared to the demand in the longitudinal reinforcement from a sample VecTor2 model (in this case $W_{\text{eff}} = 1,200\,\text{mm}$, $A_s = 11,000\,\text{mm}^2$/layer) at around the peak load stages (Figure 10). The anchorage was found to be adequate (i.e. capacity $>$ demand), especially when considering that the loads recorded in this model were significantly higher than the actual applied loads on the structure.

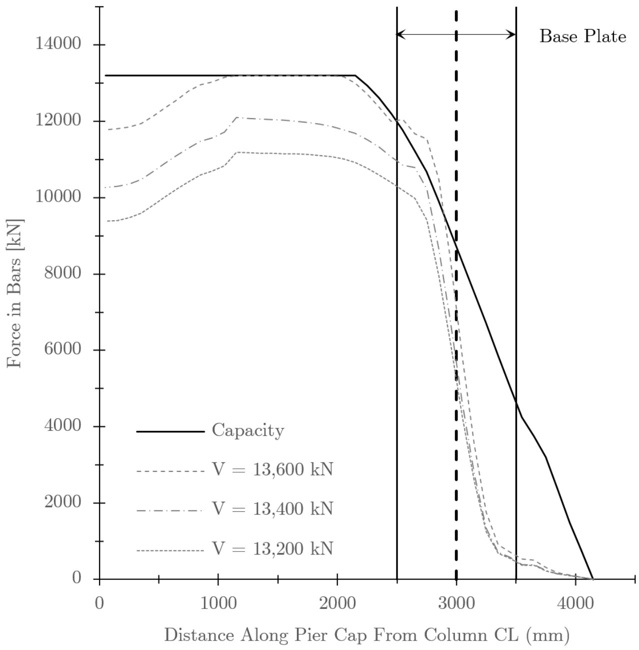

Figure 10: Demand on Longitudinal Tension Reinforcement at Applied Loads ($W_{\text{eff}} = 1200\,mm$, $A_s = 11000\,\text{mm}^2$/layer)

4 ANALYSIS RESULTS

4.1 *Establishing the Parametric Study*

A total of 40 analyses were performed to obtain a comprehensive understanding of the behaviour of the hammerhead pier. The effective width of the structure was varied between 911 mm, the width of the thinnest section of the pier, up to the full width of 2200 mm (Figure 12). Similarly, the quantity of steel included in the numerical model varied from a minimum of 7-35M bars per layer to the actual 11-35M bars per layer. This was done to account for the ambiguity in the number of bars that would be engaged by a load bearing area smaller than the full width of the pier.

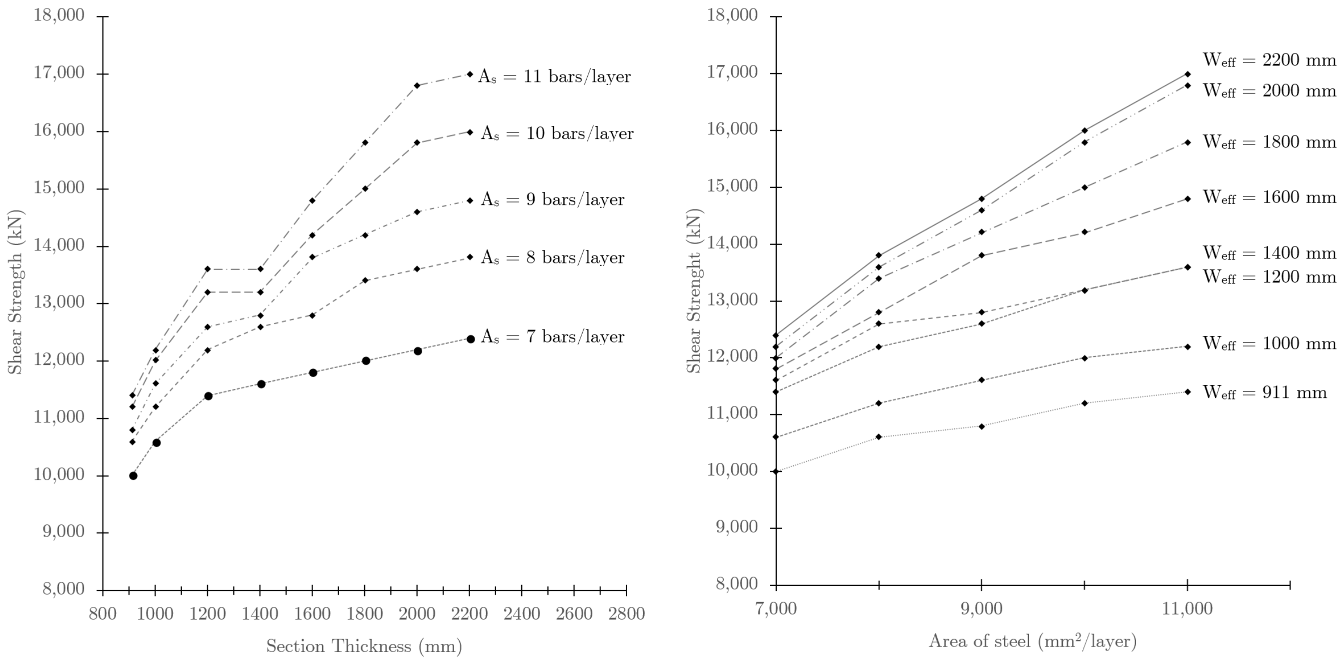

Figure 11: Influence of section thickness (left) and quantity of reinforcement (right) on ultimate capacity of pier

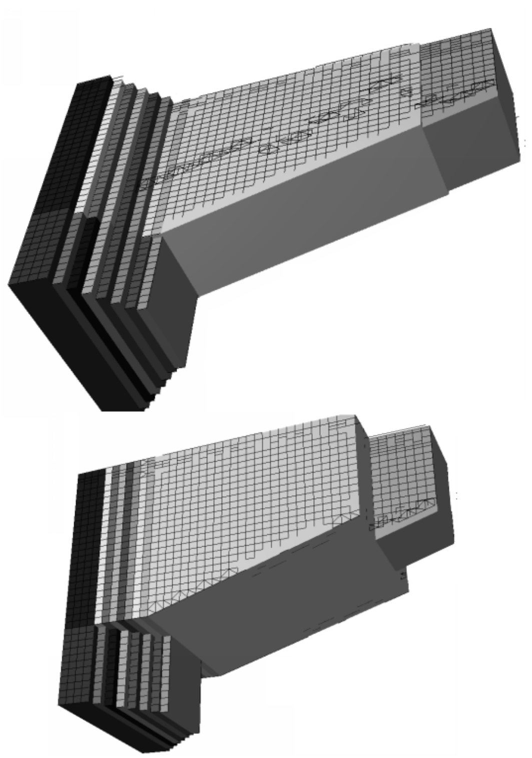

Figure 12: Effective width of pier cap varied from 911 mm (top) to 2200 mm (bottom)

4.2 *Results of Parametric Study*

Figure 11 compiles the results of all 40 analyses, with the ultimate nominal shear capacity being shown for the various widths and reinforcement quantities. It is apparent from the two plots that increasing the section thickness and quantity of reinforcement leads to gains in strength. Broadly speaking, increasing the thickness of the pier as well as the amount of steel led to proportionate gains in capacity. Generally, increasing the quantity of steel tended to provide larger gains in ultimate strength than increases in the thickness of the cross sectional area, which suggest that the amount of steel plays a major role in determining the strength of the structure. This was more pronounced in the analyses of the thicker sections, which benefited more than the thinner sections from additional steel in the tension chord.

5 DISCUSSION

5.1 *Failure Modes*

As VecTor2 is able to predict the complete load-deformation behaviour of the hammerhead pier structure, the failure mode can be understood by analyzing the element stresses and strains at failure. Figure 13 shows how the progression of failure events occurs as the effective section width is increased while maintaining the same amount of steel. If the effective section width is very thin relative to the quantity of reinforcement, then crushing of the concrete at the base of the pier occurs prior to yielding of the longitudinal steel. This can be seen in Figure 14. As the section gets larger, this pattern of events reverses and the steel yields followed by crushing causing failure.

5.2 *Integration of Results with an LRFD Framework*

Per the requirements of the National Building Code of Canada (Canadian Commission on Building and Fire Codes 2015), the factored strength of the structure must exceed the demands of the applied factored loads. The factored shear strength of the pier is thus obtained using the expression:

$$V_r = \phi_c V_c + \phi_s V_s \geq V_f$$

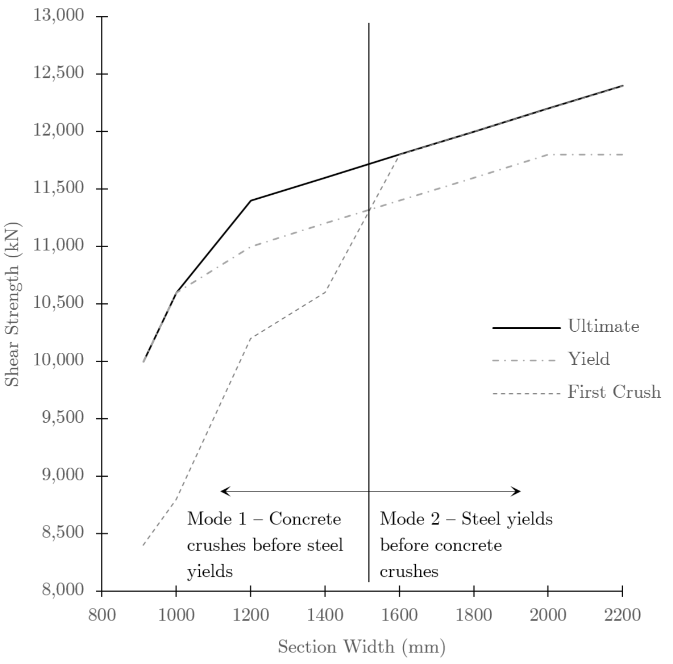

Figure 13: Influence of section width on shear strength. $A_s = 7000\,mm^2$ per layer)

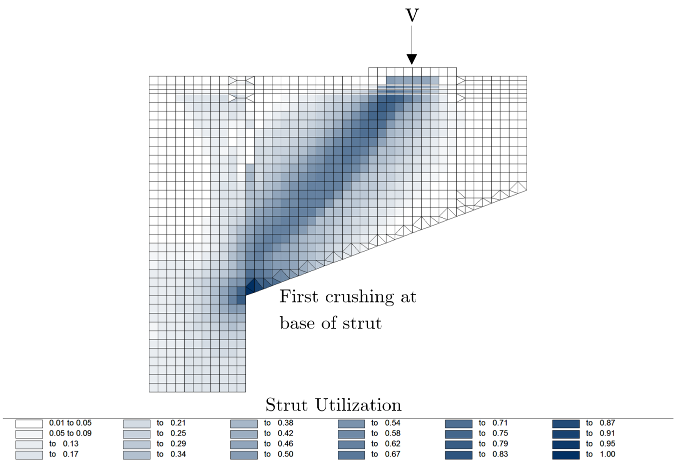

Figure 14: Crushing at the base of the pier

Where $\phi_c = 0.75$ and $\phi_s = 0.90$ are the strength reduction factors for concrete and steel accordingly (S6 Code Committee 2014). While this expression is straightforward for use with simplified design expressions, it is difficult to obtain the factored strength if the analysis is performed using NLFEA. Simply reducing the concrete cylinder strength and the steel yield strength in the model by the corresponding resistance factors is not appropriate, as doing so would result in fundamentally changing the failure mechanism of the structure.

The approach taken in this study draws inspiration from the the ACI 318-14 code's provisions for flexural design (ACI Committee 318 2014). According to these provisions, the ultimate strain in the longitudinal reinforcement can be checked at failure to classify the failure mechanism. For tensile strains greater than 0.005, the failure is said to be governed by the tension failure of the longitudinal reinforcement. Conversely, strains less than 0.005 are a mixed failure mode where the crushing of the concrete has an influence on the

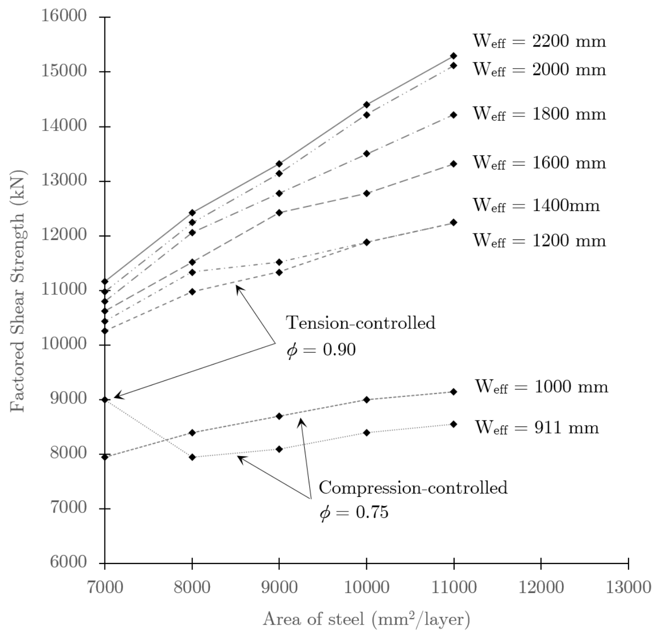

Figure 15: Factored shear strength of the pier

ultimate factored flexural capacity. When applied to the VecTor2 analysis, the maximum strain in the flexural reinforcement was checked to determine whether the failure was clearly governed by yielding of the steel. An appropriate resistance factor obtained from the Canadian Highway Bridge Code was then applied (S6 Code Committee 2014). If the capacity of the analyzed structure is clearly governed by the behaviour of the steel, multiplying the ultimate load by the steel resistance factor of 0.90 is an appropriate way to obtain the factored strength. If crushing played a role, then the factored strength was obtained by multiplying the peak load by the concrete resistance factor of 0.75.

$$V_r = \begin{cases} \phi_s V_{ult} & \text{if } \varepsilon_s \geq 0.005 \text{ at failure} \\ \phi_c V_{ult} & \text{otherwise} \end{cases}$$

It should be noted that this analysis procedure is conservative for mixed failure modes where crushing occurs shortly after yielding, since the resistance factor used is either 0.75 or 0.90. Realistically, there should be a smooth transition where mixed failure modes which are controlled by both tension and compression are treated using a blended resistance factor which lies somewhere between 0.75 and 0.90.

Figure 15 shows the factored strength of the piers following an interpretation of the analysis results. In almost all cases, the ultimate strength is governed by yielding of the flexural reinforcement, with the exception of the thinnest possible section width (911 mm) or just slightly larger (1000 mm). The transition from tension-governed to compression-governed failure modes makes the spread of possible factored strengths particularly large, with the most conservative estimates of the factored strength being almost half the value of the most optimistic case, where all of the reinforcement and the full width of the pier are effective. When assessing the actual piers in question, it was determined that even with the most conservative assumptions, the structure was able to carry the required design loads.

5.3 *Crack Pattern Comparison*

As the purpose of the numerical modelling was to verify the safety of the cracked hammerhead pier, the crack pattern predicted by VecTor2 was compared against those seen in the actual piers. Figure 16 shows the predicted crack pattern of a typical analysis when the structure is carrying 50% of its maximum capacity, as well as a representative load-displacement plot which characterized almost all of the analyses. While the pier is heavily cracked, with predicted crack widths of up to 1.5 mm, it is quite far away from failing. All of the analyses performed predicted that the load causing first cracking was a small fraction of the ultimate strength, and well below the predicted service loads.

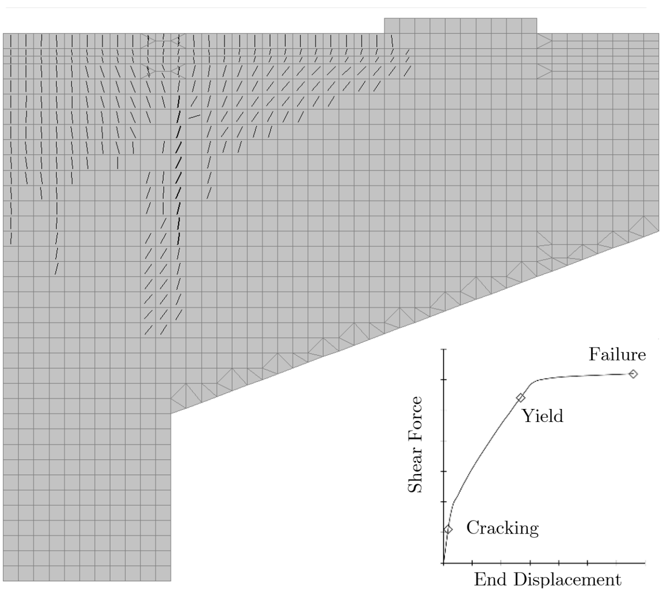

Figure 16: Typical crack pattern at 50% of member capacity

6 CONCLUSION

Nonlinear finite element analysis is a key part part of the modern structural engineer's toolbox when conducting an assessment of an existing structure which may be in distress. As shown from this case study of a cracked hammerhead bridge pier structure, the behaviour of seemingly simple structures can be complicated by subtle geometric features. In situations such as these, it is appropriate to turn to NLFEA to better understand the fundamental behaviour of these structures.

VecTor2, a 2-D NLFEA program based on the DSFM, was used to assess the piers. An extensive parametric study was performed to account for possible 3-D effects in the structure. The variables explored were the effective section width of the piers and the quantity of longitudinal reinforcement used, and the their effect on the piers' strength and failure mode were examined.

A simple procedure for using these NLFEA results within a LRFD framework was proposed, which assigns a representative resistance factor to the member's capacity by considering the strain in the longitudinal reinforcement at failure. While this method was used to obtain the factored strength for an existing structure, it is general enough to be used for designing new structures.

While a comprehensive 3-D nonlinear analysis to analyze reinforced concrete structures remains impractical for everyday use, the methodology discussed in this paper hopefully provides the practising engineer with a stronger foundation on which they can exercise their judgment when evaluating existing structures.

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