Numerical Modeling of a Caisson Foundation Retrofitted with Helical Piles

Serhan Guner(✉)

University of Toledo, Toledo, OH, USA
serhan.guner@utoledo.edu

Abstract. While the nonlinear finite element analysis methods have been commonly used for the performance assessment of existing structures, their use for the retrofit design of concrete foundations has remained limited. One reason for this is the sophisticated modeling process which requires knowledge, experience, and caution. This study demonstrates the applicability and benefit of the nonlinear finite element modeling for the performance-based structural retrofit design of caisson foundations. The foundation system investigated supports a self-supporting telecommunication tower located in Canada. The addition of new antennas and the change in the design standards requires the caisson foundations of this tower to be retrofitted with new cap beams and helical piles to resist significant additional tensile forces. A two-stage analysis and design process is conducted with the help of a continuum-type finite element analysis method, treating reinforced concrete as an orthotropic material and employing the constitutive relations of the Disturbed Stress Field Model. General modeling guidelines and the points for caution are discussed for the retrofit design of caisson foundations using nonlinear analysis methods.

1 Introduction

Nonlinear finite element analysis methods have seen significant advancements in the past decade. Various constitutive models and element formulations have been proposed. While these methods have been widely used by researchers, their practical application for the strengthening of reinforced concrete foundations has remained limited. One reason for this is the sophisticated modeling process which requires knowledge, experience, and caution. The objective of this study is to demonstrate a modeling methodology which can be employed when conducting a retrofit design in a design office environment. This methodology was developed during an actual design project to strengthen the caisson foundations of a number of existing telecommunication towers.

Self-supporting towers are commonly constructed using a triangular plan layout with three caisson foundations. Each caisson resists significant amounts of axial compression and tension loads in addition to a small shear force. Due to the changing wind direction, the axial load fluctuates between tension and compression, creating reversed-cyclic loading conditions. This makes the design of caissons for self-supporting towers more challenging than that of other types of caissons subjected to compression loads only. Caissons typically develop their tensile resistance by skin

M. Abu-Farsakh et al. (eds.), *Advances in Analysis and Design of Deep Foundations*,
Sustainable Civil Infrastructures, DOI 10.1007/978-3-319-61642-1_15

friction only, as opposed to skin friction and tip bearing for the compressive resistance. As such, retrofitting existing caissons to increase their tensile load resistance presents significant challenges.

A number of retrofit designs are used in industry to increase the axial load capacities of existing caissons. For example, weight blocks connected to caissons with epoxied dowel bars are commonly used to provide a small amount of additional tensile resistance. For larger overloads, new anchors consisting of new caissons, micro piles, or helical piles are commonly used. One challenge in designing these retrofit solutions is to ensure that the retrofitted system indeed works as a whole to carry the additional loads. The connections between the existing caissons and the new elements are one critical aspect that requires special attention due to the brittle nature of concrete which does not permit simple bolted or dowelled connections.

The literature investigating the structural behaviour of retrofitted caisson foundations remain very limited. One study was performed by Abdalla (2002) who presented a case study involving self-supporting and guyed tower foundations, and proposed repair and strengthening solutions. However, no numerical analysis and verification studies were presented. Another study was published by Guner and Carrière (2016), which forms the basis of this paper.

2 Proposed Analysis Methodology

2.1 Structure Definition

The tower examined has a height of 90 m with a face-width of 12.2 m at the base, as shown in Fig. 1. The tower is located in a residential area of Toronto, Ontario. It was designed and constructed in the early 1970's. Due to the high demand to add antennas on this tower, the tower mast has been reinforced several times in recent years. The tower has three caisson foundations; one caisson is shown in Fig. 2. There is an existing equipment building located at the centroid of the tower, which further limits the available area and the head clearance for the retrofit design. Each caisson has a diameter of 1067 mm, and a length of 10.7 m. The reinforcement includes 30-#9 longitudinal reinforcing bars and #3 circular hoops spaced at 300 mm, as indicated on the original design drawings. These drawings also specify a concrete compressive strength of 27.6 MPa, a reinforcing steel yield strength of 414 MPa, and a concrete cover of 76 mm. The soil profile includes by up to 2.7 m loose to compact sand and silt fill, 1.9 m compact silty sand, and glacial till of clayed silt and some sand, with a water table at about 11 m, as indicated in the geotechnical investigation report.

The structural analysis results indicated the maximum factored uplift and compression reactions to be 1530 kN and 1740 kN, respectively, at each caisson, considering the increased antenna loading and the latest versions of the design standards. The factored uplift capacity was calculated to be 675 kN using the geotechnical resistance factor of 0.375 in the Canadian CSA S37 standard (2001). Considering the 136 kN self-weight of the caisson, an overload factor of 2.1 was obtained. Consequently, an additional uplift capacity of 800 kN was required per caisson. Due to the limited space available on the tower site, two helical piles, each with 400 kN factored

Fig. 1. The self-supporting tower examined

Fig. 2. One of the caisson foundations to be retrofitted

tensile capacity, was employed in the proposed design (see Figs. 3, 4, and 5). The design of the helical piles was conducted in a separate geotechnical study.

The main challenge in using helical piles is the design of an effective connection between the steel pile shafts and the existing concrete caissons. One commonly used approach is to employ a reinforced concrete cap beam to provide an offset from the existing caissons, while connecting the new helical piles to the existing caissons. In this study, a depth of 1000 mm and a width of 800 mm was used to provide the required stability to the cap beam. A clear span of 700 mm was used between the caisson and the piles as per the geotechnical recommendations. This created a deep beam with a clear span-to-depth ratio of 0.7. Recall that deep beams do not satisfy the ‘plane sections remain plane’ hypothesis, and require a suitable formulation to capture the deep beam effects. The following sections present the verification studies using a nonlinear finite element method, while taking account of the deep beam effects.

2.2 Nonlinear Finite Element Modeling Guidelines

A two-dimensional nonlinear finite element analysis modeling was conducted using the computer program VecTor2, which incorporates constitutive models specifically developed for analyzing cracked reinforced concrete (Wong et al. 2013). VecTor2 employs a smeared rotating crack model based on the equilibrium, compatibility, and

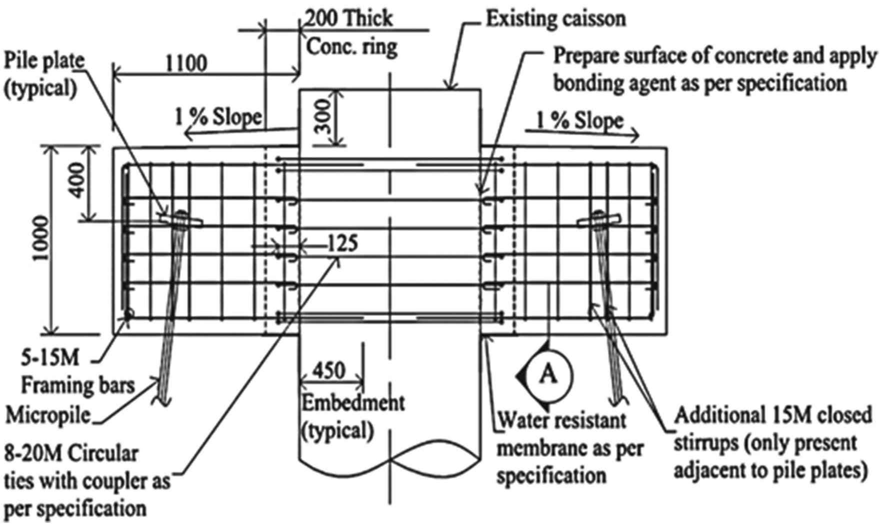

Fig. 3. Elevation of the proposed retrofit design

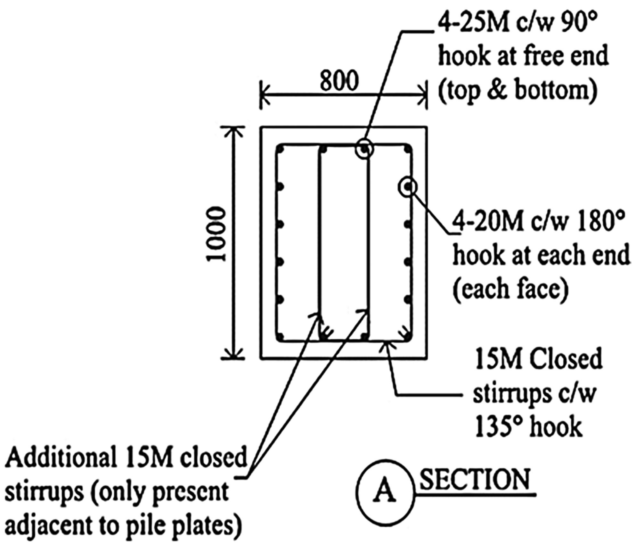

Fig. 4. Section of the proposed retrofit design

constitutive models of the Disturbed Field Model (Vecchio 2000), which is a refined version of the Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986). Although other specialized programs such as ATENA (Cervenka 2016), WCOMD (Maekawa 2016), and DIANA (2016) could also be used for this purpose, the selection of VecTor2 was made because of two reasons: (1) it accounts for a large number of second-order material behaviors models relevant to this modeling study; and

Fig. 5. Construction of the proposed retrofit design (Guner and Carrière 2016)

(2) the MCFT has been recognized internationally and adopted by many design codes such as Canadian CSA A23.3 (2014) and American AASHTO LRFD (2016).

When modeling reinforced concrete structures, proper modeling of the constitutive response and important second-order material behaviors are crucial (Guner and Vecchio 2010a, b). The material models considered in this study are listed in Table 1. Among them, three models were found to be particularly important for the cap beam examined: *the concrete compression softening* (i.e., the reduction in the uniaxial compressive strength and stiffness due to transverse tensile cracking), *the concrete tension stiffening* (i.e., the ability of cracked reinforced concrete to transmit tensile stresses across cracks), and *the dowel action* (i.e., the additional shear strength provided by the main reinforcing bars). First of all, the low amounts of stirrup reinforcement present in the existing caisson makes it prone to transverse cracking under large axial forces, which requires the consideration of 'concrete compression softening'. Secondly, the cap beam is prone to cracking and its response is sensitive to the amount of tension transmitted across cracks, requiring the modeling of the 'concrete tension stiffening' effects. Finally, the shear force transfer at the beam-caisson interface may influence the response of the entire system, such that the additional shear resistance due to the 'dowel action' should be considered. More details on these material models can be found in Wong et al. (2013).

Table 1. Material behaviour models considered

Material behaviour	Default model
Compression base curve	Popovics (NSC)
Compression post-peak	Modified Park-Kent
Compression softening	Vecchio 1992-A
Tension stiffening	Modified Bentz 2003
Tension softening	Linear
Confinement strength	Kupfer/Richart
Concrete dilatation	Variable – Orthotropic
Cracking criterion	Mohr-Coulomb (Stress)
Crack width check	Agg/5 Max crack width
Concrete hysteresis	Nonlinear w/plastic offsets
Slip distortion	Walraven
Rebar hysteresis	Seckin w/Bauschinger
Rebar dowel action	Tassios (Crack slip)

2.3 Global Design Verification

The finite element mesh was created as a result of an iterative refinement process, starting with a coarse mesh and refining it gradually. The final mesh incorporated 50 × 50 mm, 8-degree-of-freedom quadrilateral elements, with a capability to account for the geometric nonlinearities. The uplift load was applied to the bearing plate at two nodes. The final mesh is presented in Fig. 6.

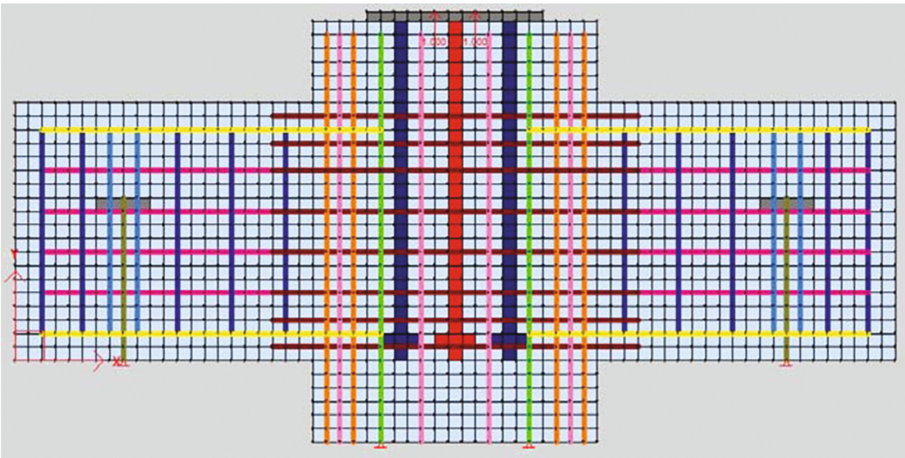

Fig. 6. Finite element model for the global design verification

Five different continuum regions were created based on the material properties as listed in Table 2, and shown in Fig. 6. To represent the 51 mm-dia anchor bolts, equivalent square areas were defined to match the finite element mesh. All reinforcing bars and the helical pile shafts were modelled using discrete truss bars to be able to observe their behaviour and to obtain their stress/strain conditions. Table 3 summarizes the material properties of the truss bars defined, which were obtained from the manufacturer specifications for the new bars and the original design drawings for the existing bars. The response of truss bars was modeled with a stress-strain curve including the Bauschinger effects, using the constitutive model of Seckin (1981) as shown in Fig. 7. The response of concrete was modelled using the plastic-offset-based nonlinear model of Palermo and Vecchio (2003) as shown in Fig. 8. This concrete model includes the nonlinear hysteresis rules for the unloading and reloading conditions. Note that some parts of the cap beam will unload, and some other parts will reload, as concrete cracking and reinforcement yielding take place.

Table 2. Continuum region properties

Region	Description	Color	$f'c$ (MPa)	fy (MPa)	Thickness (mm)
1	Concrete		30	-	800
2	Anchor Bolt (Single)		-	414	40.5
3	Anchor Bolts (Double)		-	414	81
4	Pile Head Plate		-	400	200
5	Base Plate		-	414	554

Table 3. Truss bar properties

Truss	Description	Color	Area (mm^2)	fy (MPa)	Diameter (mm)
1	Stirrups (2-15M)		400	400	16
2	Double Stirrups (4-15M)		800	400	16
3	Main Bars (4-20M)		1200	400	19.5
4	Skin Bars (2-20M)		600	400	19.5
5	Hoop bars(2-20M)		600	400	19.5
6	Helical Pile Shaft (32 Dia)		806	830	32
7	Caisson Bars (2-#9)		1290	414	28.65
8	Caisson Bars (4-#9)		2580	414	28.65
9	Caisson Bars (3-#9)		1935	414	28.65

Definition of support conditions is a critical aspect of the modeling process. Four hinges were found to represent the actual support conditions reasonably well. Two hinges were defined to support the helical piles to create conservative loading conditions for the cap beam. The other two hinges were used to ensure that the existing caisson does not exceed its calculated ultimate capacity of 675 kN using the geotechnical resistance factor of 0.375. 6-#9 bars (shown with green color in Fig. 6) were restrained for this purpose. A displacement-controlled pushover analysis was performed using an increment equal to 0.25 mm.

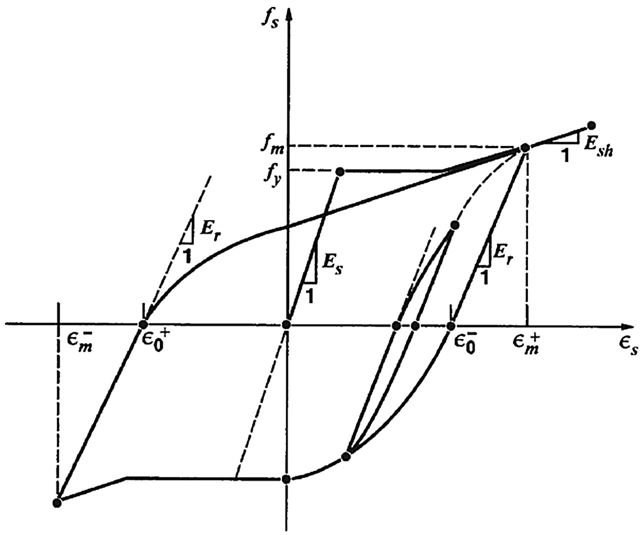

Fig. 7. Reinforcing bar response

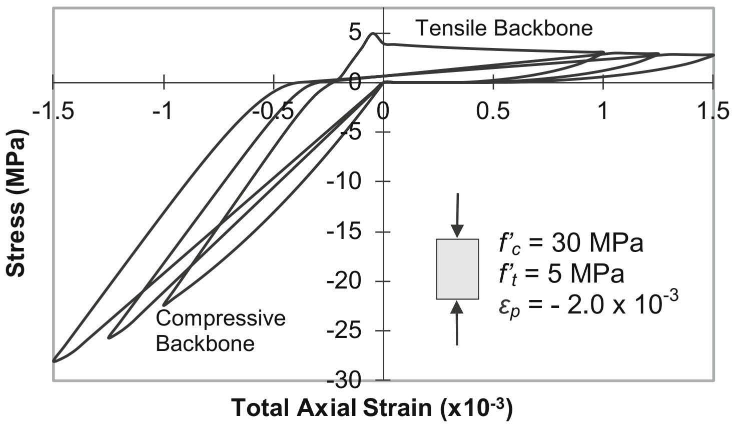

Fig. 8. Concrete response

The analysis indicated that the first concrete cracking occurs at a tensile leg load of 930 kN, as shown in Fig. 9, which is approximately equal to the service tension load. The retrofitted system exhibited a flexure-dominated response at the ultimate conditions as shown in Fig. 10. The failure mode involved yielding of the helical pile shafts at a leg load of approx. 3000 kN, as shown in Fig. 11. This is a desired failure mode, which indicates that the global response is acceptable. Figure 12 shows the load-deflection response of the global system. Since the required ultimate leg tension is 1530 kN, the global design capacity of 3000 kN is excessive. It will be seen in the following section

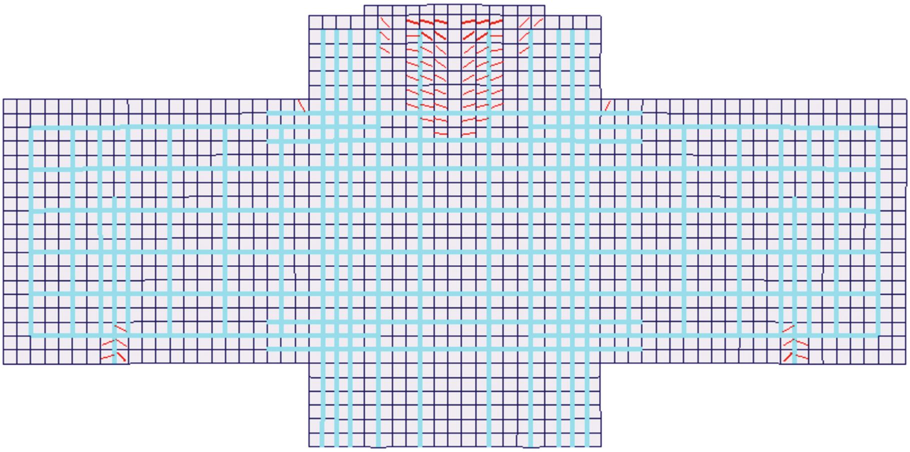

Fig. 9. Crack pattern at first cracking

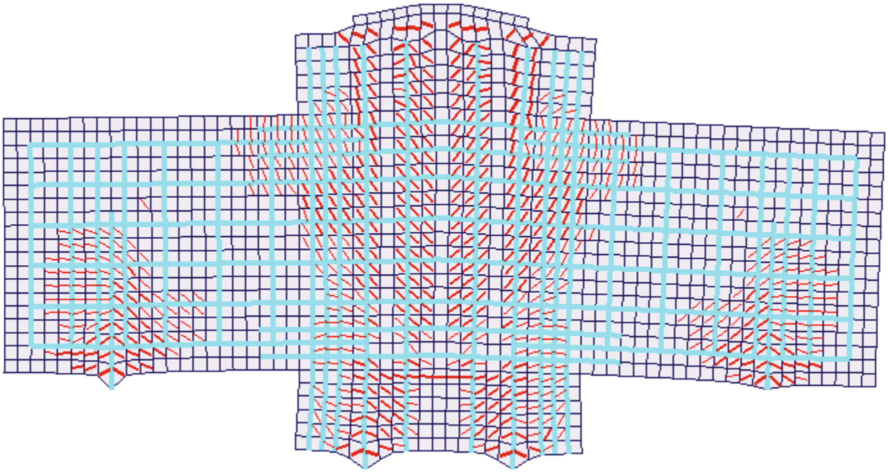

Fig. 10. Crack pattern at failure

that the design will be governed by the local response of the discontinuous dowel bars. Consequently, no change is necessary for the global design.

2.4 Local Design Verification

The site conditions and the presence of existing caisson's vertical reinforcing bars (i.e., 30-#9 – shown with orange, pink, and green bars in Fig. 6) makes it practically impossible to drill through the existing caisson to provide continuous main reinforcement to the new cap beam. As shown in Figs. 3 and 6, the main horizontal

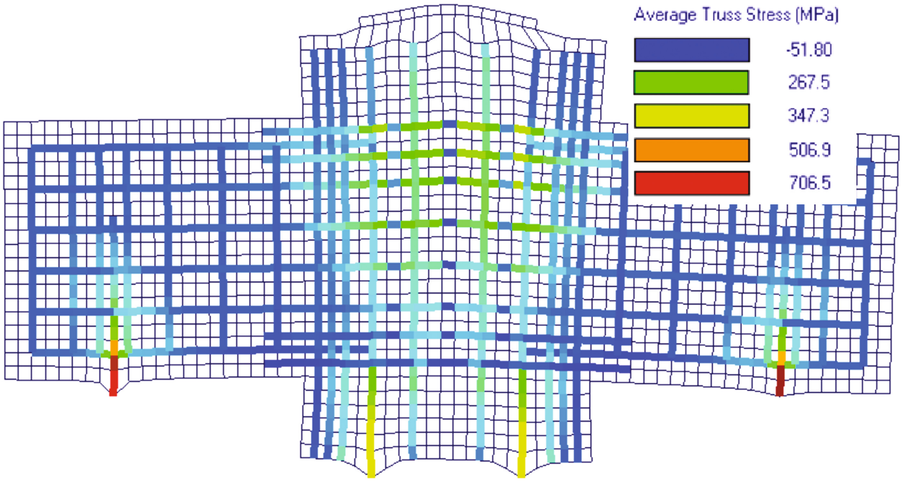

Fig. 11. Reinforcing bar stresses at failure

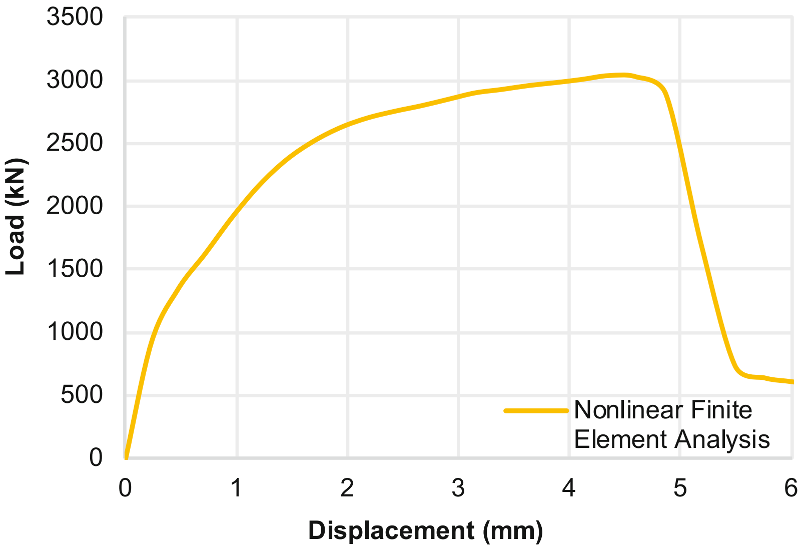

Fig. 12. Load-deflection response for the global response verification

reinforcing bars (shown with yellow bars in Fig. 6) is terminated inside the existing caisson through the use of an epoxy adhesive. The embedment length (shown in Fig. 3) required to develop the bond strength is typically provided by the adhesive manufacturer, which was in the range of 250 mm for the product that we selected. It should be noted that using the recommended bond development length will not ensure that the required bar tension can be successfully carried. A system of reinforcing bars or supports is required to carry this tension. To achieve this transfer, supplementary hoop reinforcement was used in the proposed design. Due to the obstruction of the existing tower legs, two half-circle hoops, connected with mechanical couplers, were employed. This is the most critical aspect of the proposed design; if not designed properly, it can render the entire retrofit design ineffective. To determine the required hoop quantity and to verify the resulting system response, a detailed local finite element analysis was undertaken using the program, VecTor2.

A finite element model was created using 3944 triangular elements (each with 6 degrees of freedom and 150 mm thickness) and 2054 nodes. The discontinuous reinforcing bars and the double-hoop reinforcement were modelled using perfectly-bonded discrete truss elements (each with two degrees of freedom at each node). The model was restrained with four hinges on one side, and the loading was applied uniformly on the other side with 0.1 mm displacement increments. A displacement-controlled analysis was employed to obtain the post-peak response, ductility, and failure mode. The finite element mesh is presented in Fig. 13.

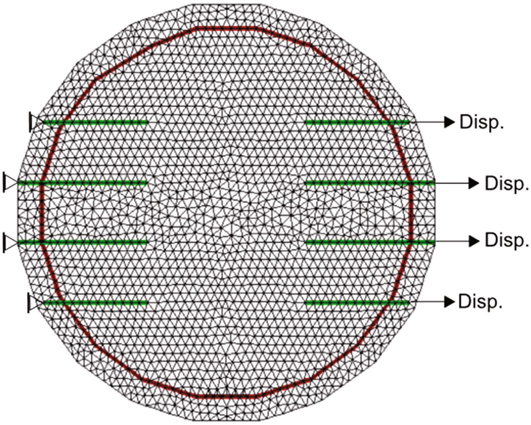

Fig. 13. Finite element model for the local design verification

In order to determine the required embedment length for the discontinuous dowel bars (shown in Fig. 3), six different models were created by varying the dowel bar embedment lengths: 650, 550, 450, 350, 250, and 170 mm for Models 1 to 6, respectively. The load-displacement responses for all six models are presented in Fig. 14. The responses of Models 1 to 5 exhibited similar behaviours: an initial peak load, followed by a sudden drop due to major cracking at the termination of the reinforcement, and a stiffening response due to the activation of the supplementary hoop reinforcement. Model 6, which had an embedment length less than the length recommended by the adhesive manufacturer, exhibited a brittle failure upon first cracking at an applied load of 200 kN. The hoops were ineffective in this model as evident from the suddenly dropping load capacity in Fig. 14.

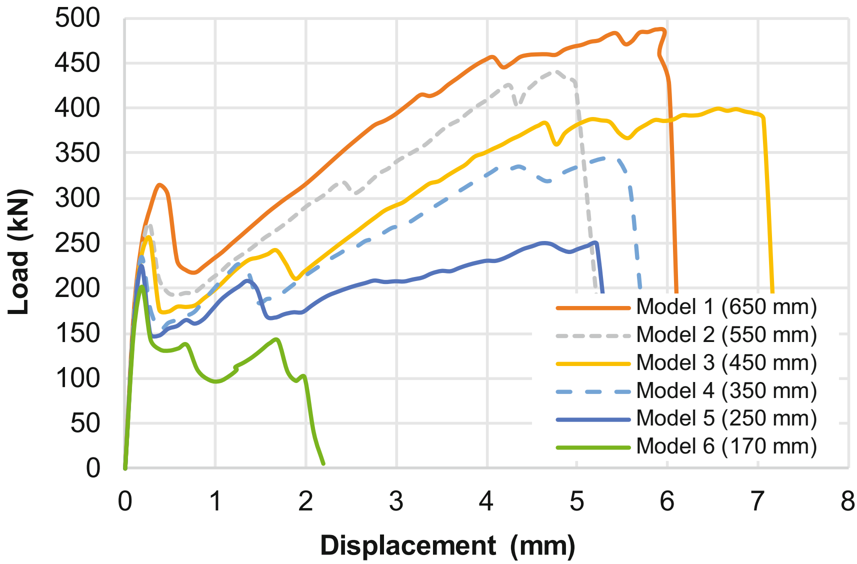

Fig. 14. Load-deflection responses for the local response verification

Analysis results indicated that the required minimum failure load of 400 kN, which corresponds to the ultimate capacity of the 4–20 M bars (shown with yellow bars in Fig. 6 and green bars in Fig. 13) was achieved with an embedment length of 450 mm (Model 3). This model exhibited a ductile response governed by the yielding of the supplementary hoop reinforcement. The three stages of cracking are presented in Fig. 15.

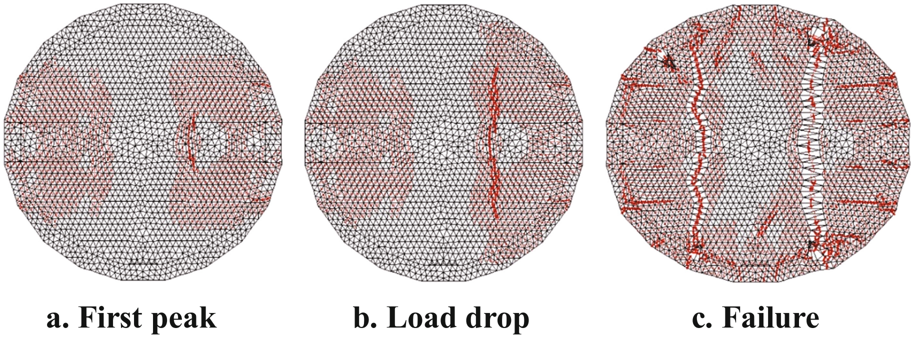

Fig. 15. Crack pattern for Model 3

The change in the dowel bar embedment length (shown in Fig. 3) affected the load capacity and the failure mode of the caisson significantly as seen in Fig. 16. An embedment length of 250 mm, which is recommended by the adhesive manufacturer,

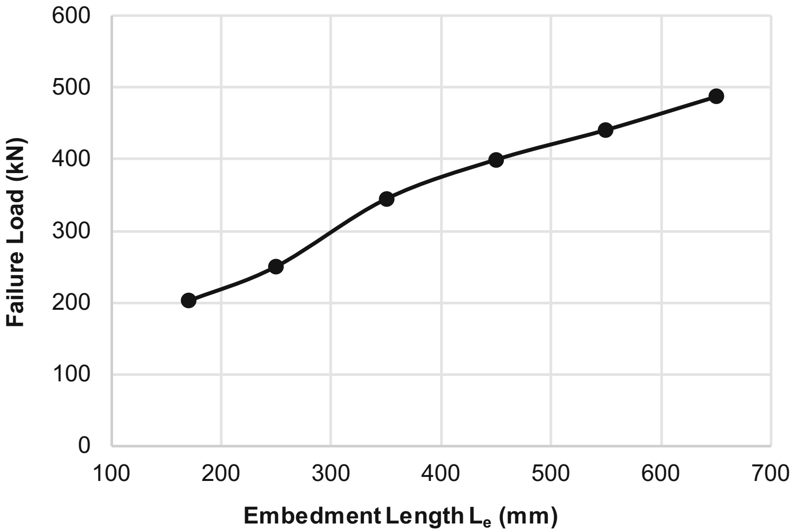

Fig. 16. Effect of the embedment length

resulted in an undesirable failure mode involving the local failure of concrete. The hoop steel was partially effective, and increased the load capacity by only 11% beyond the first peak load (as compared to 55% in Model 1). The failure load obtained was 250 kN, which is significantly lower than the required value of 400 kN.

3 Conclusions

(1) To significantly increase the tensile capacities of caisson foundations, addition of new structural elements is required.
(2) Helical piles are one viable element that can provide significant additional axial capacity in tension and compression. They are particularly useful for sites where this is limited space and limited head clearance.
(3) It is recommended that the actual load capacity of helical piles should be verified on site using at sacrificial pile tests. The complete load-deformation response should be obtained and provided to the structural design engineer for the design validation.
(4) This study demonstrated that helical piles can be connected to existing caisson foundations using reinforced concrete cap beams. It was found that the dimensions of the cap beams should be large enough to provide the required stiffness and stability.
(5) A proper analysis method must be employed to verify the global design. For deep beams, the analysis method must account for the nonlinear strain distribution. Simple sectional analysis methods with simply-supported slender beam approaches are not valid for deep beams.

(6) The critical aspects of the design (e.g., epoxy anchored bar embedment length in this study) must be verified by a proper local analysis method.
(7) Providing the recommended bond development length for epoxy anchored bars does not ensure that the bar tension can safely be carried. The designer must ensure that there are adjacent rebars available (or designed) to transfer the tension load of the terminated bars to a support point or other reinforcing bars.
(8) The analysis and design methodology proposed in this study was numerically shown to increase the uplift capacity of an existing caisson by a factor of 2.1. Overall behaviour, ductility, and the failure mode of the retrofitted system were found to be satisfactory.
(9) The proposed design has a general applicability and is suitable for applications where there is limited space around the existing caissons.

Acknowledgements. The author would like to thank Mr. Salim Khoso (a graduate student at the University of Toledo) and Mr. Sálvio Aragão Almeida Júnior (an undergraduate summer student funded by the Science Without Borders Scholarship Program) for assisting in the finite element analyses presented in this paper.

References

AASHTO: LRFD Bridge Design Specifications, Customary US Units, 7th Edn., with 2015 and 2016 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, DC (2016)

Abdalla, H.A.: Assessment of damages and repair of antenna tower concrete foundations. Constr. Build. Mater. **16**(8), 527–534 (2002)

Cervenka, V.: ATENA: software for nonlinear analysis of reinforced concrete structures, Prague, Czech Republic (2016). http://www.cervenka.cz

CSA A23.3: Design of Concrete Structures. CSA Standard A23.3. Canadian Standards Association (CSA), Mississauga, ON, 290 pp. (2014)

CSA S37: Antennas, Towers, and Antenna-Supporting Structures. CSA Standard S37-01. Canadian Standards Association (CSA), Mississauga, ON, 118 pp. (2001)

DIANA: User's Manual. TNO DIANA BV. Delft, The Netherlands (2016). http://dianafea.com/manuals/d93/Diana.html

Guner, S., Carrière, J.: Analysis and strengthening of caisson foundations for uplift loads. Can. J. Civ. Eng. **43**(5), 411–419 (2016)

Guner, S., Vecchio, F.J.: Pushover analysis of shear-critical frames: formulation. Am. Concr. Inst. Struct. J. **107**(1), 63–71 (2010a). http://www.utoledo.edu/engineering/faculty/serhan-guner/docs/JP1-107-s07.pdf

Guner, S., Vecchio, F.J.: Pushover analysis of shear-critical frames: verification and application. Am. Concr. Inst. Struct. J. **107**(1), 72–81 (2010b). http://www.utoledo.edu/engineering/faculty/serhan-guner/docs/JP2-107-s08.pdf

Maekawa, K.: WCOMD: Nonlinear Analysis Program of Reinforced Concrete Structures. University of Tokyo, Tokyo (2016). http://www.forum8.co.jp/english/uc-win/wcomd-e.htm

Palermo, D., Vecchio, F.J.: Compression field modeling of reinforced concrete subjected to reversed loading: formulation. Am. Concr. Inst. Struct. J. **100**(5), 616–625 (2003)

Seckin, M.: Hysteretic behavior of cast-in-place exterior beam-column-slab assemblies. Ph.D. thesis, Department of Civil Engineering, University of Toronto, ON, Canada, 236 pp. (1981)

Vecchio, F.J.: Disturbed stress field model for reinforced concrete: formulation. ASCE J. Struct. Eng. **126**(8), 1070–1077 (2000)

Vecchio, F.J., Collins, M.P.: The modified compression field theory for reinforced concrete elements subject to shear. ACI Struct. J. **83**(2), 219–231 (1986)

Wong, P.S., Vecchio, F.J., Trommels, H.: VecTor2 and FormWorks user's manual. Technical Report, Department of Civil Engineering, University of Toronto, ON, Canada, 318 pp. (2013)