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Analysis methodology and design verification for strengthening moment-resisting caisson foundations

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A R T I C L E I N F O

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A B S T R A C T

Many existing monopole towers rely on caisson foundations to resist the effects of large overturning moments. Increased loading on these towers, coupled with material deterioration and more stringent code requirements, often results in the overloading of the existing caissons. The strengthening of these caissons for additional overturning resistance presents many challenges and typically requires the installation of new piles. A pile cap is commonly used to connect the existing caisson to the new piles. The proper design of the pile cap has a critical importance in ensuring an effective stress transfer and creating a resilient foundation system. While a number of strengthening design approaches are repeatedly used in industry, there is a dearth of guidance and proven analysis methods to verify the performance achieved with these approaches. This study proposes a well-defined analysis methodology for the system-level structural design verification of the strengthened foundation systems. The methodology provides two routes, using either the strut-and-tie or nonlinear finite element analysis methods, to quantify the load, deformation, cracking, and failure responses. The methodology helps expose and address the weak aspects of the design using an iterative design improvement process. A common strengthening design approach from industry, involving a moment-resisting caisson strengthened with four concrete piles and a pile cap, is used as a case study to demonstrate the application of the methodology. The results demonstrate that the methodology provides significant insight into the system-level behavior both at the serviceability and ultimate limit states. The analysis results are found useful in identifying the locations where modified bar detailing could significantly improve the system performance. The improved design from the methodology provided a 56 % increase in the load capacity for the case-study foundation while still using the same amounts of reinforcing steel and concrete as compared to the initial design before the application of the proposed methodology. This paper presents the studies undertaken, conclusions reached, and recommendations made for developing resilient pile cap designs for strengthening moment-resisting caisson foundations.

1. Introduction

Monopoles typically rely on moment-resisting caisson foundations to resist the combined effects of large overturning moments and small shear and axial forces (Fig. 1). Increased loading (due to the installation of new components such as antennas, electrical equipment or power lines, or increase in the site-specific wind speeds), material deterioration over time, or the newer and more stringent design code requirements may result in the overloading of the existing monopoles. When compared to strengthening above ground components such as tower shafts, the strengthening of the caisson foundations pose a significant challenge. A part of this challenge comes from the fact that the strengthening design must satisfy the latest version of the applicable design standards [e.g., 1,2], which are generally more stringent than their previous editions in effect during the design of these caissons several decades earlier. Another challenge that needs attention is the material deterioration that takes place over time (Fig. 2).

Over the years, a number of strengthening design approaches have been used in industry (Fig. 3). They typically include the addition of new piles and a pile cap [3,4]. Discontinuous dowel bar, developed with epoxy-based adhesives, are commonly used to connect the new pile cap to the existing caisson. It is important to numerically demonstrate that the pile cap is capable of efficiently transferring stresses to the new piles, and ultimately to the soil, without exhibiting undesirable behaviors such as excessive cracking or deformations. There is, however, a lack of published guidelines and proven analysis methods to verify the system-level behavior achieved with these strengthening design approaches.

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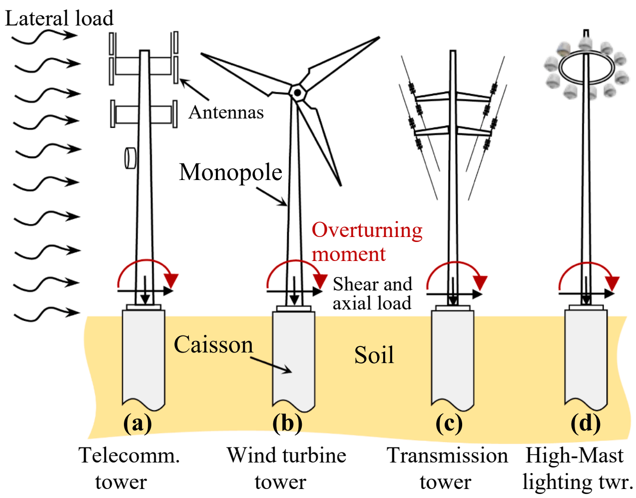


Fig. 1. Monopole towers subjected to large overturning moments.

Fig. 2. Visible signs of deterioration at the top of caissons.

2. Study objectives

The objective of this study is to develop an analysis methodology for the system-level structural design verification of strengthened foundation systems. The methodology employs a set of analysis methods, suitable for use in an engineering office environment, to predict the system response both at the serviceability and ultimate limit state conditions. The methodology is presented with a case study involving a moment-resisting caisson strengthened with a system of concrete piles and a pile cap. Four nonlinear finite element analysis and several strut-and-tie models are developed and tested, using an iterative approach, to reach a valid and efficient solution. The influences of the soil modeling, including the passive and bearing pressure contributions, are also examined. The methodology provides a general framework which can be applied to other types of moment resisting foundation systems strengthened with a pile cap and new piles.

3. Literature review

The literature investigating the behavior of strengthened tower foundations remains extremely limited. Abdalla [5] presented a case study involving self-supporting and guyed tower foundations subjected to axial and lateral loads, and proposed repair and strengthening solutions. However, no numerical analysis and verification studies were presented. Guner and Carrière [3] proposed an analysis and design methodology to increase the uplift capacities of existing caissons through the use of helical piles and reinforced concrete cap beams. Mittal and Samanta [4] investigated the failure of an under-reamed pile foundation supporting a transmission line tower and proposed a number of strengthening measures. These studies were focused on foundations subjected to axial and lateral loads with no major overturning moment effects. Schaffer et al. [6] proposed a retrofit solution using bolted connections for in-service offshore wind turbines with a monopile foundation subjected to wind and wave loadings. Chen et al. [7] investigated the strengthening mechanism of studs for embedded-ring foundation of wind turbine towers. None of these studies addresses the structural strengthening of moment-resisting caisson foundations.

4. Proposed analysis methodology

A flowchart of the proposed methodology is presented in Fig. 4. As a preliminary step, discrete geotechnical and structural analyses are performed to quantify the overloads. The geotechnical analysis is undertaken first to determine the soil capacity and calculate the acting shear and moment values along the caisson. Sectional analyses – nonlinear is preferred when possible – are then performed to determine the caisson

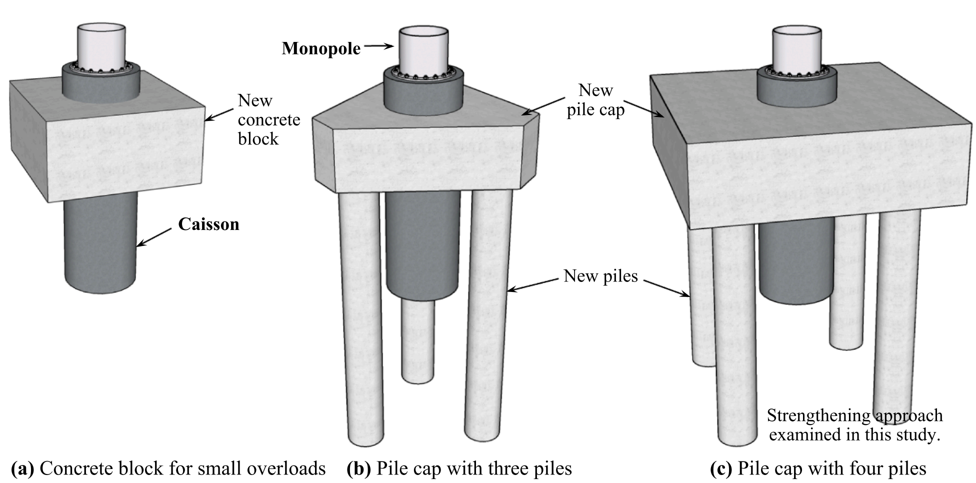


Fig. 3. Common strengthening design approaches for moment-resisting caissons.

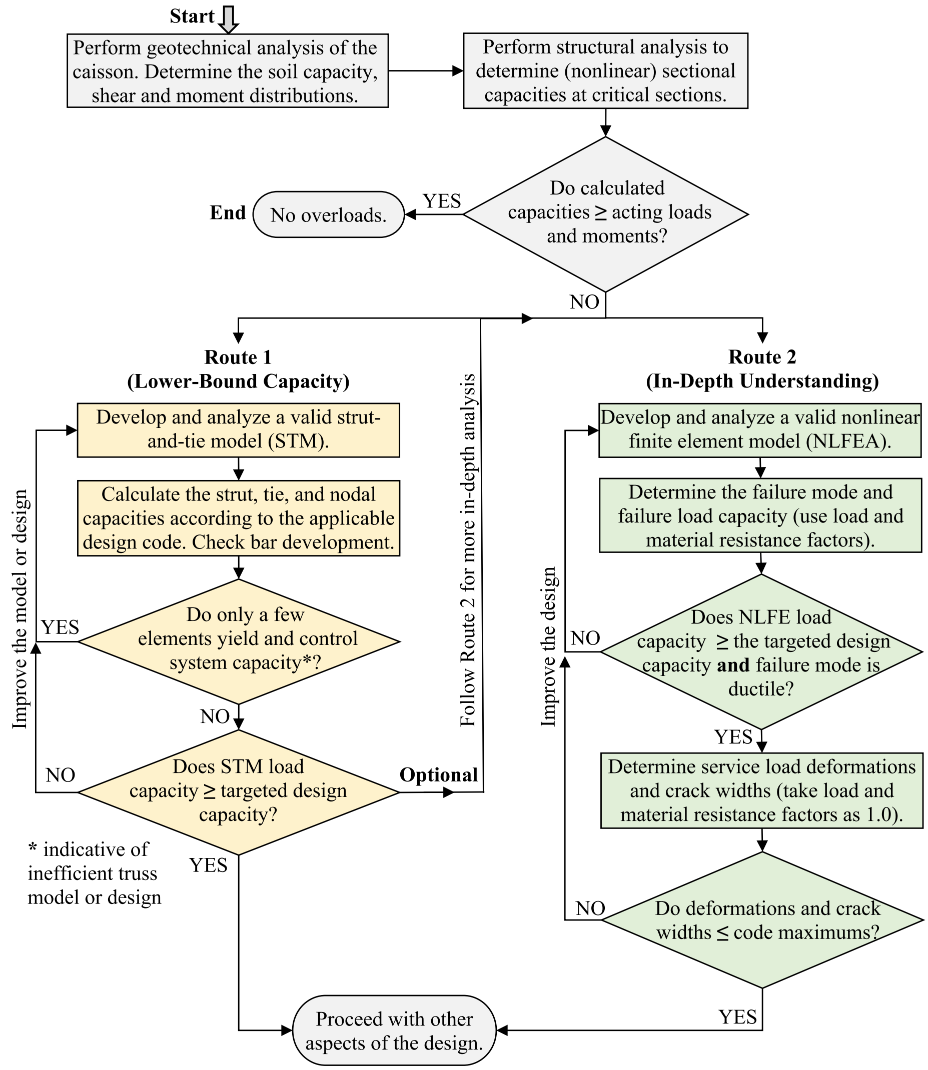
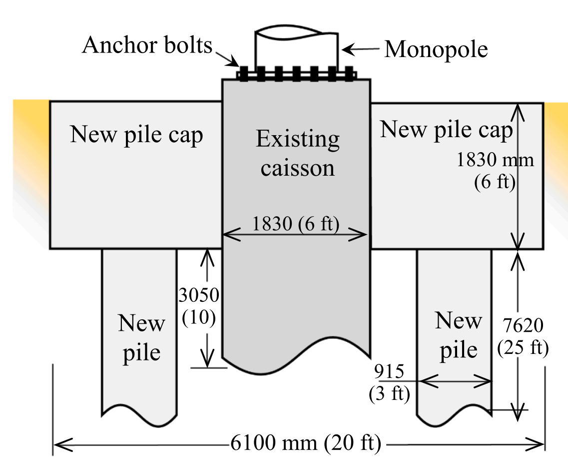


Fig. 4. Flowchart of the proposed methodology for the system-level design verification.

capacity at critical sections. These capacities are compared with the acting shear and moment values to determine the severity of the overloads. Any computer program with appropriate modeling capabilities could be used. Programs LPILE [8] and Response-2000 [9] were used in this study. Following this preliminary step, the methodology includes two routes. Route 1 uses the strut-and-tie method (STM) to provide a safe and lower-bound ultimate load capacity for the strengthened system, while Route 2 uses a nonlinear finite element analysis (NLFEA) to provide a more in-depth understanding of the system response, including the crack patterns, deformations, and failure behavior. Route 2 is also useful for the investigation of the service-load behavior or if Route 1 indicates potential problems including severe overloads. Performing both routes, as is done in this study, can verify the results obtained from each method.

5. Case-study strengthening design examined

A commonly used strengthening design was selected for the application of the proposed methodology. The existing caisson supports a telecommunication tower of 41 m (135 ft) (see Fig. 1a), located in the state of New York, and was originally designed to support factored tower reactions of 2,210 kNm (1,630 kip-ft) in overturning moment and 89 kN (20 kips) in shear. The strengthened caisson is required to resist the factored target design loads of 4,236 kNm (3,124 kip-ft) moment, 155 kN shear (35 kips), and 218 kN (49 kips) compression, based on the monopole analysis conducted according to TIA-222 [1]. The strengthening design includes the addition of four concrete piles which are connected to the existing caisson through a pile cap (Fig. 5). The reinforcing bar quantities (shown in Fig. 6) were taken from the actual strengthening design drawings developed for this site by a consulting



Refer to **Fig. 3c** for a perspective view.

Fig. 5. Elevation of the strengthening approach examined.

engineering firm, using typically industry practices. The pile cap is connected to the caisson using dowel bars developed with epoxy-based adhesives. The following sections present the studies undertaken for the application of the proposed methodology to this strengthening design.

6. Nonlinear finite element analysis (NLFEA)

The proposed methodology employs the STM before the creation of a more sophisticated NLFEA model as per Fig. 4. In this study, the NLFEA was undertaken first to develop a more in-depth understanding of the system response (including the cracking and failure behaviors), aid in the development the strut-and-tie model, and verify the results obtained.

A continuum-type, and high-fidelity modeling approach was used for the finite element modeling of the concrete foundation. The formulation is based on the Disturbed Stress Field Model (DSFM) [10], which is an extension of the Modified Compression Field Theory (MCFT) [11] – a theory adopted by several design standards [e.g., 12,13]. The DSFM employs a smeared, rotating crack approach within a total-load, secant-stiffness solution algorithm, and allows the consideration of the coupled flexure, axial, and shear effects. It examines the local stress conditions at cracks to ensure the reinforcing steel has the capacity to transmit the calculated tension across cracks. As well documented in the literature [e.g., 14], accurately modeling the constitutive response of reinforced concrete is a critical requirement for an accurate response simulation. In this study, the computer program VecTor2 [15] was used due to its extensive and experimentally validated constitutive models [e.g., 16,17]. Other specialized analysis programs, such as ATENA [18] and WCOMD [19], could also be used. The models used in this study are listed in Table 1. More information on these models is provided in Wong et al. [20].

A two-dimensional (2D) modeling approach is employed owing to the discrete wind directions according to the TIA-222 standard [1], which is further discussed in Section 6.3. While a 3D analysis may also be undertaken, it is important to ensure that the 3D analysis method

Table 1
Material constitutive models used.

Material Behaviour	Default Model
Concrete	
Compressive Base Curve	Hognestad
Compression Post-Peak	Modified Park-Kent
Compression Softening	Vecchio 1992
Tension Stiffening	Modified Bentz 2005
Tension Softening	Nonlinear (Hordijk)
Confined Strength	Kupler/Richart
Concrete Dilation	Variable-Isotropic
Cracking criterion	Mohr-Coulomb (Stress)
Crack Width Check	Agg/5 Max crack width
Crack Slip	Walraven
Reinforcing Bars	
Hysteretic Response	Seckin w/Bauchinger
Dowel Action	Tassios (Crack Slip)
Buckling	Akkaya et al. 2019

incorporates appropriate finite elements with concrete-specific constitutive models to be able to accurately capture the response of the reinforced concrete.

6.1. Modeling concrete and reinforcing steel

The concrete was modeled using 8-degrees-of-freedom quadrilateral elements (Fig. 6a). A plastic-offset-based nonlinear model was used to simulate the concrete material stress–strain response (Fig. 6c). This model includes hysteresis rules for the unloading and reloading conditions. Even in monotonic load analyses, some parts of the system will unload while some other parts reload as the concrete cracking and reinforcement yielding take place. The compressive strength ($f'c$) specified was 20.7 MPa (3 ksi) for the existing caisson and 27.6 MPa (4 ksi) for the new piles and pile cap. The cracking stress ($f't$) was calculated using the relationship $0.33\sqrt{f'_c}$ in MPa ($4\sqrt{f'_c}$ in psi) as the

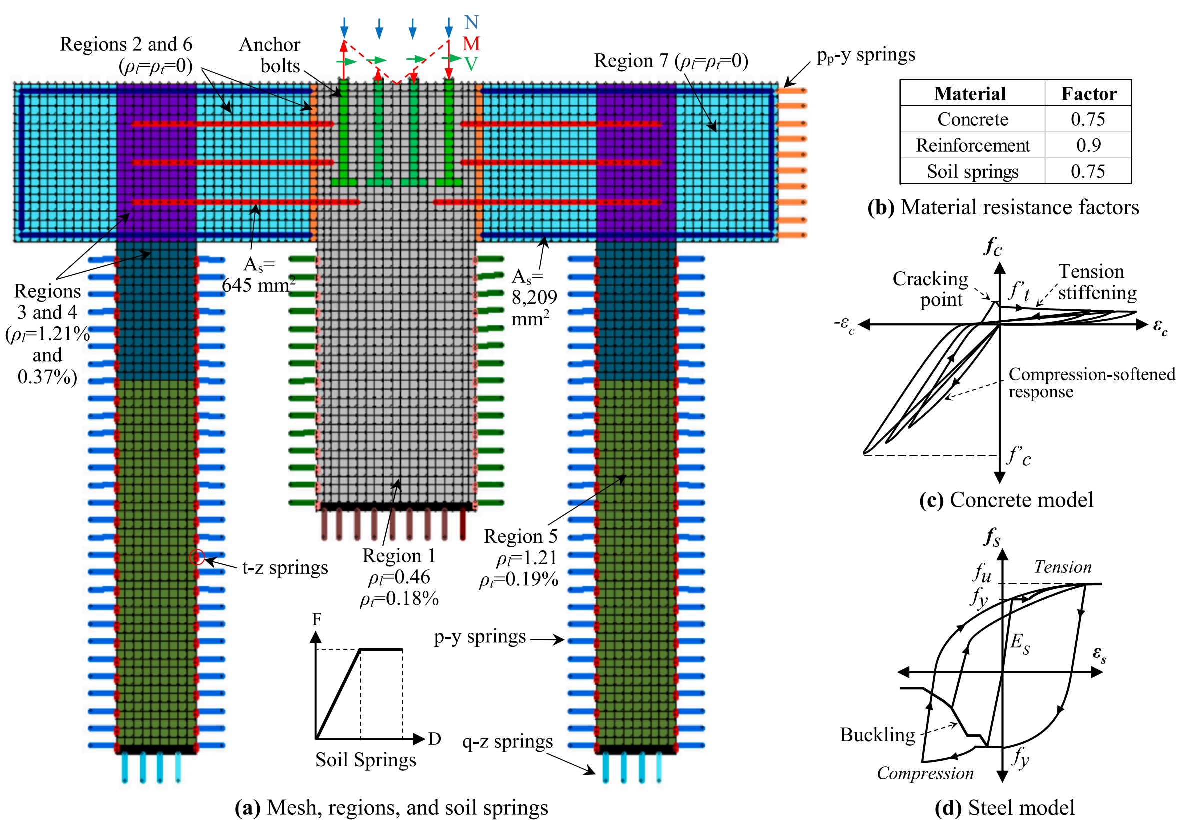


Fig. 6. Finite element model developed.

recommended lower-bound value.

The smeared reinforcement approach was used in the concrete regions with uniformly distributed reinforcement. The reinforcement percentages were calculated using Eq. (1) for longitudinal bars, Eq. (2) for hoops or spiral bars, Eq. (3) for members with rectangular sections, and Eq. (4) for members with circular sections. The calculated percentages are shown in Fig. 6a. The discrete reinforcement approach was used for the longitudinal and dowel reinforcement through two-node, two degree-of-freedom truss bar elements. The anchor bolts, which introduces the load into the existing caisson, were bundled into four bolt assemblages. A hysteretic constitutive model was used for both types of reinforcement (Fig. 6d). This model uses the hysteresis formulations of Seckin [21] in combination with the buckling formulations of the RDM model [22], which might occur for the piles under large compression forces at the ultimate limit state.

$$\rho(\%) = \frac{A_s}{A_{\mathrm{sec}}} \times 100 \tag{1}$$

$$\rho(\%) = \frac{4A_{st}(D_c - d_b)}{s \times D_c^2} \times 100 \tag{2}$$

$$\rho(\%) = \frac{2A_{st}}{s \times b} \times 100 \tag{3}$$

$$\rho(\%) = \frac{2A_{st}}{s \times D_c} \times 100 \tag{4}$$

where A_s is the total area of the longitudinal reinforcement; A_{sec} is the cross-sectional area of the concrete member; A_{st} is the area of the transverse reinforcing bar; d_b is the diameter of the transverse reinforcing bar; D_c is the diameter of the caisson or piles; and s is the transverse reinforcing bar spacing or spiral pitch. The yield stress (fy) specified for the reinforcing bars was 414 MPa (60 ksi). An ultimate stress (fu) of 620 MPa (90 ksi) was used which is the minimum value for 60-ksi deformed bars in ASTM A615-20 [23].

Seven concrete regions were created to accommodate different thicknesses, concrete, and smeared reinforced properties. For circular sections, the out-of-plane thickness was defined with an equivalent square area. A fine mesh size of 50 mm (2 in.) was used based on a mesh sensitivity study which tested mesh sizes from 100 mm (4 in.) to 40 mm (1.6 in.) to confirm that the results do not change by any significant amount when a finer mesh is used. Sharma and Guner [24] present more information on mesh sensitivity studies for pile caps.

6.2. Modeling soil

The subsurface conditions include a single layer of stiff clay and the underlying bedrock of weathered Dolostone. A nonlinear Winkler spring-based approach was employed to model the soil-foundation interaction behavior. The *p-y* springs were used to simulate the lateral soil behavior, where p represents the lateral soil reaction while y represents the lateral pile deflection. The spring parameters were derived using the site-specific geotechnical investigation report, based on the formulations of Reese et al. 1975 [25] and Welch and Reese 1972 [26]. A p-multiplier of 0.6 was used to consider the small spacing between the caisson and piles (see Figs. 5 and 7). This is a lower-bound value in the literature for clays [27]. The *q-z* and *t-z* springs were used to simulate the end bearing and skin friction behaviors, respectively, and derived based on the API [28] formulations – where q is the end bearing reaction; t is the skin friction reaction; and z is the vertical deflection of the pile. The passive pressure on the pile cap was modelled using p_p-y springs based on the formulations in Mokwa [29] which implicitly considers three dimensional and shape effects. A cohesion reduction factor of 0.5 was applied, as recommended by the site-specific geotechnical investigation report, to consider the frost depth. The developed spring parameters were further verified with the computer program GROUP [8]. It should be noted that the development of the spring parameters is highly dependent on the subsurface conditions and the foundation system details; thus, they should be determined for each specific site.

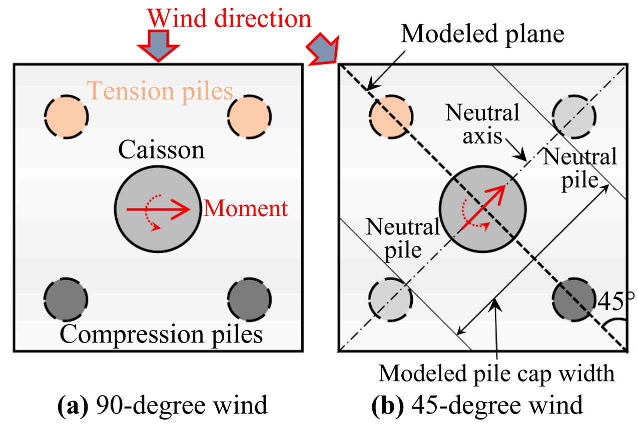


Fig. 7. Two critical loading directions.

The proper consideration of the soil response is essential for the accurate prediction of the system behavior, including the deformations at the serviceability limit state. As per the proposed methodology, Route 2 (see Fig. 4), a system-level modeling approach is undertaken using nonlinear finite elements for the reinforced concrete and nonlinear Winkler springs for the soil. This approach enables the structural design verification while considering the nonlinear behavior of the concrete in the presence of soil. The limitations of the Winkler springs, however, are well documented in the literature. It is recommended that a geotechnical finite element modeling study be separately undertaken to further examine the soil behavior, especially in the cases of more complex subsurface conditions, or cyclic or dynamic load applications.

Soil springs were modelled using two-node, two degree-of-freedom bar elements. A compression-only behavior was used for all springs except for the *t-z* springs which can resist both tension and compression. Most structural NLFEA programs, including VecTor2 [15], do not allow for a custom curve input for the springs; therefore, the calculated spring curves were simplified with an elastic–plastic behavior shown in Fig. 6a.

Three modified models were additionally created to assess the influences of the soil modeling on the system behavior. The first modified model employs the common structural modeling approach by using suitable supports (e.g., pin or fixed supports) at the element boundaries without explicitly modeling the soil. The second modified model removes the passive soil pressure springs to quantify its contribution to the system response. The third model additionally incorporates soil bearing springs under the pile cap, which is typically neglected when designing pile caps.

6.3. Load application strategies

Multiple load combinations and critical loading directions were considered using the site-specific wind speeds according to the TIA-222 standard [1]. The 45-degree wind direction, which puts two piles in tension and compression and two other piles on the neutral axis, is examined in this paper (Fig. 7b). The applied moment was resolved into force couples using a linear distribution (shown with red arrows in Fig. 6a), based on the plane-sections-remain-plane hypothesis [30–32]. The nonlinear analysis was performed with the monotonically increasing moment and associated shear force (shown in green) until the failure of the system. The dead load from the monopole (shown in blue) and self-weight of the concrete foundation elements (applied to each finite element—not shown) were also applied constantly throughout the analyses. The load factors (as per [33]) and material resistance factors (shown in Fig. 6b) were applied for the ultimate limit state analysis.

6.4. Results of the NLFEA from different modeling approaches

The moment-displacement curve for the NLFEA model, in comparison with the three modified modeling approaches, are presented in Fig. 8a. The results demonstrate that the strengthened foundation system exceeds the required target design capacity by 20 %. The structural modeling approach reached the target design capacity but provided a 20 % smaller capacity and a much larger system stiffness as compared to the NLFEA model with soil modeling (denoted as NLFEA in Fig. 8). The examination of the results from the structural modeling approach indicates that the concrete elements experienced larger strains and deformations, which resulted in more concrete cracking and a failure at an earlier load stage. The presence of soil (in the original NLFEA) allowed rigid body motions of the concrete members and resulted in a more uniform distribution of concrete strains. This conclusion agrees with those contained in Sharma and Guner [24], which found that in stiffer soils the discrepancy between the two approaches increases. Overall, the presence of soil provided a more realistic stress redistribution and stress flow through the pile cap, especially in the later, nonlinear stages of the response. The removal of passive soil pressure resulted in a 20 % reduction in the load capacity and a lower stiffness. Examination of the analysis results indicates that only half of the passive soil pressure resistance was utilized at the system failure (discussed below). If the system had failed at a later load stage, the contribution of the passive pressure might have been larger. Other studies found as high as 50 % passive pressure contribution to the lateral resistance of the pile caps [e.g., 34]. The consideration of soil bearing under the pile cap increased the system stiffness to a limited extent but did not result in any capacity increase. The soil bearing modeling was more effective in reducing the pile cap settlement (Fig. 8b).

The predicted failure mode of the system was progressive concrete and reinforcing bar failures at the interface between the existing caisson and new pile cap (Fig. 9). The existing caisson experienced a larger rotation than the new piles. This rotation activated the pile cap by exerting tensile and compressive stresses at the interface. The soil deformations played a major role in this failure by facilitating the rotation of the caisson. As the analysis progressed towards the failure load stage, increased soil deformations resulted in increased concrete cracking, and vice versa, creating a progressive failure mechanism. Some of the soil springs reached their yield point at the system failure, as schematically presented in Fig. 9. These results highlight the importance of considering the nonlinear behavior of both the reinforced concrete and soil through a holistic modeling approach.

Fig. 10 shows the predicted reinforcement stresses, which are in agreement with the crack patterns at failure. The vertical reinforcement inside the existing caisson was at its rupture strain as shown with green contours in Fig. 10a. The discontinuous dowel reinforcement that was used to connect the existing caisson to the new pile cap yielded at multiple locations and reached its rupture strain at one location as shown in Fig. 10b. In addition, the vertical reinforcement in the pile on the right experienced high stresses, indicating that the strengthening design is able to transfer the stresses to the new elements.

While the above results demonstrate that the strengthened system exceeds the targeted design capacity by 20 %, it is important to examine the response at the serviceability limit state. For this purpose, the analysis was repeated with the load and material strength reduction factors taken as 1.0. A maximum crack width of 0.5 mm (0.02 in.) was predicted at the interface between the existing caisson and the new pile cap (Fig. 11a). This value exceeds the reasonable crack width of 0.30 mm (0.01 in.) defined in *ACI* 224-01 [35] for the soil exposure condition. The yielding of the dowel reinforcement was also predicted at the same location (Fig. 11b). These results indicate the need to improve the design at the interface between the caisson and pile cap. In addition, a maximum crack width of 0.4 mm (0.015 in.) was predicted near the top face of the existing caisson at the outer, most tensioned, anchor bolt (Fig. 11a). This crack is contained in the existing caisson and should

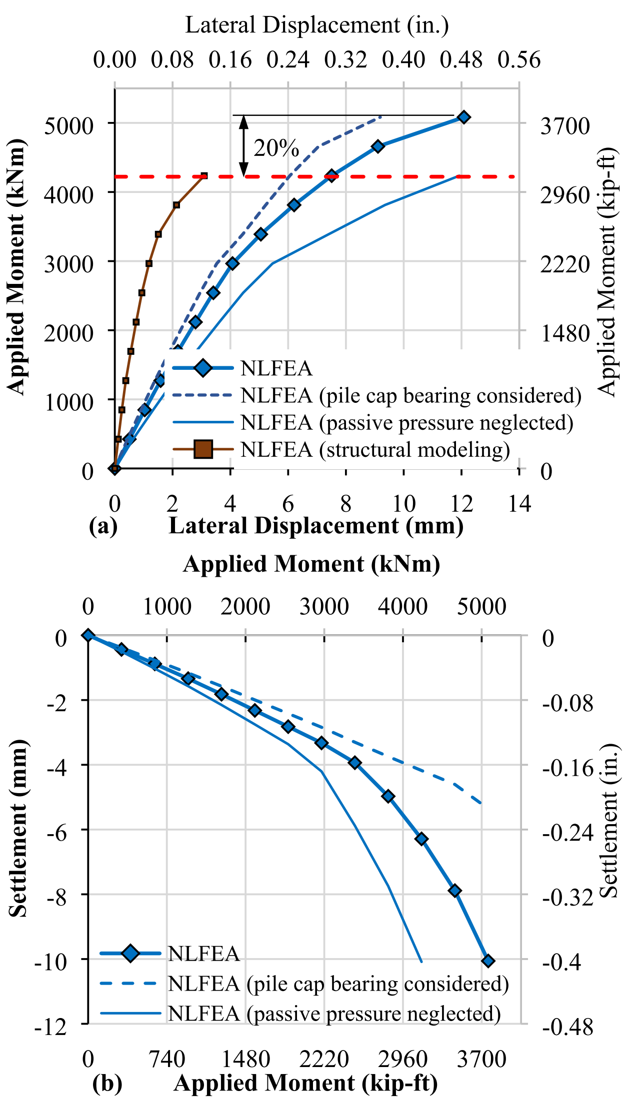


Fig. 8. Moment-displacement response predictions.

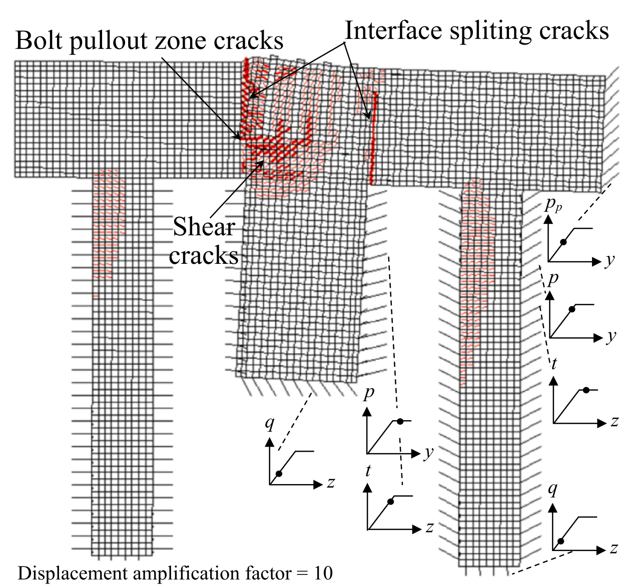


Fig. 9. Predicted crack patterns and deformations at system failure.

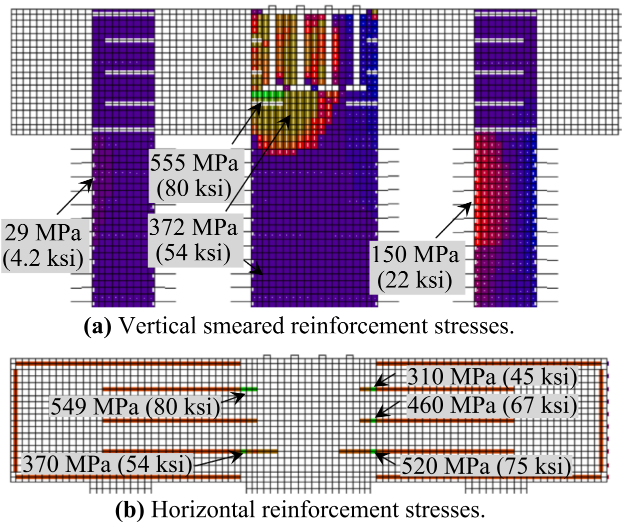


Fig. 10. Predicted reinforcement stresses at failure.

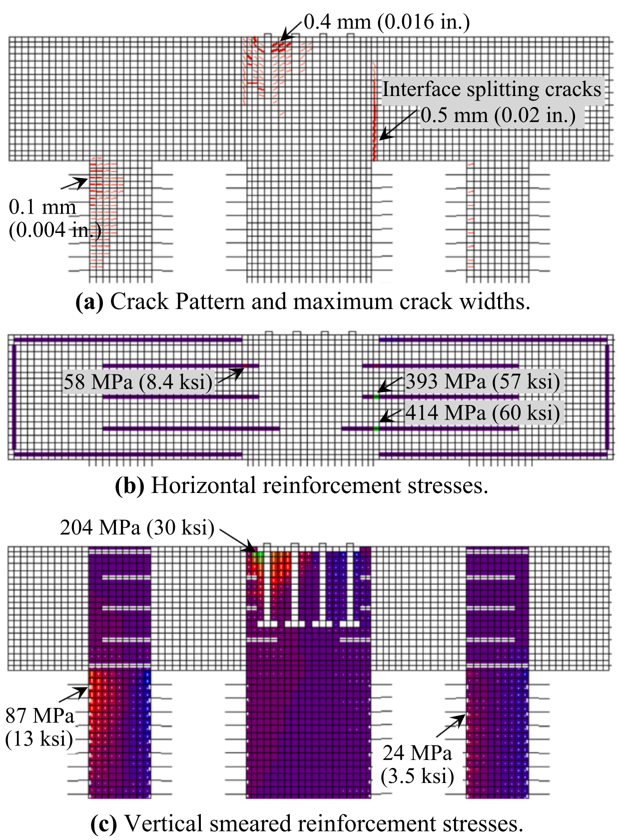


Fig. 11. Predicted serviceability limit state conditions.

have already occurred. This result indicates the need to perform a site investigation (using a visual inspection or non-destructive testing methods) to determine the actual crack widths and take remedial actions if found necessary. Cracking in this region is a common occurrence as shown in Fig. 2. The low stresses predicted for the vertical pile reinforcement indicate a satisfactory performance of the piles (Fig. 11c). The predicted vertical settlement of 2.9 mm (0.1 in.), measured at the top of the pile cap above the pile on the right, indicates a satisfactory performance.

These results demonstrate the critical benefit of the NLFEA for evaluating the service load performance which may expose the weak aspects of the design. The same strengthening design will be modelled with the STM in the next section. An improved design will be developed at the end of the paper.

7. Strut-and-Tie method (STM)

Most pile caps are deep members due to their geometry and position of the loads. Deep members exhibit a nonlinear strain distribution through their depths which invalidates the plane-sections-remain-plane hypothesis [30–32] – the basis of the slender beam or sectional analysis. In addition to the nonlinear finite element analysis (NLFEA) method, the strut-and-tie method (STM) is a valid method for analyzing deep members as recognized by design codes internationally [12,13,36,37].

The STM is a rational hand-calculation method that idealizes the cracked reinforced concrete by a truss mechanism [38–40], composed of struts, ties, and nodal zones (Fig. 12). The STM uses a plasticity-based lower-bound theorem and thus calculates a safe, low-bound load capacity for the concrete member provided that the member is sufficiently ductile [41–43]. The ductility is typically provided by sufficient amounts of grid reinforcement termed as crack control reinforcement. The STM assumes that cracked concrete carries no tension, and the steel reinforcement does not exhibit strain hardening. Both assumptions contribute to making this method more conservative than the NLFEA. The STM is used for determining the load capacity at the ultimate limit state. It does not calculate deformations, crack patterns, or crack widths, and thus is not suitable for the serviceability limit state analysis unless special techniques are employed.

Many design codes consider a region ‘deep’ if the ratio of the shear span (a) to the effective depth (d) is smaller than 2. A comparison of three analysis methods for various a/d ratios is presented in Fig. 13. The utilization is the ratio of the force (or demand) to the capacity (or strength) in percentages. If the utilization is less than 100 %, it indicates a reserve capacity; otherwise, it indicates an overload. For example, a utilization of 69 % indicates that the member has approximately 31 % reserve capacity. The sectional method is not valid and should not be used for the analysis of deep members. It is included in Fig. 13 to show the inaccuracies that may be obtained from this method. As the elements becomes deeper (see the region where a/d is less than 2.0), the capacity predictions from the sectional analysis method (used for slender members) and the STM significantly diverge. As expected, the capacity predictions converge for slender members with a/d ratios larger than 3.0 for which the sectional method is valid. While the NLFEA consistently predicts larger capacities, the STM converges with the NLFEA predictions as the member becomes deeper.

A statistical study was conducted to determine the ranges of a/d ratios used in industry for the type of strengthening design examined in this study. The commonly used combinations of design parameters (shown with A, B, C, D, D_c, and D_p in Fig. 14) were obtained from a consulting engineering firm. The foundations were divided into two regions for the purpose of calculating the a/d ratios. It was found that

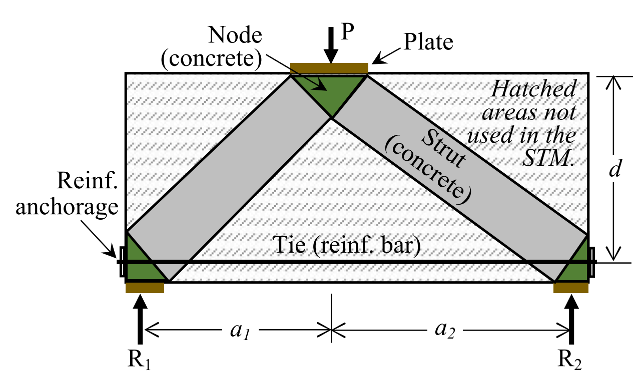


Fig. 12. Idealization of the struts, ties, and nodes.

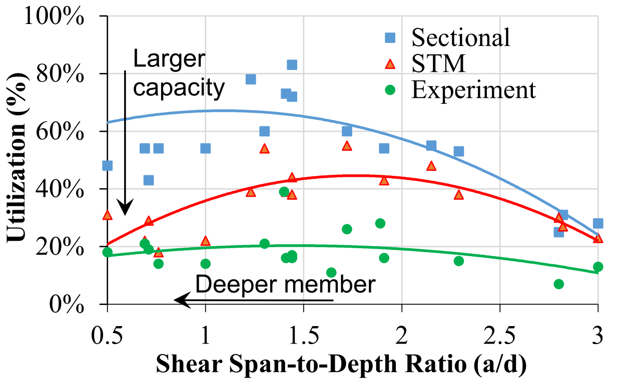


Fig. 13. Shear capacity-to-demand (utilization) predictions from three methods [44].

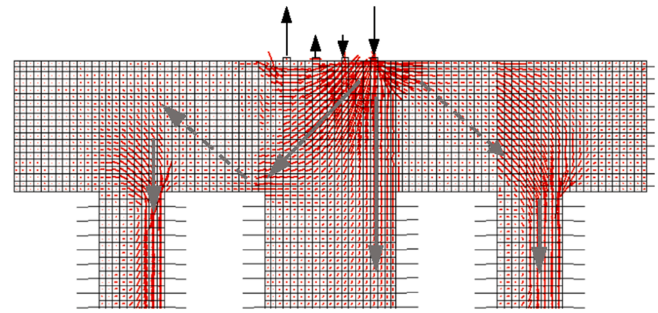


Fig. 15. Compressive strain trajectories from the NLFEA.

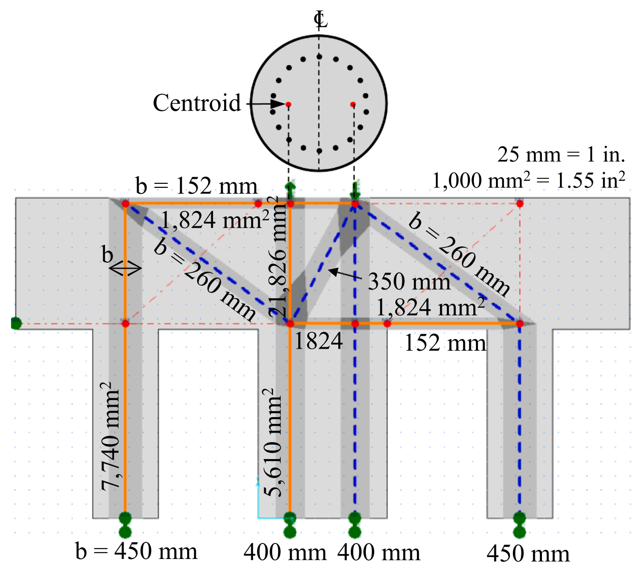


Fig. 16. Strut-and-tie model of the case-study foundation.

the *a/d* ratios were 0.33 and 1.08 for the minimum value of the parameters and 0.83 and 1.64 for the maximum value of the parameters for Regions 1 and 2, respectively. All of the ratios correspond to notably deep members. For the case-study foundation, the *a/d* ratios are 0.52 and 1.04 for Regions 1 and 2, respectively.

Developing a valid and efficient strut-and-tie model for foundation systems strengthened for moment resistance is a challenging task because these systems typically transmit tension through concrete, as confirmed by the NLFEA results. The STM, on the other hand, carries tension only through the ties whose locations are pre-set by the location of the reinforcing bars. When developing a valid and efficient strut-and-tie model, a critical task is to determine the locations of the struts. The compressive strain trajectories obtained from the NLFEA, if this analysis was performed previously, may provide some help (Fig. 15). Reference [43] provides in depth information on how to develop a valid and efficient STM.

After a suitable shape of the STM has been determined using the compressive strain trajectories, the exact locations of the center points of all nodes were determined iteratively with the help of the computer program CAST, a general-purpose strut-and-tie modeling method used for the analysis and design of disturbed regions and deep beams [45]. The final model created is shown in Fig. 16. The amount of reinforcing steel area in each tie was determined by the number of bars encased by the width of the tie (*b*). For the case-study foundation, the vertical tie inside the existing caisson bundles half of the caisson's longitudinal reinforcement, resulting in the tie areas shown in Fig. 16. Only the continuous top and bottom mesh reinforcement of the pile cap closest to the existing caisson (shown in red in Fig. 17) was considered for the horizontal ties. The moment load was applied as a force-couple at the geometrical centroid of the anchor bolts, as shown with green arrows in Fig. 16. The strut, tie, and nodal zone capacities were calculated according to ACI318-19 [36].

The results of the analysis are presented in Fig. 18 in terms of the utilization ratios, *URs* (i.e., demand/capacity), for the struts, ties, and nodal zones. The results indicate that the tie at the bottom of the pile cap

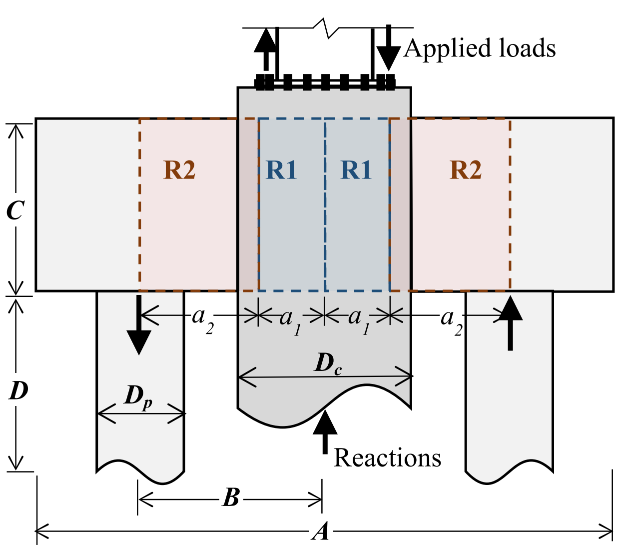


Fig. 14. Ranges of geometric variables examined.

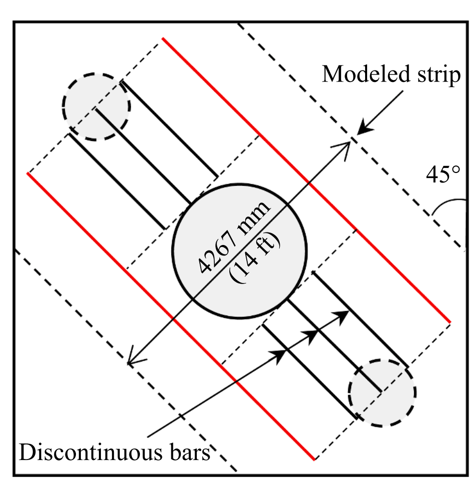


Fig. 17. Horizontal tie reinforcement in the STM.

governs the ultimate capacity. This result agrees with the results of the NLFEA which predicted large crack widths and reinforcement stresses at this location and required an improved design. The tie at the top is the second most stressed element with the UR value of 0.77. The low UR values in the struts, with the maximum value of 0.44, indicate that this is a tension governed system; therefore, a careful reinforcing bar design is required. As expected, the STM predicted the system capacity to be notably lower than the capacity predicted by the NLFEA for the reasons discussed previously (Fig. 19).

8. Recommended design improvements

Benefiting from the understanding gained from the analyses, an improved design was developed by modifying the reinforcement design (see Fig. 20) with no changes in the system dimensions. The analysis results demonstrate that reinforcement layers 1 and 5 (shown in Fig. 20a) are the main elements responsible for transferring stresses to the new piles. These layers, however, include discontinuous pile cap reinforcement which does not pass through the existing caisson and does not contribute to the tie capacity. Such discontinuous bars are commonly employed in practice due to the challenges in drilling through the existing caissons, which has a dense reinforcing bar and anchor bolt cage at this location. There is also the risk of damaging an existing bar during drilling – recall that the NLFEA results showed that the vertical bars reach their ultimate capacities at failure and thus are critical components of the system. The improved design still uses discontinuous bars but provides additional hoop bars for the stress transfer. These additional hoops may be comprised of two half-circle bars, connected using mechanical couplers. Butt welding may also be used. The improved design uses diagonally oriented main reinforcement (layers 1 and 5) and an orthogonally oriented mesh of crack control reinforcement at the top and bottom of the pile cap (Fig. 20). While diagonal main reinforcement facilitates an effective stress transfer mechanism and reduces the total amount of reinforcement, an orthogonal orientation may still be used if found more advantageous from other perspectives such as constructability.

Three layers of epoxied dowel reinforcement (e.g., layers 3–5) are also used in conjunction with the corresponding hoop bars. These dowel bars will help control cracking while the corresponding hoops provide confinement to the existing caisson at a location subjected to large and complex stresses (see Figs. 10a and 18). A critical aspect of designing deep members is the anchorage of the reinforcement. The tie reinforcement must be developed at nodal points, or at the intersection with a strut or a compressive bearing area, to be able to carry the calculated tension. A combination of hooked bars and sufficient development length is used in the final design. Not meeting this requirement may render the deep beam action ineffective and reduce the capacities towards those obtained from the sectional method (Fig. 13).

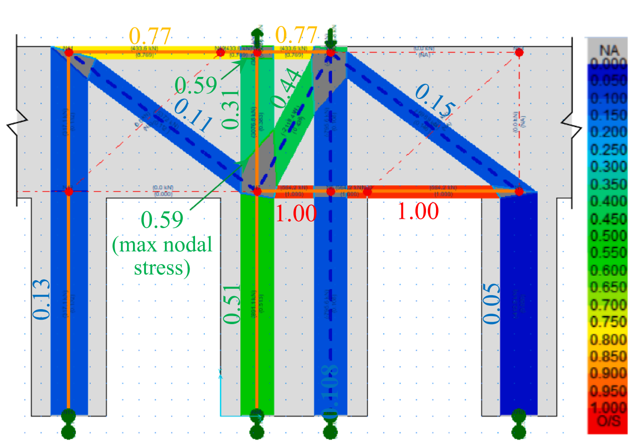


Fig. 18. Utilization ratios (URs) of the STM elements.

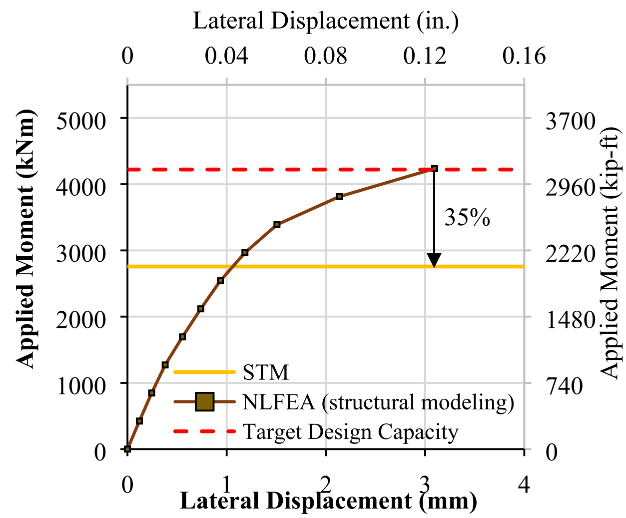


Fig. 19. Comparison of the capacity predictions.

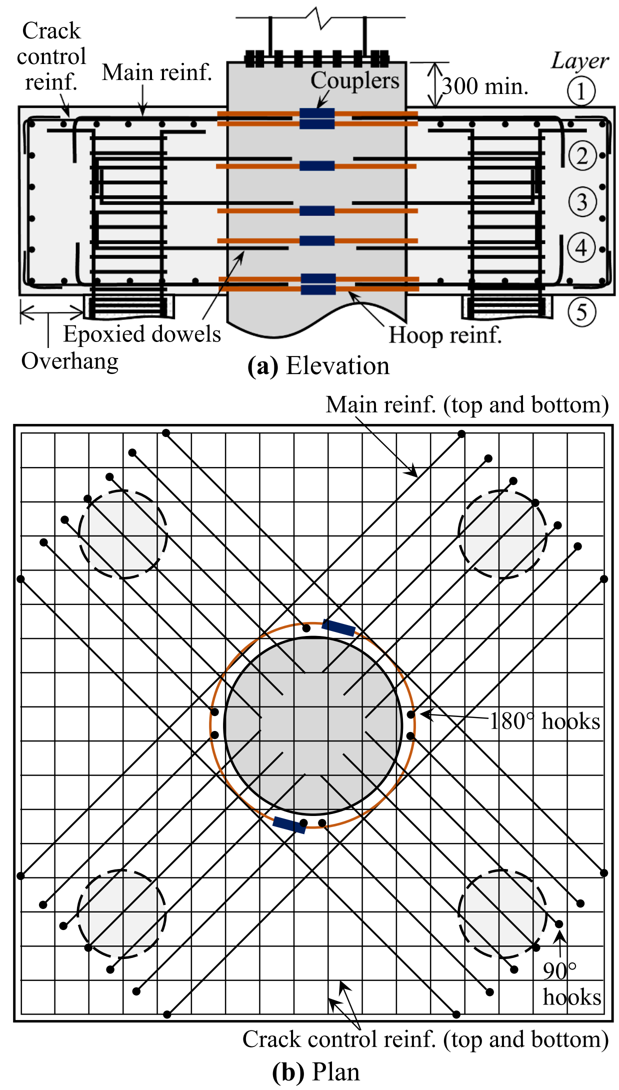


Fig. 20. Schematics of the improved design.

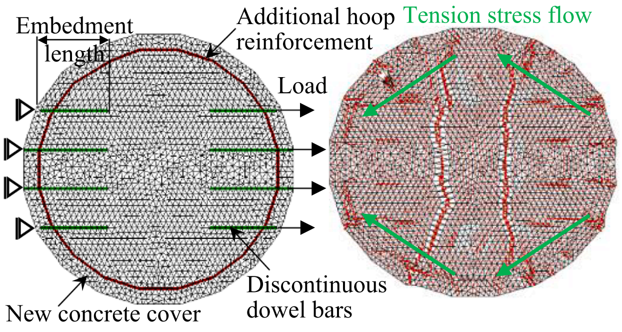


Fig. 21. Plan view of a caisson [3].

The concept of using discontinuous dowel bars in conjunction with hoop reinforcement was previously shown to be effective in transferring tensile stresses from one side of the caisson to the other (Fig. 21). The same study also highlighted the critical importance of providing sufficient dowel bar embedment depth inside the existing caisson. Refer to Guner and Carrière [3] for more details.

9. STM analysis results for the improved design

In accordance with the iterative design process, the STM was revised to account for the changes in the reinforcement design. The results indicate that the tie at the bottom of the existing caisson (i.e., the hoop reinforcement) controls the capacity (Fig. 22). Other elements of the model also experience high stresses, which shows better utilization of the system components and thus a more efficient design. The ultimate load capacity of the improved design was found to be 56 % above the capacity calculated for the original design (Fig. 23) while using approximately the same amounts of concrete and reinforcing steel. These results demonstrate the value of the insight gained from an iterative design-modeling process which could lead to significant performance improvements.

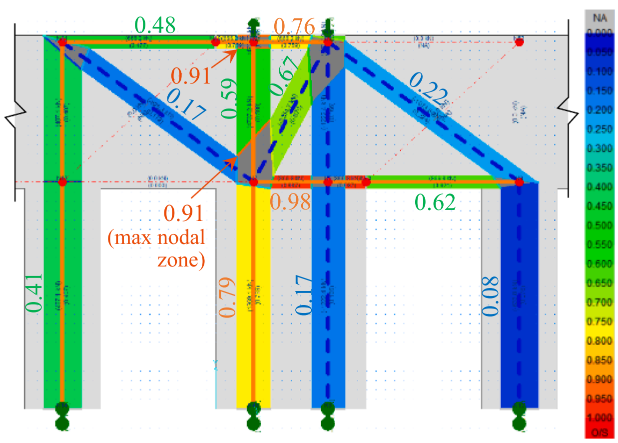


Fig. 22. Utilization ratios of the STM elements (improved design).

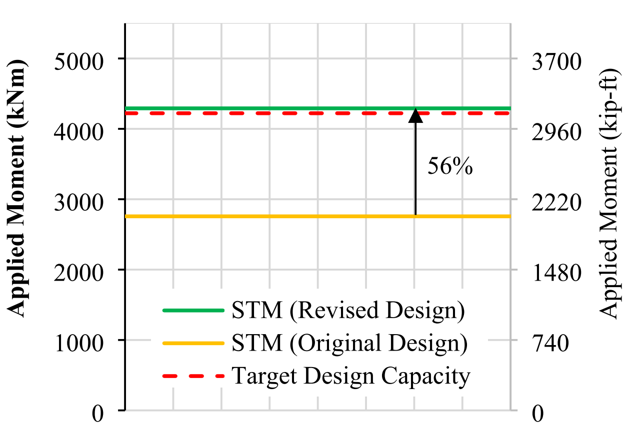


Fig. 23. Comparison of the capacity predictions from the STM.

10. Summary and conclusions

An analysis methodology is proposed for the system-level structural design verification of strengthened foundation systems. The methodology provides two routes using the strut-and-tie method (STM) and nonlinear finite element analysis (NLFEA). The STM is used to obtain a lower-bound ultimate load capacity prediction and develop an understanding of the stress flow characteristics and weaker design components. The NLFEA is used to develop a more in-depth understanding of the system response (including the crack patterns and deformations at the serviceability limit state), verify the results obtained from the STM, and investigate the influence of the soil modeling.

A common strengthening design from industry was used as a case study to demonstrate the application of the proposed methodology and reach an improved strengthening design. Four NLFEA and several strut-and-tie models were developed and tested to achieve a valid and efficient solution. The final models presented could be adopted by engineers for analyzing foundation systems of a similar nature.

The ultimate load capacity of the case-study foundation was found to achieve the target design capacity using the NLFEA. The STM calculated a load capacity 35 % below the target capacity owing to the conservative nature of this method. The NLFEA also provides the capability of investigating the serviceability behavior, including the prediction of deformations and crack widths. Comparing these values with the limits contained in the design guidelines provides additional metrics for assessing the system performance.

The NLFEA with the structural modeling strategy predicted 20 % smaller load capacity and significantly stiffer response as compared to the NLFEA with soil modeling. It was found that the inclusion of the soil allowed rigid body motions and resulted in a more realistic distribution of concrete strains. It was also found that the inclusion of the passive soil pressure contributed 20 % to the load capacity and increased the foundation stiffness noticeably. The inclusion of the soil bearing under the pile cap also increased the foundation stiffness but to a smaller extent and with no additional capacity gain for the case study foundation examined.

The results demonstrate the critical importance of undertaking a system-level analysis and design improvement process in achieving an effective strengthening design. In pursuit of this goal, the proposed methodology provides significant insight into the system-level response, including the highly stressed regions, crack patterns, deformations, failure loads, and failure modes. The analysis results were found useful in identifying the locations where modified bar detailing or increased bar quantities could lead to significant performance improvements while the bar quantities in other locations may be reduced without effecting the system response. The STM was found particularly useful in rapidly creating and analyzing modified models. The improved design from the methodology provided a 56 % increase in the load capacity for the case-study foundation while still using the same amounts of reinforcing steel and concrete. These results indicate that the methodology, when employed during the design stage in an iterative manner, could facilitate better reinforcement detailing.

The proper consideration of the soil response is essential for the accurate prediction of the system behavior, including the deformations at the serviceability limit state. The nonlinear Winkler spring approach provides a simple and effective means for the inclusion of the soil response into a high-fidelity structural finite element analysis model. This approach enables the structural design of the pile cap while considering the nonlinear behavior of the concrete in interaction with the soil. The limitations of the Winkler springs, however, are well documented in the literature. It is recommended that a geotechnical finite element modeling study be separately undertaken to further examine the soil behavior, especially in the cases of more complex subsurface conditions, or cyclic or dynamic load applications.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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