3-D FINITE ELEMENT ANALYSIS OF PILE-TO-PILE CAP CONNECTIONS SUBJECTED TO SEISMIC ACTION

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Abstract

Damage in recent earthquakes has resulted in the design of pile foundation systems becoming more conservative, particularly pile-to-pile cap connections. Ground movement has led to plastic hinges forming in the piles near their connection to the pile cap. Application of current international design practice results in pile cap joint details having congested steel reinforcement in the pile cap and this is extremely difficult to construct.

This paper reports the results from a three-dimensional finite element model that has been utilised to analyse the nonlinear behaviour of pile-to-pile cap connections. Two models of pile-to-pile cap connections have been developed, namely: plain embedment; and headed embedment and both models have been simulated under monotonically lateral loads. Results indicate that the use of a suitable headed embedment significantly improve the seismic behaviour. Based on the analysis, it is recommended that using 1.2-1.5 of pile width (or diameter) for embedment length of headed reinforcement produces adequate connections that have reduced stress concentration and crack damage at the interface between pile and pile cap.

1. Introduction

Earthquakes that have occurred globally over the last two decades have resulted in an increased expectation of acceptable performance and damage control for different structures during seismic events. Catastrophic failures of piled foundation systems in the recent earthquakes of Loma Prieta, Northridge (Nogami 1987), and Kobe (Building Research Institute 1996) have led to considerable effort being directed towards safer civil infrastructure particularly in the seismic zones. However, repair of damaged piles in high-rise building systems is impractical because of the expensive cost and difficulty associated with ground excavation.

This paper reports a study focusing on two types of connections, i. e., plain embedment and headed embedment, to improve the capacity of the connection. In the plain embedment (without treatment), the prestressed concrete pile is simply embedded in the cast-in-place pile cap. In the headed embedment (with treatment), pile strands confined with round reinforcement are exposed and embedded in the cast-in-place pile cap. In this paper, two pile units using plain embedment model which were tested by Harries and Petrou (2001), are reviewed and the validity of their hypotheses is rigorously investigated through comparison of the observed and predicted nonlinear behaviour. Teguh et al. (2004) have shown that numerical predictions for the load-displacement response of pile-to-pile connections, using finite-element methods (i. e. static pushover and time history analyses) and the Modified Compression Field Theory (Bentz 2001), provided reasonable agreement compared to experimental results. To understand the nonlinear behaviour of pile-to-pile cap connections subjected to seismic actions, refined joint models for both pile units were developed and are presented in this paper. These proposed models were analysed using the DIANA finite element analysis package.

2. Design Review of Pile-to-pile Cap Connections

An ideal pile-to-pile cap connection dissipates seismic energy by ductile yielding in plastic hinge regions of the structure above the foundations or via mechanical energy dissipating devices placed between the foundations and the structure (Park et al. 1984). In practice, earthquake loads in the form of overturning bending moments acting on high-rise building cores are translated into tension/compression loads in piles under the pile cap and each pile is checked individually for shear acting in combination with axial pile loads. However, most pile caps are designed assuming pile caps are rigid with heads fixed into the pile cap.

Current engineering practice designs pile to pile cap connections based on embedded longitudinal reinforcement and confinement reinforcement rely on careful selection and detailing around zones of potential inelastic deformation. Other regions are designed on an appropriate strength method using elastic response techniques. In line with capacity design principles, the ideal approach for the design of buildings or bridges would be for piles to remain elastic during a seismic event because of difficulties associated with inspection and repair of subsurface foundation components after an earthquake. However, under moderate conditions, this design approach may be impractical due to the potential for inelastic deformations of the piles; either in the connection region or in subgrade regions. In addition, loads and deformations imposed on the piles of a pile group may lead to unpredictable inelastic response of piles at the connections to the pile cap and/or below ground level. Consequently, a simplified approach must be developed which addresses the problem but remains analytically and numerically tractable.

3. Experimental Model

Harries and Petrou (2001) conducted cyclic lateral loading tests on two full scale pile-to-pile cap connections. The major test variable was pile embedment length in the pile cap; however the pile was embedded in the pile cap without treatment (plain embedment model). The effect of confined headed reinforcement at the connection was not investigated in the experimental tests.

Unit test variables from Harries and Petrou (2001) are listed in Table 1 and the mechanical properties of concrete and reinforcing bars are detailed in Tables 2 and 3, respectively. Each pile unit was tested under a constant axial load of approximately 0.1 $A_g f_c$' (890 kN) and a reverse cyclic load of like magnitude. Effects of secondary moment on the column were excluded. Pile embedment lengths for units P1 and P2 were 1.0 and 1.3 of pile width respectively.

Table 1: Unit test variables of pile-to-pile cap connections

Pile unit number	Unit P1	Unit P2
Initial prestressing steel (constant)	1.394×10^9 N/m^2	1.394×10^9 N/m^2
Applied axial load (increment): assumed as pressure load over the pile surface	4.260×10^6 N/m^2	4.260×10^6 N/m^2
Maximum lateral loads (monotonic & cyclic)	1.112×10^5 N	9.600×10^4 N
Pile reinforcement: 1. Low relaxation strand tendon 2. Wire plain spiral Pile cap reinforcement: 1. Main 2. Transverse	 12.5 mm (1/2”) 7 mm (0.276”), R12 22 mm (0.875”), D22 D10 - 152 mm	 12.5 mm (1/2”) 7 mm (0.276”), R12 22 mm (0.875”), D22 D10 - 152 mm
Pile dimension (B)	0.45 x 0.45 m	0.45 x 45 m
Pile cap dimension	2.14 x 0.92 x 2.14 m	2.14 x 0.92 x 2.14 m
Pile embedment length	0.610 m (1.3 B)	0.450 m (B)

Table 2. Mechanical properties of concrete

Pile unit number	Compressive strength (MPa)		Tensile strength (MPa)		Modulus of elasticity (MPa)	
	Pile	Pile cap	Pile	Pile cap	Pile	Pile cap
Unit P1	46.2	34.5	4.62	3.45	3.215×10^4	2.778×10^4
Unit P2	46.2	20.7	4.62	2.07	3.215×10^4	2.152×10^4

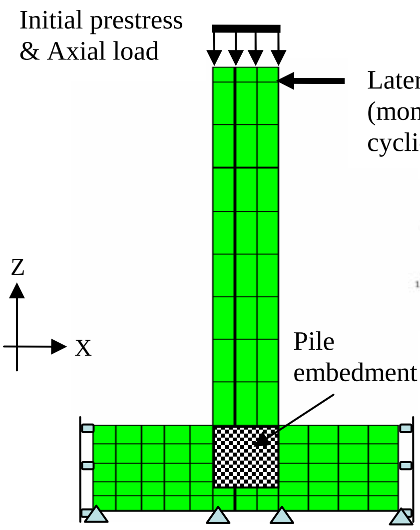
Table 3. Mechanical properties of reinforcing bars

Reinforcement	Yield stress (MPa)	Tensile strength (MPa)	Modulus of elasticity (MPa)
Tendon	1791	1882	1.94×10^5
Wire	448	630	1.86×10^5
D22	275	464	2.00×10^5
D12	275	464	2.00×10^5

4. Nonlinear Analysis by 3D Finite Element Method

4.1 Finite Element (FE) Discretisation of Model

A single prestressed concrete pile connected to a cast-in place pile cap was modelled with discrete 3-D mesh soil brick elements to match the boundary conditions and geometry of the tested pile units. The pile and pile cap were modelled by a twenty-node isoparametric solid brick element (HE20 CHX60) and the analysis based on quadratic interpolation and Gauss integration. Longitudinal and transverse bars in the pile cap, plain spiral reinforcement and tendon in the pile were modelled by the discrete truss element to include an interaction between reinforcing bar and concrete. This interaction is modelled by introducing interface and/or linkage element(s) in the interface between truss element and concrete element. All other reinforcing bars and tendons are modelled by an embedded element, assuming perfect bond. The finite element discretisation, boundary conditions, and mesh type are shown in Figures 1 and 2.



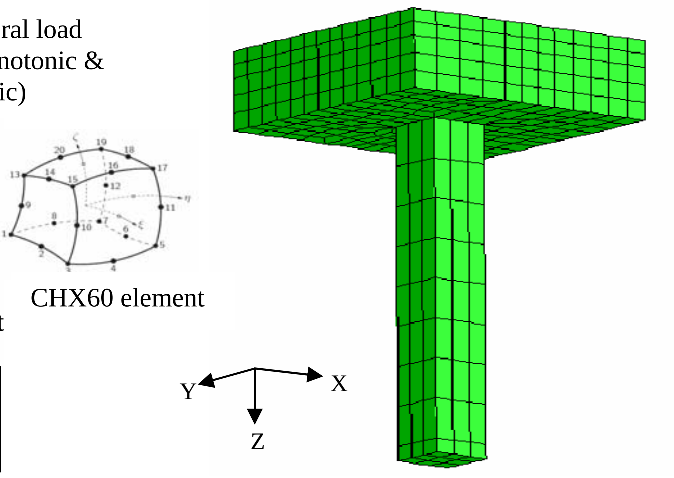


Figure 1. F.E. model boundary conditions and loading.   Figure 2. Model discretisation.

In the finite element method, reinforcement may be modelled by one of two methods. The first method, which is less computationally demanding, involves the use of embedded or smeared reinforcement. The second method, more computationally expensive, involves separate discrete modelling of the reinforcement. The second model allows for the investigation of bond-slip behaviour of reinforcement with respect to the surrounding concrete. This technique becomes computationally expensive when carried out over the entire system.

4.2. Constitutive Material Models

The constitutive relationships used for the finite element models are shown in Figure 3. The non-prestressed and prestressed reinforcements use Von Mises yield hardening criteria with constitutive models matching the behaviour determined from testing (Figure 3d). Concrete is modelled using Von Mises yield criteria for compression a tension cut-off from the concrete compressive strength, fc', to the concrete tensile strength, ft' (Figure 3a) for tension. The concrete compressive behaviour models the behaviour obtained from displacement controlled testing of concrete cylinders (Figure 3c). The Hordijk model shown in Figure 3b was used for the concrete tensile behaviour (de Witte and Kikstra 2002). It consists of elastic response to the tensile capacity followed by a nonlinear unloading branch. Cracking is modelled using both multiple fixed cracks and rotating crack formulations. The results presented in this paper are limited to the fixed crack model.

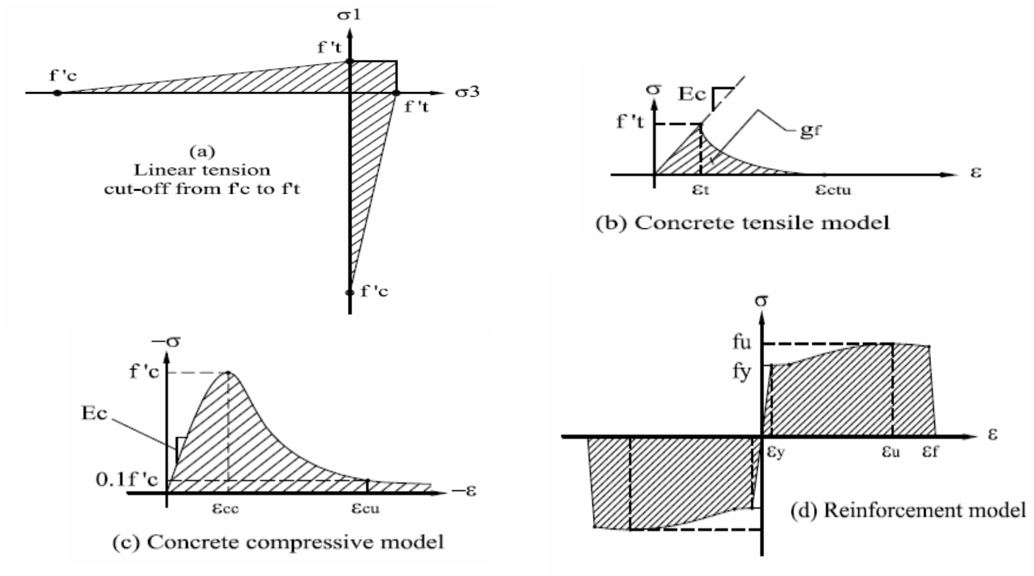


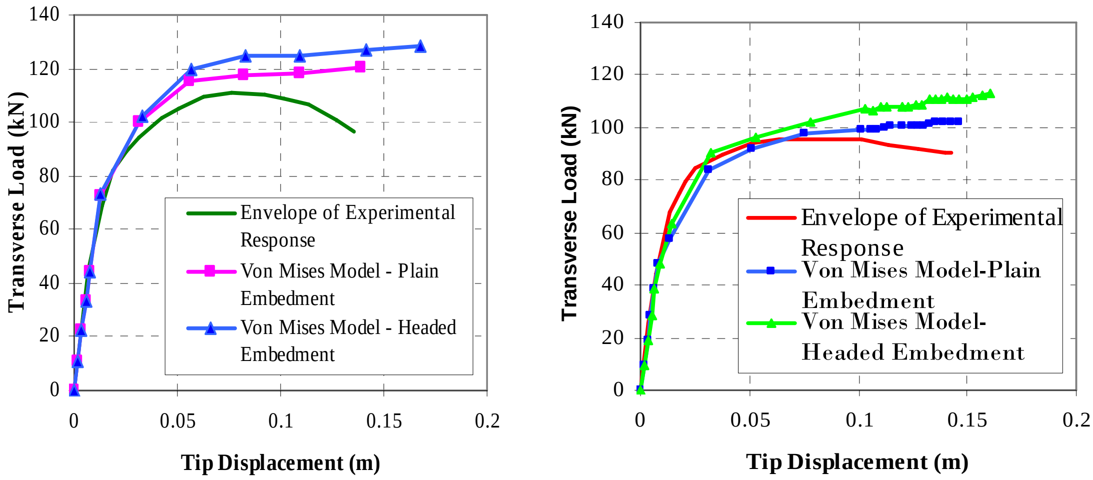
Figure 3. Constitutive material models for concrete and reinforcement

5. Results and discussion

The finite element results were obtained from analyses conducted using the program DIANA 8.1 (de Witte and Kikstra 2002). In this study, Modified Newton Raphson incremental-iterative method with tolerance for convergence of 0.0001 was used. The comparison of the load-displacement response is shown in Figure 4 and the finite element results envelope the experimental findings. As discussed earlier, the experimental investigations were performed cyclically while the finite element analyses are static pushover (monotonic). The lateral loads were applied at the point of zero moment or at the pile tip while static axial loads consist of static axial load of 0.1 fc'A_g (890 kN) and initial prestress of 1.394 kN/mm^2. The pile embedment length was varied from 1.0 to 1.5 of pile width. As seen from Figure 4a, a longer embedment length develops the flexural capacity of the pile without distress to the pile cap.

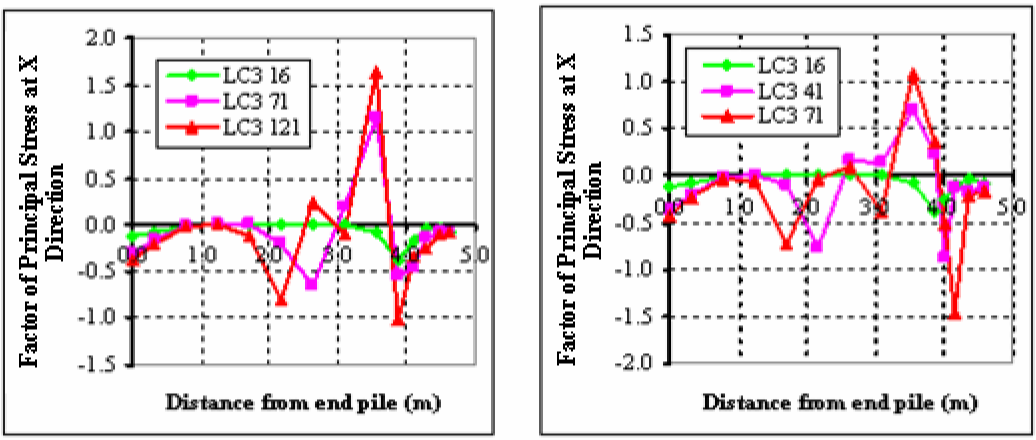
When concrete systems are subjected to cyclic loading the entire system undergoes tension-compression reversals. As a result, cracks open and close leading to a greater rate of system stiffness degradation than that observed under monotonic loading. Load reversal requires more sophisticated constitutive modelling and this issue is being investigated to better represent pile-to-pile cap connections.

A comparison of experimental and analytical results shows that the multi-directional fixed crack formulation provides a good prediction of crack formation in a three-dimensional model. Figure 6 illustrates the crack behaviour at the pile head and pile-to-pile cap joint. The joint cracking pattern observed in the experiment is captured by the analytical model. Furthermore, the crack strain is largest along the pile embedment crack, as observed in the experimental investigation. It is observed that regions of high tension occur close to the pile cap longitudinal faces. These regions of high transverse tension indicate potential locations for plastic hinges and principle stress concentrations occurred in the joint region of the pile-to-pile cap connection (Figure 5); this behaviour was observed in both pile units. Figure 5 shows the principal stress distribution along pile height at initial, middle, and final steps of monotonically increasing transverse loading (load case 3) indicated as LC3 16, LC3 71, LC3 121 for unit P1 and LC3 16, LC3 41, LC3 71 for unit P2 respectively. From these results, it is inferred that 3-D modelling is an important tool for further understanding and accurate modelling of the pile-to-pile cap connections.



a. Unit P1 b. Unit P2

Figure 4. Comparative load-displacement response

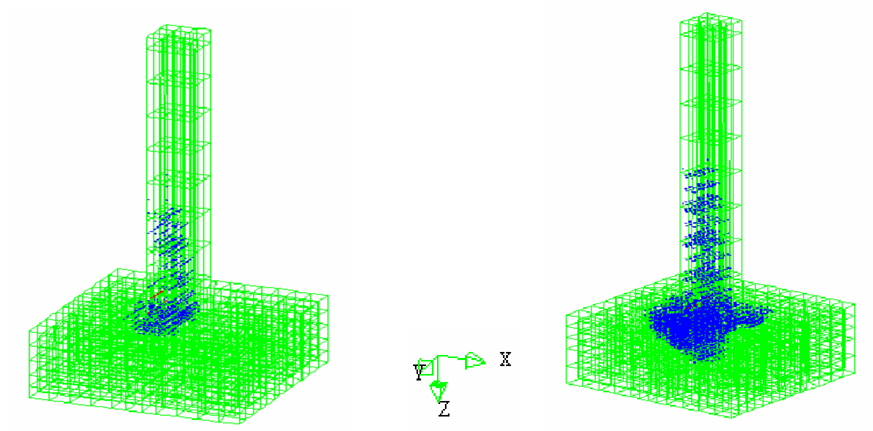


a. Unit P1 b. Unit P2

Figure 5. Principal stress distribution along the unit height with different load steps.

6. Concluding Remarks and Future Research

Current international design standards for pile-to-pile cap connections do not provide adequate guidance for reinforcement detailing at joints, however it is common sense that the resulting joint design should have enough ductility and serviceability in resisting seismic loads. The results show that the use of headed embedment as opposed to plain embedment provides effective confinement of the joint region, allowing for less congestion with comparable levels of performance. It is observed that using a longer headed embedment (1.2 – 1.5 of pile width) produces a strong connection and reduces stress concentration and crack damage at the interface and inner pile-to-pile cap connections. This study has also shown that spiral confinement at the joint and along the pile may be better achieved through the use of headed reinforcement than continuation of the pile into the pile cap (plain embedment).



a. Unit P1 b. Unit P2

Figure 6. Strain cracks pattern at the ultimate loads

The use of finite element models is effective in capturing global and local behaviour of pile to pile cap connections Additional investigation into the influence of bond-slip under cyclic loading is required. Ideally, effective tools and guidelines should be developed to assist in the practical and efficient design of pile-to-pile cap connections.

7. References

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