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Seismic performance of pile-to-pile cap connections: An investigation of design issues

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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. However, the 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 in accordance with the designers recommendations. The formation of plastic hinges in the piles remains a serious risk. A review of critical design issues and former research investigations into the soil-structure interaction of pile systems and the findings of a three-dimensional, nonlinear finite element analysis of the system is reported. Significant gaps have been identified between current practice and the performance of piling systems when subjected to seismic events. Preliminary findings indicate potential for alternate connection details to improve performance under seismic action. The paper concludes with a concise summary of current state-of-the-art design approaches and details further research requirements.

KEYWORDS: Pile-to-pile cap connections, nonlinear finite element analysis, seismic performance, international design practice, state-of-the-art design approach

1 INTRODUCTION

1.1 *Background of problem*

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 (Mizuno 1987; Nogami 1987; Sheppard 1983), 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.

In the aftermath of many earthquakes, numerous engineering inspections and investigations have been performed to assess the degree of structural damage and to evaluate the performance of various construction materials. Most of the reports, however, address only the upper structure of buildings or bridges; very little information is available on the performance of under ground structures such as pile foundation systems and their response to earthquakes. Other studies pertain to concrete piles, yet again research work specific to performance of piles and their connection to pile cap is also very limited. Research work by Pam and Park (1990b) has provided a starting point regarding the design and detailing of pile-to-pile cap connections. Specific investigation into the damage of piled foundations after earthquakes has seldom been conducted because it requires excavation and is thus costly.

Past experience has shown poor connection detail of some reinforced concrete piles and hollow prestressed concrete piles performing poor seismic performance of the pile foundation system particularly at the pile-to-pile cap connection (PPC). Lack of careful detailing and poor confinement of core concrete appears to be the reason for the failure of most of the piles. Therefore, there are still many questions left to ponder and be answered to assist the understanding of the pile-to-pile cap behaviour.

Potential inelastic damage has occurred at the interface between pile heads and pile cap as evidenced in recent earthquakes. Figure 1 shows the formation of typical potential plastic-hinges. It should be noted that because of the difficulties associated with the repair of foundation damage occurring during severe earthquakes, it is desirable to design the piles to remain undamaged. Therefore, damage due to recent earthquakes has resulted in an increased conservatism in the design of piles and pile-to-pile cap connections. In contrast, current recommendations produce connection detailing which result in high levels of congestion of steel reinforcement and extreme difficulties in construction.

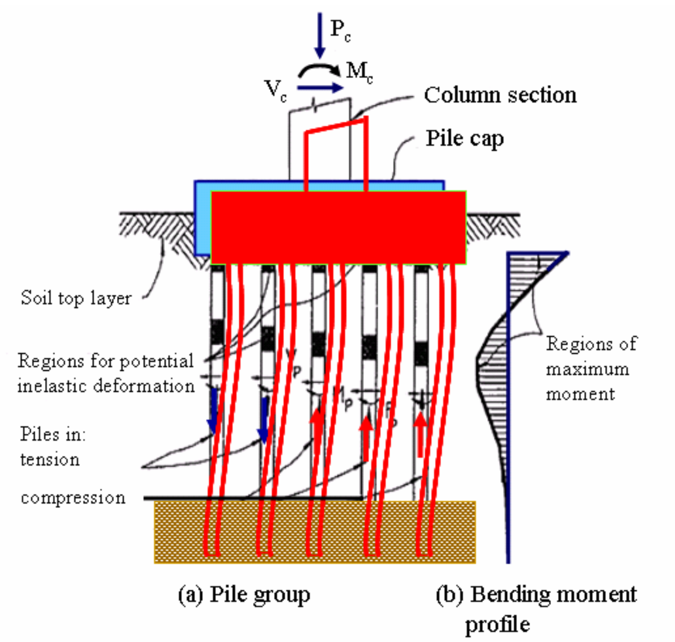


Figure 1. Response and potential plastic hinges of pile group.

Ideally, the design concept should aim to dissipate seismic energy by ductile yielding at plastic hinge regions in the structure above the foundations or mechanical energy dissipating devices placed between the foundation and the structure. i.e., the foundation should be provided with sufficient strength to ensure, as far as possible, that they remain in the elastic range while energy dissipation occurs. Moreover, the embedment detail of the connection between the precast prestressed piles and the cast-in-situ reinforced concrete pile cap is currently investigated in the University of Melbourne to fully comprehend the seismic behaviour of the overall structures.

1.2 *Aim of study*

The aim of this study is to refine the design of pile-to-pile cap connections in conjunction with capabilities of developing the moment demands on the piles as a result of the seismic events. This aim is being accomplished by subjecting analytical models of a typical connection detail of pile-to-pile cap connections. A three dimensional (3D) solid model of the pile and pile cap was generated with the finite element analysis (FEA) programs, SAP2000 and DIANA (2005), to obtain structural response that can be used to validate the analytical design procedure and to determine the types of inelastic behaviours of the system will experience under seismic loads.

In this paper, the particular case of a square prestressed pile embedded in a cast-in-situ pile cap has been analysed. Linear and nonlinear finite element and time history analyses have been conducted to investigate the behaviour of pile-to-pile cap connections under simulated seismic loads.

2 CURRENT PRACTICE

2.1 *Design standard*

In practice, the use of prestressed concrete piles and their connection to cast-in-situ reinforced concrete pile cap is inherent in pile foundations. The connection distributes shear, axial load, and moment to each pile underneath the pile cap. All connections should perform their function at all stages of loading without any distress, and with an appropriate safety factor against failure due to overload. From a previous investigation (Harries and Petrou, 2001) it has been established that piles tend to develop their flexural capacity without distress to the pile cap provided that a sufficient embedment length is furnished.

In general, design criteria for the required seismic performance of pile foundation system in a ductile earthquake resistant structure, are documented in many standards, e.g. the American Concrete Institute (ACI), the Australian/New Zealand Standards (AS/NZ), California Department of Transportation System (CALTRANS), and Japanese Codes. However these Standards do not detail the performance of pile-to-pile cap connections. As a result, research on the performance of pile-to-pile cap connections subjected to earthquake response is limited and the behaviour of these connections not well understood due to its complexity. In fact, recent research relating to the design and detailing of piles and pile-to-pile cap connections has relied heavily on the NZS 3101, the CSA Code, and the ACI Code (Joen and Park 1990).

Current design procedures for pile caps do not provide engineers with a clear understanding of the physical behaviour of these elements. The current ACI Building Code procedure/guidance for the shear design of pile caps does not predict adequately the actual behaviour because this procedure neglects certain important parameters, such as the amount of longitudinal reinforcement, and overemphasize other parameters, such as the effective depth. In order to achieve efficient connection dimensions, new advanced materials and construction techniques need to be considered.

The structure and all its components must generally be designed to resist all loads, deformations and environmental conditions likely to occur during construction and normal use, and have adequate durability. Furthermore, design loads, actions and strengths are intended to produce a very low probability of failure during the design life.

2.2 *Observation of earthquake events*

Case histories of observed reinforced and prestressed concrete piles damage and failure during global major earthquakes are presented as a representative survey in the pile foundation problems. The cases are described with extensive indications of the pile-to-pile cap performance during strong shaking and insight into modes of behaviour and failure.

The Niigata Earthquake was the second major earthquake of 1964 with the magnitude of 7.3 resulting widespread soil liquefaction related to damage and numerous failures of pile supported structures. The 7.2 m long concrete piles of 0.18 m diameter lost bearing capacity due to liquefaction, and the structure of Saiseikai Hostpital tilted and cracked (Kishida 1966).

a. Inelastic lateral deformation at pile-to-pile cap interface

b. Plastic hinges and buckling occurred within soil layers

c. Pile head released from pile cap

d. Damage at pile-to-pile cap connection

Figure 2. Pile foundation damage due to strong earthquakes (Hamada 1991; Mizuno 1987).

Figure 2a and 2b show the liquefacted soils during major earthquake affecting inelastic lateral deformation of pile-to-pile cap interface and plastic hinges and buckling along the pile length. Four years after, the magnitude of 7.8 Off-Tokachi Earthquake and magnitude of 7.4 aftershock caused serious damage to a region of Northern Japan, and in particular damage the Anenuma Bridge. Tamura et al. (1973) has undertaken post-earthquake inspection revealing cracks near the top of the piles, and over 2 ft of lateral displacement and as much as 4 in of settlement of the bridge.

The 1978 Off-Miyagi Prefecture Earthquake resulted in a number of cases of damage to prestressed concrete piles, which were principally caused by earthquake-induced vibration of the superstructure (Sugimura 1981).

The Loma Prieta Earthquake was occurred on 17th October 1989 with the magnitude of 7.0 causing the impressive failure of many pile-supported structures. It was observed that piles along an alignment that transitioned from stiff to soft foundation soils, the site response and structural connection details were the principal failure mechanism. Yashinsky (1998) provided a comprehensive summary of damage to highway systems in the Loma Prieta Earthquake such as at the Oakland Outer Harbor Pier 7, 16 in square prestressed concrete batter piles failed at or near the connection to the pile cap. It has been noted that soil liquefaction did not contribute to the failure, as upper foundation soils comprising soft clays and organic, with some alluvial sands present. Similar to severe damage over a large area caused by the Loma Prieta Earthquake is the 1991 Costa Rica Earthquake with the magnitude of 7.5 that resulted soil liquefaction relating to collapse of several pile-supported bridges. As reported by Priestley et al. (1991) two of the three spans on the Rio Viscaya Bridge collapsed due to severe abutment rotation, pile distress, and failure of an interior support, also producing from extensive soil liquefaction.

The 1995 Hyogoken-Nanbu (Kobe) Earthquake with the magnitude of 7.2 was the most destructive earthquake over 60 years to strike Japan. It was directly hit on a major metropolitan area that damaged too many pile foundation systems. The most devastated structural failure during the Kobe Earthquake was the collapse of an elevated section of the pile-supported Hanshin Expressway. Mizuno et al. (1996) has undertaken a survey over more than 30 cases of pile damage observed in precast concrete, cast-in-place concrete, and steel pipe piles. Damage patterns consisted of separation between piles and pile caps, damage near the pile head, and damage at deeper portions of piles as shown in Figure 2c and 2d.

2.3 *Theoretical design*

Design of piles to resist earthquake forces includes the interaction of lateral loads and variations in the axial loading characteristics of pile groups, as illustrated in Figure 1. Under the imposed column axial force, P_c, shear force, V_c, and moment, M_c, axial forces, P_p, shear forces, V_p, and moments, M_p develop in the piles to withstand the applied loads..

In general, the pile foundation system is designed as follows: (1) estimation of the applied forces from the column to the foundation system, (2) estimation of soil properties to determine the soil-structure interaction required for analysis, (3) design the pile cap necessary to transmit effectively the applied loads to the soil strata, and (4) selection of the pile type required to transmit the applied loads to the soil strata.

The fundamental basis of this seismic design method relies on carefully selecting and detailing locations of potential inelastic deformation. All other regions, defined to remain elastic during seismic excitation, are designed based on appropriate strength margins above the possible over strength in those regions with elastic response (Priestley et al. 1996).

In conjunction 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 this moderate condition, the design approach may be impractical due to the potential inelastic deformations that may form in the piles either in the connection region or in subgrade regions. In addition, loads and deformations imposed on the piles of a pile group, as a result of lateral translations and rotations of the pile cap, and due to seismic action, may led to unwanted inelastic response of piles at the connections to the pile cap and/or below ground level. Consequently, a simplified approach must be developed which encompasses the main features of the problem, but which is analytically and numerically tractable.

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.

2.4 *Past researches*

In view of recent proposed code changes restricting the use of prestressed concrete piling in the United States, several testing programs have been contemplated to verify prestressed concrete piles' ability to develop the required strength of the pile cap embedment and ductility at the pile-to-pile cap interface (Silva et al. 2001).

An intensive study on laterally loaded piles has also been conducted to review analytical concepts, procedures and methods for evaluating the capacity of prestressed concrete piles. Further, a new approach to analyse pile-to-pile cap connections subjected to seismic loadings is currently being formulated (Teguh et al. 2003b).

Tests on solid prestressed concrete piles have shown that properly confined members can withstand considerable displacement ductility factors without significant loss of their load carrying capacity. Furthermore, it has been demonstrated that extra longitudinal reinforcing steel improves the flexure strength but does not significantly improve the ductility of the prestressed concrete piles (Pam and Park 1990a).

In early 1983 Sheppard has initially proposed eight varieties of connection details where piles are embedded in cast-in-place reinforced concrete pile caps (Harries and Petrou 2001b; Sheppard 1983). The first three types of connections is categorized as a simple pile foundation system used in non-significant structures, for instance low-rise buildings and short-span bridges. The other types of connection details are applied to important structures requiring higher levels of seismic resistance such as high-rise buildings and long-span bridges. However, Sheppard has not yet investigated all proposed connection details.

A research project conducted by Pam (1988) investigated the adequacy of different connection details of precast prestressed concrete piles to the pile cap. Four different types of connections were used in the construction of six full scale tests. All full six scales tests were designed in accordance with the New Zealand Standard (New Zealand Standard 1982; Standard 1982) and used 400 mm octagonal precast prestressed concrete piles.

Silva (1998) investigated three standard Caltrans test units, which were tested under increasing cyclic lateral load or deformation and fully reversed varying axial loads (compression and tension). The three standard Caltrans pile test units were defined as test unit STD1, STD2 and STD3. The pile of test units STD1 and STD2 was a Class 625 pile, and for test unit STD3 the pile was a Class 1780 pile. The test unit STD1 was a full scale of 305 mm square precast prestressed concrete and the test unit STD2 was also a full scale of 356 mm diameter steel encased unreinforced concrete. The test unit STD3 was of 7/12 scale model of 356 mm diameter steel encased unreinforced concrete.

Harries and Petrou (2001) undertook experimental tests at the University of South Caroline Structures Laboratory. Two 18 in. (450 mm) square by 18 in (5.49 m) long piles were fabricated simultaneously in a 40 ft (12.2 m) prestressing bed. In this experiment, a detail connection recommended by Sheppard (1983) was adopted where the pile was simply embedded in the pile cap without any treatment so called plain embedment (i.e., no static bars). The connection details used in the first three experimental test models of pile-to-pile cap connections were based on recommendations by the New Zealand and Caltrans standards (Caltrans 1990; Standard 1982). The objectives of the studies were to check the capabilities of pile-pile cap connections to resist large inelastic deformations caused by cyclic lateral loading.

3 IDENTIFIED GAPS/WEAKNESSES IN CURRENT APPROACH

A current state-of-practice design and analysis application of the soil-pile interaction considered in the pile-to-pile cap connections is a complex soil-structure interaction of the pile foundation system and not well developed. The unavailability of standardized and validated analysis techniques, and the conservative perception, designers routinely ignore or greatly simplify the present of pile foundations in their analyses.

A special challenge of soil-structure interaction problems is in dispute over two disciplines, geotechnical and structural engineering, and the analysis is frequently broken into two parts rather than addressed in a holistic manner. In fact, a geotechnical engineer idealizes a complex multimode superstructure as a single degree of freedom oscillator; while the structural engineer often represents the potentially nonlinear soil-pile interaction with a simple linear spring. In this manner, nonlinear system interaction between the superstructure and substructure is artificially prevented. The diverse and non-standardized design approaches are basically intended to illustrate the lack of professional consensus and the gap between the current state-of-practice and the current state-of-the-art. These approaches are also considered as creative applications of limited tools to complex problems.

4 PRELIMINARY ANALYSIS

This section presents preliminary analysis of pile-to-pile cap connections focusing on two types of connections, i. e., plain embedment and headed embedment, to improve the capacity of the pile-to-pile cap connections. 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, prestressed concrete piles manufactured in Indonesia (Teguh et al. 2003a) and two pile units tested by Harries and Petrou (2001b), were reviewed and the validity of their hypotheses was rigorously investigated through comparison of the observed and predicted response from linear to nonlinear behaviour.

4.1 *Analytical approach*

To determine the capacity of a plain pile-to-pile cap connection (Figure 3a), it is assumed that a rigid body (pile) embedded in cast-in-place concrete monolith (pile cap) and based on mobilization of an internal moment arm between bearing forces C_f and C_b as shown in Figure 3b will be utilized. A parabolic distribution of bearing stresses is assumed for C_b, and C_f is computed by a uniform stress equal to $0.85fc'$. The bearing stresses are distributed over the width of the embedded pile, b. Based on these assumptions and calibrating the calculated stresses against experimental data, the required embedment length, L_e, may be determined from Equation 1, where a is the shear span of the pile (distance from pile cap to assumed point of zero moment) and β_1 is the concrete stress block factor defined in ACI 318-02, Section 10.2.7.3 (A.C.I. Committee 318 2002) as presented in Equation 2. The shear span can be increased by an amount equal to the concrete cover, c, to account for possibility spalling of the soffit of the pile cap as shown in Figure 3b. The value of b' is given by Mattock and Gaafar as the width of the

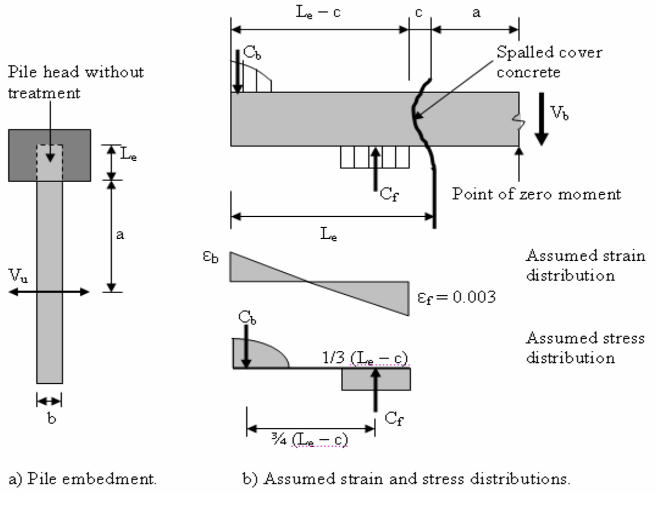


Figure 3. Analytical approach for determining pile capacity plain embedment.

element into which, in this case, the pile is embedded (Mattock and Gaafar 1982).

Using a slightly different assumed stress distribution, Marcakis and Mitchell (1980) proposed Equation 3 to determine the embedment length, L_e. This expression has also been calibrated against experimental data, where e is the eccentricity from the point of zero moment to the centre of effective embedment (Figure 3). They define b' based on a strut-and-tie approach as being effective width to assumed "tie" steel, limited by a value of $2.5b$ (Marcakis and Mitchell 1980).

$$V_u = 4.5\sqrt{f_c'}\left(\frac{b'}{b}\right)^{0.66}\beta_1 b L_e\left[\frac{0.58 - 0.22\beta_1}{0.88 + \dfrac{a}{\ell_e - c}}\right] \tag{1}$$

For $f_c' \leq 4000$ psi (27.6 MPa), $\beta_1 = 0.85$ and for $f_c' > 4000$ psi (27.6 MPa), $0.65 \leq \beta_1 \leq 0.85$, where:

$$\beta_1 = 0.85 - 0.05\left(\frac{f_c' - 4000}{1000}\right) \tag{2}$$

In addition, the nominal moment capacity of the piles corresponding to an axial load is predicted using plane section analysis program RESPONSE-2000 (Bentz 2001).

$$V_u = \frac{0.85 f_c b'(\ell_e - c)}{1 + \dfrac{3.6}{\ell_e - c)}\left(\dfrac{\ell_e - c)}{2} + c + a\right)} \tag{3}$$

Curves generated from Equations 1 and 3 are presented in Figure 4 as a comparison of the shear and moment capacities of the embedment for varying embedment length and having varying square pile sizes manufactured in Indonesia (Figure 4). It should be noted that the results from Equations 1 and 3 are similar. It can be summarized that equation proposed by Marcakis and Mitchell tends to result is slightly more conservative embedment capacity values than the values suggested by the PCI design handbook.

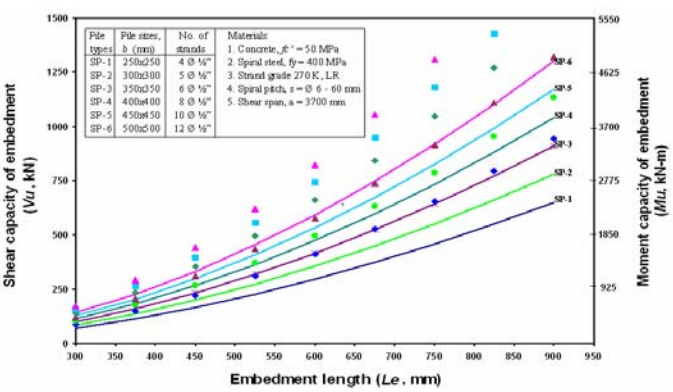


Figure 4. Embedment capacity predictions of varying pile sizes manufactured in Indonesia (Teguh et al. 2003a).

4.2 *Linear finite element analysis*

The study continued with a review of modelling assumptions to assess the effect of varying the degree of structural complexity. The most comprehensive form of analysis for reinforced concrete members uses the finite element (FE) method where individual bars and elements are separately modelled. A finite element method allows satisfactory simulation of test results, provided accurately defined constitutive models are used. A fixed base pile cap acting as a simple cantilever column was therefore employed in the model.

A three dimensional (3D) finite element analysis was performed to model the behaviour of the pile-to-pile cap connection. The objectives of this modelling were to determine elastic stress distribution and load-displacement response of the pile-to-pile cap connection due to axial and simulated seismic loads. A linear static finite element analysis as initial stage of elastic modelling was developed to simulate the seismic behaviour.

The two selected pile units (Harries and Petrou 2001b) presented in Table 1 to 3 respectively was analysed using the structural analysis software SAP2000 (Wilson 1996). Two element types were used for the finite element discretisation of the models. The concrete pile-to-pile cap was meshed (modelling surface delineation lines) using a special solid element type in order to easily perform the stress concentration or failure of concrete in cracking/crushing. Strands and reinforcing bars used in the pile and pile cap were meshed using truss elements with two different materials (i.e., strands and reinforcing bars).

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

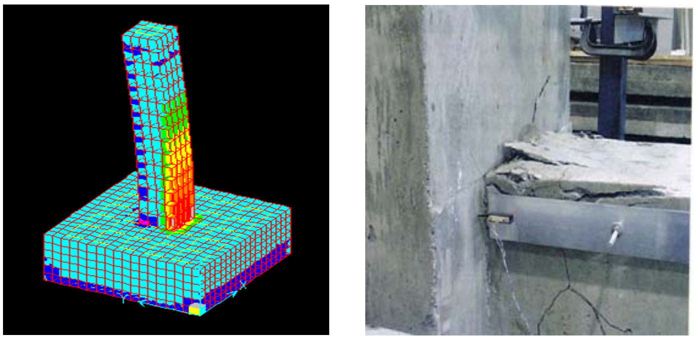
Pile unit number	Unit P1	Unit P2
Initial prestressing steel (constant)	1.394×10^9 N/m^2	1.394×10^9 N/m^2
Applied axial load (increment): assumed as pressure load over the pile surface	4.260×10^6 N/m^2	4.260×10^6 N/m^2
Maximum lateral loads (monotonic & cyclic)	1.112×10^5 N	9.600×10^4 N
Pile reinforcement: 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 0.45 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	0.61 m (1.3 B)	0.45 m (B)

Table 2. Mechanical properties of concrete

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

Table 3. Mechanical properties of reinforcing bars

Reinforcement	Yield stress (MPa)	Tensile strength (MPa)	Modulus of elasticity (MPa)
Tendon	1791	1882	1.94 x10^5
Wire	448	630	1.86 x10^5
D22	275	464	2.00 x10^5
D12	275	464	2.00 x10^5



a) FE deformed stress contour at maximum moment b) Experimental test (Harries and Petrou 2001b)

Figure 5. Stress propagation and deformation of the structure.

In the 3D linear static analysis, lateral and axial loads were predicted by the Response2000 program. This numerical analysis provided stress distribution at each element as shown in Figure 5a.

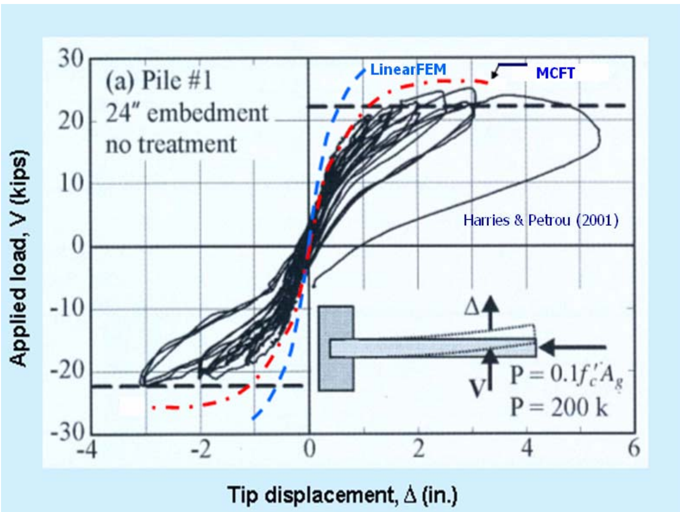


Figure 6. A comparison of load-deformation response.

As seen there are crack damage propagating along the tensile fibre as well as stress concentrations occurred at the interface between the pile head and pile cap and these stress concentrations also occur at the opposite face when the lateral load is reversed.

In addition to the experimental process, the magnitude of the structural loading was incrementally increased in accordance with a certain predefined pattern. During the investigation, the cyclic lateral load was applied three times at each load or displacement level. With the increase in the magnitude of loading, weak links and failure modes of the structure were found as shown in Figure 5b. The loading carried out in the experiment was monotonic with the effects of the cyclic behaviour and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations (Figure 6).

4.3 *Time history analysis*

Time history analysis incorporating a discrete model where the inelastic action is effectively lumped or concentrated over a short length known as a plastic hinge is described in this section. The program Ruaumoko (Carr 2001) incorporates the Giberson one-component model, which is an elastic beam with the potential to develop a plastic hinge at one or both ends. The program has also a wide range of hysteretic models for describing the inelastic behaviour of a member including the simple elasto-plastic and bi-linear models and the more complex modified Takeda degrading stiffness rule.

In the time history analysis, a simple finite element model of the pile-to-pile cap connection for test unit 1 was descretised into small elements and established using the Giberson's model with a plastic hinge at the fixed end. The lumped mass properties and cyclic loading history applied to the free end were identical to the values used for the experiment. In the analysis, input data for ground acceleration was based on the 1995 Kobe Earthquake record (Figure 7). Using the Ruaumoko 3D program, the tip displacement for unit 1 was predicted as shown in Figure 8.

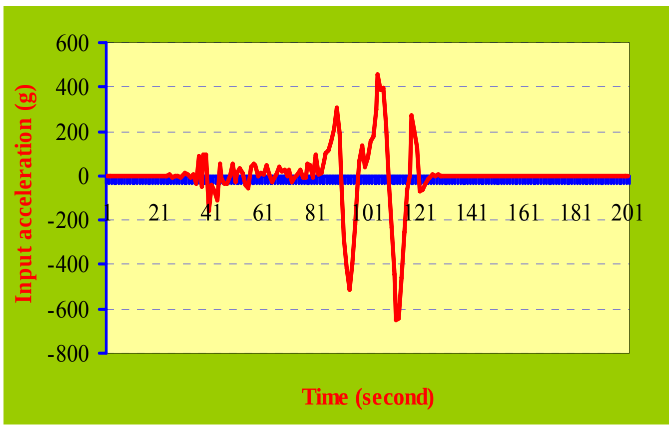


Figure 7 Ground acceleration input (Kobe Earthquake, 1995).

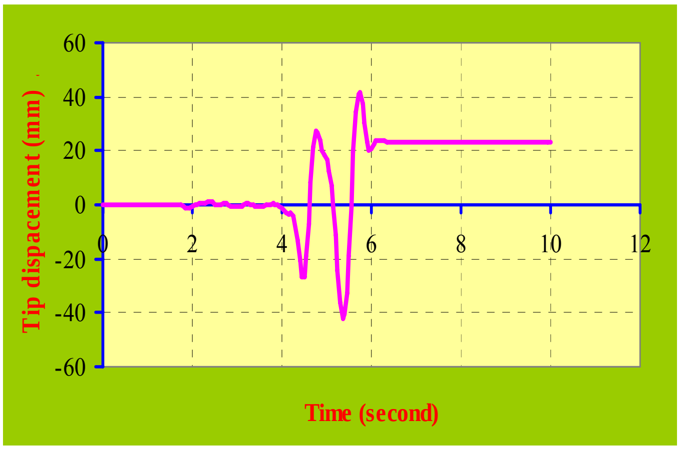


Figure 8 Time-displacement response at cantilever tip.

4.4 *Nonlinear cyclic analysis*

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 commercial finite element analysis program, DIANA 9.0 (de Witte and Kikstra 2005). Each material displayed nonlinear static behaviour in order to achieve the desired performance, and material properties had to be accurately modelled.

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 varied ranging 1.0-1.3 of pile width respectively.

A single prestressed concrete pile connected to a cast-in place pile cap was modelled with discrete 3-D mesh solid 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 was modelled by introducing interface and/or linkage element(s) in the interface between truss element and concrete element. All other reinforcing bars and tendons were modelled by an embedded element, assuming perfect bond.

In the FE method, reinforcement may generally 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.

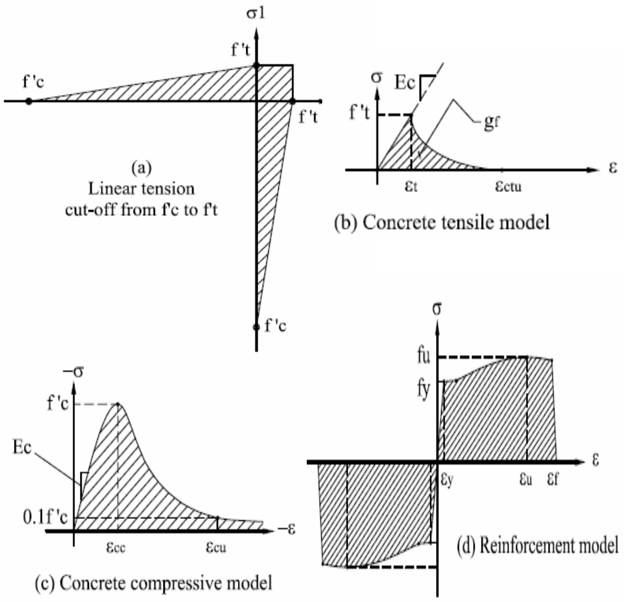
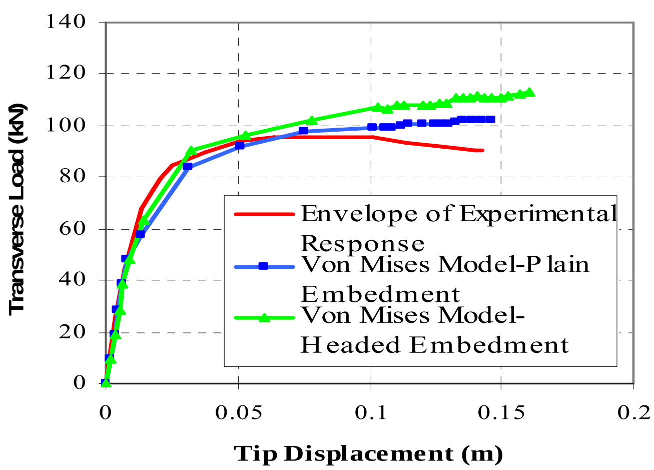
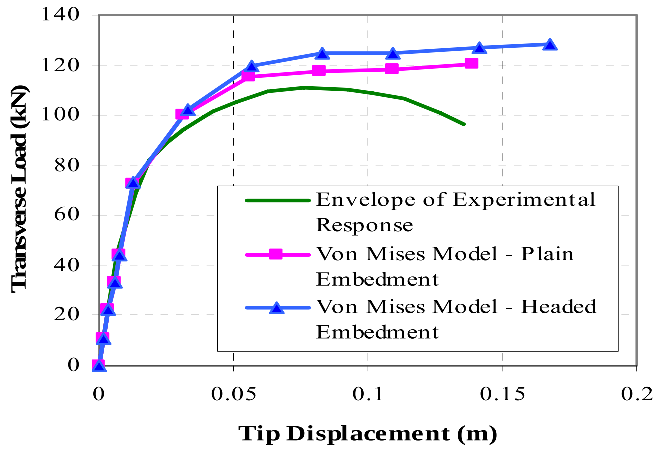


Figure 9. Constitutive material models for concrete and reinforcement.

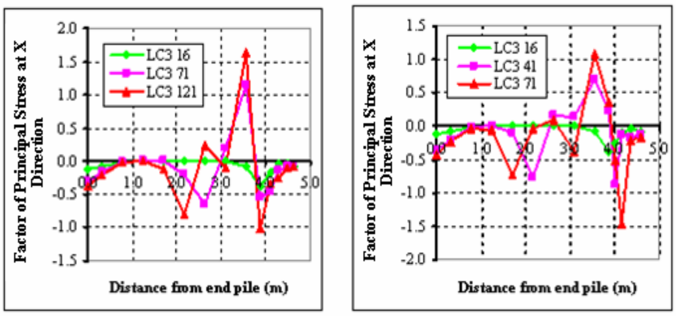
The constitutive relationships used for the FE models are shown in Figure 9. The non-prestressed and prestressed reinforcements use Von Mises yield hardening criteria with constitutive models matching the behaviour determined from testing (Figure 9d). Concrete is modelled using Von Mises yield criteria for compression a tension cut-off from the concrete compressive strength, fc', to the concrete tensile strength, ft' (Figure 9a) for tension. The concrete compressive behaviour models the behaviour obtained from displacement controlled testing of concrete cylinders (Figure 9c). The Hordijk model shown in Figure 9b 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 was modelled using both multiple fixed cracks and rotating crack formulations. The results presented in this paper are limited to the fixed crack model.



a) Unit P1
b) Unit P2
Figure 10. Comparative load-displacement response (nonlinear FE analysis).



a) Unit P1 b) Unit P2
Figure 11. Comparative load-displacement response (nonlinear FE analysis).

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 9 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 10a and 10b, a longer embedment length develops the flexural capacity of the pile without distress to the pile cap.

5 FINDING AND DISCUSSION

Seismic performance of piles is directly related to the earthquake response of pile-to-pile cap connections. The maximum loads at early stage of analysis were computed from the member yield moment capacities generated in the structure at its state of near collapse, but are limited to the values generated to the structure under elastic condition. The theoretical method presented predicts shear and moment capacities of the plain embedment of pile-to-pile cap connection and performance of square prestressed concrete piles. In addition, previous experimental tests of plain square pile-to-pile cap connections by Harries and Petrou (2001a) show good agreement with the analytical method.

This paper reports the preliminary results for a pile-to-pile cap connection with both connection details; i.e., plain and headed embedments. The more complex behaviour of pile-to-pile cap connections is being investigated and will be reported in the near future.

The response of the pile due to cyclic loading is presented in Figure 6. The predicted pile moment capacity of 285 ft-kips (386 kN-m) is shown as a horizontal dotted line. It was observed that the applied load of 13.9 kips (62 kN) caused the first crack at the pile-to-pile cap interface, while the predicted load to cause cracking of the pile was 11.2 kips (49.7 kN).

As seen from Figure 6, Response2000 (Bentz 2001) program gave a maximum load of 25.44 kips (113.2 kN), with significant inelastic behaviour close to this load. The maximum load corresponds to a displacement of 3.13 in (79.5 mm) at the pile tip. In addition, theoretical moment-curvature relationships were developed for the pile cross sections using the Modified Scott stress-strain models for spirally confined concrete. The stress-strain constitutive model for concrete developed at the University of Melbourne (Mendis 2001) took into account the increase in strength and ductility of the concrete resulting from the confinement. This model was used to modify the existing unconfined stress-strain model in the Response2000. The force-displacement responses obtained from numerical analyses procedures, finite-element analysis and Response2000 are shown in Figure 6. The load-displacement response predicted using the linear static pushover method is conservative compared to the results of the experimental investigation due to the linear material behaviour used during the analysis. However Response2000 program gave a better prediction of the load-displacement response.

The time history analysis using the Kobe Earthquake ground motion predicted a small value of maximum cantilever tip displacement of 1.72 in.

(43.7 mm) at 5.8 seconds (Figure 8 compared to the experimental maximum of 3.10 in. (78.8 mm) (see Figure 7. Therefore the response used in the time history analysis need to include other parameters such as soil pile interactions.

During the tests conducted by Harries and Petrou (2001) it was noted that, at the end of the test, the compressive fibres at the pile head (close to the interface between pile head and pile cap) spalled and crushed, and subsequent cracking propagated elsewhere near the connection (Figure 5b). Furthermore, an interfering action resulted when moment was applied at the interface of the pile and the pile cap, thereby increasing the contact pressure at the junction of the pile face and the pile cap. The maximum stresses predicted by the finite element model (Figure 5) reflected the observed crack propagation at the pile-to-pile cap interface, thus clearly representing the pile damage after testing (Figure 5b). Either the linear static pushover analysis or the time history analysis, however, did not accurately predict the pile damage except the nonlinear analysis provides the damage locations as shown in Figure 11 where the value of principal stress greater than 1.0.

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. 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 11); this behaviour was observed in both pile units. Figure 11 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.

6 CONCLUSIONS

6.1 *Current state-of-the art*

Indicative conclusions have been drawn based on the analysis of the experimental data and the comparison with simple numerical models.

The obtained numerical predictions for load-displacement response using the finite-element method and Response2000 gave reasonable agreement compared to experimental results. It is evident that more work is required to understand the behaviour of prestressed concrete pile-to-pile cap connections between experimental results and numerical predictions will assist in confirming the indicative findings. Consideration of refined numerical simulations may also be of significant help.

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.

It appears that the overall joint performance would be improved if the shear strength of non-critical sections was reduced. By non-critical, it is meant that these elements are not required to restrain transverse forces. Design of piles employing a conditional ductility approach, will result in a more economical pile foundation system. Also, a more flexible foundation system may reduce the ductility demand on the columns, potentially reducing damage to the building or bridge under seismic events. This work is continuing at the University of Melbourne.

6.2 *Future work*

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).

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.

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