Research report

Mechanisms of response and implications for the performance of pile groups in laterally spreading soils

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Section 1

Introduction

1.1. Introduction

The design and performance of pile foundations in liquefying, laterally spreading soil is an important challenge in geotechnical earthquake engineering. The interaction between the soil, foundation and superstructure during an earthquake is a complex and intense dynamic process, and the responses of the different components of the soil-foundation-superstructure system are highly interdependent. The demands on the foundation and the properties of the soil may vary significantly over the course of strong shaking. Properties such as the pile tip fixity or the stiffness of the superstructure have the potential to alter the mechanism of response of the foundation (Haskell et al., 2012b, O'Rourke et al., 1994), which may have important consequences for the prediction of foundation and superstructure performance and for the specification of inputs for state-of-practice simplified and pseudo-static analysis methods.

For example, the recent (2010-2011) series of strong earthquakes near Christchurch, New Zealand, have highlighted the influence structural restraint can have on the mode of deformation of piled bridge abutments (Haskell, 201x; Wotherspoon et al., 2011). Specifically, the decks of the short-span bridges that cross the city's Avon River were sufficiently stiff and strong to prevent significant lateral translation of the bridge abutments, in spite of the large lateral spreading ground displacements that developed. As a consequence of the restraint from the superstructure, the abutments of the affected bridges all suffered the same characteristic 'back-rotation' mode of deformation (Cubrinovski et al., 2014), with the abutment toes and the heads of the piles 'kicking out' towards the centre of the river.

This example illustrates the importance of identification of the foundation deformation mechanism for the accurate prediction of the distribution and severity of damage sustained by the foundation in a strong earthquake. For the case of pile foundations in laterally spreading soil, the foundation-soil interaction is often conceived as an inter-play between pile flexure (controlled by pile stiffness) and soil deformation (controlled by soil 'stiffness' and 'strength') (Cubrinovski et al., 2006). Much of the guidance for the use of state-of-practice analysis methods (for example the specification of p-y spring properties for the laterally spreading soil, or the identification of the critical uncertainties in the analysis (Haskell et al., 2012a)) is based on the presumption that this mechanism will develop and dominate. However, there are other mechanisms (including the back-rotation encountered in Christchurch) that may develop in preference to pile cap translation/soil deformation, under certain circumstances (several examples of which are noted by O'Rourke et al., 1994, Finn, 2005). Having recognised the potential for the development of other modes of deformation aside from pile cap translation, the issue that follows for the engineer is whether the understanding, empirical relationships, and idealisations established for one mechanism can reasonably be applied for the scenario they are trying to model in their analysis procedure.

The aim of this research is therefore to provide guidance for the design of pile groups in laterally spreading soil

Section 2 of this report covers the essential background information relevant to the study of pile foundation in laterally spreading soil and establishes the specific research objectives to be addressed. Section 3 then details the overall design of the research and the details of the experimental programme. The first of the research objectives - the description of the influence of pile tip fixity on foundation performance – is the focus of Section 4. Section 5 explores the influence of pile cap restraint on the foundation response and brings together the experimental findings and field observations from the 2010-2011 Christchurch earthquakes. The final results section, Section 6, examines in more detail lateral pile interaction effects for different pile group response mechanisms. Finally, the conclusions of the project are presented. Note that the figures and tables are not included in the main text, but rather are provided at the end of the report, after the references.

Section 2

Essential Background

2.1. Introduction

This section provides a brief overview of essential background understanding relevant to pile foundations in laterally spreading soils. Typical patterns of foundation damage and deformation observed in the field are first reviewed. The lateral loading of pile groups and lateral pile interaction effects are then briefly discussed, followed by a summary of the common conception of the main mechanisms of interaction between piles and laterally spreading soil.

2.2. Field performance of piles in laterally spreading soil

In spite of significant improvements to engineering practice in the last 40 years, failures of pile foundations in laterally spreading soils continue to occur (Finn and Fujita, 2002), emphasising the vulnerability of piles to liquefaction-induced damage. The evaluation of the foundations of expressways in the Kobe region of Japan undertaken by Matsui and Oda (1996) following the 1995 Kobe earthquake provides clear evidence of the strong correlation between the extent and severity of pile damage and the foundation soil conditions.

If subjected to lateral spreading, the extent to which the pile foundation follows or resists this displacement often provides an approximate indication of the damage the piles have sustained. Where the foundation is sufficiently strong and stiff to resist the driving pressures from spreading soil without displacing significantly, the damage suffered by the piles is typically less severe (Tokimatsu and Asaka, 1998; Finn and Fujita, 2002). This contrasting 'stiff' and 'flexible' pile behaviour suggests that the stiffness and strength of the foundation as compared to that of the displacing soil is fundamental to the nature of the soil-pile interaction, controlling the interplay between foundation damage, loss of lateral resistance, and the deformation and flow of the spreading soil.

The typical pattern of damage of piles in laterally spreading soil involves the concentration of cracking, yielding and even complete rupture of piles at the boundaries of the liquefiable soil layer(s), where there is a sharp change in soil strength and stiffness, and at the pile cap, where the piles and cap are rigidly connected. The typical double curvature deformation pattern can be found in a large number of lateral spreading field reports for pile foundations (as evident from the reviews of Ishihara (1997), Tokimatsu and Asaka (1998), and Berrill and Yasuda (2002)), suggesting that the pile deformation is more or less controlled by the demand at the top of the foundation and that the foundation's lateral resistance is derived primarily from bending of the piles and support from the non-liquefied base soil (the pressure from the intermediate liquefying soil being apparently less significant (Abdoun and Dobry, 2002)). The pattern of damage typical of floating piles is slightly different; they tend to deform in single curvature, rotating about their tips and cracking and failing at their heads. This latter example points towards the influence of pile fixity conditions with respect to the damage sustained by the piles. However the tip and head fixity also affects the piles' effective stiffness and the lateral resistance they are able to mobilise.

Finn (2005) discusses the influence of pile head rotational fixity on effective pile stiffness with reference to contrasting practice in US and Japanese for the design of grade beams between piles. Specifically, the large, stiff beams typical of Japanese construction provide significant rotational restraint to the piles, potentially reducing the lateral displacement of the foundation as compared to the 'near free-head' condition associated with much lighter grade beams typical of US practice. However, the significant rotational restraint of the pile heads also means the piles attract significant flexural demands, requiring them to be detailed to achieve sufficient strength and ductility to realise their intended lateral capacity and avoid severe and sudden loss of foundation stiffness, should the piles exceed their structural capacity.

Axial response and capacity are also relevant to pile foundations in laterally spreading soils, in particular for groups of piles connected by a pile cap. Aside from 'vertical' modes of failure (which include differential settlement and tilting, differential settlement between the foundation and the surrounding soil, or combined vertical-lateral instability).The axial capacity of piles may also affect the resistance to lateral pile cap translation that can be achieved via the frame action that is implicit to the lateral loading of a pile group. This consideration is mentioned in passing by Finn (2005) with regard to pile interaction factors and the inherent coupling of rocking and translation in the motion of pile groups during strong shaking.

2.3. Pile group effects

Group effects for laterally loaded piles can be broadly categorised as either structural, where the piles influence each other as a result of a direct structural connection, or soil-deformational, where the deformation of soil around a given pile is altered by nearby piles (the piles need not be directly connected for this sort of interaction to take place) (Cubrinovski and Ishihara, 2007). Depending on the fixity conditions of the piles (Gerber and Rollins, 2009) and their relative spacing as compared to their (effective) length, more or less of the lateral demand is resisted by frame action of the pile-pile cap-pile structure.

In studies of soil-deformational group interaction, the distinction between perpendicular (i.e. side-by-side) and parallel (i.e. front-to-back or 'shadowing') interaction effects is usually made, and particular attention is given to the influence of pile spacing (typically measured in pile diameters) on the extent of the interaction. The side-to-side interaction can be thought of as an overlapping of shear wedges or zones of plastic flow. The nature of shadowing interaction is more intuitively obvious, the soil between the leading and trailing piles being displaced (in the case of active loading) or 'held-up' (in the case of passive loading) such that the relative soil-pile deformation between trailing piles and the intermediate soil is reduced (Brown et al., 1988) and the mobilised soil resistance is altered such that, for example, the typical shear wedge is unable to form.

Typically, the shadowing interaction between piles in a group is more significant than any side-by-side interaction (Brown et al., 1988, Rollins et al., 2006). Furthermore, the bending moment distributions of different piles in a group may vary, with leading piles tending to exhibit more concentrated bending and higher peak moments, despite having the same pile head displacement as the piles behind (Rollins et al., 2006). They are thus more likely to suffer damage. If present, the pile cap provides significant rotational restraint to the piles (which thus deform in double rather than single curvature). The cap also imposes an equal displacement condition at the heads of the piles, resulting in significant structural interaction, as well as soil-deformational interaction between the piles. There is typically a difference in bending moment distribution for leading and trailing capped piles, specifically a difference in magnitude of the pile head bending moment and also a difference in its location. There is also the potential for opposing shear forces at the pile heads due to the interplay between soil-related and structural interaction.

2.4. Mechanisms of response of pile groups in laterally spreading soil

The pile stiffness has a significant influence on the foundation behaviour. Large-scale 1g testing by Cubrinovski et al. (2006a) of 'stiff' and 'flexible' single piles in liquefying, laterally spreading soil demonstrated the fundamental importance of relative soil-pile stiffness in determining the overall foundation response and the extent of relative soil-pile deformation. The flexible pile essentially followed the ground displacement, forming a plastic hinge, and eventually failing at its base, while the stiff pile displaced a small amount initially, then no further, despite the continued displacement of the soil. This contrasting response is essentially a function of the stiffness and strength of the various soil layers relative to that of the foundation, via the interplay between the deformation-induced soil demand and the corresponding compatible pile deformation.

In terms of the foundation performance, 'stiff' pile behaviour is regarded as more desirable due to the implicit reduction of pile cap displacement. However many of the case histories reviewed in Section 2.2 indicate that good performance of stiff large diameter piles is not a certainty. As noted by Haigh (2002) stiffer piles attract increased bending demands. Furthermore, stiff foundations potentially require greater support from bearing soil layers (as compared to flexible or weaker piles) in order to realise their full flexural capacity. Recent tests of pile groups in laterally spreading soils by Brandenberg (2005), Knappett (2006) and Stringer (2012) have revealed the potential for unstable forward/downslope rotation of the foundation, which implies compliance of bearing soil, both laterally and axially, and the transition from a pile bending-lateral soil deformation mechanism of response to a global instability failure of the pile group.

2.5. Summary

The foundation boundary conditions, in particular the connection to the superstructure and the pile tip fixity/bearing layer soil, have been the subject of relatively few comprehensive or systematic studies. The contrasting deformation and damage sustained by, for example, the bridge abutments in Christchurch and the long span bridges in the Niigata and Kobe earthquakes demonstrate the relevance of fixity and boundary conditions to the performance of real foundations as well as to the successful physical and numerical modelling of the interaction between piled foundations and laterally spreading soil. The primary objective of this study is thus to explore the influence of pile tip and cap fixity conditions on the mechanisms of foundation response and the consequences for foundation performance. In light of the ultimate aim to provide guidance for essential design decisions, the influence of pile spacing on the foundation response is also an objective of this research, although is discussed only briefly in this report.

Section 3

Research design

3.1. Introduction

It is evident from Section 2 that any one of a number of mechanisms of interaction between pile groups and laterally spreading soils may dominate in the field, depending on the soil, foundation, superstructure and demand characteristics for the given scenario. However, it is also evident that the links between many of these characteristics, the consequent mechanism of interaction, and the associated foundation performance are not yet well characterised, precluding the development of an overall general understanding of the system response.

In reality, the possible combinations of soil profile, pile foundation properties, and seismic demand are essentially limitless. Faced with this considerable range of soil-foundation-demand systems is desirable to do more than simply describe empirically the relationship between certain parameters and the overall foundation performance. As noted by Finn (2005), the foundation response depends on the loading mechanism and there is a need to account for this in design. The philosophy that underpins the design of this study is that between the system characteristics and the foundation performance lie different mechanisms of superstructure-foundation-soil-demand interaction, the identification, unravelling, and (ultimately) the understanding of which are fundamental to the development of any global and integrated conception of the response of pile foundations in laterally spreading soils.

Clearly the effect of all possible variables cannot be rigorously studied in a single suite of experiments, hence the need to consider at the outset the intended and anticipated outcomes from this project. The specific research objectives outlined in Section 2.5 together with the overall aim of the project (to recommend how best to design pile foundations to withstand demands from laterally spreading soils) are thus central to the overall design of the experimental programme.

3.2. Research tools

Given the emphasis of this research on foundation response mechanisms, it is essential that the nature of soil-pile interaction that occurs when lateral spreading takes place in real life is suitably replicated. Furthermore, the 'scale' of modelling needs to reflect the focus on the full soil-pile system behaviour and the overall foundation performance. An appropriate choice of experimental method is the centrifuge modelling of complete, idealised foundation-soil systems. As noted by Coehlo (2007), centrifuge modelling is particularly well-suited to problems where the response mechanisms is unclear or where there exist complex interactions between several concurrent or competing mechanisms, which is the case for pile groups in laterally spreading soil. The method is not without faults however, and care must be taken to ensure any observations are truly due to the phenomena to which they are attributed and not in fact artefacts of the test design or modelling process.

Centrifuge modelling developed out of the realisation that the non-linear stress-strain behaviour of the soil is central to the macro-scale response of soil structures, and thus correct simulation of stress is necessary if reduced-scale models are to exhibit the same fundamental behaviour as the full-scale systems they are meant to represent. The enhanced gravitational field induced by the centrifuge ensures that the stress and strain are equivalent at corresponding locations of the model and prototype. Various other quantities scale according to different relationships (which can be derived via dimensional analysis) (see Taylor (1995) for a comprehensive review of centrifuge scaling relationships).

3.3. Suite of tests

A suite of nine centrifuge tests has been undertaken, numbered JH01 through JH09 in reference to their order of completion. Recalling the key variables under investigation as summarised in Section 2.1, namely the pile cap restraint, the pile tip fixity, and the pile spacing, the suite of tests was designed such that the effect of each variable could be isolated (as far as possible). All possible combinations of pile tip and cap fixity, pile spacing, and pile properties cannot be covered in a suite of nine centrifuge tests, however the test programme was somewhat adaptive in its design, with the details and focus of each test informed by those previous. The configuration of test JH01 is illustrated in Figure 3.1, as are the variations employed for tests JH02 to JH09. The remainder of this section provides an overview of the model constituents, instrumentation, and testing procedure. More detail can be found in Haskell (201x)

3.3.1. Model container

A 'laminar' box was used for this project (Figure 3.2). It consists of disconnected, stacked rings, between which sit roller bearings and polytetrafluoroethylene (PTFE) spacers. Over the full height of the box an average shear strain of 15% can be accommodated. The simulation of gently sloping ground is achieved by placing wedges beneath the laminar box before mounting it on the centrifuge shaking table to achieve a uniform slope of 3°. However the static groundwater conditions in the model do not correspond to those of an infinite slope – instead the fluid surface assumes a profile perpendicular to the radial g-field. In this suite of tests viscous methyl cellulose fluid was used to account for the dynamic and pore fluid flow time scaling discrepancy affecting dynamic centrifuge testing.

3.3.2. Soil

When selecting soil with which to construct the models certain requirements, such as susceptibility to seismic liquefaction, must be met, however this still leaves available many options (the range of soils capable of liquefying under seismic shaking is considerably broader than once thought). Liquefaction susceptibility is not the only consideration however. These tests focus on soil-structure interaction, therefore the soil particle-pile interface, specifically the dimensions of the piles relative to the soil grains, needs to be sufficiently large for individual grain effects not to be important. On this basis the two primary soils used to construct the models are Hostun sand (for the liquefiable layer) and Fraction C Leighton Buzzard sand (for the bearing/base layer). For all tests the relative density of the Hostun sand layer was approximately 40% and the relative density of the Fraction C layer was 75-85% for JH01-JH02, and approximately 95% for JH03-JH09.

3.3.3. Model pile group

A square two-by-two pile layout is used for the pile groups of this study. Practical requirements more or less constrain the size of the pile group to a certain range. The size of the piles themselves is limited at the lower end not only by the size of the soil particles (as discussed earlier), but also by the practical consideration of instrumentation. The upper limit for the size of the pile groups is essentially controlled by the size of the container. It is important that there is sufficient space between the model foundation and the sides of the container, so that the full and correct mechanisms of soil deformation are not altered or prevented from developing. A maximum width of the pile cap (the widest part of the model foundation) of 90 mm (slightly over one third of the width of the laminar box, thereby allowing a space of approximately one pile cap width either side of the foundation) has been employed for the largest pile group.

A modular design approach has been adopted, with individual piles (either instrumented or non-instrumented) free to be swapped to different positions within the group or into another pile cap having a different spacing for different tests (Haskell et al., 2011). Figure 3.3 shows the concept and key details of the modular design.

In terms of the 'engineering' design requirements for the model pile groups, reasonable replication of the stiffness and strength of real foundations of a corresponding size is desirable, the intention being that the model groups reflect current best-practice for design to withstand lateral spreading demands. Two sets of piles have been used, one over-designed and intended to remain elastic, even under very large bending demands (essentially exhibiting a prototype strength an order of magnitude greater than a real pile of the same prototype diameter), and the second having a strength consistent with real piles (not discussed further in this report). Table 3.1 summarises the key mechanical properties of a range of model and real foundations, alongside those of the piles designed for this project. The same pile length, 200 mm, is used for all tests and the pile cap is embedded in the uppermost soil layer to a depth of approximately 20 mm at its centre.

3.3.4. Soil-foundation configuration

To replicate the pile tip fixity condition of rock-socketed piles for test JH06 a raised 'clamp block' (Figure 3.4) was used. For all other tests the pile tip embedment in the dense base layer was approximately 7D (or 12D, for test JH04). For tests JH05 and JH07-JH09, which explored the back-rotation mechanism exhibited by bridges in the Christchurch earthquakes, an 'abutment fitting' was used in order to replicate the foundation-superstructure interaction typical of these bridges (Figure 3.5). The point of contact is approximately 95 mm above the underside of the pile cap, corresponding to a prototype height of 4.18 m, which is at the upper end of the range of abutment heights of the bridges encountered in Christchurch (Haskell et al., 201x).

3.3.5. Instrumentation and measurement

The models were instrumented to measure the following soil and foundation quantities: pore pressure, acceleration, displacement, pile bending strain, and abutment compressive force (Figure 3.6). A detailed description of the instrumentation employed in the tests can be found in Haskell (201x). Tests JH03 to JH09 also employed high-speed overhead video footage of the soil surface and pile cap for qualitative assessment of the relative soil-pile cap motion at the ground surface. Visualisation of in-plane deformation of the soil surface was made possible by the creation of a grid of shallow blue sand squares, as shown in Figure 3.7.

3.3.6. Test preparation and procedure

The majority of equipment and test methods employed during seismic centrifuge tests have been developed over many years and are not unique to this project, hence only brief explanations focusing on aspects particularly relevant to these tests are provided here.

The sand was placed in the model container by dry pluviation using the Schofield Centre's automatic sand pourer ensuring consistency, both with depth and location within a single model, and between models. During the pouring process, the sand pourer was periodically paused to allow placement of pore pressure transducers (PPTs) and accelerometers at pre-defined locations within the soil. Saturation of the model was carried out under vacuum, such that the air was first removed from the soil void space before the viscous methyl cellulose was pumped into the model via ports in the base of the laminar box. The automatic saturation system 'CAMSat' (Knappett, 2006; Stringer and Madabhushi, 2010b; Stringer, 2012) was used.

3.4. Testing

The Turner Beam Centrifuge was used for all of the tests of this project and is described in detail by Schofield (1980). Its working radius is 4.125 m, with models being loaded onto a swing at one end of the beam (a counterweight of appropriate mass being loaded onto a swing at the opposite end of the beam). The simulation of earthquake ground motion is achieved via the shaking of the entire model, including its container, using an actuator mounted on the centrifuge swing. For the last few years the Stored Angular Momentum (SAM) actuator has been used for all seismic tests (Madabhushi et al., 1998). It is capable of delivering constant amplitude and frequency pseudo-sinusoidal motions. For this suite of tests a single dynamic motion of frequency 50 Hz. <1.14 Hz.>, target amplitude 8.8 g <0.2 g>, and duration of 0.5 s <22 s> was applied for all tests (within the limits of repeatability that can be achieved by the SAM actuator).

3.5. Summary

Centrifuge modelling has been selected as the physical modelling method best suited to studying the nature and mechanism of the interaction between pile foundations and laterally spreading soil, and thus meet the objectives of this project. In light of the seemingly limitless configurations of soil profile, foundation properties, and seismic demand for which lateral spreading might be relevant (not all of which can be explicitly considered in a single suite of centrifuge tests), this project focuses on those characteristics of the system over which the engineer has the greatest control, yet for which only limited guidance is presently available. The test programme was somewhat adaptive, the findings from each new test informing and influencing the design of subsequent tests.

Section 4

Pile tip fixity

4.1. Introduction

This section considers the influence of some common pile tip fixity conditions on the response of small, structurally strong pile groups. Specifically, the soil and foundation response for a 'rock-socketed' pile group (test JH06) is compared to two 'end-bearing' pile groups (JH01 and JH04) having differing bearing layer relative density and pile tip embedment depth. Key results and findings are presented here, while a more detailed discussion can be found in Haskell (201x).

4.2. General pile group response and performance

Figure 4.1 shows a photo of model JH06 during excavation. The clamp block shown in the figure was designed to provide axial, lateral, and rotational fixity to the tips of the piles. The pile cap displaced approximately 8 mm <0.35 m> downslope during the test, most of this displacement accumulating in the first 5-6 cycles of strong shaking. It would be expected that the pile cap displacement is due almost entirely to pile flexure, given the clamped pile tips and rigid connection of the piles to the pile cap. However, the measured pile cap displacement is not consistent with the pile flexural displacements, as determined from the measured pile bending strains. Specifically, the pile head displacement due to pile flexure is approximately 40 % of the directly measured pile cap displacement, suggesting the assumption of perfect fixity at the pile tips may not be valid for this particular test. Approximately 1.5 mm <0.07 m> of upward displacement of the upslope pile is required for a lateral pile cap displacement of 4.5 mm <0.2 m> (the difference between the total and flexural pile head displacements at the end of strong shaking).

Figure 4.2 shows the time histories of lateral pile cap displacement for JH01 and JH04 alongside that for JH06. It is immediately apparent that both the magnitude and the accumulation of pile cap displacement have been affected by the change in pile tip conditions, with the displacement accumulating steadily throughout the entire duration of strong shaking for both JH01 and JH04. The ultimate/residual pile cap displacement for test JH01 is approximately twice that for test JH04 and three times greater than for test JH06.

Post-test measurements of the pile cap position indicate that essentially all the of the lateral pile cap displacement can be attributed to forward rotation/unstable overturning of pile groups JH01 and JH04, which suffered 4.5 and 3.9° of permanent forward-tilt, respectively (Figure 4.3, Table 4.1). It is clear from the bending moment and flexural displacement 'snapshots' of Figure 4.4 that without the lateral, rotational and (partial) axial restraint of the clamp block, the piles are unable to realise even 20% of the flexural resistance they previously did, in spite of their significant and realistic embedment into the dense base soil. It can be deduced from the combination of significant irrecoverable downslope pile cap displacement and the small flexural pile demands that the base soil is unable to provide sufficient support to pile groups JH01 and JH04. This is the principal reason for the dominance of the mechanism of unstable collapse.

4.3. Pile deformation and damage

Figure 4.4 shows the bending moment distributions for the upslope and downslope piles of the rock-socked pile group at the instant of peak pile head bending moment and also post-shaking. As expected, the piles deform in double curvature. The bending moment distributions differ between the upslope and downslope piles near the pile head/cap, with the peak moment for the downslope pile developing at the pile cap, while for the upslope pile it develops at approximately 3 diameters (3D) below the base of the pile cap. This difference can be attributed to shadowing and structural interaction between the piles (Section 2.3). Large bending moments also develop at the pile tips/clamp block for both the upslope and downslope piles.

As shown by the time history of Figure 4.5, the bending moments develop quickly in the first 7-8 cycles of strong shaking and increase much more gradually thereafter, reaching approximately constant amplitude cycling about an average value of 11200 Nmm <950 kNm> at 0.5 s <22 s>. The trend of increasing average bending moment arises from the progressive mobilisation of kinematic demands on the piles and pile cap over several cycles, while the dynamic cycling arises from a combination of dynamic kinematic and inertial demands. The constant average bending moment after 0.5 s <22 s> together with the eventual cessation of downslope pile cap displacement suggests that the full lateral demand of the soil has been mobilised over much of the depth of the laterally spreading layer by the latter cycles of strong shaking.

Unsurprisingly, the pile bending moment distributions for JH01 and JH04 differ from those of JH06, reflecting the difference in pile tip boundary conditions, but also the differing relative soil-pile displacement and mobilisation of kinematic between the stiff flexural and unstable collapse mechanisms. As for the clamped pile group, the piles deform in double curvature, with a peak bending moment developing at the pile head. However for JH01 and JH04 the peak bending moment at depth develops near the liquefied-base layer interface (as opposed to at the pile tips). The difference in flexural demand between the upslope and downslope piles (at the instant of peak bending demand) is much greater for JH01 and JH04, as compared to JH06. However, the peak bending moments that do develop do not exceed the flexural capacity of typical state-of-practice foundations of equivalent prototype diameter (Table 3.1).Unlike for the clamped pile group, the flexural demands for the unstable pile groups remain relatively consistent throughout the duration of strong shaking, after rising rapidly in the first 1-2 cycles (Figure 4.5). Again, this reflects the inability of the pile group to derive any additional lateral resistance from the bearing soil beyond the first few loading cycles.

4.4. Soil and pore pressure response in the liquefying layer

Figure 4.6 shows the lateral soil force (per metre) distributions on the upslope and downslope piles for test JH06 at the instant of peak force on the upslope pile and post-shaking, after the excess pore pressures have dissipated. During strong shaking the uppermost 110 mm <5 m> (approximately) of soil applies a downslope demand on the pile group. The lateral force form the spreading soil is mobilised most rapidly nearest the ground surface, reaching a plateau within the first 10-12 cycles of strong shaking. At progressively greater depth the lateral kinematic demand takes longer to mobilise. The largest lateral soil force is mobilised at the pile head (i.e. the shallowest depth), which is contrary to the force distribution that would be expected if the soil strength was proportional to the initial vertical effective stress. For tests JH01 and JH04 the distributed force acting on the upslope piles follows the same 'inverted' triangular distribution (Figure 4.6). The peak kinematic soil force acting on the piles of the unstable pile groups is only 35% of that mobilised for the clamped, stiff pile group and minimal mobilisation of pile demand at progressively greater depth occurs for the unstable foundations, in spite of the steadily accumulating soil displacement.

Figure 4.7 shows the dynamic vertical effective stress time histories in the liquefiable soil layer for test JH06 (the trace for 22 mm <0.9 m> depth uses pore pressure data from downslope of the pile group due to the malfunction of the upslope PPT at this depth). It is clear from the data that the soil and pore pressure response differs between these depths, with the soil nearest the ground surface never reaching a state of 'near-zero' vertical effective stress (a prerequisite for 'liquefaction'). By contrast, the soil at greater depth does reach this state, albeit transiently and only during the first 4-5 cycles of strong shaking. At still greater depth (Figure 4.7) significant excess pore pressures are generated and sustained for the duration of strong shaking, resulting in near-zero vertical effective stress and 'full liquefaction' of the bottom of the Hostun sand layer.

It is clear from Figure 4.7 that very strong cyclic dilation, negative excess pore pressure spikes, and transient soil strength increase occur near the ground surface throughout the duration of strong shaking, resulting in transient vertical effective stresses that are in excess of the initial vertical effective stress (and at times, in excess of the initial total stress too). Unsurprisingly, the peak kinematic soil forces on the upslope piles coincide with the peak soil effective stress. Using the Equation 4.1 it is possible to estimate the lateral earth pressure coefficient at the demand peaks. For a 'typical' demand peak late in the earthquake (i.e. once the full kinematic demand has been mobilised), lateral distributed force and vertical effective stress values of 6.9 N/mm <304 kN/m> and 20 kPa give rise to a lateral earth pressure coefficient of approximately 7.5 (assuming a shape factor, α, of 4.5). This value is larger than would be expected for the passive pressure coefficient (and peak friction angle) for a clean sand. However, it is likely that the transient peak effective stress used for the calculation is an underestimate of the true value very close to the pile due to the distance of the PPT from the pile face (Haigh, 2002; Gonzalez et al., 2009). Furthermore, the shape factor α is known to take values as large as 6 (Cubrinovski et al., 2006a , Broms, 1964). These considerations suggest that, in spite of the transient increase in pore pressure and loss of effective stress, the loose liquefiable soil is able to mobilise the full passive earth pressure on the upslope piles during the peak loading cycles.

$$p = \alpha K D \sigma_v' \qquad \text{Equation 4.1}$$

where D is the pile diameter

The near-field pore pressure response for test JH01 (Figure 4.8) reflects the difference in soil-pile interaction in the liquefying laterally spreading layer between the stiff flexural and unstable overturning mechanisms. Specifically, the cyclic dilation and transient strength recovery of the loose liquefying soil layer are much less intense for the unstable pile groups (JH01 and JH04), in spite of the soil, relative density, and base acceleration input being essentially identical for all of the tests. Assuming a shape factor of 4.5, gives rise to values of K in the range of 3.0-3.2 for JH01 and JH04, respectively (the values for JH01 for peaks before 0.35 s <15.4 s>, i.e. before the pile cap came into contact with the overhead LVDT, as discussed in Section 4.6). The true earth pressure coefficients are likely to be slightly lower than these estimates due to the distance of the PPTs from the piles. Even so, the lateral kinematic soil demand appears to be lower for the unstable overturning mechanism, as compared to the stiff flexural

4.5. Soil and pore pressure response in the base layer

For JH06 the dense base layer provides support (i.e. an upslope lateral force) to the pile group throughout the duration of strong shaking, indicating that the downslope displacement of the piles is greater than that of the soil at this depth. Excess pore pressure time histories from just above and just below the liquefied-base layer interface (Figure 4.9) suggest significant 'communication' of pore pressure between the two soil layers occurs, with the excess pore pressure developed at the top of the dense layer reaching approximately the same value as that at the bottom of the loose layer, both during and after shaking. As a consequence, the vertical effective stress at the top dense layer is near zero for the duration of strong shaking (Figure 4.10), but shows significant transient dilation and strengthening as is observed at other depths. By contrast, the vertical effective stress at the pile tip level is sustained above zero, though does decrease to approximately 20% of the initial, static vertical effective stress due to the development of excess pore pressure. The implication of this link in positive excess pore pressure development is that the loss of effective stress and softening of the bearing soil layer arises predominantly from, and is largely controlled by the pore pressure development of the loose, liquefying layer above (for the soil profile considered herein). This may have important implications for the support that the base soil can afford to the pile.

As for JH06, significant positive excess pore pressures are generated throughout the dense base layer for both JH01 and JH04. The free-field excess pore pressure time histories at the top and bottom of the base layer in test JH04 are shown in Figure 4.11. The pore pressure at the top of the layer rises immediately, reaching the level of the initial effective overburden stress within the first cycle of strong shaking. The excess pore pressure at greater depth takes slightly longer to develop, rising sharply in the first cycle before accumulating more gradually over the next 7-8 cycles to also reach the level of the initial effective overburden stress. The rapid development of positive excess pore pressure throughout the base layer in the first loading cycle (and the much more gradual build up thereafter) again suggests that rapid pore pressure communication from the loose liquefying later above is partly responsible for the base layer pore pressure response. Regardless of the cause, the significant excess pore pressure development in the base layer results in transient near-zero effective stresses throughout the layer and the potential for accumulation of permanent strains via the cyclic mobility mechanism (Hyodo et al., 2002).

Focusing on test JH01 (due to the greater number of pile strain gauges employed in his test), for the downslope pile (which mobilises slightly larger lateral resistance as it encounters 'undisturbed' soil downslope of the pile group), the earth pressure coefficient falls in the range 0.7-1.0 (depending on the shape factor), which is considerably less than the earth pressure coefficient for the liquefying soil above that is driving the pile group displacement. This suggests that the full passive pressure and potential lateral support from the base layer are not mobilised by the piles. This is consistent with the effect of the positive excess pore pressure on the mechanical behaviour of dense sand, specifically the softening of the stress strain response and, in particular, an increase of the strain required for significant strength recovery (Dungca et al., 2006). Stress-strain loops for the base layer (Figure 4.12) support this interpretation, indicating that up to +/- 0.2% cyclic strain the stiffness of the base soil is very small, permitting the displacement of the piles without the mobilisation of significant soil stress. Although the overall displacement of the pile group at the pile cap level is significant, the forward pile cap rotation implies relatively small cyclic strains at the pile tip level: strains insufficiently large to mobilise the full passive resistance in the base layer.

The forward-rotation of the pile group implies not only lateral compliance of the base soil, but also vertical or axial compliance. The bearing resistance of the pile cap and the hydraulic conductivity of the base soil strongly influence the strains, dilation, excess pore pressure development, and transient strengthening of the soil surrounding the pile tips, and consequently the mobilisation of lateral and axial pile capacity in the base layer (Stringer, 2012). For the scenario considered herein, the strong dilation near the ground surface and initial embedment of the pile cap into the ground surface imply the pile group weight is likely to be predominantly supported by the upper soil layer, in spite of the net downslope displacement of the liquefying soil. Consequently, the axial loads on the pile tips can reduce without the accumulation of settlement of the pile group. Owing to the relatively high hydraulic conductivity of the coarse Fraction C base soil, any dilation of the soil immediately surrounding the pile tips is accompanied by rapid pore pressure dissipation and flow of pore fluid towards the dilating region, permitting continued dilation and loosening of the bearing soil (Stringer, 2012). The sustained positive excess pore pressures and loosened soil surrounding the pile tips, together with the associated reduction in small-strain stiffness of the bearing soil offers one possible explanation for the limited offered to the pile group by the base soil, and is consistent with the observed mechanism of incremental unstable overturning of the foundation.

4.6. Pile cap demands

Figure 4.13 shows time histories of the kinematic and inertial pile cap forces for test JH06. It is immediately apparent from the data that the kinematic pile cap demand is significantly larger than the inertial demand for the duration of strong shaking beyond the first cycle. In spite of the constant amplitude input acceleration at the model base (and pile tips), the peak cyclic pile cap inertia force decays by approximately 60% over the first 6-8 cycles, with the cyclic peak forces remaining approximately constant thereafter. The kinematic pile cap force rises rapidly in the first 4-5 cycles after which it steadily decays as the strong shaking continues. The amplitude of cycling remains approximately constant throughout.

Figure 4.13 also compares the kinematic pile cap force to the resultant kinematic force on all four piles from the upper 90 mm <4.0 m> of soil (i.e. the approximate depth of soil that acts in a downslope direction on the piles during strong shaking). It is clear that the combined kinematic demand on the piles from the laterally spreading soil significantly exceeds the kinematic demand on the pile cap, and increases as strong shaking continues (albeit more gradually later in the earthquake) as the limiting soil demand is progressively mobilised at greater depth.

For JH01, the inertial and kinematic forces are similar in terms of the magnitude and phase of peak demand up to 0.35 s <15.4 s>, at which point the kinematic demand appears to reduce (Figure 4.14). This drop in demand coincides with the time of contact of the pile cap with the overhead LVDT. The apparent decrease in the kinematic demand between 0.35 s <15.4 s> and 0.65 s <28.6 s> reflects the progressive transfer of lateral force to the LVDT and away from the piles, until the LVDT rod yielded. It is thus an artefact of the equilibrium-based calculation of pile cap forces (or more precisely, the omission of the force from the LVDT rod form the equilibrium calculation). The time history of distributed lateral kinematic force on the piles confirms this interpretation, exhibiting approximately constant peak force until 0.35 s <15.4 s>, then increasing significantly with every cycle while the pile cap was temporarily restrained, before decreasing steadily from 0.65 s <28.6 s> onwards once this restrain was lost.

Pile cap inertia-kinematic force loops (Figure 4.15) show that the phasing and cyclic amplitude of the pile cap demands are the same before and after the temporary pile cap restraint, being in phase and of similar cyclic amplitude. A second or ‘double’ peak in the downslope kinematic demand develops after 0.35 s <15.4 s> and persists post-LVDT yield, suggesting that the pile cap restraint results in some anti-phase motion between the liquefying soil and the pile cap. This interaction is explored further in Section 5 which considers the response of pile groups with external pile cap restraint, analogous to the bridge abutments in Christchurch.

The average kinematic pile demand for test JH04 increases as strong shaking continues until approximately 0.57 s <25.1 s> when it plateaus. The interaction of inertial and kinematic demands appears to change at this time, from inertial demand peaks every second cycle before 0.57 s <25.1 s> to more uniform primary peaks occurring every cycle after this time. This change is reflected in the pore pressure response in the liquefied layer near the pile cap, with a notable reduction in the magnitude of the negative pore pressure spikes after 0.57 s <25.1 s>. This implies a reduction in soil strain, most likely due to a reduction in relative soil-pile cap displacement. Comparing the inertial-kinematic demand loops for JH01 and JH04, it is clear that the inertial demand is relatively more significant for JH01 which has a smaller pile embedment in the bearing layer.

The demand loops, particularly for JH01, resemble those presented by Knappett (2006) for pile groups in laterally spreading soil suffering a similar unstable overturning mechanism of response, suggesting that simultaneous peak inerital and kinematic demands and transient reversal of kinematic demand each cycle are characteristic of the unstable overturning failure mechanism. By contrast, the demand loops for test JH06 (which is dominated by a combined flexure-uplift mechanism) are much narrower, reflecting the larger magnitude of the kinematic pile cap demand as compared to the pile cap inertia. The demand loops are aligned with the long axis in the second and fourth quadrants, indicating that the peak inerital and kinematic demand are out of phase. This has implications for the combination of inertial and kinematic demands in simplified analysis methods. To date the available guidance for the combination of inertial and kinematic demands has focused on relative natural periods of the pile foundation (and superstructure, if present) and the soil column (Tokimatsu and Suzuki, 2009). However, this presumes that the dominant mechanism is pile bending and it is unclear how global pile group instability, rocking, or lack of pile tip restraint should be taken into account within this framework. The results presented here suggest that when overturning instability is the dominant mechanism, the peak inerital and kinematic pile cap demands should be applied simultaneously. However, it must also be recognised that the unstable collapse mechanism implies the mobilisation of much lower kinematic demand on the pile group than for the much stiffer pile flexure mechanism.

4.7. Summary

The results from tests JH01, JH04, and JH06 have highlighted several interesting details that build on the existing evidence and understanding of the behaviour of pile groups in laterally spreading soil. In particular, the significant lateral displacements of the pile cap over and above those attributable to pile flexure confirm the potential impact of imperfect pile tip fixity on the foundation's performance. Furthermore, the significant excess pore pressure generation on the dense base soil due to the 'communication' of pore pressure with the loose, liquefying layer above raises doubts as to the support this soil is able to provide to the foundation (should the pile not be 'rock-socketed' into underlying bedrock) and the necessary bearing layer embedment that might be required in order to achieve sufficient fixity for the realisation of the full pile flexural capacity.

In spite of some pile uplift the pile group response in test JH06 can be described as 'stiff', with significant soil yielding and relative soil-pile displacement accumulating in the upper liquefiable layer. The strongest dilation and soil strength recovery occurs nearest the ground surface where the confining stress is lowest (and hence the soil the most dilative), The dilation is sufficiently intense for the upper half of the loose soil layer to remain 'non-liquefied', essentially forming a saturated crust of non-liquefied soil that remains relatively stiff and strong during shaking and peak transient strength (corresponding with negative pore pressure spikes) at the instant of peak pile loading (reflecting the interdependency of soil stress mobilisation and relative soil-foundation displacement). Nearest the ground surface is also where the soil deformations are the largest, and the relative soil-pile deformation the largest (for the case of 'stiff' pile groups). By contrast, the response of pile groups JH01 and JH04 is best described as, 'unstable', the foundations being unable to resist the downslope displacement of the laterally spreading soil. As a consequence of the small global stiffness of the pile groups, the mobilised soil demands were significantly lower than for the rock-socketed pile group, in spite of the foundations being structurally identical. It is clear from these tests that the lateral and axial displacements of the pile tips and the strength and behaviour of the bearing soil layer require greater scrutiny than they typically receive at present.

Section 5

Superstructure restraint and abutment back-rotation mechanism

5.1. Introduction

It was suggested in Section 2 that the fixity and 'boundary' conditions of the bridge abutments in Christchurch (and also in Costa-Rica) fundamentally affected the nature of the deformation and damage suffered by the bridge foundations under the lateral spreading demands. This section first discusses the response of model pile groups that were designed to replicate the back-rotation mechanism observed in Christchurch. In Section 5.3 a rotation-based model (and simplified calculation) for abutment stability is developed on the basis of the residual deformation and damage of the Christchurch bridges. The model is then applied to the full dynamic time history data from centrifuge test JH05, which most closely replicates the conditions encountered in Christchurch. The design of the centrifuge models and the uncertainties in the back-rotation formulation are discussed and explored in Section 5.6. Parts of this section appear in the manuscript Haskell et al (201x), which is currently in press.

5.2. Model pile group response

5.2.1. General pile group response

Figure 5.1 shows the layout of test JH05 alongside a photo of the model prior to testing. The initial and final positions of the pile cap and the in-plane deformation of the soil surface are shown in Figure 5.2. Over the course of the strong shaking the pile cap displaces approximately 2.2 mm <0.1 m> laterally via the cyclic ratchetting mechanism described in Section 4, and the pile group and soil displacement cease as soon as the strong ground motion diminishes. The lateral pile cap displacement time history in Figure

5.3 indicates that the majority of the pile cap displacement accumulates in the first 2-3 cycles, between t = 0.18 and 0.35 s <7.9 and 15.4 s>. The ground surface displacement, like the pile cap displacement, accumulates most rapidly during the first cycles of strong shaking but, unlike the pile cap, continues to accumulate throughout the entire duration of shaking, stopping abruptly when shaking ceases. Together the pile cap and ground displacement suggest that the limiting strength of the soil is reached and the pile group behaves in a 'stiff' manner (Haskell et al., 2012a) such that the pile cap displacement is relatively insensitive to the continued ground displacement beyond 6 mm <0.26 m> (in this example) as the full passive pressure has been mobilised.

The time history of abutment/deck compressive force (Figure 5.4) supports this interpretation, the force increasing with each cycle for the first 3 cycles, then cycling around an average force of approximately 500 N <970 kN> for the remainder of the strong shaking. Similarly, the combined kinematic soil demand applied to the upslope piles by the laterally spreading soil at the pile head increases rapidly between 0.18 and 0.35 s <15.4 s> and only very gradually thereafter (Figure 5.5). The kinematic demand increases much more gradually at greater depth due to the slower accumulation of relative soil-pile displacement.

5.2.2. Near-field soil response

Pore pressure time histories from shallow depth indicate that the state of near-zero effective stress was achieved in the upper layer of soil (albeit very briefly at the very top of the laterally spreading layer). The time histories also indicate that strong cyclic dilation occurred throughout the duration of strong shaking, especially at shallow depth, resulting in transient effective stress recovery and strengthening of the 'liquefying' soil. Comparison of time histories of kinematic soil demand and vertical effective stress from approximately the same depth (Figure 5.6) confirm the coincidence of the transient peak kinematic forces and the dilation-induced increases of effective stress. Assuming a shape factor, α, of 4.5 gives a value of K of approximately 6.5-7.5 at the demand peaks, which corresponds to a mobilised friction angle of approximately 49°. Again, it is likely that the transient peak effective stress used for the calculation is an underestimate of the true value very close to the pile due to the distance of the PPT from the pile face and the stronger dilation upslope of the pile group at very shallow depths that has been observed in other tests (Stringer, 2012).

Post-shaking, the distribution of lateral soil force on the piles in the upper 100 mm <4.5 m> of the Hostun sand layer increases approximately linearly with depth and corresponds to an earth pressure coefficient of approximately 6, which is very close to the value estimated for the peak kinematic demand during shaking. This suggests that the soil upslope of the pile group reaches and remains at the passive failure condition during and after strong shaking, and that the kinematic demand acting on the upslope piles at shallow depth fluctuates with the dynamic effective stress, which varies due to the transient negative excess pore pressure spikes. The downslope piles, by contrast, are subjected to much lower kinematic demands from the spreading soil, suggesting significant 'shadowing' interaction between the upslope and downslope piles (Figure 5.7). This is discussed in more detail in Section 6.

5.2.3. Pile and pile cap deformation and displacement

The majority of the lateral pile cap displacement in test JH05 occurred as a consequence of rotation of the pile cap about the point of abutment-deck contact. On the basis of the geometry of the potentiometer and abutment fittings, and assuming small angle theory, a time history of pile cap rotation can be approximated (Figure 5.8). A residual (i.e permanent) back-rotation of approximately 2° remained after the strong shaking had ceased. 'Snapshots' of the pile bending moments indicate that the upslope and downslope piles tend towards deforming in single-curvature (Figure 5.9), in contrast to the double-curvature bending typically assumed to take place if pile cap rotation is not taken into account (and as was observed for pile groups JH01, JH04, and JH06 in Section 4). The bending moment distributions do differ between the upslope and downslope piles however, with the peak moment for the downslope pile developing at the pile head, while the peak moment for the upslope pile occurs at approximately 60 mm <2.6 m> depth, or roughly 6D below the pile cap.

An interesting aspect of the significant pile cap back-rotation is the large displacement of the pile tips (or significant flexural pile deformation) it implies. Figure 5.10 shows time histories of lateral pile tip displacement both with and without pile flexure (i.e the actual tip displacement, and the hypothetical tip displacement associated with the known pile cap rotation supposing the piles were infinitely stiff). The residual lateral tip displacement estimated in this way is approximately 3 mm <0.13 m> for the downslope pile and 5 mm <0.22 m> for the upslope pile. On this basis the residual pile cap rotation comprises approximately 40 % rigid rotation associated with pile tip displacement in the base soil, and 60% rotation due to pile flexure.

5.2.4. Demands acting on the pile cap

It is apparent from Figure 5.11 that the kinematic demand from the laterally spreading soil is the dominant demand acting on the pile cap, and that the pile cap inertia is relatively insignificant by comparison. The ultimate kinematic demand acting on the pile cap increases rapidly over the first few cycles of strong shaking. After this it cycles strongly about an average value of approximately 230 N <450 kN>. Figure 5.11 shows time histories of the kinematic pile and pile cap forces, and suggests that the two demands are of the same order of magnitude and are both initially mobilised rapidly.

However, unlike the pile cap demand, the kinematic demand on the piles continues to increase throughout the duration of strong shaking as the relative soil-pile displacement accumulates and the limiting soil stress is progressively mobilised at greater depth.

5.3. Abutment rotational stability model

By explicitly considering the rotational equilibrium of the abutments about their point of contact with the rigid bridge decks, it is possible to estimate kinematic demands from the laterally spreading backfill soil are needed to initiate pile yielding.

5.3.1. Model development for Fitzgerald Bridge

The Fitzgerald Avenue twin bridges are located to the northeast of the city centre, approximately 12 km upstream of the Avon-Heathcote Estuary. The bridges were constructed in 1963-64 and are essentially the same in terms of their design and detailing. The eastern bridge carries two lanes of city/south-bound traffic and the western bridge three lanes of north-bound traffic. Each bridge comprises two spans of approximately 14 m length, constructed of 21 precast prestressed concrete deck beams, transversely post-tensioned once in place and topped with a 125 mm thick reinforced concrete deck. At the abutments and central pier the beams bear on 15 mm (5/8") thick neoprene pads. The reinforced concrete abutments and central pier are supported by 300 mm (12") square reinforced concrete piles. The nine piles beneath each of the abutments are staggered front and back at a spacing of 2.7 m (between adjacent front piles, and between adjacent rear piles); the five front piles are raked at 1:4 and the rear piles are vertical. The eight piles supporting the each central pier are vertical and spaced at 1.5 m centres, and arranged in a single line.

The northern abutments of the bridges are on the inside bank of a bend in the river, and the soil here was already known to be susceptible to liquefaction, Bowen (2007) estimating that at the northeast abutment the soil between 2.5 and 17.5 m depth would liquefy in a strong earthquake. As expected, significant liquefaction and lateral spreading did develop on the north bank of the river as evidenced by ejected sand and ground cracks parallel to the river bank, the approximate locations of which are shown in Figure 5.12. Displacement of a retaining wall caused collapse of the approach road approximately 100 m to the north of the bridge (Figure 5.13). At the south (outside) bank some ground cracking was evident, but the lateral displacement of the bank was limited.

The laterally spreading soil served to drive the abutments towards the river. However, significant lateral translation of the abutments was impeded by the bridge deck, the abutments instead rotating about the point of contact with the deck. The central piers located in the river itself suffered relatively little deformation by comparison. The closing of joints and formation of compression creases and cracks in the asphalt of the decks imply the transmission of the lateral spreading forces through the bridge superstructures, consistent with observations from other earthquakes (Boulanger et al, 2005). The global mode of deformation of the bridges arguably reflects the combination of the lateral spreading demand (both inertial and kinematic) and the stiffness of the bridge superstructure.

The severity and distribution of damage to the bridges' abutments and foundations correspond to the spatial variability of the lateral spreading ground displacement, with the northeast abutment the worst affected, suffering a permanent (i.e. residual) 'back-tilt' of approximately 7.5 º. Cracks up to 10 mm width were evident in both the north and south abutments of both bridges, as well as in the adjacent reinforced concrete wingwalls (Figure 5.13). Several deck beams also suffered cracking and loss of concrete on their bottom faces, where the compressive force due to interaction with the displacing abutments would be greatest (Figure 5.13). At the northeast abutment settlement and lateral displacement of soil towards the river exposed the uppermost 400 mm or so of several of the reinforced concrete piles (Figure 5.13). The easternmost pile (raked at 1:4) had completely sheared through at its connection to the abutment base, exposing approximately 65 mm of the longitudinal reinforcing bars. Other piles had clearly-visible tension cracks on their front faces, consistent with the bending demand induced by the back-tilting of the abutments and the associated 20-25 cm lateral displacement of the tops of the piles.

The rotation of the abutment results in a less certain pile bending moment distribution than that associated with conventional pile head translation (Haskell et al., 2012b). Arguably the overall collapse load of the abutment is dictated by its rotational stability, as depicted schematically in Figure 5.14. The rotational resistance or capacity of the abutment is likely to be derived from some combination of sources, which might include shear and bending resistance offered by the piles, axial capacities of the piles (in both tension/pull-out and compression) and the associated frame action, base friction on the bottom face of the abutment, and perhaps some moment resistance at the connection between the abutment and the bridge deck. The demand on the abutment arises primarily from two sources, namely the kinematic demand from the laterally spreading soil acting on the back face of the abutment, and the inertial force from the abutment itself. By considering equilibrium of moments about the observed point of rotation (namely the point of abutment deck contact), it is possible to estimate the lower bound kinematic soil demand the abutment can sustain. These analyses do not take into account any contribution of abutment inertia force to either the rotational demand or resistance.

In the calculations the contributions from the axial response of the piles to the abutment's rotational capacity are omitted, as the geometry of the abutment and rake of the front piles result in a trajectory of pile head displacement that is approximately perpendicular to the piles' long axis, and the liquefaction of the foundation soil limits the axial capacity the piles are able to mobilise (Stringer and Madabhushi, 2013). Similarly, the contribution from interface friction between the bottom face of the abutment and the soil beneath is neglected, as liquefaction limits the shear stress that this soil can sustain and lateral spreading movement results in loss of soil contact on underside of pile cap. The cracks observed in the upper, exposed sections of the piles are consistent with flexural failure, and on this basis it is assumed that all nine piles simultaneously develop their full plastic moment capacity, M_p, of 69 kNm. Further details of the pile and abutment calculations are provided in Haskell et al. (201x)

The demand on the abutment from the laterally spreading soil depends on both the mobilised soil stress and the distribution of earth pressure on the back face (as this controls the lever arm of the resultant soil thrust force and thus the rotational demand). The distribution of earth pressure arises from the complex interplay of the dynamic response of the backfill soil, the inertia of the abutment, the magnitude of relative soil-abutment displacement, and the manner of wall movement. Studies of seismic earth pressure distributions on retaining walls offer some insight in this regard. However, a distinction should first be made between the conventional scenarios which are primarily concerned with active earth pressures due to a net movement of the wall away from the backfill soil, and the lateral spreading case considered herein, which (due to large free-field soil displacements) concerns the opposite sense of relative soil-abutment movement and, arguably the potential to reach a passive condition in the backfill soil.

Given the complex and interdependent soil, superstructure, and foundation stresses and deformations and the many uncertainties regarding the accumulation of lateral spreading displacement during shaking, and the variation of lateral displacement with depth, the limiting triangular passive pressure distribution has been assumed to act on the back face of the abutment, and resultant thrust force is assumed to act at 1/3 H above the abutment base. As shown in Figure 5.15, the northeast abutment piles do not extend deep enough to reach competent non-liquefied soil, hence it is assumed that a single plastic hinge forms at the abutment-pile connection and the piles deform in single-curvature bending. The moment capacity of the piles, calculated on the basis of field measurements and material properties typical of the era of construction, is 69 kNm and the shear transfer at the abutment base is thus estimated to be 11.4 kN per pile, which assumes a l distribution of lateral resistance from the liquefied soil that increases linearly with depth:

$$V_p = \frac{3M_p}{2L} \qquad \text{(Equation 5.1)}$$

Where L, in this case, is the full length of the pile, M_p is the pile plastic moment capacity, and V_p the corresponding shear force at the abutment base.

Further, assuming that the centre of rotation is approximately quarter of the depth of the deck beams above the soffit, the global rotational moment capacity of the abutment is 844 kNm (which corresponds to a moment capacity of 68.6 kNm per metre width of the abutment):

$$M_{abutment} = \sum M_p + \sum V_p\left(H - \frac{3}{4}h_{deck}\right) \qquad \text{(Equation 5.2)}$$

Where H is the height of the abutment, h_{deck} is the deck thickness, and $M_{abutment}$ is the global rotational moment capacity of the abutment.

On the basis of a resultant soil thrust force acting at 1/3 H above the abutment base, a force of 56.1 kN per metre width of the abutment is calculated. Assuming the backfill soil is dry and has a unit weight of 16 kNm^{-3}, this corresponds to a horizontal earth pressure coefficient of approximately 0.96:

$$p_{soil} = \frac{M_{abutment}}{w\left(\frac{2}{3}H - \frac{3}{4}h_{deck}\right)} \qquad \text{(Equation 5.3)}$$

$$K = \frac{2p_{soil}}{\gamma_{dry}H^2} \qquad \text{(Equation 5.4)}$$

Where w is the width of the abutment, p_{soil} is the soil thrust force per metre width of the abutment, γ_{dry} is the soil unit weight (the soil is assumed to be dry), and K is the horizontal earth pressure coefficient.

The estimated earth pressure coefficient of 0.96 is well below the limiting passive earth pressure coefficient that might reasonably be expected for the fine sand behind the abutment (i.e. somewhere in the region of 3.4 to 4.2, assuming a critical state friction angle, ϕ_{crit}, of 31 to 34° and a dilatancy, ψ, of 2 to 4°), which implies that pile damage and abutment rotation will occur before the passive earth pressure is fully mobilised throughout the depth of the backfill soil. The kinematic demand from the laterally spreading soil is therefore sufficiently large to cause back-tilting instability of the abutments and induce failure of the piles, however the quantitative result is, of course, subject to considerable uncertainty. The actual kinematic demand mobilised may have been even lower, if significant inertial demands from the abutment acted in phase with the peak kinematic demand. The assumption of earth pressure distribution and pile plastic hinge locations and bending moment distribution is also a source of uncertainty (Ledezma and Bray, 2010). The sensitivity of the predicted abutment capacity to such uncertainties id explore in Section 5.6.

5.3.2. Model application to other Christchurch bridges

The back-rotation calculation has been applied to several other bridges that suffered damage and deformation due to lateral spreading in the Christchurch earthquakes, with appropriate modifications made for the geometry and soil profile for each bridge. The earth pressure coefficient corresponding to structural failure of the piles was found to fall in the range of 0.63 to 1.38 for all of the bridges with abutments supported by reinforced concrete or prestressed concrete piles, and was somewhat larger (4.55) for Anzac Bridge, which has abutments supported by steel H-piles. The key parameters and results for the surveyed bridges are presented in Table 5.1. A detailed account of the damage and deformation patterns for the surveyed bridges and the case-by-case application of the back-rotation model can be found in Haskell (201x).

5.4. Application of the abutment rotation model to dynamic centrifuge data

Having time histories of kinematic and inertial pile cap demand and pile shear forces and bending moments at the connection to the pile cap it is possible to replicate for the centrifuge teat data the back-rotation equilibrium calculation developed in Section 5.3 for the Christchurch bridge abutments. Unlike for the field cases, full dynamic data is available permitting the calculation and comparison of time histories of rotational demand (from F_{PCK} and F_{PCI}) and resistance (from V_{piles} and M_{piles}), and the evaluation of the assumptions made in the application of the calculation to the field cases, such as the decision to omit inertial demands and any contribution to the abutment/pile cap rotational resistance from axial pile forces. Figure 5.16 shows the unfiltered and low-pass filtered pile cap rotational demand and resistance time histories, determined on the basis of the following assumptions:

- The kinematic pile cap force acts at 10 mm <0.44 m> above the underside of the pile cap (resulting in a lever arm of 105 mm <4.6 m> about the point of rotation),

- The pile cap rotates about the point of contact with the deck fitting, which is located 75 mm <3.3 m> above the top face of the pile cap, and

- The pile shear forces and bending moments act at the centre of the uppermost strain gauge bridges, which are 5 mm <0.22 m> below the underside of the pile cap (resulting in a lever arm for the pile shear forces of 120 mm <5.28 m>).

It is evident from the low-pass filtered traces of Figure 5.16 that during strong shaking there is very good agreement between the calculated rotational demand and resistance. The pile cap inertial demand was initially included, but was found to make a negligible contribution to the total rotational demand, supporting the omission of inertial demand from the calculations. The full dynamic (i.e. unfiltered) demand and resistance time histories show different amplitudes of dynamic variation, with the demand apparently exhibiting much larger cyclic variation than the resistance. The rotational demand and resistance increase rapidly during the first few cycles of strong shaking, but then steadily decrease for the remainder of the strong shaking duration. Post-strong shaking, the rotational demand and resistance both decrease, but diverge, with the residual resistance remaining much larger than the residual demand.

5.5. Evaluation of the model assumptions

The close agreement between the dynamic abutment rotational demand and resistance supports the formulation of the back-rotation equilibrium as developed in Section 5.3. In particular, it supports the assumption that the pile shear forces and bending moments provide the majority of the abutment's resistance to rotation and that, for pile groups of similar geometry (i.e. relative few rows of piles and having a width much less than the length of the piles), the axial components of the rotational resistance are relatively small. This is unsurprising given the relatively low axial demand on the pile group (Knappett, 2006) and the predominantly lateral trajectory of pile tip displacement associated with the back-rotation mechanism (Figure 5.17).

It is also clear from the centrifuge test results that the inertial demand on the pile cap/abutment, and its contribution to the total back-rotation demand, is much smaller than the kinematic demand from the laterally spreading soil. Although the abutment and superstructure accelerations are likely to vary with the characteristics of the input motion (in particular the frequency content of the motion, with respect to the resonant frequencies of the soil column and the pile group), the relatively low mass of the pile cap/abutment structure limits the magnitude of the inertial forces that arise. However, this is only relevant because the bridge abutments are sufficiently stiff to mobilise such large kinematic demands from the spreading soil, thanks to the restraint provided by the bridge superstructure/deck fitting. Nonetheless, for the scenario encountered in

Christchurch, the back-rotation formulation developed here appears to be reasonable for a first approximation of the susceptibility of bridge abutments to the back-rotation failure mechanism.

5.6. Sensitivity analysis for the back-rotation model

In spite of its simplicity, the back-rotation formulation offers a 'back-of-the-envelope' or first order check of the stability of an abutment against the back rotation failure mechanism that is based on readily available and physically meaningful parameters. There are however limitations implicit to the representation of a complex dynamic problem with a deterministic pseudo-static analysis. A better understanding of the influence of assumptions implicit to the model and a more deliberate application of the back-rotation model in practice can be achieved via a straightforward sensitivity analysis (Haskell et al, 2012a). Table 5.2 summarises the primary parameters and uncertainties that feature in the abutment rotational equilibrium calculation. In the absence of any detailed or rigorous statistical information, it is still possible to estimate realistic ranges of variation for each parameter and thus develop some sense of the possible effect and implications of different parameter variations in terms of the predicted back-rotation capacity.

Considering, for example, the South Brighton and Fitzgerald Avenue bridges, Table 5.2 summarises possible ranges for the different input parameters, with comments alongside detailing the physical basis for the selected values. Figure 5.18 shows the effect of these variations on the calculated earth pressure coefficient corresponding to pile failure. It is apparent that for both bridges, the predicted capacity is particularly sensitive to the abutment height, which affects both the magnitude of the maximum soil force (that could potentially be mobilised) and the lever arm of the soil thrust. For the South Brighton bridge the predicted capacity is quite sensitive to the location of the centre of rotation, which is due to the significant depth of the bridge's deck beams and thus the large variation in the height of the centre of rotation associated with shifting it from the soffit to the top of the deck beams.

So, for these examples, it is apparent that the geometry of the abutment is a key factor affecting its susceptibility to back-rotation failure during lateral spreading. As well as providing information regarding the most critical parameters and assumptions in the calculation, the simple sensitivity analysis signposts possible design changes (for new bridges) or remediation options (for existing bridges) to optimise and improve an abutment's rotational resistance. Supposing, for example, it is not feasible to reduce the abutment height (as this might imply altering the reduced level of the deck or reducing the space beneath the bridge), some increase in rotational resistance might be achieved by designing the abutment-deck connection to pivot about the soffit of the deck. A more significant improvement might be possible via a reduction of the back rotation demand, whish could be achieved by lowering the height of the backfill soil behind the abutment. An accompanying articulated approach/'settlement slab' Priestley et al. (1996) would be required at the approach to allow traffic onto the bridge and to accommodate deformation of the soil or structure (and ensure the bridge remains useable) should lateral spreading occur. Figure 5.20 shows that lowering the backfill to 50% of the abutment height increases the earth pressure coefficient for pile failure by a factor of roughly 2.6 – 2.9 for the two bridges considered herein.

5.7. Summary

The effect of superstructural restraint on the pattern of abutment deformation in laterally spreading soil was evident from the response of bridges in the 2010-2011 Christchurch earthquakes. Centrifuge tests replicating this restraint have demonstrated the consequences for the deformation of the abutment, the abutment piles, and the interaction between the laterally spreading soil and the foundation. In particular, it has been shown that even for very strong stiff abutment piles, the force from the bridge deck/superstructure may be the primary source of lateral resistance for the abutment. This transmission of demand to the superstructure should be accounted for in the design of the bridge deck and abutment-deck connection. It also represents a potential opportunity to exploit the additional source of lateral resistance in the design of the foundation.

Modelling of the abutment back-rotation mechanisms is possibly by considering moment equilibrium of the abutments about the observed point of rotation, the point of abutment-deck collision, and assuming a distribution of pile stresses consistent with the damage sustained by the abutment piles (where visible), the global rotational capacity of the abutments can be estimated. Neglecting inertial demands, and assuming a triangular earth pressure distribution, horizontal earth pressure coefficients corresponding to rotational abutment failure are found to fall in the range of 0.63 to 1.38 for all of the bridges supported by reinforced or prestressed concrete piles, regardless of the details of the pile design and arrangement. A larger value of 4.55 is estimated for the abutments supported on steel H-piles. The assumptions made in the back-analysis of the Christchurch bridges were supported by the successful application of the model to the dynamic data from centrifuge tests JH05.

The predicted earth pressure coefficients fall within the limiting value corresponding to full mobilisation of passive earth pressure throughout the backfill soil that might reasonably be expected for typical values of friction angle and dilatancy, confirming that the kinematic demand from the laterally spreading soil was more than sufficient to induce the observed failure mechanism. The prevalence of the back-rotated abutment deformation in Christchurch suggests that this mechanism may dominate the response of bridges with longitudinally stiff and strong superstructures when they are subjected to lateral spreading demands. For such bridges, this mode of deformation should thus be considered in the design of the abutment piles and the assessment of the lateral and rotational capacity that must be provided to the abutments to ensure the abutment rotations are minimised. For modern analysis methods, this can be achieved though the careful specification of appropriate pile head boundary conditions consistent with the restraint provided by the stiff superstructure (for example Cubrinovski et al., 2014).

Section 6

Lateral pile interaction effects

6.1. Introduction

As discussed in Section 2, there is relatively little guidance available at present concerning lateral pile interaction in laterally spreading soils, regardless of the dominant mechanism of response. Yet it is necessary to account for structural and soil-deformational interaction effects if features such as differing locations of peak pile bending demand for different piles in a group are to be captured by analyses and properly accounted for in design. This section compares the kinematic soil demands on the upslope and downslope piles for all of the pile groups and response mechanisms observed in this suite of experiments.

6.2. Unstable overturning mechanism

Figure 6.1 shows the kinematic demand on the upslope and downslope piles (i.e. lateral force per unit length) from the laterally spreading soil for tests JH01 (5D pile spacing) and JH02 (2D pile spacing). The data presented correspond to the uppermost section of the piles, which is where the peak measured lateral force (per unit length) developed for both the 5D and 2D pile groups. It is clear from the data that the soil force on the upslope pile is greater than that on the downslope pile for both pile groups, indicating that some 'shadowing' interaction is occurring in both cases. However it is also clear that the ratio of downslope to upslope pile force at the peak of each cycle evolves during shaking. Figure 6.2 shows the 'demand' (i.e. positive peak) ratio for the two pile groups for each demand cycle, together with the peak upslope force per metre length plotted on the secondary y-axis.

Focusing on the 5D group, the downslope-upslope pile force ratio and the demand on the downslope pile takes approximately 10 cycles to reach its peak, which roughly coincides with the time of contact with the vertical LVDT (as discussed in Section 4). Between cycle 10 and cycle 24 the ratio remains approximately constant, at 0.9. A sudden reversal of the ratio (i.e. the force on the downslope pile becomes larger than that on the upslope pile) occurs at the end of strong shaking. A likely explanation for this is that the motion of the pile cap does not diminish as quickly as that of the soil at the ground surface and the pile cap is tending to displace further downslope than the surrounding soil, resulting in the observed relaxation of the kinematic demand on the upslope pile. However, the 'intermediate' soil between the upslope and downslope piles displaces with the pile group and the upslope pile, placing a relatively larger kinematic demand on the downslope pile.

The magnitude and evolution of the downslope-upslope pile force ratio throughout strong shaking is significantly different for the 2D pile group. Unsurprisingly, the downslope-upslope force ratio remains well below that of the 5D group until near the end of shaking, only increasing above 0.6 when the base acceleration amplitude starts to diminish and the kinematic demand on the upslope pile begins to drop (and again, the intermediate soil between the piles moves with the pile group while the force on the up-slope pile relaxes). The ratio rises rapidly in the first three cycles, but drops as liquefaction develops. The steady increase in the ratio that follows is likely due to the relatively slow accumulation of relative soil-foundation displacement due to the low global stiffness of the pile group.

Further interpreting the time histories of upslope and downslope pile kinematic force, evident in Figures 6.1a & 6.1c is a 'double peak' that begins to appear in the kinematic demand on the downslope pile. A possible cause for this is a slight phase difference between the displacement of the soil surrounding the pile head and the piles themselves. Specifically, the first (smaller) peak occurs when the soil and pile group are displacing together, and is due to the intermediate soil pushing on the downslope pile (hence there is no corresponding peak in the demand on the upslope pile). The second peak occurs when the pile group displacement slows or stops, but the lateral spreading soil displacement momentarily continues, hence the significant peak in the demand on the upslope pile that develops at the same time.

6.3. Stiff flexural and back-rotation mechanisms

Figure 6.3 shows the kinematic pile demands from the top of the laterally spreading soil layer for tests JH05 and JH06, which both exhibited stiff (albeit different) response mechanisms. Comparing Figure 6.3 to Figure 6.1, it is apparent that the shadowing interaction between the upslope and downslope piles is much more significant for the 'stiff' mechanisms of foundation response. This is further illustrated by the ratio of downslope-upslope lateral force at the demand peaks for JH05 and JH06, as shown in Figure 6.4. Specifically, after the first 9-10 cycles (for JH05) or 4-5 cycles (for JH06) the peak demand ratio remains fairly constant, taking a value of approximately 0.3-0.4 for both tests.

The calculation of the peak demand ratio was somewhat more straightforward for JH05 and JH06, as compared to the unstable pile groups, as the peak forces in the upslope and downslope piles occur simultaneously. The absence of any significant 'secondary' peaks in the downslope pile force (i.e. peaks not accompanied by a peak in the upslope pile force) following the first 9-10 cycles reflects the smaller influence of pile cap inertia on the response of the foundation for the stiff pile groups. It is likely that the relative soil-pile displacement remains too large over the course of each cycle and the incremental pile cap displacement too small for the intermediate soil to apply a secondary peak force as a result of downslope displacement of the pile cap.

6.4. Comparison between mechanisms

As noted in Section 6.2, the peak demand ratio for the 5D pile group (test JH01, that suffers unstable overturning failure) reaches a constant value after approximately 10 cycles, similar to the stiff pile groups (JH05 and JH06). This would at first tend to suggest that sufficient soil-pile displacement has accumulated to fully develop the soil deformation mechanism around the upslope and downslope piles. However, the steady-state ratio for the overturning mechanism is approximately 0.9, i.e. it is considerably larger than the steady-state ratio of 0.3-0.4 observed for the stiff pile groups. The values for the stiff pile groups are consistent with those reported by others (albeit for non-liquefied soil). For example, Kim and Yoon (2011) quote factors of 0.25-0.6 for 4D-6D spacing for monotonically loaded active pile groups in dry sands.

Without reliable measurements of dynamic soil-pile displacement it is difficult to confirm the physical basis for the steady-state demand ratio for the overturning pile group. One possible explanation is that differing displacement is required to mobilise the peak demand on the downslope and upslope piles. Specifically, it appears that the peak demand on the downslope pile is mobilised at a smaller global relative soil-foundation displacement than the peak demand on the upslope pile. The steady-state demand ratio may thus reflect approximately uniform cyclic relative soil-pile displacement for test JH01 for the majority of the strong shaking duration, rather than the full mobilisation of the soil deformation mechanism around the upslope and downslope piles.

6.5. Summary

it is clear from the test results that some 'shadowing' interaction occurs at a pile spacing of 5D, even when the soil has apparently liquefied. However, as the transient dilation-induced pore pressure suction spikes and temporary soil strength recovery approximately coincide with the kinematic pile demand peaks the mechanism of deformation of the soil around the piles likely corresponds to a non-zero (i.e. non-liquefied) soil stress state. It may therefore be appropriate to adopt lateral pile interaction factors from non-liquefied tests where liquefaction/lateral spreading-specific factors are unavailable, for cases where strong soil dilation and temporary strength recovery are expected.

It is also apparent that the mechanism of pile foundation response affects the mobilisation of demand on different piles in the group and thus the severity of shadowing interaction between the upslope and downslope piles. The differing lateral pile group interaction between the unstable overturning and stiff flexural/restrained pile groups can be explained with reference to the inferred cyclic relative soil-pile group displacement. Specifically, it appears that the lateral kinematic demand on the upslope piles depends largely on the relative soil-foundation displacement, while the demand on the downslope piles depends also on the pile group displacement in an absolute sense, due to the retained soil between the upslope and downslope piles. Therefore, the demand on the downslope piles of the unstable pile groups is almost as large as the demand on the upslope piles of these groups, which is not the case for stiff pile groups. The specification of reduction factors for lateral interaction effects thus needs to account for the mechanism of pile group interaction.

Section 7

Conclusions

It has been suggested that different (and justifiable) mechanisms of response of piled foundations subjected to lateral spreading demands might be readily achieved if the foundation fixity conditions are altered. Through only relatively minor modification of the model foundation fixity conditions in centrifuge tests intended to represent this general scenario, three distinct mechanisms have been reproduced – unstable overturning, stiff flexure, and back-rotation – and very different foundation response from one test to the next has occurred as a result.

For the short-span bridges of Christchurch, only centrifuge model JH05 satisfactorily captured important details of the soil-foundation/abutment-superstructure interaction under lateral spreading loads. In other words, it has been identified (in the field), and subsequently confirmed (in the suite of centrifuge tests) that the potentially-significant restraint of the abutment by short-span bridge decks ought to be accounted for in the design or evaluation of the likely performance of the abutments and substructure of such bridges.

The differing foundation response between the centrifuge tests that has occurred as a result of changes to the piled foundation's boundary conditions is not purely of academic importance, nor relevant only to the details of centrifuge test design. Rather, it highlights the potential for misinterpretation of the soil-foundation interaction and poor prediction of abutment displacements and pile demands if the mechanism of response is incorrectly identified in the design/analysis process. For example, the back-tilt/rotation-dominated abutment response in Christchurch led to moderate damage to abutments, piles, and bridge decks, but left the bridges more or less serviceable (pending minor road repairs) immediately after the event, because the overall displacement of the abutments was relatively limited. With this mechanism in mind, the emphasis for future design of such bridges might be the limiting of abutment rotation and slumping of the approach embankments with the aim of minimising damage to the substructure and ensuring the bridges are immediately passable by emergency vehicles following the event. However, if (in the design of similar bridges) this response is forgotten and a different mechanism is assumed to govern the abutment response, the emphasis might be other, potentially less-important aspects of the foundation design and these (apparently) more important and relevant performance criteria may be overlooked.

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