SEISMIC BEHAVIOR
OF FOUR-CIDH PILE SUPPORTED FOUNDATIONS

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Abstract

This paper discusses the results of two large-scale models of Four-Cast-In-Drilled-Hole (FCIDH) pile supported bridge piers tested under bi-directional reversed cyclic loading conditions. The test units were designed according to state-of-the-art bridge design requirements. The pile cap of the first test unit contained conventional reinforcing detailing while the second unit had headed bars. The test units performed well in terms of the ductility and energy absorption capacity. This paper reports the main findings of the test program and also discusses the implications in design of FCIDH pile supported foundations of bridge piers.

Introduction

The seismic behavior of the FCIDH pile supported footing systems is the focus of this paper. In California a Capacity Design is followed to ensure the development of plastic hinges in columns and to keep all other bridge elements, including the foundation, elastic. In general, the number of piles is obtained by distributing the column shear force, determined for the plastic hinge flexural overstrength, evenly among piles. The design for shear in the piles is consistent with this even distribution of forces. However, when reinforced concrete piles are elastic, the distribution of moment and shear force in the footing and in the piles can be significantly affected by the axial force in the piles, because of the dependency of the flexural stiffness on axial force. The moment and shear force in the piles are also affected by the rotation of the pilecap caused by the vertical stiffness of pile-soil interaction, and lateral passive soil pressure on the vertical face of the footing. Furthermore, the influence of the three dimensional geometry of the foundation on the shear direction of the elastic pile can also affect the magnitude of the bending moment acting on the piles.

The experimental program involved two half-scale seismic tests on full column-footing-pile assemblies. The test units were capacity designed using conventional and headed reinforcement. The column-pilecap and pile-pilecap joint regions were designed using the external joint strut approach proposed by Priestley et al. (1996).

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Experimental Work

To investigate the distribution of shear forces in FCIDH piles and the overall seismic response in FCIDH pile supported footings, two half-scale units were designed in accordance to current seismic design recommendations (ATC, 1996). The main difference between the test units was the amount of reinforcement and type. Unit-1 was designed with conventional deformed bars and details. Unit-2 differed from Unit-1 in that headed reinforcement was used throughout, except or the column and pile transverse reinforcement. The units were tested twice, each under a different boundary condition. In the first test, termed here Loading Phase 1, restraint devices were placed to simulate passive pressure conditions. Fifty percent of the lateral load applied to the column was resisted by the passive pressure mechanism and the piles resisted the remaining fifty percent. In the second test, or Loading Phase 2, the restraint devices were removed to represent gapping between the soil and the piles. Inelastic pile behavior was expected in this phase of testing.

The test units were the prototype pier consisted of FCIDH piles, a pilecap, a single column and a superstructure. The reinforcement details of the prototype were not utilized in the design of the test units. In order to examine the seismic performance of a FCIDH pile supported footing under laboratory conditions, the test units were built at one-half scale and without the superstructure.

The geometry and general reinforcement details of Unit-1 and Unit-2 are shown in Fig. 1. For desirable seismic response the units were designed so that a plastic hinge would develop at the base of the column. Using capacity design principles, the pilecap and joints were designed for the maximum possible forces that would develop in the column plastic hinge, considering potential strain hardening and uncertainties in material strengths. With assumed material strengths f'_c = 27.6 MPa and f_{ye}= 455 MPa. Since there were orthogonal and diagonal loading directions with two conditions of pilecap restraints in each loading direction, four loading cases were considered for the design of the test units. Unit-2 was designed after the test on Unit-1 had been completed. Unit-2 incorporated minor modifications except pilecap reinforcement, based primarily on the experience gained from testing the Unit-1, and used headed reinforcement to improve anchorage of reinforcement.

The column longitudinal reinforcement ratio of ρ_l = 0.026 and was subjected to an arbitrary axial compression equal to $0.16\,A_g f'_{ce}$. the volumetric confinement ratio of the column was ρ_s = 0.0112 and exceeded the required value of $\rho_{s,req}$ = 0.0089.

The piles of the two units were designed for the worst possible scenario, expecting pile plastic hinging caused by loading in the diagonal direction without restraint being provided by the pilecap passive pressure mechanism of lateral force resistance. A pile longitudinal reinforcement ratio of ρ_l = 0.0089 was determined to ensure the piles would remain elastic in Loading Phase 1. This reinforcement ratio was determined through an iterative process, considering all possible loading scenarios assuming that lateral loading, when applied in the diagonal direction, resulted in no additional axial force being developed in the mid-piles. The removal of the passive pressure mechanism in Loading Phase 2 meant that lateral forces applied at the top of the column had to be resisted entirely by the piles. In this situation the mechanism of plastic deformation was expected in the piles since the ratio of the yield strength of the pile group to the maximum applicable lateral load was 0.84 for both orthogonal and diagonal direction loadings. Consequently, the piles were detailed for ductility to enable the development of plastic hinges immediately below the pilecap face. Transverse reinforcement with ρ_s = 0.0087 was provided to satisfy the seismic design requirements for California bridges (ATC, 1996).

Assuming that the plastic hinge length and the curvature distribution in the piles are independent from axial force, the lateral force applied at the top of the column would can distributed in the piles in proportion to the secant flexural rigidity, providing that the length of the piles to the point of inflection is the same. The shear force for the design of the piles was derived using this concept and the compression pile was critical. The shear demand in tension pile was small enough to be neglected. With a shear strength reduction factor of ϕ_s = 0.85, the shear capacity of the compression pile greatly exceeded the demand.

Shear forces in the piles, at the ultimate curvature of the pile with highest compression, were apportioned in proportion to the secant flexural rigidities of piles. The secant flexural rigidity of each pile was obtained from moment-curvature analyses for the piles with different axial forces. A significant portion of the toal shear force was apportioned to the piles in compression.

The pilecap joints of the test units were designed with reduced amounts of reinforcement by explicitly identifying an internal force flow (ATC, 1996; Priestley et al. 1996). The joint principal tensile stress, pt, was calculated to determine whether the joint reinforcement was needed to transfer joint forces.

The average principal tensile stresses of the joints at the overstrength were estimated to be $0.52\sqrt{f_c^{'}}$ [MPa] and $0.09\sqrt{f_c^{'}}$ [MPa] for the column-pilecap and the pilecap-pile joints, respectively. When comparing these values to the joint design threshold values, it was concluded that only the column-pilecap joint should be detailed to ensure appropriate force transfer mechanism for satisfactory internal force flow through the joint (ATC, 1996; Priestley et al., 1996). Because $p_t \leq 0.29\sqrt{f_c^{'}}$ [MPa] for the pilecap-pile joint, joint shear cracking was not expected, and only nominal joint reinforcement satisfying $0.29\sqrt{f_c^{'}} / f_{yh}$ was provided in the form of spirals.

The longitudinal column bars were extended into the joint as close to the bottom pilecap reinforcement as possible. The embedment length of the column bars was 711 mm, which was almost the minimum required anchorage length obtained for #8 bars.

Tests on column-pilecap connections (Priestley et al., 1996) have indicated that to ensure the pilecap reinforcement remains elastic, the flexural reinforcement must be placed within an effective width of $b_{eff} = D_p + 2\ d_f$ (Ingham, et al., 1994), where D_p is the diameter of the column and d_f is the effective depth of the pilecap. For anchorage, this reinforcement had 90^o hooks at each end, extending down the vertical face to 254mm from the pilecap soffit.

It was assumed, as is common design practice, that the critical moment in the pile cap would develop in the vertical plane at the column face. This is non-conservative, as demonstrated by the test performance of Unit-1, and that modification was made for Unit-2, as discussed subsequently.

In the test units the piles were connected to the test base using a pin detail, see Fig. 2. The pin connection between the piles and the test base was achieved by terminating all the longitudinal column reinforcement just above the test base, reducing gross area of the pile circular section from 508 mm to 203 mm at the interface of pile and test base, and by providing a plain round steel rod at the centers of the piles.

Test Set-up and Procedure

The units were tested under bi-directional reversed cyclic loading conditions. Fig. 3 shows the test set-up of Unit-1. Two 2.5 MN capacity hydraulic actuators were placed at 90^o atop the column. Lateral force was applied to the column with these actuators. Axial load was applied to the columns using four center-hole jacks. The test units were subjected to two phases of loading. In the first phase, lateral actuators were placed at the level of the pile cap and force-controlled to resist 50% of the lateral force applied to the column. In the second phase of loading, the passive pressure restraint was removed to simulate gapping between the pile cap and the soil. Only the results of the first phase of loading are discussed in this paper.

The test was performed quasi-statically with force- and displacement-controlled cycles. The first part of the test consisted of force-controlled cycles at 25%, 50%, 75% and 100% of the theoretical first yielding of the longitudinal reinforcement in the column. The following loading steps, beyond theoretical first yielding of the longitudinal column bar in the column, were displacement-controlled. Using the measured first yield displacements in all the loading directions, an average reference yield displacement corresponding to system's displacement ductility, , was derived from the following equation:

$$\Delta_{\mu 1} = \Delta'_{y(ave)} \frac{M_y}{M'_y} \tag{1}$$

where $\Delta'_{y(ave)}$ is the average system displacement for all loading directions at the first yield of the column, M'_y is the first yield moment and M_y is the reference yield moment of the column.

Displacement-controlled reversed cyclic loading was applied afterwards to $\mu_\Delta = 1$, 1.5, 2, 3, 4, 5, where $\mu_\Delta = \Delta / \Delta_{\mu 1}$ and Δ is the lateral displacement at the top of the column. Lateral loading was applied to each normal direction with two cycles and each diagonal direction with one cycle at each system displacement ductility level in order that all the structural members experience the same level of loading.

Test Results

Unit-1

In Loading Phase 1, the column developed a plastic hinge as expected. However, unexpected, and difficult to repair, spalling of the concrete cover was observed at the pilecap soffit during this phase of loading. Spalling was caused by straightening of 90^o J-hooks that had been placed as shear reinforcement in the pilecap, see Fig. 4. Fig. 5 plots the measured lateral force-displacement hysteretic response in the E-W direction of Unit-1 during Loading Phase 1. Th e response of the unit was stable and the loss of the concrete cover in the soffit of the pilecap did not affect the response.

Unit-2

The response of this unit was as expected. In Loading Phase 1, the column developed a plastic hinge and in Loading Phase 2, plastic hinges developed in the piles. The use of headed reinforcement effectively precluded the spalling of the concrete cover in the pilecap. Moreover, modifications to accommodate the flexural moment demands taken into account in the footing design of Unit-2 hampered any inelastic behavior in this region. No damage in the joint region was observed either. Fig. 6 shows the bottom surface of the pilecap of Unit-2 at the end of the test which exhibited only minor damage compared to the bottom surface of Unit-1 as shown in Fig. 5. As for Unit-1, the hysteretic response of Unit-2 was stable and typical of a bridge component designed for full ductility response, see Fig. 7.

Discussion

Observations and data analysis of the response of the test units highlighted the following two main issues of concern that are not be currently been addressed in design:

Critical Loading Direction for Pile Design

A significant finding of the test program is that the principal direction of the pile resistance is skew with respect to the direction of the shear force in the column. This observation is made based on the inclination of the plane of bending the piles. The instrumentation deployed in the piles enabled the equation of the plane of bending to be determined.

The neutral axis locations of pile A when subjected to EW loading, that induced compression in this pile of Unit 2, are illustrated in Fig. 8. If assuming that the shear force carried by the pile is orthogonal to the direction of the neutral axis depth, then the direction of the pile shear force at peak loading was approximately 55^o with respect to the column loading direction. Skewness of the neutral axis with respect to the applied shear force implies that two dimensional frame analyses, which result in forces parallel to the loading direction, will underestimate the magnitude of the shear force demand in the piles.

Shear Distribution Between Compression and Tension Piles

Fig. 9 shows the distribution of the applied lateral force between the compression and tension piles in Unit 2 during Loading Phase 1. It is evident in this figure that the applied lateral force is very unevenly distributed between tension and compression piles. The piles in compression resist a large portion of the applied lateral force. Because of the assumption of equal stiffness in the tension and compression piles in current design, lateral forces are distributed somewhat equally between the piles. This issue, when taken together with the issue of the skewness of the shear force in the piles, will result in a significant underestimation of the pile design shear force in the piles and could lead to premature and undesirable shear failure in the foundation structure.

Conclusions

The following conclusions have been drawn from the experimental study which investigate the combined effect of axial force and moment direction of FCIDH pile supported bridge piers tested under bi-directional reversed cyclic loading conditions:

1) The pile/pilecap joints, which were designed using the external strut shear force transfer mechanism, exhibited satisfactory performance when subjected to simulated seismic loading. However, the column/pilecap joint of Unit-1 experienced spalling of the pilecap cover concrete at several J-stirrup locations, indicating straightening of the 90^o hooks. Unit-2, which used headed bars for the shear stirrups, showed no damage during the test.

2) The principal direction of elastic pile resistance was at an angle to the applied lateral force under orthogonal direction loading. This implies that the shear force in the piles is greater than that determined from a simple plane-frame analysis.

3) It was found that the piles in compression attracted greater shear force than tension piles. The compression pile shear force component in the loading direction was much greater than that of tension pile. It is noted that the current analysis procedures, in which the piles are modeled with a unique flexural stiffness value, can greatly underestimate the pile design shear force.

4) The combination of the findings described in 2) and 3) above, can result in a large underestimation of the design shear force of piles of FCIDH pile supported bridge piers and could result in premature and undesirable foundation structure failure.

5) The performance of Unit-2 using headed reinforcement was satisfactory. No damage occurred to the pilecap and joints of Unit-2. Normal minimum embedment length were satisfied in the test using headed reinforcement.

Acknowledgments

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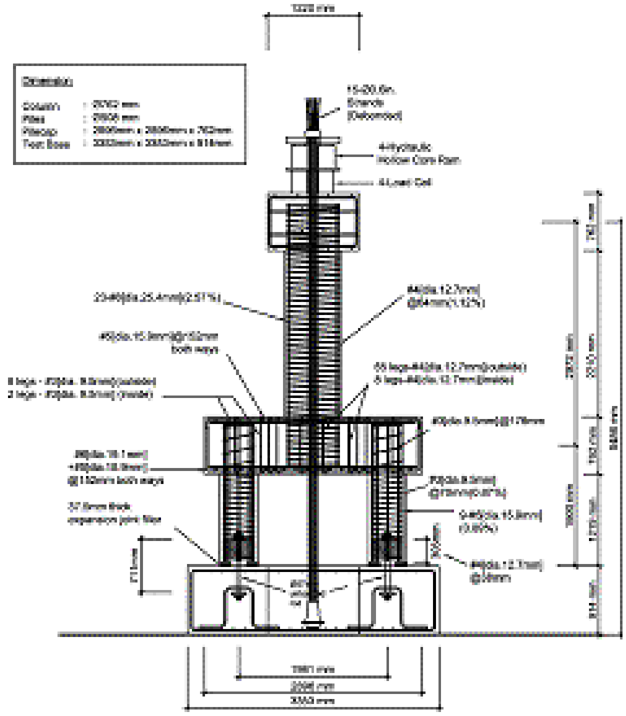
References

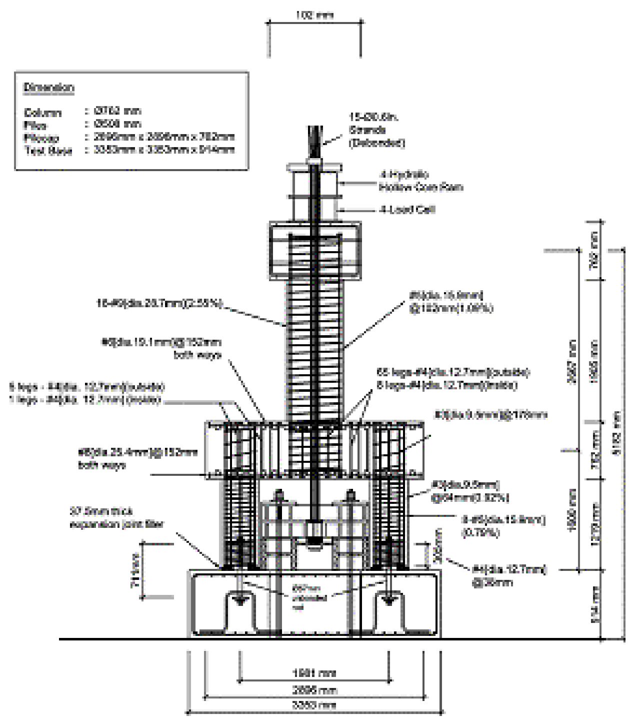
Applied Technology Council, 1996, Improved Seismic Design Criteria for California Bridges : Provisional Recommendations. ATC-32.

Ingham, J. M., Priestley M. J. N. and Seible, F., 1994, Seismic Performance of Bridge Knee Joints – Volume I, Structural Systems Research Project, Report No. SSRP-94/12, University of California at San Diego, California.

Priestley M. J. N., Seible, F., Calvi G. M., 1996, Seismic Design and Retrofit of Bridges. John Wiley & Sons, New York.

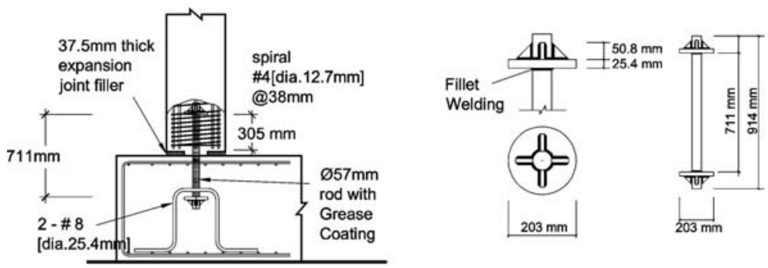


(a) Unit-1



(b) Unit-2

Figure 1- General geometry and reinforcing details of test units



(a) Pile-Test Base Pin Connection Detail

(b) Steel Rod Detail

Figure 2 − Pin connection details between pile and base block

Figure 3 - Test set-up of Unit-1

Figure 4 - Straightening of 90^o J-hook at the end of test of Unit-1

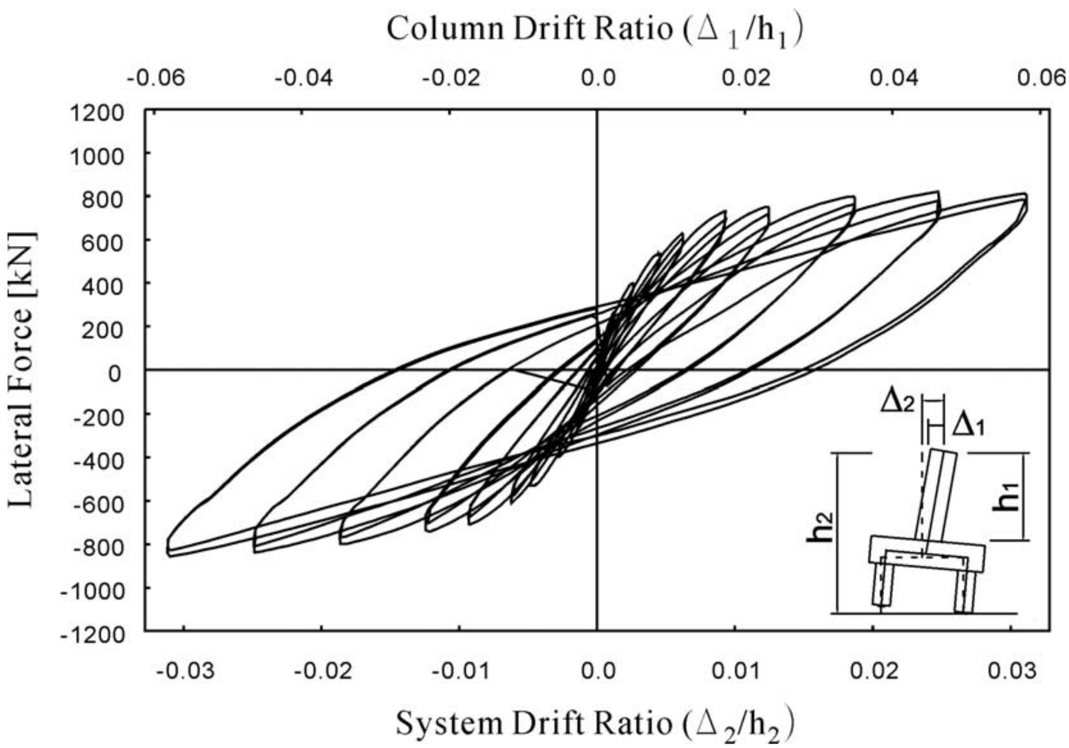


Figure 5- East-West hysteretic response of Unit - 1 during Loading Phase 1

Figure 6 - Pilecap soffit at the end of test of Unit 2

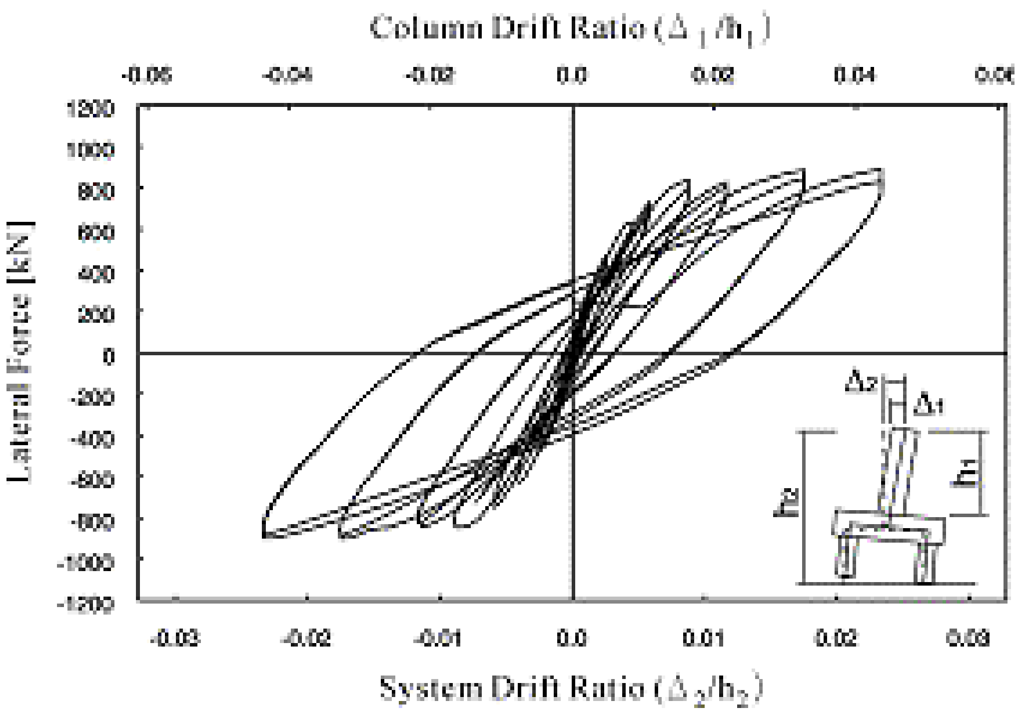
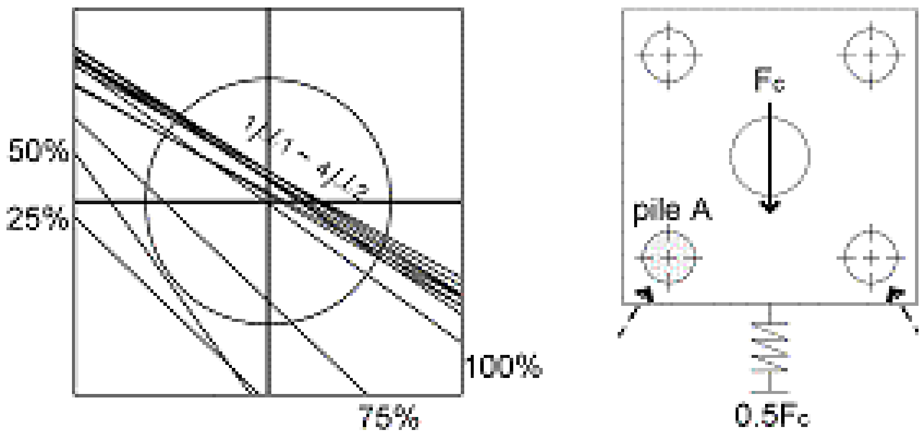


Figure7- East-West hysteretic response of Unit - 2 during Loading Phase 1



(a) Magnified view of neutral axes of pile A

(b) Loading direction

Figure 8 - Neutral axis locations for Pile A of Unit-2 during Loading Phase 1

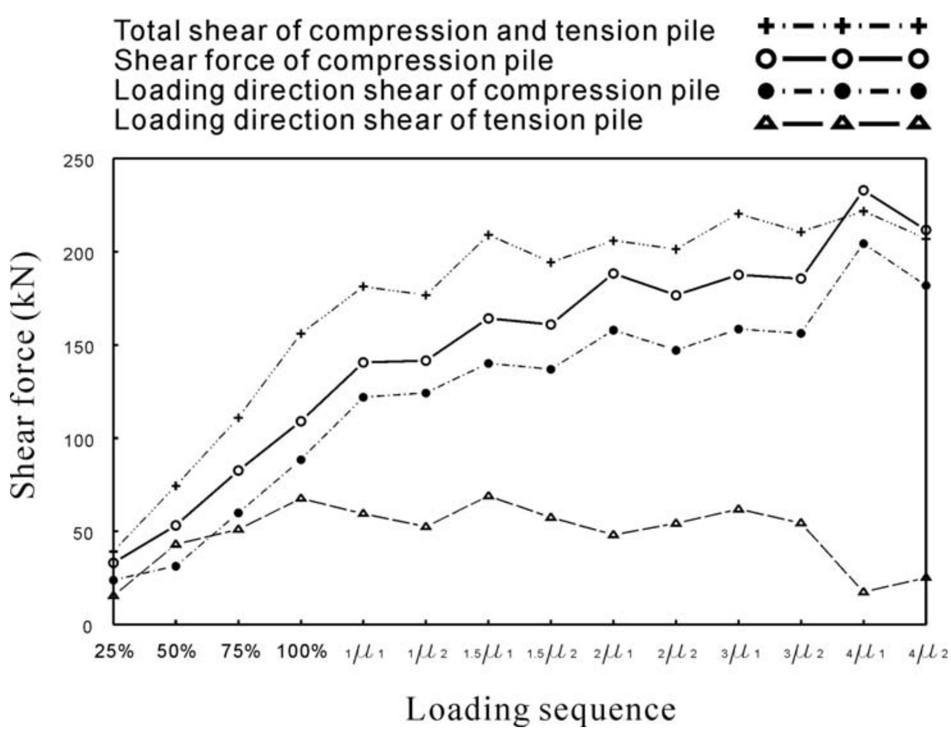


Figure 9 - Shear force distribution between compression and tension piles of Unit 2 during Loading Phase 1