

AN EXPERIMENTAL STUDY ON ROCKING RESPONSE OF BRIDGE SPREAD FOUNDATIONS

H.H. Hung¹, K.C. Chang², K.Y. Liu¹ and T.H. Ho³

 ¹ Associate Research Fellow, Center for Research on Earthquake Engineering(NCREE)/Taipei, Chinese Taiwan
² Professor, Dept. of Civil Engineering, Taiwan University/Taipei, Chinese Taiwan
³ Graduate Student, Dept. of Civil Engineering, Taiwan University/Taipei, Chinese Taiwan Email: hhung@ncree.org.tw

ABSTRACT :

Some recent studies have shown that rocking of spread foundation has beneficial effect on the dynamic performance of piers by decreasing the plastic deformation occurring at the plastic zone. This implies that the ductility demand of piers can possibly be reduced. However, rocking is still not an acceptable mode of response in design code generally. In order to gain better understanding of the problem of rocking, especially its ductility demand, and in turn have more confidence in taking the benefit of rocking mode into account during design and retrofit design process, a series of preliminary rocking experiments were performed. A total of three 3.8 m high circular reinforced concrete columns with spread foundation were tested. Two of these columns were the as-built ones with lap-spliced longitudinal reinforcement and inefficient transverse confinement, representing piers with low ductility capacity; the other one was identical to the as-built ones but wrapped with steel jacketing to shift its ductility capacity to a level fitting the requirements specified by design code. During the tests, one as-built test column and one retrofitted column were rested on rubber pad to allow the mechanism of rocking to take place; another as-built column was constrained to the strong floor during testing to represent a benchmark test with fixed base condition. Using pseudo-dynamic test and cyclic loading test, the rocking behavior of these columns undergoing different level of earthquake accelerations, including a near field ground motion were obtained and discussed. From the benchmark test, the difference between the response behavior of rocking base and fixed base foundation was highlighted. By comparing the experimental responses of retrofitted column with that of the as-built one, the interaction effect of rocking mechanism on the ductility demand and strength demand of columns was also identified.

KEYWORDS:

Rocking, Spread foundation, Pseudo-dynamic test, Bridge piers, Ductility demand.

1. INTRODUCTION

Recently, due to the vast improvements on economy and the large enhancement of design technique, the objective of bridge design has transformed gradually from collapse prevention to performance assurance that hopes bridges can still maintain a certain level of functionality after a major seismic event. Conventional design and retrofit concept is more like a sacrificial design, which makes use of the damage or plastic deformation occurring at the column base to dissipate earthquake energy. Therefore, once a bridge experienced a severe earthquake, its piers would most likely suffer serious inelastic deformation and need to be retrofitted or rebuilt. In order to reduce the costs of seismic retrofit following a major earthquake, researches that aim to substantially reduce the damage and residual displacement of bridges have become a focal point of the design and retrofitting codes of next generation. The rocking mechanism of a properly designed spread foundation just meets this requirement. The seismic isolation effect of rocking mechanism has been identified in many previous researches,



such as the pioneer work performed by Housner (1963), who noted that foundation uplift may be responsible for the good performance of several elevated water tanks during 1960 Chilean earthquake.

Based on current design philosophy of bridge piers, rocking mechanism of foundation is not allowed and not taken into account in analysis. Analysis results are often obtained based on the assumption that the foundation and soil are firmly bonded. However, spread foundation, in reality, is supported on the soil only through gravity load, and uplift of foundation did take place during past earthquake. According to Kawashima and Nagai (2006), rocking response in spread foundations was observed from the investigation for past earthquakes. These observations further postulate that even though the possible benefit gained from foundation rocking is not taken into account in design phase on the conservative side, rocking did occur during past severe earthquake. Thus both the Pros and Cons of rocking mechanism should be carefully examined. Over the last few decades, several articles have been devoted to the study of rocking behavior. However, most were focused on rigid block. Chopra and Yim (1984) were among the first few to develop a better understanding of the effects of transient foundation uplift on the response of flexible structures. Besides, most of the earlier relevant studies were performed by analytical approaches. Until very recently, the experimental studies of rocking mechanism became the focus of some studies, such as the shaking table tests of small-scale steel column performed by Sakellaraki et al. (2005) and a moderate-scale RC bridge column performed by Espinoza and Mahin (2006). Columns of both of these experimental studies were designed to remain in elastic response when excited by input ground motion, so that the plastic deformation pf column at the plastic hinge was not considered. All of these previous studies have recognized the beneficial effect of rocking; even so, relevant researches are still few and not comprehensive enough to allow practical design code be revised with confidence. In this regards, more experimental data and modeling approaches are still required. In this study, a series of preliminary pseudo-dynamic tests and cyclic loading test of three large-scale reinforced concrete columns were conducted in order to gain better understanding of the effect of foundation's uplift behavior. The results can serve as good references for future seismic evaluation and a basis for updating future design code.

2. EXPERIMENTAL PROGRAM

2.1. Test Specimens

The experimental program was designed to investigate the beneficial effect of rocking mechanism of spread footing on reducing plastic deformation occurring at the pier base and its possible benefit in reducing strength and ductility demand of piers. Three reinforced concrete columns with details shown in Fig. 1 were designed and constructed. As shown in Fig. 1, these columns with a diameter of 60 cm and a clear height of 3.4m were poorly confined and lap-spliced at a height of 38 cm above the top of foundation. The columns were reinforced with 26 No.6 longitudinal reinforcing bars, and were transversely reinforced with No.3 perimeter hoop spaced at 12.7 cm (transverse steel ratio $\rho_s = 0.39\%$). The longitudinal reinforcing bars were spliced over a length of 38 cm, which is roughly equal to 20 bar diameters. Other material properties for this test column are as follows: concrete compressive strength $f_c' = 278 \text{ kg/cm}^2$; average yield strength of longitudinal reinforcements $F_y = 3840 \text{ kg/cm}^2$. In order to investigate the rocking behavior of column with a ductility capacity that meets the requirement specified by design code, one of the test columns was wrapped with 6mm thick A36 steel jacketing prior the testing to enhance its ductility. The length of jacket was 150 cm. To insure that jacket did not bear against the footing when in compression, a vertical gap of 5cm was provided between the jacket and footing.

2.2. Test Setup

To clarify the differences in response behavior between piers with rocking base foundation and fixed base foundation, both the tests with and without foundation uplift restrained were conducted. Fig. 2 illustrates the test setup. For the case that rocking mode of foundation was restrained (Fig2a), four tie-down rods were placed through the foundation and anchored into the strong floor of laboratory. For the case that rocking mechanism was considered (Fig.2b), the square footing was rested on a 5 cm thick neoprene pad (Duro-45), simulating a spread foundation in a stiff soil. The size of neoprene pad is 180cm×180cm, a little bit larger than that of the foundation (168cm×168cm). The lateral deformation of the neoprene pad was restrained in the experiment. Besides, a special apparatus was also installed on each side of the foundation to prevent the torsion of the test



column but allow the uplift of foundation. Uplift displacements and rotations of the foundation were monitored during testing by ten dial gages and ten tiltmeters mounted on two side of the foundation. Curvatures within the plastic hinge region were also measured using a line of tiltmeters mounted along the neutral axis of columns in the potential plastic hinge zone.



Figure 1 As-built details of model columns

During the test, an axial load of 120 tonf was applied to the test column through a tap beam using two vertical actuators. The vertical loading was kept constant throughout the test to simulate the dead load of the deck, which is around $0.15A_g f_c$, where A_g is the gross cross-sectional area of the column. One horizontal actuator was used to apply the lateral force to the pier's top to simulate the seismic loading.



Figure 2 test setup (a) fixed base; (b) rocking base.

2.3. Test Schedule

The test schedule is listed in Table 2.1, which includes cyclic loading test and pseudo-dynamic loading test. The cyclic test was performed under displacement control to a drift ratio of 5% (18cm) or 6% (21.6cm). The input ground motions for pseudo-dynamic test are shown in Fig. 3, which includes two artificial earthquake accelerations and one real ground motion recorded at TCU102 observatory during Taiwan 921 Chi-Chi earthquake. These two artificial accelerations are code compatible medium earthquake acceleration (TH1) and code compatible design earthquake acceleration (TH2) for Nantou Pouli, a region of high seismicity in Taiwan. The third one (TH3) has high velocity impulses, a typical characteristic for near field ground motions. Here, the



intensity of ground motion TH3 has been scaled to have the same PGA of TH2, i.e., PGA=326gal.

For test specimen A, the footing of which was constrained to the strong floor, only cyclic loading test was performed. The test ended when the drift ration reached 5%. For test specimen B, the column was test by one pseudo-dynamic loading at first and followed by a cyclic loading test. The cyclic test was performed to a drift ratio of 6%. For test specimen C, the column was tested sequentially by three pseudo-dynamic loadings, i.e., TH1, TH2 and TH3. The same test column after being subjected to pseudo-dynamic loadings was later subjected to cyclic loading under displacement control to the drift ratio of 6%.

Test Specimens	Base	Tests
	condition	
A (lap-spliced specimen)	Fixed base	cyclic loading test
B (lap-spliced specimen)	Rocking base	pseudo-dynamic test (TH2)
		cyclic loading test
C (retrofitted specimen)	Rocking base	pseudo-dynamic test (TH1)
		pseudo-dynamic test (TH2)
		pseudo-dynamic test (TH3)
		cyclic loading test



Figure 3 Input ground motions for pseudo-dynamic test

3. TEST RESULTS AND DISCUSSION

Results obtained from the experiments are summarized in the following subsections.

3.1. Specimen A

Test specimen A was the benchmark column with short lap splices. The experimental results of which for cyclic test has been given in Fig. 4, where figures (a) and (b) show the lateral load vs. lateral displacement hysteretic response and the moment vs. rotation hysteretic response, respectively. Fig. (c) illustrates the failure after the cyclic test, at drift ratio 5%. As expected, these figures indicate that the test column exhibited sudden and significant loss of lateral resistance with low ductility under reversed cyclic deformation. The lateral strength of specimens was around 18 tonf and started to degrade at a drift ratio of 1.5% (5.4cm) due to the deterioration of the bond between the reinforcement and the surrounding concrete.

3.2. Specimen B

Test specimen B had the design details identical to specimen A but was tested with its foundation supported on a neoprene pad without uplift restraint. This test column was subjected to pseudo-dynamic test of TH2 at first, but the test was terminated unexpected when its lateral displacement reached 26 cm due to the limitation in travel range of the lateral actuator. The time when the experiment stopped was corresponding to T = 36.7 sec of TH2 (Fig.3(b)). Fig.5(c) shows the photograph of the test column after test. As can be seen, only hairline crack occurred on the column, even though the same lateral displacement would have damaged a column on a fixed base. Fig.5(a)-(b) plot the hysteretic responses of the test specimen, where figure (a) shows force-displacement curve for the system and figure (b) shows moment-rotation curve at the column base. In the moment-rotation curve, rotations were obtained as the reading of the highest tiltmeter, which was located at a height of 65cm



above the top of footing, minus the reading of another tiltmeter mounted on the footing. Thus, the rotations in figure (b) come from the elastic and plastic flexure of the column only; whereas the lateral displacements given in figure (a) come from the elastic and plastic flexure of the column plus the rocking of the foundation. By comparing figures 5(a) and (b), we can note that when the lateral displacement reached 26 cm, the plastic deformation of the column was still not very obvious.



Figure 4 Experimental results for specimen A. (a) lateral load vs. displacement; (b) moment vs. rotation at column base; (c) photograph of column at drift ratio5%



Figure 5 Experimental results for pseudo-dynamic test (TH2) of specimen B (a) lateral force vs. displacement; (b) moment vs. rotation at column base; (c) photograph of column after test.

Following pseudo-dynamic test, a cyclic loading test was performed on the same specimen. The experimental results were shown in Fig. 6. As can be observed, it shows similar pattern of response behavior to specimen A and also exhibited sudden and significant loss of lateral resistance with low ductility under reversed cyclic deformation. However, if we further compare figure 4 and 6 at the same drift ratio 5%, we can find that rocking mechanism resulted in an increase of lateral load resistance in fig. 6(a) and a decrease of plastic deformation at plastic hinge zone in fig. 6(b).







3.3. Specimen C

Test specimen C was a retrofitted column and rested on the neoprene pad without uplift restraint. As has listed in Table 2.1, specimen C was subjected to three pseudo-dynamic tests sequentially at first. The experimental results for these tests are shown in Fig. 7, where Figs.(a)-(c) represent hysteretic curves of lateral force vs. lateral displacement at the top of the column for each test, and Figs.(d) and (e) show moment vs. rotation at the column base for TH1 and TH2, respectively. Again, the moment-rotation curves in Figs.(d) and (e) can demonstrate the flexural behavior of column at plastic hinge zone. As can be observed in figure 7, for the case of medium earthquake (TH1), both force-displacement and moment-rotation hysteretic curves are almost linear, which indicates that the rocking behavior was not obvious and the plastic hinge was not formed during medium earthquake. On the other hand, the results for design earthquake (TH2) show an obvious nonlinear behavior on lateral force-displacement curve. It also demonstrates a satisfactory behavior that the residual displacement and the plastic deformation of the column were minor, and the seismic force was limited to 22tonf because the rocking can act as a form of seismic isolation. Figure 7(f) shows the time history of lateral displacement and its variation in structural period of the same test. As can be observed, rocking resulted in lengthening of the structural period of the pier system. For the last pseudo-dynamic test using TH3, a near field ground motion, the test was ended manually at T = 35 sec when lateral displacement reached a high value of 37 cm under consideration of safety, although no apparent damage was observed at that time. Another phenomenon can be found in these tests was that although time history TH2 and TH3 have the same PGA, the response behaviors of column C subjected to these two accelerations were totally different. The displacement induced by TH3 was obviously larger than that by TH2, the same result was also observed by authors in another study using analytical approach, which was not shown here. This result implies that the rocking behavior of piers under near fault ground motion may suffer a large lateral displacement at the deck level and further investigation on this issue is still needed.





After these pseudo-dynamic tests were conducted, a cyclic loading test was applied on the same test column. Fig. 8 shows the experimental results. The said figure demonstrates a clear nonlinear rocking behavior with small residual displacement and the seismic force that the pier sustained was limited to an almost constant value of 22 tonf. However, because a minor plastic deformation probably had formed at the column base due to previous pseudo-dynamic tests, the residual displacement in force-displacement curve shown in Fig.(a) and the plastic deformation in moment-rotation curve shown in Fig.(b) were not as small as expected. Fig. 8(c) shows the



photograph of the test column at the end of testing. In which, no dramatic physical damage was observed on the specimen except the wide opening of the concentrated flexural crack at the base of the column, where the steel jacketing was terminated.



Figure 8 Experimental results for cyclic loading test of specimen C. (a) lateral force vs. displacement; (b) moment vs. rotation at column base; (c) photograph of column at drift ratio 6%

3.4. Comparison and Discussion

Five tiltmeters mounted on the neutral axis of test columns at potential plastic hinge zone allow the average curvature in this area to be estimated. Average curvature was obtained as the difference between the readings of two adjacent tiltmeters divided by the distance between them. Fig. 9 shows vertical distribution of curvature in the plastic hinge region for each test column under the reverse cyclic displacements. In which, only the results for push direction were plotted for brevity. As can be seen, for the columns that were not retrofitted (specimen A and B), the curvatures are both well distributed over the plastic hinge zone. But if we compare Figs. (a) and (b) more carefully, we can find that the curvature for the rocking base case (specimen B) was smaller than that of the fixed base case (specimen A), especially at the drift ratios smaller than 3%. This phenomenon can be further explained by Fig. 10(a), where uplift displacements at foundation of specimen B under the subjection of cyclic displacement loading were depicted. As can be seen, the maximum uplift occurred at drift ratio 3%. After that, the uplift displacement decreased with the increase of drift ratio. For instance, at drift ratio 6%, the uplift was only around 0.2 mm. Thus, at that moment, the response behavior of test specimen B was similar to that of the fixed base case of specimen A. Next, by comparing the curvature distribution of specimen C (fig. 9c) and specimen B, we find that the wrapping of the steel jacket made the plastic rotation for specimen C concentrate in a very short region close to the base of columns. Besides, the curvature in other location of specimen C was much smaller than that of the non-retrofitted one (specimen B). The uplift displacement for specimen C was also given in Fig.10 (b). By comparing figure 10(a) and (b), we find that the uplift displacements for the retrofitted case were much higher than that for the non-retrofitted case. The maximum uplift of which was around 2 cm. In addition, the uplift displacement stably grew with the increase of drift ratio until drift ratio reached 5%.



In theory, if the foundation of pier is allowed to rock with uplift, then the base moment is limited to a value that required to produce uplift against the restraining forces due to gravity. The base moment limitation will then reduce inelastic deformations of the pier at the plastic hinge. From these experiments, the decrease of plastic deformation of the column at plastic hinge as a result of the energy dissipation of inelastic rocking mechanism



of footing was observed. However, we also recognize that even thought there is a good potential for spread foundation to dissipate a large amount of energy by rocking mechanism, thereby reducing ductility demand on the column, the column still should possess certain level of strength and ductility itself, in order to allow the rocking mechanism to become effective. Besides, rocking has a critical side effect of increasing displacement response at the deck level during earthquake, especially earthquake induced by near fault ground motion.



Figure 10 Uplift displacement of foundation for cyclic loading test: (a)specimen B; (b) specimen C

4. CONCLUSIONS

In this study, a series of preliminary pseudo-dynamic tests and cyclic loading test of three reinforced concrete column were conducted. Based on the observed results of these tests, some conclusions can be made as follows: (1) The isolation effect of rocking spread foundation was observed in experimental data. However, it also resulted in an increase of displacement response at the deck's level, especially for near fault ground motions; (2) If the foundation of pier is allowed to rock, the base moment can be limited to a certain value. The base moment limitation will then possibly reduce the strength and ductility demand of columns. However, the strength and ductility capacity of column should at first reach a certain level to make the isolation effect of rocking mechanism work.

REFERENCES

- Housner, G.W. (1963). The behavior of inverted pendulum structures during earthquakes. *Bulletin of the Seismological Society of America*, SSA **52:2**, 403-417.
- Kawashima, K. and Nagai, T. (2006). Effectiveness of rocking seismic isolation on bridges. 4th International Conference on Earthquake Engineering, Taipei, Taiwan, October 12-13.
- Chopra, A.K. and Yim, C.S. (1985). Simplified Earthquake Analysis of Structures with Foundation Uplift. *Journal of Structural Engineering*, ASCE, 111:4, 906-930.
- Espinoza, A. and Mahin, S. (2006). Rocking of bridge piers subjected to multi-directional earthquake excitation. *Fifth National Seismic Conference on Bridge & Highways*, San Francisco, CA, September 18-20.
- Sakellaraki, D., Watanabe, G. and Kawashima, K. (2005). Experimental rocking response of direct foundations of bridges. Second International Conference on Urban Earthquake Engineering, Tokyo Institute of Technology, Tokyo, Japan, March 7-8, 497-504.