EXPERIMENTAL STUDY ON SEISMIC PERFORMANCE OF A PRECAST REINFORCED CONCRETE BRIDGE PIER

Satoru Nonaka[1]
Masahiro Nakai[2]
Shigeki Unjoh[3]
Junichi Sakai[4]

Abstract

Shake table test was conducted to investigate the dynamic behavior of a precast reinforced concrete (RC) pier during earthquakes. The 1995 Kobe Earthquake motion recorded at the JR Takatori station was used as input ground motion to the shake table. The experimental result showed that the precast RC pier performed similar to a conventional RC pier in the viewpoint on hysteresis and energy-absorbing capacity. Moreover, buckling of the longitudinal reinforcement was not observed in the test and a part of covering concrete at bottom of the pier was spalled when the response displacement was 9.7 δ_y (156.8mm).

Nonlinear dynamic analysis was conducted to simulate the behavior of the test specimen. The analytical result calculated by using the same model of conventional RC piers showed good correlation with the test result. From these results, it was concluded that the dynamic behavior of the precast RC piers with structural conditions proposed in this study can be simply evaluated using a similar method to that used for conventional RC piers.

Introduction

Precast segmental method can reduce the term of construction at the site and make construction periods shorter. In addition, precast segmental method can improve the durability and quality of the structure by producing it at the factory. There are various precast segmental methods now. The construction steps of the method proposed in this research is as follows.

1) Precast segment produced at the factory is transported to the construction site.
2) Precast segment is piled up. Epoxy resin is spread on the bonded surface between segments.
3) Prestressing force necessary to push and to expand epoxy resin between the segments is applied.
4) The high strength mortar is grouted into the sheath.
5) Longitudinal reinforcing bars are inserted into the sheath to connect the piled up

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1. Engineer, Foundation＆Underground Structure , P.S.Mitsubishi Construction Co.,Ltd
2. Director, Foundation＆Underground Structure , P.S.Mitsubishi Construction Co.,Ltd
3. Chief Researcher, CAESAR, PWRI
4. Senior Research Engineer, CAESAR, PWRI

segments.

The seismic performance of the precast RC pier has been verified through cyclic loading tests and shear tests of the joint part. However, clarification of the dynamic behavior of the precast RC pier is not enough.

This paper presents the dynamic behavior of precast RC pier based on the shake table test and the nonlinear dynamic analysis.

Test Specimen

The specimen is shown in **Fig.1**. The specimen was designed based on the 2002 Design Specification of Highway Bridges, Japan Road Association (JRA). It was assumed the bridge pier be constructed in the city area. The scaling factor of the specimen is one-fifth. The cross section was 600mm×600mm square and thickness of the wall was 100mm. The specimen was 2785mm tall from the bottom of the specimen to the gravity point of the inertia mass. Deformed bars (SD345 D10) were used as the longitudinal reinforcement. Deformed bars (SD295 D6) were used as the hoop reinforcement, and were placed with space of 50-mm.

Six segments were separately produced. Epoxy resin was spread on the bonded surface. After the all segment were piled up, prestressing force (0.5N/mm^2) necessary to spread the epoxy resin was applied. The high strength mortar was grouted into the sheath after the prestressing, and the longitudinal reinforcing bars were inserted into the sheath. The first segment was set up just on the reinforcing bar which was placed in the upper side of the footing, and half height of the first bottom segment was filled with concrete when footing concrete was placed. (**Fig.2**)

The material properties when the specimen was tested are shown in **Table.1**.

Table.1 Material Properties

(a) Concrete and Mortar

Materials	Compression Strength (N/mm^2)	Young Modulus (N/mm^2)
Concrete(1,2,6BL)	64.7	32.4×10^3
Concrete(3,4,5BL)	58.5	30.4×10^3
Mortar Grout	45.5	-

(b) Reinforcement

Bar No.	Yield Strength (N/mm^2)	Yield Strain (μ)	Young Modulus (N/mm^2)	Ultimate Strength (N/mm^2)
SD345 D10	399	2134	18.7×10^4	547
SD295 D6	490	2487	19.7×10^4	593

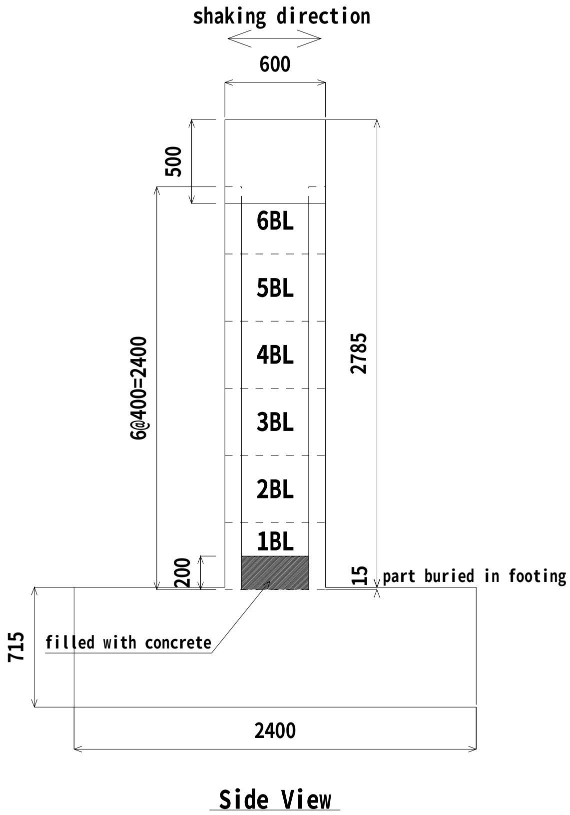


Fig.1 Specimen Views (unit: mm)

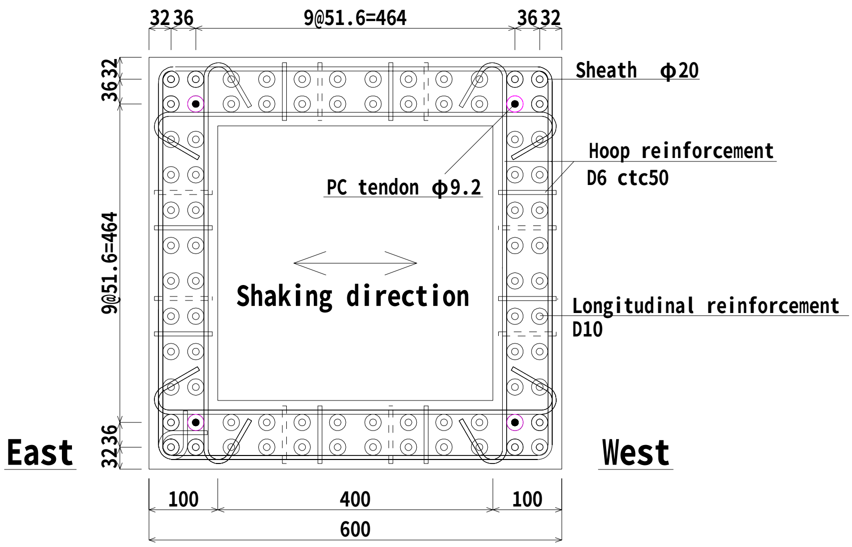


Fig.2 Specimen Cross Section (unit: mm)

Test Setup

The test setup is shown in **Fig.3**.The specimen supported steel girders and the weight plates that idealize the inertia mass and dead load of a superstructure. The total weight was 264.6kN. Response acceleration and displacement were measured at the top of the specimen. Response acceleration on the footing was also measured. CCD laser displacement sensors and accelerometers were used to measure displacement and acceleration, respectively.

EW component motion of the JR Takatori station records of the 1995 Hyogo-ken Nanbu Earthquake was used as input earthquake ground motion. The scaling factor of time was $1/\sqrt{5}$. The amplitude scale tested is shown in **Table.2**. These scales were determined based on the nonlinear dynamic analysis. The amplitude scale for Case1 was determined as 15% of the waveform which was a level that the crack was predicted, and 60% was a level that the longitudinal reinforcement was yielded. 150% was a level that permissible displacement was generated. However, the specimen did not suffer significant damage even after 150% excitation, the specimen was further excited by 200% and 250% level.

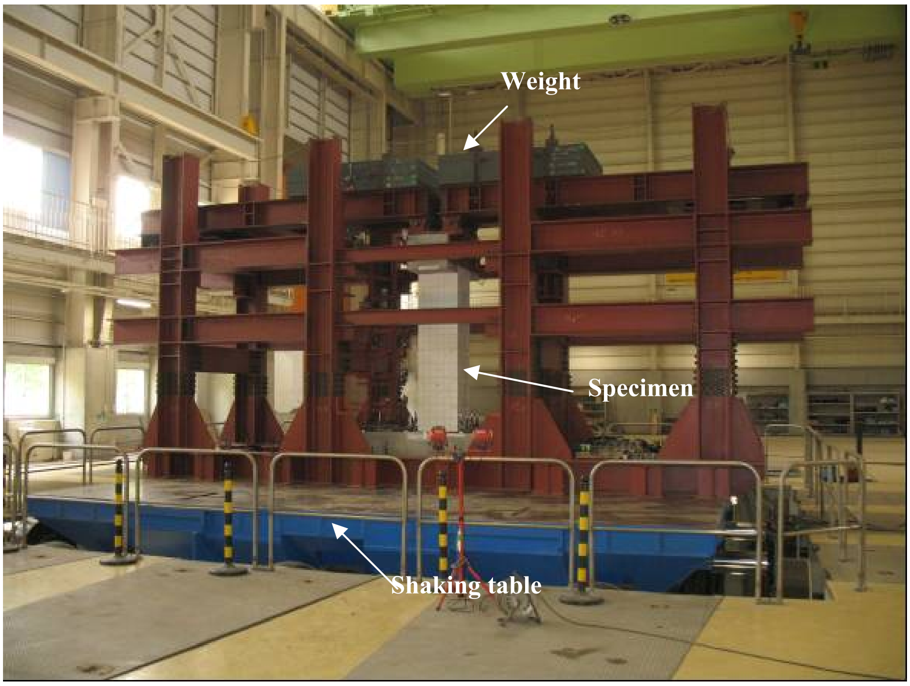


Fig.3 Test Setup

Table.2 The amplitude scale

Case	1	2	3	4	5	6	7
The amplitude scale	15%	60%	100%	120%	150%	200%	250%

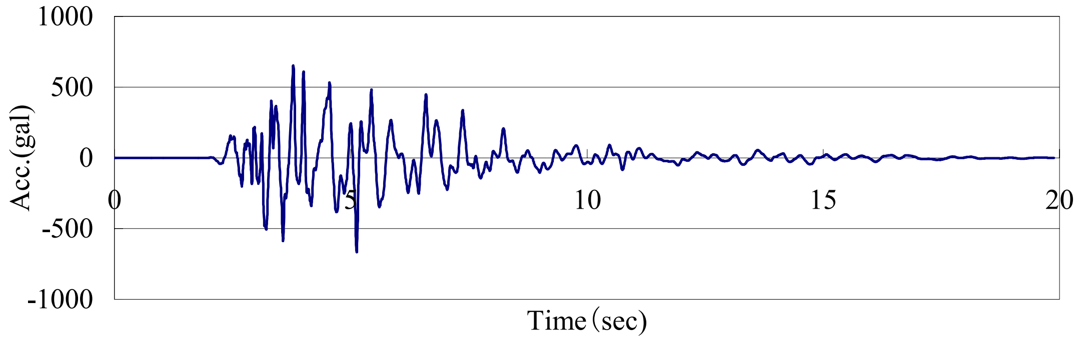


Fig.4 Input waveform (The scaling factor of time is 1/√5)

<u>**Test result**</u>

Damage Situation

Fig. 5 shows the distribution of longitudinal reinforcement strain in the direction of height. **Fig. 6** shows the width of joint opening between segments.

In Case1 (15%), the maximum strain of longitudinal reinforcement was less than 661 μ. No damage was found. In Case2 (60%), the maximum strain of longitudinal reinforcement was 1783 μ, which was almost yield strain. A few cracks were found in the segment at 100 to 200mm high (h=100 to 200) from the base. In Case3 (100%), the maximum strain of longitudinal reinforcement was 6409 μ at the base. At h=300mm from the base, the maximum strain of longitudinal reinforcement was 7718 μ. The width of joint opening between 1BL and 2BL was 2.6mm. In Case4 (120%), the crack at h=150mm was developed. For this reason, the maximum strain of longitudinal reinforcement at h=300mm and the width of joint opening between 1BL and 2BL were smaller than in Case 3. In Case5 (150%), cracks did not concentrate in the joint and distributed from 1BL to 2BL. The damage situation after Case 5 is shown in **Fig.7**. In Case6 (200%), the width of the crack at h=150mm extended further. It was thought that plastic hinge was formed at h=150mm, and the stress concentrated on here. In Case7 (250%), a part of spalling of the cover concrete was found, but buckling of the longitudinal reinforcement did not occur. There was no slipping out at the boundary part of footing and 1BL. The damage situation after Case 7 is shown in **Fig.8**.

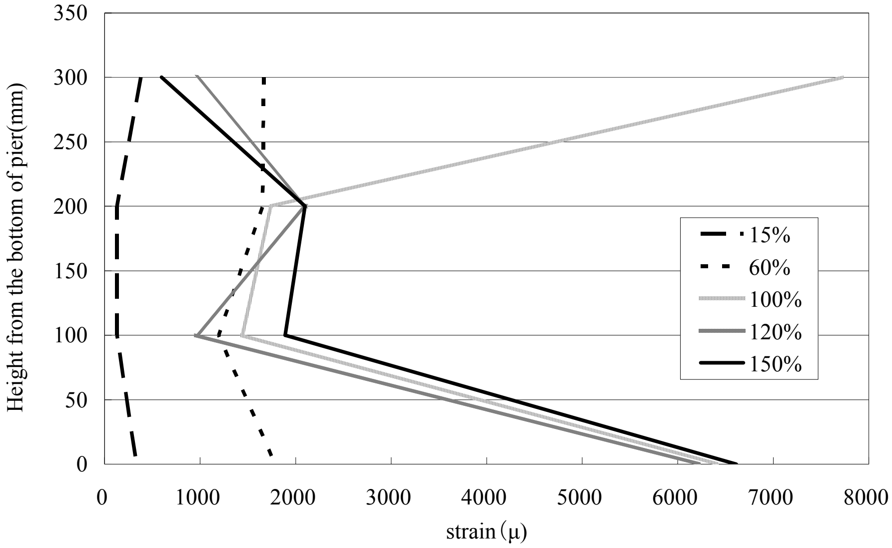


Fig.5 Distribution of longitudinal reinforcement strain

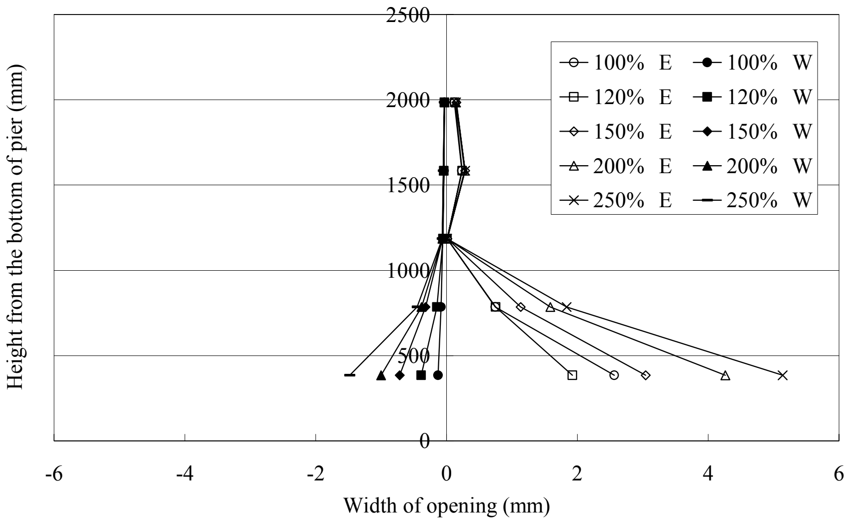


Fig.6 Width of joint opening between segments

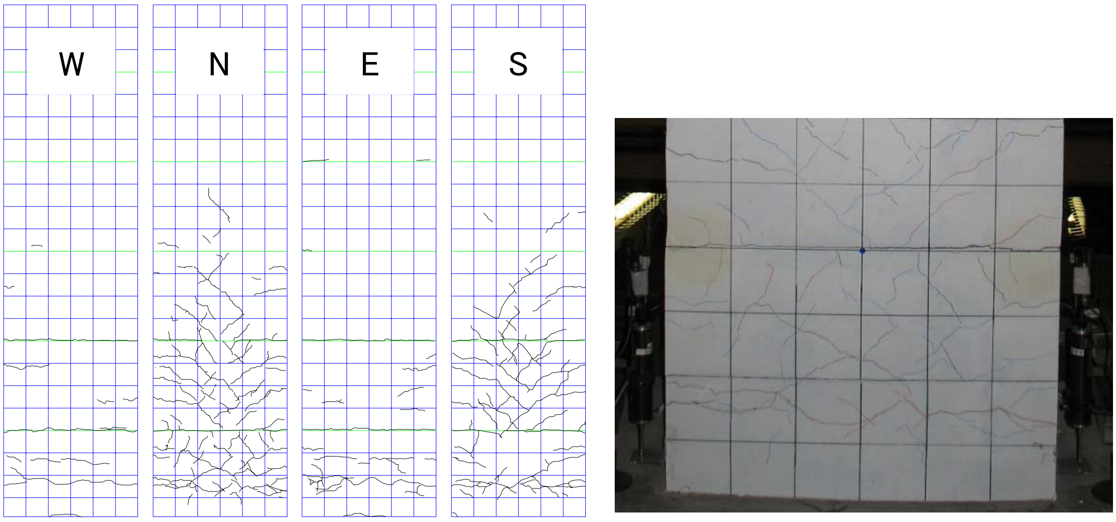


Fig.7 Damage after Case5 (150%)

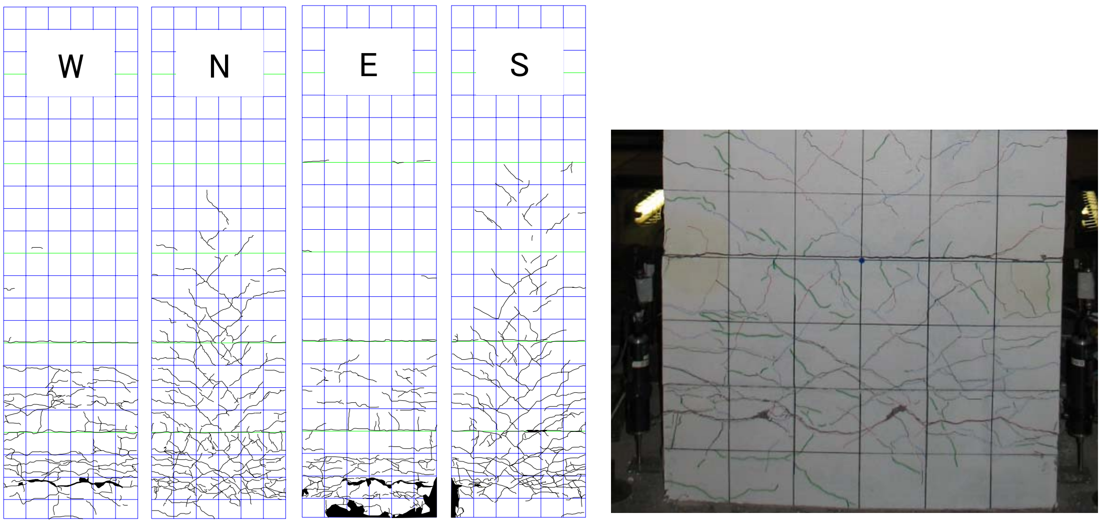


Fig.8 Damage after Case7 (250%)

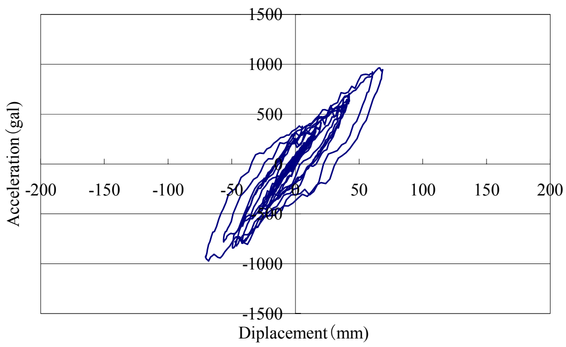
Deformation Performance and Lateral Strength

Tble.3 shows the maximum response acceleration, the maximum response displacement and residual displacement at the top of specimen for each shaking step. In Case2(60%), longitudinal reinforcement was yielded, so response displacement(16.2mm) at this time was defined as yield displacement (δ_y). In Case5(150%), the response displacement was 4.4 δ_y which was almost allowable ductility capacity(μ a=4.04) based on 2002 JRA design specifications. In Case6(200%), although the response displacement was 7.0 δ_y(112.2mm), the residual displacement was only 1.0mm. In Case(250%), the response displacement was 9.7 δ_y (156.8mm) and the residual displacement was 29mm which exceeded the allowable residual displacement (δ_{Ra}=28mm) based on 2002 JRA design specifications .

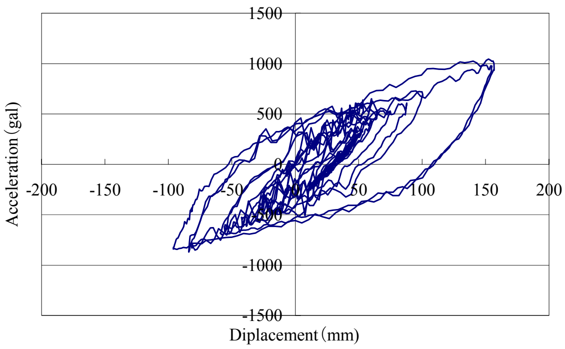
Table.3 Maximum response acceleration, maximum response displacement and residual displacement of each step at the top of pier

		Case1 15%	Case2 60%	Case3 100%	Case4 120%	Case5 150%	Case6 200%	Case7 250%
Max response acceleration(gal)	＋(East)	295	666	945	933	963	1033	1044
	－(West)	-310	-691	-898	-947	-972	-942	-870
Max response displacement(mm)	＋(East)	3.6	15.7	40.3	44.7	68.5	112.2	156.8
	－(West)	-4.4	-16.2	-34.2	-51.8	-70.5	-95.6	-96.1
Residual displacement(mm)		0.0	0.1	2.9	4.5	2.3	1.0	29.0

Fig.9 shows the hysteresis loop between acceleration at the top of pier vs. response displacement hystereses. The area of the hysteresis loop that means the energy-absorbing capacity is similar to the conventional RC columns.



(1) Case5 (150%)



(2) Case (250%)

Fig.9 Acceleration vs. displacement at the top of pier hysteresis

Fig.10 shows the relation between lateral force vs. lateral displacement obtained from test result as well as the calculation result based on 2002 JRA design specifications. The yield and ultimate displacements of the specimen based on the 2002 JRA design specifications were 14.5 mm and 80.5 mm, respectively, resulting in the ductility capacity of 4.04. The computed flexural strength was 236.4kN.The lateral force of test result is computed by multiplying the response acceleration by the inertia mass.

The lateral force of the test result was 19% larger than the calculation result based on 2002 JRA design specifications. Moreover, even when the response displacement was exceeded the displacement capacity, the lateral force did not decrease. From these result, the designed value using the 2002 JRA design specifications was evaluated well.

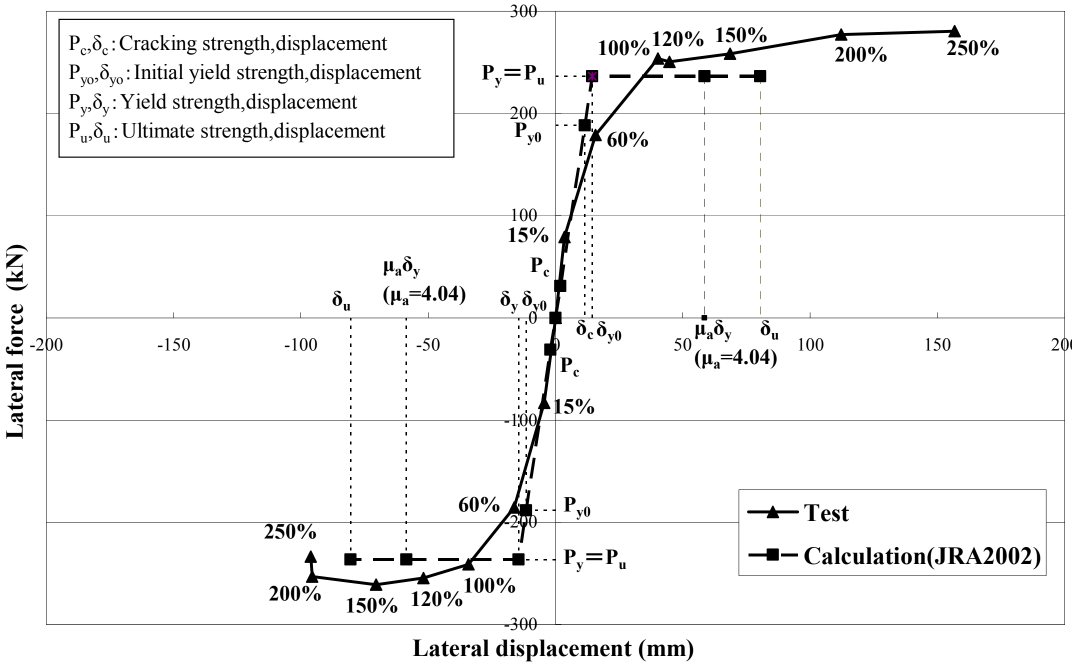


Fig.10 Lateral force vs. lateral displacement

Analytical Simulation

Nonlinear dynamic analysis was conducted to simulate the behavior of the test specimen. Analytical model is shown in **Fig.9**.

The support beam and footing were modeled as rigid beam element. The pier was modeled as nonlinear beam element, and the rotation spring element was assumed at the plastic hinge region. A skeleton curve of this nonlinear element was modeled as a bi-linear curve. Modified Takeda model was used for hystereses model. The analytical material properties were assumed based on the experimental

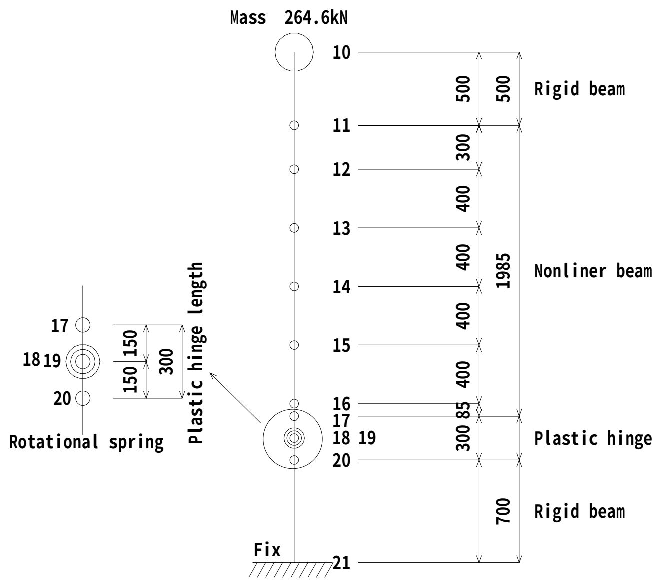
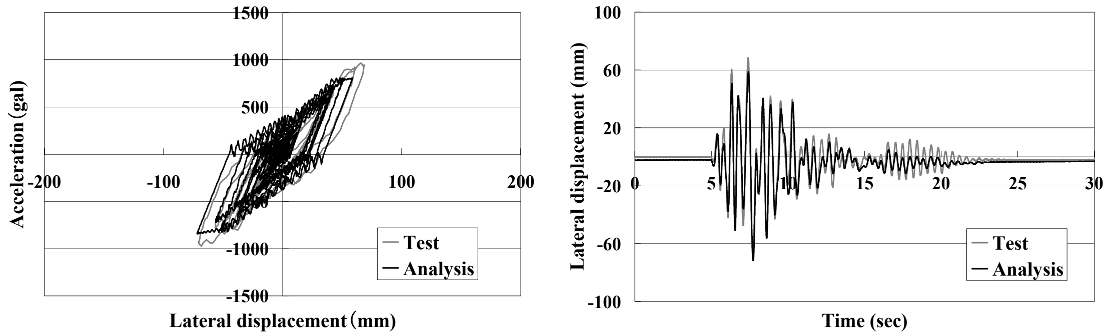


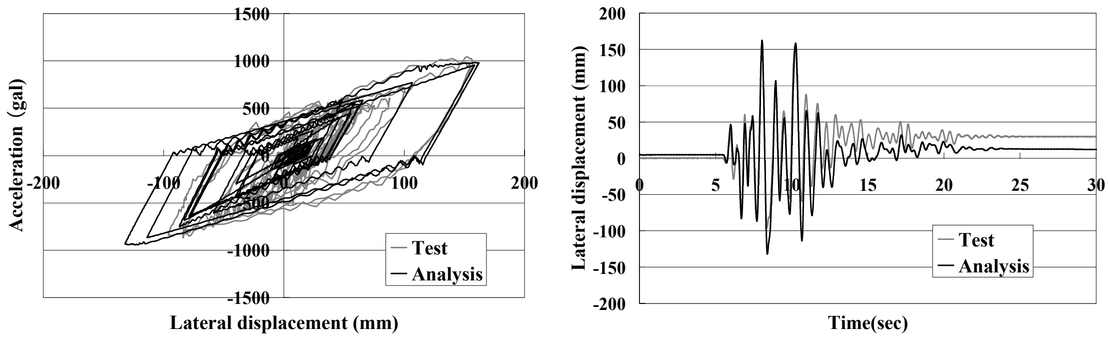
Fig.11 Analytical model

values listed in **Table.1**. The acceleration recorded on the footing was used as input wave. The input motion was continuously applied from 15% to 250%.

Fig.12 shows the comparison between the test result and the analytical result. The analytical model showed good correlation with the test result as the response acceleration, the maximum response displacement and the hysteresis.



(1) Case5 (150%)



(2) Case7 (250%)

Fig.12 Comparison between test result and analytical result

Conclusion

(1) In Case5 (150%), although the response displacement was ductility capacity (μ_a=4.04) based on 2002 JRA design specifications, crack did not concentrate on the joint part but were developed in the plastic hinge region.

(2) The buckling of the longitudinal reinforcement was not observed and a part of covering concrete at the bottom of pier was spalled when the response displacement was 9.7 δ_y (156.8mm). Moreover, there was no slipping out at the boundary part of footing and 1BL. This is because the first segment was placed just on the reinforcing bar of the footing and half height of the first segment was filled with concrete.

(3) The hystereses of precast RC pier showed similar properties to conventional RC piers. The lateral force was larger than the calculation result based on 2002 JRA design specifications. Moreover, even when the response displacement was exceeded the displacement capacity, the lateral force did not decrease.
(4) The analytical result which used the same model of conventional RC piers showed good correlation with the test result. From these results, it was concluded that dynamic behavior of precast RC pier and conventional RC pier can be evaluated in a similar method.

References

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