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Evaluation on Seismic Performance of Limited Ductile RC Bridge Piers by Pseudo-Dynamic Test

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Abstract

Pseudo dynamic test for seven circular RC bridge piers has been carried out to investigate their seismic performance subjected to expected artificial earthquake motions. The objective of this experimental study is to investigate the hysteretic behavior of reinforced concrete bridge piers, which have been widely used for railway and urban transportation facilities. Important test parameters are confinement steel ratio, and input ground motion. The seismic behavior of circular RC bridge piers under artificial ground motions has been evaluated through displacement ductility, cumulative energy input, and dissipation capacity. It can be concluded that RC bridge piers designed in a limited ductile behavior provision of Eurocode 8 have been determined to show good seismic performance even under moderate artificial earthquakes.

Keywords: pseudo-dynamic test, RC bridge piers, artificial earthquake, displacement ductility, energy ductility

1. Introduction

Even though earthquakes have several economic, social, psychological, and even political effects in the areas and the countries where they take place, most Koreans thought until lately that Korea is located rather far away from active fault areas and immune from the earthquake hazards. Recently, it has been observed in the Korean Peninsula that the number of minor or low earthquake motions have increased year by year. Many historic records and recent seismic activities indicate that Korea should belong to a moderate seismicity region. Furthermore, collapse or near collapse of bridge superstructures during the 1995 Kobe earthquake and the 1996 Northridge earthquake stimulated the establishment of seismic design codes for various infrastructures which could be appropriate for geological and topographical conditions in Korea. Therefore, the objective of this pseudo dynamic test is to investigate the seismic performance of circular reinforced concrete bridge piers subjected to earthquake motions, and then to study possible ways of enhancing the ductility of concrete piers in the plastic hinge region. Considering that the Korean Peninsula is located in a moderate or low seismicity region, the seismic performance of test specimens with the limited ductility for bridge piers designed in accordance with Eurocode 8 has been evaluated by the pseudo dynamic test.

2. Description of Experiment

2.1 Material Properties of Test specimen

D10 and D6 deformed steels have been used as longitudinal and lateral confinement steels in RC test specimens, respectively. Yielding stress from the tensile coupon test is 460.60 MPa for D10 deformed steel and 431.20 MPa for D6 deformed steel. A target compressive strength of concrete was $\mathbf{f}_{ck}$ = 23.52 MPa at 28 during days.

2.2 Test Program

Circular solid RC piers of the Hagal bridge, located in Kyung-Gi Do province, Korea, were adopted as a prototype of the test specimen. The bridge had been seismically designed in accordance with the provisions of Korea Highway

Design Specification. Test specimens have been nonseismically or seismically designed in accordance with the provisions of Korea Highway Design Specification.[1] Further test specimens were designed in accordance with a limited ductility for bridge piers in Eurocode 8.[2] Fig. 1 and Table 1 show detailed dimensions and properties of test specimens, respectively. Seven test specimens have been prepared for the pseudo dynamic test to investigate their seismic performance: one for pilot test, three for KHC (Korea Highway Cooperation) artificial earthquake, and three for Kaihokus artificial earthquake. Important test parameters are input ground motion, and confinement steel ratio. The applied scale factors of dynamic similitude between the prototype and the specimen is 3.4. As shown in Table 2, the scale factors for diameter, force, mass, and time are S, S^2, S^3, and S, respectively. The damping ratio of test specimen was measured to be 4.78% from the free vibration test.

2.3 Input Ground Motion

Initial PGA, 0.154g of both loading patterns, was determined on the basis of Korea Highway Bridge Design Code.[1] As shown on Table 3, PGA(Peak Ground Acceleration) values for loading pattern 1 started from 0.154g and gradually increased by approximate 0.1g to the final PGA. The sequence of PGA values for loading pattern 2 are also shown on Table 3. Two types of input ground motions for this experiment have been used, of which accelerations are shown in Fig. 2.

Their PGA values are 0.20g for KHC artificial earthquake and 0.36g for Kaihokus artificial earthquake. These artificial earthquakes are based on rock soil condition. Their duration was 24 seconds. It should be noted that the initial

Table 1 Test specimen description

Specimen designation [1)]	Space (cm) of confinement steel (D6)		Plastic hinge length (cm)
	PHR [2)]	NPHR [3)]	
NS-PL-LP1	12.5	12.5	-
NS-PD-LP1	12.5	12.5	-
ML-PD-LP1	4.5	5.5	50
S-PD-LP1	3.0	4.5	25
NS-PD-LP2	12.5	12.5	-
ML-PD-LP2	4.5	5.5	50
S-PD-LP2	3.0	4.5	25

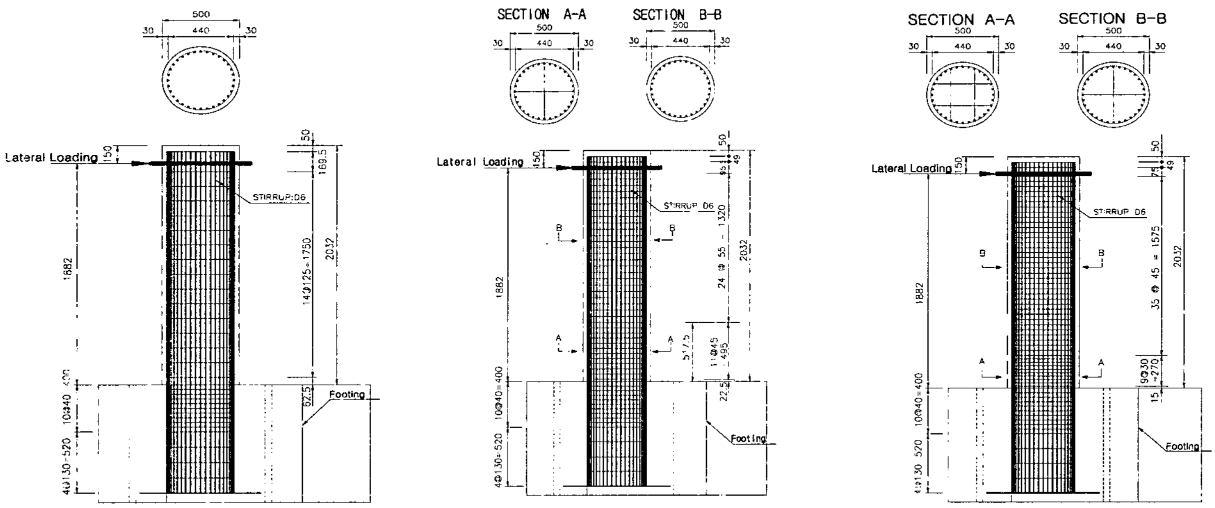
[1)] NS : Non-seismic design, ML : Limited ductile design,
S : Seismic design, PL : Pilot test,
PD : Pseudo-dynamic test LP1,2 : Loading pattern
[2)] PHR : Plastic hinge region
[3)] NHPR : Nonplastic hinge region

Table 2 Scale factor

Scale factor (S)		Dimension	
		Prototype	Specimen
Length/Diameter (m)	S	6.4/1.7	1.882/0.5
Mass (kg*sec^2/cm)	S^3	1,616.11	41.119
Acceleration (g)	1/S	0.154	0.5236
Force (KN)	S^2	4,879.57	422.08
Stiffness (KN/cm)	S	1,080.54	317.98
Frequency (Hz)	1/S	12.034	3.5396

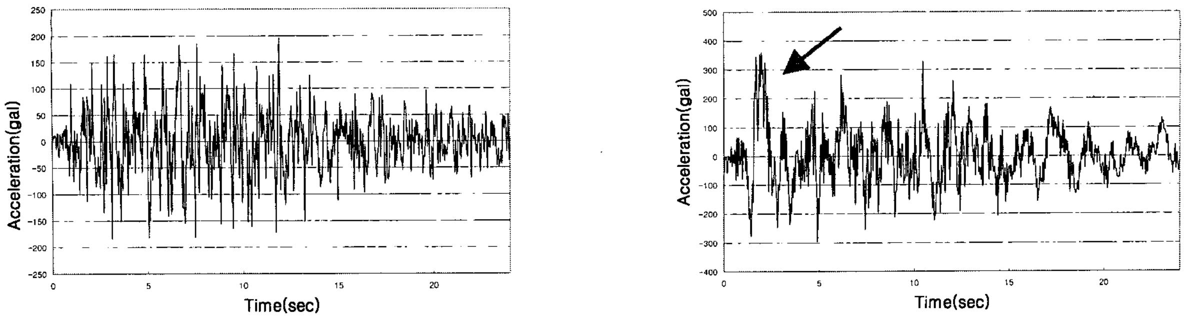
Table 3 Loading patterns

Loading pattern	Artificial earthquake	Sequence of input acceleration	
LP1	KHC (PGA : 0.2g)	1	0.154g
		2	0.220g
		3	0.300g
		4	0.400g
		5	0.500g
LP2	Kaihokus (PGA : 0.36g)	1	0.154g
		2	0.220g
		3	0.260g



(A) Nonseismic (B) Limited ductile (C) Seismic

Fig. 1 Detailed dimension



(A) KHC Artificial earthquake (B) Kaihokus artificial earthquake

Fig. 2 Artificial ground acceleration

part of Kaihokus artificial earthquake could cause failure, as marked by an arrow in Fig. 2(B).

3. Experimental Program

3.1 Data Acquisition

The 1,000KN actuator was used for the pseudo dynamic test. Its maximum stroke was ±250mm. When lateral force is imposed on the test specimen by the actuator, it may be desirable to maintain a constant axial force. A hydraulic axial force controller was employed for this purpose. The applied axial force was 422.08KN, which corresponds to the weight of bridge superstructure. During the pseudo dynamic test, lateral displacements were measured by 2 displacement transducers, which are located at 0.0cm, and 94.2cm from the loading point of test column downward, as shown on Fig. 3. Further 2 LVDTs were placed on the footing to check an unexpected displacement during test. Curvatures have been also measured in the plastic hinge zone of each test column by using 8 clip gages, as shown in Fig. 4. Strain gages were attached to measure the plastic strain of confinement and longitudinal steels in the plastic hinge region.

3.2 Preliminary Experiment

The pseudo dynamic test is similar to standard step-by-step nonlinear dynamic analysis procedures in that computer software controls the response to be divided into a series of time step. Within each step, the governing equation of motions is numerically solved for the incremental structural deformation. In pseudo dynamic test, the ground motions as well as the structure's inertial and damping characteristic are specified numerically through a conventional dynamic analysis.

However, the characteristics of the structure's restoring force are measured directly from the specimen during the test. Scheme of pseudo dynamic test is shown in Fig. 5. Explicit Newmark-β method used for the algorithm of this pseudo dynamic test.[3] Pseudo dynamic test is proceeded with displacement control system. Calculated displacement at every step was exactly input to the test specimen through the Actuators. Restoring force and displacement at lateral loading point of the specimen were measured and calculated at every step, as shown in Fig. 5. So as to assure the reliability of the pseudo dynamic test, the displacement of pseudo dynamic test and the FEM analysis at the lateral loading point of test specimen were compared to check the interface between equipment and software of the pseudo dynamic test. A good agreement between experimental and analytical result was observed (Fig. 6). Analytical result was calculated by the FEM program, SARCF[4](Seismic Analysis of Reinforced Concrete Frame) allowing for behavior in inelastic range.

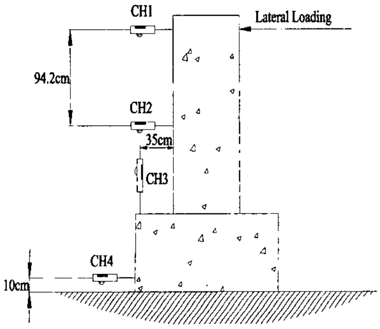


Fig. 3 LVDT setup

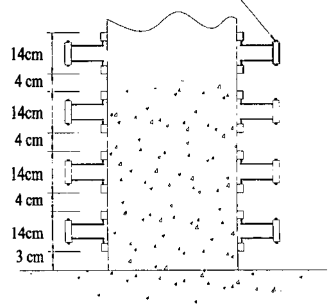


Fig. 4 Clip gages setup

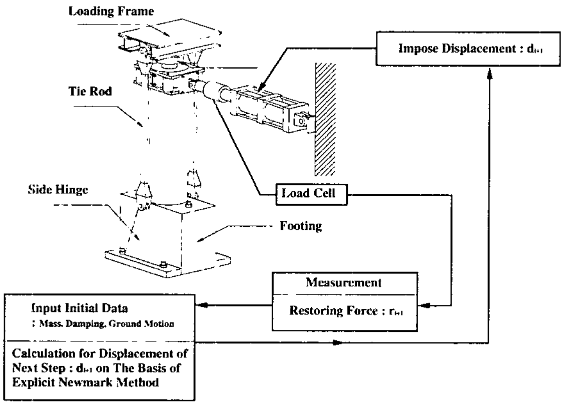


Fig. 5 Scheme of pseudo dynamic test

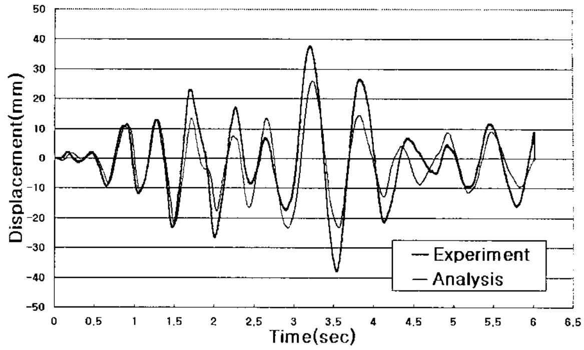


Fig. 6 Pilot test result

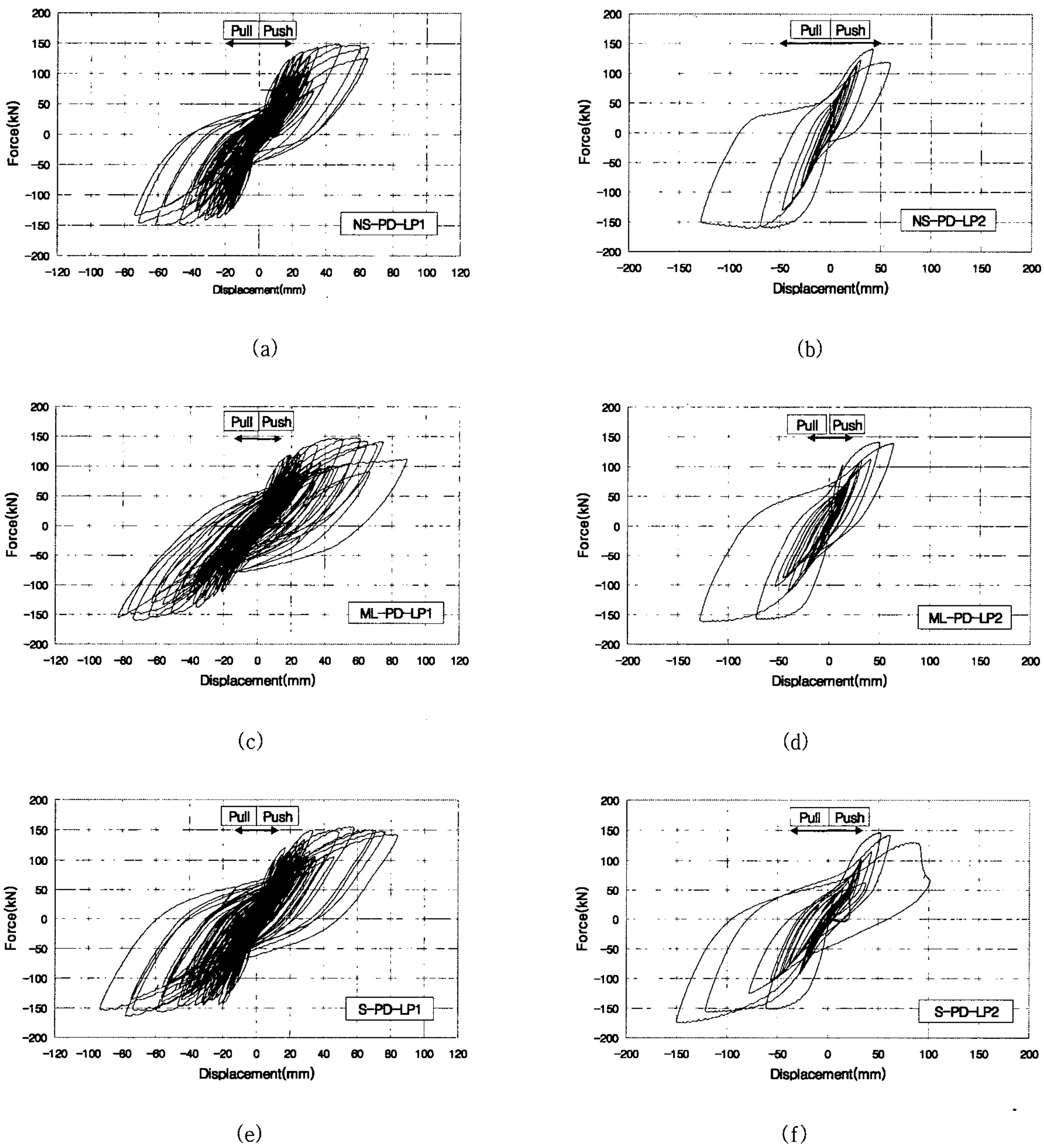
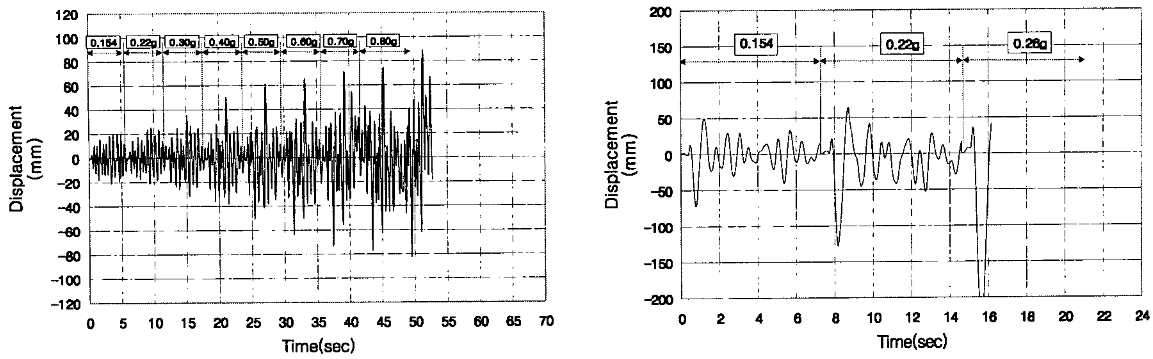


Fig. 7 Hysteric curve



(A) Loading pattern 1 (B) Loading pattern 2

Fig. 8 Typical time displacement history of ML-PD-LP1 and ML-PD-LP2

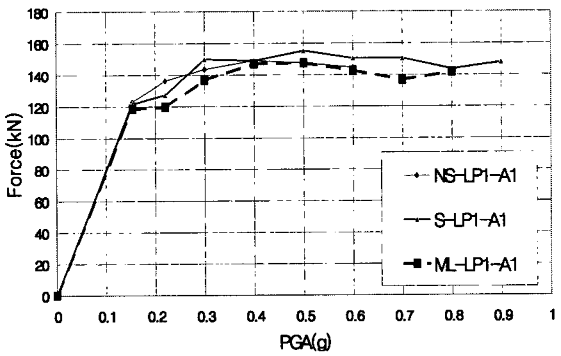


Fig. 9 Strength degradation (LP1)

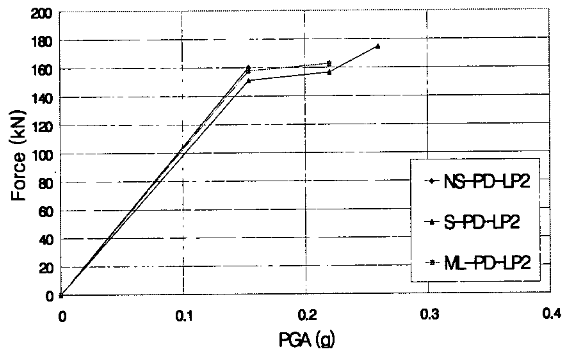


Fig. 10 Strength degradation (LP2)

4. Test Results

4.1 Time-Displacement History and Hysteric Curve

All test specimens displayed similar failure patterns. Mechanism of column failure was started by cracking of cover concrete, spalling of cover concrete, buckling of the confinement or longitudinal steel, crushing of the core concrete, and then breaking of the longitudinal steel in sequence.

Fig. 8 shows typical time-displacement history for each loading patterns. Fig. 7 shows hysteric curve of test specimens subjected to both artificial ground accelerations. It can be seen from Fig. 7 that large ductility was obtained from ML-PD-LP1 and S-PD-LP1, and lower ductility from NS-PD-LP1. It is in particular interesting to note that NS-PD-LP2 specimens severely damaged at 0.22g, ML-PD-LP2 and S-PD-LP2 damaged at 0.26g, respectively.

4.2 Strength Degradation

All test specimens under loading pattern 1 has shown a similar trend of strength degradation until 0.4g, but became somewhat different from 0.5g to the failure state. As shown in Fig. 9 and 10, limited ductile and seismic specimens show similar strength degradation. It can be noted that significant strength degradation in NS-PD-LP2 specimen was measured at 0.154g, and ML-PD-LP2 and S-PD-LP2 specimen at 0.26g.

4.3 Displacement Ductility

Seismic performance of RC bridge piers can be evaluated by a displacement ductility. Yield displacement has been distinctly defined by many researcher in the world. In general, it is used that definition of yield displacement is determined on the basis of i) the strain of steel in the plastic hinge region reached its yielding point, ii) the equivalent elasto-plastic system using stiffness of elastic range, iii) the same quantity of energy absorption. As shown in Figure 11, the yield displacement in this study was calculated by extrapolating the straight line between the origin and 0.75 V_i of the force-displacement point to the lateral loading V_i, which corresponds to the nominal flexural capacity of test specimen.[5, 6] As shown on Fig. 11, meanwhile, Δ_u was defined as an experienced maximum displacement when the strain of longitudinal or confinement steel exceeds its ultimate strain(ε_{su} =0.03) but the strength on the descending branch of the force-displacement envelope curve is above 0.85Vmax.

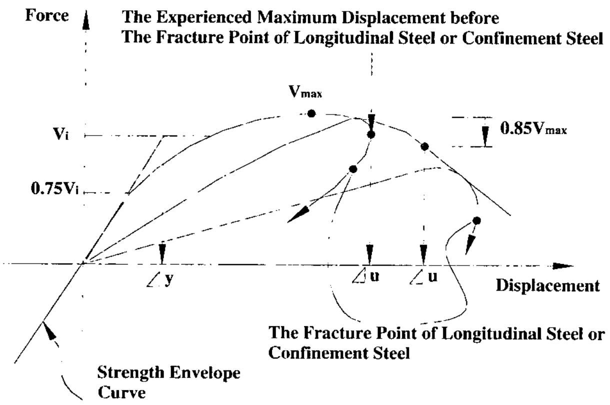


Fig. 11 Yield and ultimate displacement definition

Table 4 Displacement ductility

Specimen	μ_Δ	R Factor
NS-PD-LP1	5.14	3.040
ML-PD-LP1	6.00	3.316
S-PD-LP1	7.62	3.773
NS-PD-LP2	4.07	2.670
ML-PD-LP2	5.81	3.259
S-PD-LP2	6.65	3.508

However, when the strength on the descending branch of the force-displacement envelope curve is dropped below 0.85Vmax but the strain of longitudinal or confinement steel does not reach the ultimate strain(ε_{su}=0.03), Δ_u is defined as the displacement corresponding to 0.85Vmax. Table 4 shows the results of displacement ductility, $\mu_\Delta = \Delta_u / \Delta_y$, and response modification factor, R=$\sqrt{2\mu_\Delta - 1}$. Limited ductile specimens designed in accordance with limited ductile provision of Eurocode 8 showed that its displacement ductility was enhanced by 1.16-1.42 times comparing with nonseismic specimens designed in accordance with the conventional code. It can be said that even nonseismic test specimens by and large satisfy the requirement of R factor, which is specified as 3 for single bridge columns in Korea Highway Bridge Design Provisions. Limited ductile and seismic test specimens displayed a good seismic performance.

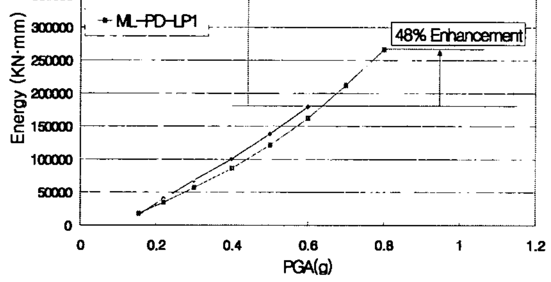


Fig. 14 Cumulative input energy (LP1)

4.4 Cumulative Energy Analysis

For the measurement of energy capacity of test specimen at a given PGA, cumulative input energy and dissipation energy was analyzed. Cumulative input energy is defined as the cumulative workdone of the actuator as shown in Fig. 12. The amount of dissipated energy in each load cycle was calculated from the area of hysteretic loop between two consecutive displacement peaks as shown in Fig. 13. As shown in Fig. 14 and 15, limited ductile specimens showed that cumulative input and dissipation energy was enhanced by approximately 1.5-2.6 times as against nonseismic specimen designed in accordance with a conventional code.

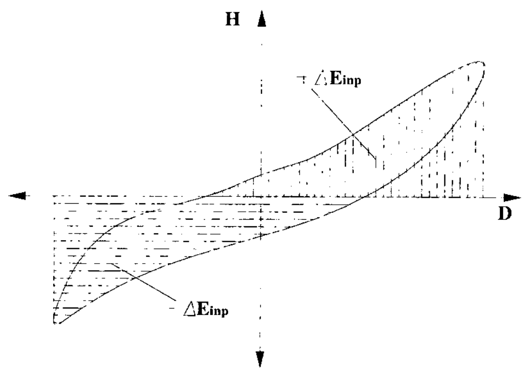


Fig. 12 Input energy definition

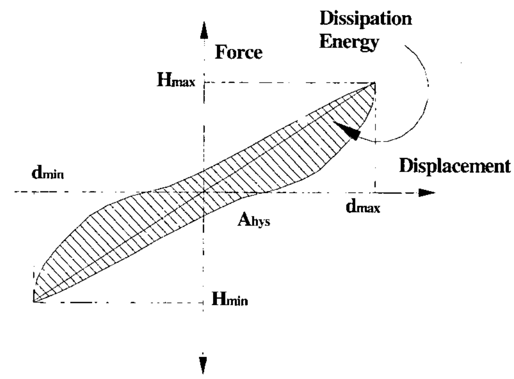


Fig. 13 Dissipation energy definition

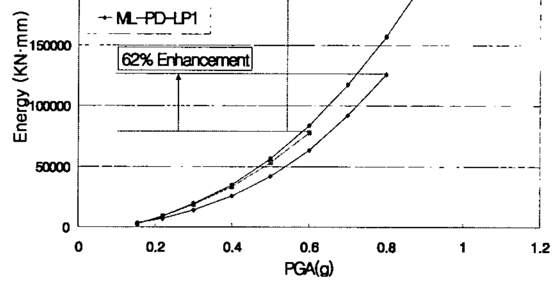


Fig. 15 Cumulative dissipation energy(LP1)

5. Conclusions

So as to analyze seismic behavior of concrete bridge piers, the pseudo dynamic test was done for circular solid RC columns which were 1 to 3.4 scaled models of Hargal bridge piers located in KyungGi-do province. The followings can be concluded from the tests

1) Nonseismically designed RC bridge piers had resistant capacity under KHC artificial earthquake, but exhibited a notable damage at 0.22g of a Kaihokus artificial earthquake.
2) Both limited ductile and seismic design specimens showed a similar seismic resistant capacity. The former were designed in accordance with Eurocode 8, and the latter were designed in accordance with Korea Highway Bridge Design Specification.
3) The limited ductility design concept should be more desirable for RC bridge piers in moderate or low seismic region.

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