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https://hdl.handle.net/2324/8002

出版情報：九州大学工学紀要. 67 (3), pp.85-94, 2007-09-20. 九州大学大学院工学研究院
バージョン：
権利関係：

Memoirs of the Faculty of Engineering, Kyushu University, Vol.67, No.3, September 2007

A Numerical Analysis of Seismic Retrofitting Effect on Reinforced Concrete Piers by Using PCM Shotcrete Method

by

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(Received August 1, 2007)

Abstract

The behavior of typical rectangular bridge columns with sub-standards design details for seismic forces was investigated. The objective of the investigation is to evaluate seismic performances of reinforced concrete (RC) piers retrofitted by using polymer cement mortar (PCM) shotcrete method based on numerical simulating these load-deformation behaviors with three-dimensional finite element analysis. In experimental piers, the poor performance of this type of column attested to the need for effective and economical seismic upgrading techniques. A shotcrete method utilizing PCM to retrofit existing bridge columns was carrying out. PCM was wrapped around the column to increase confinement and to improve the behavior under seismic forces. In this study, there were two models, namely, one model was not retrofitting column and the other was retrofitting column by PCM. It was found that the numerical behavior through the elastic and non-elastic ranges up to failure showed good agreement with the data from the experimental full-scale pier tests. The comparison between experimental and numerical result was obtained that the lateral load increased the load carrying capacity by 136% for experimental pier and by 165% for the finite element model.

Keywords: Seismic retrofitting, Reinforced concrete piers, Polymer cement mortar (PCM) shotcrete method

1. Introduction

In the last decade, repair and seismic retrofit of concrete structures with CFRP (carbon fiber-reinforced plastic) sheets has become more and more common. The strengthening of RC piers with

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wrapped CFRP sheets to improve seismic performance is one of the major applicable soft as new strengthening method. The wrapped CFRP sheet around the plastic hinge region of RC piers provides not only enough shear strength which results in a ductile flexure failure mode in according to the concept of strong shear and weak flexure, but also confinement of concrete in the plastic hinge region to increase the ductility of the piers.

Saadatmanesh[1)] found that the strength and ductility of bridge concrete column can be significantly increased by wrapping FRP straps around the columns due to the confinement of concrete and prevention of the buckling of longitudinal reinforcement bars. The confinement effectiveness of various influence parameters, such as concrete compressive strength, thickness and spacing of FRP straps and type of FRP, were studied. A stress-strain model for concrete confined by FRP was suggested and used to predict the compressive strength and strain. With the confinement of FRP, a desirable ductile flexural failure mode rather than a brittle shear failure mode could be achieved in seismic strengthening for concrete columns. In Saadatmanesh's subsequent study on the strengthening method of bridge concrete columns pre-failed in a severe earth-quake using wrapped FRP sheets, the enhancement of concrete compressive strength and strain were also found.

Z. S. Wu[2)] conducted a numerical investigation on seismic retrofitting performances of reinforced concrete columns strengthened with CFRP sheets. The material and constitutive models, verified in previous research and adopted in the research, could simulate the nonlinear seismic behavior of concrete members. Darwin, Pecknold's equivalent uniaxial strain models could be used to simulate well nonlinear behavior of RC columns strengthened with FRP sheets, and mesh dependent behavior could be improved if the mesh configurations are made in good way, in which mesh size approximates the spaces of cracks. In 2D-FEA, failure criteria of concrete under biaxial stresses state, lateral confinement effect and reduction of compressive strength after cracking must be considered in order to simulate nonlinear behavior of RC columns. Total strain model of Vecchio, Collions could simulate the nonlinear behavior of RC columns, and mesh dependent behavior could be improved through using the material models based on fracture mechanics. But it is necessary to consider failure criteria of concrete under biaxial stresses state, lateral confinement effect and reduction of compressive strength after cracking. Compressive models of concrete have a small effect on capacity of RC columns strengthened with CFRP sheets, but have a large relation with post-peak behavior. The integration of compressive fracture energy model with failure criteria is an important content hereafter. Based on the comparisons load-deformation relationship, the 2D-FEA results have good agreement with the test result, therefore, 2D-FEA can simulate the seismic performance of columns strengthened with FRP sheet or not, according to result of his research. Seen from the strain growths in columns, the interaction mechanics of CFRP with concrete is revealed. CFRP sheets used for seismic strengthening can restrain the growth of shear strain and tensile strain in inflection point effectively. In compressive zone at the bottom of column, CFRP sheets can confine concrete's expansion, and improve the ductility of RC columns.

In this paper, polymer cement mortar (PCM) was used to retrofit existing bridge columns as shown in **Fig.1**. PCM was wrapped around the column to increase confinement and to improve the behavior under seismic forces. The LUSAS finite element program was used to simulate the behavior of reinforced concrete piers. 3-D finite element model used solid elements for concrete and PCM, and bar elements for steel reinforcements. This model could help to confirm the theoretical calculations as well as to provide a valuable supplement to the laboratory investigations of behavior.

2. Experimental Program

The test specimens used for the actual experiment are two of No.1 until No.2, that is, No.1 is the control pier specimen which is a model for existing un-retrofitted bridge pier, and No.2 is the pier specimen retrofitted with the proposed PCM shotcrete method.

The piers were designed with a scale factor 1/5 that of the prototype bridge column. The overall height of the units was 3.045 m. The height of columns was 2.555 m of pins where the cyclic loading was applied to the top of footing. A 1130 x 800 mm concrete block was designed as footing for the specimens, as shown in **Fig.2**. The column had starter bars with a lap length equal to 20 times the bar diameter (20D), 320 mm for rectangular columns reinforced with D16 mm and D10 mm. For the big footing reinforced with D16 mm and D13 mm for the small footing, respectively. The column had a 300 x 300 mm rectangular cross section. The thickness of PCM was 36 mm.

The effect of an earthquake on the column specimen was simulated by reversed cyclic loading. Two independent loading systems were used to apply the load to the specimens as shown in **Fig.3**. The axial load (P_v) of 100 kN (5.3 % and σ = 1.1 N/mm^2 of concrete design compressive strength) was applied to the column, before applying the lateral loads to the specimens. Lateral forces (P_h) were generated by an HP-computer controlled, hydraulic actuator mounted on the reaction frame. The actuator was capable of moving the top of specimen in both positive and negative directions. The position of loading from center pin was 2.555 m. Lateral loading both positive and negative was assumed to be a displacement control.

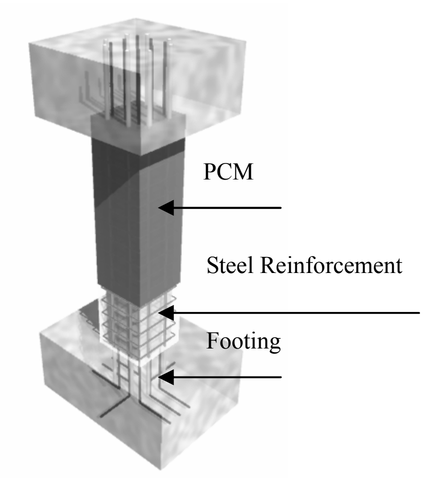


Fig. 1 PCM of Pier Specimens.

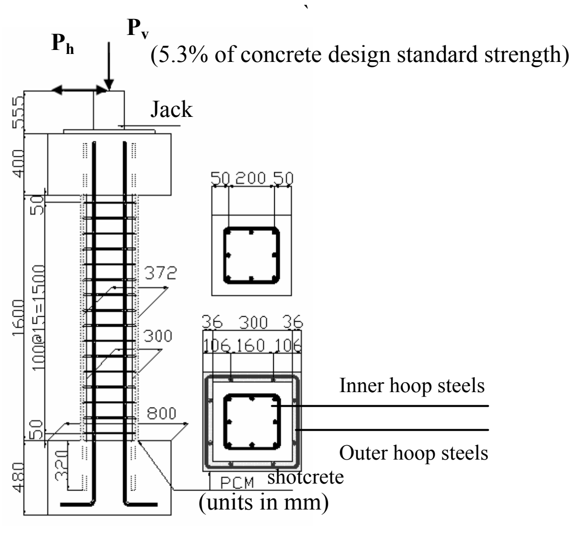


Fig. 2 Experimental Piers.

3. Finite Element Models

3.1 Element types

A bar element was used to model the steel reinforcement. Three nodes were required for this element as shown in **Fig.4(a)**. A solid element, hexahedral twenty nodes, were used to model the concrete and PCM in LUSAS. The solid element had twenty nodes with three degrees of freedom at each node, translations in the nodal x, y, and z directions. The element was capable of plastic deformation, and cracking in three orthogonal directions as shown in **Fig.4(b)** and **Fig.4(c)**, respectively. Bar element could be used with solid element for analysis of reinforced concrete structures as in LUSAS theory[3)].

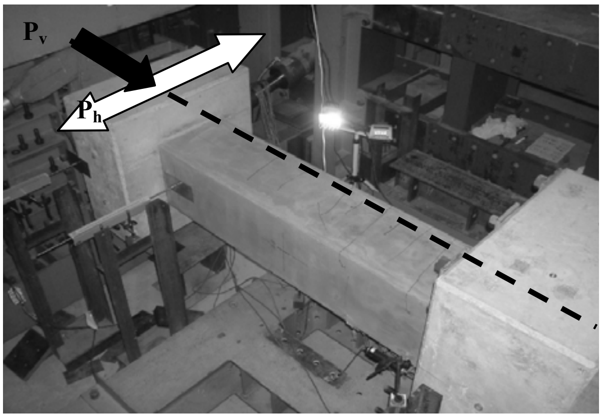
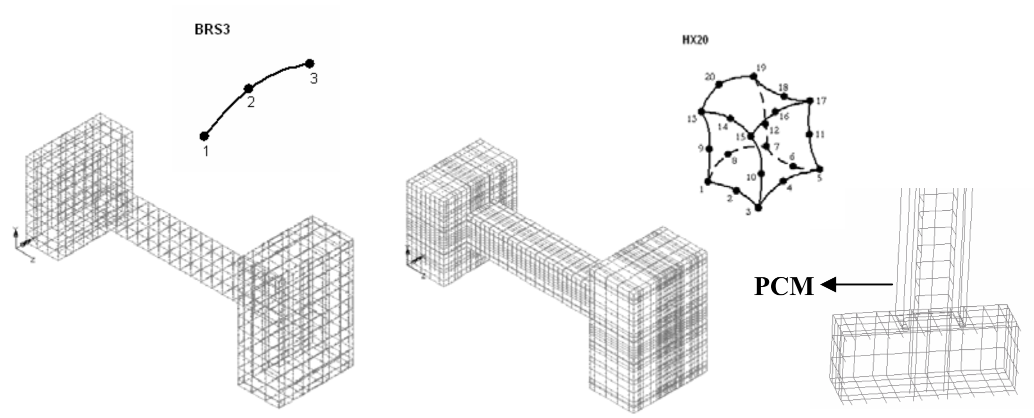


Fig. 3 Test Setup.



(a) Element of Steel Rebar (Bar Element)

(b) Element of Concrete (Solid Element)

(c) Element of PCM (Solid Element)

Fig. 4 Elements of Specimens.

3.2 Material properties

Concrete: Solid elements are capable of predicting the nonlinear behavior of concrete materials using NONLINEAR 94 (Elastic: Isotropic, Plastic: Multi-Crack Concrete). The Multi-crack concrete model is a plastic-damage-contact model in which damage planes form according to a principal stress criterion and then develop as embedded rough contact planes[3)]. Concrete is a quasi-brittle material and has very different behaviors in compression and tension. The tensile strength of concrete is typically 8-15% of the compressive strength as shown in **Fig.5**.

According to LUSAS theory[3)] if no data for the strain at peak compressive stress, ε_{cp}, is available it can be estimated from:

$$\varepsilon_{cp} = 0.002 + 0.001\frac{(f_{cu} - 15)}{45} \quad , \text{ where } \ f_{cu} = 1.25 f'_c \tag{1}$$

Any value for ε_{cp} should lie in the range: $0.002 \le \varepsilon_{cp} \le 0.003$ and as a guide, a reasonable value for most concretes is 0.0022. It is important that the initial Young's modulus, E, is consistent with the strain at peak compressive stress, ε_{cp}. A reasonable check is to ensure that:

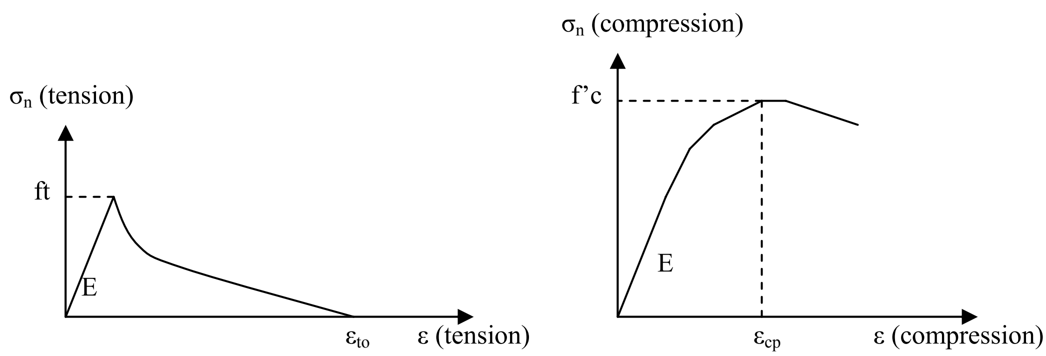
$$E > \frac{1.2 f'_c}{\varepsilon_{cp}} \tag{2}$$

where:

f'_c : specified compressive strength, MPa.
f_{cu} : specified ultimate compressive strength, MPa.
f_t : specified tensile strength, MPa.
ε_{cp} : specified the strain at the peak compressive stress.
ε_{co} : specified the ultimate strain at the end of the compressive softening curve.
ε_{t0} : specified the ultimate strain at the end of the tensile softening curve.

Poisson's ratio for concrete is assumed to be 0.2 and is used for all piers.

Steel Reinforcement: Steel reinforcement in the experimental piers was constructed with typical steel reinforcing bars. Elastic modulus and yield stress for the steel reinforcement used in this FEM study follow the design material properties used for the experimental investigation. The steel for the finite element models is assumed to be an elastic-perfectly plastic material and identical in tension and compression. A Poisson's ratio of 0.3 is used for the steel reinforcement. For steel frame and reinforcement material, the stress potential model that assumed strain-hardening coefficient as $Es/100$ was used. **Figure 6** shows the stress-strain relationship used in this study. Material properties for the concrete and steel reinforcement are summarized in **Table 1** and **Table 2**, respectively.



(a) Exponential softening surface in tension (b) Nonlinear behavior in compression

Fig. 5 Stress-Strain Curve for Concrete and PCM.

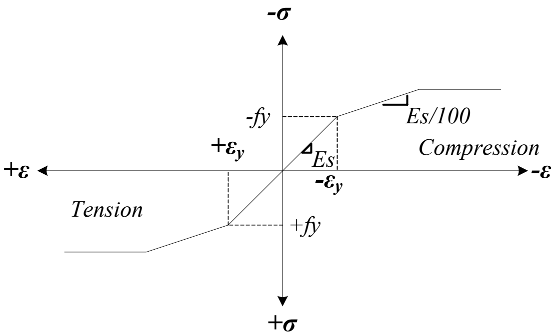


Fig. 6 Stress-Strain Curve for Steel Reinforcement.

Table 1 Summary of Material Properties for Concrete.

Specimen	*f"c*, MPa	*ft*, MPa	*fb*, MPa	*Ec*, MPa	*υ*
No. 1	26.8	2.62	3.12	27700	0.2
No. 2	27.8	2.61	3.49	31500	

Table 2 Summary of Material Properties for Steel Reinforcement.

Specimen		Specification	*ϕ*, mm	*fy*, MPa	*ft*, MPa	*Es*, MPa	*υ*
No. 1	Main steel rebar	SD295A	16	349	487	200000	0.3
No. 2 (inner)	Hoop steel rebar		10	354	511		
No. 2 (outer)	Main steel rebar	SD345	16	398	552		
	Hoop steel rebar		10	421	581		

Table 3 Summary of Material Properties for PCM.

Specimen	*f'c*, MPa	*ft*, MPa	*fb*, MPa	*Ec*, MPa	*υ*
No. 2	47	1.84	8.49	21700	0.2

PCM: for this study, the PCM is assumed to be isotropic material, where the properties of the PCM can be shown in **Table 3**.

3.3 Modeling methodology

Due to the symmetrical nature of the control and retrofit piers, only the half of the control pier (No.1) and retrofit pier (No.2) were modeled. This approach reduced computational time and computer disk space requirements significantly. The steel reinforcement was simplified in the model by ignoring the inclined portions of the steel bars present in the test piers. Ideally, the bond strength between the concrete and steel reinforcement should be considered. However, in this study, perfect bond between materials was assumed.

The thickness of the PCM created discontinuities, which were not desirable for the finite element analysis. Perfect bond between concrete and PCM was assumed. A convergence study was carried out to determine an approximate mesh density. **Figure 7** shows the finite element model.

4. Comparison of Results

4.1 Lateral load-lateral displacement

Lateral deflections were measured at the centre of the top face of the piers. **Figure 8** shows the lateral load-lateral displacement plots for control pier (No.1) and retrofit pier (No.2). The numerical lateral load-lateral displacement plots in the quadratic interpolation line. In general, the load-deflection plots and design lateral load-lateral displacement plots for the piers from the finite element analysis agree quite well with the experimental data, design value was calculated based on specifications for highway bridges PART V about seismic design[4]. After first cracking, the stiffness of the finite element model is again higher than that of the experimental design piers. There are several effects that may cause the higher stiffnesses in the finite element models. First, microcracks are present in the concrete for the experimental piers, and could be produced by drying shrinkage in the concrete and/or handling of the piers. On the other hand, the finite element models do not include the microcracks. The microcracks reduce the stiffness of the experimental piers. Hereinafter, perfect bond between concrete and steel reinforcing is assumed in the finite element analysis, but the assumption would not true for the experimental piers. As bond slip occurs, the composite action between the concrete and steel reinforcing is lost. Thus, the overall stiffness of the experimental piers is expected to be lower than for the finite element models (which also generally impose additional constraints on behavior).

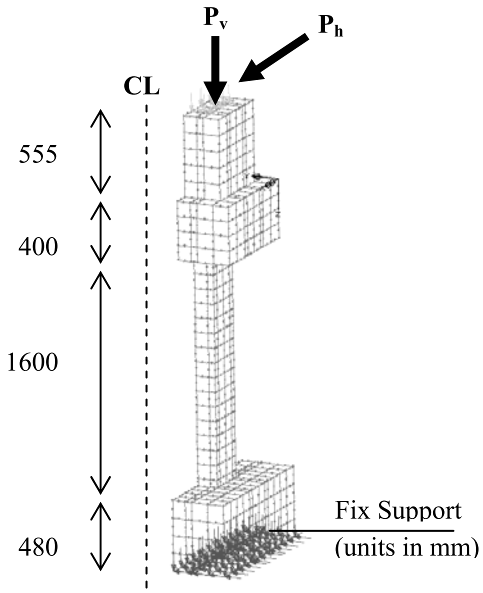
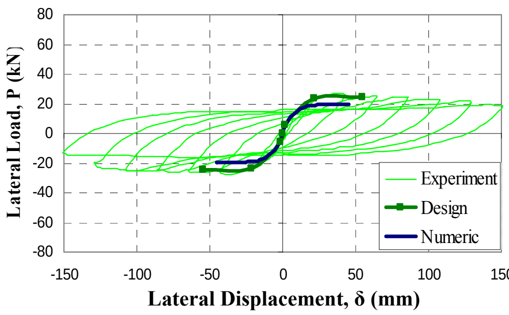
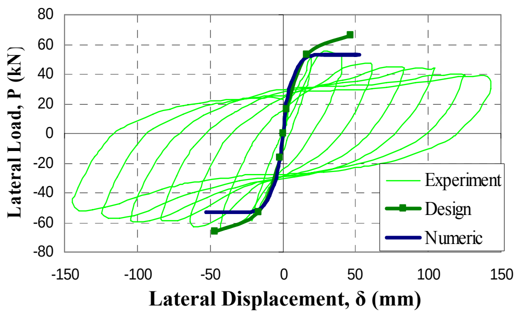


Fig. 7 Finite Element Model.



(a) No.1



(b) No.2

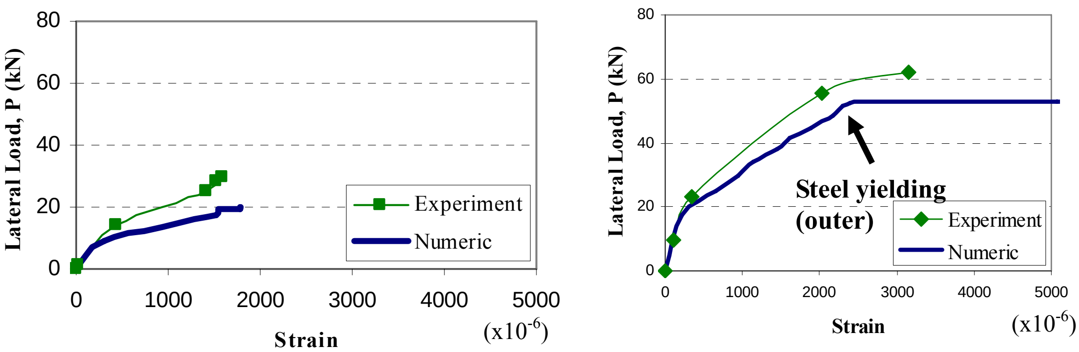
Fig. 8 Lateral Load-Lateral Displacement.

4.2 Tensile strain in main steel reinforcing

Comparison of the load-tensile strain in main steel reinforcing plots from the finite element analysis and experimental data for the main steel reinforcing at the height of 75 mm are shown in **Fig.9**. For both the specimen No.1 and No.2 of piers in the linear range (before concrete cracking) the strains from the finite element analysis correlate well with those from the experimental data. In the nonlinear range, the trends of the finite element and the experimental results are generally similar. The finite element analysis supported the experimental results that the main steel rebar at the height of 75 mm for specimen No.1 has not yielded at failure, while the steel rebar for the specimen No.2 of pier yields but at a lower load.

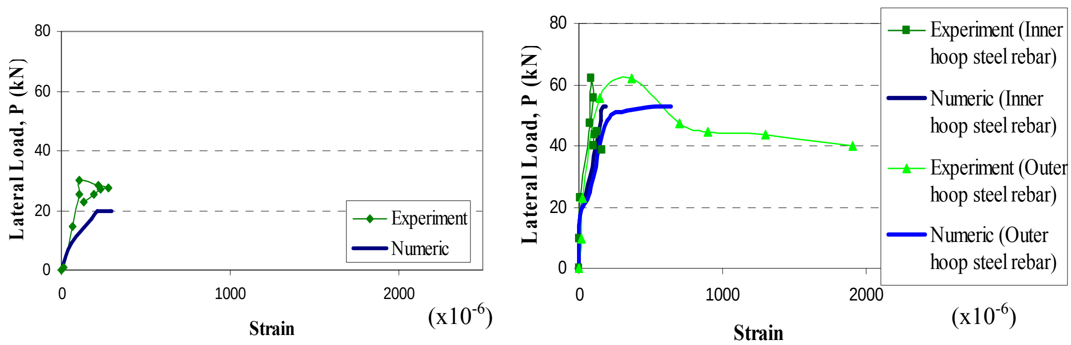
4.3 Tensile strain in hoop steel reinforcing

In this analysis, both hoop steels and concrete are considered as perfect bonding. Hereinafter, the strain in x-direction can be thought as the strain of hoop steels. As shown in **Fig.10**, the tensile strain in specimen No.1 grows with increase of deflection deformation. This condition is the same to that of test, which the inclining cracks appears and widened, and the shear failure happened, lastly. However, after PCM were used to retrofit seismic performance of columns (specimen No.2), the lateral strains were restrained and become smaller and almost keep constant after ultimate capacity at inner hoop steels. But, hoop steels at outer become larger.



(a) No.1 (b) No.2

Fig. 9 Lateral Load-Tensile Strain for Main Steel Rebar.



(a) No.1 (b) No.2

Fig. 10 Lateral Load-Tensile Strain for Hoop Steel Rebar.

4.4 Yield and ultimate lateral load

Table 4 shows comparison between the yield and ultimate lateral loads of the experimental, design and FEA values. The final loads for the finite element models are the last applied load steps before the solution diverges due to numerous cracks and large deflections. It is seen that the LUSAS models underestimate the strength of the piers, as anticipated.

In the experiment, the failure modes for the piers were predicted. The specimen No.1 (as the control pier) failed in shear. The specimen No.2 (retrofit pier) failed in flexure, with yielding of the steel reinforcing. Crack patterns obtained from the finite element analysis at last converged load steps and the failure modes of the experimental piers agree very well. For the finite element model of the specimen No.1, smeared cracks spread over the high shear stress region and occur mostly at the ends of the column from the bottom of column. The side of crack occurs following the lateral loading direction. Diagonal cracking has a marked effect on the failure mechanism of bridge columns, and can develop well outside the potential plastic hinge zone[5], particularly in columns with inadequate transverse reinforcement, as was shown for the specimen No.1. When the lateral load reached 27.3 kN, the column longitudinal bars were suddenly separated from the core concrete, the lateral load dropped significantly, and the test was terminated[6]. The finite element program accurately predicts that the specimen No.1 fails in shear. For the specimen No.2, numerous cracks occur at the ends of the column from the bottom of column rather than underneath near support location. The crack pattern and steel yielding at the bottom of column (near support) for the finite element retrofit pier the experimental results that the pier fails in flexure.

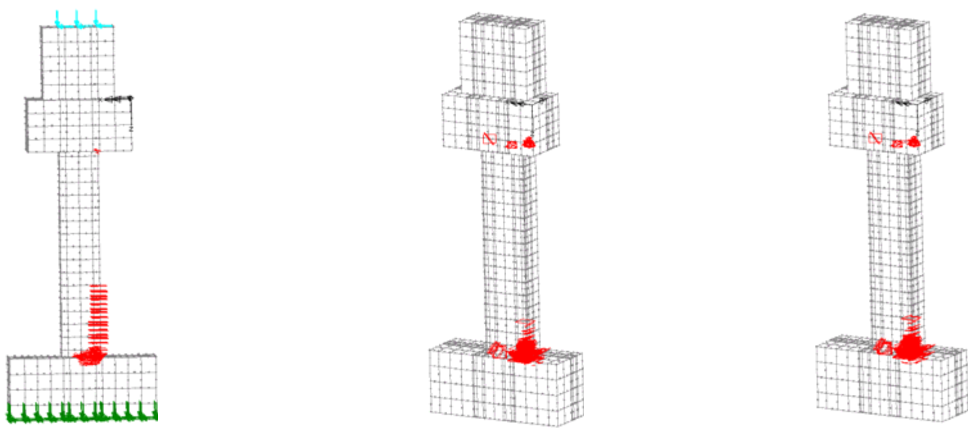
4.5 Crack propagation for concrete

The LUSAS program records a crack pattern at each applied load step. **Figure 11** shows crack propagations developing for each pier.

Table 4 Comparison of Yield and Ultimate Lateral Loads.

Pier	Yield Lateral Load (kN)			Ultimate Lateral Load (kN)			
	Exp.	Design	FEA	Exp.	Design	FEA	% Difference
No.1	25.3	23.6	18.0	27.3	24.3	20.0	26.7%
No.2	52.2	52.9	48.0	64.3	66.2	53.0	17.6%
Percent Gain over Control Pier	106%	124%	167%	136%	172%	165%	-

P_h = 20 kN (The final load from FEA) P_h = 48 kN (Main steel rebar yielding) P_h = 53 kN (The final load from FEA)



(a) No.1 (b) No.2

Fig. 11 Crack Propagations.

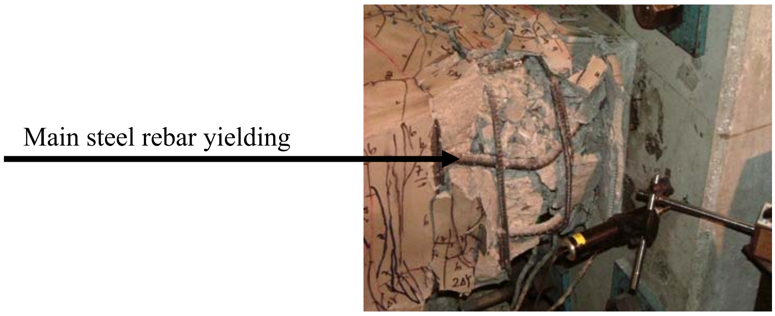


Fig. 12 Main Steel Rebar Yielding in Experiment.

The cracks appear at the bottom of column on the specimen No.1. For the specimen No.2 model the crack pattern and steel yielding at the bottom of column (near support). Hereinafter, for experimental test result can be shown in **Fig.12**.

5. Conclusions

The general behaviors of the finite element models showed good agreement with observations and data from the experimental full-scale pier test.

(1) Perfect bond between concrete and steel reinforcing was assumed in the finite element analysis.
(2) The finite element program accurately predicted that the specimen No.1 fails in shear and the specimen No.2 fails in flexure. These conditions were similar with experimental results.
(3) The finite element analysis supported the experimental results that the main steel rebar at the height of 75 mm for specimen No.1 has not yielded at failure, while the main steel rebar for the specimen No.2 yields. The lateral load increased the load carrying capacity by 136% for experimental pier and by 165% for the finite element model.

Acknowledgements

The authors wish to express their gratitude to Association of PCM shotcrete method for RC structures, Japan for financial support for this research, Mr. Kenji KOBAYASHI (Doctoral student in Kyushu University) and Mr. Lee Tung PHUONG (Master student in Kyushu University) for their help and collaboration in this work.

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