- Technical paper -

NUMERICAL STUDY ON SEISMIC BEHAVIOR OF RC BRIDGE PIERS WITH BOND CONTROLLED REINFORCEMENT

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ABSTRACT: Three Dimensional nonlinear finite element analysis was carried out to verify the enhancement of seismic performance of reinforced concrete bridge piers such as shear strength and ductility by controlling bond of longitudinal reinforcement. Proposed analytical method was found to model the global hysteretic behavior and failure mode of unbonded RC piers producing an excellent correlation with that of experimental results. The enhancement of seismic performance and alteration of the failure mode of unbonded pier was found to be due to the change in internal mechanism to the one resembling a tied arch.
KEYWORDS: reinforced concrete pier, seismic performance, finite element analysis, unbonded bars, bond, shear, ductility

1. INTRODUCTION

Hyogoken-Nanbu Earthquake in 1995 has given numerous examples of catastrophic shear failure of RC bridge piers leading to their fatal collapse [1]. Following the earthquake, design earthquake load in Japanese design codes has been drastically increased. To satisfy the seismic performance required by new design codes, an enormous amount of shear reinforcements have to be provided in RC bridge piers. A large quantity of reinforcement however makes its arrangement complicated and congested creating constructability problems [2]. It is therefore important to look for some alternative methods to improve shear capacity without relying heavily on shear reinforcement alone.

Elimination of the bond between longitudinal bar and concrete leads to a major change in stress distribution inside the concrete [3]. By unbonding longitudinal bars, no flexural crack occurs in the unbonded shear span of RC piers and concrete body mainly remains under the state of diagonal compression [4,5]. Since this behavior is favorable in preventing shear failure, Pandey et al. [6] performed an experimental study to investigate the seismic performance of RC piers reinforced with poorly bonded and completely unbonded longitudinal bars. The results showed that with unbonding longitudinal bars, mode of failure of RC bridge piers can be completely changed from undesirable shear mode to a ductile flexural mode.

The main purpose of this study is to develop an analytical model to investigate the enhancement of seismic performance of RC bridge piers with bond controlled longitudinal reinforcement. The aim is also to clarify the change in the internal mechanism which is responsible for the abrupt change in the behavior of piers with unbonded longitudinal bars compared to those with ordinary deformed bars.

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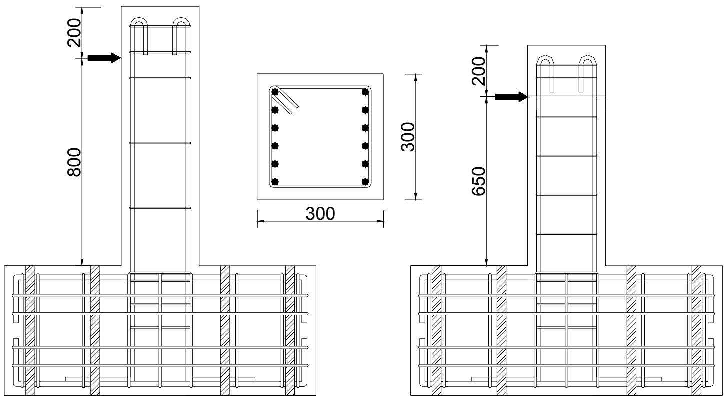
2. EXPERIMENTAL STUDY

In order to study the influence of unbonded reinforcement on seismic behavior of RC piers, Pandey et al. [6] conducted an experimental investigation. **Table 1** shows the description of four specimens tested under reverse cyclic loading.

Table 1- Specimen description with experimental parameters

Sp. No.	a/d ratio	Bond condition	Concrete f_c', MPa	Longitudinal bars		Lateral ties	
				A_s	f_y, MPa	Size and spacing	f_{wy}, MPa
A-1	3.0	ordinary deformed bars	32.54	12-D16	380.18	D6@250 mm	396.60
A-2		unbonded deformed bars	33.69	12-D16	380.18	D6@250 mm	396.60
B-1	2.5	ordinary deformed bars	28.76	12-D16	380.18	D6@150 mm	396.60
B-2		unbonded deformed bars	30.47	12-D16	380.18	D6@150 mm	396.60

Specimens were categorized into Series-A and Series-B depending on their a/d (shear span-to-depth) radio. Specimens A-1 and B-1 of Series-A and Series-B respectively were purposely designed to fail in shear. The ratio of shear strength to flexural strength for both the series was 0.8. **Fig. 1** shows the details of the test specimen. Twelve bars with diameter of 16 mm were provided as longitudinal bars and the amount was kept constant in both the series. Deformed bars with diameter of 6 mm were used as lateral reinforcement and were provided at the spacing of 250 mm and 150 mm in Series-A and Series-B respectively. In specimen A-2 and B-2 longitudinal bars from the bottom of the column to the loading height were completely unbonded by inserting them into spiral sheaths before casting the specimens.



Series-A Series-B

Fig.1-Details of test specimens

3. FINITE ELEMENT ANALYSIS

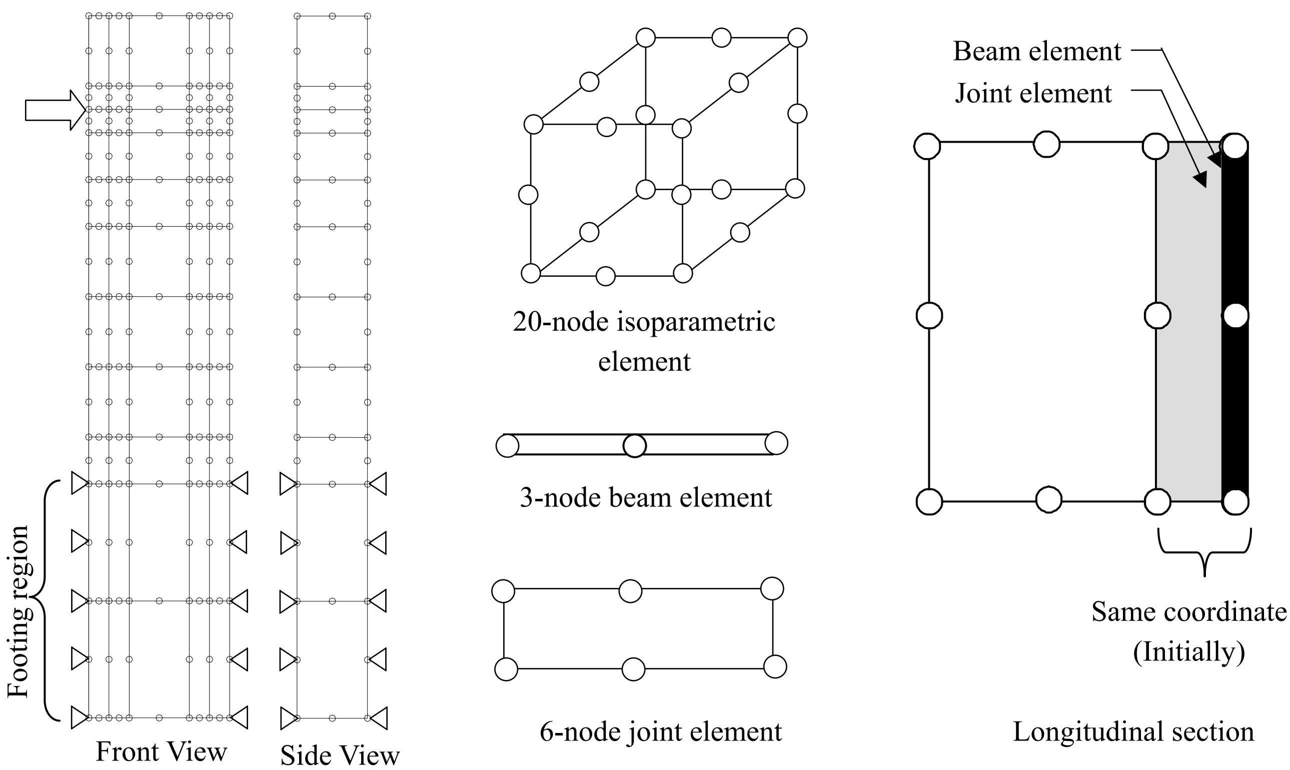
Three-dimensional nonlinear finite element analysis was performed to represent the quasistatic reversed cyclic loading test.

3.1 NONLINEAR MATERIAL MODEL

The nonlinear material model for reinforced concrete is composed of several models to characterize the behavior of concrete and reinforcing bars. Models for concrete can be broadly divided into the one before and after cracking which is separated by a cracking criterion. In order to model the nonlinearity of concrete before cracking, elasto-plastic fracture (EPF) model is used [7]. For the cracked concrete, the constitutive relations for reinforced concrete based on average stress average strain smeared crack concept are employed. Once the concrete gets cracked, the behavior turns to anisotropic in the crack direction. The orthogonal four-way fixed crack model is used to obtain three constitutive laws of cracked concrete including normal stress transfer parallel and normal to the crack axis and shear stress transfer along the crack interface which are termed as compression, tension and shear transfer model respectively. Compression Model is similar to the EPF model which assumes uniaxial stress condition and uses equivalent stress-strain relationship [8]. The effect of the orthogonal tensile stress on the compressive strength is considered by using compression field theory [9]. Tension stiffening model [10] and tension softening model are used to model the behavior of concrete in tension. Shear transfer models are used to model shear stress transfer along the crack surface. Steel model capable of representing Bauschinger effect is used in the analysis [11]. In 3-D constitutive model, post cracking formulation derived from uniaxial tension is generalized to the spatially arbitrarily inclined crack. Anisotropic concrete tension fracture and mean yield level of reinforcement is evaluated my 3-D RC zoning concept [12].

3.2 FINITE ELEMENT MODEL

Finite element mesh of the RC pier is as shown in the **Fig. 2** (a). **Fig. 2** (b) shows the details of the elements used in the model. Twenty-noded isoparametric solid element with eight point gauss integration scheme is used to model concrete. Reinforcing bar is modeled as one dimensional three-noded beam element with six degrees of freedom per node including three translation and three rotation degrees of freedom. Bond between steel and concrete is modeled by one dimensional six-noded joint element. Schematic diagram of the connection of beam element to solid element with joint element is shown in Fig. 2 (c). By varying shear slip stiffness of the joint element, various bond conditions from the perfect bond as that of ordinary RC column to the perfect unbond can be achieved. In footing region, concrete is modeled as solid elastic concrete elements with the boundary nodes restrained in all three directions.

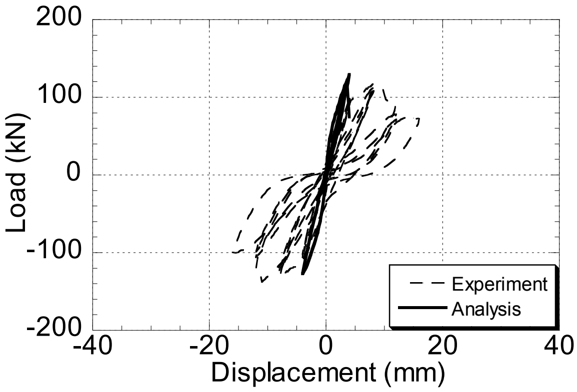


(a) Finite element mesh (b) Type of elements (c) Beam and joint elements

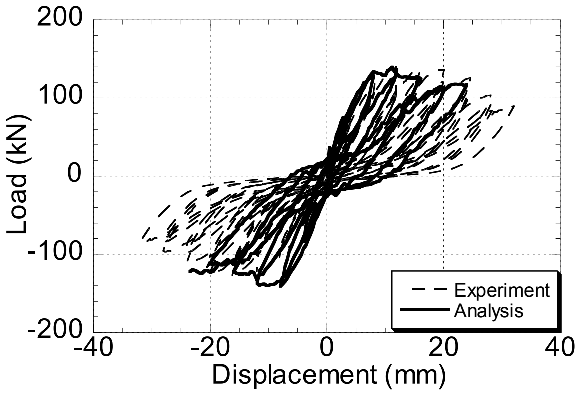
Fig. 2 Details of finite element model

3.3 RESULTS AND DISCUSSION

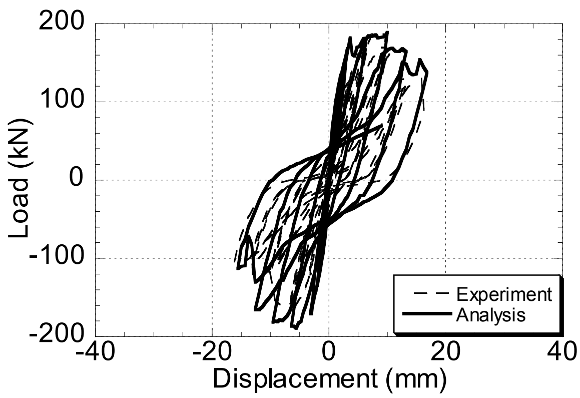
Cyclic analysis on RC bridge piers is conducted to characterize the global hysteretic behavior including stiffness, strength, ductility, energy absorption capacity and residual deformation. The load displacement curves determined at the loading point were measured in the experiment and calculated in the simulation. The comparisons of these curves are shown in **Fig. 3**.



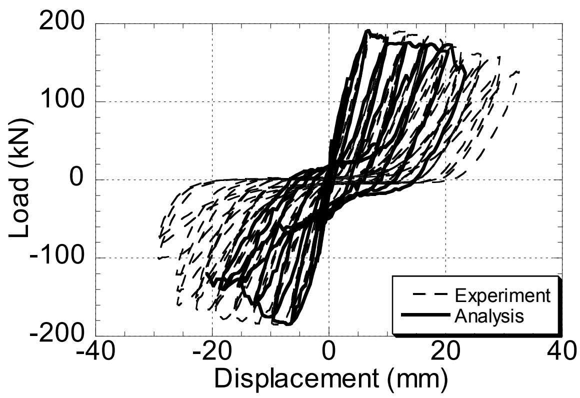
(a) A-1 (Control specimen with a/d = 3.0)



(b) A-2 (Unbonded specimen with a/d = 3.0)



(c) B-1 (Control specimen with a/d = 2.5)



(d) B-2 (Unbonded specimen with a/d = 2.5)

Fig. 3-Comparasion of experimental and analytical hysteretic behavior

The analytical results show a close agreement with the experimental results. Specimen A-1 failed in shear prior to the yielding of longitudinal bar while specimen B-1 failed in shear after flexural yielding. By unbonding longitudinal bars both specimens A-2 and B-2 showed a change in the failure mode from shear to flexure. A ductile behavior is also observed in unbonded piers. **Table 2** shows the comparison of analytical and experimental results.

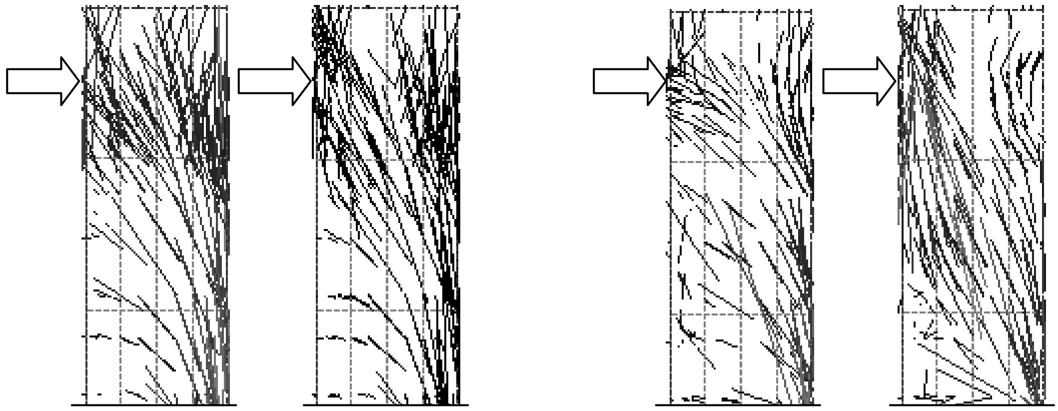
Table 2 - Comparison of maximum load and ductility

Specimen	Maximum load (kN)			Ductility		
	Experimental	Analytical	Exp./Ana.	Experimental	Analytical	Exp./Ana.
A-1	125.50	129.46	0.97	-	-	-
A-2	133.16	137.10	0.97	3.62	3.02	1.20
B-1	188.48	189.18	1.00	-	-	-
B-2	189.43	191.53	0.99	4.48	3.44	1.34

It can be clearly observed that the proposed numerical analysis method can well predict the maximum load carrying capacity. The model however is found to underestimate the ductility. Fig. 3 shows that the global hysteretic behavior including area of energy absorption and residual displacement can be well predicted by the proposed model.

3.4 STRESS DISTRIBUTION

The stress flow diagram obtained from the analysis for specimens B-1 and B-2 both before and after the initiation of the first flexural crack is shown in **Fig. 4**. Specimen B-1 showed a curvilinear flow of principal compressive stress both before and after the occurrence of flexural crack. This stable behavior is attributed to the stress transfer from steel to concrete due to the presence of bond.



B-1 B-2 B-1 B-2

(a) Before the first flexural crack (b) At the lateral load of 100kN

Fig. 4-Stress flow diagram

Before the first flexural cracking, stress flow pattern of Specimen B-2 is similar to the one of B-1. Before cracking, the tensile stress developed in the concrete is responsible for this type of stress flow. A clear change in the behavior of Specimen B-2 is observed after the appearance of the first flexural crack. The stress in the column is transferred from loading point to the support by straight diagonal thrust lines resembling the behavior of tied arch. Since the pier is in the state of diagonal compression, the stress condition of concrete is favorable in preventing shear failure.

4. CONCLUSION

3-D nonlinear finite element analysis was carried out on RC piers with both bonded and unbonded longitudinal reinforcements. Based on this study following conclusions can be drawn:

1. Described model is able to model the behavior of both bonded and unbonded RC piers with reasonably accurate estimation of maximum load, failure mode and ductility.
2. An abrupt change in the behavior of unbonded pier occurs after the initiation of the first flexural crack which is an onset to new mechanism.
3. Behavior of unbonded pier matches to the one of tied arch with a straight thrust line joining support and loading point. Due to this stress transfer mechanism, unbonded piers do not fail in shear and shows an enhanced ductility.
4. Small amount of hysteretic energy absorption is evident in the unbonded piers which are basically due to the opening of wide flexural crack at the column-footing joint.

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