EFFECT OF SHEAR REINFORCEMENT ON FAILURE MODE OF RC BRIDGE PIERS SUBJECTED TO STRONG EARTHQUAKE MOTIONS

Atsuhiko MACHIDA[1] And Khairy H ABDELKAREEM[2]

SUMMARY

Nonlinear 3D FEM was utilized to carry out inelastic response analysis of RC piers subjected to earthquake motions in order to analyze the role of shear reinforcement on final failure mode of the piers. Final failure mode of RC piers with different ratios of shear reinforcement, axial stress level, main reinforcement ratios and L/h ratios was determined considering different typesof damage indices of concrete such as; shear and normal stresses and strains, axial and lateral strains, plastic strains in axial, lateral or principle directions and cracking patterns. In the analysis, plastic strains (axial, lateral, and principle) were focused since the level of collapse depends on the quantity and sign of the plastic strains. Final failure mode of piers is highly affected by shear reinforcement ratio and axial stress level, remarkably affected by pier size and slightly affected by main reinforcement ratio. Failure mode changes significantly from severe diagonal failure to flexural failure by increasing shear reinforcement ratio and the rate of change depends on the mentioned parameters. From such comprehensive analysis, we plotted design curves from which the designers can determine the required shear reinforcement ratio at which failure mode changes from diagonal shear failure to flexural failure. A model was proposed to determine the plastic hinge height at which high plastic strains occur. Experimental results were carried out using cyclic loading tests and the results were used to verify the analytical ones.

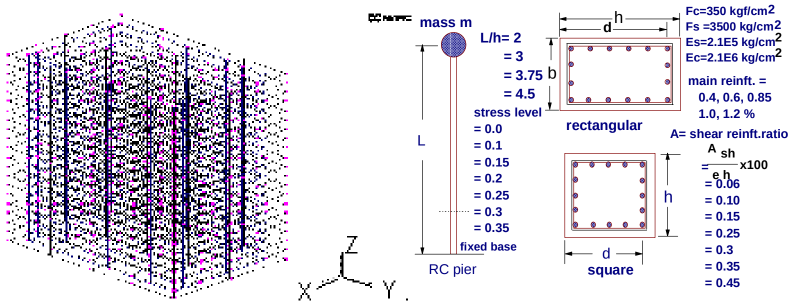
INTRODUCTION

During recent earthquakes, such as the Great Hanshin Earthquake in Japan, January 17, 1995, many of the reinforced concrete piers or columns of highway and railway bridges suffered mainly severe diagonal shear failure in addition to other features of damages. The recognized reason for such severe collapse was that the piers were designed in such a way that they were provided with insufficient quantities of shear or lateral reinforcement which resulted in non-ductile behavior of the piers during the motion. Such severe collapse makes us to more deeply recognize that many of our RC bridges did not have sufficient strength and ductility to resist strong ground shaking. It is very important to analyze dynamic failure mechanisms and to investigate the causes of the severe shear failure. An accurate estimation of ductility levels is also necessary. As demonstrated in the previous studies (Machida and Mutsuyoshi 1988, Okamura and Maekawa 1991, Maekawa and Shawky 1997 and Bathe 1996), shear reinforcement has significant role on final failure mode of RC piers. Shear reinforcement shares both of concrete and longitudinal reinforcement in resisting shear stresses and thus increases shear strength of piers. Also, shear reinforcement increases ultimate strength and the corresponding strain due to confinement, increases ductility level of piers and prevents buckling of longitudinal bars. In the previous studies, the effect of shear reinforcement on failure mode of piers was determined either experimentally or theoretically based on two-dimensional analysis, which is not accurate enough for such structures. Consequently, it is required to evaluate accurately the limits of the quantities of shear reinforcement, which are needed to prevent or to reduce the occurrence of diagonal failure. In this study, nonlinear 3D FEM approach was utilized to clarify the role of shear reinforcement on final failure mode then to determine the limits of shear reinforcement ratios at which failure mode changes from diagonal shear to flexural failure.

In 3D model, concrete was modeled as 8-node isoparametric 3D element and 2-node 3D-truss element for modeling both of longitudinal and transverse reinforcement. The super structure was represented by concentrated

[1] *Prof., D. Eng., Saitama University, Japan*
[2] *Assistant Professor, D. Eng., Assuit University, Egypt*

mass at top of the pier and mass of pier was lumped at the joints. Nonlinear Newmark approach (Bathe 1996) was used to solve the nonlinear equation of motion. A finite element software named MARC was utilized in the analysis. Figure 1 (a, b) shows the simplified 3D FE model and parameters of the study respectively. Von Mises criterion with normality flow rule was adopted to consider nonlinearity of steel reinforcement. Nonlinearity of concrete was adopted through constitutive equations, which were considered in two stages. For uncracked concrete, we used a model based on theory of plasticity (Zhishen and Takada-aki 1990 and Ge and Usami 1994) to model concrete in compression as in Figure 2. We modified the model to consider the effect of transverse reinforcement on increasing both of ultimate strength and corresponding strain due to confinement. The modification was based on a model proposed by Mander et al (Park and Paulay 1990, Watson and Park 1994 and Park and Priestley 1982) as in Figure 3. For cracked concrete, we used constitutive equations based on smeared crack model as shown in Figure 4 (a, b). Just the cracking occurs perpendicular to the principle tensile stress, then concrete drops into orthotropic coordinates with axes parallel and normal to cracks (Lotfi 1992). More details were illustrated (Khairy Hassan 1999). In this study, shear reinforcement ratio was defined as in which is the area of stirrup branches in the direction considered, and are spacing of stirrup and depth of the pier respectively. Stress level was defined as in which P is the axial load, is the concrete strength and is the area of concrete section. Percentage of main reinforcement was defined as where is the area of main steel in tension side, d and b are effective depth and width of pier respectively. For each case of study, the pier was subjected to El Centro motion at the base, which was scaled to have 0.8g peak acceleration to bring it to the same level of severity of the Great Hanshin earthquake. In other cases, piers were subjected to cyclic displacement at top then hysteresis loops were plotted. The study was based upon stresses, strains, and principal and lateral plastic strains. Each pier was subjected to concentrated mass at top from the superstructure in addition to lumped mass of the pier at the joints.



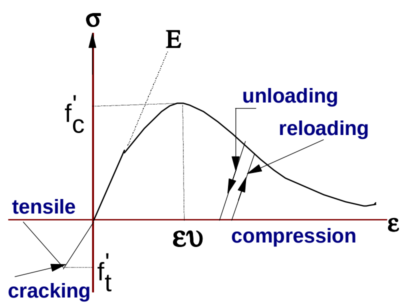
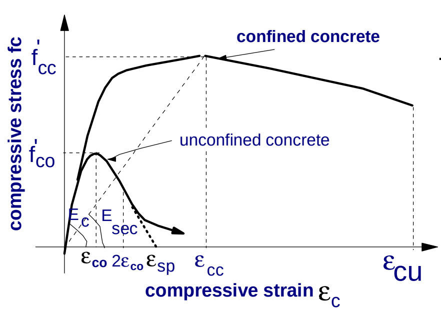


Figure 1 a) 3-D model. b) Parameters of the study. Figure 2: Modeling of concrete in compression.



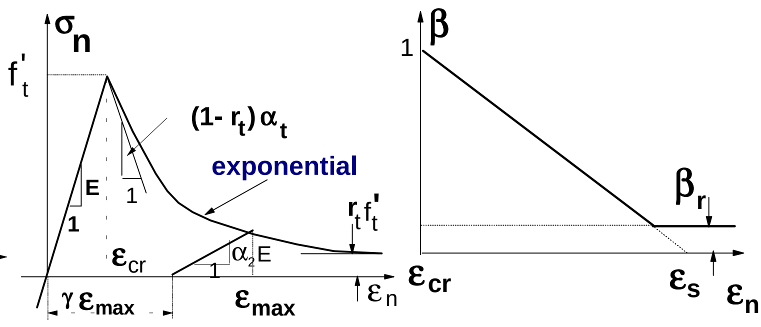
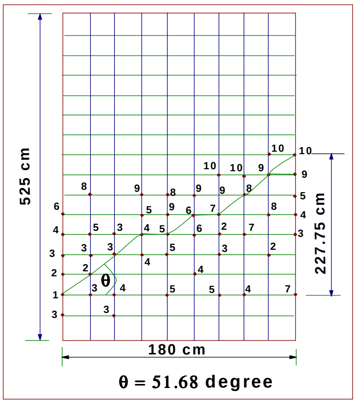


Figure 3: Park model. Figure 4 a) Tension stiffening. b) Retention factor.

FAILURE MODE

In the analysis, propagation and angle of inclination of cracking were obtained during each increment of applying the motion. Figure 5 (a, b) illustrates crack propagation at a certain height of pier with shear reinforcement ratios of 0.05 % and 0.35 % respectively. In Case A, failure mode was diagonal and angle of inclination for the most severe cracking was 51.68 degrees. In case B, failure mode was flexural and the angle of inclination of cracking line ranged between 19.8~28.4 degrees. Effect of shear reinforcement on cracking pattern, propagation of cracking and angle of inclination is very clear from sequence of numbering of cracked points. In our analysis, we considered principle and lateral plastic strains and stress strain distribution as damage indices in addition to cracking pattern and its angle of inclination. We give few analytical and experimental cases and the readers can refer to Khairy Hassan (1999) for more illustration.



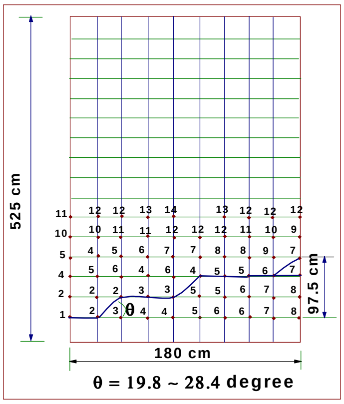


Figure 5a) Crack propagation for RC pier with shear reinforcement = 0.05%.

Figure 5b) Crack propagation for RC pier with shear reinforcement = 0.35%.

We analyzed RC pier with three different shear steel ratios and we obtained the principle plastic strain at the position where cracking occurred. Figure 6 illustrates the principle plastic strain as time domain for the pier with different shear reinforcement ratios of 0.05, 0.12 and 0.35 % respectively. The principle strain was calculated at the shown height of the pier at each increment at the indicated levels for each case. In the first case, tensile plastic strain occurred with big quantity associated with diagonal shear failure at height of 45 cm. The quantity and position of tensile plastic strain change significantly with the increase of shear steel to 0.12 % as it is shown. The sign of plastic strains changed from tension to compression by increasing shear reinforcement from 0.05 to 0.35 %. Increasing shear reinforcement from 0.05 to 0.35 %, failure mode changed from severe diagonal (associated with tensile plastic strains) to flexural failure (associated with compressive plastic strain).

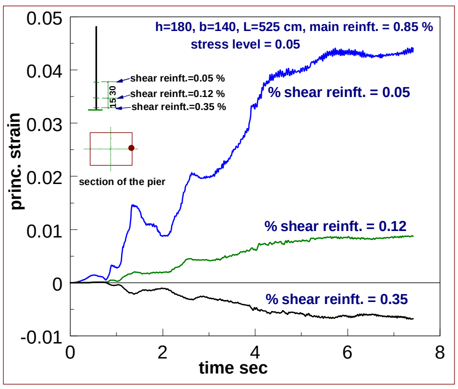
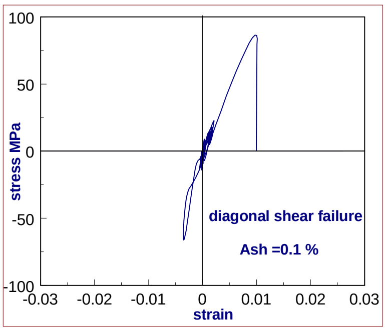


Figure 6: Effect of shear reinforcement on principle plastic strain of RC pier.

For more illustration and to analyze the reasons of failure, we analyzed a square RC pier of 200 cm width, 600 cm height and main reinforcement ratio of 1.0 %. Figure 7 (a, b) illustrates the lateral stress strain relation at the collapsed position of the pier for two ratios of shear reinforcement of 0.1 and 0.3 % respectively. The collapsed position in the first case (0.1 %) was obtained at 50 cm height from the base of the pier whereas it was at the base in the second case (0.3 %). In the first case (0.1 %), failure occurred due to tensile stresses associated with diagonal shear failure. In the second case (0.3 %), collapse occurred due to compressive stresses associated with flexural failure.



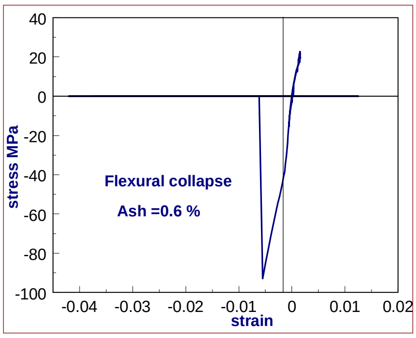


Figure 7: Lateral stress strain relaion at collapsed position of RC pier.
Case a) Shear reinft.= 0.1%. Case b) Shear reinft.=0.3%.

The effect of shear reinforcement on changing of failure mode from diagonal to flexural mode was very obvious from the experimental results, which we have carried out in the Structural Material Laboratory of Saitama University using cyclic displacement (Khairy Hassan 1999). Figure 8 shows photos of final collapse of specimens B, D and H of shear reinforcement ratios of 0.118, 0.236 and 0.472 % respectively. In specimen B, severe diagonal crack occurred and failure mode was diagonal failure. This crack almost disappeared in specimen H. and its final failure was flexural failure. Based on comprehensive analysis of many piers, we concluded that shear reinforcement has significant influence on changing failure mode of bridge piers from diagonal shear failure to flexural failure and this influence depends greatly on axial stress level at top of the pier, main reinforcement ratio and L/h ratio. This is a matter of course, however we considered herein many indices to clarify this conclusion and to establish the following criteria. If angle of inclination of cracking $\alpha \geq 45$ degree and associated with tensile principle plastic strain and based on propagation of the cracking then failure mode is diagonal (at a certain height above base). If $\alpha \leq 45$ degree and associated with compressive plastic strain and based on propagation of cracking, then failure mode is flexural (near the base). In the following part of study, we are trying to answer the following question: what is the required shear reinforcement ratio at which failure mode changes from diagonal to flexural mode?

Figure 8: Photos 1, 2 and 3 for final failure mode of tested specimens B, D and H respectively.
Shear Reinft.=0.115%. Shear Reinft.=0.236%. Shear Reinft.=0.472%.

RECOMMENDED RATIOS OF SHEAR REINFORCEMENT

Figure 9 (a, b) illustrates effect of shear steel on changing both of the quantity and sign of principle plastic strain (Figure 9a) and lateral plastic strain (Figure 9b). The effectiveness of shear reinforcement on carrying shear stresses is very high at the beginning because of occurrence of diagonal cracking of concrete and this is why the rate of decrease of plastic strain is high. If shear reinforcement ratio is low, diagonal cracking occurs and hence concrete does not carry shear stresses. In that case, shear reinforcement carries mainly shear stresses. This explains why the rate of decrease of the curve is significant in the beginning. However, if shear reinforcement ratio is high, then diagonal cracking does not occur and hence concrete section carries shear stresses. In this case, the role of shear steel on carrying shear stress is reduced and its function tends to confine the concrete core. The rate of decrease of tensile strain becomes low after certain limit because shear steel carries little shear stresses. Increasing shear steel after certain limit will increase of confinement.

This is supported by the result of Figure 10 in which cracking angle was plotted for piers having different shear steel ratios and stress levels. The rate of decrease of angle of cracking is high in the beginning then it becomes smaller after the indicated limits for different shear steel ratios as it was explained. In the analysis, the authors have utilized both of the principle plastic strain and lateral plastic strain to determine the ratios of shear reinforcement required to avoid diagonal shear failure, i.e. to avoid occurrence of tensile plastic strain as shown in Figure 9 (a, b). As shear steel ratio increases, principle and lateral tensile plastic strain decrease gradually till a certain level at which sign of plastic strain changes from tensile (associated with shear failure) to compressive (associated with flexural failure).

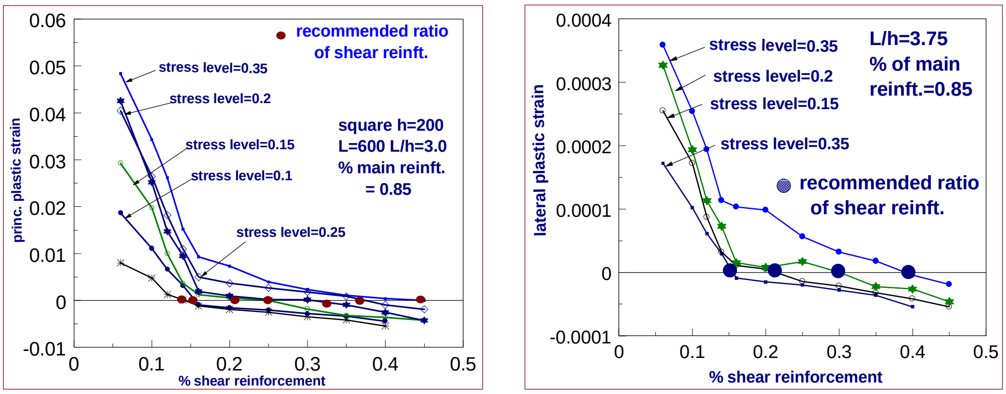


Figure 9: Determination of the recommended ratios of shear reinforcement from
a) Priciple plastic strain. b) Lateral plastic strain.

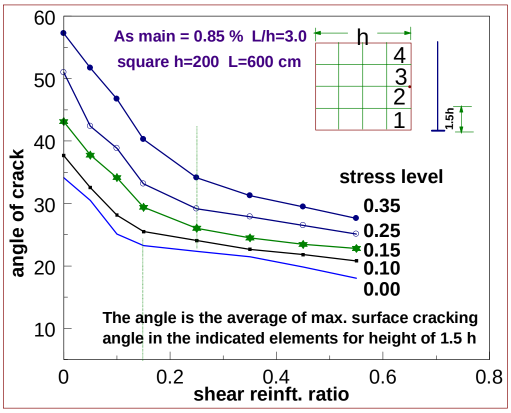


Figure 10: Effect of shear reinforcement on maximum angle of cracking.

The ratio of shear steel corresponding to change of the sign of plastic strain from tension to compression was determined and referred as the recommended ratio for design of RC bridge piers. The recommended shear reinforcement ratios were determined based on the condition that diagonal shear failure does not occur (tensile plastic strains vanish) taking into account effect of stress level, pier size and main steel ratio. The recommended ratios were plotted as groups of design curves which can be used for design. Figure 11 (a, b, c) illustrates examples of the different groups of design curves.

By investigating the curves, one can estimate the effect of shear steel, stress level, L/h ratio and main steel ratio on the behavior of piers. It was found that as stress level increases, the required ratio of shear reinforcement increases. This is due to crushing of concrete at the ends of the diagonal cracks which causes decrease of ductility of piers and hence, higher quantity of shear reinforcement is needed. Also, it was found that the piers having smaller L/h ratio behave in better way than that of bigger L/h ratio. As L/h ratio is high, effect has significant influence on reducing ultimate capacity of piers especially when stress level is high. Also, as size of pier increases, its shear strength decreases.

It should be mentioned that in deriving the design curves, the purpose was to eliminate completely the possibility of occurrence of shear failure by letting tensile plastic strain vanish. This explains why the recommended ratios of shear reinforcement are relatively high and in some cases not practical. It is more practical to allow some minor tensile plastic strain to occur and to determine the required ratios based on this assumption. As previously mentioned, the rate of decrease of principle plastic strain with the increase of shear steel is very high in the beginning then the rate decreases. The ratio of shear reinforcement corresponding to the significant change of the slope of the curve can be considered as recommended shear reinforcement ratio. This point is illustrated in Figure 12 for some cases. Following this assumption leads to reduction of recommended ratios of shear steel by 10 to 35 %. Higher value of reduction is pointed for piers with high stress level and vice versa.

We demonstrate that shear reinforcement has three main functions on behavior of RC piers; to carry shear stresses, to confine concrete core and to prevent buckling of main bars. In this study, the authors clarified the limits and the range of the functions of shear steel on carrying shear stresses. In another study, effect of shear steel on preventing buckling of main bars and confining concrete core was focused.

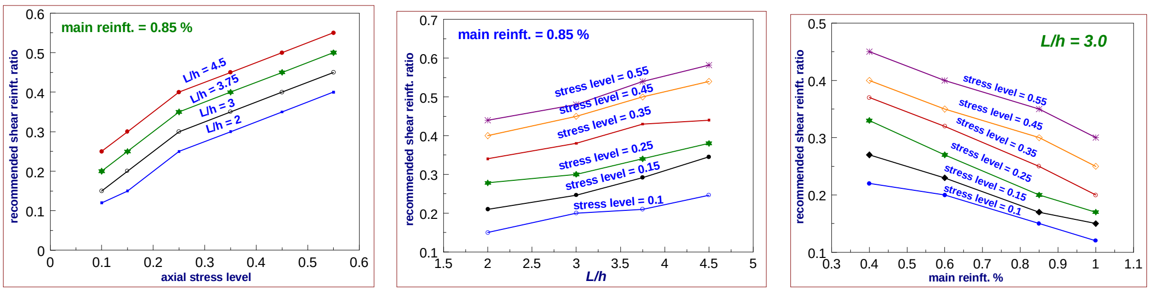


Figure 11: Recommended ratio of shear reinft.
a) An example of group A. b) An example of group B. c) An example of group C.

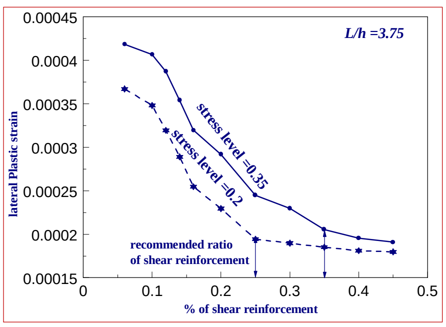


Figure 12: Recommended shear reinft. ratios based on allowing some tensile plastic strains.

THE PLASTIC HINGE LENGTH

Plastic strains are usually very high within a certain length measured from the base of pier where maximum straining actions occur. The mentioned length is referred as the plastic hinge length Lc. We determined the height of plastic hinge where plastic strain is 0.004 for many cases of RC piers with different properties We found that Plastic hinge length is highly dependent on axial stress level and L/h ratio. We proposed a numerical model to predict the plastic hinge length as a ratio of the pier width h considering the effect of stress level and L/h ratio. Figure 13 illustrates plot of the model for different L/h ratios The model is represented as follows:

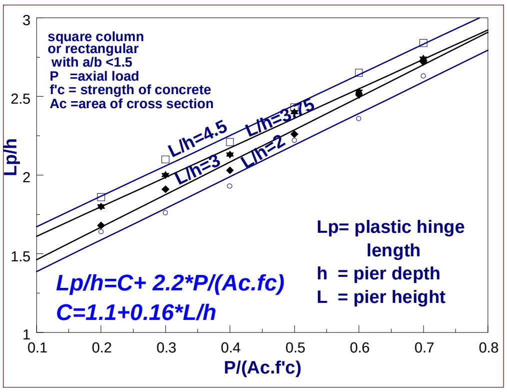


Figure 13: Proposed model to determine Plastic hinge height.

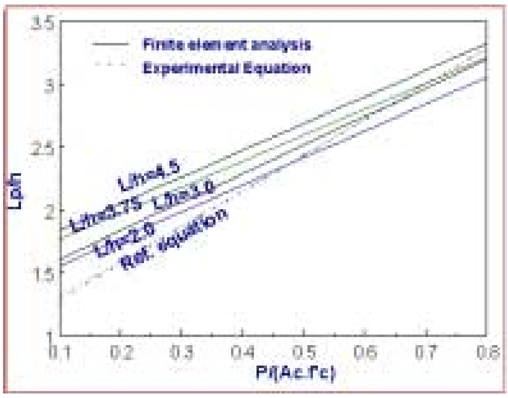


Figure 14: Comparison of the proposed model with reference model.

$$L_c/h = 2.2\, P/A_c f'_c + C \tag{1}$$

where C is a parameter depending on L/h ratio as follows:

$$C = 1.1 + 0.16\, L/h \tag{2}$$

Figure14 shows a comparison of the current model with a previous empirical model of Park and Paulay (1990) and Watson and Park (1994). The current model considered effect of axial stress level and L/h ratio and was based on results of actual earthquake. The reference model is an empirical one and considered only the effect of axial stress level. As a comparison, the current model gives higher values of plastic hinge length than the previous model and this is more safe. The current model is more conservative for piers subjected to earthquake motions.

CONCLUSIONS

Based on nonlinear 3D FEM and experimental results, we clarified that by increasing shear reinforcement of to a certain level, failure mode of RC pier changes from severe diagonal shear to flexural failure mode.

Groups of design curves were plotted to determine the recommended ratio of shear reinforcement which is required to avoid severe diagonal shear failure taking into account the effect of axial stress levels, main reinforcement ratio and height of pier relative to its width ratio.

An accurate model was proposed to determine the height of plastic hinge of RC pier considering effect of stress level and the height of the pier relative to its width.

REFERENCES

Bathe, K. (1996) "*Finite Element Procedures*", Printice Hall, Engelwood Clifs, U.S.A.

Ge, H. and Usami, T. (1994) "Strength Analysis of Concrete-Filled Thin-Walled Steel Box Columns", *J. of Construction. Steel Research* 30, 259-281.

Khairy Hassan (1999) "*Analytical Study on Seismic Resistant Characteristics of RC Bridge Piers*", A Thesis Submitted for Doctor of Engineering Degree, Department of Civil Engineering, Saitama University.

Lotfi, H. (1992) "*Finite Element Analysis of Fracture of Concrete and Masonry Structures*", Ph.D. thesis, Department of Civil Engineering, University of Colorado.

Machida, A. and Mutsuyoshi, H. (1988), "Evaluation of Ultimate Deflection of Reinforced Concrete Members", Proceeding of Ninth World Conference on Earthquake Engineering, Vol. VI, August.

Maekawa and Shawky (1997) "Collapse Mechanism of Hanshin RC Bridge Piers", *EASEC-5*, Australia.

Okamura, H. and Maekawa, K. (1991), "*Nonlinear Analysis and Constitutive Models of Reinforced Concrete*", University of Tokyo.

Park, R. and Priestley, N. (1982) "Ductility of Square-Confined Concrete Columns", *Journal of Structural Division*, Vol. 108, No. St1, January, 929-951.

Park and Paulay (1990) "Bridge Design and Research Seminar", Vol. 1, *Transit New Zealand*.

Watson, S. and Park, R. (1994) "Simulated Loading Tests on Reinforced Concrete Columns", *ASCE Journal of Structural Engineering*, Vol. 120, No. 1-6, 1825-1849.

Zhishen, Wu. and Takada-aki. (1990) "A Hardening / Softening Model of Concrete Subjected to Compressive Loading", *Journal of Structural Engineering*, Vol. 36B, March.