Computers and Concrete, *Vol. 14, No. 4 (2014) 387-403*
DOI: http://dx.doi.org/10.12989/cac.2014.14.4.387

Structural performance assessment of deteriorated reinforced concrete bridge piers

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(*Received May 9, 2012, Revised May 10, 2014, Accepted July 26, 2014*)

Abstract. The aim of this study is to assess the structural performance of deteriorated reinforced concrete bridge piers, and to provide method for developing improved evaluation method. For a deteriorated bridge piers, once the cover spalls off and bond between the reinforcement and concrete has been lost, compressed reinforcements are likely to buckle. By using a sophisticated nonlinear finite element analysis program, the accuracy and objectivity of the assessment process can be enhanced. A computer program, RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology), is used to analyze reinforced concrete structures. Material nonlinearity is taken into account by comprising tensile, compressive and shear models of cracked concrete and a model of reinforcing steel. Advanced deteriorated material models are developed to predict behaviors of deteriorated reinforced concrete. The proposed numerical method for the structural performance assessment of deteriorated reinforced concrete bridge piers is verified by comparing it with reliable experimental results. Additionally, the studies and discussions presented in this investigation provide an insight into the key behavioral aspects of deteriorated reinforced concrete bridge piers.

Keywords: structural performance; deteriorated; reinforced concrete; bridge piers; material models

1. Introduction

Many existing reinforced concrete structures deteriorate due to lack of attention to durability issues. Evaluation of such structures is needed to determine strength and safety at the time of investigation.

Deterioration in structural elements due to long term environmental attacks leading to carbonation and/or alkali-silica chemical reaction of concrete and corrosion of reinforcement will reduce structural stiffness and strength of the structure (Tapan 2007).

Corrosion of reinforcement is the most common type of deterioration of reinforced concrete bridge piers that may result in loss in the mechanical properties of reinforcement, loss of bond between concrete and reinforcement, and loss in effective cross-sectional area of steel.

Corroded reinforcement in deteriorated structures that are mainly subjected to dynamic loads can decrease their ductility and robustness significantly, because the ultimate strain and elongation of the reinforcement are reduced. Hence, ductility of reinforcements embedded in concrete, in

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http://www.techno-press.org/?journal=cac&subpage=8 ISSN: 1598-8198 (Print), 1598-818X (Online)

terms of ultimate strain, ductile area and elongation, decrease at a much greater rate than those of bare bars (Du *et al.* 2005a, Du *et al.* 2005b).

In recent years, considerable efforts have been made to understand the deterioration mechanisms of reinforced concrete structures (Dagher and Kulendran 1992, Rodriguez *et al.* 1997, Lundgren 2002, Fang *et al.* 2006, Bhargava *et al.* 2006, Shayanfar *et al.* 2007, Hanjari *et al.* 2011). However, during the same period, little attention has been given to the performance assessment of deteriorated reinforced concrete bridge piers.

This study aims to develop an assessment procedure for determining the structural performance of deteriorated reinforced concrete bridge piers. A nonlinear finite element analysis program (RCAHEST, Reinforced Concrete Analysis in Higher Evaluation System Technology), developed by the authors (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007, Kim *et al.* 2009), was modified to develop an analytical evaluation method for the structural performance of deteriorated reinforced concrete bridge piers. Advanced deteriorated material models were developed and programmed in FORTRAN to predict the structural behaviors of deteriorated reinforced concrete. To assess the ability of the RCAHEST program to predict the structural performance of deteriorated reinforced concrete bridge piers, computed results using the program were compared with the experimental and analytical results.

In this study, column strength interaction diagram was also developed to predict the design resistance of deteriorated reinforced concrete bridge piers. It takes into account the location of deteriorated column face, corrosion rate, loss of reinforcement, loss of concrete, and buckling of corroded reinforcement. These material properties are incorporated in the proposed model to develop column strength interaction diagram.

2. Nonlinear finite element analysis program, RCAHEST

A two-dimensional finite element model for the structural performance assessment of deteriorated reinforced concrete bridge piers is developed in this study. The model was created and analyzed using general-purpose finite element software, RCAHEST (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007, Kim *et al.* 2009). RCAHEST is a nonlinear finite element analysis program used for analyzing reinforced concrete structures.

The structural element library, RCAHEST, is built around the finite element analysis program shell named FEAP, developed by Taylor (2000). The elements developed for the nonlinear finite element analyses of reinforced concrete bridge piers are a reinforced concrete plane stress element and an interface element (see Fig. 1).

The nonlinear material model for the reinforced concrete comprises models for concrete and models for the reinforcing bars. Models for concrete may be divided into models for uncracked concrete and for cracked concrete. For cracked concrete, three models describe the behavior of concrete in the direction normal to the crack plane, in the direction of the crack plane, and in the shear direction at the crack plane, respectively (see Fig. 2). The basic and widely-known model adopted for crack representation is based on the non-orthogonal fixed-crack method of the smeared crack concept. The post-yield constitutive law for the reinforcing bar in concrete considers the bond characteristics, and the model is a bilinear model, as shown in Fig. 3. To account for buckling of reinforcing bars, the average stress-strain behavior after concrete crushing is assumed to be linearly descending until the 20% of the average steel stress. This relation was derived from a

2D or 3D Spring element	4 nodes PSC shell element	2D or 3D Flexibility-based fiber beam-column element	4 nodes Elastic shell element
Joint element	FEAP		4 nodes RC shell element
Reinforcing or Prestressing bar element	Interface element	RC plane stress element	2D Elasto-plastic plane stress element

Fig. 1 RCAHEST nonlinear finite element analysis program

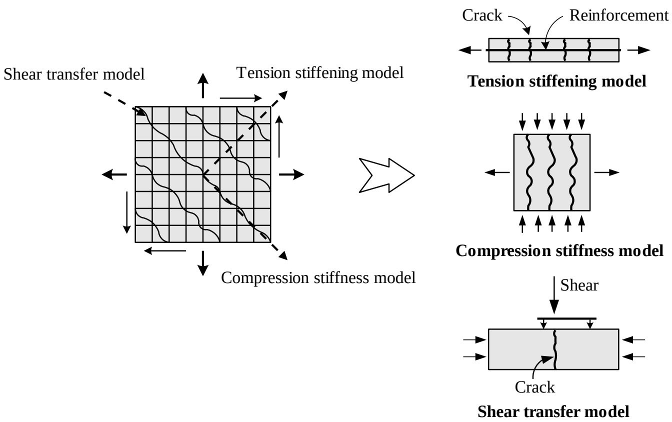


Fig. 2 Construction of cracked concrete model

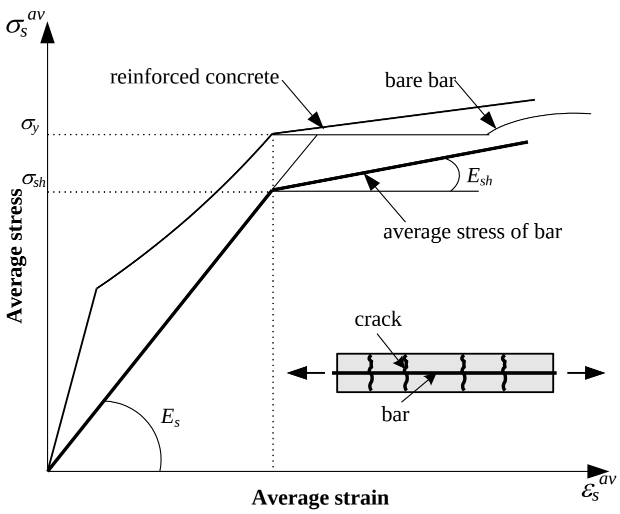


Fig. 3 Model for reinforcing bar in concrete

parametric study using finite element analysis (Kim *et al.* 2005).

The transverse reinforcement confines the compressed concrete in the core region and inhibit the buckling of the longitudinal reinforcing bars. This study basically adopted a model proposed by Mander *et al.* (1988). The models consider the yield strength, the distribution type and the amount of the longitudinal and transverse reinforcing bars to compute the effective lateral confining stress and the ultimate compressive strength and strain of the confined concrete (Kim *et al.* 2003).

Details of the nonlinear material model used have been provided by the authors in previous research (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007, Kim *et al.* 2009).

3. Nonlinear material model for deteriorated reinforced concrete

Reinforced concrete structures may deteriorate due to design and construction faults, chemical attacks or change in existing environment. Regardless of the reason, deterioration of reinforced concrete members limits the service life of the structure (Tapan 2007).

Corrosion of reinforcement is the principal cause of deterioration of reinforced concrete elements. The deteriorating effects of corrosion are evident in the loss of stiffness, loss of strength, and rust stains that the structure will experience. Corrosion of reinforcing bar can greatly influence the bond strength between deformed bars and concrete.

The effect of corrosion is modeled as a change in geometry and properties of corroded reinforcement and surrounding concrete that is, a reduction of steel area and ductility, removal of spalled concrete, modification of concrete response due to corrosion cracks, and modification of bond-slip properties (Hanjari et al. 2011).

Reversed cyclic loading can cause deterioration in the bond between reinforcement and concrete. Several other factors also have effects on bond behavior under cyclic loading. These factors include concrete strength, cover thickness, diameter of the reinforcing steel bar, and the loading rate. Confinement reinforcement was found to effectively reduce bond degradation under cyclic loading (Fang *et al.* 2006).

The following empirical formula developed by Du *et al.* (2005a) to evaluate residual capacity of corroded reinforcing bars was adopted in the analytical model presented in this study.

$$f = (1 - 0.005 \cdot Q_{corr}) \cdot f_y \tag{1}$$

$$A_s = (1 - 0.01 \cdot Q_{corr}) \cdot A_{so} \tag{2}$$

$$Q_{corr} = 0.046 \cdot \frac{I_{corr}}{D_b} \cdot t \tag{3}$$

where f = yield strength of corroded reinforcement; f_y = yield strength of non-corroded reinforcement; Q_{corr} = amount of corrosion of reinforcement (%); A_s = average cross-sectional area of corroded reinforcement; A_{so} = initial cross-sectional area of non-corroded reinforcement; I_{corr} = corrosion rate of reinforcement in real structure ($\mu A / cm$); D_b = diameter of non-corroded reinforcement; and t = time elapsed since the initiation of corrosion (years).

Non-uniform reinforcement corrosion along the height and cross-section of a bridge piers leads to partial or complete loss of bond between corroded steel bars and the surrounding concrete. Bond strength between reinforcement and concrete decreases with reduction in confinement by transverse reinforcement. In seismic regions, the effect of confinement reinforcement in reducing bond deterioration under cyclic loading enhances the energy absorption and dissipation capabilities.

Generally, the cracked concrete may resist a certain amount of tensile stress normal to the cracked plane by the bond effect between the concrete and the reinforcing bars. A refined tension stiffness model is obtained by transforming the tensile stresses of concrete into the component normal to the crack and improved accuracy is expected, especially when the reinforcing ratios in orthogonal directions are significantly different and when the reinforcing bars are distributed only in one direction.

The tensile stresses in the directions of reinforcing bars may be obtained using the following relations:

$$\frac{\sigma_{xt}}{f_t} = \left(\frac{\varepsilon_{cr}}{\varepsilon_{xt}}\right)^c \tag{4}$$

$$\frac{\sigma_{yt}}{f_t} = \left(\frac{\varepsilon_{cr}}{\varepsilon_{yt}}\right)^c \tag{5}$$

where f_t = tensile strength of concrete normal to the crack plane; ε_{cr} = cracking strain of concrete; and ε_{xt} , ε_{yt} = tensile strains of concrete in the axial directions of x, y reinforcing bars. The bond parameter c in Eqns. (4) and (5) reflect the influence due to the difference in bond characteristics where the values of 0.2 for welded wire, 0.4 for deformed bar, 0.6 for undeformed bar, and 2.0 for plain concrete are basically recommended. The bond parameter is also determined considering aging degradation.

4. Strength interaction diagram for deteriorated column

Strength interaction diagrams presented here have been developed based on a design procedure that accommodates the KHBD (2005) code provisions for reinforced concrete compression members. Section strength is accurately calculated by a strain compatibility procedure and an analytical representation of the nonlinear stress-strain relations of reinforcing steels.

Most reinforced concrete columns are symmetrically reinforced about the axis of bending. However, for deteriorated reinforced columns, nonuniform cross-section loss of steel bars leads to large eccentricity and moment, and the concrete column becomes unsymmetrical.

For a deteriorated column, once the cover spalls off, compressed reinforcements are likely to buckle. In other words, if the exposed corroded bar length, exceeds a critical length; the reinforcement will buckle before reaching its yield capacity and load carrying capacity of the deteriorated column will be reduced (Tapan 2007).

Damaged geometry and material properties of the concrete column section were included in the proposed method. Material behavior of deteriorated concrete and reinforcement were investigated, and the stress-strain properties of the deteriorated concrete and reinforcement were basically used (see Fig. 4). An analytical model was developed to determine the strength interaction diagram of deteriorated columns. A flowchart outlining the basic operation of the developed strength interaction diagram is presented in Fig. 5.

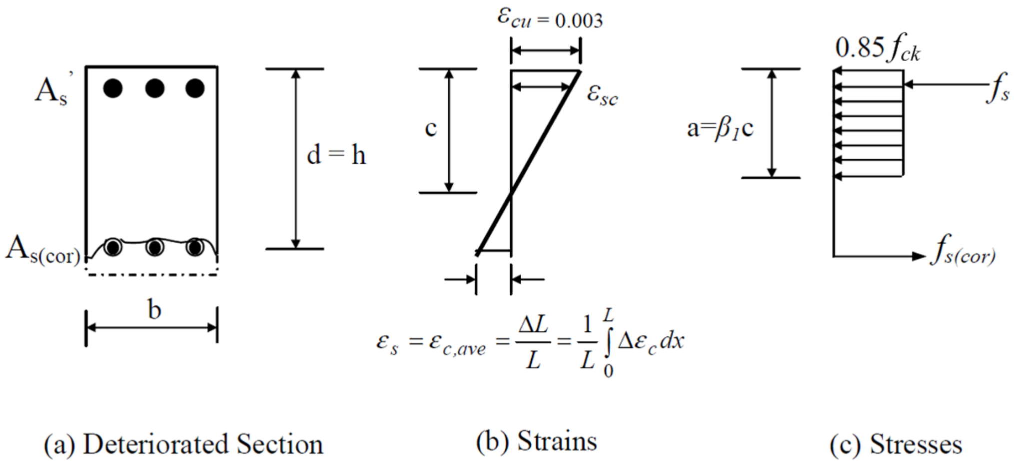


Fig. 4 Stresses and Strains for a corroded reinforced concrete column section (Tapan 2007)

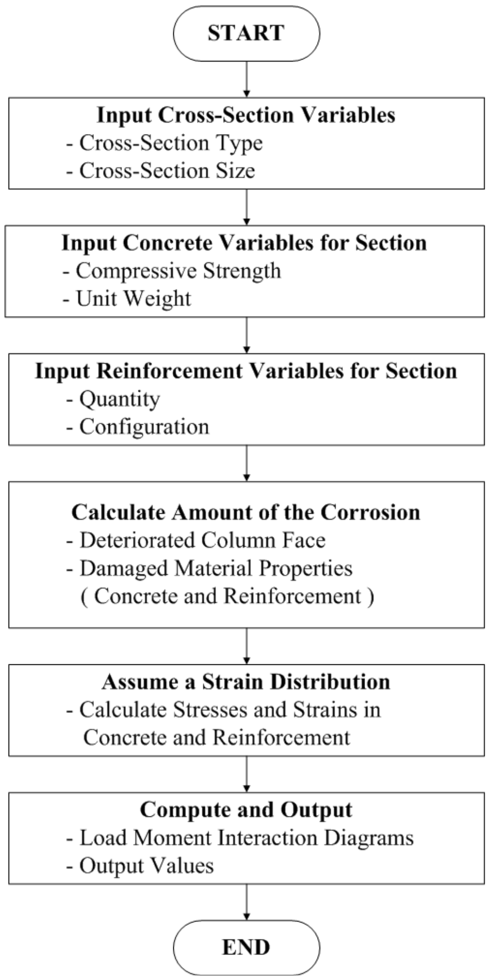


Fig. 5 Flowchart of column strength interaction diagram

5. Verification in structural level

5.1 Deteriorated reinforced concrete beams with fracture stirrups

For evaluating the effect of broken stirrups in reinforced concrete beams on their shear capacity and ductility, an experimental program of reinforced concrete beams with shear reinforcement, whose anchorage and bond were removed, was conducted (Toongoenthong and Maekawa 2005a, Toongoenthong and Maekawa 2005b). Fig. 6 shows the specimen details and reinforcement arrangement.

The tested compressive strength of the concrete after standard curing of 28 days was 35 MPa. The high yield strength of both the tension and compression reinforcement was 645 MPa for producing a comparatively large flexural capacity to ensure that shear failure would occur. The yield strength of the shear reinforcement was 345 MPa. The concrete cover of the stirrup was 22 mm. The loading was applied with combined flexure-shear forces.

The differences in surface condition at the legs of stirrups was additionally addressed for each side of the beam by wrapping the extreme ends of stirrup-legs with a 5 cm long strip of vinyl tape (see Fig. 6). The aim of the vinyl tape wrapping was to create smooth surfaces for eliminating bond. This setup was designed as an analogy of corroded broken web reinforcement or ruptured stirrups. Premature anchorage of web steel is thought to cause degradation of reinforcement capability and the loss of bond close to the tip of cut-off steel may occur as a result.

Fig. 7 shows the finite element discretization and the boundary conditions for specimen, and the finite element model consists of eight-noded plane stress elements. The load was proportionally increased until failure occurred.

Fig. 8 shows the load-displacement relation at the mid-span of the non-damaged beam. The experimental shear capacity at the peak was approximately 380 kN that the computed shear capacity corresponds to the yield limit state of web steel. The proposed analytical model successfully predicts the load-displacement relationship of non-damaged beam.

The experimental and analytical crack pattern after failure is also shown in Fig. 9. In reality, diagonal shear cracking was initiated in the web. The final failure stage was then reached with the formation of an unstable shear failure path between the loading point and the beam supports.

Fig. 10 shows the experimental and analytical load-displacement relation of the damaged beam exhibiting shear failure in the shear span containing the broken web reinforcement. A significant reduction in shear capacity can be clearly seen. The damaged beam containing cut-off shear reinforcement reached the experimental ultimate capacity of 240 kN, which is a decrease in shear of approximately 37% compared with the non-damaged reference beam. The value given by specimen was similar to the analytical results.

Starting with the reference case of no-damage, we have the analytical results, which match fairly well both the experimental capacity and the cracking pattern. Distributed diagonal shear cracks can be simulated well owing to the existence of web reinforcement as shown in Fig. 11.

Nonlinear finite element analysis was confirmed as an efficient tool for computational performance assessment of damaged beams with broken stirrups of no anchorage, by considering the bond deterioration zone of ten times the diameter of the stirrup steel bar. The numerical results show fair agreement with the experimental facts both in terms of load-deflection responses and failure crack patterns.

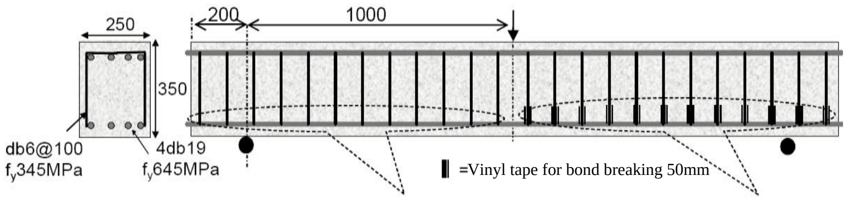
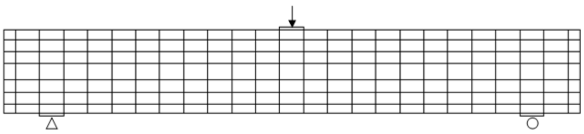


Fig. 6 Details of specimen (Unit: mm) (Toongoenthong and Maekawa 2005b)



8-node RC element	**175**
8-node Elastic element	**3**

Fig. 7 Finite element mesh for specimen

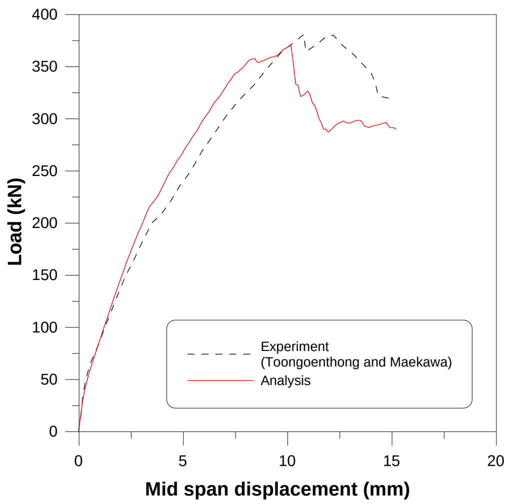
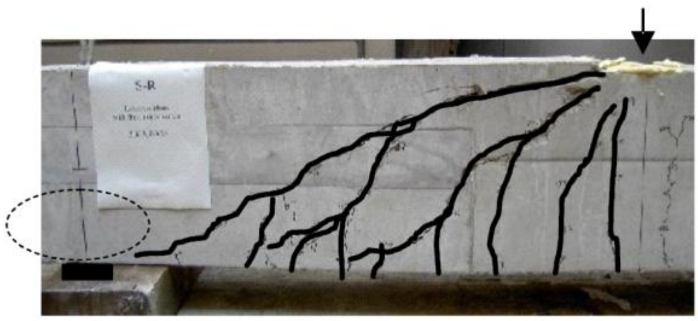
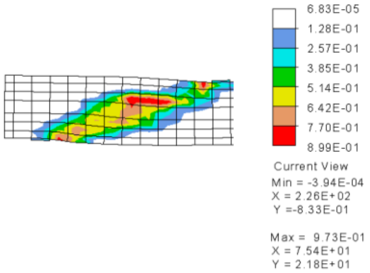


Fig. 8 Load-versus-displacement relationship for reference beam



(a)

Fig. 9 Failure pattern for reference beam: (a) experiment (Toongoenthong and Maekawa 2005b); and (b) analysis



(b)

Fig. 9 Conitnued

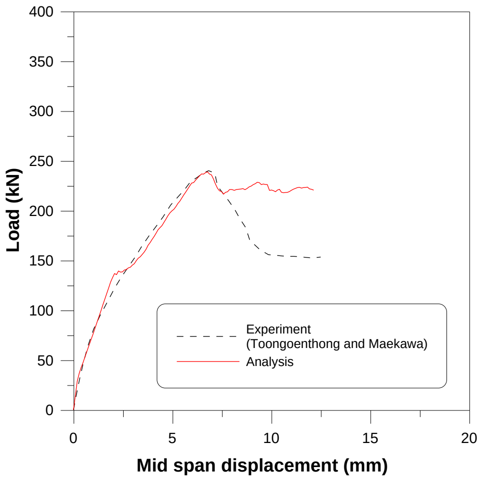
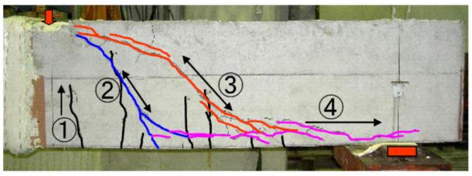
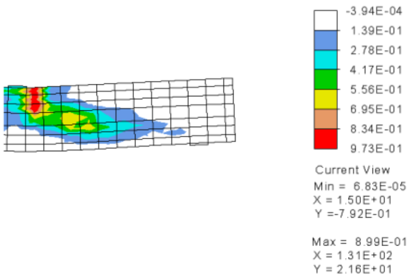


Fig. 10 Load-versus-displacement relationship for damaged beam



(a)



(b)

Fig. 11 Failure pattern for damaged beam: (a) experiment (Toongoenthong and Maekawa 2005b); and (b) analysis

5.2 Load carrying capacity of deteriorated reinforced concrete columns

Available experimental and analytical data for the effects of corrosion on short columns subject to axial loads combined with moments used to verify the accuracy of proposed model.

Fig. 12 shows column that was tested as well as the loading test arrangements (Rodriguez *et al.* 1997). Type 2 columns had four 16 mm main bars with 6 mm stirrups at 150 mm centers. 3% sodium chloride (by weight of cement) was added to the mixing water, and a current of 100 $\mu A/cm^2$ was applied to all of the reinforcement in the 1200 mm central test section for approximately 100-200 days to obtain the required level of corrosion. Those cases are illustrated in Fig. 13 below with brief information about the deterioration level. The measured corrosion rates offer the possibility of being able to calculate the reduction in reinforcement diameter directly.

Fig. 14 shows both the strength interaction diagram developed by Tapan (2007) and the one developed by proposed model for Case - 1 where it was assumed that the left side of the concrete cover was deteriorated. Fig. 15 and Fig. 16 show the strength interaction diagrams computed for Case – 2 and Case - 3 using proposed model and the interaction diagram that was developed by Tapan (2007). For comparison purposes the results of the experimental tests are also plotted on same diagrams. The column strength interaction diagrams developed using proposed models reasonably agree with the ones developed by Tapan (2007). However, in compression field the developed model shows higher strength. That is probably because of different models used for computing buckling stresses, residual strength of corroded reinforcement and strains at corroded regions.

The results show that the proposed model is progressive and is capable of predicting the load carrying capacity of deteriorated reinforced concrete columns with reasonable accuracy.

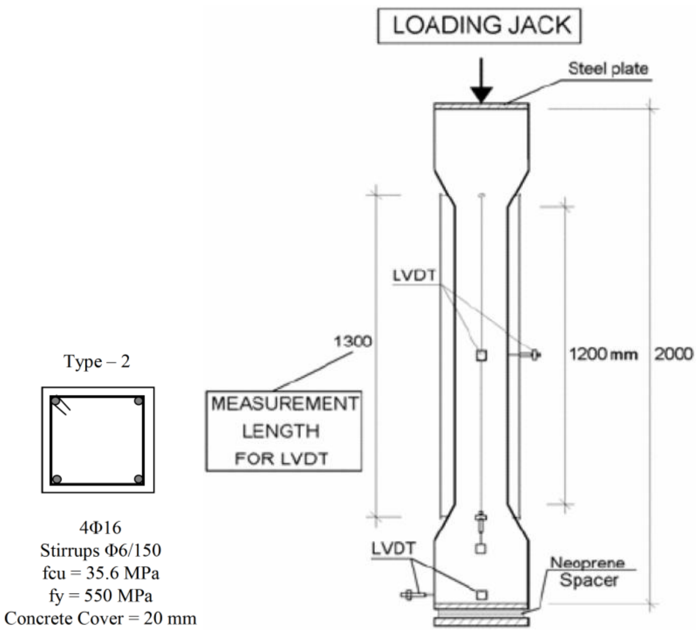


Fig. 12 Tested columns and loading test arrangement (Rodriguez *et al*. 2007)

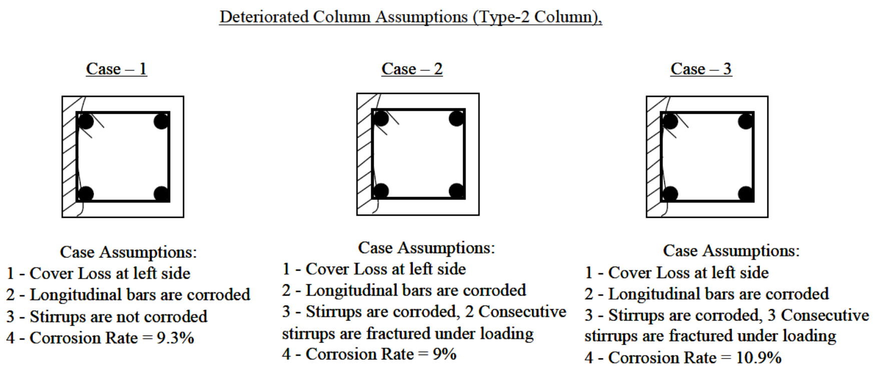


Fig. 13 Deteriorated columns (Rodriguez *et al*. 2007)

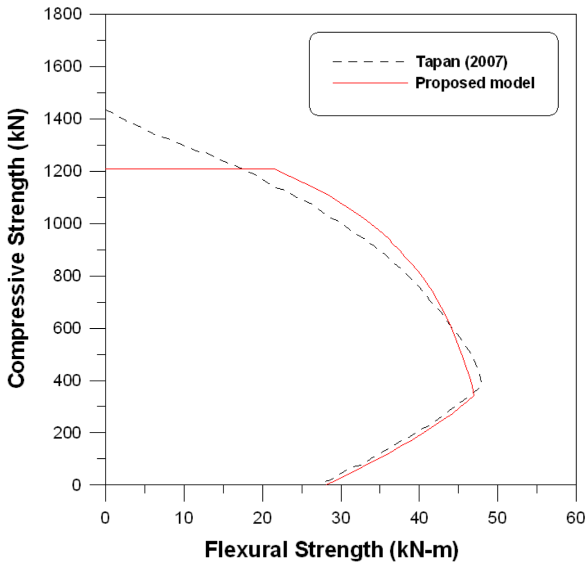


Fig. 14 Column strength interaction diagram for specimen Type-2 (Case-1)

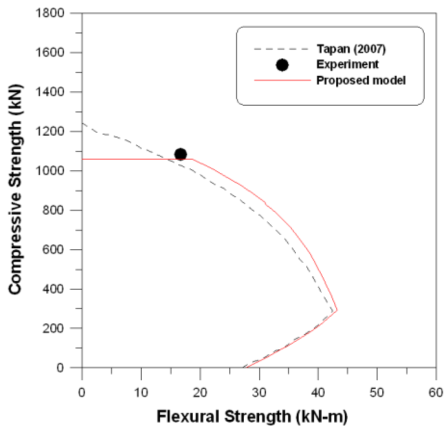


Fig. 15 Column strength interaction diagram for specimen Type-2 (Case-2)

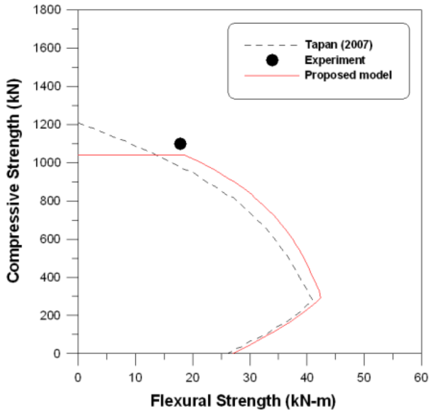


Fig. 16 Column strength interaction diagram for specimen Type-2 (Case-3)

6. Application to deteriorated reinforced concrete bridge piers

In this application example, the effects of reinforcing details on the progression of corrosion damage in reinforced concrete bridge piers are studied.

The applied bridge columns had circular cross sections and were reinforced with well-distributed longitudinal reinforcement (Fig. 17). The tested compressive strength of the concrete was 22.4 MPa. The yield strength of the longitudinal reinforcement and transverse reinforcement were 351.4 MPa and 392.3 MPa, respectively. It is considered appropriate to use the current code provisions (KHBD 2005) on the concrete confinement for the potential plastic hinge regions (Kim *et al.* 2007).

Fig. 18 shows a drawing of the bridge column with an aspect ratio of 4 in the test apparatus. The 2000 kN MTS hydraulic actuator attached to the top of the bridge column applied the lateral displacement history. Specimens were subjected to two cycles each at lateral displacement amplitudes of 0.25, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0 and 7.0 percent until failure. The column specimen was tested under a $0.10 f_c^{'} A_g$ constant compressive axial load to simulate the gravity load from bridge superstructures.

A three-dimensional finite element analysis is tedious and expensive, and requires a high number of elements to achieve a good accuracy. Therefore, two-dimensional eight-noded smeared elements are used to model the experimental specimens. Fig. 19 shows the finite element discretization and the boundary conditions for 2 dimensional plane stress nonlinear analyses of the reinforced concrete bridge piers. The interface element between the footing and the column enhances the effects of the bond-slip of steel bars and the local compression. The interface model for the boundary plane connecting two reinforced concrete elements with different sections is based on the discrete crack concept, which uses the relationships between the stress and the localized deformations. Fig. 20 shows the method for transforming a circular section into rectangular strips to using the plane stress elements of RCAHEST. For rectangular sections, equivalent strips are calculated. After the internal forces are calculated, the equilibrium is checked.

The lateral load-displacement response for specimen is shown in Fig. 21. The analytical results show reasonable correspondence with the experimental results. The column strength interaction diagram for specimen is shown in Fig. 22. For the normal strength columns, the analyses provide conservative column strengths compared with the test results.

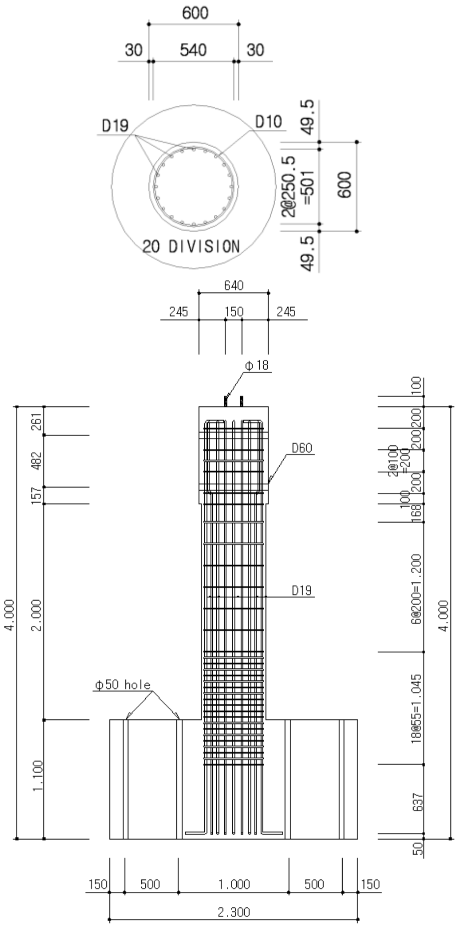


Fig. 17 Bridge column geometry and reinforcement (Unit: mm)

Fig. 18 Experimental setup

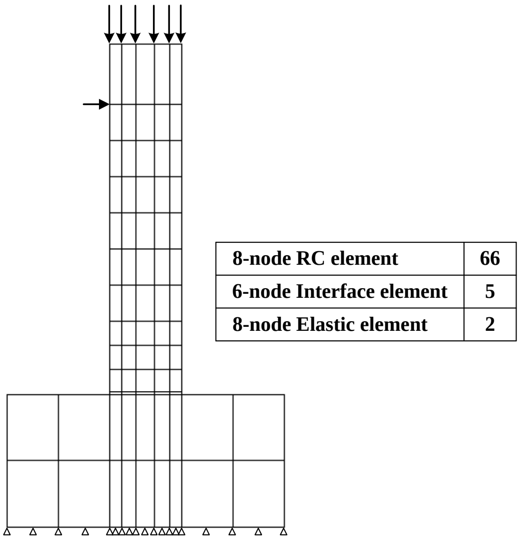


Fig. 19 Finite element mesh for analysis

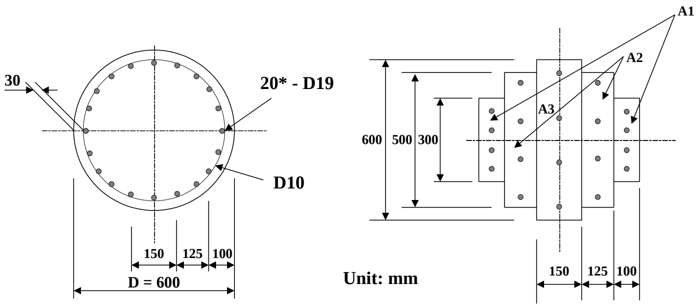


Fig. 20 Transformation of a circular column to an idealized equivalent rectangular column

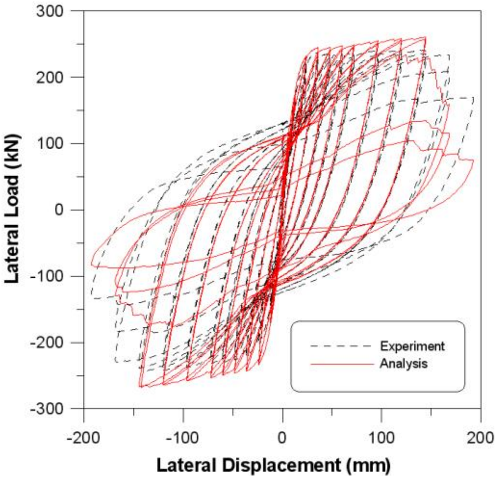


Fig. 21 Lateral load-displacement relationship for specimen

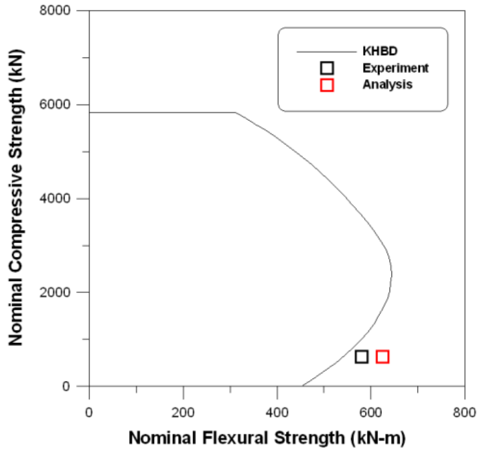


Fig. 22 Column strength interaction diagram for specimen

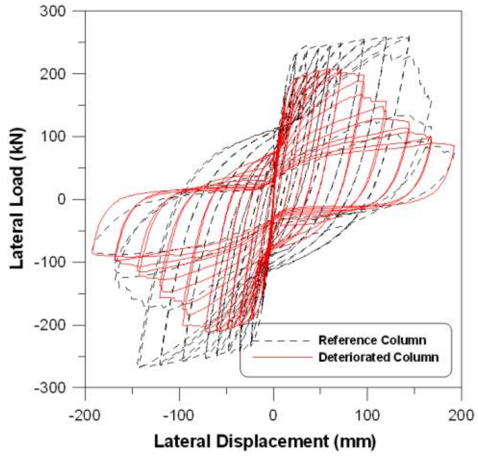


Fig. 23 Lateral load-displacement relationship

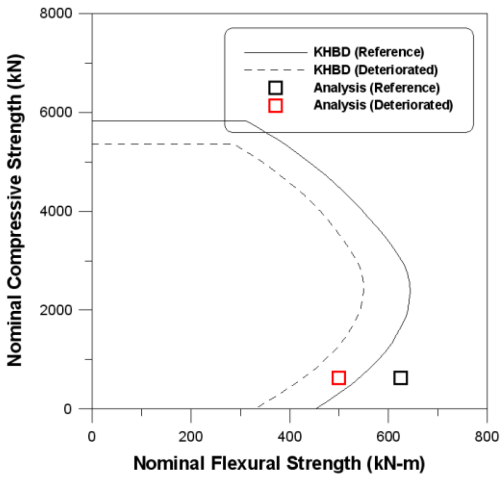


Fig. 24 Column strength interaction diagram

It is assumed that deterioration stage is completed as reduced reinforcement ratio reaches 0.15 for longitudinal reinforcement and 0.25 for transverse reinforcement, respectively (Naus *et al.* 1996). The bond parameter considering aging degradation is 0.7 determined by linear interpolation. Here, reduced reinforcement ratio is defined according to the lost sectional area, and bond deterioration is represented by reduced tension-stiffness equivalent to plain concrete in the analysis.

An analytical comparison between the load-displacement values for the deteriorated and reference specimen is shown in Fig. 23. Reinforcement corrosion has significant effects on load carrying capacity of the reinforced concrete bridge piers (see Fig. 24). The reduction of area of steel in a member will in turn decrease the strength of the member. The loss of bond strength between the concrete and steel may further contribute to the loss of strength of the member.

The comparison of analytical results and interaction diagrams yields that the proposed model seems to accurately represent the behavior of deteriorated reinforced concrete bridge piers and for the limited available experimental data the proposed model is considered conservative. It is expected that, by using the proposed method, the structural performance of deteriorated reinforced concrete bridge piers can be predicted accurately, and this enables more rational and reliable design of bridge piers. However, further comparisons should be done with the availability of new experimental data.

7. Conclusions

This paper attempted to establish a framework for assessment of the structural performance of deteriorated reinforced concrete bridge piers. An experimental and analytical study was conducted to quantify performance measures and examine one aspect of detailing for deteriorated reinforced concrete bridge piers. The RCAHEST can be used for examining the structural performance of these structures that have been previously investigated. From the results of the experimental and analytical studies, the following conclusions were reached.

1. The proposed analytical method along with results of detailed investigation of deteriorated reinforced concrete bridge piers will improve the understanding of effects of deterioration on structural members. The numerical model also provides a tool that may be used to develop a better understanding of the mechanisms of damage propagation due to corrosion of the reinforcement, delamination, and spalling of reinforced concrete structures.
2. Nonlinear finite element analysis may be used to investigate the load-deflection response of deteriorated reinforced concrete structures. It is observed that for the limited available experimental data the proposed model is conservative and is capable of predicting the load carrying capacity of deteriorated reinforced concrete columns with reasonable accuracy. Also, failure modes and ductility may be checked for seismic resistant design.
3. The proposed model gives a good quantitative estimate of the strength of corroded concrete column, which will help in establishing a cost-effective repair system for deteriorated reinforced concrete bridge piers.

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