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INVESTIGACIONES SOBRE SISMOLOGIA E INGENIERIA SISMICA EN EL JAPON

APPLICATION OF FEM (FINITE ELEMENT METHOD) TO RC (REINFORCED CONCRETE) STRUCTURES

Hiroshi Noguchi

...VENCION DE DESASTRES

SISTEMA NACIONAL DE PROTECCION CIVIL

CENTRO NACIONAL DE PREVENCION DE DESASTRES

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Investigaciones sobre Sismología e Ingeniería Sísmica en el Japón

P R E S E N T A C I O N

Una de las aportaciones más importantes de la parte japonesa dentro del Convenio de Cooperación Técnica entre la Agencia de Cooperación Internacional del Japón (JICA) y el Centro Nacional de Prevención de Desastres es el envío de expertos de corto plazo. Dentro de las actividades de estos expertos están la asesoría en aspectos específicos de proyectos de investigación, la impartición de seminarios y conferencias, y la participación en eventos técnicos.

En las numerosas conferencias dictadas desde que se inició el Convenio en 1990, se han presentado resultados y experiencias en líneas actuales de investigación y desarrollo tecnológico en materia de prevención de desastres sísmicos en el Japón.

Con objeto de dejar un testimonio permanente del contenido de las conferencias más sobresalientes y de mayor interés para la comunidad ingenieril y científica, el CENAPRED ha emprendido la publicación de esta serie como parte de los Cuadernos de Investigación.

CONTENIDO

PROLOGO

El desarrollo reciente y acelerado de las computadoras digitales y de criterios de comportamiento de materiales, basados en resultados de ensayes de laboratorio, ha conducido a la realización de una gran cantidad de estudios analíticos. Entre ellos destacan los que han aplicado el Método del Elemento Finito (MEF).

En esta publicación, H. Noguchi discute las aplicaciones del MEF para analizar estructuras de concreto reforzado. En la primera parte, Noguchi introduce al lector, de manera formal aunque relativamente simple, en los conceptos básicos de análisis de este tipo de estructuras. La descripción secuencial de los criterios de modelado pone en evidencia el refinamiento experimentado y el consecuente mejoramiento en la precisión de los cálculos cuando éstos se comparan con resultados experimentales. La interacción de los mecanismos de resistencia a cortante (transferencia de cortante a través de grietas, acción de dovela, adherencia y anclaje) en elementos de concreto reforzado es discutida e ilustrada mediante resultados de análisis. El empleo del MEF para estudiar el comportamiento de elementos de concreto, para los cuales los efectos del cortante son importantes, pone de manifiesto el potencial del método para extender los resultados experimentales. Con él se puede evaluar el efecto de las variables más significativas de un fenómeno, así como cuantificar la interacción de los diferentes mecanismos resistentes. Lo anterior permite reducir el número de ensayes de laboratorio lo que evidentemente conduce a proyectos más baratos y eficientes. Un ejemplo de ello es el programa de análisis de estructuras de concreto reforzado con materiales de alta resistencia. En este proyecto, de la comparación de los resultados experimentales y analíticos, se verificaron e incluso se modificaron algunas conclusiones obtenidas de los ensayos. Noguchi explica sucintamente, aunque con éxito, los parámetros que afectan significativamente la precisión de los resultados obtenidos del análisis.

Como se señala en el texto, el MEF no sólo es una herramienta poderosa para académicos; también es útil para ingenieros estructuristas. Un uso correcto del MEF facilita la visualización del flujo de fuerzas en las estructuras que es dato fundamental para el diseño. Sin embargo, los análisis (aun elásticos) deben realizarse y evaluarse con plena conciencia. Los ingenieros estructuristas que usan (e inclusive "abusan") este método no deben perder de vista que un análisis correcto no es sustituto de un diseño y detallado adecuados.

Sin duda alguna que, conforme se avance en la modelación de estructuras (en particular en el intervalo inelástico), se extenderá la aplicación del MEF. En estas circunstancias, la revisión de la literatura y la discusión de Noguchi serán de gran utilidad.

CONTENTS

1. Analytical Techniques of Shear in Reinforced Concrete Structures by Finite Element Method

1.1 Introduction

It was fifteen years back when the reinforced concrete (RC) beams were first analyzed with the finite element method (FEM) by Ngo and Scordelis (1967). For these fifteen years, the objects of analysis have been made wider in the scale from members: beams, columns, beam-column joints, shear walls, footings and floor slabs, to overall frames including shear walls, box-shaped or cylindrical shear walls, bridges, shell structures, prestressed concrete atomic reactor vessels (PCRV) and atomic reactor buildings. For the loading conditions, the time dependent problems under the long-term loading like creep, shrinkage and temperature stress have been treated besides the problems under the short-time loading as the static analysis, while on the other hand the problems under the earthquake have been treated as the dynamic analysis.

The state-of-the-art reports on how the analytical studies of RC structures by FEM had developed were written by Scordelis (1972), Kawamata (1974), Schnobrich (1976), Noguchi (1976), Wegner (1976), Scordelis (1978), Aoyama and Noguchi (1979), (1979), Gerstle (1981), Meyer (1981), Argyris (1981), Gergely (1981), Eibl (1981) and Okamura (1981).

As the finite element idealization and the creation of the boundary condition are fairly free in FEM, the modelling of the shear behavior: the propagation of concrete cracking, the bond between reinforcing bars and concrete, the shear transfer across cracks and the dowel action of reinforcing bars are sufficiently possible. Therefore, FEM has developed into one of the most powerful instruments in the analysis of the strength and deformation of the shear dominant RC structures, using the appropriate constitutive laws and the failure criterion of concrete and reinforcing bars under multi-axial stresses. In this report, the previous studies on shear analysis of RC structures by FEM, were surveyed and the previous analytical models on the material behavior were introduced. The subjects of future investigation were also pointed out.

1.2 Characteristics of Shear Analysis of RC Structures by Finite Element Method

The previous studies on the shear problems of beams, the most basic members, were introduced and the characteristics of shear analysis of RC members by FEM were surveyed.

In Ngo and Scordelis' study (1967), FEM was first applied to the analytical method in RC structures to represent the bond behavior by a bond linkage element which consisted of two orthogonal springs, assuming the bond stress as the function of the bond slip. In this study, the simple beams with the pre-set inclined and flexural cracks between elements, as shown in Fig. 1, were analyzed.

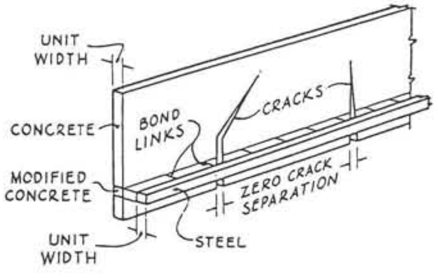


Fig. 1.1 Analytical Model for Simple Beam (Ngo, 1970)

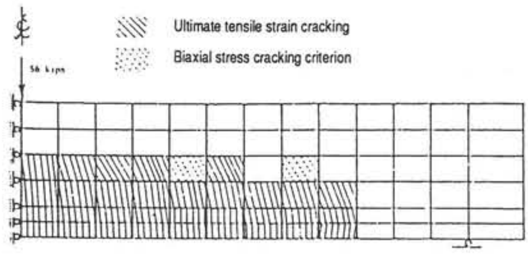


Fig. 1.2 Analytical Crack Pattern for Simple Beam (Franklin, 1970)

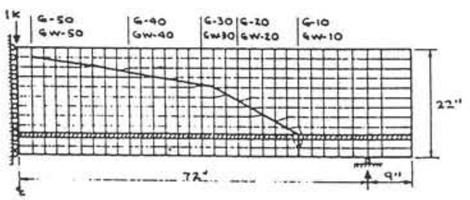


Fig. 1.3 Finite Element Idealization for Analysis of Simple Beam (Ngo, 1970)

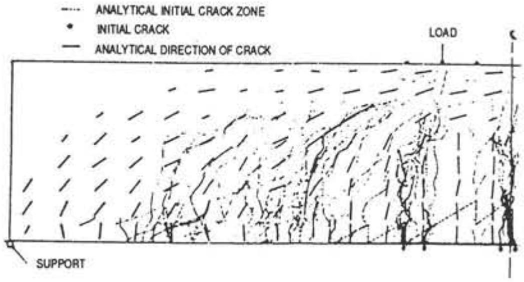


Fig. 1.4 Comparison between Analytical Crack Pattern and Experimental Result (Isobata, 1971)

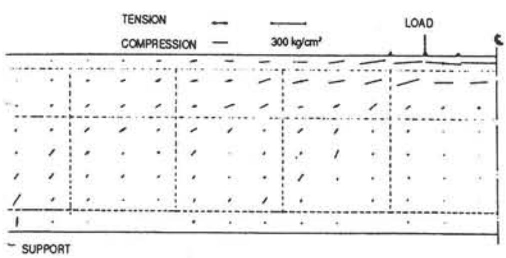


Fig. 1.5 Principal Stress Distribution in Simple Beam (Isobata, 1971)

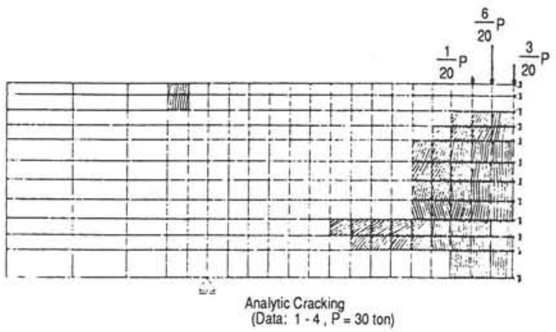


Fig. 1.6 Analytical Crack Pattern for Simple Beam without Bond (Kokusho and Takiguchi, 1971)

Franklin (1970) proposed an automatic propagation model of concrete cracking. In this model, cracks were assumed to occur perpendicularly to the maximum principal stress direction. When a crack occurred, the stiffness in the perpendicular direction to the crack was set to zero, and cracking release loads were subjected to the overall structures in order to consider the redistribution of stresses. The analytical crack pattern of a simple beam was shown in Fig. 1.2. From this figure, it is known that cracks occur in the some overall area, and the crack spacing can not be represented. The stiffness estimation of a cracked element is also difficult.

Ngo, Franklin and Scordelis (1970) reported the elastic analysis of simple beams with pre-cracks, as shown in Fig. 1.3. The bond was represented by the bond linkage element, and the effect of stirrups, the dowel action of tensile longitudinal bars crossing the inclined crack, the aggregate interlock on the crack surfaces and the splitting crack along the longitudinal bar near the support were considered in this analysis. The analytical results were compared with the Kani's theory (1966) and the shear resistance mechanisms were investigated in detail. The results were as follows. The aggregate interlock increased the dowel shear force, and this was contradictory to the general opinion. There were two areas: near the loading point, and the supporting point, where the effect of stirrups was not observed. This was corresponding to the Kani's theory. Stirrups had the function to restraint the maximum principal stress near the crack tip or in the dowel area. In this study, the phenomena, which could not be grasped with the experimental approach, were ascertained with the analytical approach.

Isobata (1971) analyzed simple beams with the shear span ratio, a/d = 2.7, by the elastic plastic theory in which concrete was regarded as the orthotropic material. A comparison of the crack pattern between the analytical result and the experimental one was shown in Fig. 1.4. The analytical crack pattern indicated that cracks occurred on the overall tension area and the crack position was indefinite. But as the experimental crack position could not be recognized as a necessary consequence and the analytical crack direction gave a good agreement with the experimental one, it was pointed out that the crack effect was represented sufficiently by the smeared crack model. It was significant that near stirrups, the balance of forces as the truss analogy took the shape of the increase of stirrup tensile forces and the concentration of concrete compressive stresses along the crack, as shown in Fig. 1.5.

Kokusho and Takiguchi (1971) applied the plasticity theory based on the Drucker-Prager criterion to the concrete model. In their model, cracking was represented with the smeared crack model, and the bond slip was represented with the bond link. The analytical results of simple beams were compared in detail with the experimental results on the deflections, strains in reinforcing bars and concrete. The failure of beams was not discussed, but the analytical results of the RC beams with a small shear span ratio and a high reinforcement ratio, the analysis of which has been difficult, gave a good agreement with the experimental results. The effects of the bond characteristics on the behavior of beams were analytically discussed. It was pointed out that the shear cracking load increased and the opening of the flexural crack was fairly remarkable, as shown in Fig. 1.6, in case of the poor bond.

Valliappan (1971) applied the plasticity theory based on the von Mises criterion to the models of concrete and reinforcing bars and analyzed simple beams. Muto and Miyashita (1972) analyzed a cantilever beam subjected to the reversed cyclic loading by the plasticity model based on the Drucker-Prager criterion. Though the analytical results were stiffer than the experimental results, it was the first analytical study on the reversed cyclic loading in Japan.

Fukushima (1972) and Lassker (1973) analyzed simple beams, assuming concrete to be an orthotropic material and using the bond link model for the bond behavior. In the Lassker's study, the propagation of the shear cracks was discussed with a smeared crack model.

Suidan (1973) conducted the three-dimensional analysis of a simple beam, as shown in Fig. 1.7, assuming the perfect bond and the constant shear stiffness of cracked elements. The analytical results gave a good agreement with the experimental results to the range of the tensile yielding of longitudinal bars, but the shear span ratio a/d of the beam was 4.5, and the flexural behavior was dominant.

Ono and Adachi (1973, 74) developed the Ngo's model (1970) and analyzed simple beams (a/d = 1.9). In their model, the plasticity theory based on the Drucker-Prager criterion was applied to the concrete model, and the aggregate interlock on the crack surface was represented by the spring stiffness degradation of the crack linkage element. The analytical principal stress distribution was shown in Fig. 1.8. It was pointed out that the effects of the aggregate interlock on the overall stiffness of the member and the progress of failure were not so distinguished.

Salem (1974) applied the plasticity theory based on the octahedral shear stress criterion to the concrete model, and analyzed slender beams and deep beams without stirrups. When the perfect bond was assumed for the longitudinal bars which were modelled as linear or two-dimensional elements, the stiffness of the analytical result was higher than the experimental result. When the bond slip was represented by a tie-link (bond link) between a two-dimensional bar element and concrete, bond yielding occurred in the neighborhood of a support near the maximum strength, and the stiffness was degraded in the better agreement with the experimental result. At the initial stage, the beam action was dominant, but the arch action became distinguished, as shown in Fig. 1.9, with the propagation of cracks.

Cedolin (1977) pointed out that it was important to evaluate properly the shear stiffness of cracked elements in the analysis of the shear dominant member, and analyzed simple beams without stirrups, the specimen OA1 without stirrups, and A1 with stirrups (a/d = 4.0), with the parameter of the shear stiffness, G. The analytical load-deflection curves were compared with the experimental results, as shown in Fig. 1.10. When the value, G was constant, the analytical load-deflection curve gave a good agreement with the experimental one, but the analytical ultimate strength was a little high. When the value, G was decreased as a function of the strain, ε_1, normal to the crack, the analytical ultimate strength was in good accord for the specimen OA1.

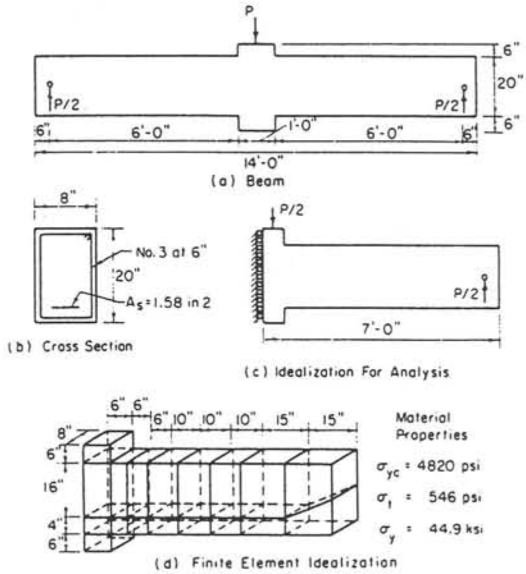


Fig. 1.7 Dimensions of Simple Beam Specimen and Finite Element Idealization (Suidan, 1973)

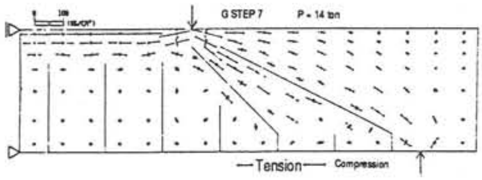


Fig. 1.8 Principal Stress Distribution in Simple Beam (Sato and Ono, 1974)

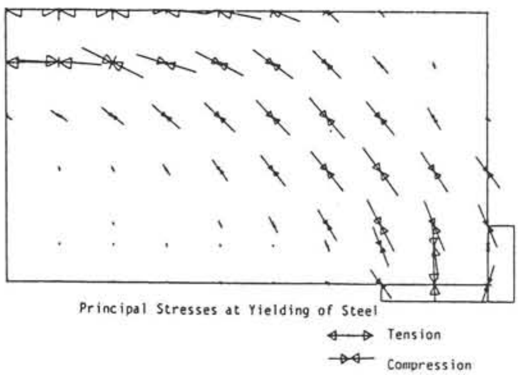


Fig. 1.9 Direction and Relative Magnitudes of Principal Stresses in Deep Beam (Salem, 1974)

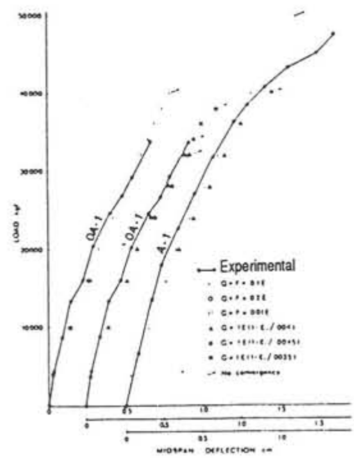


Fig. 1.10 Load-Deflection Curves (Cedolin, 1977)

Ohta and Nakazawa (1978) analyzed a simple beam, a/d = 1.35, by FEM, in comparison with their proposed beam theory model. Their analytical model was similar to the Kokusho-Takiguchi model (1971). The analytical load-deflection curve was stiffer than the experimental one.

Ikeda and Uji (1980) conducted the elastic finite element analysis of simplified model specimens without stirrups, as shown in Fig. 1.11, in order to study the relations between the shear strength of simple beams and the bond characteristics. When the failure patterns were classified into the four groups: diagonal tension failure, shear compression failure, splitting failure and flexural failure, it was pointed out that the failure pattern and the shear strength could be estimated very well from the comparisons between the analytical and experimental results.

Osanai, and Kokusho (1980) analyzed the cantilever beam specimens, finite element idealization of which was shown in Fig. 1.12, using the bond-link and the crack-link. It was pointed out that the strain distribution of longitudinal bars, the load-deflection relations and the initiation and propagation of flexural cracks could be simulated, but the propagation of inclined cracks was a subject for the further study.

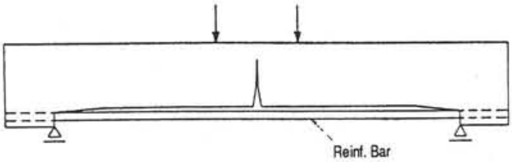
Saouma (1981) developed the discrete crack model and proposed a method to change the finite element meshes finely according to the propagation of cracks. The first and the final crack patterns were shown in Fig. 1.13.

Blaauwendraad (1981) indicated that RC structures sometimes displayed a failure behavior which was dominated by one or a few discrete sharp cracks. Their proposed model was able to predict such failure types, considering that discrete cracks were allowed to develop in any direction, and to cross any element. The analytical crack pattern of a simple beam was shown in Fig. 1.14, as compared with the experimental result.

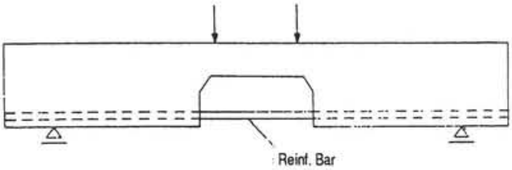
Plauk (1981) studied the behavior of simple beams by the mechanical model, into which the different bond properties near cracks and between cracks were incorporated. It was pointed out that the crack spacing could be predicted very well, as shown in Fig. 1.15, by even a smeared crack model.

Beukel (1981) analyzed two beams which had different types of shear failure by almost the same model as Blaauwendraad's. The ultimate crack patterns were shown in Fig. 1.16. The new continuous inclined crack which came into being in the very last second of the experiment could not be simulated, but the two different patterns of cracking could be well simulated. The agreement between the load-deflection curves of the experiment and analysis was also satisfactory to the shear failure.

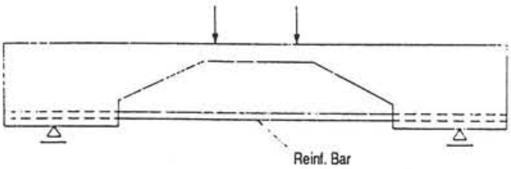
Niwa and Okamura (1981), Maekawa (1981) proposed the concrete model, in which the strain softening tendency after the maximum compressive strength and the orthotropic material after cracking were considered by the total strain theory. The finite element idealization of a deep beam



(a) Analytical Model for Unbonded Specimen after Flexural Cracking



(b) Analytical Model for Bonded Specimen after Flexural Cracking



(c) Analytical Model for Bonded Specimen after Diagonal Cracking

Fig. 1.11 Analytical Models for Bond Effect on Shear Behavior (Ikeda and Uji, 1980)

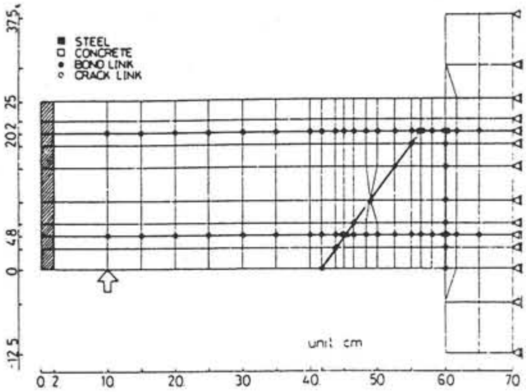
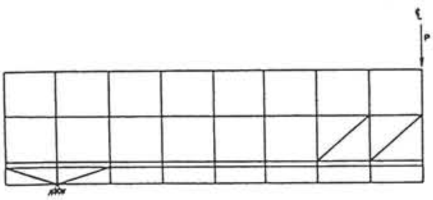
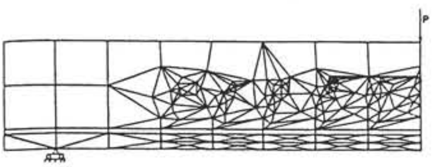


Fig. 1.12 Finite Element Idealization for Analysis of Simple Beam (Osanai and Kokusho, 1980)



(a) Original Analytical Model



(b) Final Mesh

Fig. 1.13 Analytical Crack Patterns (Saouma, 1981)

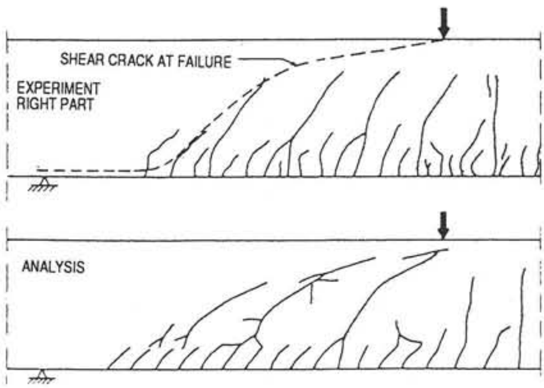


Fig. 1.14 Crack Patterns (Blaauwendraad, 1981)

and the comparison of the analytical crack pattern with the experimental one was shown in Fig. 1.17. It was indicated that the analysis was capable of predicting the crushing near the bottom of diagonal concrete struts and the flexural tension failure. However, the slip along diagonal crack was not predicted by the analysis. The modelling of the shear transfer across cracks and the failure of cracked concrete were pointed out as the problem to be solved.

In the previous studies on RC beams by FEM, as stated above, there were various modelling methods of constitutive materials. However, in the greater part of the previous studies, much emphasis was laid on the comparisons and agreements between the analytical and experimental results on the load-deflection curves, crack patterns, strains in reinforcing bars and concrete and principal stress distributions. It will be necessary not only to develop the rational model for the bond behavior, aggregate interlock and dowel action, but also to study the correspondence of analytical results to the contribution rates of shear resistant elements which has been obtained in the many previous experiments. The shear resistance mechanisms which have been studied by the previous macroscopic model like Kani's model (1966) will be also the object of analytical studies by FEM.

Noguchi (1982) studied the shear resistance mechanisms of simple beams (a/d = 2.4) without stirrups from the comparisons of the analytical shear contributions of the three shear resistant elements: the compression zone of concrete, the dowel action of the tension reinforcement and the aggregate interlock, with the experimental results. The analytical deformation conditions of the two different failure type specimens: the shear compression failure and the dowel splitting failure, were shown in Fig. 1.18.

Many analytical studies have been carried out on RC shear walls. The only names of researchers and references were listed as under. Iwashita (1967), Franklin (1970), Cervenka (1970), Yuzugullu (1973), Berg (1973), Colville (1974), Araki and Yamaguchi (1974), Ono and Adachi (1976), Darwin (1976), Agrawal (1976), Sato and Shirai (1977), Ohnuma and Kawamata (1977), Kushiyama and Arai (1977), Koike and Takeda (1977), Arai (1978), Mitsukawa and Baba (1978), Nomura and Sato (1978), Kokusho and Hayashi (1978), Sato and Shirai (1978), Nomura and Ono (1978), Grootenboer (1978), Mehlhorn (1978), Cedolin (1979), Sato and Shirai (1979), (1980), Morinaga and Takagi (1980), Mochizuki and Kawabe (1980), Aktan (1980), Seya and Matsui (1981), Kano and Takagi (1981), and Murakado and Yoshizaki (1981).

The shear analyses of column, beam-column joints, box-typed and cylinder-typed shear walls, prestressed concrete reactor vessels (PCRV) have been also carried out actively by FEM. Above all, the analytical studies on the PCRV have rendered great services in the development of the analytical method of RC structures by FEM. The main studies and references on the finite element analysis of the PCRV were listed as under. Zienkievicz (1972), Argyris (1974), (1978), Goodpasture (1978), Muto and Morikawa (1980), Imoto (1980), Mochida and Okada (1980), Isobata (1981), Mikame (1981), Prestress Concrete, Special Issue: PC structures for the nuclear facilities (1981).

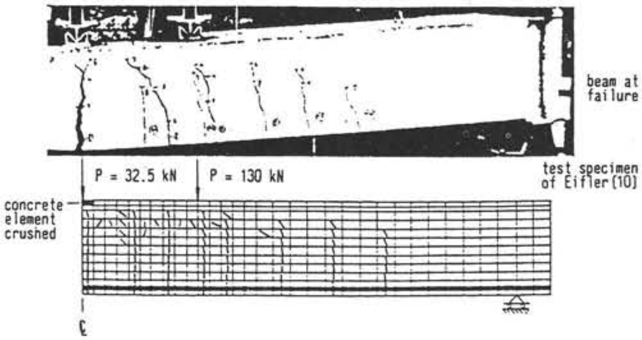


Fig. 1.15 Crack Patterns (Plauk, 1981)

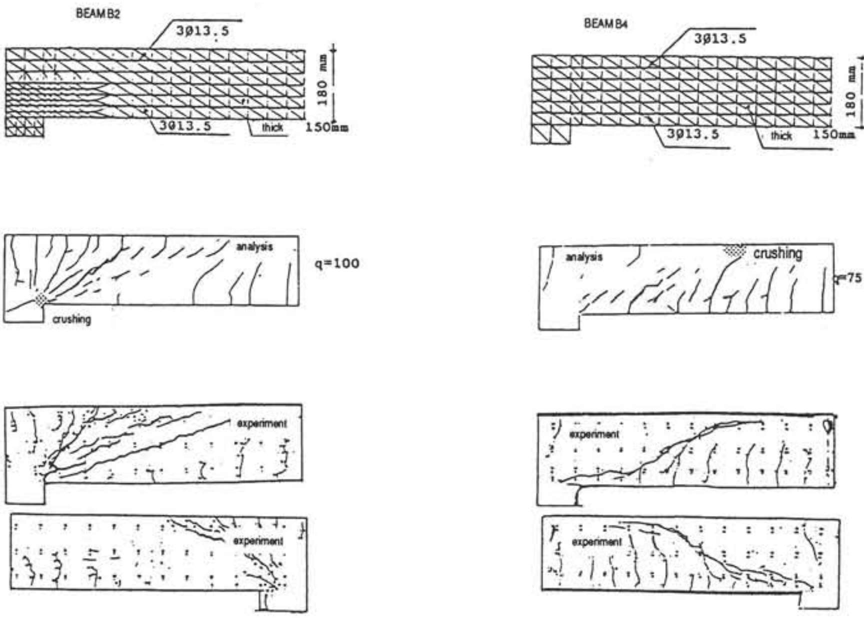
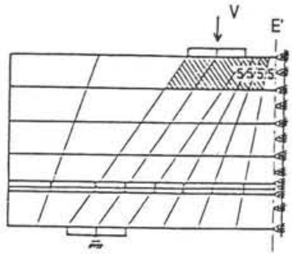
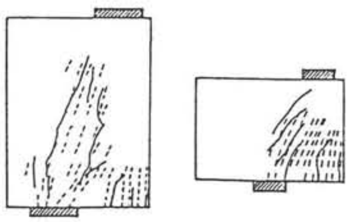


Fig. 1.16 Finite Element Idealization and Crack Patterns (Beukel, 1981)



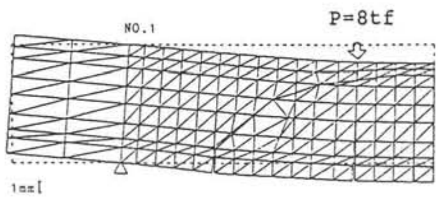
Idealization of
Finite Elements



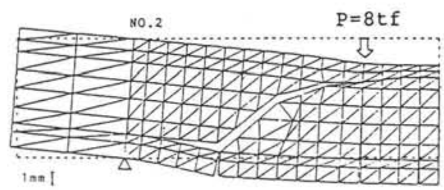
(a) T5 (b) T7

Predicted and Observed
Crack Pattern

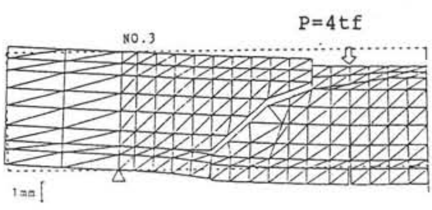
Fig. 1.17 Finite Element Idealization and Crack Patterns (Niwa and Okamura, 1981)



(a) Analytical Model for Shear Compression Failure Type Specimen Without Pre-Crack



(b) Analytical Model for Shear Compression Failure Type Specimen with Pre-Inclined Crack



(c) Analytical Model for Dowel Splitting Failure Type Specimen with Pre-Inclined Crack and Roller in Compression Zone

Fig. 1.18 Analytical Deformations (Noguchi, 1982)

1.3 Stress-Strain Relation of Concrete under Multi-axial Stresses

The stress strain relation of concrete under biaxial or triaxial stresses has been represented by numerous formulae.

Franklin (1970) defined secant moduli in each principal direction by calculating effective strain, where the stress strain relation under biaxial stresses was transferred to that under uniaxial stress. But concrete was considered isotropic by adopting the smaller modulus among them.

Kupfer and Gerstle (1973) proposed an isotropic model which was represented by the nonlinear formulae of bulk modulus K and shear modulus G (Fig. 1.19). The moduli were defined by the function of octahedral shear strain and stress based on their own test results under biaxial stresses. Cedolin, Crutzen and Poli (1977) applied this method to a triaxial stress model. Here K and G were defined by the functions of octahedral normal strain and octahedral shear strain, respectively (Fig. 1.20). Kotsovos and Newman (1978) represented K and G as the functions of octahedral normal stress and octahedral shear stress, respectively (Fig. 1.21). These formulae were defined as the secant moduli and, when incremental method was adopted, the tangent moduli were obtained by their differentiation. In these models only monotonic loading was considered.

Yamada (1970) presented the nonlinear matrix of stress strain relation by the plasticity theory based on the Drucker-Prager's yield criterion. Kokusho and Takiguchi (1971) applied this method to two-dimensional problems. The constant in the function, which represented the nonlinearity of concrete, was obtained from the strength of Kupfer's biaxial tests (Fig. 1.22). This method was adopted by many researchers. Two-dimensional problems were studied by Muto and Miyashita (1972), Takeda and Imoto (1973), Ono and Adachi (1973), and Shirai and Sato (1981). Noguchi (1977) pointed out that the nonlinear matrix of stress strain relations obtained by these models needs studies on the possibility of whether it could represent the true stress strain relation of concrete. As the results of comparative studies, Noguchi showed that the strains in both principal direction obtained by the plasticity model were considerably less than experimental results under high stresses. For three dimensional problems, Takeda and Imoto (1975) presented an analytical method. Muto and Inoue (1976) analyzed reinforced concrete columns under repeated lateral forces, where the constant in the yield criterion was determined by simulating the results of column tests under uniaxial forces, such as the axial stress in concrete and the strain in lateral reinforcement (Fig. 1.23).

Kupfer·Gerstle (1973)

$$\frac{K_S}{K_O} = \frac{\frac{G_S}{G_O}}{e^{-(c\gamma_{oct})p}} \tag{1}$$

$$\frac{G_S}{G_O} = 1 - a\left(\frac{\tau_{oct}}{f_c^{'}}\right)^m \tag{2}$$

K_s : secant bulk modulus $\quad$ K_o : initial bulk modulus

G_s : secant shear modulus $\quad$ G_o : initial shear modulus

$f_c^{'}$: uniaxial compressive strength of concrete

a, m, c, p: nondimensional constants

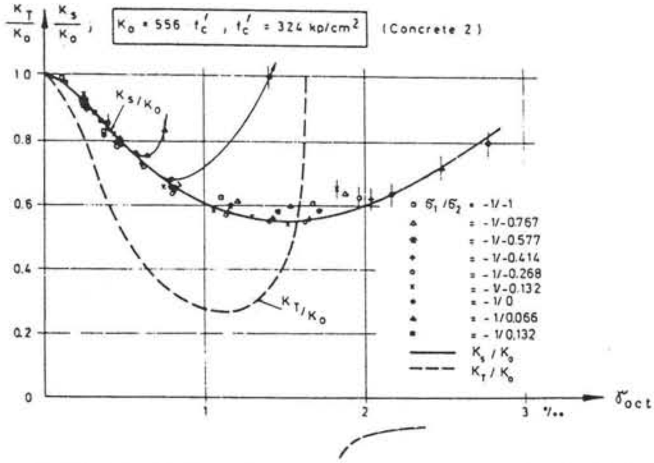


Fig. 1.19 (a) Bulk Modulus - Octahedral Shear Strain Relation

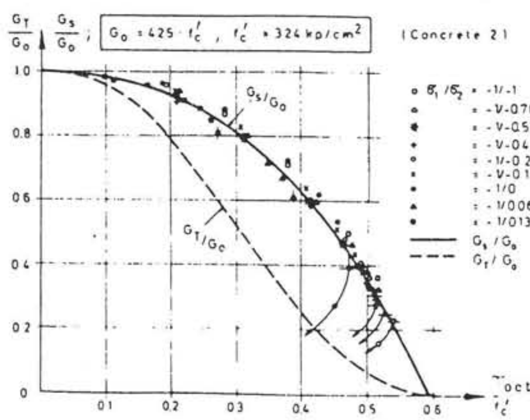


Fig. 1.19 (b) Shear Modulus - Octahedral Shear Stress Relation

Cedolin·Crutzen·Poli (1977)

$$\frac{K_S}{K_O} = ab^{-\frac{\varepsilon_{oct}}{c}} + d \tag{3}$$

$$a = 0.85,\ b = 2.5,\ c = 0.0014,\ d = 0.15$$

$$\frac{G_S}{G_O} = pq^{-\frac{\gamma_{oct}}{r}} - s\gamma_{oct} + t \tag{4}$$

$$p = 0.81,\ q = 2.0,\ r = 0.002,\ s = 2.0,\ t = 0.19$$

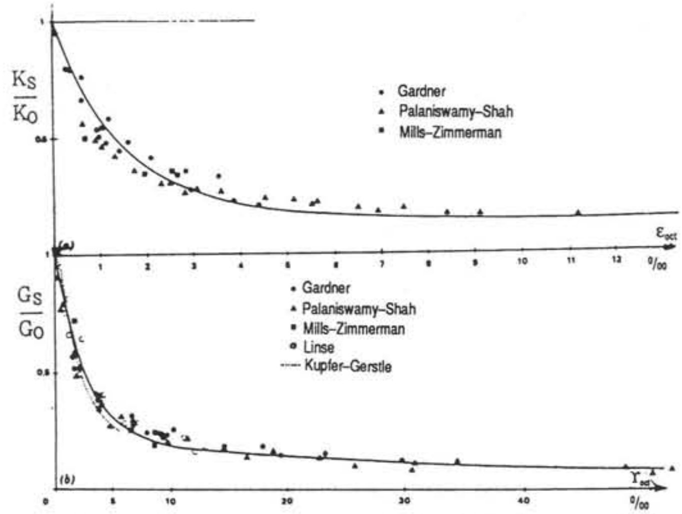


Fig. 1.20 Bulk Modulus - Octahedral Normal Strain Relation and Shear Modulus - Octahedral Shear Strain Relation

Kotsovos·Newman (1978)

$$\frac{K_S}{K_O} = \frac{1}{1 + 0.52\left(\frac{\sigma_{oct}}{f_c'}\right)^{1.09}} \tag{5}$$

$$\frac{G_S}{G_O} = \frac{1}{1 + 3.57\left(\frac{\tau_{oct}}{f_c'}\right)^{1.7}} \tag{6}$$

$$K_o = 13.23 \times 10^3\ \text{N/mm}^2$$
$$G_o = 13.55 \times 10^3\ \text{N/mm}^2$$

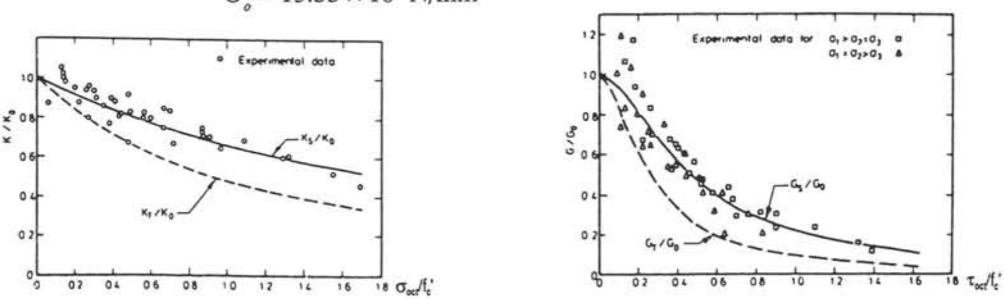
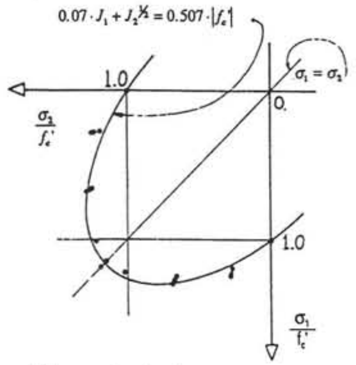


Fig. 1.21 (a) Bulk Modulus - Octahedral Normal Stress Relation **(b) Shear Modulus - Octahedral Shear Stress Relation**



= uniaxial compressive strength

• experimental results by H. Kupfer, H.K. Hilsdorf and H. Rusch

Fig. 1.22 Biaxial Strength in Principal Stress Space (Kokusho and Takiguchi, 1971)

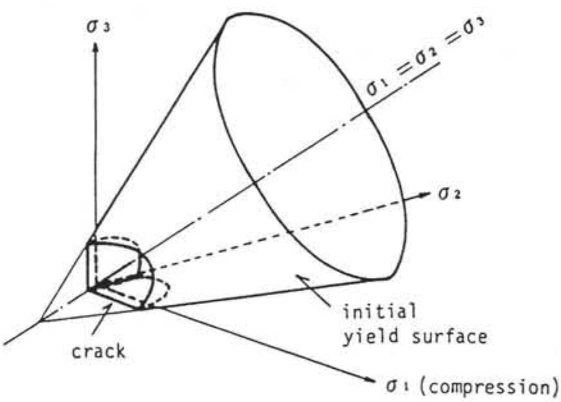


Fig. 1.23 Initial Yield Surface in Principal Stress Space (Muto and Inoue, 1976)

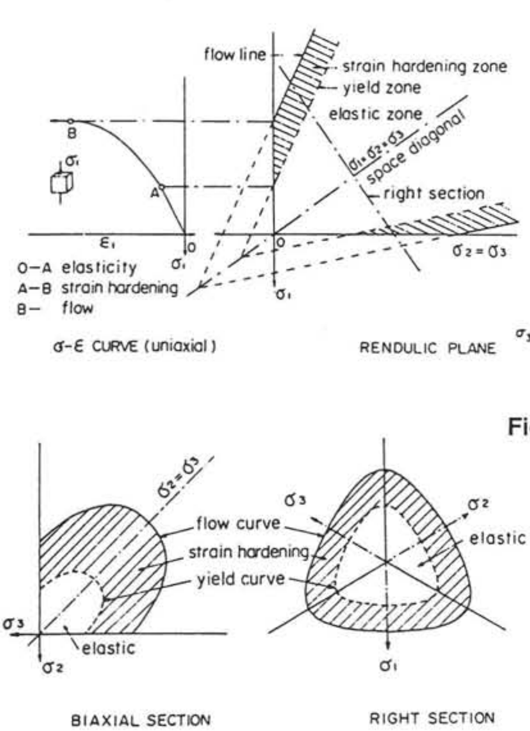


Fig. 1.24 Yield and Flow Curve in Principal Stress Space (Iwashita and Hirano, 1971)

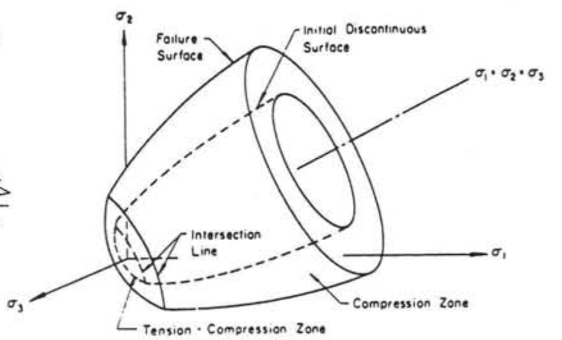


Fig. 1.25 Failure and Initial Discontinuous Surfaces in Principal Stress Space (Chen, 1975)

Above-mentioned methods were based on the Drucker-Prager's yield criterion, and the parameter representing the nonlinearity of concrete was only one. Iwashita and Hirano (1971) defined another criterion including one more parameter of the third invariant to trace experimental results more accurately (Fig. 1.24). Chen (1975) defined a further complicated function with three parameters (Fig. 1.25).

Darwin and Pecknold (1974) proposed an orthogonally anisotropic model with different tangent moduli in each principal direction. The Young's modulus was determined from the Saenz's formula by seeking the equivalent uniaxial strain and the shearing modulus was determined to be constant in any direction. Noguchi (1977) applied this model to two dimensional problems. Elwi and Murray (1979) developed this idea to three dimensional axisymmetric model. Isobata (1978) also studied axisymmetric problems where modulus and Poisson's ratio were functions of normal strains.

Bažant (1976) developed an entirely different approach for modelling the nonlinear response of concrete by means of an extension of the inelastic endochronic model which was introduced for metal initially by Valanis. In this model, the inelastic strain accumulation was characterized by a scalar parameter, called intrinsic time. This model could predict strain-hardening, unloading diagrams and contraction of hysteresis loops. Further by extending the theory to concrete, it could express the sensitivity of the intrinsic time increments due to hydrostatic pressure, the inelastic dilatancy due to shear straining and the strain-softening that occurred after the peak stress. The function required the specification of 19 constants, which all pertained to the compressive strength of concrete.

Okamura (1981) proposed the total strain formulation by using iterative procedure. This method calculated the stresses by total strains and constitutive equations which contained invariants with regard to coordinate transformations as parameters. This method was used for the following reasons: (a) Strain softening tendency is easily modeled. (b) No accumulation of errors occur in estimating the stresses. (c) Experimental data are directly used for making the constitutive equations. But Argyris (1981) pointed out that this method restricted the range of application primarily to monotonic loading regimes.

Each Formulation is executed generally in the same way under biaxial or triaxial stresses. Parameters in the formulae are determined by the application to experimental results according to each stress condition. Here it should be emphasized that the stress-strain relation of concrete should be expressed adequately as well as the yield criteria under multiaxial stresses. From this view point many researches have been made regarding the failure criteria under biaxial stresses. Especially experimental results obtained by Kupfer et al. are considered to be reliable although the results are restricted to the condition of constant stress ratios. On the contrary test data under triaxial stresses are very few. Especially this tendency is so outstanding about strains that the stress strain relation under triaxial stresses should be studied more hereafter.

1.4 Behavior of Concrete after Peak Stress

The stress strain of concrete after peak stresses is represented by descendent slope called strain-softening which shows the behavior in-between ductile and brittle state. Ductility of structures is strongly influenced in accordance with how this characteristic is assumed.

The simplest way of considering this behavior is by means that the stiffness of the element is assumed to be zero after the element has reached the fracture criterion defined by stresses or strains and the internal stresses are replaced by the equivalent nodal forces. In the plasticity theory models adopted by Kokusho and Takiguchi (1971, Fig. 1.26) or Muto and Miyashita (1972), this approach was used because the plastic behavior of concrete was considered to be the process of strain-hardening and its hardening ratio was defined as the function of monotonically increasing plastic work. Muto and Inoue (1976) considered that a reinforced concrete column was composed of core concrete and cover concrete. In this model the former was represented by three dimensional elements in which the same approach was applied, but the latter was represented by one dimensional elements which had descendent slope after peak stress (Fig. 1.27).

In the model proposed by Takeda and Imoto (1975), a zero tangent modulus was used on the downward sloping portion of the stress-strain curve, and the unbalanced internal stresses were replaced by the nodal forces. In spite of the assumption based on the plasticity theory this model could introduce the strain softening in approximation (Fig. 1.28). Darwin and Pecknold (1976) used a similar method by assumption of the stress-strain relation shown in Fig. 1.29.

In above-mentioned models the formulation after compressive failure was executed by different methods from prior handling before failure. In the endochronic model proposed by Bazant (1976) strain-softening was included in the constitutive relations without the necessity for defining compressive failure surfaces (Fig. 1.30).

The total strain formulation adopted by Okamura et al. could calculate falling gradient continuously with an iterative procedure.

Furthermore, there were many cases where consideration was neglected regarding compressive failure although the idea was proposed. In the case in which failure was assumed, its adequacy was not obvious because fractured elements were very few in the analyzed structure. This is one of the important problems which requires the further research.

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \\ d\tau_{12} \end{Bmatrix} = \frac{1}{1-\nu^2} \begin{bmatrix} E_1 & \nu\sqrt{E_1E_2} & 0 \\ \nu\sqrt{E_1E_2} & E_2 & 0 \\ 0 & 0 & \frac{1}{4}(E_1E_2 - 2\nu\sqrt{E_1E_2}) \end{bmatrix} \begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \\ d\gamma_{12} \end{Bmatrix} \tag{7}$$

Constitutive Equation by Orthogonally Anisotropic Model (Darwin and Pecknold, 1974)

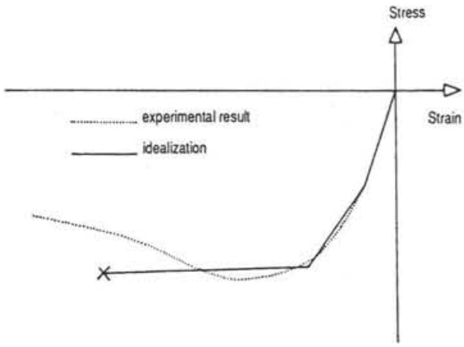


Fig. 1.26 Stress-Strain Relation under Uniaxial Compression (Kokusho and Takiguchi, 1971)

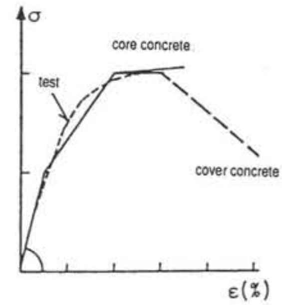


Fig. 1.27 Stress-Strain Relation under Uniaxial Compression (Muto and Inoue, 1976)

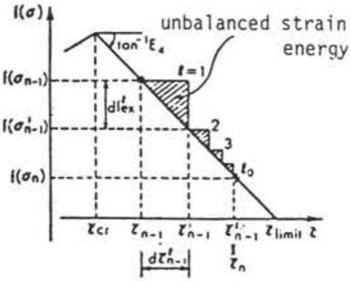


Fig. 1.28 Analytical Procedure of Strain-Softening (Takeda and Imoto, 1975)

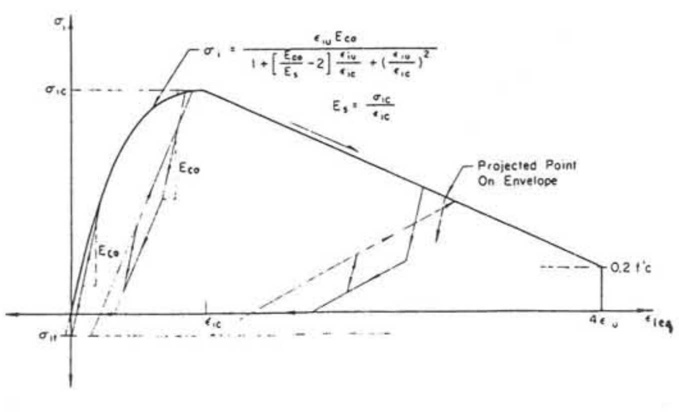


Fig. 1.29 Stress-Strain Relation under Cyclic Load (Darwin and Pecknold, 1976)

1.5 Cracking and Its Propagation

There are basically following two approaches for the idealization of cracking, damage and localized fracture.

- discrete crack model
- smeared crack model

Discrete Crack Model

Ngo and Scordelis (1967) put in cracks beforehand (Fig. 1.1). The same beams with different crack patterns were analyzed in the elastic range, and stresses in concrete, and steel and bond stress were obtained. This resulted in showing the possibility of applying the finite element method to reinforced concrete structures.

In the model proposed by Nilson (1968) the elements were disconnected at their common corners when the average value of principal tensile stresses in two adjacent elements exceeded the tensile strength (Fie. 1.31). If the principal stresses were sloping to the border line of the element, a crack was set up at the border line nearest to the plane at right angle to the principal stress direction. After the propagation of cracks, loads were released completely and the sequent model was reloaded.

Ngo, Scordelis and Franklin (1970) predetermined the shape and location of the crack with linkage elements connecting two adjacent nodes (Fig. 1.32). Cracks were formed by setting the stiffness in the perpendicular direction to the crack at zero. This method was adopted by Ono and Adachi (1973). It is useful in the case when crack patterns can easily be derived from the experimental results, but if not, there is apprehension of obtaining stresses or strains extremely deviated from actuality.

Taylor (1972) proposed a model in which the inclination of the crack was modified automatically by moving grids of quadrilateral elements in the direction of the principal strain. In this model the ability of changing the inclination was limited to some extent. Noguchi (1977) devised a more flexible model by adopting triangular elements with six nodes (Fig. 1.33).

Saouma and Ingraffea (1981) presented an analysis procedure that automatically simulated discrete crack nucleation and extension, with the length of the crack being governed by a fracture mechanics criterion (Fig. 1.14).

This discrete crack model can predict the spacings and widths of actual cracks, so the representation of internal shear transfer between crack surfaces is comparatively easy. But, because this requires reformulation of the finite element topology after each crack propagation, it possesses complex computational problems and needs much computation time. It can be said

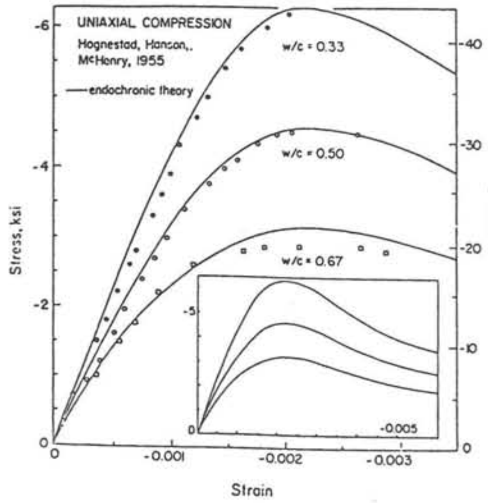


Fig. 1.30 Fit of Uniaxial Stress-Strain Data (Bazant, 1976)

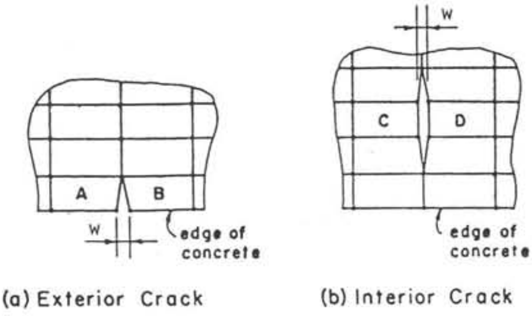


Fig. 1.31 Development of Cracking (Nilson, 1968)

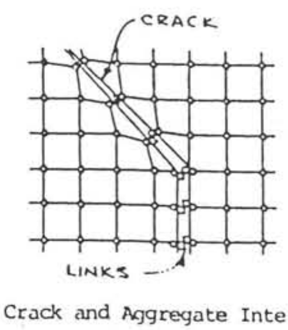


Fig. 1.32 Linkage Element (Ngo, Scordelis and Franklin, 1970)

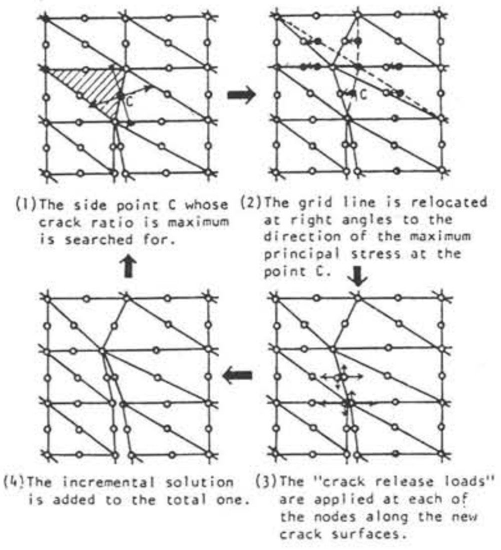


Fig. 1.33 Analytical Procedure of Crack Propagation (Noguchi, 1977)

that this approach is suitable in case of a structure in which discrete cracks must be apprehended as actual crack lines for the reason that the shear transfer between crack surfaces, bond and dowel actions have great influences on the fracture process of the structure.

Smeared Crack Model

In the model proposed by Franklin (1970), when the principal tensile stress exceeded the failure criterion, the stiffness in the principal direction was set at zero and the element was considered to become orthogonally anisotropic because cracks were formed at the right angle to that direction. This approach was adopted by Liu (1971), Kokusho and Takiguchi (1971), Muto and Miyashita (1972), Isobata (1970), Takeda and Imoto (1973), Suidan and Schnobrich (1973), Fukushima and Obata (1972), Darwin and Pecknold (1976), and Muto and Inoue (1976).

This approach assumes innumerable cracks in an element, so crack spacings and widths are hard to evaluate. But it becomes rather easy because the movement or separation of nodes is not necessary. Hence it is useful for the structures in which many cracks occur, the prediction of crack patterns are difficult, or much computing time is necessary because its shape is complex or large.

1.6 Interface Shear Transfer (Aggregate Interlock)

After cracking the shear stresses are transferred along the rough surfaces of narrow cracks through aggregate interlock. The analytical methods for considering this effect are classified into two groups according to the approach of cracking, that is, discrete crack model or smeared crack model.

Discrete Crack Model

Ngo, Scordelis and Franklin (1970) adopted joint elements between adjacent nodal points where the stiffness parallel to the crack was assumed to be a constant value representing the aggregate interlock. This was the very first attempt in application of analyzing the effect.

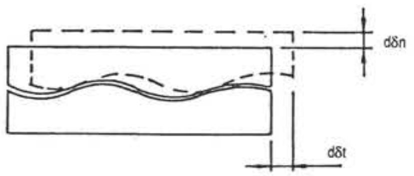
Ono and Adachi (1973) adopted this approach assuming the stiffness of joint elements from the formula proposed by Fenwick and Paulay (1968). Here the shear force-slip relation was represented as a function of crack width and concrete strength. In the analyses the crack width was assumed to be constant throughout the incremental step, and the stiffness parallel to a crack was evaluated by the formula.

Bažant (1981) and Tsubaki (1981) pointed out that the asperities of the rough crack surfaces would tend to cause a spreading or dilatation of the crack upon sliding as shown in Fig. 1.34. When this dilatation was constrained either by surrounding portions of the structure and its supports, or by reinforcing bars crossing the crack, compression resulted in the concrete which could strengthen the structure but might also under some conditions cause over-stress in the reinforcing

Shear Stress along Cracked Surfaces (Fenwick and Paulay, 1968)

$$V_{cr} = (467/w - 8410)(0.0255\sqrt{f_c'} - 0.409)(\delta - 0.0436w) \tag{8}$$

f_c' : compressive strength of concrete (psi)
w : crack width (in) V_{cr} : shear stress (psi)
δ : relative displacement across cracked surfaces (in)



$$\begin{Bmatrix} d\sigma_{nn} \\ d\sigma_{nt} \end{Bmatrix} = \begin{bmatrix} B_{nn} & B_{nt} \\ B_{tn} & B_{tt} \end{bmatrix} \begin{Bmatrix} d\delta_n \\ d\delta_t \end{Bmatrix} \tag{9}$$

Fig. 1.34 Crack Displacements (Bažant and Tsubaki, 1981)

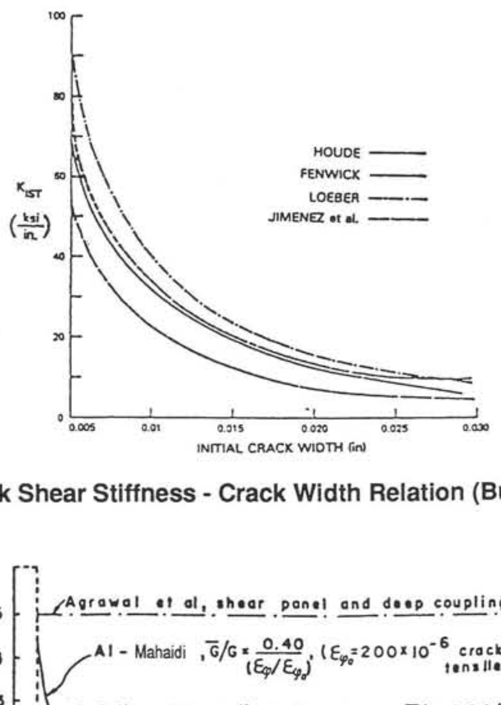


Fig. 1.35 Crack Shear Stiffness - Crack Width Relation (Buyukozturk, 1979)

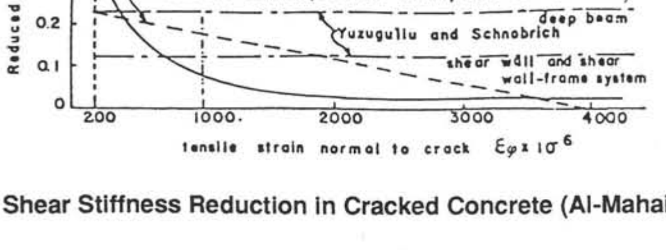


Fig. 1.36 Shear Stiffness Reduction in Cracked Concrete (Al-Mahaidi, 1979)

steel in tension. This behavior can be represented by a stiffness matrix, and its value should be determined by experimental results. But for the lack of experimental data the matrix was simplified by assuming two parameters indicating crack friction and dilatancy.

Buyukozturk (1979) showed the comparison on the relation of crack shear stiffness vs. initial crack width in different studies (Fig. 1.35).

Smeared Crack Model

When smeared cracks are assumed, the shear resistance is averaged and included as a reduced shear modulus in the material stiffness matrix for the elements representing the crack region.

Franklin (1970), and Darwin and Pecknold (1976) assumed that shear modulus G of cracked concrete was one-fourth of Young's modulus E in the direction with no crack.

Suidan and Schnobrich (1973) assumed 0.5 G. Isobata (1978), and Muto and Inoue (1976) assumed 0.3 G and 0.2 G respectively. Here G is elastic shear modulus.

Okamura et al. (1981) analyzed deep beams with and without assuming shear transfer effect. From the results it was pointed out that the estimation of shear transfer was an important problem in order to predict the slip failure mode.

Takeda and Imoto (1975) evaluated the reduced shear modulus as the function of crack width, which was obtained by integrating strains in the direction perpendicular to the crack, and the strength and shape of aggregates.

Shirai and Sato (1981) defined the modulus as a function of crack width, concrete strength and crack spacing which was evaluated by Morita's formula.

Al-Mahaidi (1979) showed the reduction of shear modulus due to cracking which were used by different analyses (Fig. 1.36). The range is surprisingly wide.

It is not clear at the present how much influence this shear transfer effect has on actual structures. At least this effect should become important for a structure in which shear resistance is dominant, especially under nonproportional load histories. But the evaluation of this effect is very difficult, so it is desirable that the study on simple test specimens are pursued in the beginning. And furthermore, Mattock (1974) pointed out that under repeated loading the strength of shear transfer was reduced to 80 percent of the value under monotonical loading and that its slip was large and its crack width was small comparatively. When a structure is subjected under repeated forces such as earthquake loads, this behavior should be considered in future.

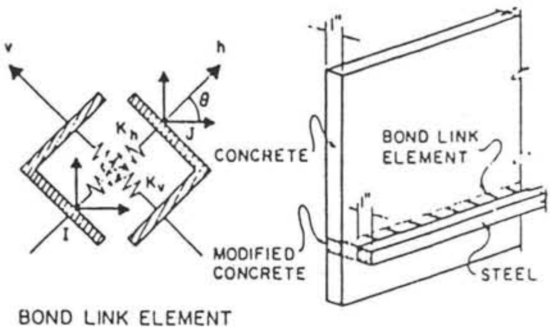
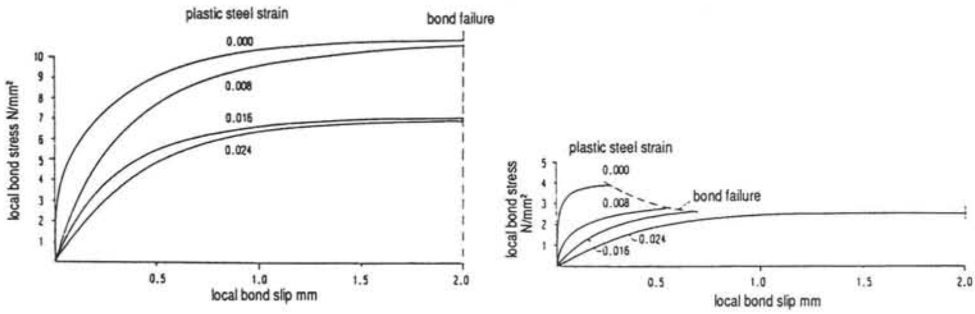


Fig. 1.37 Bond Link Model (Ngo and Scordelis, 1967)



(a) Bond Stress-Slip Relations for Bond in Uncracked Regions or in Some Distance from Cracks

(b) Bond Stress-Slip Relations for Bond near Cracks

Fig. 1.38 Different Bond Properties near Cracks and between Cracks (Eifler, 1974)

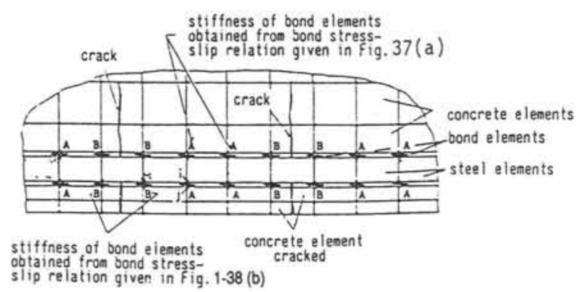


Fig. 1.39 Representation of Bond in Cracked Reinforced Concrete Beam (Plauk, 1981)

1.7 Modelling of Reinforcing Bars

The reinforcing bar was generally assumed to be a linear element in the previous finite element analysis of RC members. In the analysis of shear walls, the reinforcing bars were often distributed uniformly throughout the concrete element. It is necessary to consider the dowel action from the local bending of the reinforcing bar in the analysis of the shear dominant member. In this case, the reinforcing bar was often represented by the linear strain two-dimensional element.

The stress-strain relations of the reinforcing bar was often assumed to be linear elastic or bi-linear (perfect elasto-plastic or strain-hardening type). The von Mises criterion was frequently used in the two-dimensional element.

The above-mentioned simplified model has been used in the previous shear analyses under the monotonic loading. However, the more refined model under the reversed cyclic loading will be necessary in order to study on the restoring force characteristics and the shear failure after the flexural yielding. The material model under the reversed cyclic loading, which was used in the previous fiber model for the flexural dominant member, will be a good reference. (Fujii and Aoyama (1973), Bazant (1977), Kokusyo and Takiguchi (1980))

1.8 Bond Between Reinforcing Bars and Concrete

The analytical model for the bond between the reinforcing bar and concrete was reviewed in detail by Noguchi (1976), (1977) and AIJ Committee (1980). Therefore, the recent analytical model will be mainly reviewed in this chapter.

The analytical deflection, which was obtained on the assumption of the perfect bond, showed a tendency to become smaller than the experimental deflection, as the failure progressed in the member. This is more remarkable in case of the concentrated reinforcing arrangement like a beam and a column than a shear wall and a slab, and indicates that it is necessary to consider the bond-slip behavior in the analysis of reinforced concrete.

The bond link element proposed by Ngo and Scordelis (1967) has been used most generally as the analytical model of the bond mechanism. The reinforcement node and the concrete node are connected by the bond link element which was composed of the two orthogonal springs, as shown in Fig. 1.37. The bond slip between the reinforcing bar and concrete and the corresponding bond stress transfer are expressed by the deformation characteristics of the springs, which are given on the assumption that the bond stress is represented as the function of the relative slip.

Muguruma (1972), Kosaka (1973), Kokusho (1974), Labib (1978) investigated the applicability of the bond link model by applying the model into the simplified specimen like the pull out bond specimen. Kokusho indicated that the analytical strain in concrete within twice as much distance from the surface of the deformed bar as the diameter of the deformed bar was smaller than

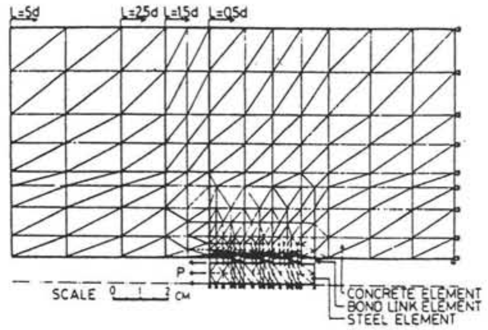


Fig. 1.40 Finite Element Idealization (Kokusho and Yoshida, 1981)

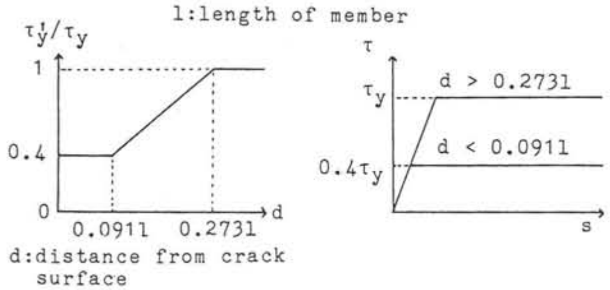
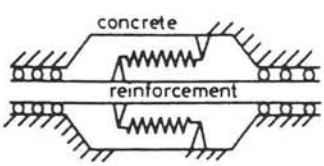
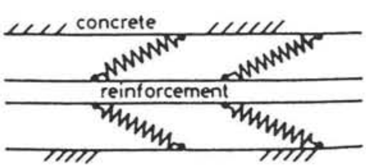


Fig. 1.41 Reduction of Bond Elastic Limit according to Distance from Crack Surface (Nomura and Sato, 1978)



(a) One-directional Spring Model (Stauder, 1973; Schafer, 1975)



(b) Inclined Spring Model (Van Mier, 1978)

Fig. 1.42 Spring Models

the experimental strain from the analysis of pull out specimens, but there was little difference between the analytical and experimental strains at more than twice as much distance as the diameter of the deformed bar. Labib analyzed the tensile specimens, considering the crack which occurred around the deformed bar. It was indicated that the analytical primary crack pattern and crack spacing gave a good agreement with the experimental results. In this model, the modelling of the deformation characteristics of the bond link spring perpendicular to the axis of the deformed bar was made a trial, and it was shown that the bond behavior of the deformed bar could be predicted to a certain extent.

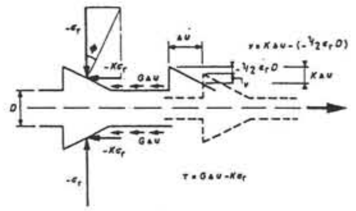
The bond link model was applied to the analysis of the member in many studies. Franklin (1970) investigated the effect of the bond characteristics on the failure mode of the beam. Kokusho and Takiguchi (1971) inquired into the difference of the crack pattern, deformation, strain according to the bond characteristics. Consequently, it was pointed out that the analytical shear cracking load and the load when the stirrup strain began to increase got higher, as the bond properties became poorer. Ohtzuki (1975) investigated the effect of the modelling for the bond stress-relative slip curve, the concrete stress-strain curve, the failure criterion on the analytical results of RC beams without stirrups. It was indicated that the effect of the bond characteristics on the crack propagation, deformation and strength was the most distinguished, and the shear cracking load got higher and the shear strength got lower, as the bond properties became poorer.

Muto and Inoue (1976), (1981), Sugano and Inoue (1980), (1982) adopted the bond link into the three-dimensional analytical model under the reversed cyclic loading, and investigated the confinement effect of the tie in the analysis of the column.

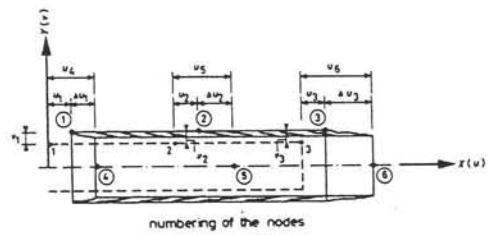
In the analysis of the shear wall, the perfect bond had been generally assumed by Darwin (1976) and others. Mitsukawa and Baba (1978) analyzed the shear wall using the bond link, assuming that the deformation characteristics of the spring parallel to the reinforcement axis were based on the bond stress-bond slip rule under the reversed cyclic loading, which had been proposed by Morita (1975). The distributions of the bond stress and bond slip were investigated in this analysis.

Noguchi (1980), (1981) studied the effect of the bond characteristics of the beam main bar in the beam-column joint on the shear resistance mechanisms of the joint and the restoring force characteristics of the overall frame, using the bond link model.

Plauk (1981) pointed out that the bond stiffness and bond stress near cracks were significantly lower than in some distance from the crack surface or in uncracked regions for equal values of bond slip, as shown in Fig. 1.38, from the results of Eifler's test (1974), and the different bond properties near cracks and between cracks were incorporated in to the finite element model with the bond link, as shown in Fig. 1.39. The analytical crack spacing of a simple beam gave a good agreement with the experimental result, even if the smeared crack model was used.



(a) Teeth Model



(b) Bond Element Model

Fig. 1.43 Bond Models for Deformed Bars

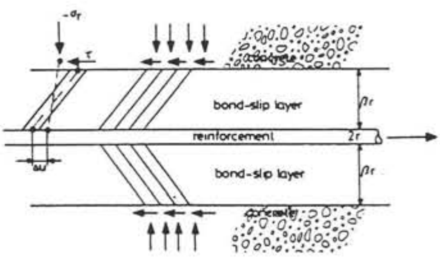


Fig. 1.44 Bond-Slip Model (De Groot, 1981)

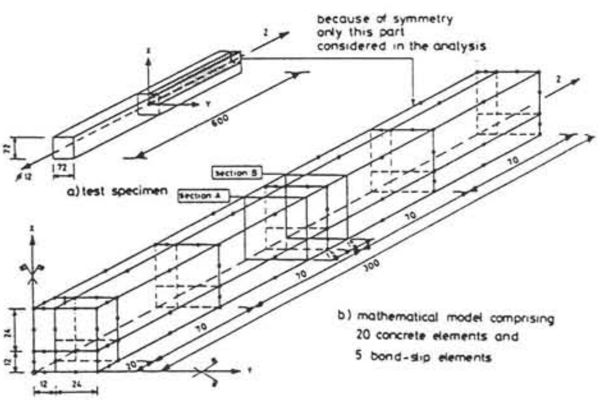


Fig. 1.45 Finite Element Idealization in Three-dimensional Analysis of Tensile Specimen (De Groot, 1981)

Kokusho and Yoshida (1981) investigated the relations between the bond stress-bond slip curve and the distance from a crack surface by both the analytical and experimental approaches. The finite element idealization in the axi-symmetric nonlinear analysis was shown in Fig. 1.40. The spring stiffness of the bond link which connected the lug of a deformed bar and concrete was given as the very large value in compression, and as the almost zero value in tension and in shear. The analytical bond stress and bond slip were fairly smaller than the experimental results, but gave a good agreement in a qualitative tendency.

It is known that the bond properties in round bars are dependent upon the strain in reinforcing bars and the distance from the crack surface. Nomura and Sato (1978) analyzed the column with the bond link, assuming the reduction of the elastic limit of the bond according to the distance from the crack surface, as shown in Fig. 1.41. There was a remarkable difference between the normal bond and the perfect bond in the analysis of the shear failure type column with the shear span ratio, a/h = 2.0, subjected to the monotonic loading. In the perfect bond model, the crack pattern was almost flexural type and the maximum strength was fairly lower than the experimental result. In the normal bond model, the deflection was fairly smaller than the experimental result, but the crack propagation was not far from the experimental result. In the analysis under the reversed cyclic loading, it was pointed out that the deflection was smaller than the experimental result, and the shape of the hysteresis loop under the unloading was different from the experimental result. It was indicated that the elucidation on the bond mechanism of the round bar under the reversed cyclic loading was necessary in future.

The slip on the reinforcing bar surface is represented by the bond link. In case of the deformed bar, the concrete around the bar follows the displacement of the bar to the tolerable degree, making contact with the lug of the bar. Consequently, the internal cracks occur from the bar lug, and propagate into the surrounding concrete. Therefore, it seems to be difficult to explain the behavior of the concrete around the deformed bar by the bond link model.

Ikeda (1973) proposed the finite element model composed of the spring system, considering the inclined crack around the deformed bar. In this model, the coupling springs were arranged in the axial direction between the deformed bar and concrete elements in order to represent the shear deformation of the concrete and the flexural deformation after the internal crack occurred.

Kokusyo and Yamanaka (1974) proposed the bond zone model. The relative slip was transformed into the shear deformation of the bond zone around the deformed bar, by reducing the shear stiffness of the concrete in the bond zone. It was shown that the analytical strain in concrete within twice as much distance from the surface of the deformed bar as the diameter of the deformed bar by the bond zone model, was nearer to the experimental result than by the bond link model.

Noguchi (1977) applied the orthotropic theory into the internal crack zone around the deformed bars. In this model, the elasticity modulus, nearly perpendicular to the internal crack was decreased together with the shear modulus. It was indicated that the analytical longitudinal strain in the concrete around the deformed bar gave a good agreement with the experimental result from the analyses of the previous pull out and tensile specimens.

Ma and Bertero (1976) developed the stiffness reduction method in the concrete layer around the deformed bar, and proposed the model in which the initiation and propagation of the internal crack were considered by modelling the lugs of the deformed bar with fairly good approximation to reality It was shown that the inclination of the analytical internal cracks gave a good agreement with the Goto's experimental results, and the stiffness degradation of the bond stress-bond slip curve from the internal crack initiation was observed.

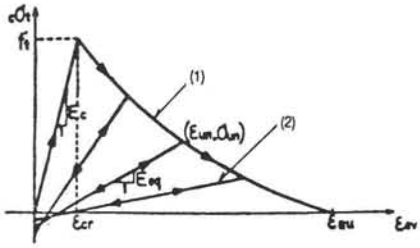
As the one-directional spring model proposed by Stauder (1973) and Schafer (1975), as shown in Fig. 1.42 (a), and the two-directional independent springs model were difficult to represent the bond mechanism of the deformed bar, Van Mier (1978) proposed the inclined spring model, as shown in Fig. 1.42 (b).

Gijsbers (1978) considered the roles of the surface friction and the interlock friction, which were the components of the bond mechanism of the deformed bar, in the teeth model as shown in Fig. 1.43 (a), and analyzed the tensile specimen using the bond element model, as shown in Fig. 1.43 (b). De Groot (1981) developed this model into the bond-slip model, as shown in Fig. 1.44, and conducted the two-dimensional and three-dimensional elastic plastic analysis of the tensile specimen, using the bond-slip element, in which the bond-slip model and the deformed bar element were incorporated. The finite element idealization in the three-dimensional analysis was shown in Fig. 1.45.

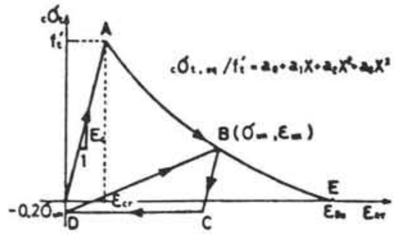
As the other models of the bond mechanism, Hassan (1973) assumed the bond stress distribution along the reinforcing bar. Viwathanatepa (1979) and Sauma (1980) represented the bond behavior by the bond layer and the shear element in the almost same way as the Bertero's original model (1976) and the Kokusho and Yamanaka's bond zone model (1974). Bazant (1981) proposed the bond model which was adapted to his proposed crack band model.

When the reinforcing bars are distributed closely in the overall member like a slab and a shear wall, the concrete between cracks restrains the reinforcing bar by bond, and bears a part of tensile forces. In this case, the tension-stiffening of concrete is assumed after cracking, and the downward portion of the tensile stress is provided in the apparent tensile stress-strain curve of concrete. Arai (1978) gave a consideration to the confinement of the concrete between cracks by bond in the estimation of the reinforcement stiffness in the cracking zone of the shear wall. Sato and Shirai (1978) adopted the effective tensile stress-average strain curves for the concrete, as shown in fig. 1.46 (a), in the analysis of the shear wall, and the modified models were proposed, as shown in Fig. 1.46 (b), (1979), (c), (1980).

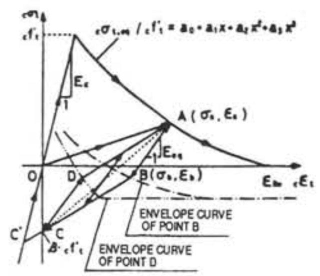
The many analytical models for the bond mechanism have been proposed as stated above. In



(a) (1978)



(b) (1979)



(c) (1980)

Fig. 1.46 Effective Tensile Stress - Average Strain Curves for Concrete (Sato and Shirai)

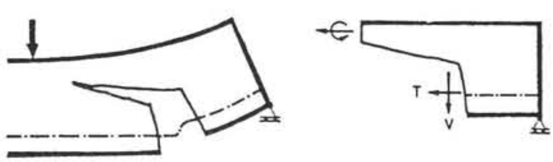


Fig. 1.47 Dowel Action of Reinforcing Bars (Ngo, 1970)

the most of the previous analytical studies with the bond link, the bond stress-slip relations are assumed to be independent on the position of reinforcing bars, but it is necessary to consider the degradation of the bond characteristics near cracks. The relations to the modelling method of cracks, the deformation characteristics in the direction perpendicular to the reinforcement axis and the modelling of the downward portion of the bond stress-slip relations are pointed out as the future subject.

For the bond mechanism of a deformed bar, it will be necessary to study the contact problem between the lug of the deformed bar and concrete in detail with a good approximation to reality in order to quantify the characteristic values for the envelope curve of the bond stress-slip relations. It is also an important subject to study the effect of the bond mechanism of the deformed bar including the bond splitting crack, especially the displacement of concrete in the perpendicular direction to the reinforcement axis by the wedging action, on the behavior of the overall member.

The bond stress-slip relations were often assumed from the results of the previous bond test in the analysis of RC member with the analytical bond model. Kaku (1978) pointed out that the bond characteristics in the member was affected by the magnitude of shear forces, the existence of the shear reinforcement, the combination effects of the bond splitting and the dowel action, the reaction forces of the support. The bond characteristics in beam-column joints were affected furthermore by axial forces, flexural forces and the effect of transverse beams. Viwathanatepa (1979) indicated that the bond characteristics of the beam main bar through the joint were affected by the stress condition of the main bar and the confinement effects of column main bars and ties on concrete, and underwent a change according to the position of the main bar. In future, it will be necessary to make both experimental and analytical investigation of the bond characteristics of the member, and to investigate the effect of the bond on the shear behavior and the shear strength of the member.

1.9 Dowel Action of Reinforcing Bars

The some shear forces are contributed by the dowel action of reinforcing bars, which is caused by the local bending of the reinforcing bars at the location where an inclined crack and longitudinal reinforcing bars intersect in a simple beam. (Fig. 1.47)

Ngo (1970) modelled the reinforcing bar in a linear strain triangular element which was able to represent the local flexural behavior. In this study, the effective dowel length was defined as the length of the section where the bond was in failure, as shown in Fig. 1.48. The dowel stiffness was in inverse proportion to the effective dowel length. As it was difficult to quantify the effective dowel length, it was assumed to be about 5 cm for the diameter of the reinforcing bar, 2.9 cm as a matter of convenience. When the dowel shear force increased, the dowel splitting crack occurred along the longitudinal reinforcing bars from the location where an inclined crack and longitudinal bars intersected. This phenomenon was represented by separating the concrete elements along the longitudinal bars. The redistribution of stress and the decrease of the dowel shear force were

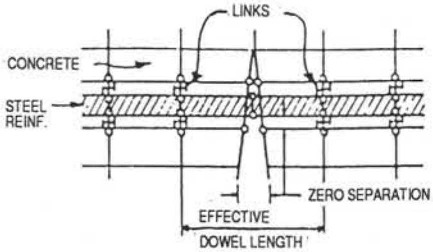


Fig. 1.48 Effective Dowel Length (Ngo, 1970)

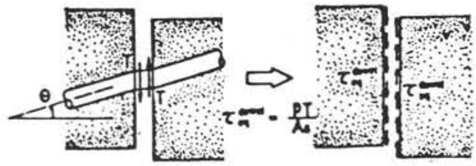


Fig. 1.49 Analytical Model for Dowel Action (Sato and Shirai, 1978)

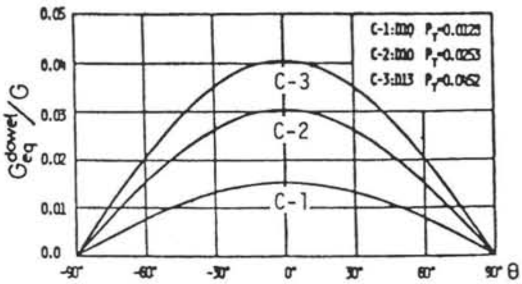


Fig. 1.50 Equivalent Shear Stiffness of Dowel Action (Sato and Shirai, 1978)

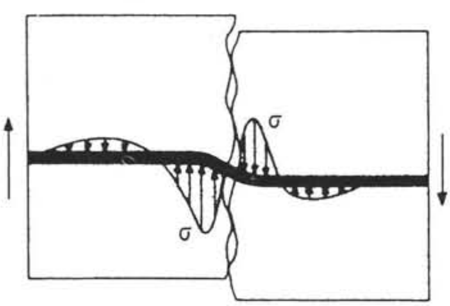


Fig. 1.51 Normal Stress Distribution Caused by Dowel Deformation of Reinforcing Bar (Grootenboer, 1981)

remarkable in the dowel splitting crack model. In this analysis, many unknown phenomena for the dowel action were explained. Though the focus was placed on the effective dowel length in this analytical model, and the large value was assumed for the spring stiffness of the bond link in the perpendicular direction to the reinforcing bar, it seems that the role of this spring is important in the actual dowel action, and the effective dowel length is dependent upon the spring stiffness.

Sato and Ono (1975) analyzed the simple beams with the predicted crack pattern, considering the dowel action of the longitudinal bars by a similar model to Ngo's (1970), and compared the analytical dowel shear force with the experimental result.

Sato and Shirai (1978) transformed the dowel force into the equivalent shear stress as shown in Fig. 1.49 in the analytical model of shear walls, and proposed the equivalent shear stress-shear strain relations based on the Dulacska's relative slip-dowel force relations (1972). The ratios of the equivalent shear stiffness to the elastic shear stiffness were plotted against the angle at which the reinforcing bar inclined with crack surfaces, as shown in Fig. 1.50. From this figure, it was pointed out that the equivalent shear stiffness of the dowel action was no more than 5 x 10-2 of the elastic shear stiffness.

Grootenboer (1981) indicated that it was necessary to provide the separate bond layers at the both sides of a reinforcing bar, because the bond layers did not remain in the axi-symmetric condition on account of the high normal stress in the concrete at the one side of the reinforcing bar caused by the dowel deformation of the reinforcing bar. (Fig. 1.51) But the perfect elastic plastic was assumed in the relations between the dowel force and the displacement in the perpendicular direction to the reinforcement axis in the plane stress program. The effect and the magnitude of the dowel force were pointed out as the future subject.

There are not so many researches for the modelling of the dowel action. It is necessary to compare the parametric analysis for the effecting factors of the dowel action like the spring constants of the bond link with the simple test, in which the focus is placed on the dowel action. As there is much accumulation of the experimental data on the contribution rates of major shear resistant elements including the dowel action in beams, it will be possible to clarify the effect of the dowel action on the overall behavior of a beam by comparing the above-stated parametric analysis of the dowel action with the previous experiment.

1.10 Conclusions

In this paper, the characteristics of the shear analysis of RC structures by the finite element method were generally reviewed, and the previous studies on the modelling of the basic mechanical properties of reinforced concrete were reviewed.

The primary points at issue in the shear analysis of RC structures by the FEM seem to be as follows.

1. There are limits on the experimental information on the modelling of materials.

2. It yet remains to be difficult to analyze the shear dominant RC member.

3. The computational program is complexed, and requires a lot of computational time.

4. It is difficult to develop the equation for design from the nature of the numerical computation.

On the other hand, there are many advantages as follows.

1. Very wide application is possible to the subject of the analysis.

2. Not only the strength but also the deformation and the internal stress state can be obtained.

3. The institution of the boundary condition is easy.

These advantages outweigh the drawbacks, and the finite element method has developed into one of the most powerful shear analytical methods.

As subjects for further investigation, it will be necessary to develop the analytical model for the basic properties of reinforced concrete into the more rational one, comparing with the simplified experimental results. Not only are the analytical results of a few specimens compared with the corresponding experimental results, but also it is necessary to investigate the shear resistance mechanisms under the reversed cyclic loading by the more systematic parametric analysis, observing the internal stress state which is difficult to be grasped in the experimental studies.

It will be an important subject for the further investigation to develop the macroscopic model by which the overall structural behavior can be grasped more easily from the feedback of the analytical results by the finite element method in order to bind up the products of research with the design.

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Co-author: Prof. Norio Inoue, Associate Professor, Tohoku University (Former in Kajima Corporation) —

2. Finite Element Analysis of Reinforced Concrete Structures in Japan

2.1 Introduction

Many analytical studies of reinforced concrete members or structures have been done recently by the Finite Element Method due to the development of computers and analytical methods based on fundamental experimental data. When the general view of the studies is observed, many cases are found in which experimental results were investigated analytically, and the specific problems for reinforced concrete were made clear by the feedback between the experiments and the analyses. This approach should clarify the unknown which might be impossible to investigate only with experimental studies. Further for the buildings of nuclear power plants or offshore structures, the analytical studies are sometimes considered to be more favorable because the structures are so large and complicated, hence, experimental studies are very difficult in the adequate scale. Such thoughts are also seen in the case where the number of necessary test specimens is excessively large in accordance with the variety of loading and boundary conditions. Currently this approach is still supplemental but it should become possible to make up for experimental studies and reduce the number of necessary test specimens when the accuracy of the analyses becomes higher in future. From the view of structural design the current design formulae are established empirically from the test results obtained for each member such as columns, beams or shear walls. This sometimes causes discrepancy when a formula is used in the inapplicable range or a formula is applied as a matter of convenience to the structure which does not belong to any member under classification. For these members FEM in considered to be useful to clarify their behavior, but this approach is still thought as being supplemental.

From these backgrounds active movements have appeared to rearrange the present status and to look over the future prospects of the FEM. In Europe IABSE held a colloquium on "Advanced Mechanics of Reinforced Concrete" at Delft in 1981 [2-1] and in the United States the ASCE published the "State-of-the-Art-Report" [2-2]. In Japan the Japan Concrete Institute held the colloquium on "Shear Analysis of RC Structures" in 1982 and 1983 [2-3], where the fundamental problems of reinforced concrete were discussed especially on analytical procedures. In the colloquium the authors [2-4] presented the review of the analytical approaches by FEM and pointed out future problems concerning each basic item as follows: (a) Stress-strain relation of concrete under multi-axial stresses, (b) behavior of concrete after peak stress (c) cracking and its propagation, (d) internal shear transfer, (e) modelling of reinforcing bars, (f) bond between reinforcing bars and concrete, (g) dowel action of reinforcing bars.

In this chapter the application of FEM is reviewed in accordance with each member such as columns, beams, beam-column joints, shear walls and structures of nuclear power plants. These members have distinct characteristics from one another in the size, space and arrangement of bars, the effect of bond and the dimension in which calculations are performed. Consequently it seems wise to assort respective items to which applications are made so that the problems may be clarified. In the presentation herein only short time loads are considered excluding creep, thermal loads and impact loads. The subjects are selected from among those that are comparatively new and have distinct characteristics.

2.2 Columns

A column contains comparatively thick reinforcing bars which are arranged rather sparsely. From this specific feature the characteristics of bars including the effect of correlation with surrounding concrete, are shown remarkably well, hence the bond behavior and the dowel action become important. From an analytical view each bar can be modelled individually, and the bond behavior can be represented easily by bond links which have assumed bond-slip relation. The failure modes of columns are classified into the bending-type, the shear-type, the bond-type etc. corresponding with the shear span ratio and the quantity of reinforcement. But it is considered very difficult for only one model to simulate all of the failure mode types. As to the loading condition a column is subjected to not only lateral forces but also axial ones and besides the direction of stresses change every moment in accordance with the increase of lateral forces. This sometimes causes difficulty so that some additional means for modelling of cracking or its propagation are necessary, or in some cases the questions have to be left undecided. Furthermore in case of analyzing a column with axial forces, which is deformed in two horizontal directions like circular displacements, the analytical method is required which can treat cyclic loads in the three dimension. In this model it is worthwhile to note that the concrete confined by hoops is in 3-dimensional stress condition, and for its evaluation the 3-dimensional constitutive law is applied.

Nomura et al. [2-5] defined the yield and failure of concrete by the Mohr-Coulomb's formula and represented reinforcement by axial elements. Bond was modelled by bond links in which the strength of the bond was reduced corresponding with the distance from a crack surface. Columns under cyclic loads were analyzed, and the influence of axial forces to the shape of hysteresis was presented. In the results the area surrounded by the hysteresis loop was small comparatively (Fig. 2.1). The assumption of the hysteresis characteristics should be reconsidered. Furthermore from the study [2-6] in which the pullout of main bars at the end was represented by spring elements, the influence of the pullout was pointed out to be very important because the obtained story deflection increased 35% to 25% when this effect was considered. In considering that Okamura et al. [2-7] represented the bond behavior of reinforcement embedded in massive concrete from the strain in reinforcement vs. slip relation, this procedure seems effective for modelling the anchorage problems.

Noguchi et al. [2-8] represented concrete by an orthogonally anisotropic model and modelled reinforcement by two-dimensional elements for expressing the dowel effect (Fig. 2.2). Bond was represented by bond links in which the detailed characteristics of bond were incorporated, such as the difference of their properties after the bar yielding or the arrival of the ultimate bond strength.

Further the varied properties were assumed for the upper portion and the lower portion, and the deterioration of bond near a crack surface was given consideration. From the analyzed results the fact was pointed out that the assumed bond characteristics severely affected the distribution of the strain of main bars, the bond stress and the bond slip. In this study the detailed analyses were executed on the basis of the precise properties of each material although crack locations were predetermined to represent them by crack lines. This shows that FEM is an useful approach to clarify the influence of each factor to the behavior of reinforced concrete members.

Watanabe [2-9] investigated the shear failure mechanisms of columns caused by the splitting bond failure by considering the ultimate bond stress obtained by experiments for splitting bond failure. In the analytical model the stress-strain relation of concrete was represented by a simple bi-linear curve, and reinforcement was modelled by axial elements. Bond was represented by bond links, and cracks were modelled by crack links of which locations were established beforehand (Fig. 2.3). Furthermore from the obtained stress distribution of concrete the failure mechanism was simplified, and the macroscopic model which could represent its failure more adequately was established. In this study a microscopic model like FEM was used to create a macroscopic model. This approach seems to be the future trend.

Inoue et al. [2-10] proposed a nonlinear three-dimensional model in which the core concrete confined by hoops was represented by three-dimensional elements with the plastic theory, and reinforcement and bond were modelled by axial elements and bond links, respectively. In this model the hysteretic characteristics of each element were assumed. This approach was applied to analyze columns under cyclic lateral forces in one direction and to those which were deformed in two horizontal directions (Fig. 2.4). The obtained results showed that their three-dimensional behavior was simulated well. When analyzing a column under two-directional lateral forces, the three-dimensional FEM like this becomes necessary to investigate the effect of shear forces which cannot be estimated by a fiber model, although this analysis is comparatively difficult because the stresses change every moment in accordance with the looped hysteresis. But in the results of the

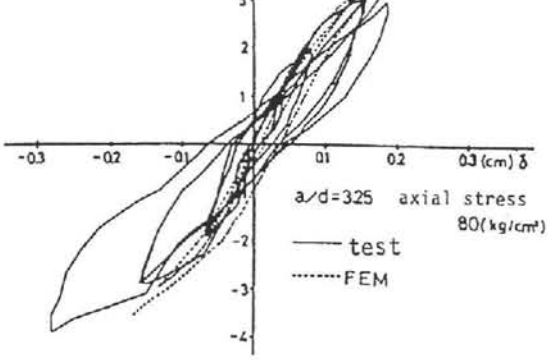


Fig. 2.1 Cyclic Load - Deflection Diagram [2-5]

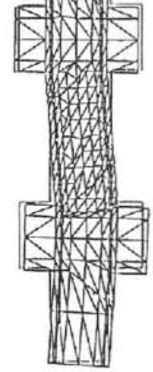


Fig. 2.2 Deformation of Column [2-8]

shear-deflection relation, the obtained area surrounded by the hysteretic loop was small comparatively. This tendency was also seen in the Nomura's analysis [2-5], so the assumption of the hysteretic characteristics require reconsideration.

A column is a member which carries vertical loads. Its failure leads directly to the collapse of the building, and so it is very important to investigate the ductility after its yielding. Especially the influence of cyclic loads should be clarified. But in many cases the analyses by FEM were executed under monotonical loading up only to the adequate strength. This tendency of loading below the strength is seen also in the analyses of cyclic loading. Further the influence by axial forces to the shear capacity of a column has not been cleared by experiments, and the ductility after the peak loads is still beyond comprehension. From now on its is hoped that a proposal will be made for the analytical model in which the ductility of a column after peak loads can be evaluated with consideration for the effect of axial forces and in which the influence of cyclic loading can be discussed.

2.3 Beams

A beam is a fundamental member which has often been analyzed by FEM to clarify the mechanisms of shear resistance in reinforced concrete. The making of analytical model is almost the same as for columns, from the structural characteristics. The authors [2-4] have already made presentations on the detailed review about the beams in past colloquium, so in this report the researches after the review are presented.

Noguchi et al. [2-11] made simple-supported beams without stirrups in which cracks were installed beforehand with the different crack depth, and measured the shear resistance of the elements after the occurrence of inclined cracks, as following: concrete, dowel action of reinforcement and aggregate interlock. In the analytical study by FEM the mechanism of each

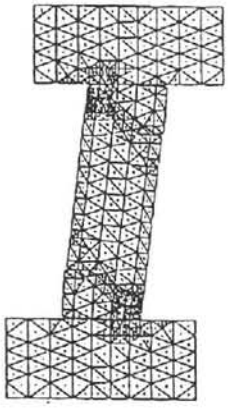


Fig. 2.3 Deformation of Column [2-9]

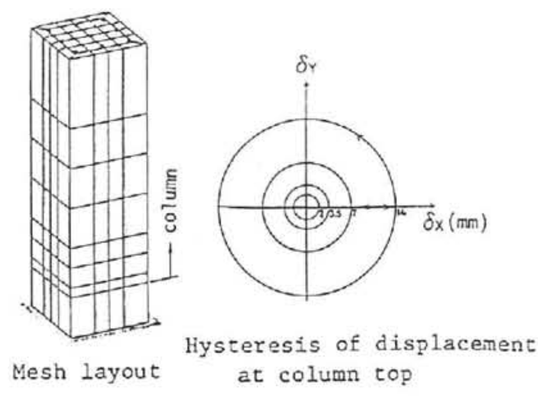


Fig. 2.4 3-Dimensional Model of Column [2-10]

factor was presented, and the obtained shear stress well explained the experimental results in the resistance ratio (Fig. 2.5). This approach seems effective to clarify the resistance role of each element in reinforced concrete by both experiments and analyses.

Kokusho and Hayashi et al. [2-12] executed the test of a beam under bending and shearing forces with prior installed cracks to investigate the bond-slip characteristics of a cracked member. From the analytical results by FEM in which the bond characteristics obtained by the tests was assumed as bond links (Fig. 2.6), it was pointed out that the bond-slip relation should be varied with the correspondence to the distance from a crack surface to simulate the distribution of the strain of longitudinal bars and bond stresses. In this approach one dominant factor for the behavior of reinforced concrete was picked out and investigated by both experiments and analyses. This is a favorable approach expected in the future.

Fukuhara [2-13] made parametric analyses of beams with prior cracks by an elastic model to grasp their mechanical behavior in the vicinity of the inclined cracks which occurred at the ultimate stage under shear forces and to gain the fundamental data for proposing experimental formulae of ultimate shear strength (Fig. 2.7). The obtained results were studied quantitatively in stresses and crack width. This elastic analysis seems to be useful when considering the fact that many specimens have to be calculated to obtain the formulae of the ultimate strength analytically.

Niwa [2-14] analyzed deep beams by a nonlinear FEM assuming smeared cracks in order to clarify the mechanism of their shear resistance (Fig. 2.8). Some parameters which affected the shear capacity were picked out, and their influence to the distribution of compressive stresses in the diagonal concrete struts was grasped quantitatively. Finally from these results a formula of the ultimate shear strength for deep beams was proposed. This approach, in which a design formula was obtained analytically by the parametric calculations, is highly significant to connect the FEM with the design.

Kokusho et al. [2-15] analyzed beams with an opening to study the effect of the reinforcement around it (Fig. 2.9). In this study two models, with and without bond, were calculated. The obtained results showed that the bond influenced the behavior of a X-shaped reinforcement largely, but it did not influence that of a ring-shaped reinforcement. This is an example that certain fundamental behaviors can be grasped by a simple approach like the elastic analysis.

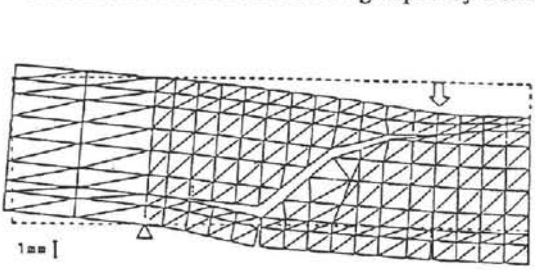


Fig. 2.5 Deformation of Beam [2-11]

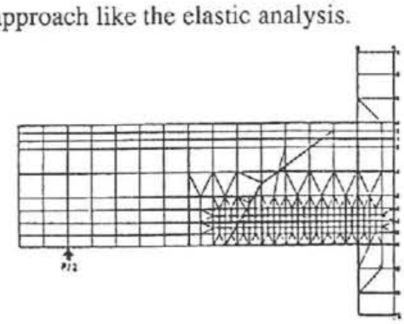


Fig. 2.6 Mesh Layout of Beam [2-12]

Obata and Katsumizu [2-16] pointed out that the applied element mesh layout affected the strength, deformation, crack distribution and stress in longitudinal reinforcement in the analysis of a simple-supported beam. In this study the variations were considered, such as the fineness or coarseness of the mesh size and the orthogonal or oblique division (Fig. 2.10).

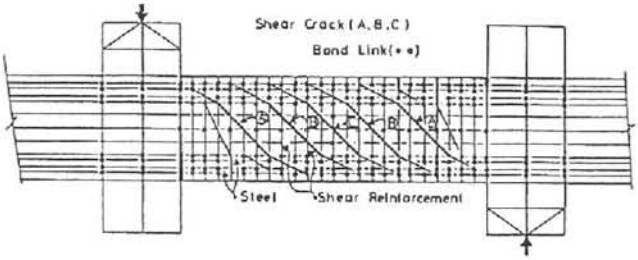


Fig. 2.7 Mesh Layout of Beam [2-13]

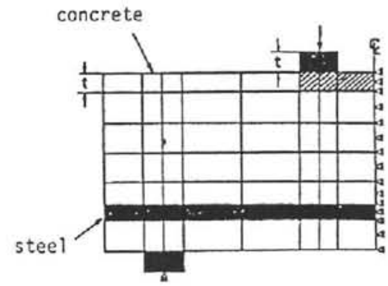


Fig. 2.8 Mesh Layout of Deep Beam [2-14]

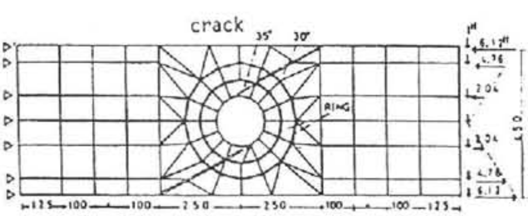


Fig. 2.9 Mesh Layout of Beam with Opening [2-15]

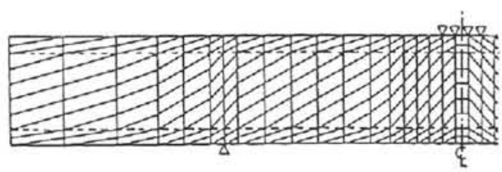


Fig. 2.10 Oblique Mesh Layout of Beam [2-16]

The FEM analyses make it possible to grasp the internal stress and strain in a beam which cannot be obtained easily by experiments. Consequently it is expected that the various failure mechanisms of a beam corresponding with the quantity of lateral reinforcement, and the shear span ratio is clarified by FEM. Furthermore by investigating the resistance ratio of each element quantitatively and comparing it with the results from a macroscopic model, it is desired that the current experimental formulae are substantiated analytically, and a more reasonable design formula is proposed in reference with the results of parametric studies by FEM. Regarding the method of modelling, the degree of minuteness in the adopted method should be considered in accordance with the degree as to how it will affect the analytical results.

2.4 Beam-Column Joints

In the recent design of reinforced concrete frames the stresses in a beam-column joint have become large comparatively in accordance with the increase of the ultimate strength of beams and columns, because it is emphasized that the strength and ductility must be secure in the beams and columns. Consequently the necessity of its study has increased, but there exist not many analytical researches. This is because it is very difficult to model the nonlinear behavior such as the cracking and crush of concrete under complicated high stress condition, and the bond slip of longitudinal reinforcement in a beam at the joint in correspondence with the deterioration of bond.

Shimodaira et al. [2-17] analyzed cross-shaped joints under low stresses (Fig. 2.11). In this model the concrete was assumed to be elastic. Cracking was represented by separating the adjoining nodal points, and the bond was modelled by bi-linear springs. The obtained results showed good coincidences with the experimental ones in the distribution of stress and strain, cracking and its propagation at the joint.

Otsuki et al. [2-18] analyzed statically indeterminate frames with joints by a nonlinear model to comprehend the influence of the difference in assumptions like the mesh layout, the stress-strain relation and failure criteria of concrete and the bond-slip relation. In this study the inclined cracks were neglected at the joint because the assumed failure mode was restricted to the yield of a beam and the bond failure of main bars (Fig. 2.12).

Ohwada [2-19] analyzed joints elastically by 3-dimensional FEM to investigate the influence of the parameters such as the shape of joints, the existence of orthogonal beams and the width of beams. Recently in another research [2-20] the behavior after cracking was studied elastically with a model, incorporating reinforcement and bond, in which cracks were installed beforehand (Fig. 2.13). In considering that the effect of orthogonal beams may decrease when they are subjected to cyclic loads up to the vicinity of their bending yield, such study in nonlinear behavior is desired.

Kamimura et al. [2-21] investigated by three-dimensional elastic analysis the influence of the eccentricity of beams jointed eccentric to a column.

Noguchi et al. [2-22] analyzed cross-shaped joints under cyclic loads, paying attention to the constitutive relation of concrete, the opening and closing of a crack and the bond property.

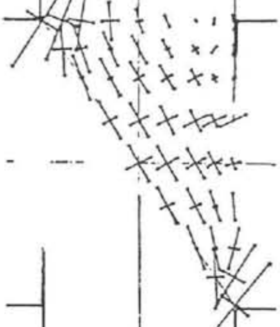


Fig. 2.11 Principal Stress Distribution in Beam-Column Joint [2-17]

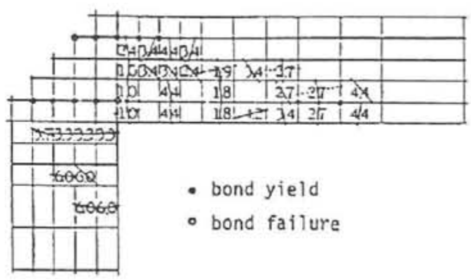


Fig. 2.12 Distribution of Crack in Beam and Column [2-18]

The obtained results indicated the experimental behavior that the deterioration of hysteretic characteristics was influenced by the shear failure of concrete and the deterioration of bond of the beam main bar in the joint (Fig. 2.14).

Hayashi and Kokusho et al. [2-23] analyzed a joint by applying assumed bond characteristics equal to that obtainable from an experiment in which the bond property of beam reinforcement was measured preponderantly within the joint. The results showed a good coincidence with the experimental ones. Especially it is an interesting indication that the deterioration of bond near a crack surface and the axial stress near the surface of a column were influential in expressing the bond properties of a beam.

Until now the number of the analytical examples on beam-column joints is small. Hereafter it is expected that the mechanisms of shear resistance are clarified by investigating analytically the change of internal stress distribution and the deterioration of bond, and that the analysis becomes useful for establishing the design method of joints. Especially the important problems are as follows; the correlation between the bond deterioration of beam reinforcement and the shear failure of a joint, the effect of lateral reinforcement in a joint, the change of the mechanisms of shear failure under cyclic loads after the bending yield of beams.

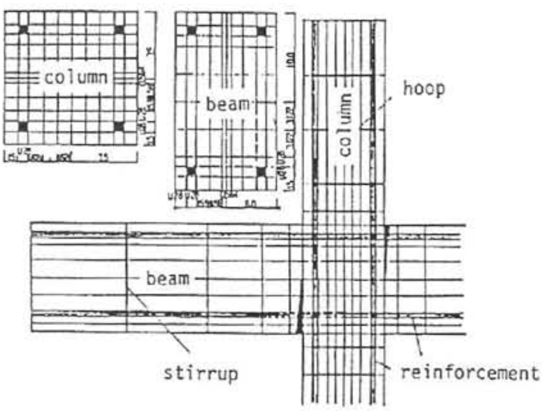


Fig. 2.13 3-Dimensional Mesh Layout of Beam-Column Joint [2-19]

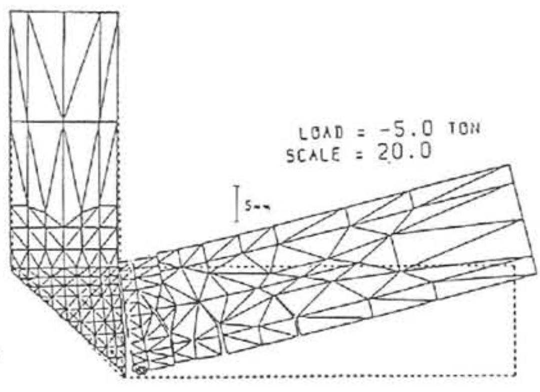


Fig. 2.14 Deformation of Beam-Column Joint [2-22]

2.5 Shear Walls

Generally in a shear wall many number of reinforcing bars are placed. Due to this feature it is rather difficult to model each reinforcement individually and to represent bond by bond links. Consequently in many cases the average behavior of a shear wall was studied by using a distributed representation of the reinforcement. In this modelling, there are cases whereas the bond is considered as perfect, but better results were obtained when evaluation was made of tension stiffening for the bond behavior. But for the investigation of bar arrangement or the bond behavior in detail, the modelling methods used in the analysis of a column or a beam, in which a bar is modelled by an axial element and bond represented by bond links, are sometimes adopted. As to the purpose of the analysis the adopted modelling method retains certain specific feature corresponding with the subjects of the investigation, such as, to evaluate experimental results of the total behavior of a shear wall, or the microscopic behavior of the crack spacing and the bond characteristics. In the study to design a shear wall there are many problems to be investigated as follows: the adequate arrangement of reinforcing bars around an opening, the behavior of the joints in a precast wall or postcast wall, the stiffness and strength of partial walls (wing, wainscot and/or hanging portion of a wall). When these subjects were analyzed, the most respective suitable modelling method was proposed for evaluating the most noted points and the studies corresponding with its aim was executed.

Mochizuki et al. [2-24] represented the nonlinear behavior of concrete by the Darwin-Pecknold's method, and assumed the uniform distribution of reinforcement and perfect bond. By using this method shear walls with surrounding frames, under cyclic pure shear forces, were analyzed, in which a nondimensional hysteretic loop was assumed from the experimental results of stress vs. crack width relation (Fig. 2.15). This showed a simple method to simulate the cyclic behavior but some problems to be cleared remain such as generality of the hysteretic loop obtained from the specific test specimens, and the application to a shear wall in which the yield of concrete in compression was dominant.

Shirai et al. [2-25] assumed the plastic theory for concrete and the uniform distribution of reinforcement. Cracks were represented by the smeared model, but the proposed procedure was able to evaluate tension stiffening, crack spacing and width, aggregate interlock and dowel action quantitatively by using the bond theory which was based on the results of pullout tests under two axial stresses considering the perpendicular compression to the direction of reinforcement. The application to a shear wall of three stories and one span showed good results (Fig. 2.16). This approach is useful to evaluate each factor in detail although cracks are modelled by smearing.

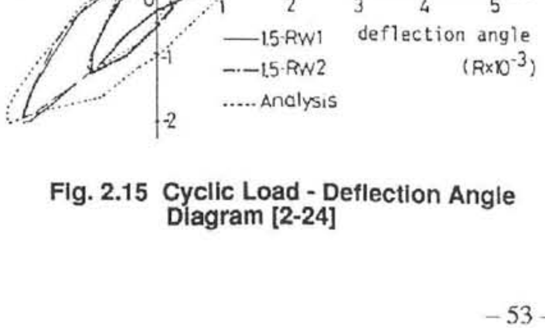


Fig. 2.15 Cyclic Load - Deflection Angle Diagram [2-24]

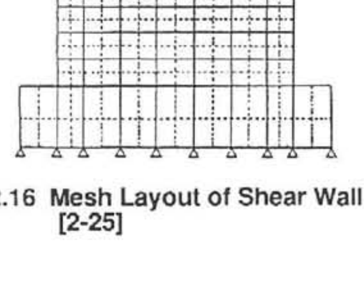


Fig. 2.16 Mesh Layout of Shear Wall [2-25]

Noguchi et al. [2-26] executed microscopic analyses of the Collins' panel tests by a discrete crack model (Fig. 2.17). In this study the dominant factors, such as strength of concrete, reinforcement ratio, yielding strength of reinforcement and loading method, were selected and the shear stress vs. shear strain relation of panels were compared in correspondence with each parameter. Finally by evaluating the influence of each parameter, the formulae to evaluate the maximum shear capacity were proposed. Here not only qualitative but also quantitative analyses were executed instead of experimental studies in which difficulty was seen to conduct the many tests corresponding to each parameter. This approach showed the useful applicability of FEM. However, panels with deformed bars should be investigated in the future because the bond characteristics might show special difference in the Collins' tests in which welded wire meshes are used. Inoue and Koshika et al. [2-27] analyzed a reinforced concrete panel under pure shear by a smeared crack model (Fig. 2.18). In this study reinforcement was modelled by axial elements and the property of bond links were assumed with consideration for the difference of bond strength corresponding with the distance from a crack surface. Furthermore the mesh dimension of the concrete was kept smaller than the crack space. From the results it was concluded that in order to simulate the crack pattern the bond strength near a crack surface should be decreased. In considering that these studies of panel tests were conducted mainly to grasp the total behavior macroscopically and to obtain the constitutive relation of a reinforced concrete panel, the meaning of a microscopic analysis by FEM can be said to exist in the study to pick up the dominant factors, which cannot be evaluated well only by experiments, and to substantiate the constitutive relation analytically.

Inoue et al. [2-28] [2-29] presented a method in which the principal stress strain relation proposed by Collins, from reinforced concrete panel tests, was assumed in compression-tension, and the orthogonally anisotropic model was assumed in compression-compression. In the applications to shear walls with different shear span ratios and reinforcement ratios, it was shown that this method could grasp the yielding propagation and ultimate strength corresponding with each parameter (Fig. 2.19). This approach had less assumptions comparatively because it was

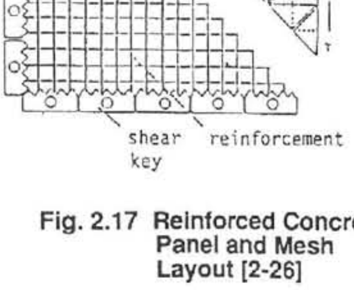


Fig. 2.17 Reinforced Concrete Panel and Mesh Layout [2-26]

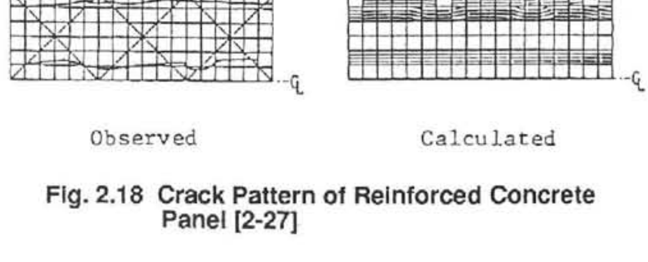
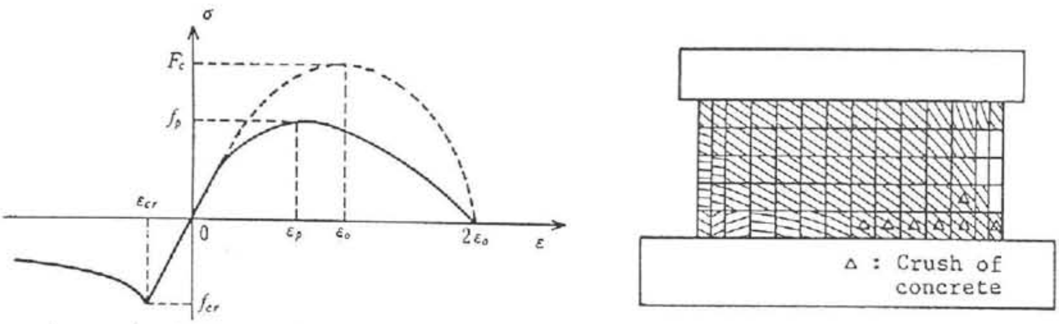


Fig. 2.18 Crack Pattern of Reinforced Concrete Panel [2-27]

more macroscopic on the basis of the test results in which reinforced concrete panels were considered to be the minimum elements. Consequently in cooperation with the iterative calculation procedure using secant modulus, the inquiry of obtained results was simple and this approach is thought to be useful for the design purpose. However, it is very important to know the applicability of the test results on which this approach is based.



Assumed stress - strain curve of cracked concrete

Crack pattern

Fig. 2.19 FEM Analysis of Shear Wall based on Collins' Theory [2-28] [2-29]

As to the study of openings, Seya et al. [2-30] analyzed shear walls with an opening by a model in which the plastic theory was assumed for concrete and reinforcement was modelled by an axial element. From the obtained stress distribution and crack pattern, it was concluded that the analyses could give the feature of the ultimate failure in the experimental results. Sotomura et al. [2-31] analyzed shear walls with many small openings by the ADINA program and presented that the obtained results showed good coincidences with the experiments regarding the shear vs. deformation relation, the loads at initial cracking and the feature of crack propagation (Fig. 2.20). Noguchi et al. [2-32] analyzed shear walls with different arrangement of reinforcement for an opening with a model in which reinforcement was modelled by axial elements and bond was represented by bond links (Fig: 2.21). The obtained results showed that the difference in the reinforcing method had much influence on the deformation and the failure behavior of shear walls.

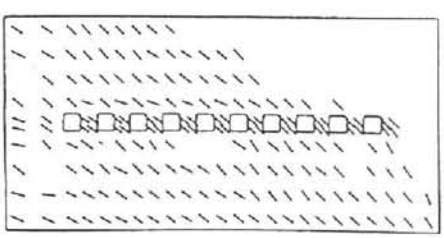


Fig. 2.20 Crack Pattern of Shear Wall with Many Openings [2-31]

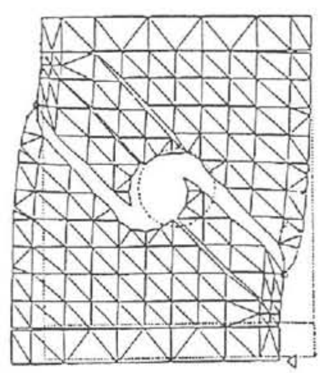


Fig. 2.21 Deformation of Shear Wall with Opening [2-32]

It can be said that in the microscopic study like this a corresponding microscopic modelling is necessary. In this study cracks were assumed beforehand by crack links at determined portion derived from the experimental results. But from the view of the structural design it will be an important point whether the location of cracks can be presumed adequately without preliminary experiments.

On the study paying attention to concrete joints, Kokusho et al. [2-33] analyzed a postcasted shear wall which was placed with dowell reinforcement to strengthen an existing building (Fig. 2.22). In this analysis the joints were modelled from the experimental results of joints with the dowel reinforcement by spring elements which were orthogonal mutually. From the analytical results in which the resistance ratio of the dowel and concrete was estimated, it was concluded that the behavior of a postcasted shear wall could be grasped if the property of the joint was given. Suenage et al. [2-34] represented the joint by an anisotropic finite element considering the friction, bond and dowel action to analyze a precasted shear wall with the horizontal joints. The application to a walled frame well represented the load vs. deformation relation and the nonlinear behavior. Higashi et al. [2-35] represented the vertical and horizontal joints of a precasted shear wall by bond link elements, and determined the horizontal stiffness from shear tests and the vertical stiffness from uni-axial compression tests. Noguchi and Muruta [2-36] estimated the stiffness and the failure criteria of the spring elements which jointed the frame and the postcasted shear wall on the basis of the latest test data of dowel and the analytical results presented by Kokusho et al. The application showed that the failure process of the joints had much influence on the total behavior of a shear wall from the study in which the dowel effect was investigated on the ultimate strength and deformability of the postcasted shear wall (Fig. 2.23).

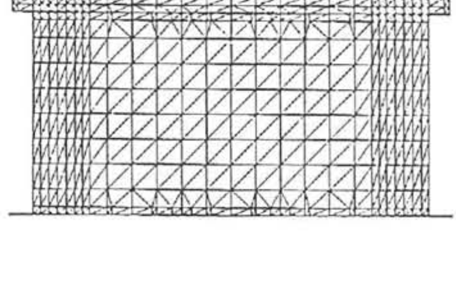


Fig. 2.22 Mesh Layout of Postcasted Shear Wall [2-33]

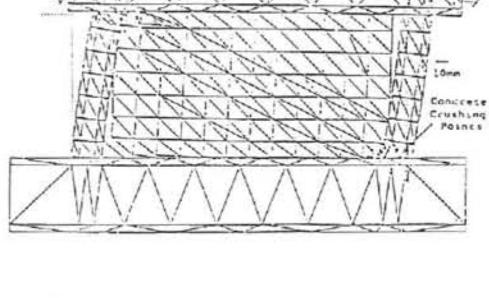


Fig. 2.23 Deformation of Infilled Shear Wall [2-36]

Nomura et al. [2-37] studied the effect of partial walls which were placed in a frame (Fig. 2.24). In the analysis the nonlinear behavior of concrete and reinforcement was evaluated by assuming the equivalent elastic stiffness. Here the initial stiffness, the yielding strength and the ultimate strength were compared with the current design formulae to investigate their applicability. Aizawa et al. [2-38] analyzed walled frames with an opening of different shapes and sizes to obtain the equivalent opening ratio and presented the applicable limit of the modelling by frames. These studies investigated the applicability of the design formulae by parametric analyses and should increase in accordance with the improvement of the analytical accuracy.

Furthermore Ono and Adachi et al. [2-39] investigated the resistance ratio of the shear wall panel and its surrounding frame in a multi-storied shear wall by a discrete crack model (Fig. 2.25). Kano et al. [2-40] studied the same problem by the smeared crack model. As to the calculation approach Arai [2-41] proposed an iterative method for monotonically increasing loading, and Mitsukawa and Baba [2-42] pointed out the usefulness of a generalized code with many options of modelling.

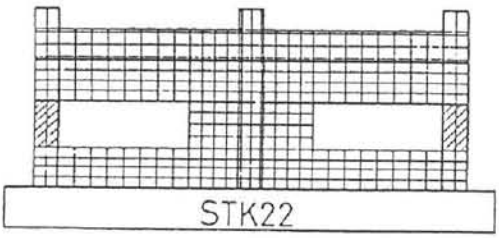


Fig. 2.24 Mesh Layout of Shear Wall [2-37]

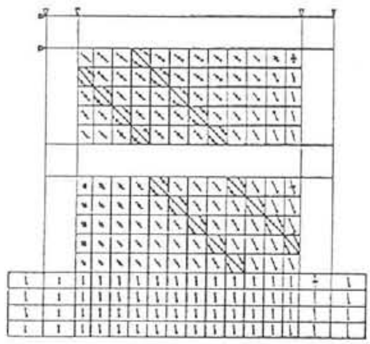


Fig. 2.25 Principal Stress Distribution of Shear Wall [2-39]

In considering that the greater parts of a shear wall are under tension-compression stresses in shear failure, it is very important in the analysis how to consider the decline of the compressive strength and stiffness in cracked concrete, which was shown in the reinforced concrete panel tests by Collins. Ueda et al. [2-52], Maekawa [2-43] and Morikawa [2-53] used methods to decrease them beforehand, and Noguchi [2-26] represented their decline by assuming the characteristics under bi-axial tension-compression stresses without deteriorating the concrete strength itself. Inoue et al. [2-28] [2-29] assumed the stress strain relation proposed by Collins as it was. This problem should be cleared urgently. About the shear slip failure observed in experiments some studies have been started like the Ueda's proposition [2-52] in which the failure was represented by assuming the limit criteria of shear strain. But this is a future problem and should be investigated to clear its mechanism and the means to represent it by analyses. Furthermore there are problems to be studied such as the change of the ultimate shear strength corresponding with the confinement of the surrounding frame and the influence induced by the difference of the following loading methods; push, push-pull and distributed loading.

2.6 Structure of Nuclear Power Plant

As to the structure of nuclear power plants there are the box-typed and cylinder-typed shear walls, and prestressed concrete reactor vessels (PCRV) among others. In the analyses they should be handled as three dimensional structures because they are subjected to out-of-plane forces besides in-plane forces. The loading condition is very complicated. A cylinder-typed shear wall under internal pressures or thermal loads can be analyzed as an axisymmetric problem and can be handled like a two dimensional problem by using solid plane elements for concrete and axial elements for reinforcement. But when subjected to non-axisymmetric forces like earthquake loads the entire structure must be considered and then in many cases it is analyzed by the model in which the out-of-plane bending is represented by layered elements, reinforcement is replaced with equivalent plates and smeared cracks are assumed. On the occasion when subjected to earthquake loads after cracking by internal pressures, the direction of principal stresses changes greatly, and new cracks occur in the aslant direction to the previous cracks. So it becomes very difficult to represent the cracks by the generally used method in which the orthogonally anisotropic model is assumed after cracking. Also, it becomes a basically complicated problem because the shear transfer between cracks is dominant. For these reasons the studies to such loading conditions are very few.

Isobata [2-44] proposed an orthogonally anisotropic model by introducing parameters concerning Young's modulus and Poisson's ratio to analyze three dimensional structures like a prestressed concrete pressure vessel (PCPV) as a three dimensional axisymmetric problem (Fig. 2.26). The application to the PCPV under internal pressures showed that not only the deformation but also

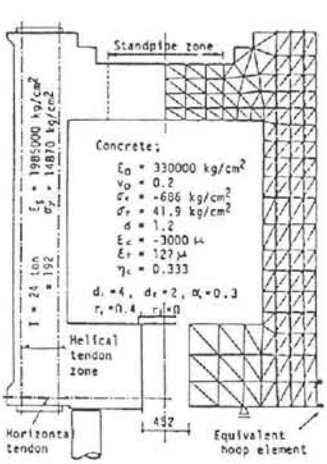


Fig. 2.26 Model of PCPV [2-44]

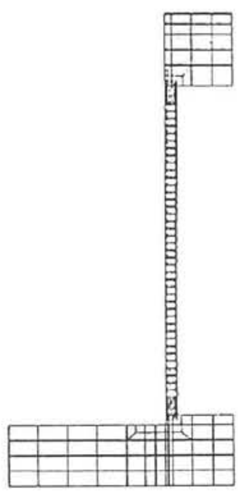


Fig. 2.27 Mesh Layout of Reinforced Concrete Containment Vessel [2-45]

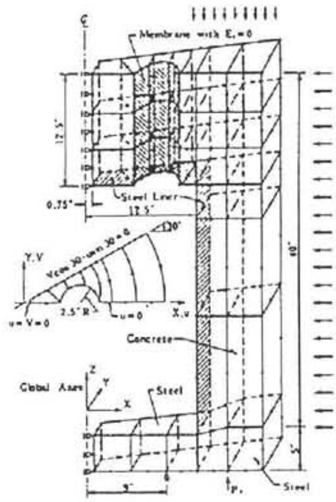


Fig. 2.28 3-Dimensional Model of PCRV [2-46]

the crack propagation could be grasped in a three dimensional structure with thick cross sections and that the obtained results simulated the experimental crack pattern.

Miyashita and Morikawa et al. [2-45] analyzed a reinforced concrete containment vessel under internal pressures by the three dimensional axisymmetric model in which the constitutive relation of concrete was defined by the plastic theory, and reinforcement and bond were represented, respectively (Fig. 2.27). This is the same approach as the two dimensional problem and makes it possible to investigate the deformation and the propagation of nonlinearity in a structure of revolution with thick cross sections like the Isobata's model.

Imoto [2-46] proposed an analytical method using three dimensional solid elements for three dimensional structures with complicated shapes like a PCRV. As to the yield condition of concrete a cracking criterion reflecting the effect of the intermediate principal stress for tension, and a generalized Drucker-Prager's criterion for compression were adopted to precisely represent the three dimensional stress state. The application to a PCRV under internal pressures showed a good coincidence with the observed deformation and yielding propagation (Fig. 2.28). In future the clarification of the applicability to the stress strain relation of plain concrete under triaxial stresses is desired, although this is a problem in experiments.

Ohuchi et al. [2-47] analyzed a cooling tower under horizontal forces by a layered model assuming that the in-plane stiffness of concrete was obtained from the plastic theory and reinforcement was replaced with the equivalent plates (Fig. 2.29). The obtained results simulated the experimental ones on the deformation, the crack pattern and the strain in reinforcement. But it was pointed out that the observed shear failure at the throat of the cooling tower could not be interpreted by the analysis although its possibility was shown. The clarification of this failure is a very important problem pertaining to shear walls in general.

Fujita et al. [2-48] analyzed a cylindrical shear wall under internal pressures and a box-typed shear wall under lateral forces by the layered model using the orthogonally anisotropic theory

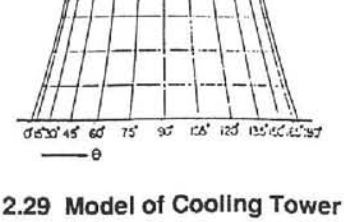


Fig. 2.29 Model of Cooling Tower [2-47]

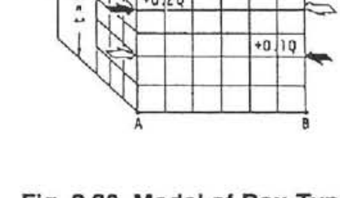


Fig. 2.30 Model of Box-Typed Shear Wall [2-48]

proposed by Isobata (Fig. 2.30). Obtained results showed good coincidences with the observed ones on the deformation, the stress distribution and the strain distribution of reinforcement.

Imoto et al. [2-49] analyzed a prestressed concrete containment vessel (PCCV) using the Ohuchi's method and investigated the method of estimating the bending and shear stiffness for the lumped mass models, which have been used widely in earthquake response analyses (Fig. 2.31). This was conducted by separating the rotation, the bending deformation and the shear deformation from the FEM results to obtain each nonlinear characteristic. This approach seems to be very useful for obtaining the necessary load-deflection curve in the executing dynamic analyses.

To obtain the load-deflection curve of box-typed shear walls, Inada et al. [2-50] made parametric studies with the Fujita's layered model using the orthogonally anisotropic theory and investigated the influence of each parameter such as the reducing ratio of shear rigidity, the shear span ratio, the reinforcement ratio, the axial stress and the loading method (Fig. 2.32). Furthermore the shear stress vs. shear strain relation, which was obtained by separating the bending and shear deformation from the analytical results, was simplified to the trilinear curves and compared with the current formulae. In considering that there are not many experiments of box-typed shear walls and that it is difficult to generalize the load-deflection curve only with the test data, this is a useful approach to fill up the insufficient data.

Shiraishi et al. [2-51] analyzed a reinforced concrete cylindrical shell and a prestressed concrete one under horizontal forces with the layered model using the Darwin-Pecknold's for the constitutive relation of concrete. From the results it was concluded that the deformation, the strain in reinforcement and the crack pattern were well grasped and that the induced prestresses caused the increase of ultimate strength and the decrease of the ductility. But the observed shear failure could not be simulated although the obtained shear distribution showed possibility.

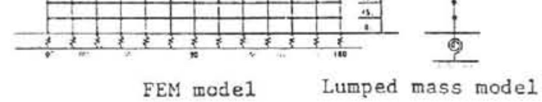


Fig. 2.31 FEM Model and Lumped Mass Model of PCCV [2-49]

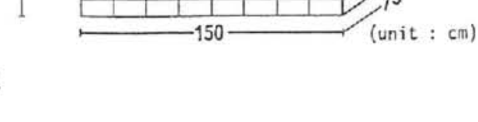


Fig. 2.32 Model of Box-Typed Shear Wall [2-50]

Ueda et al. [2-52] defined the constitutive relation of concrete used in the layered model with reference to the experimental results of the reinforced concrete panel tests by Collins (Fig. 2.33). That is, it was assumed that the stiffness and strength of concrete in the parallel direction to a crack after cracking were reduced in tension-compression region and that the stiffness of concrete dropped to zero when the shear deformation reached the assumed limit value. The former represented the deterioration of concrete after cracking and the latter represented the shear slip failure by shear strain. These assumptions are very important and should be cleared urgently as a general problem of shear walls, in considering that the greater parts of the wall are in the tension-compression stresses when shear failure occurs. The applications to a cylindrical shell and a box-typed shear wall showed good agreements with the experimental results on the load vs. deformation relation and the failure pattern.

Morikawa [2-53] analyzed reinforced concrete cylindrical shells under lateral forces with internal pressures or thermal loads with the layered model assuming the plastic theory for concrete (Fig. 2.34). To represent this complex loading condition, it was proposed that the aggregate interlock after cracking in two directions was modelled precisely assuming the equivalent shear rigidity and that the previous cracks in two directions closed and an inclined crack occurred newly when the principal stresses in the inclined direction reached the assumed cracking criterion. The obtained results indicated good comprehension of the experimental behavior although the shear slip failure could not be simulated. This approach seems to be useful when the loading direction greatly changes.

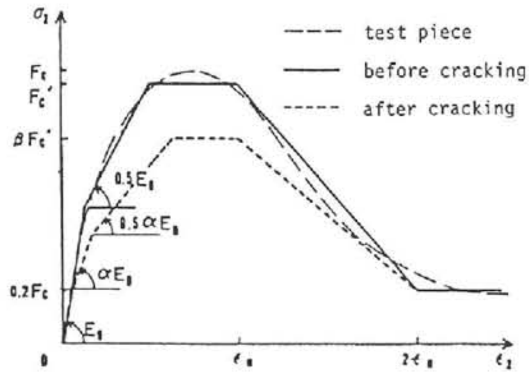


Fig. 2.33 Assumed Stress - Strain Curve of Cracked Concrete [2-52]

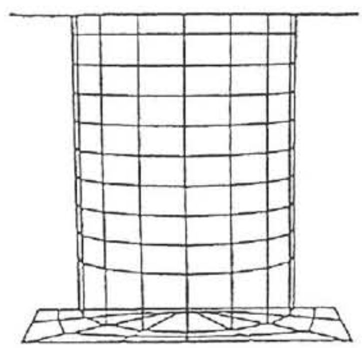


Fig. 2.34 Model of Cylindrical Shell [2-53]

The structures of a nuclear power plant are generally very stiff, but in many cases their dynamic studies are demanded due to their importance. In such analyses the adequate load-deflection curve has to be assumed and the certain suitable formulae based on the experimental studies are used. But until now there are not enough experimental data and so the FEM analyses possess expectation to fill the insufficient points. Furthermore because the structures have in general very complicated shapes and are under complicated loading combination due to their functions, the current design formulae might not be applied adequately in some cases. These reasons make the necessity of the FEM analysis very high. Especially in considering that the confirmation has become necessary lately of their safety on the nonlinear behavior in the worst earthquakes besides the allowable design, the FEM application to these structures may be the most popular and useful. From the experimental view, as the actual structure is so huge, the test specimens become very small comparatively. Namely, in many cases the scale is reduced to approximately 1/30. Consequently the scale effect should be studied by the analyses on the basis of the material property besides experimental investigations.

2.7 Conclusions

In this chapter a description was presented for the respective structural specific features. As to the future direction to proceed it depends on each researcher's opinion whether the development of computing program codes being applied to each structural feature is a better approach or not. But the authors think it is preferable to solve structural members, such as columns, beams, beam-column joints and shear walls, by one program. Because they are made from the composite material of reinforcement and concrete with not such difference in size, and they could be calculated by the same model if it was assumed on the basis of the true material property. Furthermore such possibility of the FEM analyses is its strong point over the macroscopic analyses. However, as each member has its own structural feature it seems effective to use a general code in which a suitable option can be selected corresponding with the following points; the discrete crack or the smeared crack, modelling reinforcement by an individual element or by a distributed element, representing the dowel action and the aggregate interlock by a discrete model or by an distributed model. Furthermore when calculating the whole structure it will be necessary to develop a more macroscopic model on the basis of the test results of fundamental members like the Collins' panel tests or the analytical results obtained by the microscopic FEM analyses. In these analyses it is desirable that the better modelling is proposed on the basis of the latest knowledge concerning the dominant factors to the nonlinear behavior of reinforced concrete, and that the FEM analyses can contribute to the clarification of actual phenomena and are applied to the actual design by promoting their usefulness.

Finally in considering that similar analytical results are obtained in some cases even though researchers use rather different models individually, it is desired that they should exchange their knowledge and make a comparison with each other without keeping it on the personal basis. Such opportunities are desired not only domestically but also on a world-wide basis.

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Co-author: Prof. Norio Inoue, Associate Professor, Tohoku University (Former in Kajima Corporation) —

3. Finite Element Analysis of Shear Behavior of RC Members with High Strength Materials

-Panels, Shear Walls, Beams, Columns and Beam-Column Joints-

3.1 Introduction

In this study, reinforced concrete (RC) structural members with high strength concrete and reinforcement were analyzed using nonlinear finite element (FEM) in "FEM WG": Working Group of Constitutive Equations and Finite Element Method (FEM) (Chairman: Prof. H. Noguchi, Chiba University), in Sub-Committee on High-Strength Reinforcement (Chairman: Prof. S. Morita, Kyoto University) in New RC Project entitled "Development of Advanced Reinforced Concrete Buildings Using High-Strength Concrete and Reinforcement (Chairman: Emeritus Prof. H. Aoyama, the University of Tokyo), started in 1988. Most of the object specimens in the analysis were tested in Sub-committee on Structural Performances (Chairman: Prof. S. Otani, the University of Tokyo) in New RC project.

The constitutive models for FEM analysis of high strength RC structures were derived from the systematic basic experiment which was carried out in the "FEM WG". In the analysis, the emphasis of the investigation was laid on the shear behavior of RC members with high strength materials in order to make good use of the merit of FEM. The shear performance of RC members with high strength materials was also compared with that of RC members with ordinary strength materials. The five kinds of RC members: panels, shear walls, beams, columns, and beam-column joints, were analyzed in order to verify the applicability of FEM programs on the structural members of buildings.

Main items of the investigation in this study were as follows:

1. Modeling of nonlinear constitutive rules of high strength materials and its implementation to several FEM codes, including a platform program. The FEM analytical program, "FIERCM", which was developed by Prof. M. P. Collins, Dr. N. J. Stevens and Prof. S. M. Uzumeri in the Univ. of Toronto [3-1], was modified for high strength materials and used as a platform program. The original FEM codes developed in several universities, institutes and construction corporations were used for the comparisons and verifications.
2. RC members with high strength and ordinary strength materials were analyzed using several FEM codes as shown on Table 3-1 and the reliability of the program codes were investigated from the comparative analyses.
3. The shear strength and deformation of RC members with high strength materials were investigated by FEM parametric analyses using several programs including a platform program.
4. Application of FEM analysis on the structural design of New RC building structures. (Analysis of a large-scaled box column of a New RC boiler building in a steam power plant using high strength materials.)

5. Guideline for the nonlinear FEM analysis of RC members with high strength materials. This guideline especially gives the know how and instructions for the nonlinear FEM analysis of RC members with high strength materials for design engineers and experimental researchers.

In this paper, the outline of the research results performed by the "FEM WG" was introduced by putting the emphasis on the above mentioned items No. 2 and No. 3.

3.2 Analytical Models for High-Strength Materials

3.2.1 Concrete

3.2.1.1 Stress-Strain Curves of Concrete

As for the main characteristics of the high strength concrete, the compressive stress-strain curve is nearly straight at the upward portion and the strength decay after the peak is remarkable as shown in Fig. 3- 1. The equation proposed by Fafitis and Shah [3-2] is commonly used for the high strength concrete.

The uniaxial compressive strength obtained form the test of cylinders is used for the analysis. The splitting strength obtained form the test of cylinders is used in the analysis of beams, columns and beam-column joints, but a square root of the compressive strength (unit: kgf/cm^2) is used in the analysis of panels and shear walls. Because the splitting strength is too large for panels and shear walls as compared with the previous test results.

The failure criteria on the bi-axial or tri-axial principal stress planes should be considered adequately as shown in Fig. 3- 2. The modeling of the confinement effects by lateral reinforcement is important especially for the analysis of beams, columns and beam-column joints.

The models proposed by Kent-Park [3-3][3-4] and Sakino in New RC WG for Confined Concrete are used in this study. While these models were proposed for the flexure problems, the models are also used in the analysis of flexural shear problems for convenience' sake. In high strength concrete, the confinement effects are not remarkable unless high strength reinforcement is used for the lateral reinforcement.

3.2.1.2 Tension-Stiffening Characteristics

Tension-stiffening model is commonly used for in the analysis for the representation of bond in the concrete between neighboring cracks. It was observed that the stress-decay after cracking was remarkable in high strength concrete and also in the specimens with large amount of reinforcement in the previous RC panel test performed by K. Sumi, Hazama-gumi Corp. in New RC FEM WG.

3.2.1.3 Compressive Strength Reduction Factor

It was pointed out that the strength and stiffness parallel to cracks of RC panels under tension and compression are lower than those of uncracked concrete under uniaxial compression in the previous panel test [3-5]. Therefore, in the analysis, the reduction factor was multiplied on the uniaxial compressive strength. It was pointed out that the reduction factor for the compressive strength of high strength concrete is smaller than that of ordinary strength concrete from the panel test. The reduction factors used for this study are shown in Fig. 3- 3. These factors were derived from the previous basic test performed by K. Sumi, Hazama-gumi Corp. in New RC FEM WG.

3.2.1.4 Shear Stiffness Reduction Factor

The shear stiffness reduction factor is used for the representation of shear transfer across a crack. This factor is considered to decrease according to the crack width like Al-Mahadi's model [3-6]. The shear stiffness reduction factor of high-strength concrete is also considered to decrease because of a crack passing though aggregates. As there is not much data on the reduction factor on high strength concrete, the factor in accordance with the ordinary-strength concrete is used.

3.2.2 Reinforcement

The multi-linear or bi-linear model is used for the stress-strain curve of reinforcement. It should be noted that as for high strength reinforcement, the length of yielding plateau is shorter and the yielding ratio (yielding strength/tensile strength) is larger than those in ordinary strength reinforcement.

3.2.3 Bond

The bond stress-slip relationships are necessary for bond-link elements, but it is not easy to decide the relationships because of the effects of cover thickness, concrete strength, a position from a crack and so on. The relationships are decided in reference to the previous proposals [3-7] and the experiences of the analysis.

There is a tendency that the bond strength and stiffness of high strength concrete is proportional to σ_B or $\sigma_B^{2/3}$ (σ_B: compressive strength) as compared to those of ordinary strength concrete.

The research results obtained in the Working Group "Bond and Anchorage" (Chairman: Prof. R. Tanaka, Tohoku Univ. of Technology) in Sub-Committee on high strength reinforcement in the New RC project are useful and discussed in this study.

3.3 Comparable FEM Analyses of RC Members with High Strength Materials

3.3.1 Beams, Panels and Shear Walls

Comparable FEM analyses of RC beams, panels and shear walls were carried out using several FEM computer codes including a platform program, "FIERCM" (Stevens) [3-1] in order to verify the constitutive rules for RC with high strength materials.

The total number of analyzed specimens were 48.

1) Beams: 20 specimens
   - Ordinary strength: 4 specimens, selected specimens by JCI Committee on Shear Strength of RC Structures [3-8]
   - High Strength: 16 specimens
     New RC (PB, B series) tested by Prof. F. Watanabe, Kyoto Univ.
     ASB Series tested by Prof. H. Noguchi, Chiba Univ.
2) Panels: 12 specimens
   - High strength: New RC (Panel series) tested by Mr. K. Sumi, Hazama-gumi Corporation
3) Shear Walls: 16 specimens
   - Ordinary strength: 2 specimens, JCI selected specimens
   - High strength: New RC (NW series) tested by Prof. T. Kabeyasawa, Yokohama National Univ.
     New RC (No. 1 - No. 8) tested by Prof. Y. Kanoh, Meiji Univ. and Japan National Land Development Corporation

The FEM analyses were carried out by the following seven universities, institute and corporations.

a) Chiba University, Prof. H. Noguchi
b) Nihon University, Prof. N. Shirai
c) Building Research Institute, Dr. H. Shiohara
d) Ohbayashi-Gumi Ltd., Mr. K. Naganuma
e) Hazama-gumi Corporation, Mr. K. Sumi
f) Fujita Corporation, Mr. K. Uchida
g) Meiji University, Mr. H. Takagi

As each example of the comparable FEM analyses, finite element idealization, analytical and experimental results and crack patterns are shown in Figs. 4 to 6 for a beam, a panel and a shear wall, respectively. The detailed investigation of the comparable analytical results are omitted in this paper.

The main problems which were pointed out for the constitutive laws from the comparable analyses are as follows,

1. The confinement effects by high strength lateral and longitudinal reinforcement on the ultimate shear strength of beams with high strength concrete were more remarkable than those in beams with ordinary strength materials.
2. Shear transfer effects by dowel action of reinforcement across a crack were considered to be relatively large when longitudinal and lateral reinforcement ratio were high in beams.
3. The analytical results of panels and shear walls gave a good agreement with the test results. When the failure mode is reinforcement yielding type, the modeling of the stress-strain curves of reinforcement is important, and it is desirable that the analytical curve is resemble to that obtained from the material test. When the failure mode is concrete compressive failure type, the modeling of the compressive strength reduction factor is important. As for the reduction factor, the analytical results where the effects of concrete strength and tensile principal strain were considered, gave a better agreement than those where only the effect of tensile principal strain was considered. As the analytical compressive strength reduction factor was based on the panel test, it is considered that the reduction factor is larger in beams and columns, the thickness of which is relatively larger than that of panels.

3.3.2 Columns

3.3.2.1 Analysis of Columns -1

The six short columns ($b \times D$ = 20cm × 20cm, a/D = 1.0) of NSK series with high strength concrete, σ_B = 58MPa and high strength reinforcement, σ_B = 735MPa, tested by H. Noguchi in Chiba Univ. were analyzed by H. Noguchi and A. Nimura in Chiba Univ. using their original FEM code. The parameters in the test were loading method (reversed cyclic and monotonic), lateral reinforcement (p_w = 0.3, 0.6, 1.8%), axial stress ratio ($n = \sigma_0 / \sigma_B$ = 0, 0.3, 0.6). The failure mode in the test was shear compression failure. Finite element idealization of the specimen is shown in Fig. 3- 7.

In order to investigate the confinement effect, specimen NSK-7 with high ratio of reinforcement, p_w = 1.8% was analyzed using a plain concrete model, Kent-Park model [3-3] and modified Kent-Park model [3-4], as shown in Fig. 3-8. The improvement of the strength decay after a peak is considered in Kent-Park model, and the increase of the strength and the improvement of the ductility are considered in the modified Kent-Park model. From Fig. 3- 8, it is indicated that the analytical results by the Kent-Park and modified Kent-Park models gave a better agreement with the test results as compared with that by plain concrete model. It is considered that the confinement effects of lateral reinforcement on core concrete gave an increase of the shear strength of the column.

As shown in Fig. 3-9, the analytical results under monotonic loading gave a good agreement for the shear strength with the test results of specimen NSK-2 under monotonic loading in the test of specimen NSK-1. The shear strength under reversed cyclic loading was lower than that of NSK-2 under monotonic loading.

The analytical results for parameters of lateral reinforcement and axial stress ratio are shown in Figs. 10 and 11, respectively, as compared with the test results. The analytical results gave a similar tendency with the test results.

3.3.2.2 Analysis of Columns -2

JCI four selected specimens with ordinary strength materials and eight specimens tested by Prof. K. Minami in Fukuyama University in New RC project with high strength materials were analyzed by Dr. K. Naganuma, Ohbayashi-gumi Corp., using their original FEM code, "FINAL".

The analytical results of columns with ordinary strength materials gave a good agreement except bond splitting failure type specimen No. 3 and specimen No. 4 with high ratio of lateral reinforcement. As shown in Fig. 3-12, the plane stress analytical results of specimen No. 4 where the confinement effect is not considered gave a lower maximum strength than the test result. The previous three-dimensional analytical results by K. Naganuma et al. [3-9] gave a good agreement with the test result. This indicated that the confinement effect is necessary to be considered in the analysis of the columns with high-ratio of lateral reinforcement.

The analytical results of New RC columns, with high-ratio of lateral reinforcement with ties and sub-ties, p_w=1.19%, gave lower strength and smaller deformation as compared with the test, as shown in Fig. 3-13. This tendency is remarkable in the case of high-axial stress ratio, $n = \sigma_0 / \sigma_B = 0.3$. If the lateral reinforcement ratio is high, it is considered that the confinement effects increase. In this condition, three-dimensional FEM analysis or plane stress FEM analysis in which the confinement effects are considered is necessary.

3.3.3 Analysis of Interior Column-Beam Joints

Three joint shear failure type specimens, OKJ-1,3,6 and two bond deterioration type specimens, MKJ-1,3 were tested and analyzed by H. Noguchi and T. Kashiwazaki in Chiba University, using their original FEM code. This test was supported in New RC Working Group, "Beam-Column Joints and Frames" (Chairman: Prof. S. Nomura, Tokyo University of Science).

The main parameter was concrete compressive strength σB: 55MPa, 71MPa, 109MPa for OKJ=1, 3, 6 and 86MPa, 100MPa for MKJ-1, 3, respectively. The shear reinforcement ratio in the joint, pw, was 0.54%. The dimension and finite element idealization are shown in Figs. 14 and 15, respectively.

The analytical results on the joints shear stress-concrete strength are shown in Fig. 3- 16, as compared with the test results and previous other test data for from ordinary to high strength concrete.

The analytical maximum joint shear stresses of the joint shear failure type specimen, OKJ-1, 3, 6 located on the area of from $5 \times \sqrt{\sigma_B}$ to $6 \times \sqrt{\sigma_B}$ (Unit: kgf/cm^2) [$1.57 \times \sqrt{\sigma_B}$ to $1.88 \times \sqrt{\sigma_B}$ (Unit: MPa)], and they gave a good agreement with the test results.

The analytical story shear force-story displacement relationships of specimens OKJ-1, 3, 6 and MKJ-1, 3, shown in Fig. 3-17 gave good agreements with the test results for both joint failure type, OKJ series and bond deterioration type, MKJ series.

The analytical deformation and crack pattern of specimens, OKJ-1, MKJ-1 at the maximum strength is shown in Fig. 3-18, as compared with the crack pattern of the test results. In the analysis of specimen, OKJ-1, the joint shear compression failure occurred after the compression failure of a beam. This was corresponding to the test results. In the analysis of specimen, MKJ-1, beam flexural yielding occurred, and an opening of the crack of a beam at the face of a column became wide because of the increase of slip out of beam longitudinal reinforcement. In the test, the joint shear compression failure was observed under the large deformation, story rotation angle $R_s = 1/33$rad., after beam flexural yielding. But in the analysis this was not observed.

The analytical principal stress distributions of specimens, OKJ-1, MKJ-1 at $R_s = 1/200$rad. and $R_s = 1/33$rad. at the maximum strength are shown in Fig. 3-19. Though the compressive principal stress flew widely in the joint like a fan for both specimens at $R_s = 1/200$rad., the width of the compressive strut in the joint decreased and the compressive stress concentrated along the diagonal line in the joint at $R_s = 1/33$rad. This phenomenon was remarkable in the specimen, MKJ-1, in which the bond deterioration after the beam flexural yielding was dominant.

3.4 FEM Parametric Analyses of RC Members with High-Strength Materials

3.4.1 Beams

3.4.1.1 Analysis of Test Specimens

The five RC beams, ASB-1 - 4, 6 ($B \times D$ = 20cm × 30cm, a/D = 2.33) with high-strength materials tested by H. Noguchi and A. Amemiya in Chiba University were analyzed by H. Noguchi and N. Kobayashi, including parametric analyses using their original FEM code.

The shear reinforcement ratios of the parameter were designed according to AIJ Guideline [3-10] using a concrete strength reduction factor in the draft of CEB Model code 1990, as shown in Fig. 3-20. The finite element idealization is shown in Fig. 3-21. The symmetrical condition around a center point was used. The analytical shear force-story displacement relationships gave a good agreement with the test results except the specimen ASB-3, as shown in Fig. 3- 22.

The analytical ultimate shear strength and the amount of shear reinforcement relationships are shown in Fig. 3-23, where the comparisons with the test results and the calculated results by the "A method" of AIJ Guideline using AIJ equation for the compressive strength reduction factor:

$\nu \times \sigma_B = \sigma_B \times (0.7\text{-}\sigma_B/2000))$, σ_B: concrete strength (kgf/m^2).

The analytical and calculated results gave a slightly conservative results. But it should be noted that the yielding of shear reinforcement and the compressive failure of the strut concrete were assumed in the "A method", but the yielding of the shear reinforcement was slightly observed in the FEM analysis.

3.4.1.2 FEM Parametric Analyses of Beams for the Amount of Shear Reinforcement

1) The Effects of Shear Reinforcement Ratios

The analytical shear strength and shear reinforcement ratio relationships are shown in Fig. 3-24, compared with the calculated results by the "A method" of AIJ Guideline. Here, the amount of beam longitudinal reinforcement was assumed to be large in order to avoid the flexural yielding. When p_w = 0.6%, the yielding of the shear reinforcement was remarkable, and the concrete crushing was not observed. When p_w = 0.8%, 1.2%, the yielding of shear reinforcement was seldom observed and the concrete compressive failure at the edge was observed at p_w = 1.2%. When p_w = 1.8%, 2.4%, 3.0%, the yielding of shear reinforcement was not observed, and the compressive failure of concrete strut occurred.

2) The Effects of Concrete Confinement Models with a Constant Value of $p_w \times \sigma_{wy}$

The FEM parametric analysis were carried out setting the value of $p_w \times \sigma_{wy}$ as a constant value for the specimen ASB-2. The Sakino model proposed in New RC WG on Confined Concrete and the modified Kent-Park model [3-4] were used for the confinement effects on the stress-strain curves by shear reinforcement. The comparisons of the stress-strain curves by the two confinement models are shown in Fig. 3- 25.

The stress-strain curves by the New RC Sakino model and the modified Kent-Park model are shown in Fig. 3-26. The analytical shear force-deformation curves are shown for both models in Fig. 3-27. The analytical results by the modified Kent-Park model gave a difference for the ultimate shear strength according to the shear reinforcement, but there are almost no differences for the ultimate shear strength by Sakino model.

3.4.2 Columns

The column specimens which were tested by H. Noguchi and already mentioned in 3.3.2.1 were analyzed by H. Noguchi and A. Nimura in Chiba University, using their original FEM code. The parameters were shear reinforcement ratio: P_w = 0.3%, 0.6%, 1.2%, 1.8%, and axial stress ratio: $n = N/{}_cN_u$ = 0, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.8, 1.0, where ${}_cN_u$ is axial strength considering longitudinal reinforcement.

The analytical shear strength-axial stress ratio relationships as a parameter of shear reinforcement ratio are shown in Fig. 3- 28, compared with the test results. From the analysis of RC columns with ordinary strength materials performed by A. Zhang and H.Noguchi [3-11], it was reported that the increase of shear strength is dominant in the case of low shear reinforcement ratio according to the increase if the axial stress ratio as compared with high shear reinforcement ratio.

As for the high strength materials, there was so much tendency that the shear strength increased in the case of low shear reinforcement ratio, according to the increase of the axial stress ratio. In this case, the increase of the shear strength according to the increase of the shear reinforcement ratio was remarkable even for the high axial stress ratio. The analytical results gave reasonable agreements with the test results for this tendency. It is considered that this tendency is depend on the confinement effect of shear reinforcement on core concrete.

The analytical shear strength-shear reinforcement ratio relationships as a parameter of axial stress ratio are shown in Fig. 3-29. Though the increase of the shear strength according to shear reinforcement ratio becomes a little blunt in the case of high axial stress ratio, this tendency is not remarkable than reported in ordinary strength concrete.

FEM parametric analysis of columns with high strength materials was also carried out by K. Naganuma in Ohbayashi-Gumi Ltd. The basic column specimens were mentioned in 3.3.2.2. The parameters were shear reinforcement ratio and axial stress ratio, $N/BD\sigma_b$. The analytical shear strength-shear reinforcement ratio relationships are shown in Fig. 3-30. From this figure, the increase of the shear strength is large as the axial stress ratio is small. When the axial stress ratio is larger than 0.45, the effect of the difference of shear reinforcement ratios on the shear strength is small. The analytical shear strength and axial ratio relationships are shown in Fig. 3- 31. It is shown in this figure that the increase of the shear strength is remarkable as the shear reinforcement ratio is small.

In this analytical model, the confinement effects by shear reinforcement on core concrete are not considered. As mentioned in 3.3.2.2, this analysis gave a lower strength than that in the test results in the case of higher ratio of shear reinforcement, $P_w = 1.19\%$, under high axial stress, $n = 0.30$.

The analytical shear strength-shear reinforcement ratio relationships and the analytical shear strength-axial stress ratio relationships are shown in Figs. 32 and 33, respectively. The calculated strength by "A method" and "B method" in AIJ Guideline and the modified Arakawa's equation [3-12]. The modified Arakawa's equation gave a similar tendency for the increase of the shear strength depending on the axial stress ratio, but the analytical strength was higher than that in the modified Arakawa's equation.

The analytical shear strength located between "A method" and "B method" in AIJ Guideline [3-10] based on the equation in the draft of CEB model code [3-13] for the axial stress ratio less than n = 0.3. The analytical strength was nearly corresponding to "A method" for no axial stress, and it was also corresponding to "B method" for the axial stress ratio, $n = 0.3$. The analytical strength gave a higher results than "B method" for the axial stress ratio higher than $n = 0.3$.

3.4.3 Beam-Column Joints

3.4.3.1 Effects of Joint Shear Reinforcement Ratios and Concrete Strength

The beam-column joints were parametrically analyzed for the shear reinforcement ratio, $P_w = 0, 0.09, 0.18, 0.36, 0.54, 0.9, 1.2, 2.4\%$, and concrete strength, $\sigma_B = 21, 36, 51, 65, 80, 100, 120$MPa, by H. Noguchi and S. Takezaki in Chiba University, using their original FEM code. The basic specimen, AT-4, was tested also by them, with concrete strength, σ_B = 80MPa, and the yielding strength of beam longitudinal reinforcement, ${}_s\sigma_y$ = 556MPa and the yielding strength of shear reinforcement in the joint, σ_{wy} = 804MPa.

The analytical story shear force-story displacement relationships for 4 specimens of AT series with two failure modes of joint failure and beam flexural yielding gave reasonable agreements with the test results.

1) Effects of Joint Shear Reinforcement Ratios

In the parametric analyses, the amount of longitudinal reinforcement was increased in order to obtain the joint shear strength preventing the beam flexural yielding. The analytical story shear force-story displacement relationships are shown in Fig. 3-34. As for the specimens with small amount of joint shear reinforcement, $P_w = 0, 0.09, 0.18\%$, the maximum strength is smaller than the other specimen with larger than $P_w = 0.18\%$. The analytical joint shear strength-joint shear reinforcement ratio relationships are shown in Fig. 3-35. The joint shear strength increased from $P_w = 0$ to $P_w = 0.36\%$, and nearly reached the top of strength at $P_w = 0.54\%$. Even if $P_w = 2.4\%$, the remarkable increase of the strength is not observed.

2) Effects of Concrete Strength

The analytical joint maximum shear stress-concrete strength relationships are shown in Fig. 3-36, compared with the previous test results. The analytical joint maximum shear stress increase in proportion of $\sigma_B^{2/3}$ (σ_B: concrete strength), and they are located just above the curve of $6 \times \sqrt{\sigma_B}$ (Unit: kgf/cm^2), ($1.88 \times \sqrt{\sigma_B}$ (Unit: MPa).

3.4.3.2 Effects of Column Shear Force, Bond in Beam Longitudinal Reinforcement and Beam and Column Longitudinal Reinforcement Ratios

The analysis of beam-column joints are possible for the impractical condition using FEM. The following items were investigated by Dr. H. Shiohara in Building Research Institute, using FEM code, "FIERCM" developed by Dr. N. J. Stevens et al. in the University of Toronto [3-8].

1) In the previous design philosophy of beam-column joints, it has been considered that the shear force of a beam-column joint at the joint failure, $Q_j = T + C - Q_c$ is assumed to be constant. The joint maximum strengths are compared by the analysis of specimens (Case 1 and Case 3 in Fig. 3-37) with only the difference of the distance between contra-flexural points of columns.

As a result of the analysis, the joint maximum shear strength was not constant but changed by the changes of column shear force, when the distance between the contra-flexural points of columns was changed. The summation of input moments from both side beams was constant. Therefore, the assumption that the joint shear force = $(T + C - Q_c)$ at the joint failure is constant is considered not to be adequate.

2) In order to investigate the effects of bond in beam longitudinal reinforcement through a joint on the joint shear strength, the comparisons are made between a no bond specimen and a sufficient bond specimen.

As a result shown in Fig. 3-37, in the case of no bond in beam longitudinal reinforcement through a joint, the maximum input moment from beams to the joint was about a half of that in the case of perfect bond in beam longitudinal reinforcement.

3) The effects of beam and column longitudinal reinforcement ratios, which have not been considered in the previous design, are investigated. As a result shown in Fig. 3-37, the maximum input moment and the joint shear strength increased a little as beam longitudinal reinforcement ratio increased. But the effects of column longitudinal reinforcement ratio was not observed.

3.4.4 Shear Walls

3.4.4.1 Analyses of New RC Shear Walls

Fourteen shear wall specimens with high strength materials were analyzed including parametric analyses by Prof. N. Shirai, Nihon University using the modified "FIERCM." The FEM code, "FIERCM", which was originally developed by Dr. Stevens, Prof. Collins and Prof. Uzumeri in the University of Toronto [3-8], was modified by installing the constitutive equations for high strength materials developed in "FEM WG." The object specimens were composed of the following two series,

1) NW series: 6 specimens tested by Prof. T. Kabeyasawa, Yokohama National University
2) No. series: 8 specimens tested by Prof. Y. Kanoh, Meiji University and Japan Land Development Corporation

The analytical shear strength of these fourteen specimens are compared with the test results as shown in Fig. 3-38. The errors in FEM prediction were less than 5% for NW series and less than 12% for No. series.

3.4.4.2 Parametric Analyses of Shear Walls

The parametric analyses of shear walls were carried out in order to complement the parameter zone where the test data were not available. The four parameters were changed as follows,

1) Concrete Strength: $\sigma_B = 200 - 1000 \text{kgf/cm}^2$
2) Wall reinforcement ratio: $p_w = 0.2 - 1.45\%$
3) Column longitudinal reinforcement ratio: $p_s = 1.5 - 6.25\%$ (omitted in this paper)
4) Shear span ratio: $h_w / L = 0.875 - 2.063$ where the basic specimen had the same dimension and properties as the New RC specimen, No. 3.

The calculated results by the previous macroscopic model: Shohara-Katoh model [3-14], experimental equation: Hirosawa's equation [3-15] modified from Arakawa's equation and design equation in AIJ Guideline are compared with the FEM parametric analytical results for the four parameters in Figs. 39 - 41.

1) Effects of Concrete Strength

As for the effects of concrete strength on the shear strength shown in Fig. 3-39, as the compressive strength reduction factor, ν, Nielsen's equation: $\nu = 0.7 - \sigma_B/2000$ (kgf/cm²) was adopted in the current AIJ guideline equation-1 [3-10]. CEB draft equation: $\nu = 3.68 \times \sigma_B^{2/3}/\sigma_B$ (kgf/cm²) was used for AIJ Guideline equation-2. Modified CEB draft equation by "FEM WG": $\nu = 3.68 \times \sigma_B^{2/3}/\sigma_B$ (kgf/cm²)> 0.5 was used for AIJ Guideline equation-3. The AIJ Guideline equation-3 gave the best agreement for the effect of concrete strength. The current AIJ Guideline equation-1 gave the results apart from the test and FEM analytical results as the concrete strength increased. Therefore, it is pointed out that the Nielsen's equation gave an excessive prediction for the reduction of the compressive strength of cracked high strength concrete.

2) Effects of Wall Reinforcement Ratios

The effects of the amount of wall reinforcement, $P_w \times \sigma_y$ (σ_y: yielding strength of reinforcement) on the shear strength are shown in Fig. 3-40. The AIJ Guideline-3 gave the best agreement with the test and FEM analytical results. However, all of the design equations gave an excessive prediction of the effects of wall reinforcement depend on the increase of $P_w \times \sigma_y$. Though the angle of the truss mechanism is assumed as $\cot \phi = 1$, there may be rooms for further investigation for high strength concrete.

3) Effects of Shear Span Ratios

The effects of shear span ratio on the shear strength are shown in Fig. 3-41. AIJ Guidelines equation-3 and Hirosawa's equation gave relatively good agreements with the test and FEM analytical results.

3.4.5 Panels

The estimation of shear behavior of RC panels is very important for the investigation of the shear strength and deformation of shear walls. The effects of the parameters which were not investigated in New RC panel test were parametrically analyzed. The parameters were reinforcing details: the combination of p_t and σ_y, uni-axial compressive stress and bi-axial compressive stress. Concrete strength was $\sigma_B = 70$MPa, and the failure mode was shear compression failure of panel concrete.

The analytical shear stress-shear strain relationships are shown in Figs. 42 - 44. From Fig. 3-42 for the effects of the combination of p_t and σ_y from $p_t \times \sigma_y = 20$MPa, it is indicated that the analytical stiffness after shear cracking and the maximum strength increased as the wall reinforcement ratio, pt increased, even if $p_t \times \sigma_y$ was constant. From Figs. 43 and 44, it was observed that the axial compressive stress gave contributions toward the increase of the shear cracking strength and the shear strength. This contribution was more remarkable in the case of bi-axial compression than that in the case of uni-axial compression.

3.5. Concluding Remarks

In this paper, the principal research fruits were introduced. As for the high strength materials of reinforced concrete, the modeling of the stress-strain relationships, tensile strength, the compressive strength reduction factors of cracked high strength concrete, tension stiffness, shear modulus reduction factors of cracked high strength concrete and bond characteristics were discussed and established from the basic tests performed in New RC project. These analytical models were installed into several FEM codes including a platform program, "FIERCM."

The principal members of RC buildings: panels, shear walls, beams, columns and beam-column joints, were analyzed systematically by the Working Group members using the several FEM codes. The analytical results gave generally reasonable agreements with the test results, but the following problems were pointed out.

1) Evaluation of the Compressive Strength Reduction Factors of Cracked High Strength Concrete

The analytical results of panels and shear walls gave good agreement with the test results, but the analysis of beams and columns gave conservative results as compared with the test results. This tendency was more remarkable for the specimens with high ratio of shear reinforcement. As the reduction factor was based on the panel test, it is considered that the reduction factor is larger in beams and columns, the thickness of which is relatively larger than that of panels. The further investigations of the reduction factor are needed.

2) Confinement Effects of Cracked High Strength Concrete

The analysis gave a conservative results for beams and columns with high ratio of shear reinforcement. In this case, the confinement effects for the shear strength should be considered. In this study, the confinement effects were considered as the estimation of the stress-strain curves for the convenience of the plane stress analysis using the confinement model originally proposed for flexural confinement problems. The further investigations are needed for this conventional approach. The three-dimensional approach will be necessary for the essential solution of the problems of confinement effects.

3) As for the tension stiffness models and shear stiffness reduction factors, the further investigations are needed for several effecting factors.

4) The general estimation of bond in RC members with high strength materials are needed considering several effecting factors.

From the systematic FEM parametric analyses, the effects of the following parameters on the shear strength were investigated, and the applicability of the design and experimental equation was verified.

1) Beams: $p_w \times \sigma_y$
2) Columns: axial stress ratio and p_w
3) Beam-column joints: p_w, σ_B
4) Shear walls: σ_B, $p_w \times \sigma_y$, p_t, and shear span ratio
5) Panels: $p_w \times \sigma_y$, σ_0 (uni-axial and bi-axial stresses)

As the further research subjects, the investigations of the shear and bond resistance mechanisms of RC members are necessary in order to verify the concepts of the truss and arch mechanisms proposed in the previous macroscopic models and to propose the more rational macroscopic models and design equations for the shear strength and deformation of RC members with high strength materials.

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Table 3-1 Systematic Finite Element Analyses of Specimens Selected by JCI and Tested in New RC Project

	Beam	Panel	Wall	Beam-column Joint	Column
Noguchi	[Ihzuka] JCI ASB NRC	[Ihuzuka, Zhang, Stevens] JCI NRC	[Ihzuka] JCI NRC HiRC	[Ihuzuka, Zhang] NRC {Joint WG}	[Zhang] NRC HiRC
Shirai		[Stevens] Collins (PV, PB)	[Stevens] JCI NRC		
Shiohara	[Stevens] NRC (PB, B) JCI			[Stevens] NRC	[Stevens] JCI HiRC NRC
Naganuma	[FINAL, Stevens] JCI NRC (PB, B) ASB		[FINAL] JCI NRC		[FINAL] JCI NRC
Sumi		[Stevens] NRC			
Takagi Shiraishi Suzuki {Wall WG}			[Meiji Univ.] NRC		
Uchida	[Fujita] NRC (PB, B) ASB				

[Researcher's or program's name]
{Working Group in New RC Project}

JCI : Specimens selected by Japan Concrete Institute
NRC : Specimens tested in National Research Project (New RC)
ASB : Specimens tested by Chiba University
HiRC : Specimens tested by Chiba University and Kajima Corporation

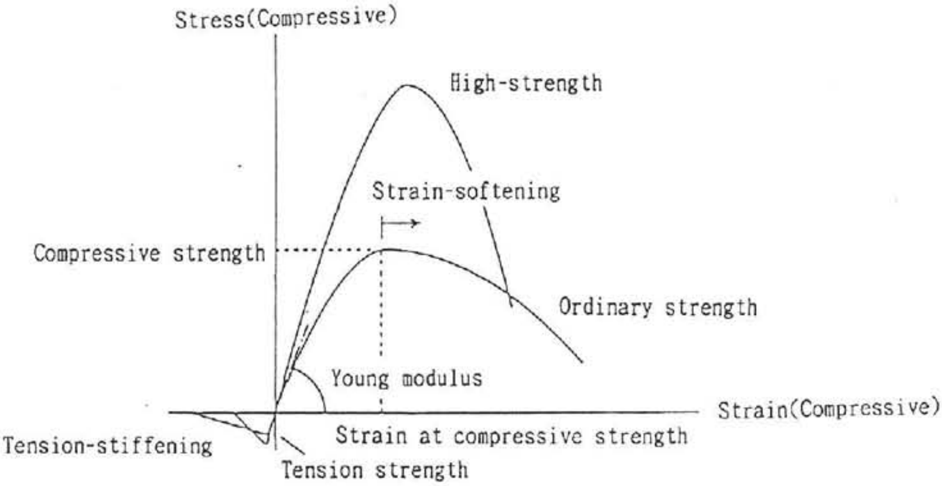


Fig. 3-1 Stress-strain Relationships of Concrete

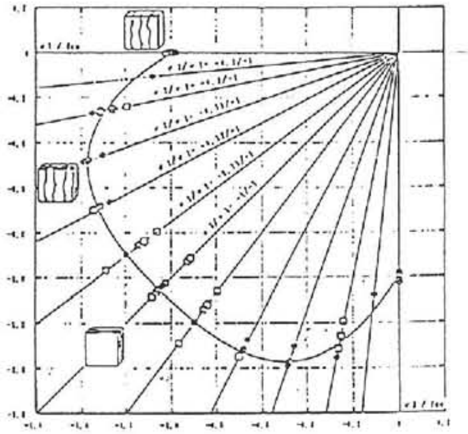


Fig. 3-2 Biaxial-criterion of High Strength Concrete

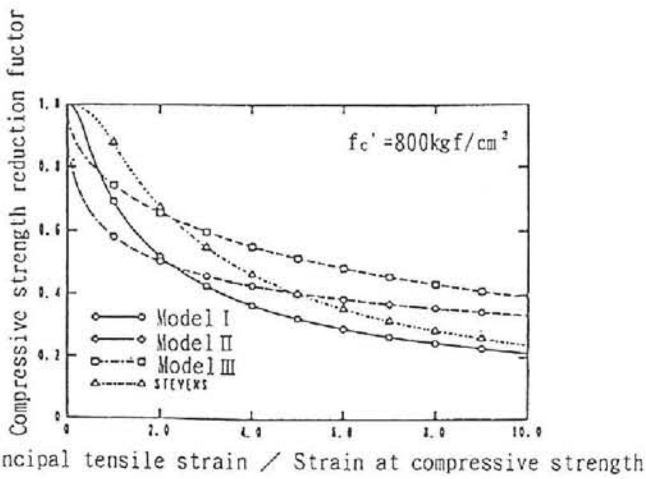
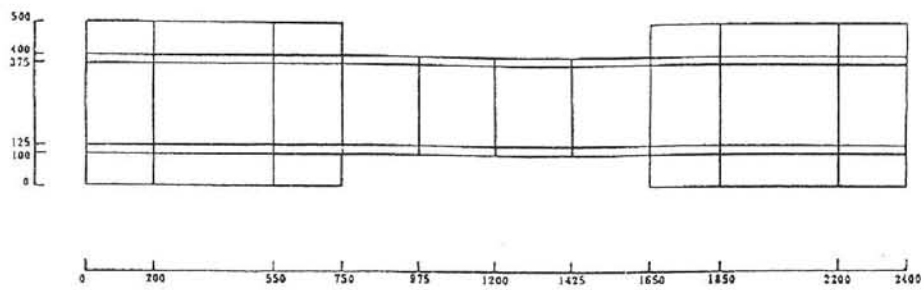


Fig. 3-3 Compressive Strength Reduction Factor for High Strength Concrete

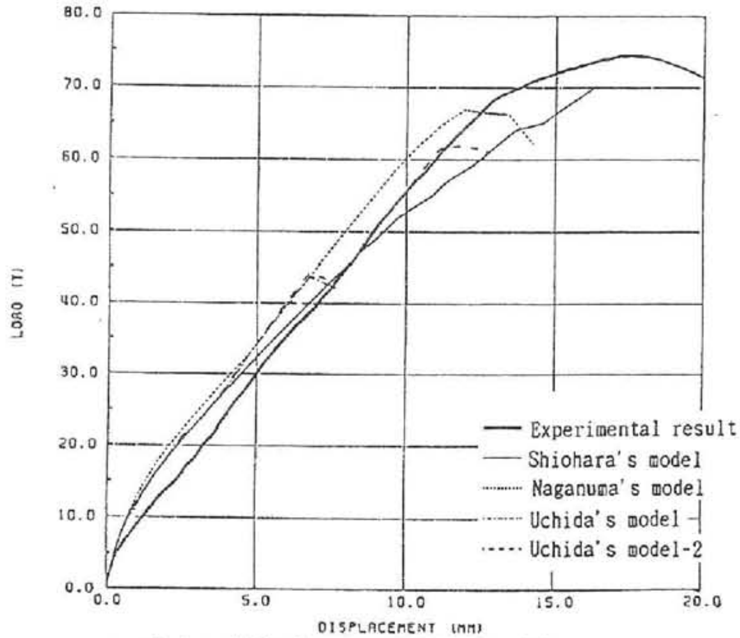
Specimens for analysis



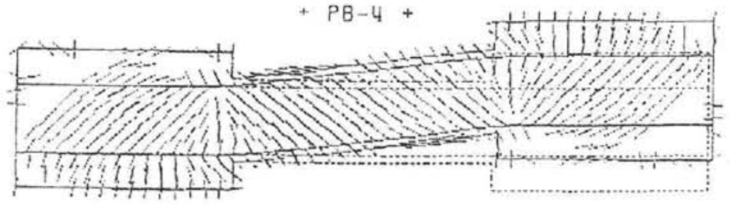
a) Finite Element Idealization

b) Comparisons of Analytical Results with Test Results of Beam PB4

	shear strength	Failure mode
Experimental result	74.5 tf	Flexural yielding
Shiohara's model	69.8 tf	—
Naganuma's model	66.8 tf	Flexural Compression failure
Uchida's model-1	43.6 tf	·Shear compression failure
Uchida's model-2	61.9 tf	Shear compression failure(edge)



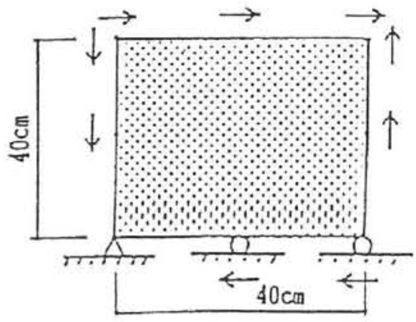
c) Load-Displacement Relationships



d) Crack Pattern (PB-4 at Maximum Strength)

Fig. 3-4 Finite Element Idealization and Analytical Results for New RC Beam PB4 Tested by Prof. F. Watanabe, Kyoto University

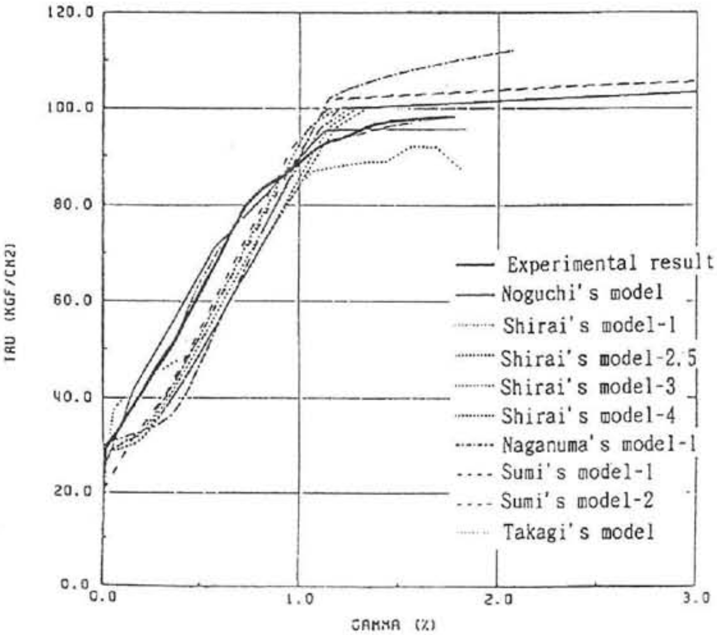
A panel is idealized as a single element.



a) Finite Element Idealization

b) Comparisons of Analytical Results with Test Results of Panel 8-8-8

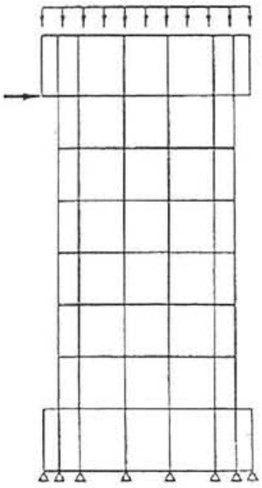
	shear strength	Failure mode
Experimental result	98.1 kgf/cm²	Cut off of reinforcement
Noguchi's model	95.5 kgf/cm²	Cut off of reinforcement
Shirai's model-1	104.3 kgf/cm²	—
Shirai's model-2	104.3 kgf/cm²	—
Shirai's model-3	103.8 kgf/cm²	—
Shirai's model-4	114.0 kgf/cm²	—
Shirai's model-5	104.3 kgf/cm²	—
Naganuma's model-1	112.0 kgf/cm²	Yielding of reinforcement
Sumi's model-1	106.5 kgf/cm²	Cut off of reinforcement
Sumi's model-2	105.8 kgf/cm²	Cut off of reinforcement
Takagi's model	92.0 kgf/cm²	—



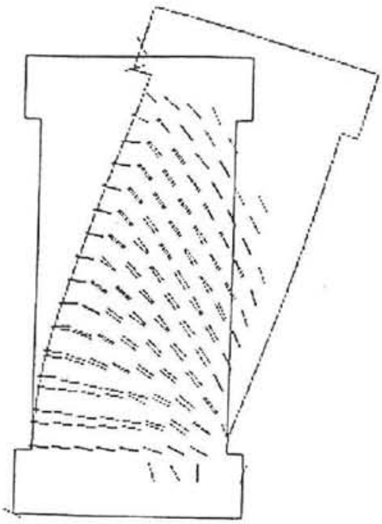
c) Load-Displacement Relationships

+8 −8 −8+

Fig. 3-5 Finite Element Idealization and Analytical Results for New RC Panel Specimen, 8-8-8, Tested by K. Sumi of Hazama Corporation.



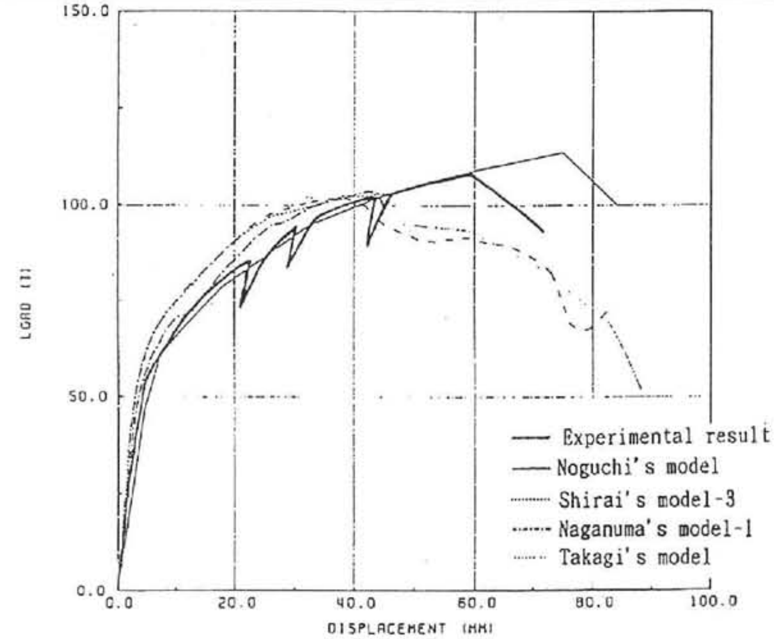
a) Shirai's Model



d) Crack Pattern (NW-1 at Maximum Strength)

b) Comparisons of Analytical Results with Test Results of Shear Wall NW-1

	shear strength	Failure mode
Experimental result	108.4 tf	Flexural failure
Noguchi's model	113.5 tf	Flexural yielding failure
Shirai's model-3	103.3 tf	—
Naganuma's model-1	103.6 tf	Compressive failure at the bottom of compression columns after flexural yielding
Takagi's model	101.9 tf	Compressive failure at shear wall after column flexural yealding



c) Load-Displacement Relationships

+ NW-1 +

Fig. 3-6 Finite Element Idealization and Analytical Results for New RC Shear Wall Specimen NW-1 Tested by T. Kabeyasawa of Yokohama National University

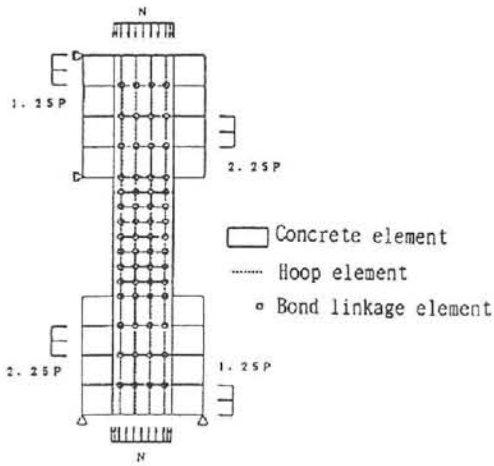


Fig. 3-7 Finite Element Idealization of Columns

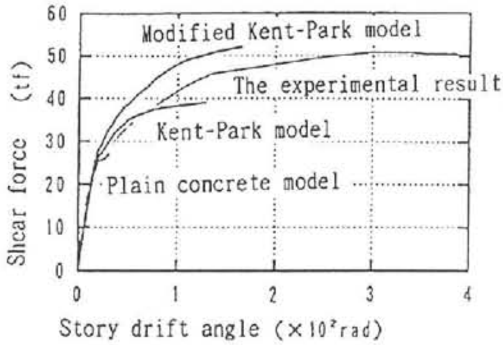


Fig. 3-8 Shear Force-Story Drift Angle Relationships (Influence of Confinement Effects(p_w =1.8%))

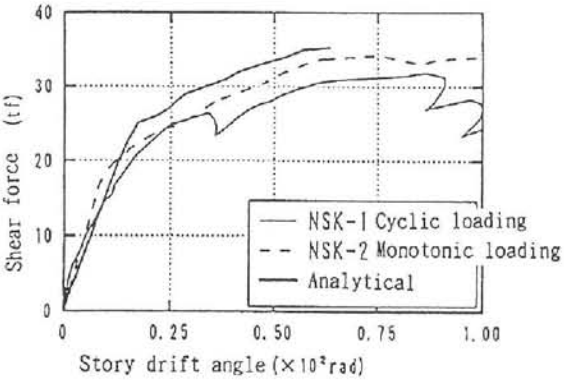


Fig. 3-9 Shear Force-Story Drift Angle Relationships (Loading Methods)

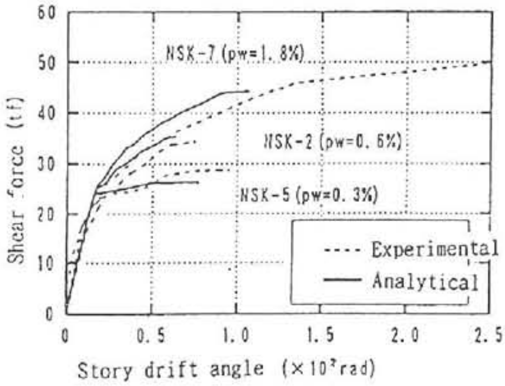


Fig. 3-10 Shear Force-Story Drift Angle Relationships (Lateral Reinforcement Ratio, p_w)

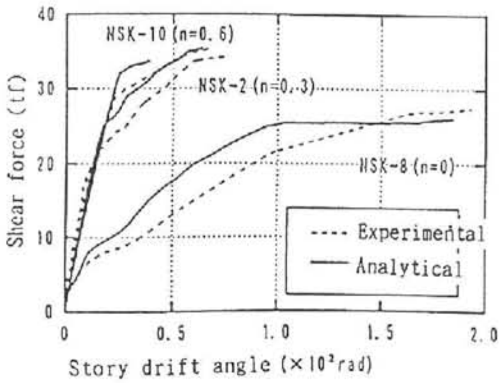


Fig. 3-11 Shear Force-Story Drift Angle Relationships (Axial Stress Ratio, n)

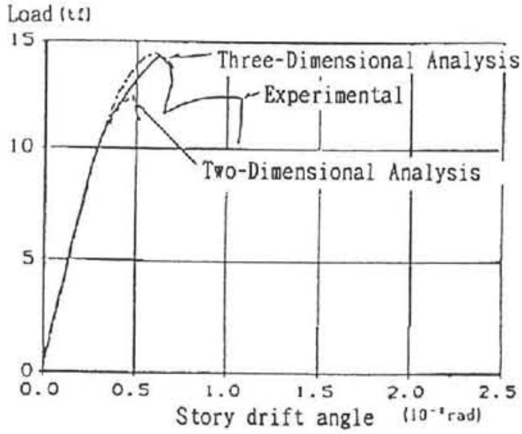


Fig. 3-12 Load-Story Drift Angle Relationships

JCI selected specimen No. 4

σ_B=241kgf/cm²
Main bar: 6-D10
Lateral reinforcement: 2-6ϕ

Axial stress ratio: n=0.581

σ_t=20.3kgf/cm²
p_g=0.68% σ_y=4458kgf/cm²
p_w=0.79%
σ_y=3804kgf/cm²

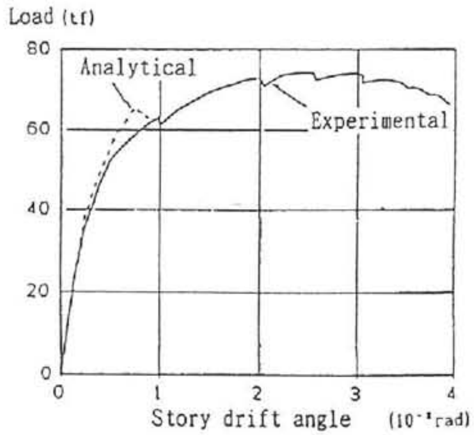


Fig. 3-13 Load-Story Angle Relationships

New RC selected specimen CA06-3-4

σ_B=735kgf/cm²
Main bar: 12-D19
Lateral reinforcement: 4-D10

Axial stress ratio: n=0.30

σ_t=49kgf/cm²
σ_y=7565kgf/cm²
p_w=1.19%
σ_y=10911kgf/cm²

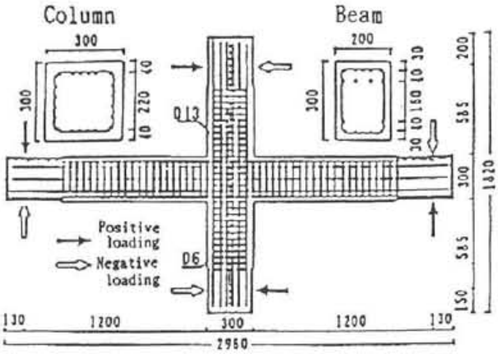


Fig. 3-14 Arrangement of Reinforcement (OKJ-1)

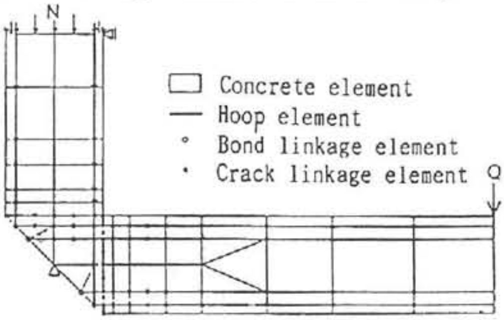


Fig. 3-15 Finite Element Idealization (OKJ)

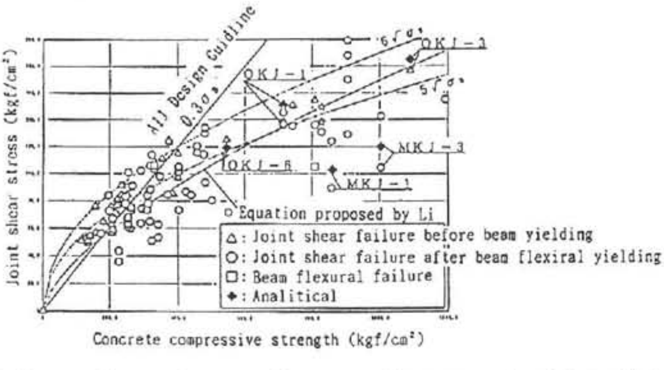


Fig. 3-16 Joint Maximum Shear Stress – Concrete Compressive Strength Relationships

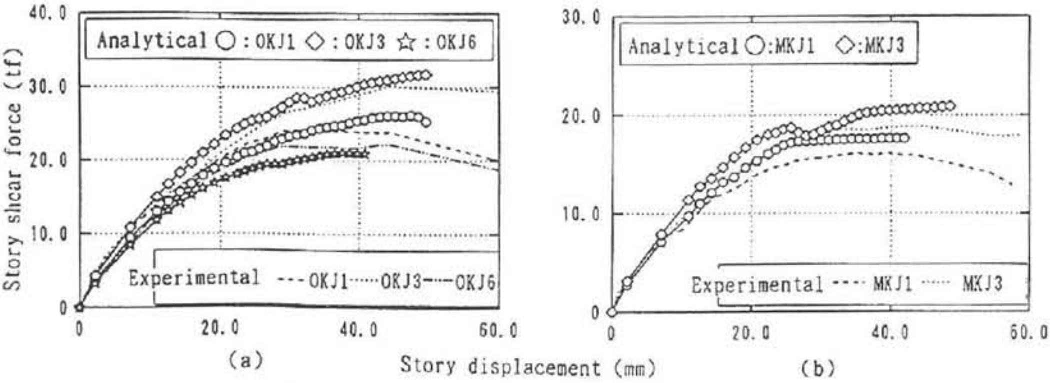
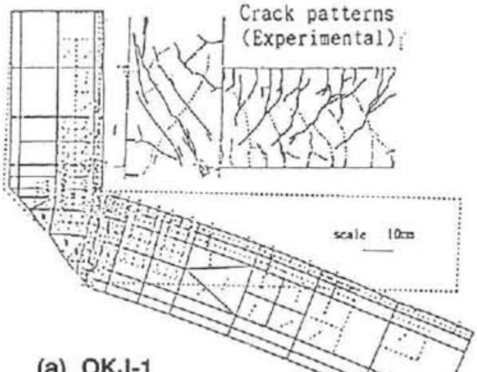
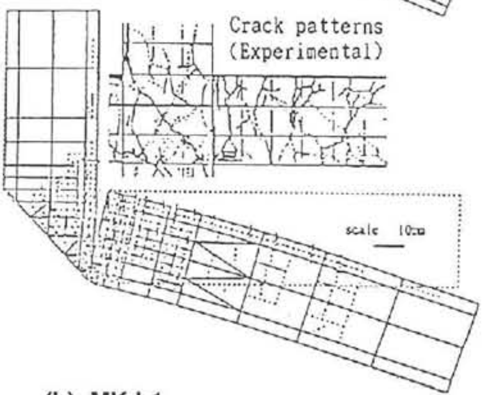


Fig. 3-17 Story Shear Force-Story Displacement Relationships

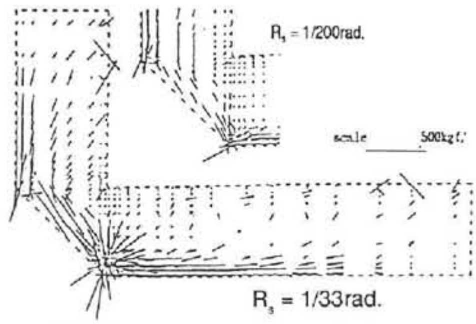


(a) OKJ-1

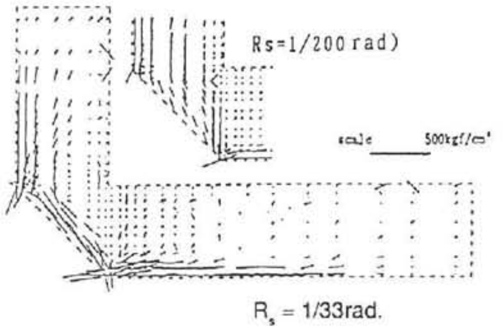


(b) MKJ-1

Fig. 3-18 Crack Patterns
(at maximum loading R_s=1/33rad.)



(a) OKJ-1



(b) MKJ-1

Fig. 3-19 Deformation Patterns
(at maximum loading R_s=1/33rad.)

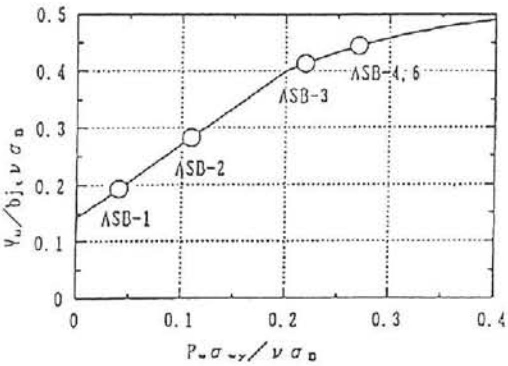


Fig. 3-20 Planning of Beam Test (Ultimate shear strength-amount of shear reinforcement relationships)

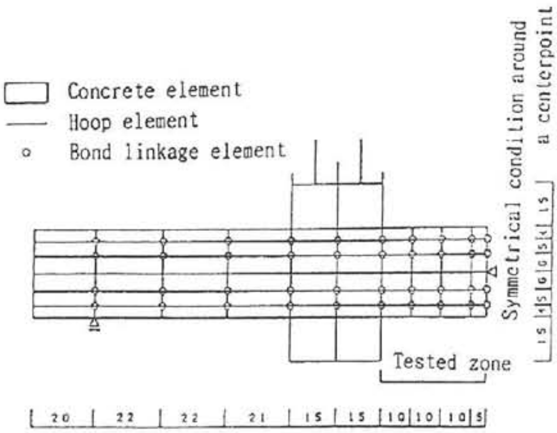


Fig. 3-21 Finite Element Idealization

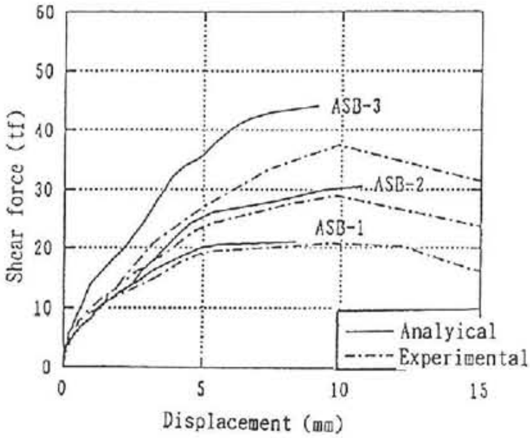


Fig. 3-22 (a) Shear Force-Displacement Relationships

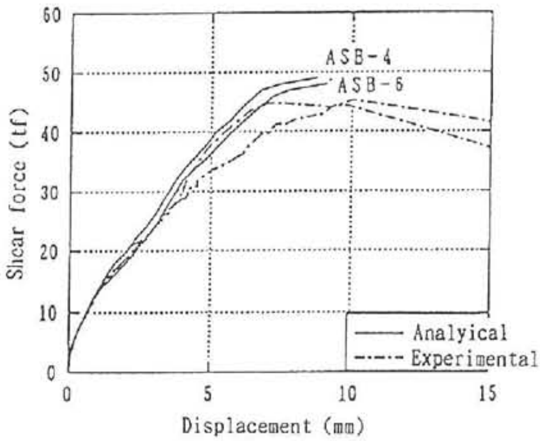


Fig. 3-22 (b) Shear Force-Displacement Relationships

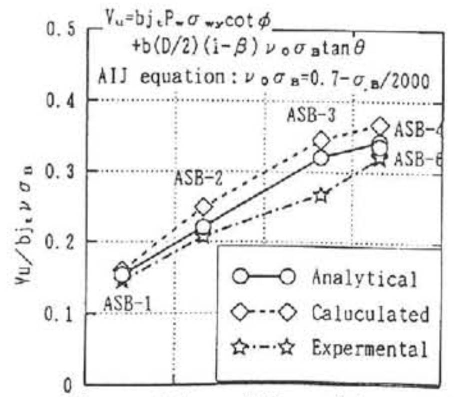
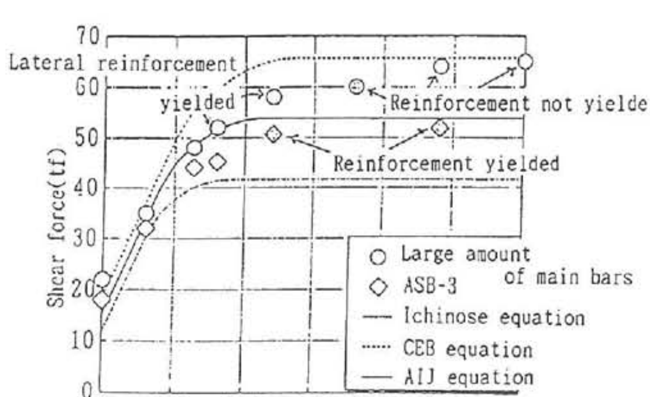


Fig. 3-23 Ultimate Shear Strength-Amount of Shear Reinforcement Relationships



Fig, 3-24 Shear Force-Shear Reinforcement Ratio Relationships

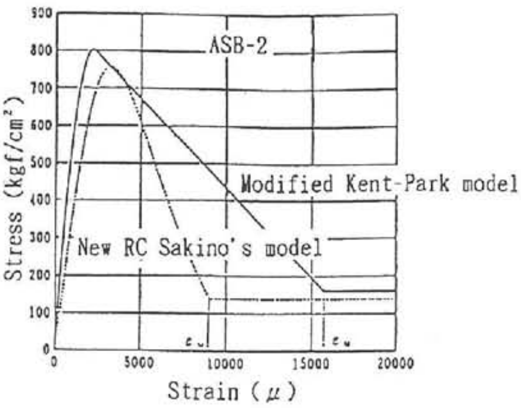


Fig. 3-25 Comparisons of the Stress-Strain Curves by the Two Confinement Models

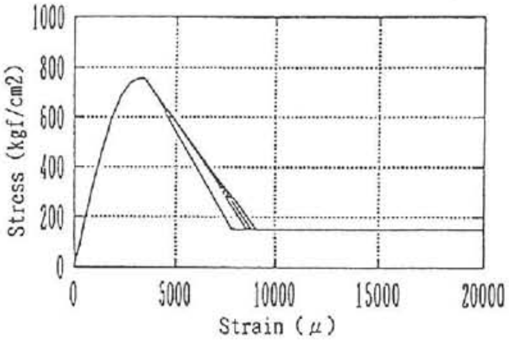


Fig. 3-26 (a) Stress-Strain Relationships Sakino's Model

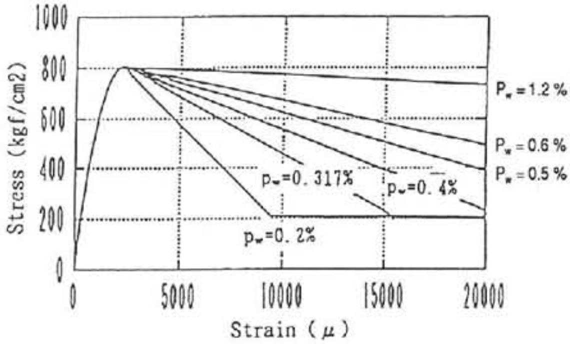


Fig. 3-26 (b) Stress-Strain Relationships Modified Kent-Park Model

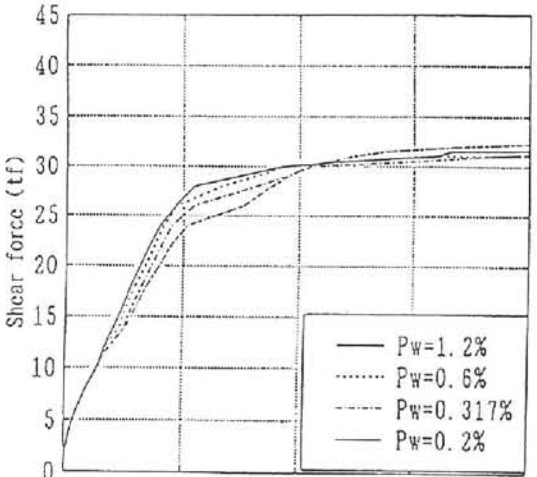


Fig. 3-27 (a) Shear Force-Displacement Relationships

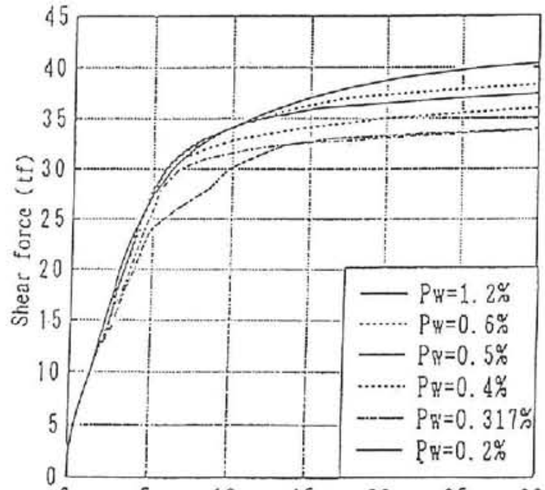


Fig. 3-27 (b) Shear Force-Displacement Relationships

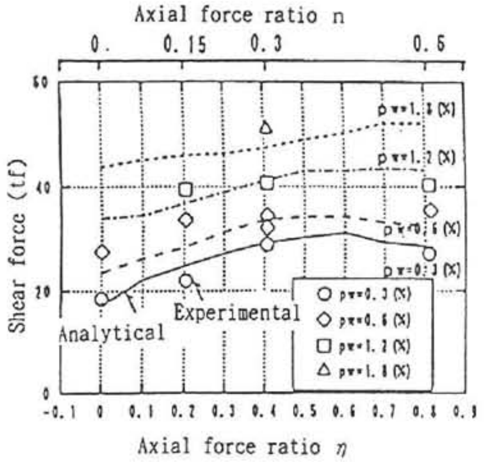


Fig. 3-28 Shear Force-Axial Force Ratio Relationships

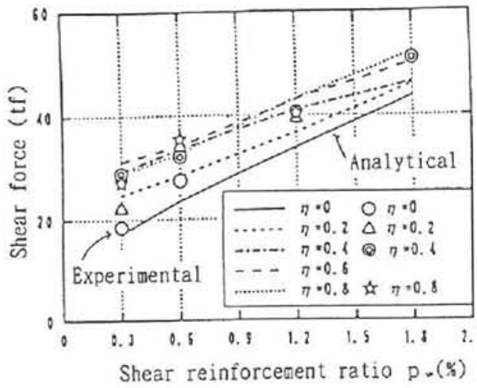


Fig. 3-29 Shear Force-Shear Reinforcement Ratio Relationships

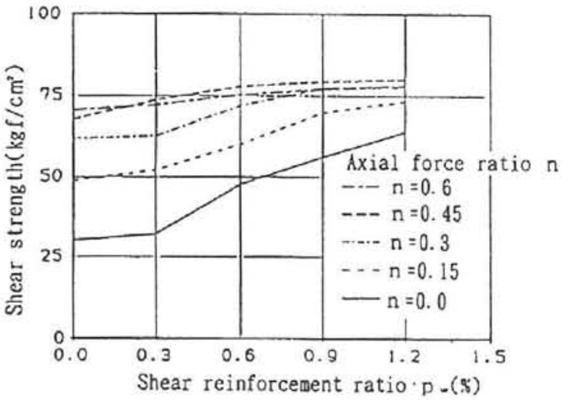


Fig. 3-30 Shear Stress-Shear Reinforcement Ratio Relationships

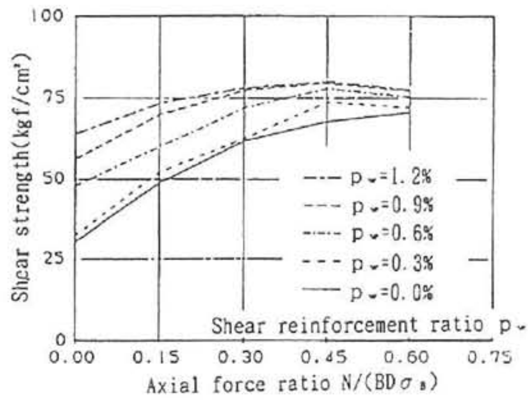


Fig. 3-31 Shear Stress-Axial Force Ratio Relationships

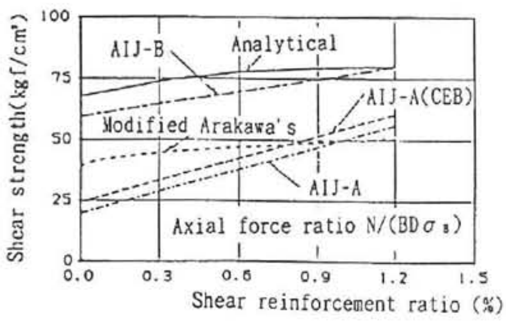


Fig. 3-32 Shear Stress-Shear Reinforcement Ratio Relationships

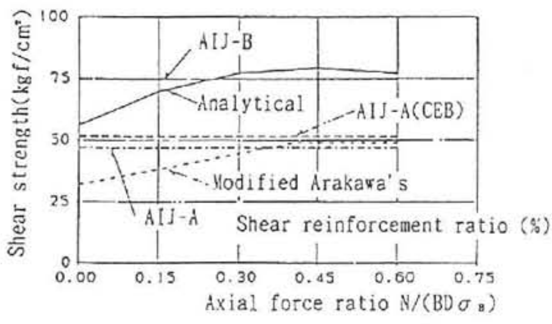


Fig. 3-33 Shear Stress-Axial Force Ratio Relationships

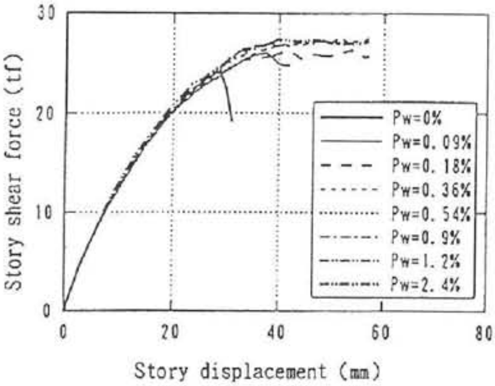


Fig. 3-34 Story Shear Force-Story Displacement Relationships

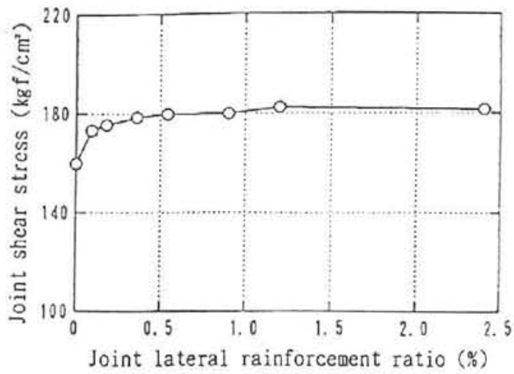


Fig. 3-35 Joint Shear Stress-Joint Lateral Reinforcement Ratio Relationships

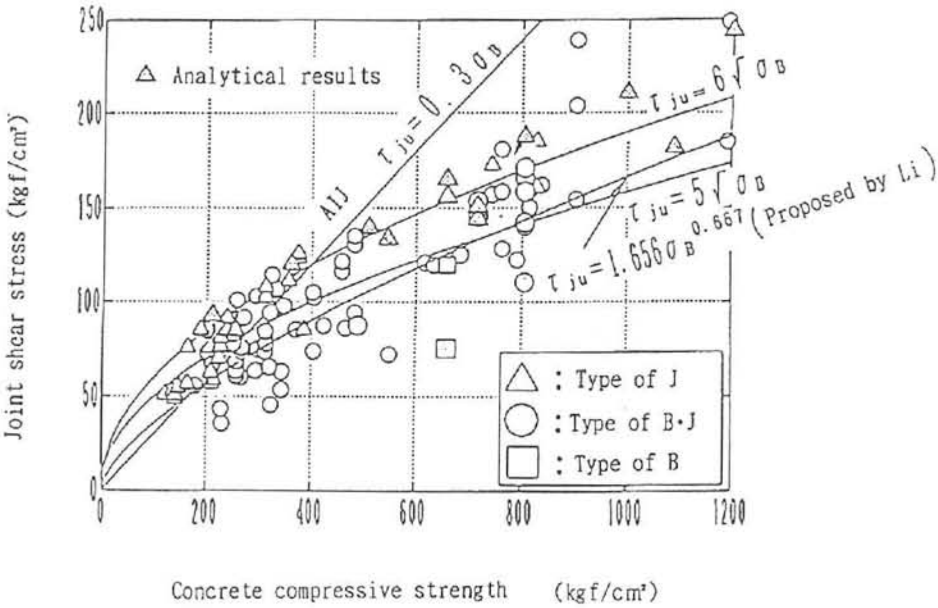
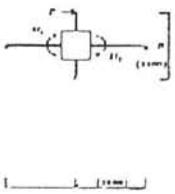
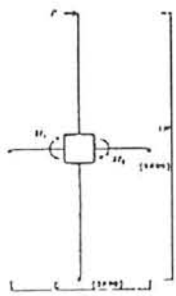


Fig. 3-36 Joint Shear Stress-Concrete Compressive Strength Relationships

FEM Analysis (to certify)

	Bond		Influence of Story Shear Force
	Arrangement J-11 (perfect bond)	Arrangement J-11 (no bond)	
	CASE 1	CASE 2	CASE 3
Diameter of Main Bars Strength of Main Bars Strength of Concrete	D19 σ_y=800 (MPa) σ_B=40 (MPa)	← ← ←	← ← ←
Bond condition	perfect	no	perfect
H and L			
Maximum Shear Force of Beam-column Joints (tf)	117.6	61.4	132.3
Maximum of M1+M2 (tf)	28.5	15.0	28.0
Maximum of Story Shear Force (tf)	22.8	11.9	5.6

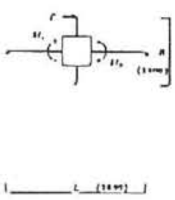
	Influence of strength of Main Bar		
	Both Column and Beam Ordinary Strength	Only Beam Ordinary Strength	Only Column Ordinary Strength
	CASE 4	CASE 5	CASE 6
Area of Main Bars of Beam Area of Main Bars of Column Strength of Main Bars of Beam Strength of Main Bars of Column Strength of Concrete	Double of CASE 1 Double of CASE 1 σ_y=400 (MPa) σ_y=400 (MPa) σ_B=40 (MPa)	Double of CASE 1 Equal to CASE 1 σ_y=400 (MPa) σ_y=800 (MPa) ←	Equal to CASE1 Double of CASE 1 σ_y=800 (MPa) σ_y=400 (MPa) ←
Bond Condition	Perfect Bond	Perfect Bond	Perfect Bond
H and L			
Maximum Shear Force of Beam-Column Joints (tf)	126.4	124.8	118.6
Maximum of M1+M2 (tf)	30.7	30.3	28.8
Maximum of Story Shear Force (tf)	24.5	24.2	23.0

Fig. 3-37 Parameters of Analytical Specimens

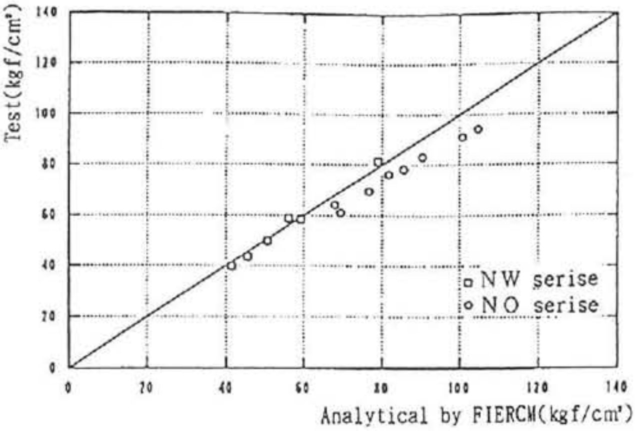


Fig. 3-38 Comparisons of FEM Analytical and Test Results for Ultimate Shear Strength

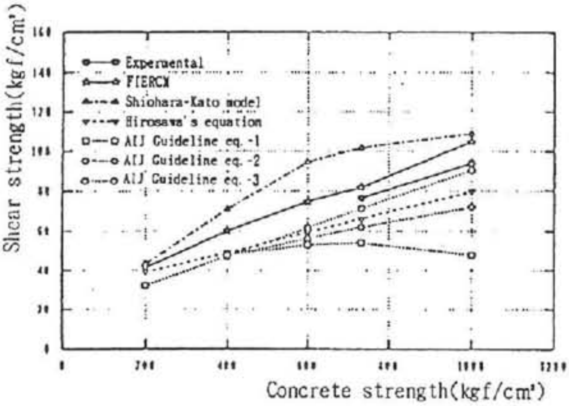


Fig. 3-39 Effects of Concrete Strength

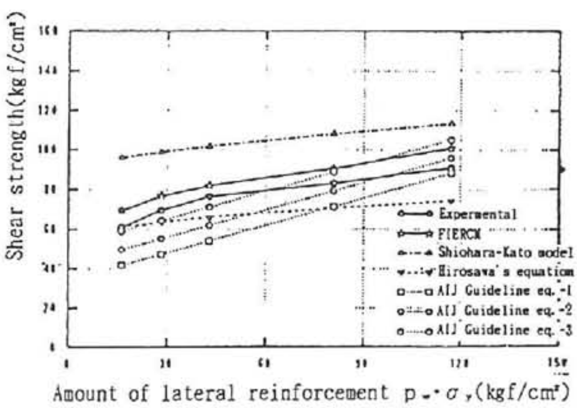


Fig. 3-40 Effects of $p_w \cdot \sigma_y$

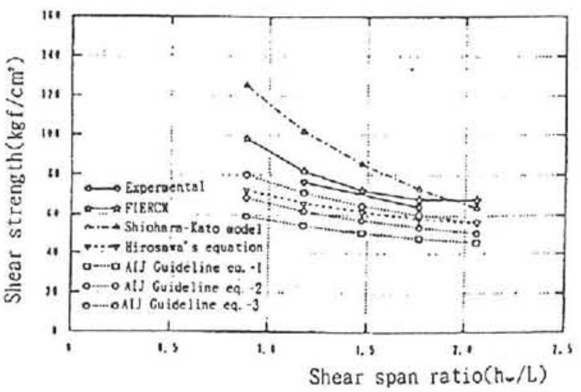


Fig. 3-41 Effects of Shear Span Ratio

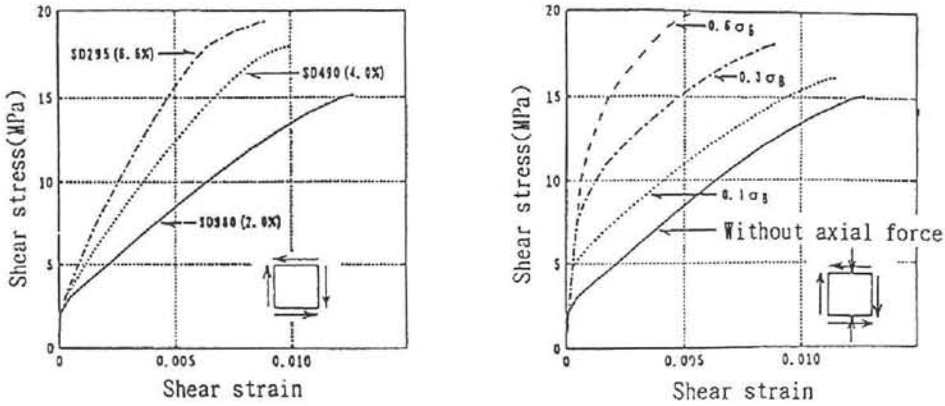


Fig. 3-42 Combinations of P_t and σ_y (p_t=2.0%+SD980, p_t=4.0%+SD490, p_t=6.6%+SD295, for $p_t \cdot \sigma_y$=200MPa)

Fig. 3-43 Variations of Uni-axial Compressive Stresses

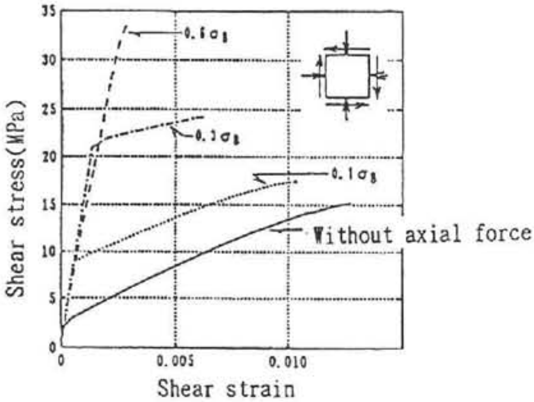


Fig. 3-44 Variations of Bi-axial Compressive Stresses

NOTA BIOGRAFICA DE HIROSHI NOGUCHI

El profesor Noguchi recibió su grado de Doctor en Ingeniería de la Universidad de Tokio en 1976. Fue profesor asistente en la Universidad de Chiba durante el periodo de 1977 a 1979 y profesor asociado de 1979 a 1990. Desde 1990 es profesor titular y desde 1993 jefe del Departamento de Arquitectura.

Fue investigador visitante en la Universidad de Toronto durante 1984 y 1985. Sus líneas de investigación son el análisis no lineal de estructuras de concreto empleando el Método del Elemento Finito y el estudio de mecanismos de resistencia al cortante en elementos de concreto reforzado. Es miembro de AIJ, JCI, ACI, ASCE e IABSE. También participa en el comité ACI–ASCE de Análisis por Elemento Finito de Estructuras de Concreto Reforzado.

TITULOS PUBLICADOS

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AV. DELFIN MADRIGAL Nº 665, COL. PEDREGAL SANTO DOMINGO
DELEGACION COYOACAN, MEXICO D.F., C.P. 04360

TELEFONOS: 606-98-37, 606-97-39, 606-99-82
FAX: 606-16-08