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PERFORMANCE EVALUATION METHOD FOR REINFORCED CONCRETE BUILDINGS

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SUMMARY

This paper describes proposed design process of earthquake resistant RC structures based on performance evaluation methods. In this process, limit states are defined by the damages. Crack width is used as an index of damages. As a result of the analysis based on the process, the crack width of each member is calculated. Flexural and shear deformation sums to the deformation of members. The shear deformation after cracking, and the crack width that referred to the shear deformation, are both calculated using the truss theory. Experiments were executed to verify the methods. The calculation corresponded well to the experimental results.

INTRODUCTION

For structural design, the Building Standard Law in Japan was revised in 1998, and new technical specifications in the form of the Law Enforcement Order and a series of Notifications of Ministry of Construction were issued in 2000. A performance-based design concept was introduced in the revised law. Architectural Institute of Japan published “guidelines for Performance Evaluation of Earthquake Resistant Reinforced Concrete Buildings” in 2004[1]. In the guidelines, it is described that damage evaluation methods should be proposed for the performance-based design, and the residual crack width was adopted as one of the index of damages.

For the performance-based design, the evaluation method of the earthquake resistant performance for the reinforced concrete members must be developed, as mentioned above. The performance means not only the restoring force characteristics (degradation of stiffness) but also the relationship between damage and deformation or restoring force. In the performance-based design process, limit states of structure are

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defined for the design criteria. There are three states, limit state of serviceability, limit state of reparability (or limit state of damage control) and limit state of safety. Each state is defined by the damages of members consisted in the structure. The yielding of reinforcement and the crack width are used as the index of the damages. As a result of the inelastic frame analysis based on the individually proposed process, the crack width of each members are calculated at each step. Examples of design process of R/C buildings according to the proposed methods are introduced in this paper.
The objective of this research is to develop the performance evaluation method base on the truss theory, for RC members with emphasis on the damage evaluation. This paper describes about the proposed method for earthquake resistant RC members and the comparison of test results and calculation by proposed method. And also, an example of design process for RC building according to the proposed methods is introduced in this paper.

EVALUATION OF RESTORING FORCE CHARACTERISTICS FOR RC MEMBERS

Idealization of Restoring Force Characteristics of RC Members

As a general rule, the restoring force characteristic of RC member was assumed to change its stiffness at the shear and flexural cracking of the concrete and the tensile yielding of the reinforcement. The restoring force characteristics would be defined as force-deformation relationship, and the deformation of RC member subjected to lateral load was assumed to be separated into the shear deformation and the flexural deformation, as shown in **Figure 1**. The restoring force characteristics affected to the shear and the flexural deformation were derived from shear stress-shear strain relationships and moment-curvature relationships, respectively. Because an RC beam or column is designed to behave dominantly in flexure, being prevented from the failure in less ductile modes, the shear deformation is assumed to be elastic irrespective of shear cracks. However, there was observed that the shear stiffness was degraded after yielding of the longitudinal reinforcement in beam and the shear crack width opened widely in ductile columns. Therefore, the shear stress-strain relationship was assumed to change its stiffness at the shear cracks of the concrete and the yielding of the reinforcement in this study.

Truss Theory

Shear Stress-Shear Strain Relationship of Shear Panel (Tsuda [2])

Figure 2 shows an idealization for the shear stress-shear strain relationship of shear panel test. **Figure 3** shows the model for the truss theory. The degraded stiffness after cracking is called as ‘Truss Stiffness’ in this paper.

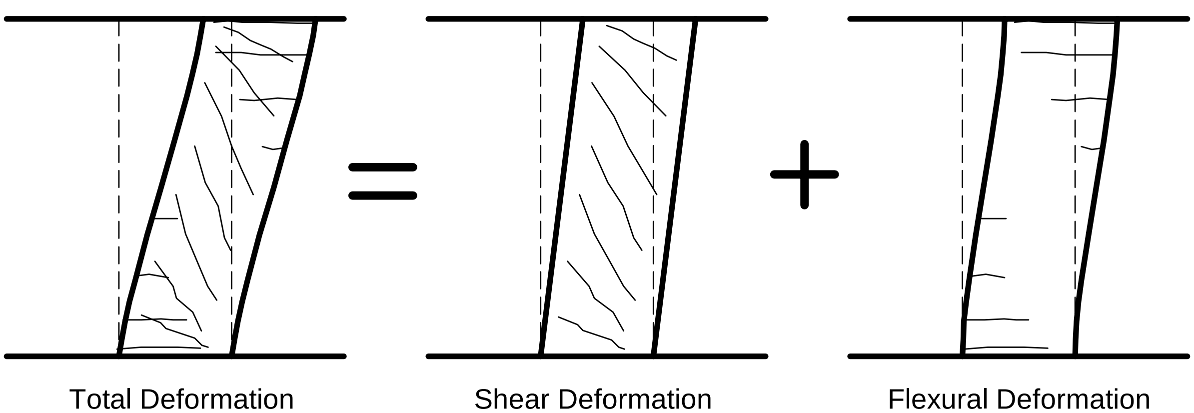


Fig. 1 Image of Deformation

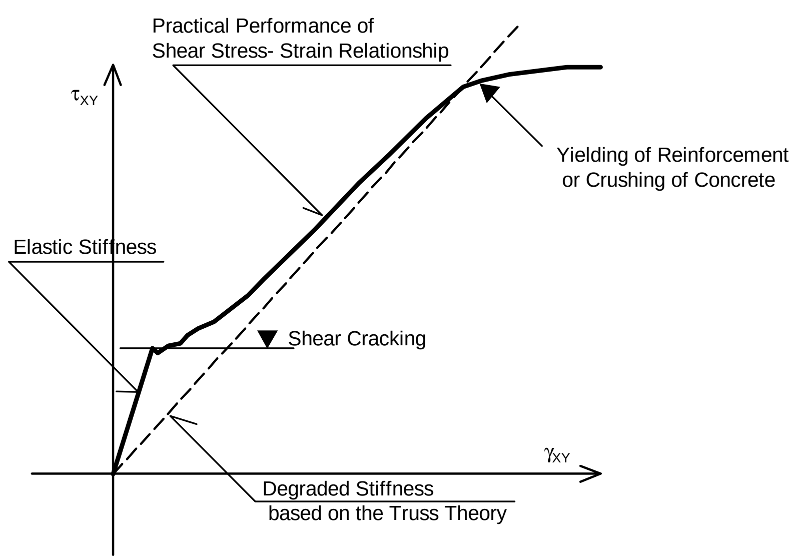


Fig. 2 Idealization of shear stress-shear strain relationship

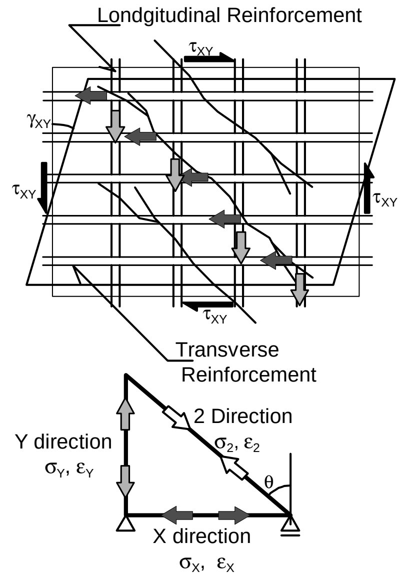


Fig. 3 Model for truss theory

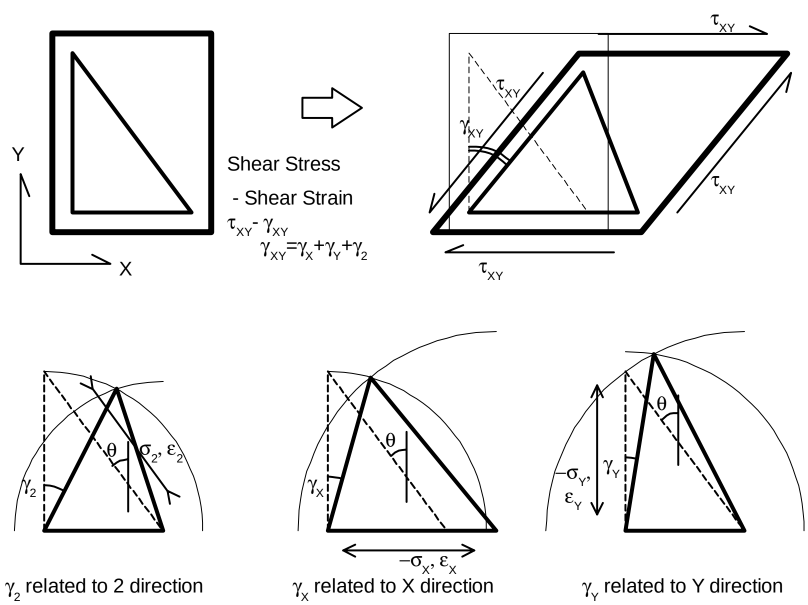


Fig. 4 Shear strain affected to the stress in each direction

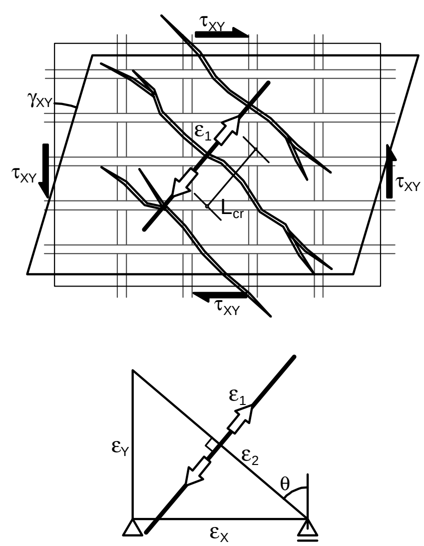


Fig. 5 Model for crack width

It is assumed as follows; 1) The shear stress balances with the stress of transverse (X direction) and longitudinal (Y direction) reinforcement and the principal compressive stress of concrete (parallel direction to the crack). 2) After cracking of concrete, the tensile stress of transverse and longitudinal reinforcement balances with the compressive stresses of concrete in the X and Y direction, respectively. 3) The shear strain is derived as summation of the shear strain affected by the axial strain in each direction (**Figure 4**). 4) The direction of concrete principal stress and strain is coincident.

From these assumptions above, the Truss Stiffness can be calculated as follows.

$$\gamma_2 = \tau_{XY} \Big/ \left(\frac{\sigma_2}{\varepsilon_2} \cdot \cos^2\theta \cdot \sin^2\theta \right) \quad \text{.......} (1)$$

$$\gamma_X = \tau_{XY} \Big/ \left(\frac{\sigma_X}{\varepsilon_X} \cdot \frac{1}{\tan^2\theta} \right) \quad \text{.......} (2)$$

$$\gamma_Y = \tau_{XY} \Big/ \left(\frac{\sigma_Y}{\varepsilon_Y} \cdot \tan^2 \theta \right) \quad (3)$$

$$\tau_{XY} = Gt \cdot \gamma_{XY} = Gt \cdot (\gamma_2 + \gamma_X + \gamma_Y) \quad (4)$$

$$Gt = \frac{1}{1 \Big/ K_2 \cdot \cos^2 \theta \cdot \sin^2 \theta + \tan^2 \theta \Big/ K_X + 1 \Big/ K_Y \cdot \tan^2 \theta} \quad (5)$$

where, γ_2 = shear strain affected by the principal compressive stress of concrete (2 direction), γ_X = shear strain affected by the stress of X direction, γ_Y = shear strain affected by the stress of Y direction, γ_{XY}, τ_{XY} = shear strain and shear stress respectively, θ = the angle between Y direction and 2 direction, Gt = shear stiffness of the truss model, K_2, K_X, K_Y = the stiffness of 2, X and Y direction, normally written by;

$$K_2 = Ecd \quad (6)$$

$$K_X = p_x \cdot E_{sX} \quad (7)$$

$$K_Y = p_y \cdot E_{sY} \quad (8)$$

where, Ecd = degraded Young's modulus of concrete(=70% of Young's modulus of concrete approximately) , p_x, p_y = steel reinforcement ratio in X and Y direction respectively, E_{sX}, E_{sY} = Young's modulus of the steel in X and Y direction respectively.

The angle between Y direction and 2 direction θ can be calculated by following expressions,

$$(1/K_X - 1/K_Y) \cdot \cos^4 \theta - 2 \cdot (1/K_2 + 1/K_X) \cdot \cos^2 \theta + 1/K_2 + 1/K_X = 0 \quad (9)$$

This expression was derived from the theory of minimum potential energy.

Crack Width

Figure 5 shows the calculation model for the crack width. The crack width W_{cr} could be written as

$$W_{cr} = \varepsilon_1 \cdot L_{cr} \quad (10)$$

The principal tensile strain ε_1 could be calculated from the assumptions described above. The crack spacing L_{cr} is a function of the bond strength of concrete and the steel spacing in X and Y direction.

Evaluation Method for Earthquake Resistant Wall

Restoring force characteristics for flexure (Tsuda [3])

Moment-curvature relationship for earthquake resistant wall is derived from the 'plane-sections analysis' based on the assumption that plane sections remain plane. The flexural rotation is calculated by integration of the distribution of curvature. Plastic hinge region, where the curvature assumed to be constant, is written by

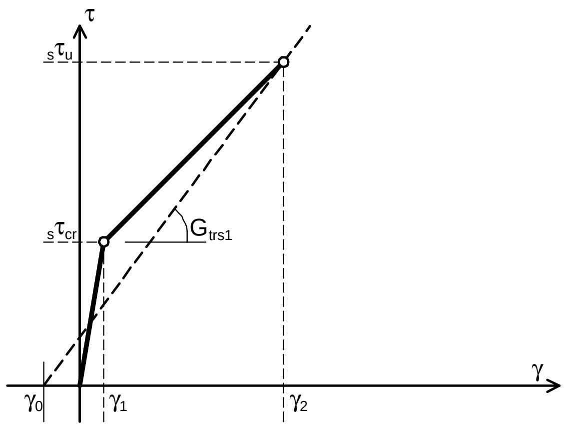
$$h_p = 0.15 \cdot \left(\frac{M}{Ql} \right) \cdot l \quad (11)$$

where, M/Ql = shear span to depth ratio, l = distance between columns at right and left side of the wall.

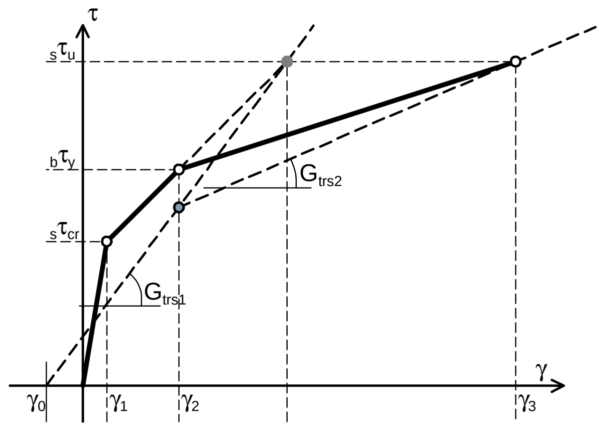
Restoring force characteristics for shear

The shear stiffness after shear cracking is calculated based on the truss theory, in consideration of the influence of axial force, the restraint in lateral direction by columns at each side of the wall and degradation after yielding of the longitudinal reinforcement. The idealization of the shear stress-strain relationships is shown in **Figure 6**.

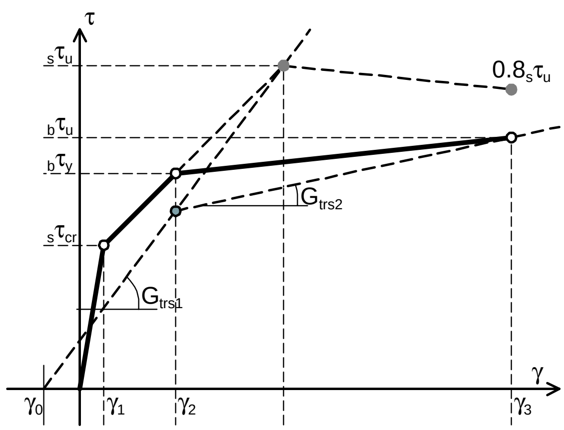
The Truss Stiffness G_{trs1} is calculated using following expressions.



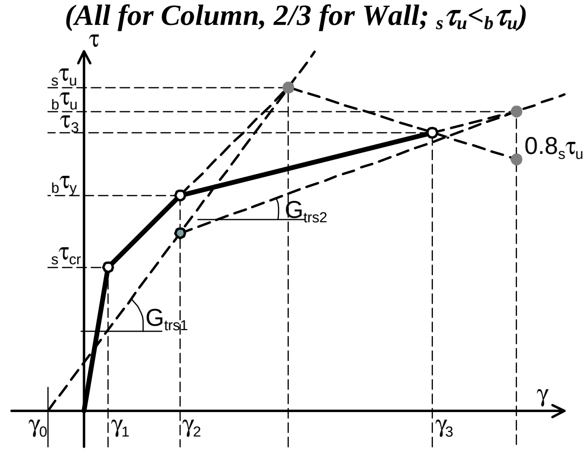
(a) Shear failure before yielding



(b) Yielding of longitudinal reinforcement
(All for Column, 2/3 for Wall; $_s\tau_u<_b\tau_u$)



(c) Yielding of longitudinal reinforcement
(All for Column, 2/3 for Wall; $0.8_s\tau_u>_b\tau_u$)



(d) Yielding of longitudinal reinforcement
(All for Column, 2/3 for Wall; $0.8_s\tau_u<_b\tau_u$)

$_s\tau_u$ = ultimate shear stress of shear failure mode, $_b\tau_u$ = ultimate shear stress of flexural mode,
$_b\tau_y$ = shear stress at flexural yielding, $_s\tau_{cr}$ = shear stress at shear cracking or flexural shear cracking

Fig. 6 Idealization of shear stress-shear strain relationship for earthquake resistant wall

The influence of axial force is considered as γ_0 in Figure 6, which is given by

$$\gamma_0 = \sigma_0 / (K_Y \cdot \tan\theta) \quad (12)$$

where, σ_o = average axial stress over entire wall and column cross sectional area.
For the wall located at the highest floor or the lowest floor, the restraint in lateral direction by columns at each side of the wall is considered as follows;

$$K_X = p_x \cdot E_{sX} + K_f \quad (13)$$

$$K_f = \alpha \cdot Ec \cdot {}_cI_e \cdot l / (t_w \cdot h^4) \quad (14)$$

where, α = 360 for the single-story wall, 22.5 for multi-story wall, Ec = Young's modulus of concrete, $_cI_e$ = moment of inertia of the column, t_w, h = thickness and height of the wall, respectively.
In consideration of difference of concrete strength σ_B, K_2 is defined as follows;

$$K_2 = 0.168 \cdot \sigma_B{}^{0.38} \cdot Ec \quad (15)$$

The Truss Stiffness after yielding G_{trs2} is calculated from the assumptions as follows;
1) The longitudinal reinforcement of the tensile column and the wall assumed to be neglected in Y direction.
2) The stiffness of the concrete K_2 is degraded as follows;

$$K_2 = 1.6 \times 10^{-3} \cdot \sigma_B{}^{0.71} \cdot Ec \quad (16)$$

3) The restraint in lateral direction by columns at each side of the wall is disappeared(K_f=0).

Evaluation of crack width (Tsuda [4])

The average of the shear crack spacing is calculated by the following expression, which is modification of the proposition by Adachi[5]. And the maximum of the shear crack spacing is calculated as follows;

$$_sL_{cr.ave} = \frac{3 \cdot \sigma_t \cdot b_e \cdot t_w}{n \cdot \tau_{\max} \cdot (\phi_x + \phi_y)} \cdot \frac{2.6 - 0.93 \log[0.5 \cdot (S_x + S_y)]}{\cos\theta_{cr}} \quad (17)$$

$$_sL_{cr.\max} = 29.2 + 1.53 \cdot {}_sL_{cr.ave} \quad (18)$$

where, σ_t = tensile strength of the concrete , $b_e = 0.5 \cdot (S_x / \cos\theta + S_y / \sin\theta)$, ϕ_x, ϕ_y = length of circumference of the transverse and the longitudinal reinforcement, respectively, S_x, S_y = spacing of the transverse and the longitudinal reinforcement, respectively, τ_{max} = bond strength between the concrete and the reinforcement , n = number of layers of the reinforcement.

The principal tensile strain ε_1 is a function of the shear strain γ and the Truss Stiffness G_{trs1}, as written by

$$\varepsilon_1 = \alpha_1 \cdot \gamma + \beta_1$$

$$\alpha_1 = [\tan\theta_{t1} / K_X + 1/(K_Y \cdot \tan\theta_{t1}) + 1/(K_2 \cdot \cos\theta_{t1} \cdot \sin\theta_{t1})] \cdot G_{trs1} \quad (19)$$

$$\beta_1 = [-1/(K_2 \cdot \cos^2\theta_{t1}) + (1 - \tan^2\theta_{t1})/K_X] \cdot G_{trs1} \cdot \sigma_0 / K_Y$$

After yielding of the reinforcement, the principal tensile strain ε_1 is written by

$$\varepsilon_1 = \alpha_2 \cdot \gamma + \beta_2$$

$$\alpha_2 = [\tan\theta_{t2} / K_X + 1/(K_Y \cdot \tan\theta_{t2}) + 1/(K_2 \cdot \cos\theta_{t2} \cdot \sin\theta_{t2})] \cdot G_{trs2} \quad (20)$$

$$\beta_2 = \varepsilon_{1-2} - \alpha_2 \cdot \gamma_2$$

where, ε_{1-2} , γ_2 = principal tensile strain and shear strain at the yield point, respectively.

For the flexural crack width, *Eq.(10)* is adopted in almost the same way described above. Additional crack width derived from the slip movement of the reinforcement from the basement W_{slip} would be considered. The expression rewritten by

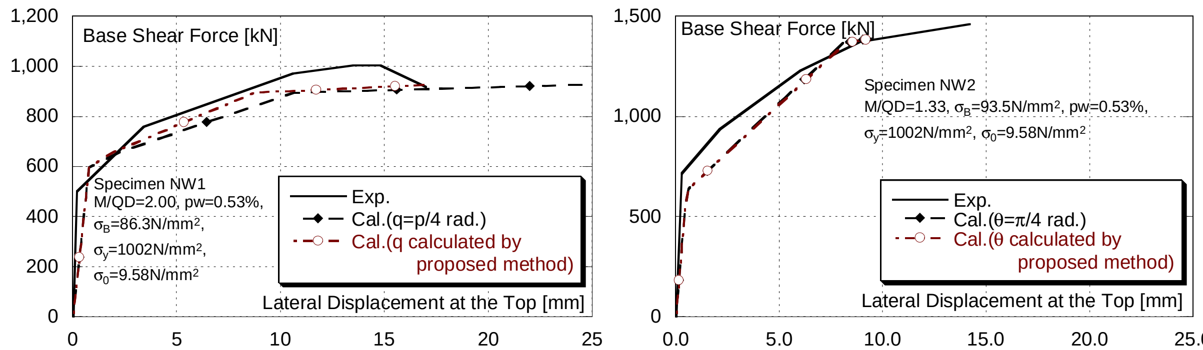
$$W_{cr} = \varepsilon_1 \cdot L_{cr} + W_{slip} \quad (21)$$

Verification by Loading Test (Tsuda [4], Matsumoto [6], Sato [7], Shiga [8],)

From **Figure 7** to **Figure 9**, the model is compared to the experimental data of other researchers.

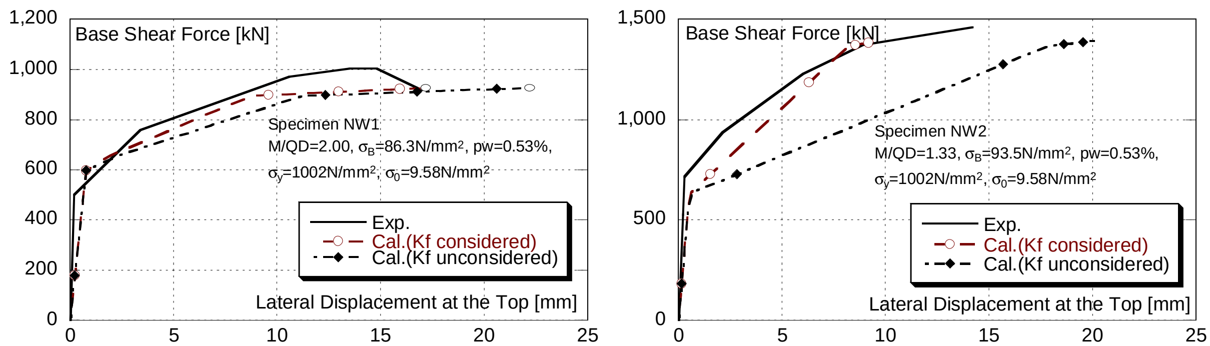
Figure 7 shows the influence of the restraint of the columns. In consideration of K_f, the calculation corresponded well to the experimental results. **Figure 8** shows the influence of assuming of θ, the angle of the principal compressive stress to the Y direction. However θ could be calculated by the expressions in previous section, it can be simplified θ=45 degree as shown in **Figure 8** (b) for *M/QD* which is less than 2.0. **Figure 9** shows that the calculated results by the proposed method correlated well with the test data of multi-story wall. The ratio of the shear deformation to the total deformation at the top floor is not small. It can be seen that the shear stress-strain model affected to the total displacement of the wall.

In **Figure 10**, the calculations of the crack width are compared to the experimental data of other researchers. The shear crack width was measured over wide area. Some of the widest data at the peak of the loading cycles are plotted. The calculation corresponded well to the experimental results.



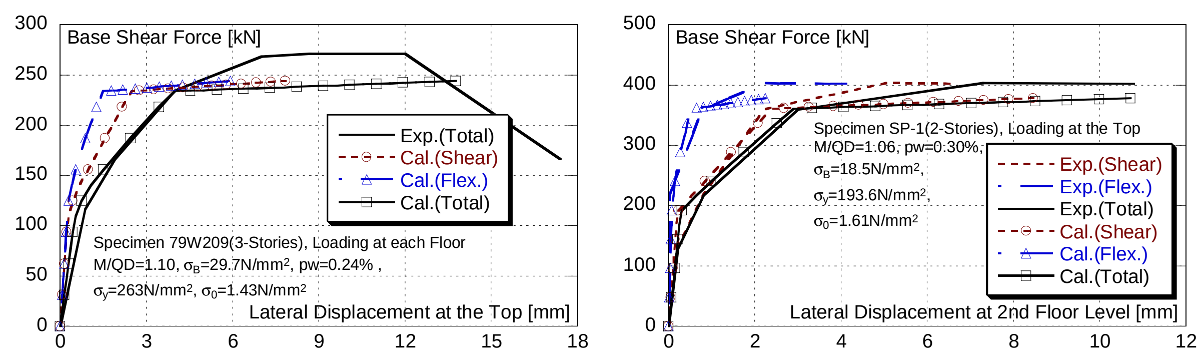
(a) Specimen NW1 *(b) Specimen NW2*

Fig. 7 Influence of restraint from columns (Matsumoto [6])



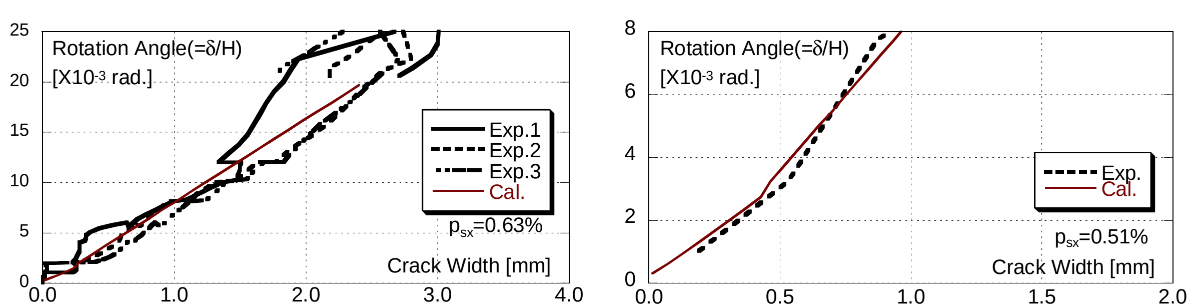
(a) Specimen NW1 *(b) Specimen NW2*

Fig. 8 Influence of angle θ (Matsumoto [6])



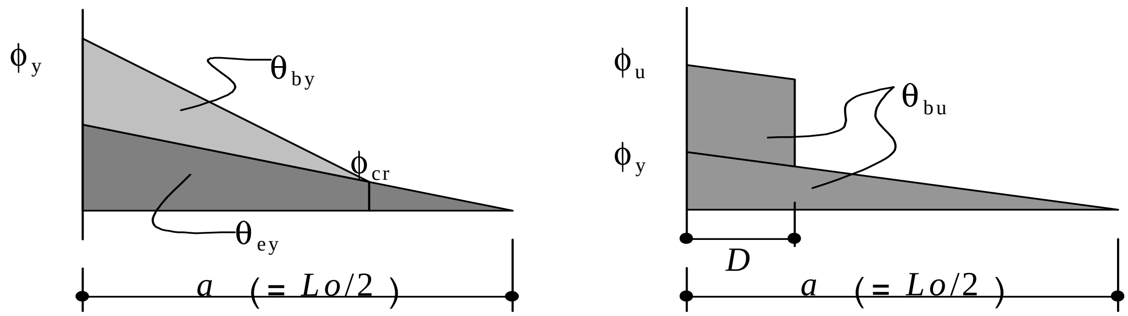
(a) Specimen 79W209 (Sato [7]) *(b) Specimen SP-1 (Shiga [8])*

Fig. 9 Shear force-displacement relationship



(a) Specimen MC (Tsuda [4]) *(b) Specimen SP-1 (Shiga [8])*

Fig. 10 Deformation-shear crack width relationship at the peak of loading cycles



(a) Yield point **(b) Ultimate point**

ϕ_{cr}, ϕ_y, ϕ_u = the curvature of flexural crack, yielding and ultimate strength calculated by 'plane-sections analysis', θ_{ey}, θ_{by}, θ_{bu} = the rotation angle of flexural crack, yielding and ultimate strength integrated from the curvature distribution

Fig. 11 Curvature distribution assumptions

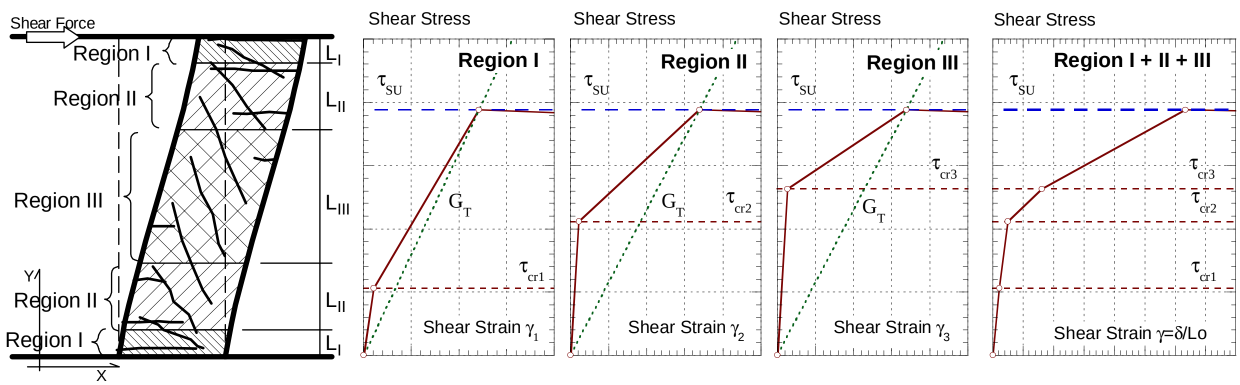


Fig. 12 Idealization of shear deformation for Beam and Column

Evaluation Method for Beam and Column

Restoring force characteristic for flexure

After flexural cracking of beams or columns, the yield point and the ultimate point are defined for flexure. The moment-curvature relationship is calculated by the 'plane-sections analysis'. The flexural deformation at each point calculated as integration of the curvature distribution assumed as shown in **Figure 11**.

The rotation caused by the slip movement at the end of the beam or column written by

$$\theta_{sy} = 0.5 \cdot \frac{a_{st} \cdot \sigma_y}{\psi_{st} \cdot \tau_{st}} \cdot \varepsilon_y \cdot \frac{1}{d - x_n} \quad \text{...} (22)$$

where, a_{st} = cross sectional area of the reinforcement, σ_y, ε_y = yield stress and strain of the reinforcement, respectively, Ψ_{st} = length of circumference of the reinforcement, d = distance from the extreme compressive fiber to the centroid of tensile reinforcement in the section, x_n = distance from the extreme compressive fiber to the neutral axis in the section, τ_{st} = bond strength ($= 0.7 \cdot \sigma_B{}^{0.667}$ [9]).

The total of the deformation affected by flexure is calculated by summation of θ_{ey}, θ_{by} and θ_{sy}.

Restoring force characteristic for shear(Sugimoto [10])

When the Truss Theory is adopted for beams or columns, it assumed to be separated into 5 regions as shown in **Figure 12**. Region I assumed to be effected by the restraint of beam-column joint for strain in the X direction. Region I and II assumed to be effected by yielding of the longitudinal reinforcement, while yielding of the transverse reinforcement assumed to effect to region II and III. The shear strain of the beam or column would be defined as summation of 5 region's shear strain defined based on the Truss

Theory. The previous expressions from *Eq.*(4) to *Eq.* (9) can be adopted to calculate the shear stress-strain relationship for each region, basically.
Only for region I, $1/K_X$=0 is assumed. For region II and III, to consider the influence of cover concrete, *Eq.*(7) is changed into following expression;

$$K_X = 1 \Big/ \left(\frac{1}{p_X \cdot E_{sX}} + \frac{s \cdot b_r^{\,4}}{384 \cdot Ec \cdot I_{co} \cdot j_e} \right) \qquad (23)$$

where, s = spacing of the transverse reinforcement, b_r = spacing of the longitudinal reinforcements hooked by the transverse reinforcement, j_e = maximum distance between compressive and tensile reinforcement, Cs = thickness of cover concrete. I_{co} = moment of inertia of cover concrete ($= 8 \cdot s \cdot Cs^3 / 12$)
For columns, to consider the influence of the axial force, *Eq.*(8) is changed into following expression;

$$K_Y = \frac{\sigma_0 + K_{YO} \cdot (\varepsilon_{y0} - \varepsilon_0)}{\varepsilon_{y0}} \qquad (24)$$

where, $K_{YO} = p_y \cdot E_{sY}$, $\varepsilon_{y0} = \varepsilon_y \cdot \min(1.0, Vsu/Vmy)$, σ_0, ε_0 = average axial stress and strain over entire cross sectional area, respectively, ε_y = yield strain of the longitudinal reinforcement, Vsu = shear strength of the column, Vmy = flexural yielding strength of the column.
The idealization of the shear stress-strain relationship is shown in **Figure 13**. For shear failure mode, the 3rd point is defined as the ultimate shear strength (τ_U). At this point, it is assumed as following;
1) The concrete stress declines to 80% of the maximum compressive strength because of stress softening.
2) The stress of the transverse reinforcement reaches at 110% of the yield stress and the strain reaches at approximately 10000μ because of the strain hardening.
The Truss Stiffness G_{MU} in **Figure 13(b)** is calculated by the assumption that the Young's modulus of the steel E_{sY} in *Eq.*(24) is reduced as 1% of the elastic stiffness for region I and II.

Evaluation of crack width
Minimum of the shear crack spacing ${}_sL_{cr.min}$ is calculated by the following expression, which modified for beams or columns from the proposition by Adachi [5]. And maximum of the shear crack spacing ${}_sL_{cr.max}$ is defined as twice of ${}_sL_{cr.min}$.

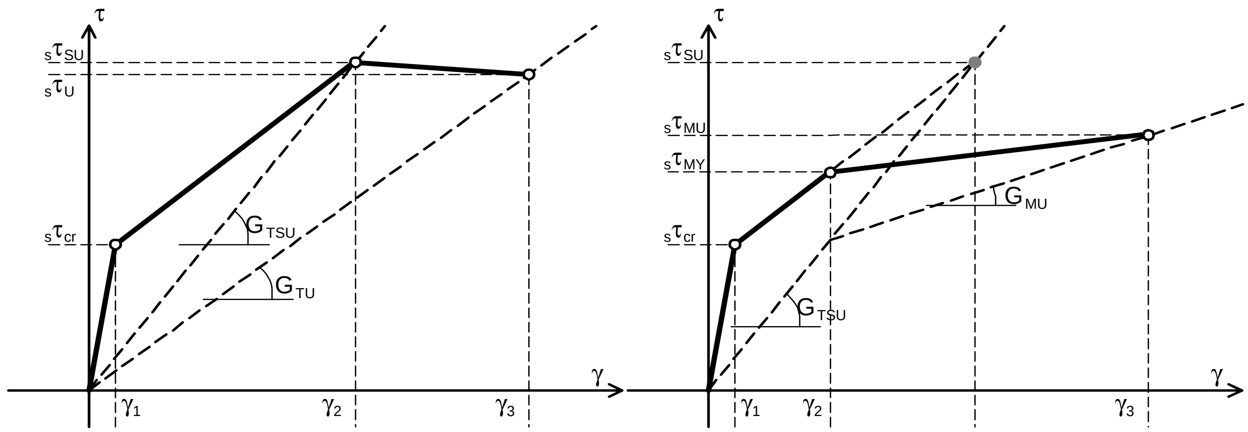
$${}_sL_{cr.\min} = \frac{2 \cdot \sigma_T \cdot b_{ew} \cdot t_c}{\tau_x \cdot \phi_x + \tau_y \cdot \phi_y} \qquad (25)$$

where, σ_T = tensile strength of the concrete($= 0.33 \cdot \sqrt{\sigma_B}$), b_{ew} = equivalent width($= S_X \cdot \sin\theta + S_Y \cdot \cos\theta$), t_c = twice as the thickness of cover concrete, τ_x, τ_y = bond strength of the longitudinal and the transverse reinforcement, respectively($= 0.7 \cdot (1 + \sigma_o / \sigma_B) \cdot \sigma_B^{0.67}$, $= 0.7 \cdot \sigma_B^{0.67}$), S_X, S_Y = average spacing of the longitudinal and the transverse reinforcement, respectively.
The principal tensile strain ε_1 is function of the shear strain γ and the Truss Stiffness G_T, as written by

$$\varepsilon_1 = [\tan\theta / K_X + 1/(K_Y \cdot \tan\theta) + 1/(K_2 \cdot \cos\theta \cdot \sin\theta)] \cdot G_T \cdot \gamma \qquad (26)$$

Verification by loading test for beams and columns

Outline of loading test
1 beam of about 1/2 scale and 2 columns of about 2/3 and full scale were tested. The dimensions and the reinforcement of the beam test specimen is shown in **Figure 14**, and the dimensions and the test setup of the column test specimens are shown in **Figure 15**. The properties of the test specimens are given in **Table 1**. The material properties are given in the table, also. The beam specimen F-L was simply supported at each end of the stab, and was subjected to anti-symmetric bending moment reversals in the span length of 2 meters. The column specimens were subjected to constant axial force and anti-symmetric bending moment reversals by lateral force making the top and the bottom stab parallel.

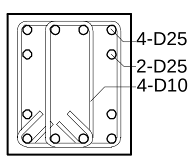
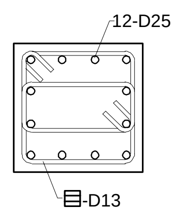
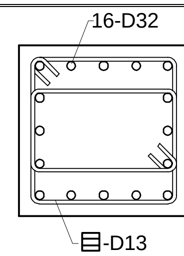


(a) Shear Failure Mode ***(b) Flexural Yielding before Shear Failure***

τ_{SU} = maximum shear stress of shear failure mode , τ_U = ultimate shear stress,
τ_{MY} = shear stress at flexural yielding, τ_{MU} = ultimate shear stress after flexural yielding

Fig. 13 Idealization of shear stress-shear strain relationship for beam or column

Table 1 Properties of test specimen for the verification

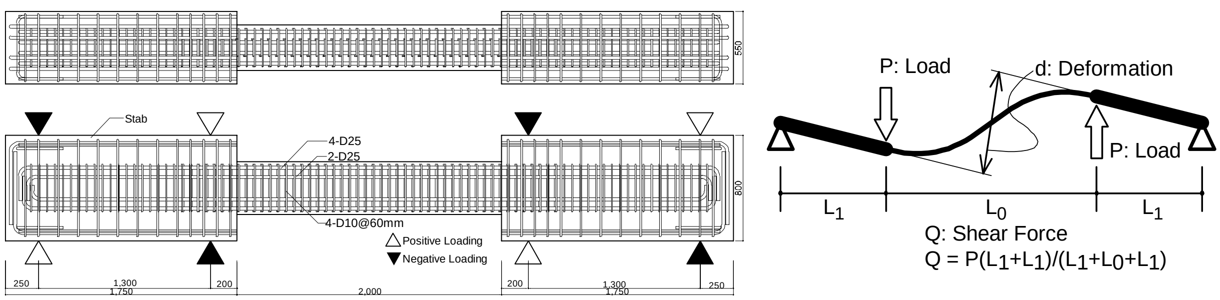
Specimen	F-L	S13-N (2/3 scale)	S13-N-R (full scale)
Section			
B X D [mm X mm]	350 X 400	470 X 470	700 X 700
Height or Length [mm]	2000	1410	2100
Longitudinal Reinforcement	6-D25 (Top/Bottom)	12-D25	16-D32
Material Properties σ_y / Es [N/mm^2]	507 / 1.96 X10^5	1006 / 1.94 X10^5	1014 / 1.96 X10^5
Transverse Reinforcement	4-D10@60	4-D13@140	4-D13@100
Material Properties σ_{wy} / Eh [N/mm^2]	825 / 2.09 X10^5	337 / 1.89 X10^5	337 / 1.89 X10^5
Concrete Properties σ_B / Ec [N/mm^2]	25.8 / 2.51 X10^4	40.2 / 2.70 X10^4	42.4 / 2.99 X10^4
Axial Force [kN]	0	1292	2867

σ_y, Es: observed yield stress and Young's modulus of longitudinal reinforcement,
σ_{wy}, Eh: observed yield stress and Young's modulus of transverse reinforcement,
σ_B, Ec: observed compressive strength and Young's modulus of concrete

Test results and verification

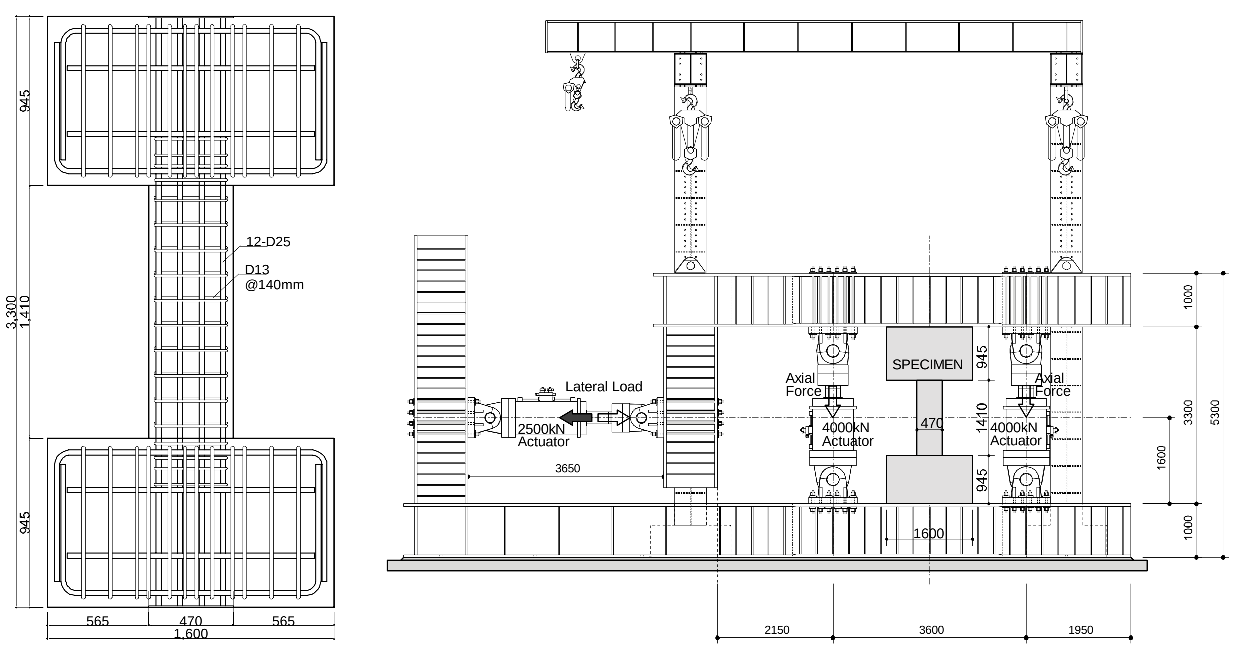
Figure 16 shows the comparison of the observed and the calculated restoring force characteristics and the shear crack width-deformation relationship of the beam specimen F-L. The observation is shown only envelope of the positive direction. It was observed that the first yielding of the longitudinal reinforcement occurred when the deformation reached 19 mm. The calculated final point, the deformation was about 36mm, was derived from the assumption that the strain of the extreme compressive fiber in the section (ε_{cu}) reached 3000 μ for the flexural analysis. The result of the other case of analysis, which was assumed as ε_{cu} = 10000 μ, was plotted in the same figure. It is difficult to define ε_{cu} for calculating the ultimate deformation after yielding. For practical design, the assumption of ε_{cu} = 3000 μ gives underestimation of ultimate deformation. The calculations are correspondent with the experimental results very well for both of the restoring force characteristics and the crack width.

Figure 17 and **Figure 18** shows the comparison of the observed and the calculated restoring force characteristics and shear crack width-shear displacement relationship of the test specimen S13-N and S13-N-R, respectively. Both specimens were failed by shear before yielding of the longitudinal reinforcement. Though the transverse reinforcement ratio is about 0.7% for both specimens, the size of the specimen S13-N-R was about 1.5 times as that of the specimen S13-N. The calculations are corresponded well to the experimental results, in spite of the difference of their sizes.



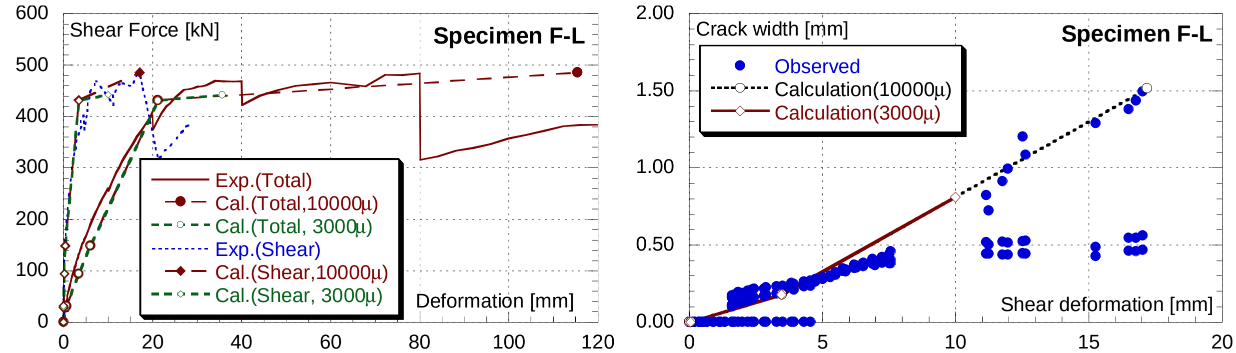
(a) Dimensions and reinforcement of beam test specimen ***(b) Definition of force and deformation***

Fig. 14 Outline of the beam test



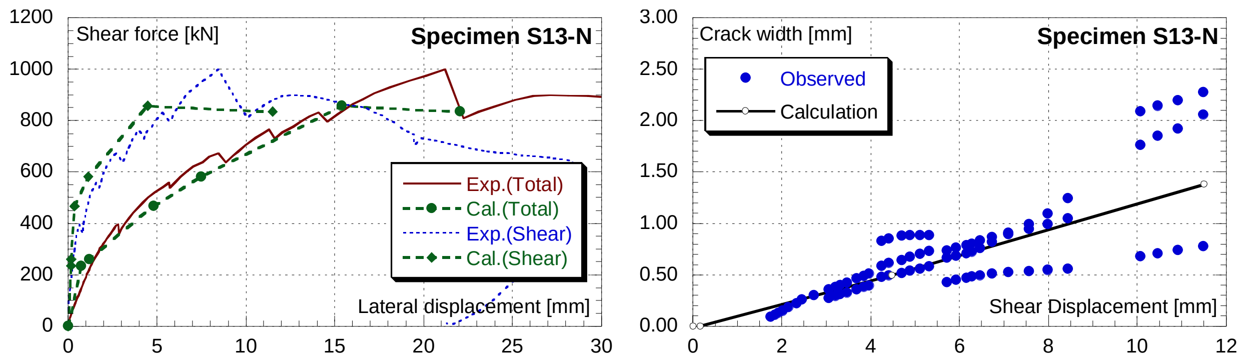
(a) Specimen S13-N ***(b) Test setup***

Fig. 15 Outline of the column test



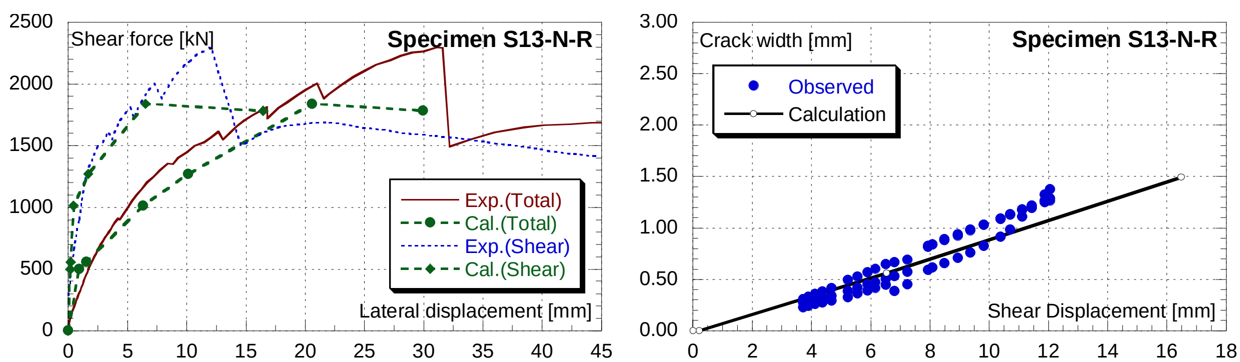
(a) Shear force-deformation relationship *(b) Crack width*

Fig. 16 Comparison of observed and calculated result for Specimen F-L



(a) Lateral load-displacement relationship *(b) Crack width*

Fig. 17 Comparison of observed and calculated result for Specimen S13-N



(a) Lateral load-displacement relationship *(b) Crack width*

Fig. 18 Comparison of observed and calculated result for Specimen S13-N-R

EXAMPLE OF DESIGN PROCESS FOR RC STRUCTURE

Analytical Model

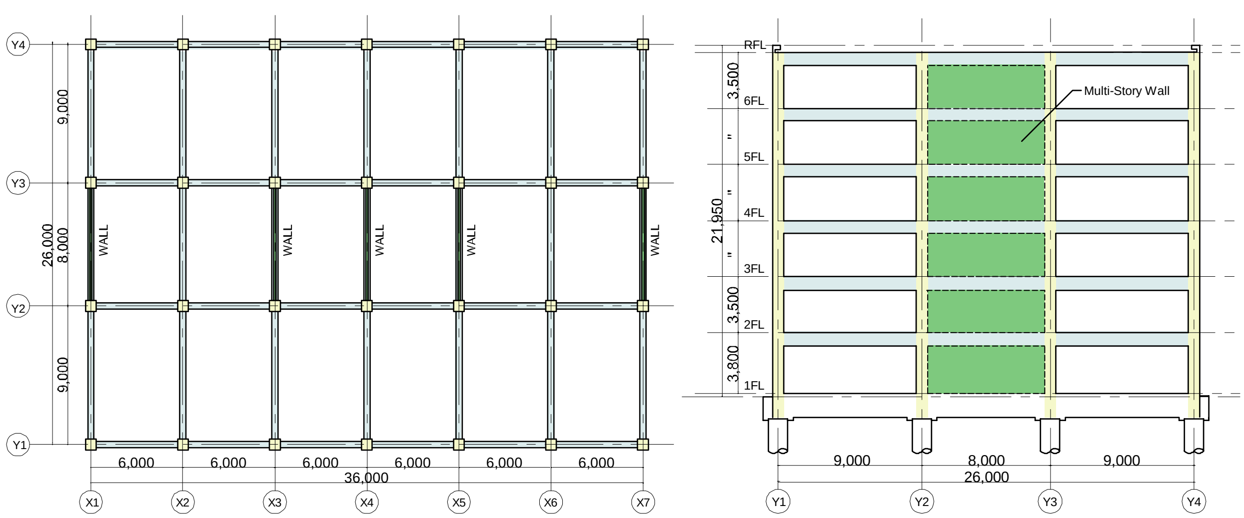
The evaluation methods proposed in the previous sections were installed into three-dimensional inelastic frame analysis program "DREAM-3D"(Nagahara [11]) which was originally developed. In this program, the multi spring model (Lai [12]) is used for the flexural deformation of walls and columns. For beams and the shear deformation of walls and columns, the program was modified to adopt the proposed methods. A six-story RC wall-frame structure was studied for example using the program "DREAM-3D". The outline of the analytical process will be described in this section.

Figure 19 shows the plan and the sectional elevation of the RC structure designed for example. It was designed as a six-story office building in the guidelines [13].

Design Process
The design process is based on the design procedure proposed in the guidelines [1]. This procedure would be applied to the structure, which was completed normal structural design.
1) Nonlinear static analysis under monotonically increasing horizontal forces “pushover analysis”, is carried out.
2) Each limit state is defined based on the damages of the members consisted in the structure.
3) Nonlinear earthquake response analysis of multi-degree of freedom system is carried out.

Calculation of Limit State
Three limit states of the structure are defined as follows, Limit State of Serviceability, Limit State of Reparability (or limit state of damage control) and Limit State of Safety. Limit State of Reparability could be classified into two levels; i.e., minor or medium damage and major damage. This classification should be determined in consideration of the requirement of repair.
Table 2 shows each limit state and correspondent image of the damage. The limit state of each story was derived from the damages of members consisted in each story.
Figure 20 shows the relationship between story shear force-story drift as a result of calculation and each limit state defined based on the damages of consisting members.

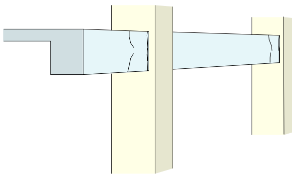
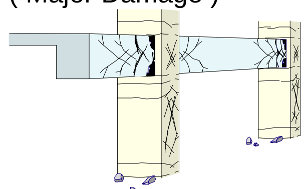
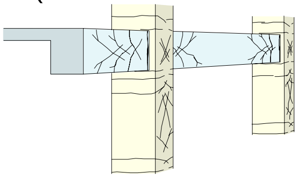
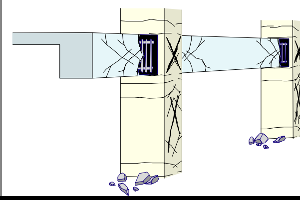
Response Analysis of Multi-degree of Freedom System
The story shear force-story drift relationships were idealized into tri-linear curves for the response analysis. The idealizations were shown in **Figure 20**, also. The Takeda model (Takeda [14]) was selected for the hysteresis model. Ordinary earthquake acceleration record of NS component of Hachinohe(1968) was used to carry out the response analysis. 2 cases were conducted for standardized waves of the maximum velocity as 250mm/sec and 500mm/sec. The example of damage evaluation is shown in **Figure 21**, which was drawn by post-process of “DREAM-3D”. In this case, each story response was less than the Limit State of Reparability II. It means that the residual crack width as the maximum damage of the members after the earthquake was less than 2.0mm.



(a) Floor plan *(b) Typical elevation (X4 Axis)*

Fig. 19 Example structure ([13])

Table 2 Limit states and damages

I) Limit State of Serviceability 	$_rW_{cr}$: Less than 0.2mm Repair: Unnecessary	III) Limit State of Reparability II (Major Damage) 	rWcr: Less than 2.0mm Repair: (ex.) Concrete casting
II) Limit State of Reparability I (Minor or Medium Damage) 	rWcr: Less than 1.0mm Repair: (ex.) Epoxy injection	IV) Limit State of Safety 	rWcr: Over 2.0mm Repair: Impossible

rWcr = residual crack width after earthquake

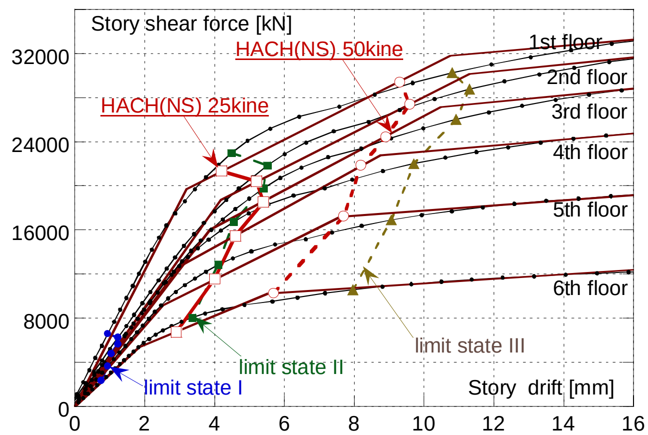


Fig. 20 Story shear force-story drift relationship

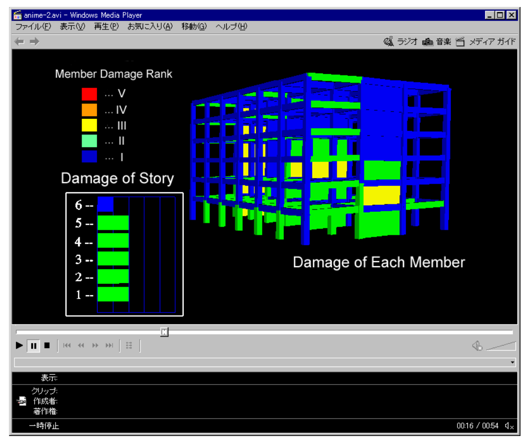


Fig. 21 Example of damage evaluation

CONCLUSIONS

For the performance based design process, the damage evaluation methods for earthquake resistant reinforced concrete members were developed and proposed in this paper. The special features of the proposed methods were following;

1) The shear stress-strain relationship assumed to be degraded its stiffness not only at cracking of the concrete but also at yielding of the reinforcement.
2) The shear stiffness after crack formed was calculated based on the truss theory.
3) The shear crack width was calculated as the product of the crack spacing and principal tensile strain derived from the Truss Theory.

The proposed methods were verified its correspondence with accurate behavior by wall, beam and column tests. The restoring force characteristics and the crack width-displacement relationship calculated by the proposed methods corresponded well to the experimental results.

The calculation procedures were installed into the original program ”DREAM-3D”. For the example of the design process, a six-story RC building were analyzed by using the program.

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