ELASTO-PLASTIC BEHAVIOR OF SHEARWALL WITH ROTATIONAL PC-PANEL AND UNBONDED STEEL PLATE

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SUMMARY

In this paper, a new type of PC(precast concrete)-shearwall having excellent ductility and good energy absorption capacity is proposed, and elasto-plastic behavior of the shearwall is investigated both experimentally and theoretically. Tests were performed with a full-scale wall, 1/2-scale models of the wall and 1/2-scale models of two-stories-one-span frame. A simple and satisfactory design procedure for the shearwall is also proposed.

INTRODUCTION

Conventional PC-shearwalls, which work as one of earthquake resistant components of steel structure, generally have poor flexural capacity compared with steel frames surrounding them. Under seismic shear loading, many shear cracks are first developed in a concrete panel with small deformation, then sudden crushing of the concrete panel often occurs before the frame reaches to an ultimate state, and the structure may undergo significant damages. The proposed shearwall is designed to avoid these demerits of the concrete panel by using an unbonded steel plate.

MECHANISM OF SHEARWALL

The proposed shearwall consists of a rotational elastic PC-panel and an elasto-plastic unbonded steel plate. One side of the panel is fixed to upper and lower beams with the unbonded plate which penetrates through the panel vertically and connects to the panel only at its center. The other side of the panel is fixed to the beams with gusset plates and high strength bolts. When the seismic shear force is transferred from the frame to the shearwall, a lower-part of the unbonded plate becomes longer while an upper-plate becomes shorter, vice versa, and consequentry the panel moves as if rotated, as shown in Fig.1. Under a major earthquake, seismic energy is predominantly absorbed and dissipated into the plate by the form of elastic and plastic strain energy. Whereas the PC-panel only rotates according to the deformation of the unbonded plate, stresses in the panel remain elastic and cracks barely propagate in the panel. Therefore both strength and energy absorption capacity of the panel are maintained and crushing of the panel can be avoided.

TEST-1

A test was performed with a full-scale wall in order to investigate basic mechanical characteristics of the shearwall. The test specimen is shown in Fig.2. The unbonded plate penetrates vertically through the left side of the panel. The ends of the plate connect to an upper loading

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beam and a lower base beam of the testing frame, and its center is anchored to the panel by diagonal reinforcing bars. The cross section of the unbonded plate is rectangular(6 x 150 mm), but the thickness is 19mm at the both ends and at the center. Release agent was painted on the unbonded plate. According to a pull-out bond test carried out on this plate previously, the maximum bond stress between the plate and the concrete is approximately 0.27 kgf/cm², and it is negligible. At the right side, the top and bottom of the panel are connected to the beams of the testing frame by gusset plates and high strength bolts. The concrete of the panel is normal weight concrete and its strength is 302 kgf/cm². The thickness of the panel is 150mm. Two sheets of reinforcing wire-mesh (ϕ6, @ 100mm) are set in the panel. At the ends of the panel,the unbonded plate is surrounded by spiral hoops(D6, @ 50mm). The yield strength of the unbonded plate is 43.1 kgf/mm². Horizontal shear force was applied to restrict movement of the upper beam parallel to the lower beam.

RESULTS OF TEST-1

Shear force-deformation hysteresis loop is shown in Fig.3. The deformation is measured by the relative displacement angle between the upper loading beam and the lower base beam of the testing frame. The specimen exhibited very satisfactory behavior of excellent ductility and good energy absorption capacity. Tensile cracks became visible on the tensile zone of the panel at the deformation about R=2/1000 rad., but the stiffness of the shearwall barely decreased. A broken line in the figure presents the result of the calculation discribed later. The pattern of the deformation of the shearwall, shown in Fig.4, illustrates clearly that the panel rotates almost completely. The deformation of the shear-wall can be resolved into three components,

$$R = \Theta_r + \Theta_p + \Theta_j$$

where R is the total deformation of the shearwall, Θ_r is the rotational compornent, Θ_p is the deformation of the concrete panel, Θ_j is sum of the deformations of the upper and the lower joints. These components are shown in Fig.5. It can be seen from this figure that the rotational component which is induced by elongation and contraction of the unbonded plate, amounts to more than 60 percent of the deformation at the yielding of the unbonded plate. After the yielding of the unbonded plate, the rotational component increases further while the increment of the deformation of the panel is small. Therefore it can be concluded that the seismic energy is absorbed and dissipated predominantly as the plastic strain energy of the unbonded plate.

DESIGN PROCEDURE

The stress distribution in the shearwall can be explained as shown in Fig.1. A relation between the shear force (Q) and the axial force of the unbonded plate (N) is expressed as following,

$$\frac{h}{2}\cdot Q = b\cdot N \qquad (a)$$

where h is the height of the shearwall and b is the horizontal distance between the plate and the joint of the panel. The shear strength of the wall can be obtained by substituting $\sigma y \cdot A$ in Eq.(a), where A is cross sectional area and σy is the yield strength of the plate. The elastic strain of the plate is given by $N/(A \cdot E)$, where E is elastic modulus of the plate. Then the rotational component Θ_r can be given by a following equation,

$$\Theta_r = \left(\frac{N}{A \cdot E} \right) \cdot \left(\frac{h'}{b} \right) = \frac{h\ h'}{2 \cdot A \cdot E \cdot b^2} \cdot Q$$

where h' is length of the plate, as shown in Fig.5. By introducing a coefficient k to expresse the total deformation of the wall as $R=\Theta_r/k$, the elastic stiffness of the wall becomes,

$$Q/R = k \cdot \frac{2 \cdot A \cdot E \cdot b^2}{h \cdot h'} \qquad \text{(b)}$$

The coefficient k relates to the stiffness of the panel and the joint. From Fig.5 the coefficient k is assigned 0.6 for this speciemen. The shear strength and the elastic stiffness of the wall obtained by Eq.(a) and Eq.(b) are shown in Fig.3 by a broken line. The calculated result shows good agreements with the experimental result.

TEST-2

The next tests were performed with two 1/2-scale models of two-stories-one-span steel frames with the shearwalls. The specimens are shown in Fig.6. The cross section of columns of the frame are box-shape(250 x 250mm) and the section of the beams are H-shape (Depth:350mm, Width:175mm). The mechanical characteristics of materials of the frame and the unbonded plate were listed in Table-1. The member sizes of the beams and columns were the same for both test frames.

Table-1 Material Properties

Member	Thickness mm	Yield Strength kgf/mm^2	Elastic Modulus kgf/mm^2
Column	11.3	37.4	20.2×10^3
Beam flange	11.6	34.3	20.2×10^3
Beam web	5.7	33.8	19.2×10^3
Unbonded plate	8.6	39.0	19.5×10^3

Two different types of shearwall were used for each frames. The shearwalls for the first frame are 1/2-scale models of the specimen for the TEST-1 except the shape of the unbonded plate. The shearwalls for the second frame are double size of the panel which take the shape of fastening two panels symmetrically, as shown in Fig.7. The cross section of the unbonded plate is rectangular (45 x 8.6mm), but its width becomes wider (75mm) at the ends to prevent buckling. The thickness of the panel is 83mm. Light weight concrete is used for the panels and its strength is 283 kgf/cm^2. Horizontal shear loadings are applied at the top of the columns. The bottom of the columns are fixed to the basement of the testing apparutus by pin-joint.

RESULTS OF TEST-2

Shear force-deformation relationships are shown in Fig.8. This figure indicates that these specimens have excellent ductility and good energy absorption capacity. After the unbonded plates yield, the bearing capacity of the system continues to increase further. The reduction of stiffness is mainly due to yielding of the beams. The ultimate failure of both frames were occured by the shear buckling of the beams.

ANALYSES

The test frames were analyzed using the finite element model. The model of No.2- frame is shown in Fig.6 by broken lines. The beams and columns were modeled by the elements that possess flexural and axial stiffnesses. The unbonded plates and the concrete panels were modeled by vertical and diagonal elements respectively, that have only axial stiffness.(1) Moment-rotation diagram and interaction diagram of the beams were approximated bilinear.(2) The stiffness of the diagonal element of the panel shown in Fig.9 was obtained by the tests, which is similar to TEST-1, on 1/2-scale models of the shearwall carried out separately. One of the results of the analyses is shown in Fig.10 with the experimental result. This figure shows that these results are in good agreements in initial stiffness, maximum strength and collapsing progress. Fig.11 shows an example of bending moment distribution obtained by the analysis and the experiment. This shows excellent agreements between these results. Therefore the behavior of frames with the shearwall can be predicted accurately by using this model.

CONCLUSIONS

The test results indicate that the proposed shearwall has following good structural characteristics.

1) Since the pattern of the stress distribution in the shearwall is rather simple and strength of the shearwall are determined only by cross sectional area of the unbonded plate, the correct earthquake resistant capacity of the shearwall is evaluated easily.

2) Stiffness of the shearwall is affected mainly by stiffness of the unbonded plate, i.e. cross sectional area and length of the plate.

3) After yielding of the unbonded plate the seismic shear force is not transferred to the panel any further. Thus undesirable failure of the shearwall can be avoided and exceedingly flexural elasto-plastic behavior with good energy absorption capacity can be achieved.

4) The behavior of frames with the shearwall can be predicted more accurately by replacing the shearwall with a simple truss model.

REFERENCES

(1) Bruce F.Maison and Egor P.Popov "Cyclic Response Prediction for Braced Steel Frames" Journal of the Structural Division, ASCE, July 1980

(2) Stavros A.Anagnostopoulos "Post-Yield Flexural Properties of Tubular Members" Journal of the Structural Division, ASCE, September 1979

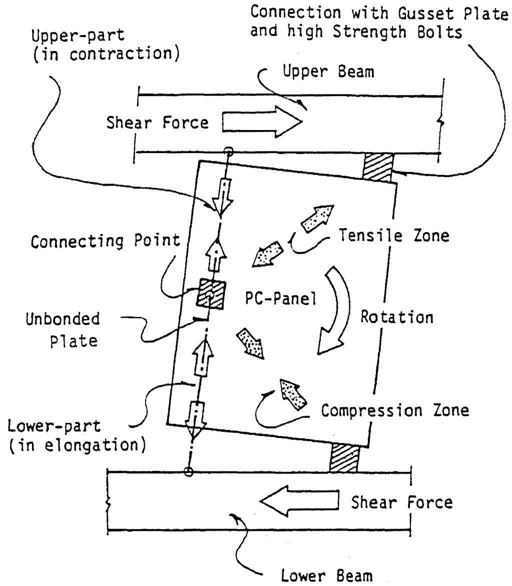

FIG.1
MECHANISM OF SHEARWALL

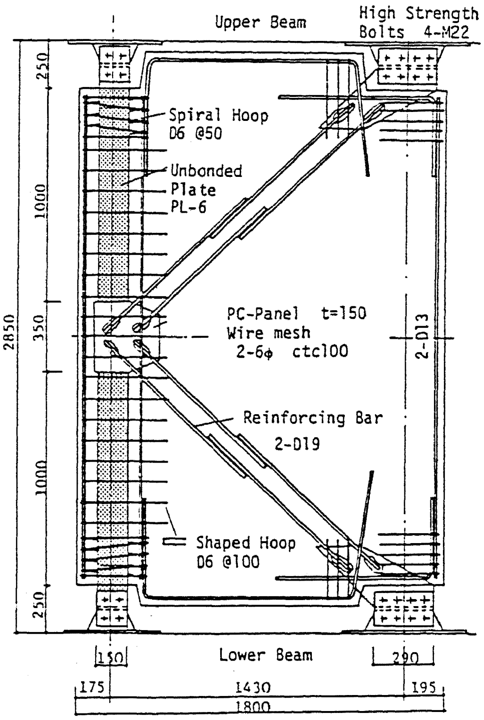

FIG.2
FULL-SCALE TEST SPECIMEN

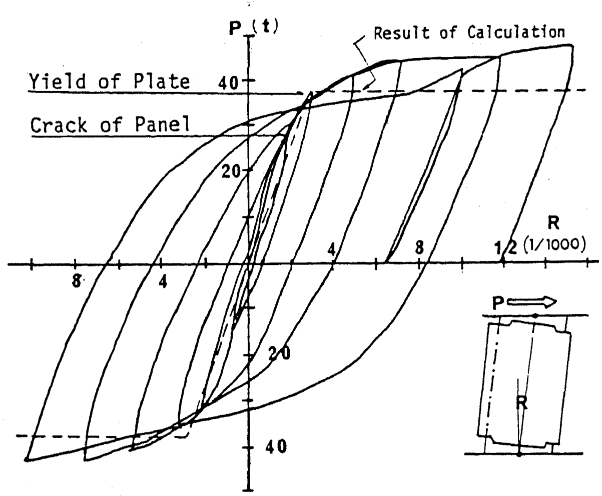

FIG.3
SHEAR FORCE-DEFORMATION DIAGRAM

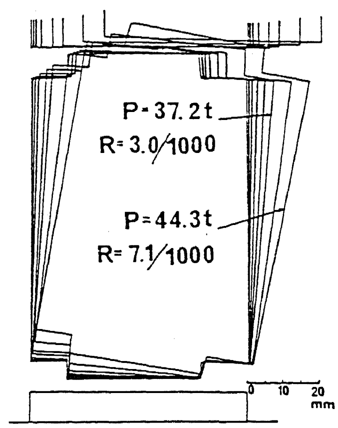

FIG.4
PATTERN OF DEFORMATION

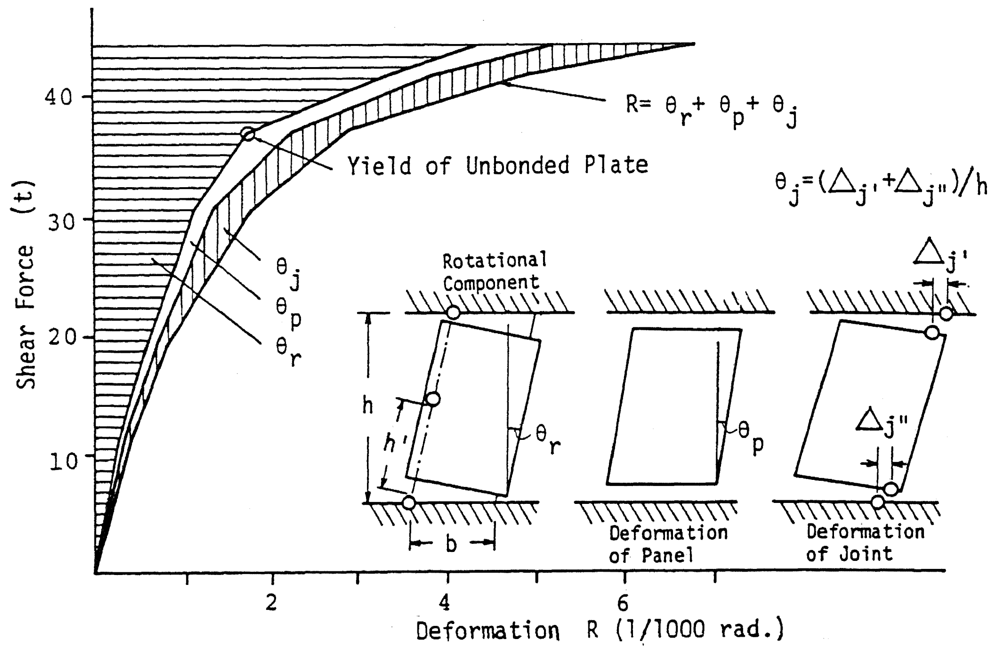

FIG.5 COMPONENTS OF DEFORMATION

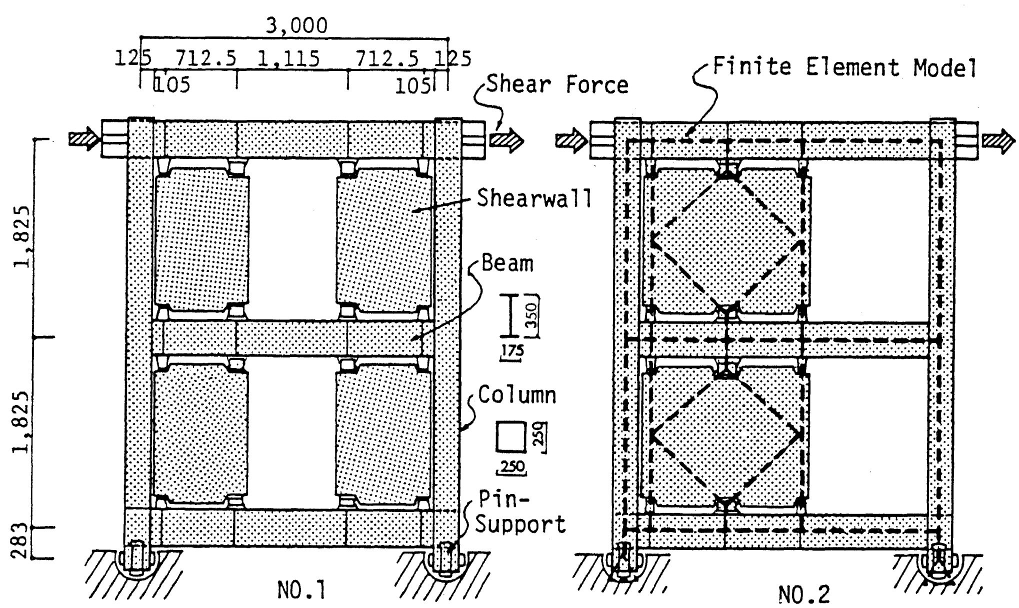

FIG.6 FRAME SPECIMENS

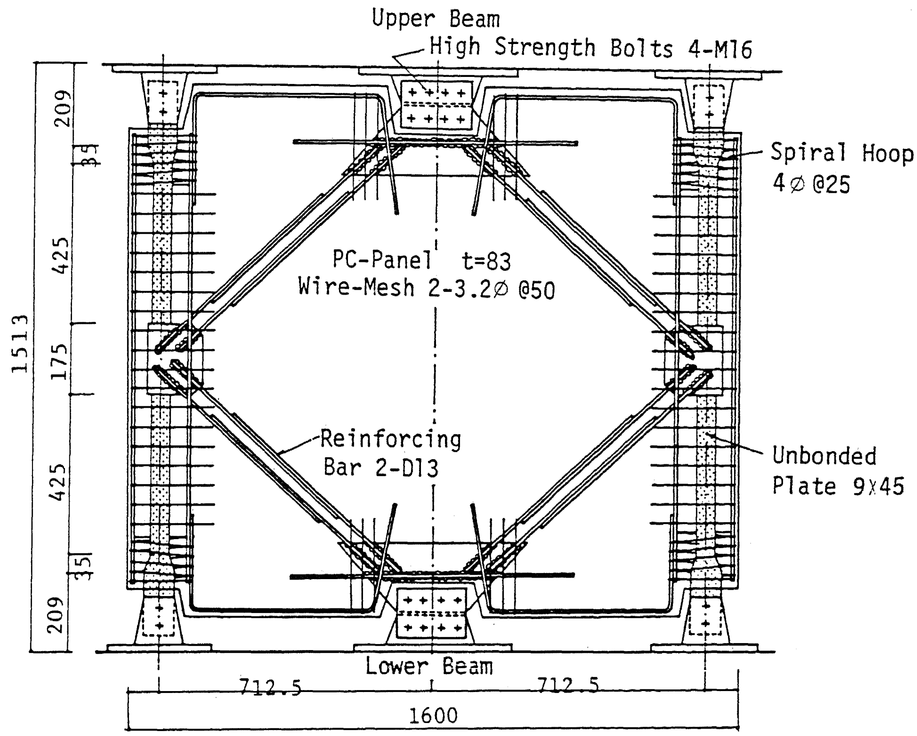

FIG.7 PANEL FOR NO.2-FRAME

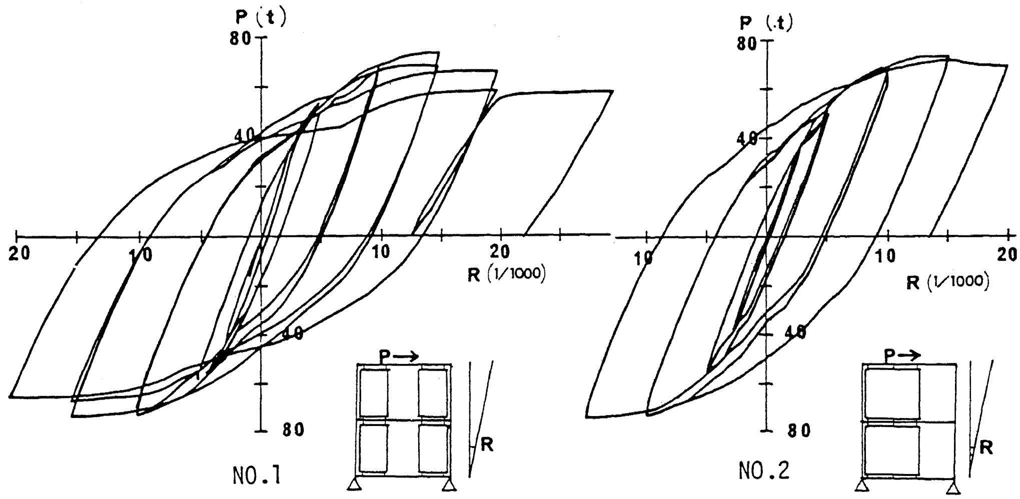

FIG.8 SHEAR FORCE-DEFORMATION DIAGRAMS

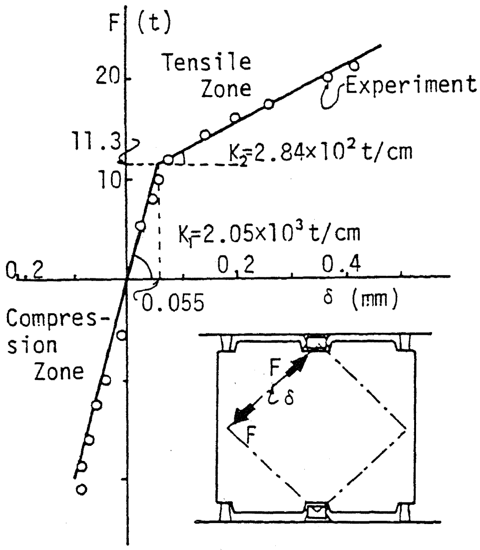

FIG.9
STIFFNESS OF DIAGONAL ELEMENT

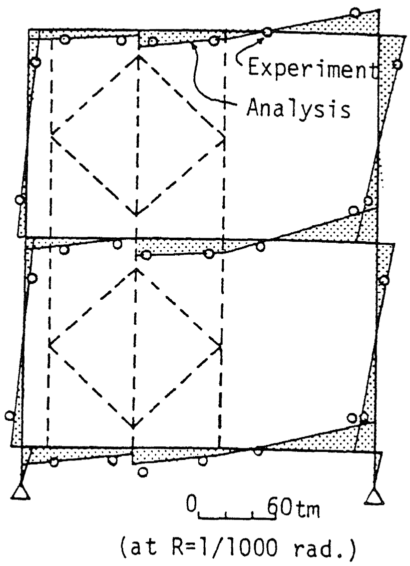

FIG.11
DISTRIBUTION OF BENDING MOMENT

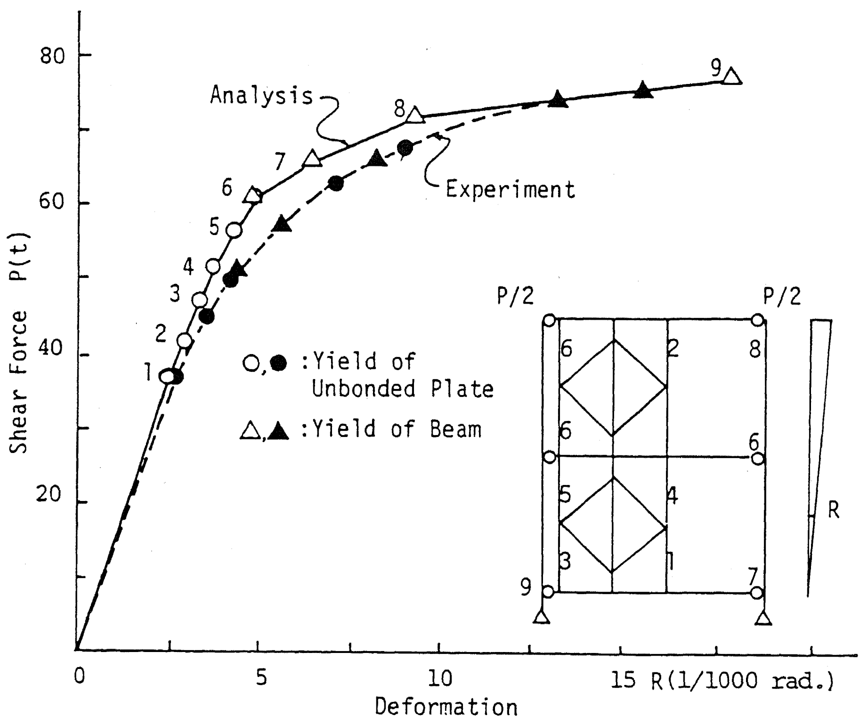

FIG.10 SHEAR FORCE-DEFORMATION
DIAGRAM AND COLLAPSING PROGRESS