INELASTIC BEHAVIOR OF LOW-RISE STEEL FRAME BASED ON A WEAK BEAM-TO-COLUMN CONNECTION PHILOSOPHY TO EARTHQUAKE MOTION

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SUMMARY

Weak-panel-strong-column frame, in this paper, with wide-flange steel members based on an aseismic design concept is fundamentally suggested. Deterministic response analysis on the inelastic behavior of such frames is performed. The discussion is mainly placed on the relation between the dissipated plastic energy quantity (called as "damage") distribution and the yield story-shear coefficient distribution over the height of the frames characterized by the panel-column yield strength ratio in a variable.

INTRODUCTION

It is known that the total input energy quantity per unit mass caused an earthquake motion to a frame is almost constant wherever the plastic hinge formations develop in the frame. This implies that if the input energy is unsuitably distributed over the height of the frame, damage concentration in excess of the energy absorption capacity of relatively weak stories into them would happen with less contribution of the energy absorption in the other stories of the frame (Ref. 1). When dealing with frames in aseismic design, therefore, both sufficient energy absorption capacity and reasonable dynamic characteristics, which is provided for the frames so that the input earthquake energy is effectively distributed, should be simultaneously take into account in their mutual relation.

In general, the plastic deformation of members such as beams and columns is employed to secure the energy absorption capacity of frames. The plasticity of columns in frame under earthquake motion tends to induce damage concentration without the suitable allotment of the input energy, such as was actually experienced in the four-story steel frame concentrately damaged into the 3rd story to Miyagiken-oki earthquake(1978) in Japan. The phenomenon may be attributed to the fact that the yielding of only columns in the frame results in no story-story deformation coupling. In this sense, especially, weak-beam-strong-column philosophy dependent on the plastic energy absorption of only beams is rational and recommended because the excessive damage concentration might be effectively avoided.

However, it seems that in the practical design the realization of the weak-beam-strong-column frame is not always easy and economical, because column strength more than 1.6 times beam strength is required as a reasonable balance in the seismic response (Ref. 2), besides the complete plastic hinge formations may be difficult in beams to which continuous concrete slabs attach. Furthermore, the large deformation of beams over its elastic limit induces local or lateral buckling of the beams. They are apt to give a limit to the stable inelastic behavior or the energy absorption capacity of the frames.

On the other hand, beam-to-column connections, so-called panel zones, in frames generally have stable and ductile restoring force characteristics, as well known, of their shear deformation. Hence, the panel zones in frames might

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serve the source for the plastic energy absorption as aseismic structural elements based on the more positive utilization. The energy absorption capacity of frames with elastic members dependent on the plastic deformation of only panels, namely herein weak-panel-strong-column frames, was already discussed on the static behavior (Ref. 3). In this paper, we focus on the another problem, damage distribution, in the two basic requirements as previously mentioned for the rational aseismic frame design.

BACKGROUND OF ASEISMIC DESIGN CONCEPT

The weak-panel-strong-column frames with plastic panel zones, elastic beams and columns up to the maximum strength are suggested as being basically expected
(1) to relax somewhat of the restriction on the width-to-thickness ratio of wide-flange sections due to no plastic hinge formations in beams and columns.
(2) to ease the lateral support design against inelastic lateral buckling of beams and columns, in addition
(3) to avoid damage concentration as well as weak-beam-strong-column frames during strong earthquake motions.

DAMAGE DISTRIBUTION OF UNIFORM SHEAR-BODY

For the comparison with the damage distributions of the weak-panel-strong-column frames investigated later on, the damage distribution of a uniform shear-body which is regarded as an idealized frame as shown in Fig. 1 is analized. In the uniform shear-body, the damage distribution over the height can be evolved as follow.

$$E_i/E_o = \sqrt{(1-(i-1)/N)^3} - \sqrt{(1-i/N)^3} \qquad (1)$$

where E_i = plastic energy quantity dissipated in i-th segment
E_o = total plastic energy quantity dissipated
N = number of the segments divided in the system

In the analytical derivation of Eq. (1), the followings are assumed.
(1) Maximum shear force distribution is predicted by root-mean-square method in the modal response analysis.
(2) Velocity response is constant for any modes of the system.
(3) Cyclic behavior is replaced to monotonic.
(4) Restoring force characteristics is elasto-perfectly-plastic.
(5) Ductility factors are same in all elements in the system.
(6) The system is divided into N segments along the height so that it corresponds to N-story frame.

According to Eq. (1), for example, the damage distribution in percentage over five segments (N=5) of the system is numerically given as listed in Table 1.

A CONDITION FOR FORMATION OF WEAK-PANEL-STRONG-COLUMN FRAME

The restoring force characteristics of panel zone is assumed to be tri-linear type on the nondimensional shear stress-strain relation in the analysis as shown in Fig. 2. The model is based on the experimental results (Ref. 4).

At first, let us introduce the next parameter α, termed panel-column yield strength ratio, to primarily provide for the weak-panel-strong-column frames to strong earthquake motions.

$$\alpha = {}_pM_y/\Sigma_c M_p \quad (={}_pM_y/\Sigma f\cdot{}_cM_y) \qquad (2)$$

where ${}_pM_y$ = yield panel moment ($=t\cdot D_b\cdot D_c\cdot \tau_y$)
$\Sigma_c M_p$ = sum of full plastic moments of columns attached to a panel
$\Sigma_c M_y$ = sum of yield moments of columns attached to a panel
f = shape factors

In case of frames with relatively small yield strength of panels to members, generally, the members start yielding after progressing of plastic deformation of the panels. Employing beam-column subassemblage with α approximately modeled from the condition existing at any floors in frames subjected horizontal loads as shown in Fig. 3, the value of panel ductility γ/γ_y when the yielding at first occurs with the columns can be obtained as

$$(1 - \zeta - \eta)/((1 - \zeta)((\gamma/\gamma_y - 1)H' + 1)) = \alpha \qquad (3)$$

which is illustrated in Fig. 4 on H'=0.05. It can be understood that plastic panel frame with elastic members carrying into the maximum strength can be theoretically realized, if we provide the frame with

$$\alpha < 0.42 \qquad (4)$$

as the necessary and the most compatible condition.

OUTLINE OF METHOD OF EARTHQUAKE RESPONSE ANALYSIS

Finite element technique together with consistent mass model is applied to the response analysis of frames. Elastic plastic stiffness matrices of panel zone and member are alternatively formulated in terms of these structural element models which have four nodes and eight degrees of freedom as shown in Fig. 5. In their formulations, the main assumptions are the followings.
Panel zone:
(1) Shear stress-strain relation is the tri-linear model.
(2) Bauschinger effect based on Prager-Ziegler kinematic model (Ref. 5) is taken into account.
(3) Yield is von Mises rule.
Member:
(1) Bending, shear and axial deformations are taken into account.
(2) Geometric nonlinearity of columns to account for P-delta effect is considered.
(3) Prager-Ziegler kinematic rule is retained in the generalized hinge method with the strain hardening coefficient 0.02.
(4) Yield curve is the approximate interaction between axial force and bending moment such as in Fig. 6.

Proceeding with the synthesis of elastic plastic stiffness matrix [K] of overall frame from the individual stiffness matrix of the structural elements, we can get, in general incremental form,

$$[M]\{\Delta\ddot{x}\} + [D]\{\Delta\dot{x}\} + [K]\{\Delta x\} = -[M]\{r\}\Delta\ddot{y} \qquad (5)$$

$\{\Delta x\}$ = incremental displacement vector
$\{r\}$ = vector with unit and null in horizontal and vertical components, respectively

wherein [M] is composed of the consistent mass matrices of panels and beams, [D] is assumed the linear combination, Rayleigh damping model, of the elastic stiffness and the mass matrix with damping constant 1% for both the fundamental and second natural frequencies of the frame.

Eq. (5) is solved by step-by-step direct integration used Newmark's β method. In the computer programming time increment between 1/100 and 1/2000 sec is

automatically changeable according to the yieldings and the reversals.

El Centro(1940 NS) and Taft(1952 EW) earthquake accelerograms magnified their peak accelerations as listed in Table 2 are used in the scalar $\Delta\ddot{y}$ in Eq. (5).

ESTIMATION OF DAMAGE QUANTITY OF FRAME

Premultiplying Eq. (5) by the velocity vector, and integrating with respect to time, we get the energy equillibrium equation of the overall frame. Then, the damage quantity W_R in the frame is estimated by the following concerned with the third term in left hand side in Eq. (5).

$$W_R = \int_0^{t+\Delta t}\{\dot{x}\}^T\{F_R\}dt = 1/2\cdot\Sigma\{\Delta x\}^T[\{F_R\}_t + \{F_R\}_{t+\Delta t}] \qquad (6)$$

where $\{F_R\}$ = restoring force vector

ANALYTICAL MODEL OF WEAK-PANEL-STRONG-COLUMN FRAME

Model Frames Five-story-one-bay model frame for the deterministic response analysis is selected as the typical low-rise story frame. The overall dimensions and the story dead loads are shown in Fig. 7. Five different panel shear strength distributions are provided for the frame. To begin with, the panel thicknesses of the one namely W0 among the five frames are decided by the design formula* and the lateral shear distribution factor A_i with the standard base-shear coefficient C_0=0.2 in the current Japanese code. Thereafter, the section properties of the beams and columns are designed so that W0 constitutes the weak-panel-strong-column frame with α=0.4, which is satisfied with the relation Eq. (4) in the preceding section. Thus, the model frame, W0, is basically followed with the current design provision of Japan except the ultimate-state design procedure. The next, in the other four frames, the panel thicknesses are decreased or increased under the same section properties of the beams and columns as W0 in order to concentrate the effect of the panel shear strength in the study. The constant section properties of the beams and columns in the five model frames are given in Table 3. The α values and the panel thicknesses in the five model frames named W0, W1, W2, W3 and S1 are summarized in Table 4. Herein, it should be noted that only S1 unsatisfies with the condition α<0.42. The panel thicknesses of S1 correspond to them designed by the allowable shear stress $f_s=\tau_y$ in this case. Table 5 shows the natural periods of all model frames.

Yield Story-shear Coefficient Distribution The damage distributions are most closely related to the yield story-shear coefficient distributions of the frames in general. The panel shear strength distributions provided for the model frames are equivalent to assign three different yield story-shear coefficient distributions to them as shown in Fig. 8; the ones are approximate to A_i, the others are respectively very apart from it. The dispersed distributions, which would result in the damage concentrations in ordinary frames, from A_i are consciously supposed that these may exist in actual frames, because no attention is paid to the panel shear strength distributions in the design practice.

The A_i in Japanese design code is similar to the shear coefficient distribution transformed from the seismic coefficient in UBC of USA, and to f(x) (Ref. 6) which is decided so that the response story ductilities are uniform along the height of lumped-mass system. The comparison of these distributions is shown in Fig. 9. In Table 6, the base-shear coefficients of the model frames are listed.

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*Herein, the allowable shear stress for panel is $4/3\cdot\tau_y$ (τ_y=yield shear stress)

RESULT OF ANALYSIS AND DISCUSSION

Maximum Story Drift The maximum story drift envelopes are shown in Figs. 10, 11 and Fig. 12. The figures indicate that the envelopes have the difference of no consequence, that is, no concentrated large drift into the relatively weak stories is arised, whereas providing the heavily different yield story-shear coefficient distributions for the frames. However, the larger the panel shear strength in the frames is, the more the envelopes due to the two different earthquake motions scatter.

Panel Ductility The envelopes of the maximum shear deformation ductilities of the panel zones are shown in Figs. 13, 14 and Fig. 15. Herein, the plotted value in a floor represents the larger response value of a panel than the another panel in the same floor level of the frame. The distributions of the panel ductilities in the figures are smooth as a whole.

In the figures, we should notice that the maximum ductilities are under around $\gamma/\gamma_y=14$ in no excess of the stable limit $\gamma/\gamma_y=20$ in Fig. 2 in all of the model frames, in this case, to 400 gal peak acceleration of the earthquake motions. According to the static analysis of weak-panel-strong-column frames, the panel ductility $\gamma/\gamma_y=20$ yields the story drift to column height $\delta/h=1/40\sim 1/20$. The relation between the dynamic response in the figures and the story drift to column height in Figs. 10, 11 approximately corresponds to the static relation in the linear interpolation.

$\delta/h=1/40\sim1/20$ would be an aseismic index to get desirable dynamic behavior of the weak-panel-strong-column frames in the stable region without the decay of the restoring forces of stories.

Damage Distribution The distributions of the dissipated plastic energy quantity estimated by Eq. (6), which are represented by the percentage of the quantity in each story to it in the overall frame, are shown in Figs. 16, 17 and Fig. 18 for W0, W1 and W2, W3, and S1, respectively. The damage distribution, Eq. (1), of the uniform shear-body is also plotted in the figures.

Eq. (1) nearly lies between W0 and W1 in Fig. 16. Especially, the damage distributions of W0 are more close to Eq. (1). The comparison of W1 with W0 gives that the upper stories of W1, in which α values are smaller than them in W0, absorb the input earthquake energy relatively more than the 1st story, as a matter of course. The time histories of the dissipated energy rate of the beam, columns and panels composed of the 1st story in W0, W1 and S1 are shown in Figs. 19, 20 and Fig. 21, respectively. Apparently, the panels in W1 absorb more than them in W0 on the rate. This implies that if there is no plastic energy absorption capacity of the 1st story columns concerned with width-to-thickness ratio and so on, the aseismic constitution such as W1 with smaller values α rather than W0 might be desirable because of the unavoidable plastic hinge developments at the column bottoms.

W2, W3 with relatively weak panels respectively located at the 2nd and 3rd, and 4th floor tend to absorb slightly more the input earthquake energy in the neighbour related stories than in the other stories, as shown in Fig. 17. However, less damage concentrations arise; besides, these damage distributions are similar to Eq. (1). Thus, the damage concentration into the relatively weak stories in the weak-panel-strong-column frames, even if which have the extremely dispersed yield story-shear coefficient distributions, is effectively reduced by one-story to next-story coupling spreading the coupling effect to the other stories. Furthermore, the figures for the weak-panel-strong-column frames show that the differences of the damage distributions caused to the two different earthquake motions are less.

On the contrary, as can be seen in Fig. 18, the damage distributions of S1 seem to be scattered more dependently on the characteristics of the earthquake motions. Only the 1st story in S1 to El Centro dissipates the plastic energy near 50% of the total quantity in the overall frame in spite of providing the most suitable yield story-shear coefficient distribution. Due to the larger shear strength of the panels in S1 with α=0.53(0.42), the 1st story columns absorb the input earthquake energy more with the less energy absorption of the panels as shown in Fig. 21. The result may give that no reason we should always provide strong panels for frames exists.

CONCLUSION

The fundamental situation about the weak-panel-strong-column frames is mentioned at first in this paper. Thereafter, the inelastic response behavior especially the damage distribution of such frames to strong earthquake motions is investigated. The followings can be emphasized in the study.

(1) For the suitable damage distribution of frames, relatively strong panels to column strength in the frames are not always needed.

(2) In the weak-panel-strong-column frames suggested herein, the dissipated plastic energy quantity is adequately distributed over the height, almost independently on the yield story-shear coefficient distributions.

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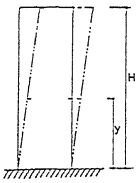

Fig.1 Uniform shear-body

Table 1 Damage distribution of uniform shear-body

Story	Damage Rate
5	8.9
4	16.3
3	21.2
2	25.1
1	28.5
Total	100.0(%)

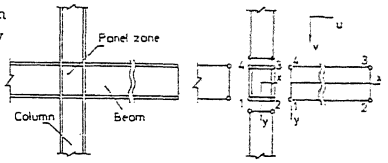

Fig.5 Structural element models

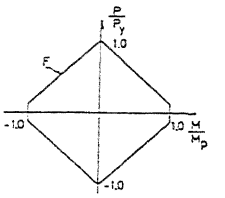

Fig.6 Axial force-bending moment interaction

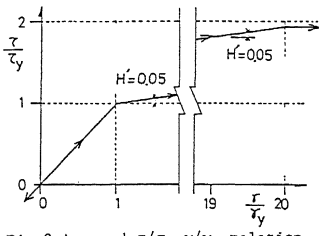

Fig.2 Assumed τ/τ_y-γ/γ_y relation

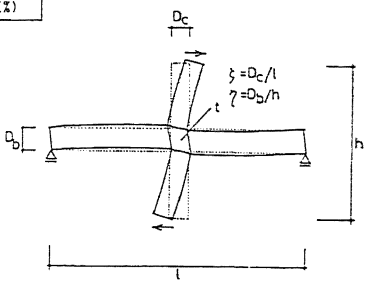

Fig.3 Beam-column subassemblage

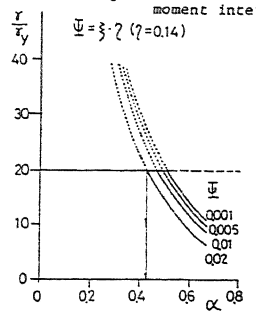

Fig.4 γ/γ_y-α relations

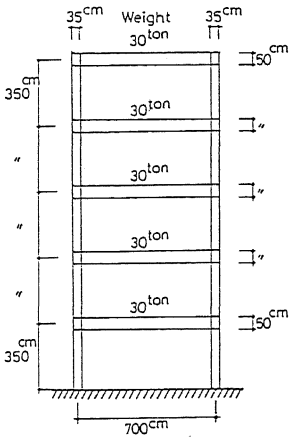

Fig.7 Overall dimensions and story dead loads of model frame

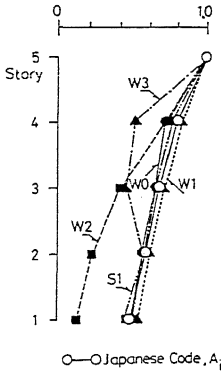

Fig.8 Yield story-shear coefficient distributions

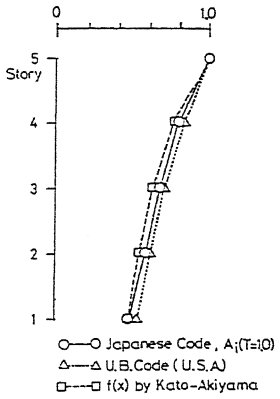

Fig.9 Story-shear coefficient distributions

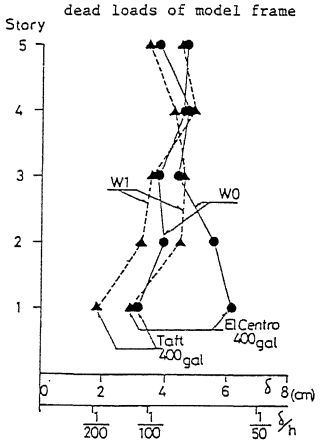

Fig.10 Maximum story drifts(W0,W1)

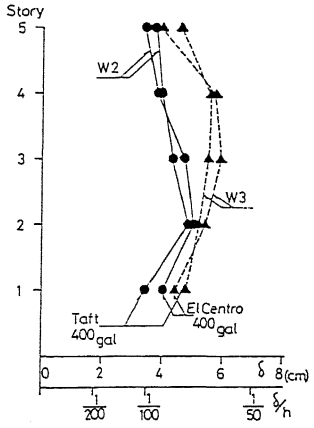

Fig.11 Maximum story drifts(W2,W3)

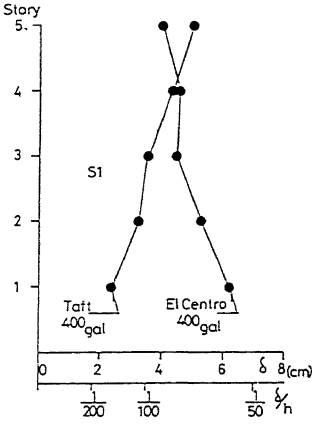

Fig.12 Maximum story drifts(S1)

Table 2 Earthquake motions input

	El CEntro (1940,NS)	Taft (1952,EW)
Max. Acc.	400 gal	400 gal
Dur.	6 sec	8 sec

Table 5 Natural periods of model frames (sec)

Name / Modes	W0	W1	W2	W3	S1
1	1.075	1.115	1.116	1.137	1.065
2	0.372	0.382	0.379	0.387	1.370
3	0.196	0.199	0.198	0.202	0.194

Table 6 Base-shear coefficients of model frames

Name	W0	W1	W2	W3	S1
$_yC_b$	0.15	0.07	0.04	0.15	0.20
$_mC_b$	0.46	0.38	0.30	0.46	0.49

$_yC_b$---Coefficient at yielding of panel.
$_mC_b$---Coefficient at $2\tau_y$ of panel and full plastic moment of column.

Table 3 Section properties of members

Name	W0, W1, W2, W3, S1			
Story	I (cm4) Column	I (cm4) Beam	M_p (t·cm) Column	M_p (t·cm) Beam
5	15200	36500	4250	4250
4	30400	36500	4480	8730
3	42600	48600	5290	9770
2	60800	73000	6070	11360
1	60800	73000	6070	12140

I ··· Sectinal Moment of Inertia
M_p··· Full Plastic Moment

Table 4 α values and panel thicknesses

Name / Floor	Weak Panel Frames W0	W1	W2	W3	—— S1
Top	0.4 (0.7)	0.343 (0.6)	0.4 (0.7)	0.4 (0.7)	0.53 (0.9)
5	0.4 (1.3)	0.277 (0.9)	0.4 (1.3)	0.4 (1.3)	0.53 (1.7)
4	0.4 (1.6)	0.300 (1.2)	0.4 (1.6)	0.3 (1.2)	0.53 (2.1)
3	0.4 (1.8)	0.245 (1.1)	0.2 (0.9)	0.4 (1.8)	0.53 (2.4)
2	0.4 (2.0)	0.265 (1.3)	0.2 (1.0)	0.4 (2.0)	0.53 (2.6)

Values in ( ) stand for thickness of panel (cm)

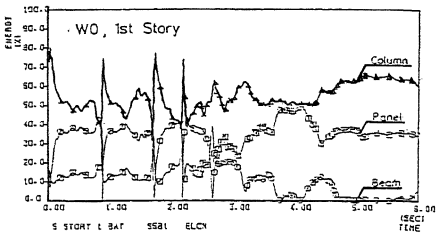

Fig.19 Time history of dissipated energy rate(W0)

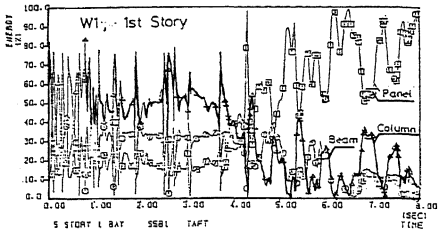

Fig.20 Time history of dissipated energy rate(W1)

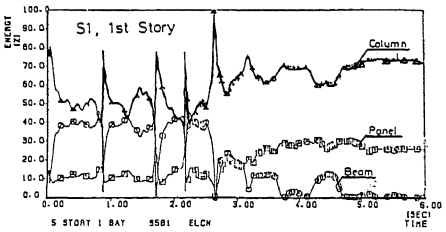

Fig.21 Time history of dissipated energy rate(S1)

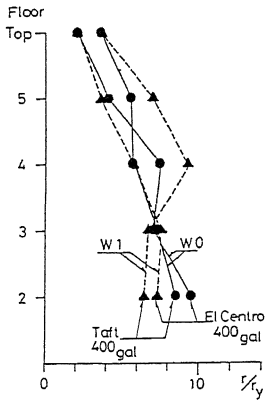

Fig.13 Panel ductilities(W0,W1)

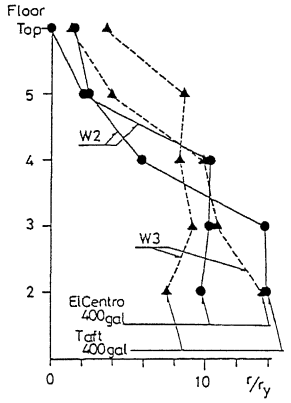

Fig.14 Panel ductilities(W2,W3)

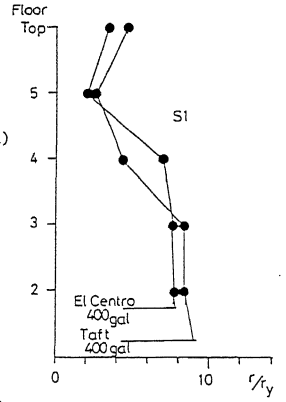

Fig.15 Panel ductilities(S1)

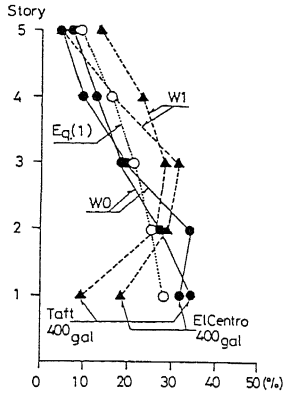

Fig.16 Damage distributions(W0,W1)

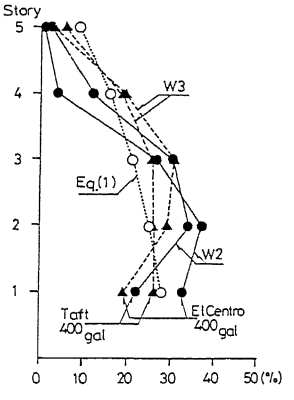

Fig.17 Damage distributions(W2,W3)

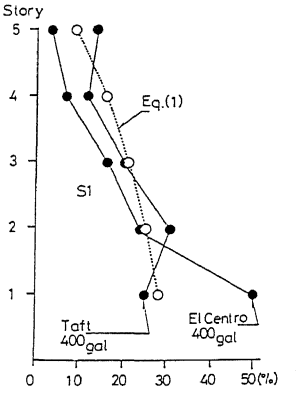

Fig.18 Damage distributions(S1)