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COMPOSITE MOMENT RESISTANT FRAME DESIGN - DUCTILITY DEMAND

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ABSTRACT

The effect of the shear-panel zone design on the seismic response behavior of the composite moment resisting building frame has been assessed. The shear-panel zones of a composite moment resisting frame (MRF) have been designed considering different criteria, in which the shear strength of composite shear-panel zone has been varied relative to the shear force in the shear-panel zone at the beam yielding level. Nonlinear dynamic analyses have been carried out for above building frames, with the numerical joint model correlated to full-scale pseudodynamic test results.

The results obtained from design-analyses investigations of the ductility demand indicate that the composite MRF shall be designed with the composite shear-panel zone considering the web-steel, the flange effect and the concrete contribution for the shear-force in this panel zone corresponding to the connecting beam yield level. Such a capacity aseismic design of the joint would result into a higher ductility capacity of the structure as a whole, enabling both beam and shear-panel zone to participate in nonlinear behavior. This would also prevent higher ductility demand on either of the two components, thus preventing an early failure of either element and the subsequent potential collapse of beam-column joint and the structure.

KEYWORDS

Composite building frame, beam-column joint, shear-panel, ductility demand, seismic response

INTRODUCTION

In order to assess the influence of the composite shear-panel design on the seismic response behavior of the composite moment-resisting building frame, a single frame of a 4 story (3.5 m story height) building was taken for the study. The 3-bay frame spaced at 5 m, with a total height of 14 meters (Fig. 1) was designed for the maximum EC 8 (Eurocode No. 8 1988) earthquake zone. The shear-panel zones of the above frame were designed considering different criteria. The shear strength of the shear-panel zone was varied with respect to the shear force in the shear-panel zone at the beam yielding level. Nonlinear dynamic analyses were carried out for the above building frames subjected to the ground motion time history of the 1986 Kalamata earthquake. The results obtained from the case study are critically discussed with reference to both the structural period and the story drift as well as the relative and the extent of plastifications in the shear-panels, the beams and the columns.

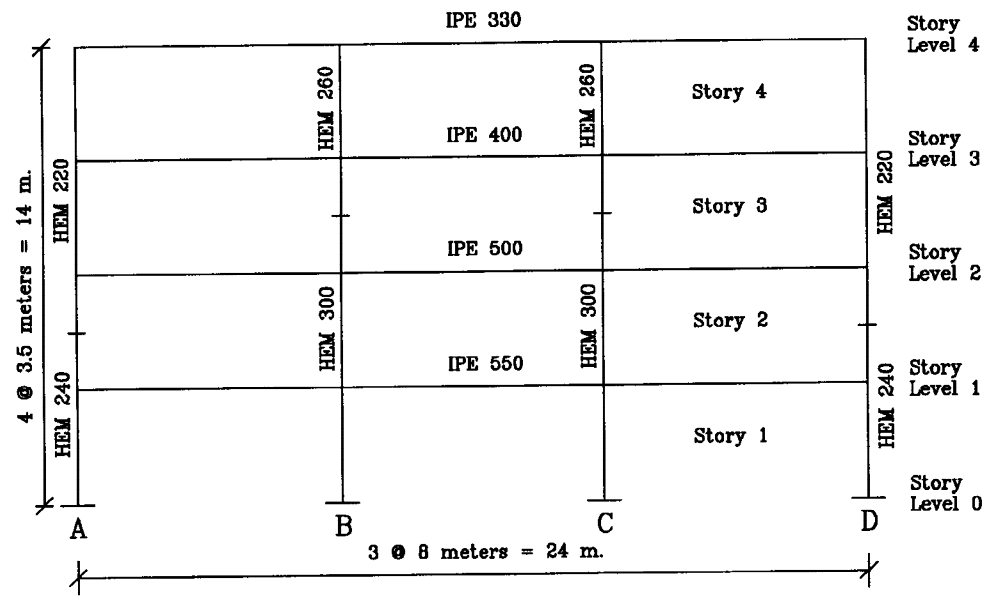

Fig. 1. Composite frame

COMPOSITE FRAME DESIGN

The building structural system employed the composite moment resisting frame with fully welded connection in the the major axis of the columns. In the other direction the beam to column connections were taken as shear connected. The lateral load resistance in this direction is provided by shear walls. The moment resisting frame, assuming fully symmetrical conditions, considered for the study is shown in the Fig.1. The composite beam and column sections are of European type with the reinforced concrete encased between the rolled-section steel flanges.

A single frame of an office building with a loading class of 2 kN/m^2 was designed with a reinforced concrete slab of 18 cm thickness connected to the partially encased composite beam sections through the shear-stud connectors. The member stiffnesses were modelled assuming the the filled-in concrete provides 25 % contribution of the total concrete and the composite slab action.

Seismic loading parameters are as follows:

Seismic zone coefficient 'α' = 0.33
Importance factor 'I' = 1
Soil type = B
Behavior factor 'q' = 6
Assumed damping of 3 %.

Equivalent lateral load analysis was carried out using the program SAP 90 (Wilson and Habibullah 1988) to arrive at the forces for design of members. Several iterations of analysis and design were carried out to obtain the optimum frame design, fulfilling both the interstory drift requirement (within 1.5 %) and the strong column - weak beam criteria (i.e. the sum of the plastic moments of the columns at the joint to be greater than the sum of the beam plastic moments). The designed member sections are shown in the Fig. 1. The standard rolled steel sections St.37-2 and the concrete of grade C 25 were used for the design.

COMPOSITE SHEAR-PANEL DESIGN

To assess the influence of the composite shear-panel zone action on the seismic response behavior of composite moment-resisting building frame, the following 4 joint-design cases have been studied:

Case 1 - The shear-panel zone remained elastic. The shear-panel zone strength at the first yielding (Q_1,, Fig. 2) was designed at 1.33 times the beam-yielding shear-level (Q_{by}).

Case 2 - The shear-panel zone was allowed to yield. The shear-panel zone strength at the first yielding (Q_2, Fig. 2) was designed at the beam-yielding shear-level (Q_{by}).

Case 3 - The shear-panel zone was allowed to yield. The shear-panel zone strength at the second yielding (Q_3, Fig. 2) was designed at the beam-yielding shear-level (Q_{by}).

Case 4 - The shear-panel zone was allowed to yield. The shear-panel zone strength at the second yielding (Q_4, Fig. 2) was designed at 0.75 times the beam-yielding shear-level (Q_{by}).

Q_{by}, the shear force in the shear-panel zone at the beam-yielding level, is defined as,

$$Q_{by} = (M_{by}/d_b) - Q_{sc}$$

where, M_{by} is the beam-yield moment,

d_b the depth of the beam about flange centers and

Q_{sc} the shear force at the column.

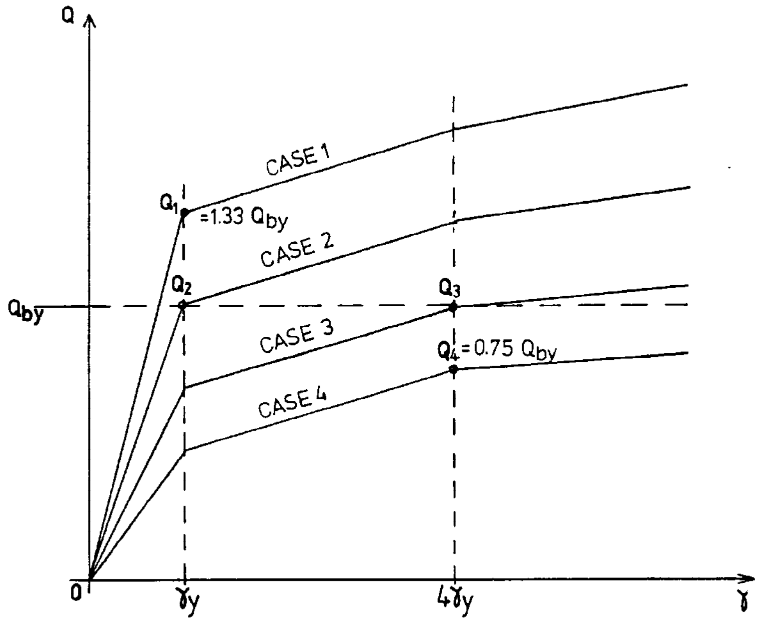

Fig. 2. Shear-panel zone design criteria

NONLINEAR DYNAMIC ANALYSES

The nonlinear dynamic analyses were performed for different joint cases using the program DRAIN-2DX (Prakash and Powell 1992). The structure was modelled in 2-dimensions as an assembly of nonlinear beam-column elements (beams and columns) and nonlinear rotational connection springs at nodes (shear-panel zone). Bi-linear behavior was considered for the beam-column elements. The modelling of the shear-panel zone was done using three parallel rotational springs. Steel part was represented by two bilinear springs with inelastic unloading to obtain tri-linear behavior, including the characteristics of both the steel column-web and the column-flange bending effect. The concrete part was modelled with a 'bilinear elasto-plastic with gap' spring. The numerical joint model was correlated to the full-scale pseudodynamic test results (Pradhan and Bouwkamp 1994).

As the beam yield moment is governed by negative bending for both the clockwise and anti-clockwise bending of the beam, the strength contribution of the slab was discarded. However, the member stiffnesses were modelled assuming that the filled-in concrete provides 25 % contribution of the total concrete and the composite slab action. The degradation in strength of the concrete has been assumed to be compensated by discarding the strain hardening of the steel after the 4 γ_y shear-distortion level (i.e. the end line of the ultimate base curve of the shear-panel zone is assumed to be a straight line). Kalamata earthquake (1986) record was used as an input excitation.

The results of the analyses studies are critically discussed in the following sections, covering the structural period, story drift, story shear and extent of plastifications in the shear-panel, beam and columns.

Structural Period

The design of the shear-panel zone with different criteria led to different strength and stiffness of the shear-panel zone having consequent impact on the structural period of the frame as shown in the Tab. 1. From the above table it could be observed that the stiffness of the frame decreases with the consequent increase in the period of the structure as one goes from Case 1 to Case 4 (i.e. from elastic shear-panel zone to weaker shear-panels). However, the results indicate that the shear-panel designed as per Case 2 did not decrease the stiffness of the structure appreciably compared with Cases 3 and 4. The fundamental period of the structure is the mostly affected one compared to the periods at the higher modes.

Table 1. Period of the structure

Cases	Mode 1	2	3	4
Case1	0.810	0.310	0.207	0.158
Case 2	0.821	0.313	0.207	0.158
Case 3	0.859	0.325	0.211	0.159
Case 4	0.893	0.333	0.212	0.159

Story Drift and Story Shear

Maximum story drift for each story is shown in the Fig. 3. It could be noted that in the cases with the weaker shear-panel zones (Cases 3 and 4), the story drift is smaller than in the cases with the strong panel zones. This indicates that the structure with the weaker shear-panel zone inspite of being flexible, shows less drift contrary to the expectation of higher drift level. This is due to the dynamic response of the structure for a given earthquake input. The above observation holds good for the structural periods lying beyond the peak of response spectrum of the input excitation. The reduced stiffness of the structure leads to a smaller force input to the structure. This could be noted from the results shown in the Tab. 2. depicting the base shear for each case.

Table 2. Maximum base shear

Cases	kN + ve	-ve
Case 1	1291.60	1295.50
Case 2	1275.40	1277.70
Case 3	930.80	1182.50
Case 4	993.80	1078.00

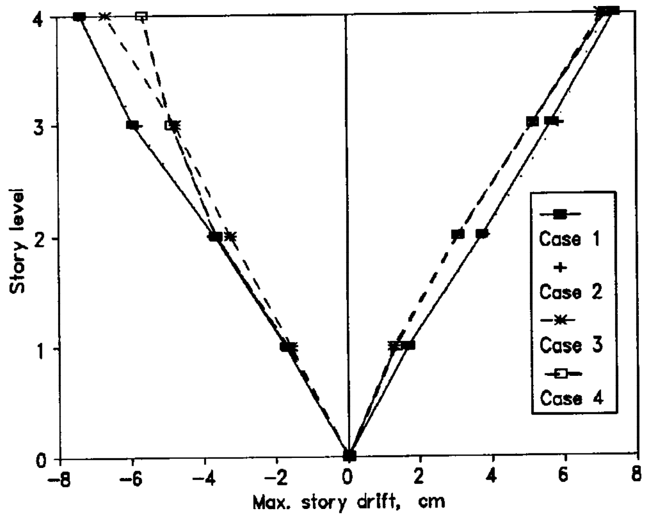

Fig. 3. Maximum story drift

It is observed that in all the cases the maximum drift ratio is less than 0.9 % (Fig. 4), i.e. within the limits of 1.5 % as stipulated in the code. In fact, during the PSD tests (Pradhan and Bouwkamp 1994) the cyclic response indicated that a drift ratio of 7 % could be accommodated without loss of stable hysteresis behavior of the joint. The story shear for the Case 1 and Case 2 (stronger shear-panel zones) is larger compared to the Case 3 and Case 4 (weaker shear-panel zones) (Fig. 5).

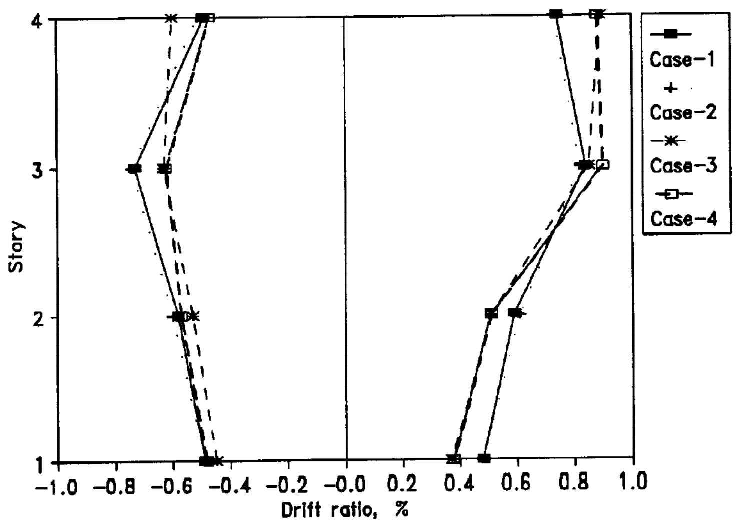

Fig. 4. Maximum drift ratio

Yielding and Damage

The potential damage due to earthquake to the beams, shear-panels and columns for the cases of different composite shear-panel design (Case 1 to Case 4) is discussed in this section.

For comparison, the extent of the damage and the sequence of yielding for all the 4 cases are shown in the Figs. 6 to 9, wherein:

a) the filled circles (•) indicate the extent of the damage in the beams and columns in terms of the average accumulated plastic hinge rotations,
b) the empty circles (o) indicate the extent of the damage in the shear-panels in terms of the average

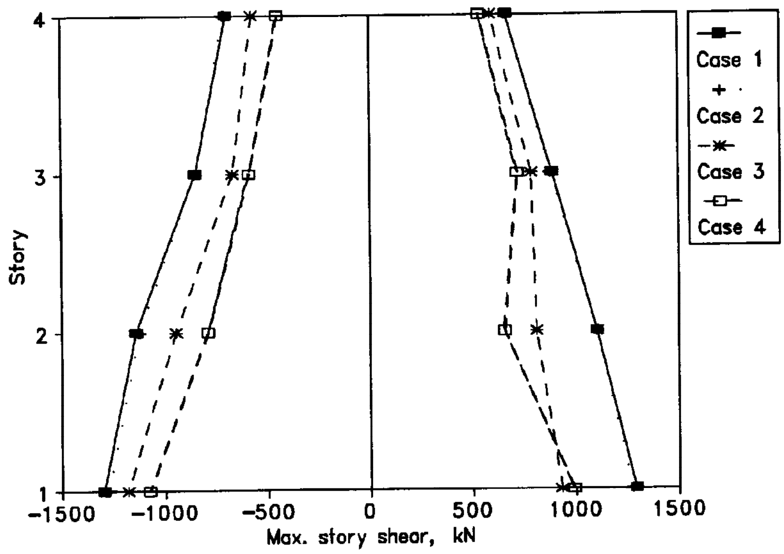

Fig. 5. Maximum story shear

accumulated plastic hinge rotations after the first yielding, and

c) the numbers indicate the first yielding time-range.

The results show that in Case 1 the shear-panels remained elastic. The yielding started within one second of the earthquake in the beams and in the first floor columns between 1 to 2 seconds (Fig. 6). In Case 2 no appreciable difference in both the location of the plastification and the extent of damage could be observed compared to Case 1, except that the third floor level shear-panels also plastified, but to a lesser extent with the delayed yielding of the first floor level beams (Fig. 7). In Case 3, the shear-panel zones in the second, third and the fourth floor level plastified, thereby reducing the level of damage to the beams (Fig. 8). In this case the columns remained elastic. In Case 4, yielding occurred in all the shear-panels except one of the panels, while first, second and third (almost) floor level beams did not yield (Fig. 9). Furthermore, average maximum beam-end rotations and the maximum shear-panel distortions for Case 3 and Case 4 are smaller compared to Case 1 and Case 2. The shear-panel yielding (causing larger distortions) results in a decrease in the rotational demand on the beam (smaller rotations).

With the above observations, it is could be concluded that in the first two cases (Case 1 and Case 2) a larger ductility demand on the beams resulted, while the shear-panel remained elastic. In Case 3 a uniform distribution of the yielding happened for both to the beams and the shear-panels with balanced ductility. But in Case 4, the damage is primarily in the shear-panels imposing less or no ductility demand to the beam. Hence, for the design of the composite shear-panel, the criteria as used in Case 3 is desirable since it leads to a balanced ductility demands in the shear-panels and the beams and thus preventing potential collapse of either one of the elements due to the excessive ductility demand. Also, the potential stress concentration on the connection welds would be reduced due the shear-panel yield deformations in the same direction as that of the beam-end rotation.

CONCLUSION

The design-analyses investigations of the ductility demand on the composite moment resistant frame show that the extent of the post-earthquake damage and ist distribution on the beams, the shear-panel zones and the columns are quite altered depending upon the shear-panel design criteria employed. It is recommended that the capacity design of the composite shear-panel zone is carried out taking into consideration the strength of the web steel, flange-effect and the concrete contribution to meet the beam-yielding shear level. Such a capacity design of the joint would result in higher ductility capacity of the structure as a whole and

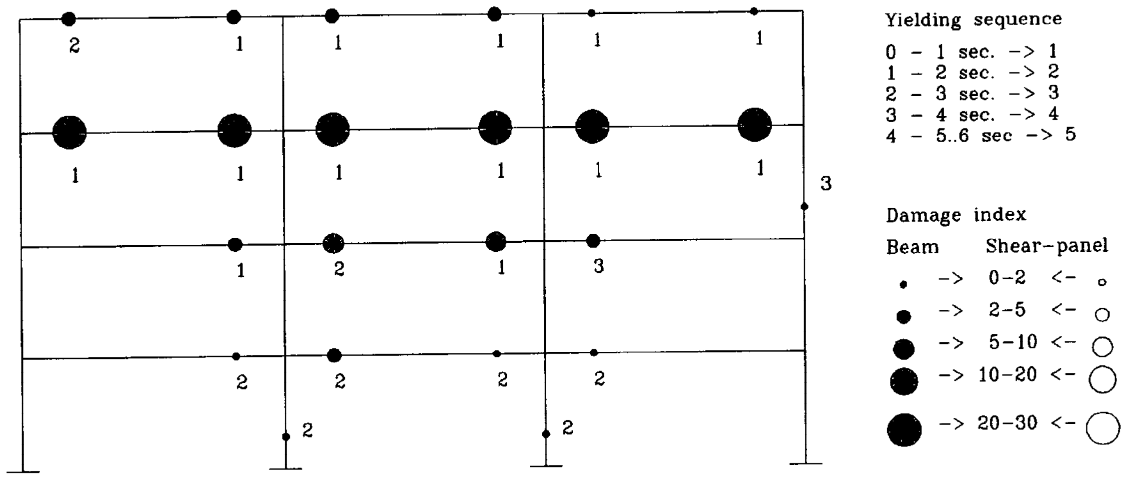

Fig. 6. Yielding and damage - Case 1

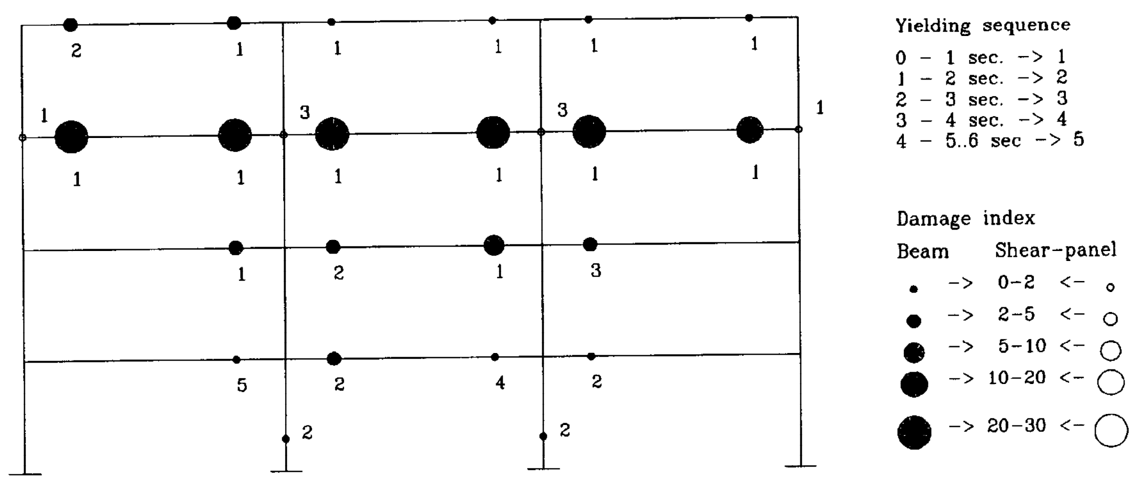

Fig. 7. Yielding and damage - Case 2

thus enabling both the beam and shear-panel to participate in nonlinear behavior. This would prevent the higher ductility demand on either of the two components, thus preventing an early failure of either the beam or the shear-panel zone. Also, this would prevent potential stress concentration on the connection preventing the potential collapse of the beam - column joint and the structure.

With due consideration to the building site soil (anticipated response spectrum) and the fundamental period of the structure, the above design approach could lead to an effective smaller base-shear input to the structure and smaller story drift. Further it is necessary to consider the contribution of concrete in the design of the composite joint. The concrete in the shear-panel zone has beneficial effects in the following way:
a) provides larger stiffness and strength (at elastic response level - wind and small earthquake), b) in moderate earthquake cases, the composite shear-panel participates in the energy dissipation along with the beam, and c) in large earthquake cases, the concrete would 'fuse-off'(crushing degradation) in the panel zone after a certain ductility level and then the reserve shear-panel web steel participates fully in the energy dissipation.

From the aseismic structural design point of view, it is highlighted that the composite moment resistant frame shall be designed to obtain a balanced distribution of ductility demand to both beams and shear-panel zones, for which the joint should be capacity designed as semi-rigid. Here, the semi-rigidity is obtained through the composite shear-panel zone.

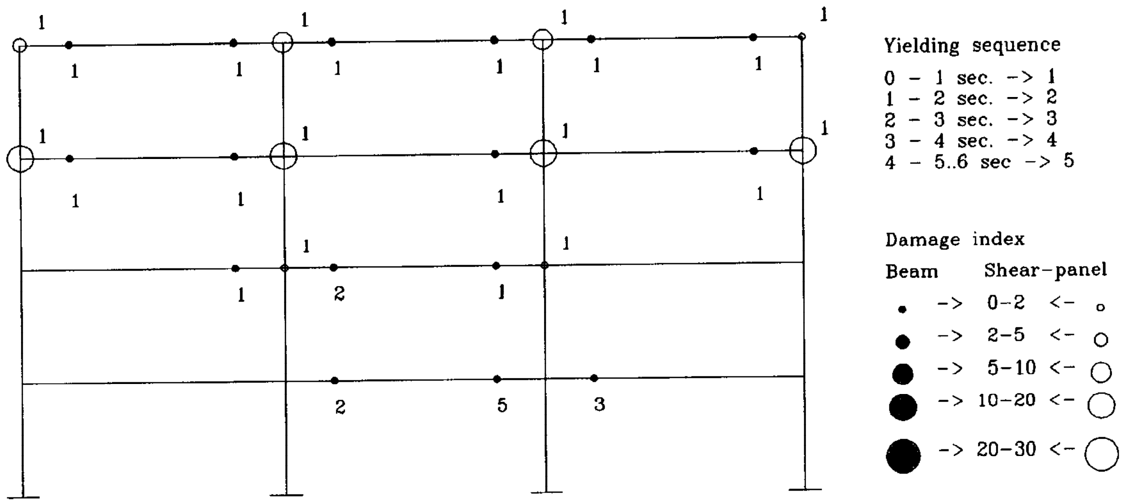

Fig. 8. Yielding and damage - Case 3

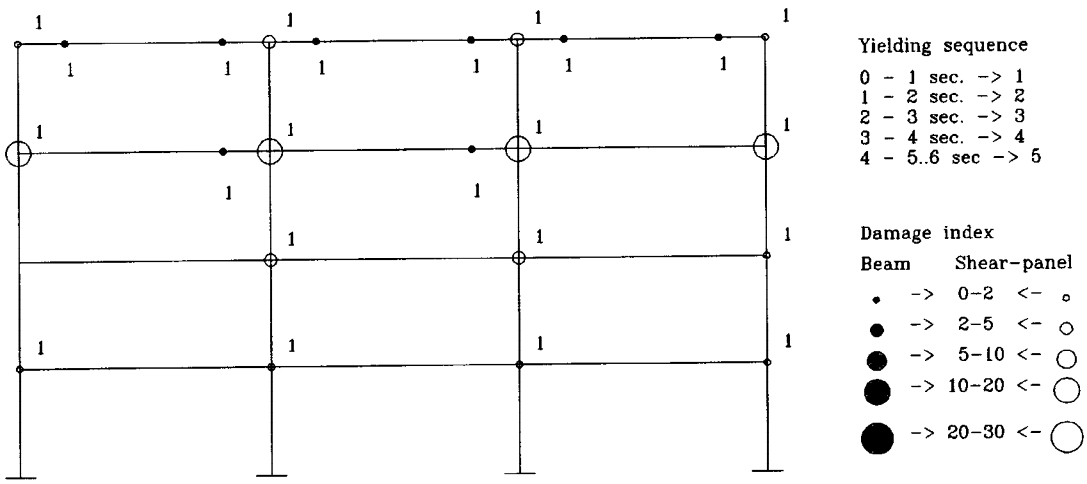

Fig. 9. Yielding and damage - Case 4

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