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SEISMIC BEHAVIOR OF PRECAST REINFORCED CONCRETE STRUCTURES

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ABSTRACT

Recently precast reinforced concrete (PCa) construction has been widely used even in seismic zones, where, in the past, their use was severely prohibited. One of the problems on this structural type is the connection between members, which can be a potential weak point that develops sliding failure mode in the member, and then an inadequate behavior of the structure. Therefore, in the design process of a PCa member or structure it should be considered important the sliding failure mode.

The purpose of this study is to establish a relation between a limit value for the sliding displacement in the construction joint of PCa members, and the maximum responses of the structural system under ground motion excitation. An allowable sliding displacement value for the construction joint is proposed, so that the structural responses could be within the limits of the values permitted by the design codes.

KEYWORD

Precast structures, sliding displacement, construction joint, non-linear behavior, seismic analysis, maximum responses.

INTRODUCTION

In precast concrete (PCa) moment resisting frames the construction joints perpendiculars to members' axis are indispensable, but it is potentially weak to sliding failure mode. Therefore, in design of PCa structures, the sliding failure mode should be taken into account. In the past, many direct shear tests have been studied in order to make clear the shear transfer mechanism along joints or cracks in reinforced concrete (RC) beam members. However, direct shear tests will not recognize the effect of stress distribution along the members with damage or failure near the connection. So that, it was developed an experimental program to investigate the effect of sliding along construction joint on stiffness and strength degradation, as well as shear transfer capacity, on PCa beam members under cyclic loading.

Besides, it was considered important to establish a relation between the behavior characteristics at construction joint of PCa beam members and the maximum responses of the whole PCa structural system. Until now there is almost no information about the maximum sliding displacement allowable at construction joints in terms of the maximum responses of the structural system. So, once the maximum allowable sliding displacement is selected, it will be easy to develop the formulation to know the strength in flexural and shear behavior taking into account the sliding at the construction joint.

In this study, based on experimental results on PCa beams with different types and characteristics of construction joints (López, et.al., 1992, Kusu, et.al., 1992), it is proposed a restoring force characteristics model to be used at the end of a two elements member model. Then, considering a regular building structure, it is done a parametric study considering different values for the restoring force model, and the corresponding values of sliding displacement at construction joint. Ground motions of very different characteristics are taken into account. Finally, there are defined relations between the values for the maximum responses of the whole structure, the restoring force characteristics of members, and the level of sliding displacement at construction joint.

From these results it is proposed a limit allowable value for sliding displacement on the construction joint of PCa beam members. So that, if during the design process of a PCa beam member, the sliding displacement is less than the allowable proposed value, it can be supposed that the maximum responses of the structural system will be within the limit values proposed by the seismic design codes.

EXPERIMENTAL RESULTS

To investigate the shear transfer mechanism for PCa elements with joint at mid-span, and with joint at the member-end, two series of PCa beams specimens (López, et al., 1992, Kusu, et.al., 1992) subjected to lateral load reversals were tested. The details of the connection were varied to consider the contribution of different possible shear transfer mechanisms. The specimens were designed to fail in shear within the beam, or at the joint vicinity. Shear force - member deformation, and the hysteresis characteristics at both ends of the whole member were studied and discussed, and herein only the most relevant results are presented.

All of the specimens with joint at mid-span, showed shear diagonal tension failure after the apparition of splitting crack along the longitudinal reinforcement, caused by the dowel action in both sides of the connection. On the other hand, specimens with joint at member-end failed in shear compression. Both series of specimens showed slip-type hysteresis shape. The representative hysteresis shapes are shown in Fig.1. Strength degradation and slip deformation are considered as the hysteresis parameters with more influence on maximum responses. Both parameters were studied, as well as their relation with sliding displacement at construction joint.

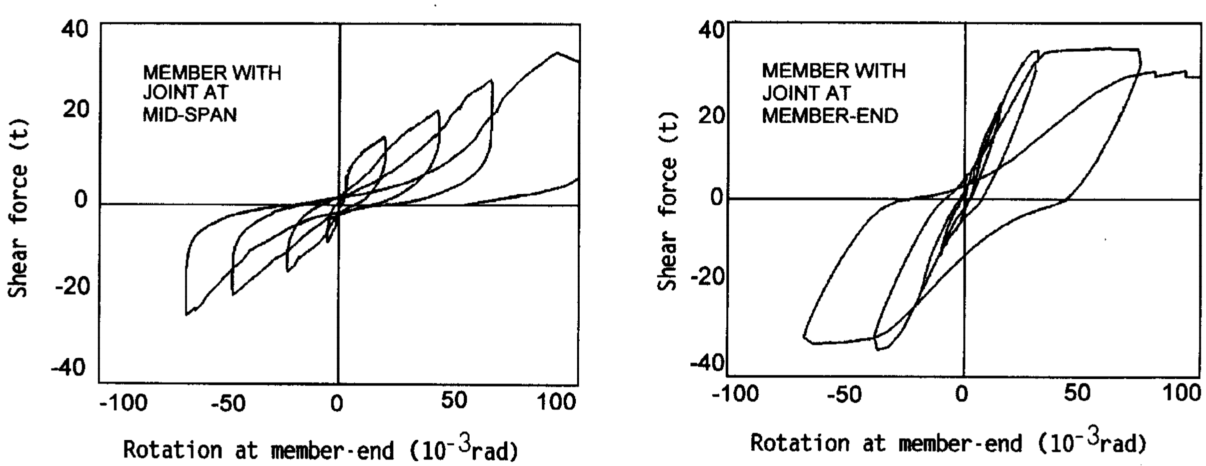

Fig.1 Representative hysteresis shapes of experimental results.

Strength degradation and slip deformation

Strength degradation was defined as the percentage of strength reduction for a peak displacement value with respect to the previous strength for the same peak displacement value (Fig.2)

$$\eta = F_{i+1}/F_i \qquad (1)$$

where: F_i is the strength at the peak of the i-th cycle, and F_{i+1} is the strength at the same displacement in which was measured F_i, for the i+1-th cycle.

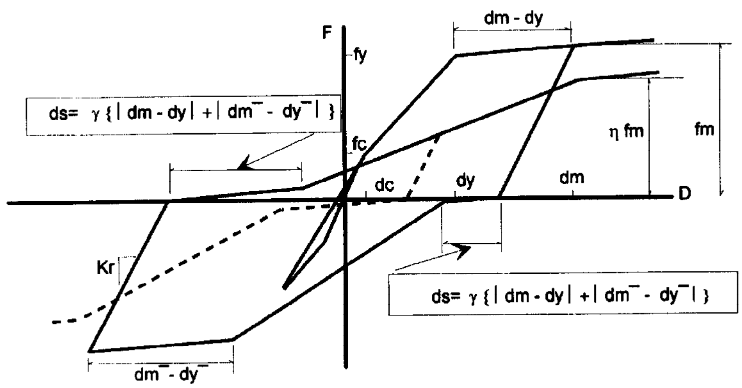

Fig.2 Definition of strength degradation and slip deformation index. Hysteresis model used for analysis.

The strength degradation index η was computed for all specimens, with joint at mid-span, and joint at member-end. Also, it was measured the sliding displacement at construction joint. In Fig.3 it is shown the relation between the strength degradation index η and the sliding displacement at construction joint.

It can be clearly noticed that for both cases, members with joint at mid-span and members with joint at member-end, if the sliding displacement at the construction joint becomes bigger, the strength degradation index η decreases. For members with joint at member-end, for small values of sliding displacement, the strength degradation index decreases considerably, and it is observed a strength degradation of 20 % for a sliding displacement of 1.0 mm. On the other hand, for members with joint at mid-span, almost 3.0 mm of sliding displacement at construction joint is accepted, while the strength degradation gets to 20 %.

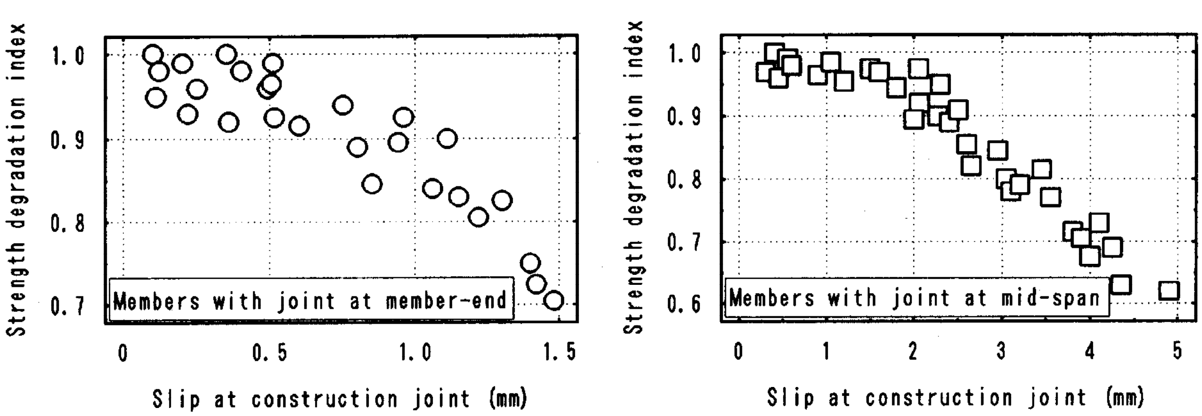

Fig.3 Relation between η index and sliding displacement at joint.

Slip deformation index γ, was defined as the relation between the total deformation in post-yielding stage, and the total deformation in re-loading zone with instantaneous stiffness equal to the stiffness in post-yielding stage (Fig.2)

$$\gamma = [\ |dm - dy| + |dm^{-} - dy^{-}|\ ] / ds \qquad (2)$$

where: dm and dm⁻ are the maximum positive and negative displacement in post yielding stage; dy and dy⁻ are the positive and negative yielding displacement respectively; and, ds is the deformation in re-loading zone with instantaneous stiffness equal to the stiffness in post-yielding stage.

In the same way as the strength degradation index η, the relationship between the slip deformation index γ, and the sliding displacement at construction joint was computed. The results are shown in Fig.4.

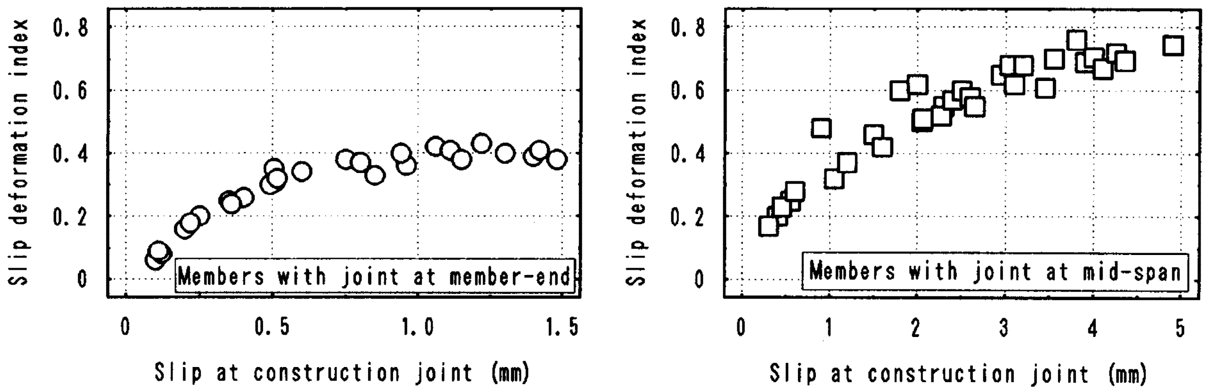

Fig.4 Relation between γ index and sliding displacement at joint.

It can be observed that the slip deformation index γ becomes bigger as the sliding displacement at construction joint increases. For members with joint at mid-span, the sliding displacement is large from the beginning, and the slip deformation index gets to large values. For a sliding displacement between 3.5 to 5.0 mm, the slip deformation index results in 75%. Contrarily, for members with joint at member-ends, the sliding displacement values at construction joint resulted small, and so the slip deformation index. For a sliding displacement between 1.0 to 1.5 mm, the slip deformation index results in 40%. It should be noticed the good correspondence between the value of γ and the sliding displacement at construction joint, independent of the location of the construction joint. It can be observed that for sliding deformation at construction joint of approximately 1.0 mm, in both cases, the slip deformation index γ becomes between 0.35 and 0.40.

<u>Hysteresis Model</u>

Taking into account the experimental results and the tendency of behavior of PCa structural members, it was considered a simple hysteresis model. The model is a modification of the named Takeda model (Takeda, et. al., 1970) with the peculiarity that the model considers strength degradation (index η) as a function of the strength for the previous maximum displacement peak, and slip deformation (index γ) as a function of total post-yielding stage deformation. The graphic representation of the hysteresis model is shown in Fig.2.

EFFECT OF SLIDING DISPLACEMENT AT CONSTRUCTION JOINT OF PCa MEMBERS ON EARTHQUAKE RESPONSE

The sliding displacement at construction joint of PCa members can be undesirable because the energy dissipation capacity at beams ends is reduced by pinching in a hysteresis shape, and also by strength degradation under cyclic loading in post-yielding stage. Earthquake response analyses were carried out in order to investigate the effect of strength degradation, slip deformation and sliding displacement at construction joint on maximum response.

The earthquake response analyses were carried out by the computer program "Dandy" (Kabeyasawa et.al., 1983). Each member was represented by a one component model, in which an inelastic rotation spring was placed at member ends. The beam-column joint was assumed to be rigid. The hysteresis models placed at beam ends were selected to simulate the pinching and strength degradation behavior caused by bond deterioration, shear cracking and sliding displacement at construction joints (Hysteresis model presented above).

The earthquake response analyses were carried out on 6 and 12 stories moment-resisting frames with a 6.0 m span and a uniform story height of 3.0 m. The buildings were designed in accordance with Japanese Building Standard Code, to form a weak-beam strong-column frame structure. Total height, total weight, fundamental period and design shear coefficient are listed in Table 1.

A part of building (a fish-bone model, Fig.5), consisting of a continuous column and beams connected on both sides of the column, was removed from the prototype building by cutting beams at the inflection point located at the beams mid-span (Kitayama et.al, 1986). From non-linear static analyses under monotonically increasing loads of an inverted triangular distribution, the base shear coefficient at the yielding mechanism were calculated to be 0.3 for both structures.

Table 1 General characteristics of structures

Case	H (m)	W (t)	T (s)	Design shear coef.
6 stories structure	18.5	267.0	0.50	0.25
12 stories structure	36.5	526.0	0.78	0.25

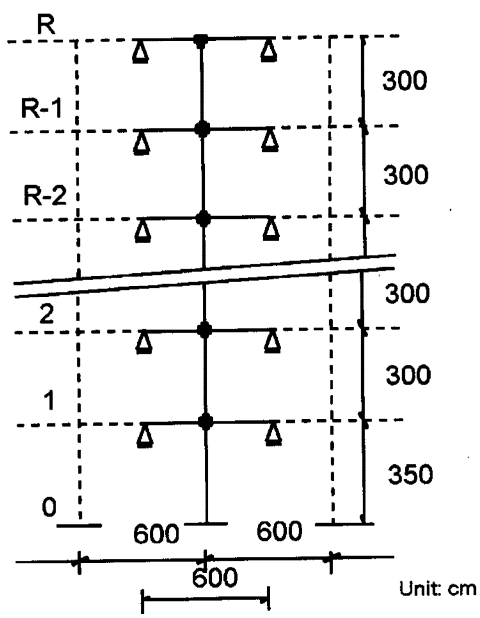

Fig. 5 Analytical model of the structure (fish-bone model).

Viscous damping matrix was assumed to be proportional to instantaneous stiffness matrix, and the initial elastic damping factor for the first mode was chosen to be 0.05 of the critical. Input earthquake motions were 1940 El Centro (NS) record, 1978 Tohoku (NS) record and 1985 Michoacan SCT (EW) record. The records were selected to include ground motions having different dominant period. The general characteristics of the earthquake ground motions are shown in Table 2. The intensity of ground motions was adjusted to develop maximum ductility factors of approximately 4.0 at beam-ends in the structure when the model selected is the Takeda model (η=1.0, and γ=0.0).

Table 2 General characteristics of the ground motions records

Record	EC	TO	SC
td (s)	0 - 20	0 - 20	35 - 70
a_{max} (cm/s^2)	342	258	138
T_{pmax} (s)	0.46	0.95	1.98

td: Duration used for analysis

T_{pmax}: Period at maximum elastic response

a_{max}: Maximum input acceleration

The parameters η and γ of the hysteresis model were selected so as to appreciate their effect on maximum responses. Considering the experimental results it was decided to use Takeda model (η=1.0, γ=0.0) to simulate a good behavior of reinforced concrete structures. A combination of parameters was selected so that γ may vary from 0.0 to 0.5, and η may vary from 1.0 to 0.8. In Table 3, all the cases considered in the analyses are shown, and so the equivalent damping ratio h_{eq} for each case.

Table 3 Cases of study

Case	1	2	3	4	5	6	7	8	9	10
η	1.0	0.9	0.9	0.9	0.9	1.0	0.8	0.8	1.0	1.0
γ	0.0	0.0	0.1	0.3	0.5	0.5	0.5	0.0	0.3	0.1
h_{eq}	0.235	0.219	0.214	0.204	0.191	0.194	0.187	0.213	0.210	0.231

The attained ductility factors of beam ends are shown in Fig.6. The distribution of the ductility factors on the structure are similar in shape for almost all the cases. The change in the h_{eq} value of the model considering only slip deformation index did not affect the ductility demand, however, if the strength degradation is considered, the ductility demands vary drastically with the value of h_{eq}. The maximum drift radios of each story are shown in Fig.7. The maximum drift ratio also shows important variation with the strength degradation index. It was concluded that the effect of slip deformation or pinching shape of hysteresis model on the response amplitudes was relatively small, and the effect of strength degradation could be considered important depending on the combination of both parameters η and γ.

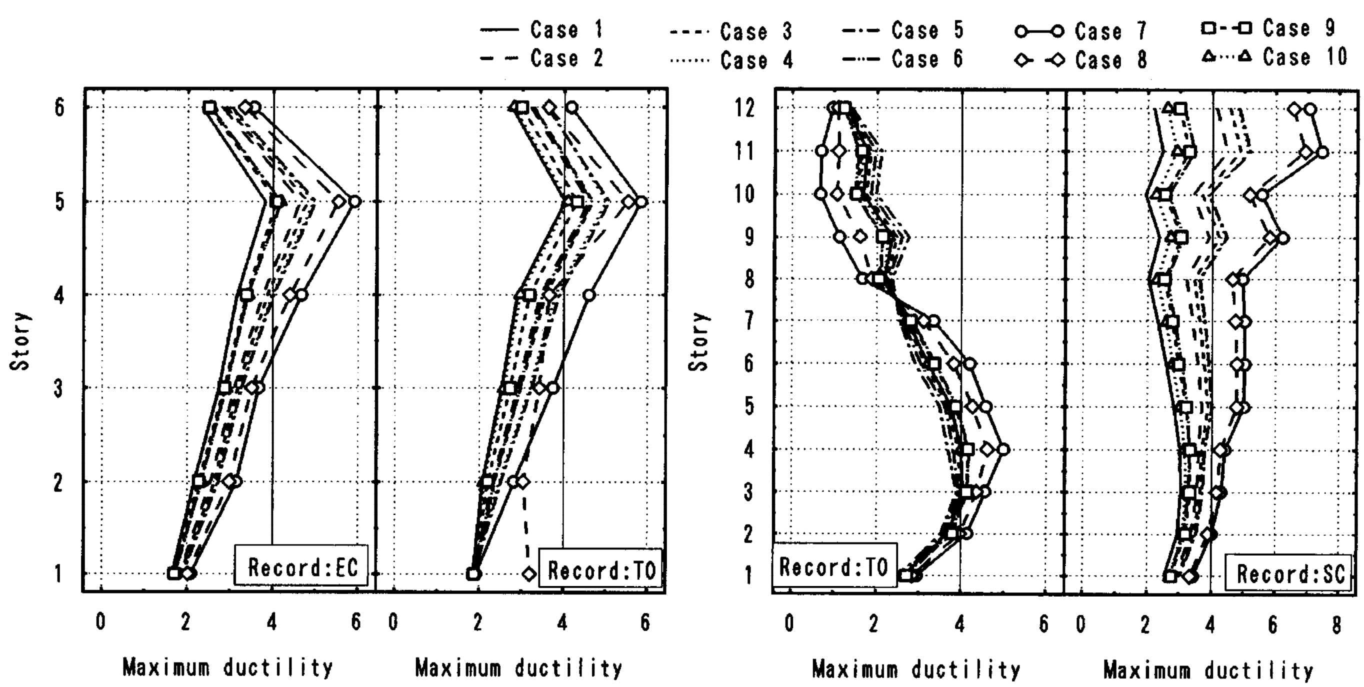

Fig.6 Maximum attained ductility factors at beam-ends.

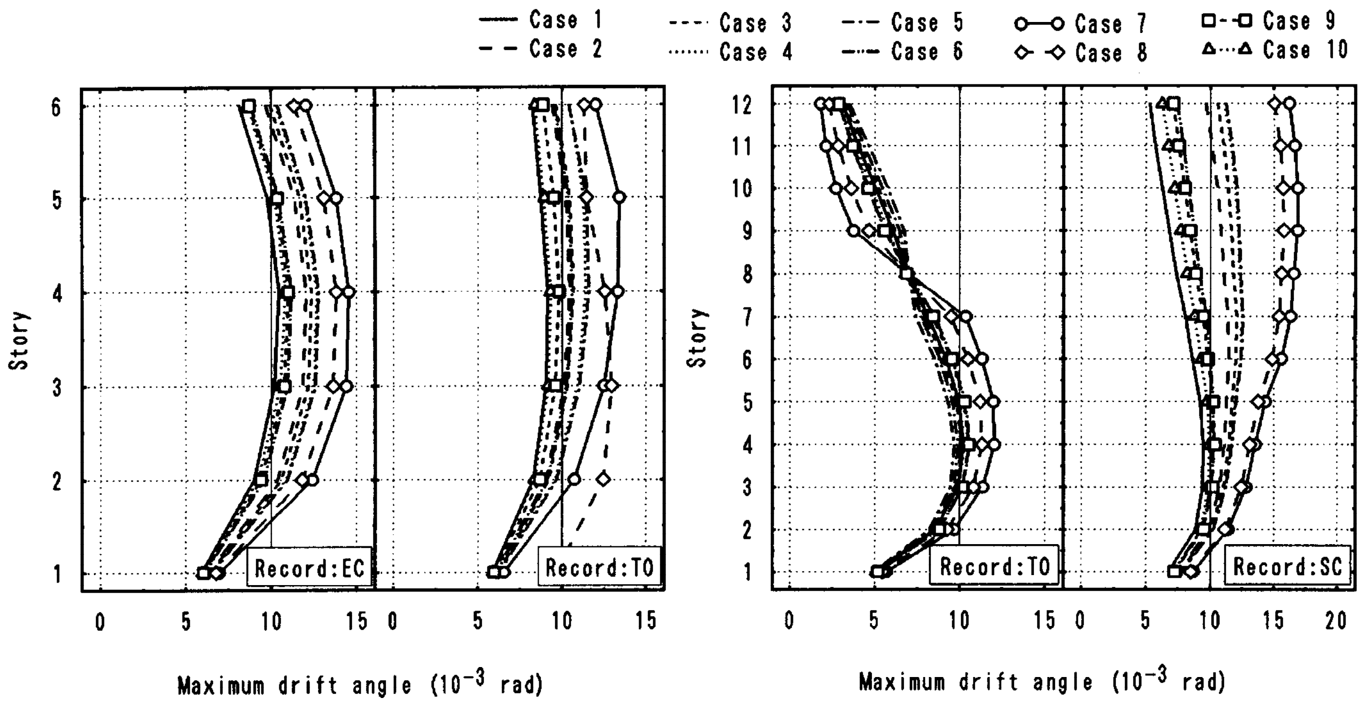

Fig.7 Maximum story drift angle ratios.

<u>Limitation of slip deformation, strength degradation index and sliding displacement.</u>

The buildings were designed to develop, at collapse mechanism, the maximum ductility at beam-ends not more than 4.0, and to present the maximum story drift angle equal to 1/100 rad. The combination of index γ and η should be such that the maximum responses are within the limit design values stated before.

From the analyses results it could be selected an allowable lower limit for the pair of index γ and η such that maximum responses could be within the limits values permitted by design process. The combination of η and γ becomes as follows:

$$\eta \geq 0.8 \qquad ; \text{for } \gamma \leq 0.2$$

$$\eta \geq 1/3\,(\gamma - 0.2) + 0.8 \qquad ; \text{for } \gamma \geq 0.2 \tag{3}$$

$$\eta \leq 1.0$$

The limitation for the pair of values of γ and η determined from analyses results, together with the experimental results, yields to establish a limitation of values for sliding deformation at construction joint. In Fig.8, it is shown the pair of values of γ and η obtained from experimental results, together with the allowable limit values obtained from analyses. It could be established that for members with joint at mid-span it can be admitted a sliding displacement at construction joint not more than 3 mm; and for members with joint at beam-end a sliding displacement at construction joint not more than 1.0 mm.

CONCLUSIONS

From the relations between experimental results on PCa beam members and analytical results on a 6 and 12 stories PCa structures, the conclusions can be as follows:

1. Sliding displacement at construction joint located at member-end has important relevance on the strength degradation of structural members.

2. Maximum responses of a structural system show considerable variation with strength degradation of structural members.

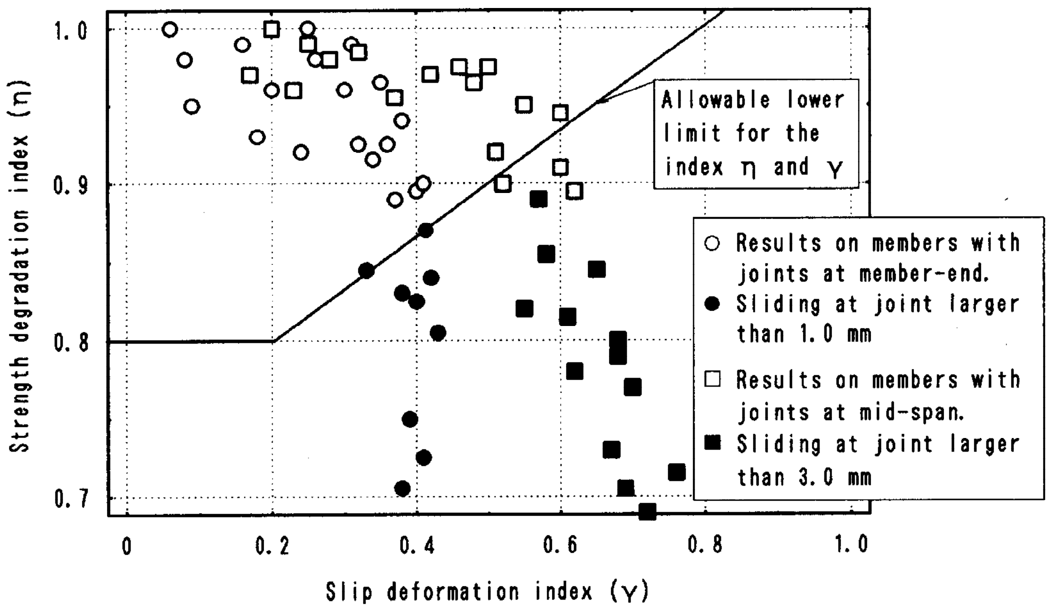

Fig.8 Allowable lower limit for η and γ index, and its relation with experimental results.

3. It was found as an allowable value for sliding deformation on the construction joint of PCa members, the value of 1.0 mm for joints at member-ends; and the value of 3.0 mm for joints at the mid-span of the member, for the maximum ductility of 4.0 and the maximum story drift angle of 1/100 rad. The design of a PCa member of which the sliding displacement at construction joint should be not more than an allowable value for sliding displacement, should provide a structural behavior within the limit values proposed by the seismic design codes.

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