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Ductility-based seismic design of precast concrete large panel buildings

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Abstract. Two approximate methods based on mechanism analysis suitable for seismic assessment/design of structural concrete are reviewed. The methods involve use of equal energy concept or equal displacement concept along with appropriate patterns of inelastic deformations to relate structure's maximum lateral displacement to member and plastic deformations. One of these methods (Clough's method), defined here as a ductility-based approach, is examined in detail and a modification for its improvement is suggested. The modification is based on estimation of maximum inelastic displacement using inelastic design response spectra (IDRS) as an alternative to using equal energy concept. The IDRS for demand displacement ductilities are developed for a single degree of freedom model subjected to several accelerograms as functions of response modification factor (R), damping ratios, and strain hardening. The suggested revised methodology involves estimation of R as the ratio of elastic strength demand to code level demand, and determination of design base shear using $R_{design} \leq R$ and maximum displacement, determination of plastic displacement using IDRS and subsequent local plastic deformations. The methodology is demonstrated for the case of a 10-story precast wall panel building.

Key words: precast concrete; seismic design; inelastic response spectra.

1. Introduction

The deficiency of precast concrete construction for regions with high seismicity, where relatively large energy dissipation capacity is needed, has been long recognized (Fintel 1977, Mueller 1981,

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Park 1981, Dowrick 1987). The problem basically stems from insufficient detailing at connections to accommodate desirable hysteretic responses. Seismic behavior of various types of precast concrete large panel construction has been the subject of numerous recent studies (Schricker and Powell 1980, Harris and Abboud 1981, Oliva *et al.* 1988, Pekau and Hum 1991, Soudki *et al.* 1995, Kianoush *et al.* 1996). While most analytical and experimental studies have revealed different aspects of seismic deficiencies of panel wall construction or have investigated new connection components or systems for such construction, there are some that discuss various design aspects and code provisions (Johnson 1981, Aswad 1981, Popoff 1981, Fintel and Ghosh 1981). A particular study of interest in this paper (Clough 1985) actually proposes remedial measures to make precast construction more competitive in seismic regions. In general, solution methods that recognize the essence of this "jointed" construction and recommend design and detailing schemes that do not just emulate those suitable for monolithic reinforced concrete construction, can more realistically address the issue.

Although design codes (e.g., ACI 1995) generally contain explicit requirements with respect to tension ties to ensure integrity of precast construction, they do not provide such specific requirements for ductility. This problem is especially crucial for jointed construction with relatively weaker stiffness, strength, and ductility at the connections compared to the members being connected.

In one of the comprehensive studies on jointed precast prestressed concrete buildings (Clough 1985) referred to as Clough's method in this paper, an approach is proposed to design the structure for a desirable ductility level and to detail the connections according to the required deformations in order to provide the necessary energy dissipation capacity such that an acceptable degree of continuity and redundancy is obtained. The suggested approach assumes that the failure mechanism of the structure can be modeled as a Single Degree of Freedom (SDOF) system with Elastic Perfectly Plastic (EPP) force-deformation relation. Moreover, to calculate the maximum inelastic displacement, Clough's method assumes that the Equal Energy Concept (EEC) (Newmark and Hall 1973) is applicable. This entails that the fundamental period of the structure be within a certain range. The maximum inelastic displacement (at the top of the building), which is the displacement in the SDOF failure mechanism model, is then related to the rotations in the plastic hinges with the use of kinematics. Finally, guidelines are suggested to detail joints in order to accommodate the desirable capacity based on the calculated joint rotation demand at critical sections. Thus, the method proportions and details joints to provide the required deformations. One of the drawbacks of this methodology is its dependence on the EEC which says that the maximum energy absorbed by an SDOF system with relatively short period (e.g., in the range of 1/8 second to 1/2 second) subject to seismic excitation is approximately the same whether the system shows a Linear Elastic (LE) force-deformation relation or an EPP one (Fig. 1a). By equating the expressions for such energies (Newmark and Hall 1973), a fixed relation can be obtained between displacement ductility factor μ and response modification factor R. The derived relation can then be used to determine the maximum inelastic displacement of the EPP model from the maximum displacement of the LE model. Although these may be conservative in many cases, some studies (Bertero *et al.* 1976, Mahin and Bertero 1981, Sucuoglu *et al.* 1994) have shown that such a fixed relation could lead to inaccuracies for certain period ranges, primarily because of the sensitivity of the response to input ground motion. The general conclusion is that by developing period dependent relations in the form of response spectra between μ and R, the suggested methodology can be improved.

While the more recent Clough's Method is suggested for the design of jointed precast concrete structures, an earlier method suggested by Park and Paulay (1975) considers reinforced concrete framed buildings. In their method, Park and Paulay assume that the relation between μ and R can be obtained from the EEC or Equal Displacement Concept (EDC), depending on the period. Park later suggested the use of their method, which with some modification fits in the general capacity design approach of New Zealand, for precast concrete buildings (Park 1990). Although the main emphasis of this paper is with respect to suggesting a modification for Clough's method, it is also of interest to discuss the similarities and differences between this method and Park and Paulay's method.

The objectives of this paper are:

(i) To discuss the background to Clough's method, to draw some comparison with Park and Paulay's method, and to indicate the suitability of the proposed modification for the Park and Paulay's method as well.

(ii) To develop demand displacement ductility factor spectra for several values of R.

(iii) To discuss various attributes of ductility spectra and their sensitivities to parameters such as post-yield stiffness and damping ratio.

(iv) To illustrate an application of Clough's method as modified in this paper to a 10-story coupled shear wall building and compare results with those based on the original Clough's method.

2. Explanation of Park and Paulay's and Clough's Methods and their comparison

For ultimate limit state of seismic resistant design or assessment, it is necessary to predict the inelastic behavior of the structure under severe earthquakes. Although inelastic dynamic time history analysis provides the most accurate insight, the complexities involved have led to development of alternative approximate approaches (Newmark 1965, Shibata and Sozen 1976, Saiidi and Sozen 1981). One such approximate method is the "static collapse mechanism" approach (Park and Paulay 1975) which will be discussed subsequently.

One of the basic measures of inelastic response has been the displacement ductility factor, μ,

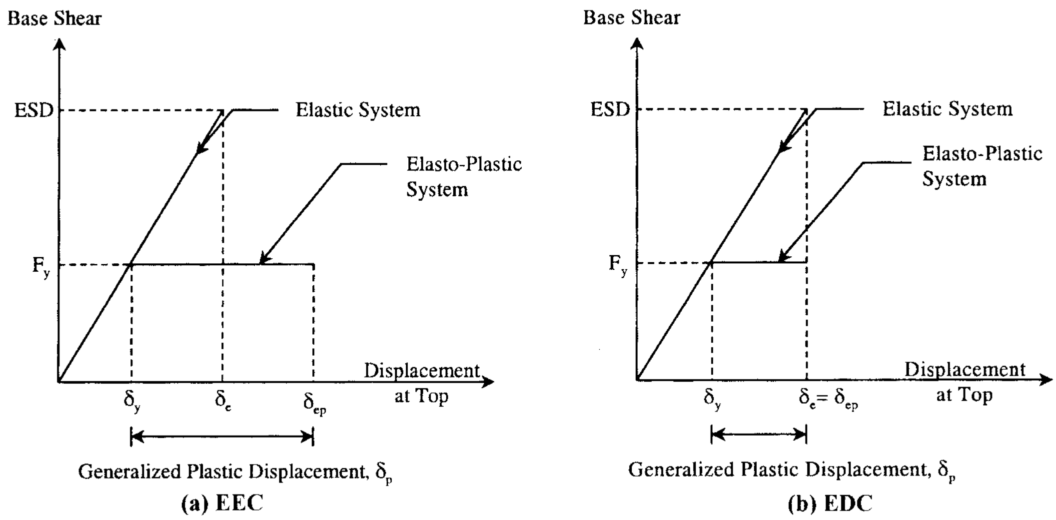

Fig. 1 EDC and EEC for estimating inelastic displacements of an EPP system

defined as the ratio of maximum total displacement δ_{ep}, to displacement at first yield δ_y (Newmark and Hall 1973). It has been suggested (Park and Paulay 1975) that an estimate for μ may be obtained by assuming that the maximum total displacement response of a building can be taken equal to maximum elastic displacement response obtained from a dynamic analysis and that the displacement at first yield may be taken equal to the displacement under the action of seismic code prescribed (equivalent static) lateral forces and obtained from a linearly elastic static analysis. This suggestion is in effect based on the assumption of equal maximum displacement for a structure with force-deformation relation being LE or EPP (Fig. 1b).

Now well known as the equal displacement concept (EDC), it was originally stated for SDOF systems with relatively long fundamental periods *T.* This conclusion was drawn as a result of the method of obtaining inelastic acceleration design response spectra (IDRS) directly from linear elastic design response spectra (LEDRS) (Newmark and Hall 1973) by dividing the latter by μ. For such cases, the response modification factor, *R*, which is defined as the factor by which the elastic response forces should be divided to obtain the design forces, is equal to μ. The response modification factor, *R*, or the factor by which LEDRS is divided to obtain IDRS in moderate period range is $(2\mu-1)^{0.5}$. This ratio can be obtained based on EEC. Park and Paulay have suggested that the response modification factor obtained based on the EDC and EEC can be applied to multistory reinforced concrete frames in an approximate sense, taking the maximum displacement to be the roof displacement (Park and Paulay 1975). It is recognized that since the plastic hinges at failure condition do not all form simultaneously, the load-displacement relationship for the entire structure is actually nonlinear. However, as an approximation, a bilinear model strictly applicable to an SDOF system can be assumed. For making an approximate assessment of flexural curvature or rotation ductility demand of plastic hinges to achieve a given displacement ductility, Park and Paulay (1975) suggested an approach based on static collapse mechanism. Without going through the details, the essential aspects of the approach are explained here. For simplification, it is assumed that the critical sections of the frames will have bilinear moment-curvature characteristics and that yielding occurs at all critical sections simultaneously giving rise to a beam sway or a column sway mechanism (Figs. 2a and 2b). Priestley and Calvi (1991), in discussing the capacity-design assessment procedure, have added that in many cases a mixed mechanism, shown in Fig. 2c, could form where flexural failure at some sections combined with shear failure at other sections brings about the collapse. According to Priestley and Calvi (1991), one way to develop the probable

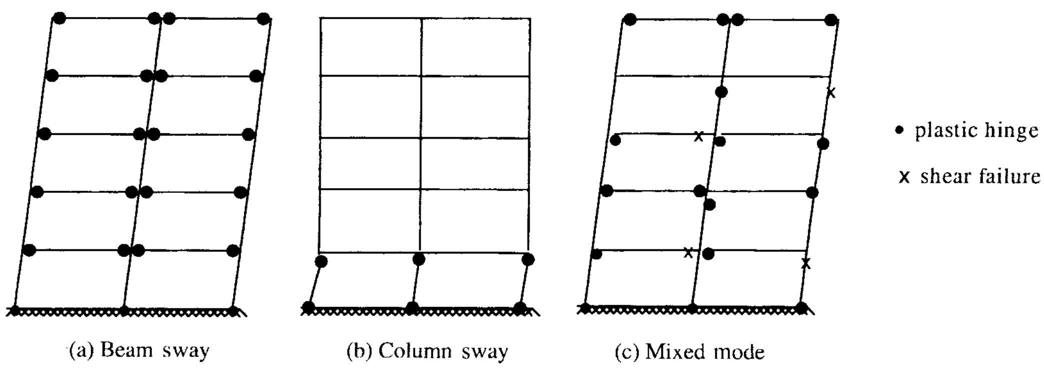

Fig. 2 Plastic collapse mechanisms

sidesway mechanism of an existing building is based on capacity determination, where by comparison of flexural and shear capacity of members, local failure modes of elements are first identified. Then by comparing capacities of beams and columns that frame into a joint, the basic sidesway mode can be determined. By interpolation between basic mechanisms, one can determine the appropriate mixed mode mechanism.

Using the column curvature distribution, which has the shape of the bending moment diagram at the initiation of yielding in the frame and based on principles of elastic structural analysis, one can derive an expression for the yield displacement, δ_y, at the roof level as a function of column curvatures. Next, depending on the assumed sidesway mechanism and using kinematics, appropriate expressions can be derived for the plastic displacement, δ_p, at the roof level in terms of column or beam plastic hinge rotations. The maximum total displacement δ_{ep} is the sum of δ_p and δ_y. Finally, using the definition of displacement ductility factor, $\mu=\delta_{ep}/\delta_y$, one can obtain a relation between μ and plastic hinge rotation or curvature demand. Based on this method, an estimate of curvature ductility factor for plastic hinges in static collapse mechanism may be obtained and compared to the critical section deformation capacity. Therefore, the method, as suggested, can be used for seismic assessment of an existing reinforced concrete framed structure.

In a somewhat similar type of approach, Clough's method can be used to estimate plastic hinge rotations (local deformations) of the failure mechanism of the lateral force resisting system in precast concrete buildings. The method is primarily proposed for design and in this regard, guidelines for the selection of a lateral force resisting system that will yield with a predictable failure mechanism modeled as an SDOF system are suggested. Accordingly, based on the EEC, using the following relation between R and μ:

$$\mu=\frac{R^2+1}{2} \tag{1}$$

and the definition of displacement ductility factor, one can obtain the following expression for δ_{ep}:

$$\delta_{ep}=\frac{R^2+1}{2}\delta_y. \tag{2}$$

It is recalled that R is defined as the elastic force response or Elastic Strength Demand (*ESD*)

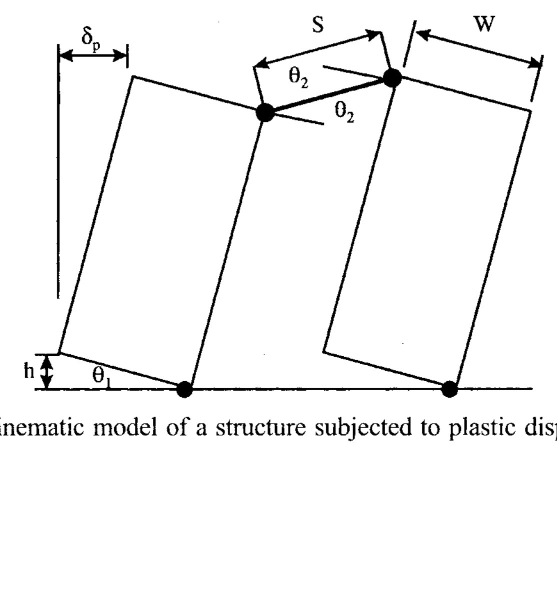

Fig. 3 Kinematic model of a structure subjected to plastic displacement

divided by the seismic code design force level or required yield strength F_y ($R=ESD/F_y$). Using this relation, the suggested approach then proceeds with a kinematic analysis to relate the plastic component of global displacement, δ_p, which is assumed to occur at the roof level, to plastic hinge rotations at the yielded joints. Fig. 3 shows a typical failure mechanism of a structure made up of two wall panels connected by a beam at the top. The kinematic model shown simulates the condition of wall panel rotations resulting in plastic hinge formation at toe of each panel and at the ends of the beam. Proportioning and detailing of the joints will then be such that the required local deformation capacity is provided.

While initially Park and Paulay's method (Park and Paulay 1975) was suggested as a method for approximate assessment of the ductility demand at plastic hinge locations of reinforced concrete frames, the approach with some modification was later used in the context of "capacity design" for seismic design of precast concrete structures (Park 1990). In this sense, both Clough's and Park's methods recommend that the primary lateral load resisting system be chosen so that plastic hinges will form at desirable locations (Clough 1985, Park 1990). In the initial Park and Paulay suggestion for ductility assessment of reinforced concrete frames, since there is not a prior designation of a desirable framing system (it deals with an existing system), more than one possibility for the collapse mechanism may have to be investigated. This is particularly true for existing structures. Obviously, although it is suggested for the design of new buildings, it is possible to use Clough's method for assessment of existing buildings or designs if the possibility of various collapse mechanisms is also taken into consideration. Another difference between the two methods is that while in Park and Paulay's method the relation between response modification factor and displacement ductility can be obtained from either the EDC or EEC depending on period, in Clough's method, due to inherent rigidity of low to midrise wall type precast concrete buildings considered suitable to develop an SDOF mechanism, only the EEC is used.

Priestley has also used the concept of static collapse mechanism for seismic assessment of buildings and bridges initially in the context of a capacity-design approach and later as an element of a proposed displacement based method (Priestley and Calvi 1991, Priestley 1997, Priestley and Park 1987). Displacement-based design method, which is still under development and its various possible forms and aspects are under revision (Moehle 1992, Wallace 1994, Calvi and Pavese 1995, Kowalsky *et al.* 1995, Calvi and Kingsley 1995), is different from the force-based method in that it compares the deformation demands with deformation capacity. In force-based methods, the displacement is checked not to exceed a prescribed limit. The following steps are outlined by Wallace (1994) and are used here as an example of one overview of the displacement-based method:

(i) Ensure minimum level of building strnegth.
(ii) Characterize earthquake demands at the building site.
(iii) Characterize global (building) displacement response.
(iv) Characterize local (structural element) deformation capacity.
(v) Relate global and local deformation.
(vi) Establish detailing requirements.

All proposed displacement-based approaches use some form of elastic displacement response spectra. Definitions of displacement-based design or assessment as suggested by other researchers (e.g., Kowalsky *et al.* 1995, Priestley 1997) are slightly different from the above. However, the essential idea is to design critical sections to satisfy the local demand deformations (according to some performance criteria) which can be related to global displacement. The procedure used in this paper essentially follows the above general steps. However, the way some parameters are determined and emphasized is different. For example, whereas in the displacement-based method global displacement is read from an elastic displacement spectra, in the method proposed here, first displacement ductility demand is read from inelastic displacement ductility spectra for a desirable value of response modification factor (R). Then global displacement is determined as a function of local ductility demand and the selected value of R. The procedure of this paper is thus defined here as a ductility-based approach.

Although determination of IDRS directly from LEDRS has been a widely accepted approach for calculation of preliminary design forces, the reliability of this approach has been questioned primarily because maximum response of systems with LE and EPP behavior models are obtained with different earthquake ground motions (Bertero *et al.* 1976, Mahin and Bertero 1981). These studies have shown that the calculated displacement ductility demand can be significantly different from the maximum displacement ductility specified for design, especially for short period ranges (e.g., $T<0.4$ sec). It has been suggested in these studies that by constructing charts to give displacement ductility demand as a function of damping ratio, fundamental period and response modification factor instead of relying on EDC or EEC, more accurate results will be obtained. In order to improve the accuracy of Clough's approach in this respect, and for that matter, the static collapse mechanism approach of Park and Paulay, as well, displacement ductility demand spectra are constructed in the following sections. To develop such spectra, inelastic dynamic time history analysis of SDOF system with bilinear force-deformation relation with or without strain hardening and subjected to a selected ensemble of earthquake ground motions must be carried out.

3. Construction of displacement ductility spectra

Inelastic dynamic time history method of analysis has been known to provide the most accurate

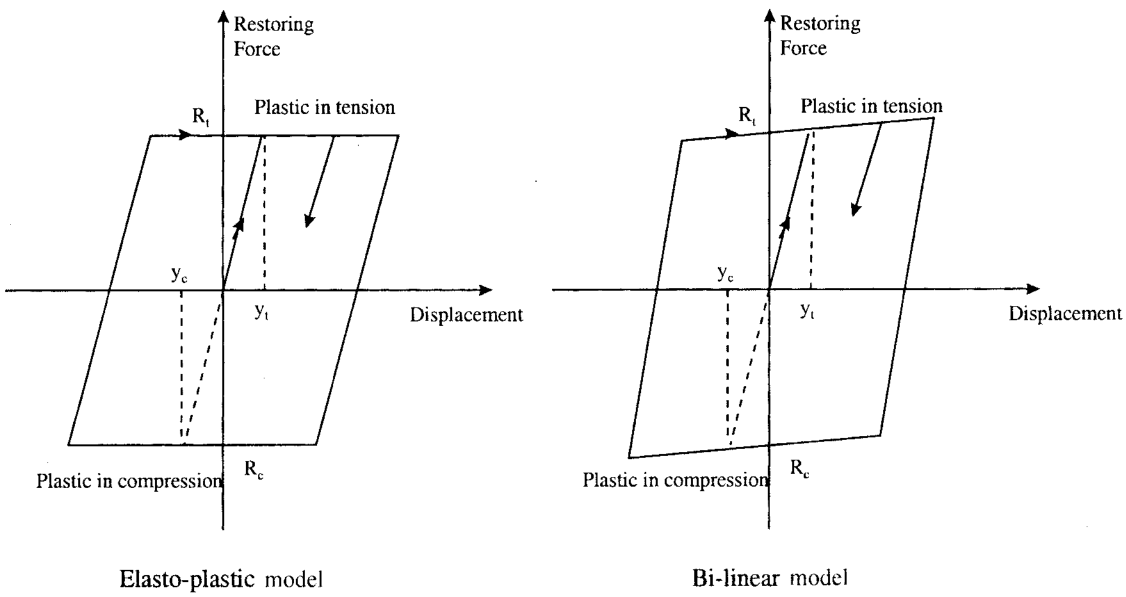

Fig. 4 Hysteretic models

results in deriving IDRS which are plots of pseudo velocity, pseudo acceleration, and relative displacement versus period for different values of damping ratio, displacement ductility factor and strain hardening parameters. However, spectra can also be developed for other response parameters such as ductility. For this study, a computer program was developed (Astarlioglu 1997) to determine the dynamic response of SDOF systems with an EPP or bilinear hysteric model shown in Fig. 4.

The incremental dynamic equilibrium of the model subjected to a base excitation was established assuming a velocity dependent damping force, a displacement dependent elastic force, and an acceleration dependent inertial force (Paz 1991). Assuming that acceleration varies linearly but material properties remain constant during a given time increment, we can obtain the response by a step-by-step numerical integration process. The seismic excitations are selected from an ensemble of accelerograms with an average v_{max}/a_{max} ratio of 0.848 m/sec/g (Sucuoglu *et al.* 1994). The 15 digitized records, typical of U.S. west coast earthquakes, that are used in this study, are listed in Table 1. The records cover geological conditions ranging from rock to general alluvial categories, with v_{max}/a_{max} ratios in the range of 0.266 m/sec/g to 1.593 m/sec/g. This range encompasses the range of 0.660 m/sec/g to 1.397 m/sec/g as average values for rock and alluvium, respectively, for west coast earthquakes studied by Seed *et al.* (1976) to develope a relation between peak ground acceleration and maximum ground velocity for various geological conditions. The basis for using v_{max}/a_{max} ratios instead of peak acceleration alone in selected ground motion records is explained by Seed *et al.* (1976). Among other reasons, the relation of destructiveness of earthquake with maximum velocity is notable.

The records used are unscaled since the parameter of interest to be determined, R, is the ratio of ESD to F_y, both being functions of peak ground acceleration. The following steps outline the construction of displacement ductility demand spectra.

(i) Specify damping ratio, β, initial to post yield stiffness ratio, α, and the earthquake acceleration

Table 1 Characteristics of ground motion records

Site	Location	Year	Distance (km)	Orient. (°)	a_{max} (g)	v_{max} (m/sec)	v_{max}/a_{max} (m/sec/g)
Bear Valley	Melendy Ranch	1972	10.1	61	0.480	0.173	0.361
Bear Valley	Melendy Ranch	1972	10.1	331	0.516	0.137	0.266
Coyote Lake	San Yasidro	1979	8.6	230	0.417	0.438	1.051
Imperial Valley	El Centro	1940	4.6	180	0.348	0.323	0.928
Kern County	Taft	1952	50.4	111	0.179	0.177	0.989
Loma Prieta	Corralitos	1989	1.0	0	0.630	0.552	0.876
Loma Prieta	Santa Cruz	1989	23.2	0	0.442	0.212	0.480
Parkfield	Cholame-Shandon 2	1966	20.9	65	0.489	0.779	1.593
Parkfield	Cholame-Shandon 5	1966	21.6	355	0.355	0.225	0.634
Parkfield	Cholame-Shandon 5	1966	21.6	85	0.434	0.254	0.586
San Fernando	Pacoima Dam	1971	7.8	164	1.170	1.132	0.968
San Fernando	Pacoima Dam	1971	7.8	254	1.075	0.575	0.534
San Fernando	Castaic Old Ridge Rt.	1971	27.0	21	0.315	0.165	0.524
Whittier	Tarzana	1987	43.3	90	0.537	0.242	0.451
Whittier	Garvey Reservoir	1987	3.0	330	0.477	0.198	0.415

record. Select a period range and a value for response modification factor, R.

(ii) Determine the maximum elastic restoring force for the SDOF system with LE model during the earthquake for a prescribed period. Set this force equal to elastic strength demand, *ESD*.

(iii) For the same period value, determine the maximum elastoplastic displacement, δ_{ep}, for the SDOF system with bilinear model and maximum restoring force. Calculate displacement ductility demand, $\mu=\delta_{ep}/\delta_y$, where δ_y is the displacement corresponding to F_y.

(iv) Increase the value of the period and repeat steps (ii) and (iii).

(v) If desired, choose different values of damping ratios and/or post yield stiffness, α, and repeat steps (i) to (iv).

Based on the described procedure, displacement ductility demands were determined (Astarlioglu 1997) for several values of R. Only ductility spectra using R values of 2, 3, 4 and 5 are presented. It is noted that R=1 corresponds to the LE response. Two values of α are used for the bilinear models: α=0 which gives the EPP model and α=10% for bilinear model. Moreover, three values of damping ratio are considered: 5%, 10%, and 15%. The ductility demands for each of the selected 15 records are obtained for 39 period values in the range 0.1 sec to 2.0 sec. Based on the spectra determined for each record (a total of 15 earthquake records), the averages of these spectra are plotted for different values of the parameters considered. Fig. 5 shows an example of such an average spectra (for α=0 and β=5%), and Fig. 6 shows the corresponding standard deviation. The curves approach horizontal lines beyond 2.0 second; therefore, there was no need to expand the period range.

4. Parametric study

Unlike EDC and EEC, each of which are based on a given value of displacement ductility demand over a certain period range, the displacement ductility demand spectra plotted in Fig. 5 show high sensitivity to period, particularly in the low period range. For smaller periods, e.g., T<0.4 sec,

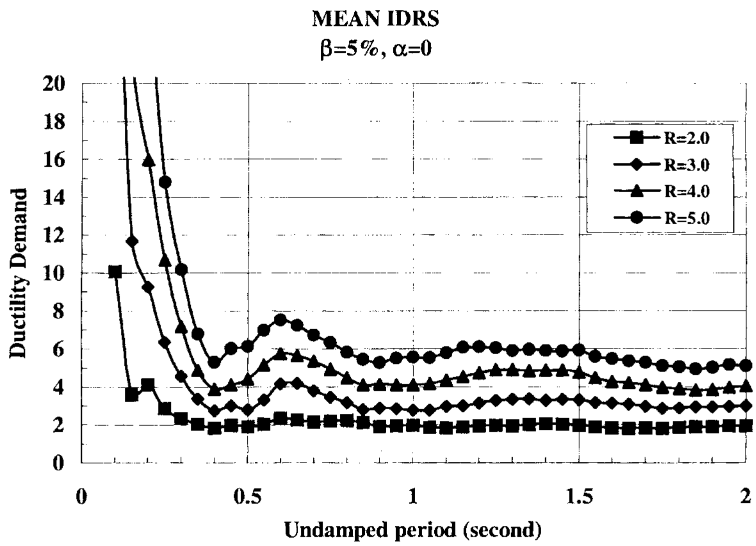

Fig. 5 Average IDRS

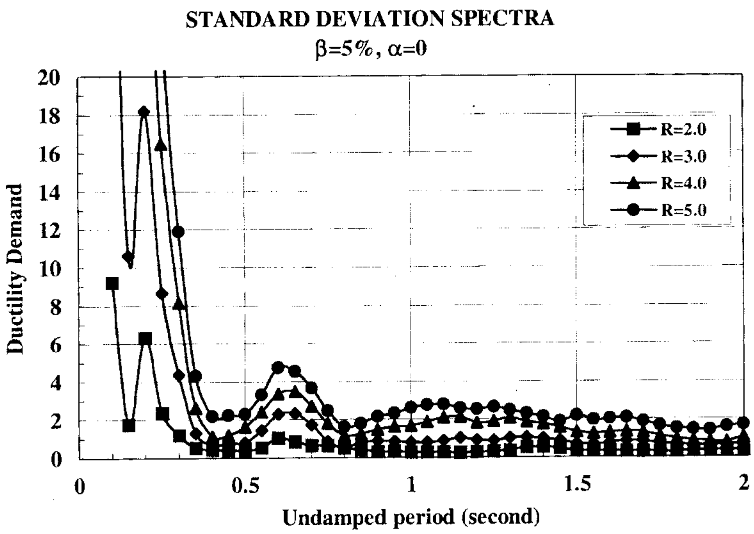

Fig. 6 Standard deviation spectra

the ductility demands increase very rapidly as period decreases. For such stiff structures, the ductility demands become excessively high and exceed those by EEC by a great margin. Moreover, the ductility demand increases with response modification factor. For periods generally larger than 0.4 sec, however, the variation in ductility demand for a given response modification factor is small and for periods larger than 1.0 sec, it approaches the numerical values of R, as is also expected based on EDC. It should be noted, however, that for individual records (i.e., for site specific spectra), the variation in ductility demand is larger and could be in excess of those specified by EDC. In fact, the individual spectra determined for each record show that ductility demand also increases with v_{max}/a_{max} ratio. Such large differences between ductility demands for different records can be seen by observing increased values of standard deviations especially in lower period ranges.

The effect of different damping ratios on displacement ductility demand spectra is shown in Fig. 7. Despite the larger effect of damping on elastic response spectra shown in Fig. 8, the damping effect on displacement ductility demand spectra (Fig. 7) is small. Since the response modification factor is defined as the ratio of *ESD* to F_y, both of which are obtained for the same damping ratio, the effect of β on ductility demand decreases as R gets smaller. It should be noted that for R=1, that is LE behavior, damping ratio will lose its effect on μ since μ=R=1 regardless of damping ratio. However, as R increases, which means μ also increases, the damping ratio will have its effect in the post elastic region, and that is why in the figure shown the effect of β on μ is larger for larger R values. A study (Sucuoglu *et al.* 1994) has shown that the effect of damping on inelastic response to an ensemble of earthquake records gets smaller with increase in ductility. Another study (Mahin and Bertero 1981) has shown that as damping increases, the difference between ductility demand based on dynamic analysis of the EPP system and the ductility specified in obtaining IDRS directly from LEDRS gets smaller. In general, it has been well documented (Bertero *et al.* 1976) that the effect of damping is significantly smaller in SDOF systems with EPP behavior than LE force-deformation relation.

Comparison of the charts for 0%-strain hardening (EPP) with those for 10% strain hardening indicates that with increase in strain hardening, ductility demands decrease slightly in regions with

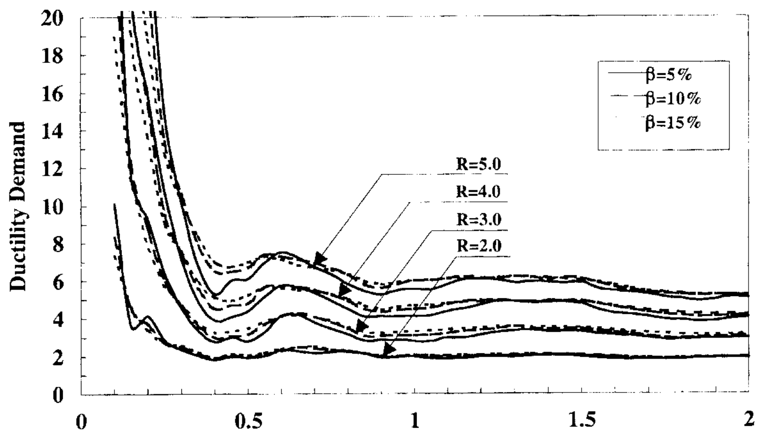

Fig. 7 Effect of damping on IDRS

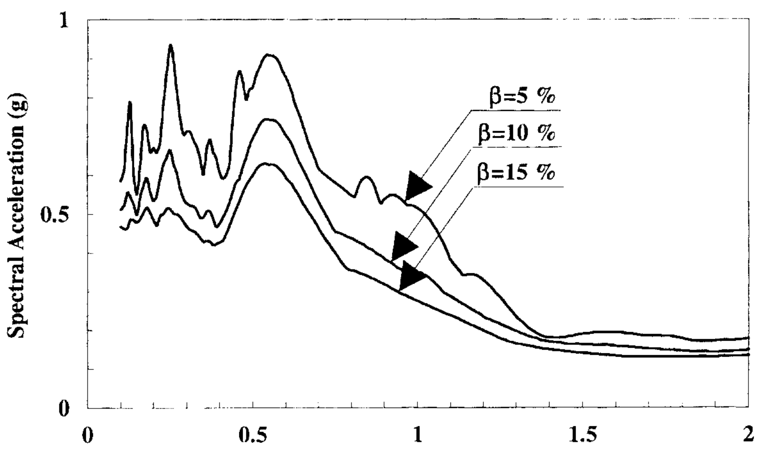

Fig. 8 Effect of damping on LEDRS

periods greater than about 0.4 sec, but the decrease is significant in the small period range. This effect is slightly more so for larger values of response modification factor. This can be seen more clearly in Fig. 9, which plots the average ductility demand for R=2.0 and 5.0 for an EPP system and a bilinear one with 10% strain hardening.

It is finally of interest to compare the displacement ductility demand plotted versus period in Fig. 10 for R=2.0 and 5.0 obtained with the assumption of EDC and EEC with that obtained according to the procedure suggested in the paper. It can be seen that for periods from 0.1 sec to 0.5 sec, EEC gives approximately the arithmetic mean of actual values in this range; however, depending on the period, the actual values can be significantly different from those given by the EEC, more so for larger ductilities. For periods between 0.5 sec and 1.5 sec for R=5, the EDC gives smaller ductility demands, but for R=2, the result is close to that given by IDRS. In general, deviations of EDC curves from the curves of actual values are smaller than those of the EEC.

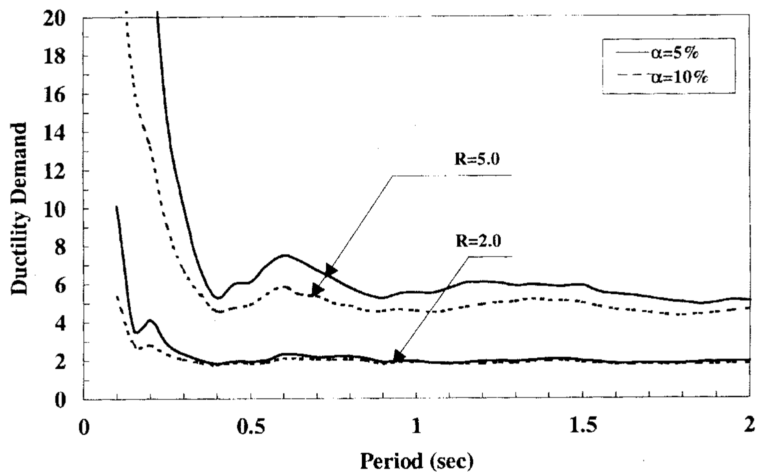

Fig. 9 Effect of strain hardening on IDRS

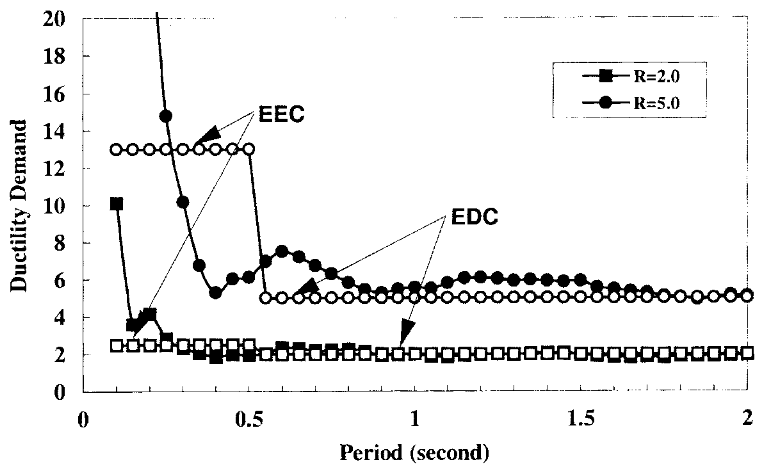

Fig. 10 Comparison of EEC and EDC with IDRS

5. Application of the methodology

The step-by-step procedure suggested by Clough (Fig. 11) and as modified here to use displacement ductility demand spectra instead of the relation based on EEC can be described as follow:

(i) Calculate the fundamental period of the building using empirical formulas given by ATC 3-06 (ATC 1978), UBC (ICBO 1994), NEHRP (BSSC 1994), BOCA (BOCA 1996) or any other seismic code of interest. Alternatively, one can determine the period more accurately as a result of using a dynamic computer structural analysis based on an elastic model with gross section properties.

(ii) Based on a seismic code being used for elastic strength analysis, and with response modification factor R=1 (i.e., linear elastic), determine the elastic strength demand *ESD* (See Fig. 1) by performing a static analysis of a linearly elastic (LE) model of the precast concrete building.

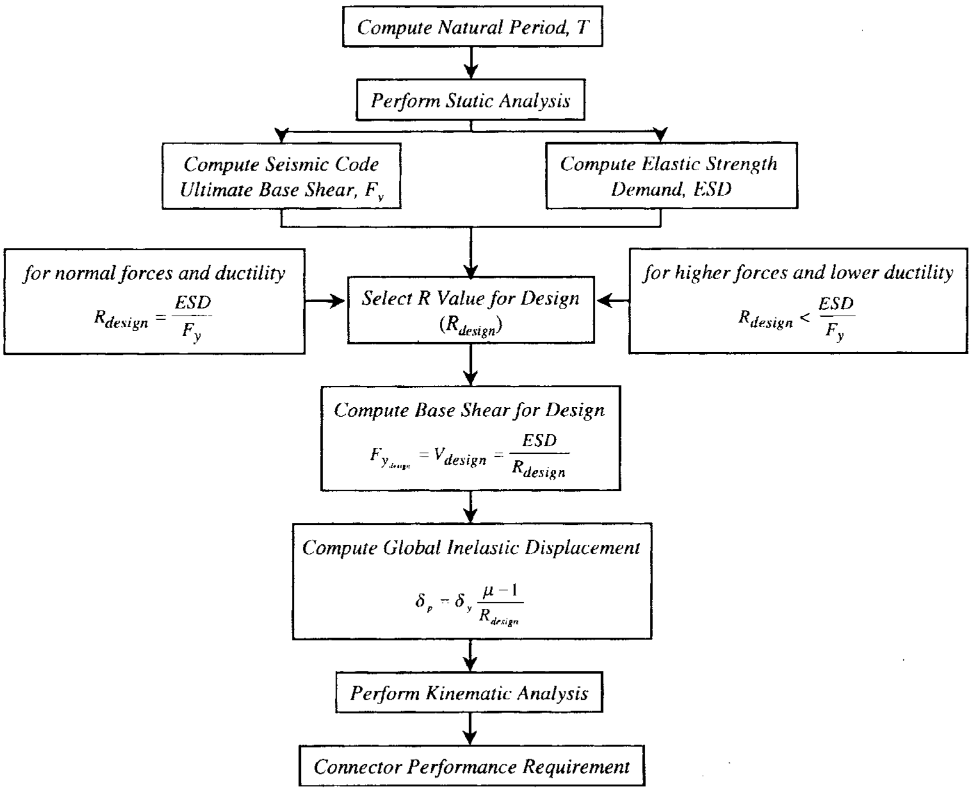

Fig. 11 Seismic design methodology for precast structures

This model could be the same as the model for optional computer analysis in step (i). The *ESD* is the base shear for this model. As part of this computer analysis, the maximum elastic roof displacement, δ_e, is also determined.

(iii) Based on the applicable seismic code being used for the required ultimate capacity determination and the LE model, determine the required ultimate inelastic capacity, F_y. This seismic code can be any seismic code required to be used for the site in question (in any country). The seismic code used in step (ii) should provide for an R factor. It is understood that not all seismic codes specify an explicit R factor. Therefore, the applicable seismic code in this step can be different from that in step (ii).

(iv) Calculate R as the ratio of *ESD* to F_y if these are obtained according to different codes. This would be the R value used in calculating F_y if the same code is used as that for ESD. Considering the definition of the reinforced concrete resisting systems which governs the R value in the same seismic codes, select the "design" response modification factor, R_{design}, value appropriate for the lateral load resisting system. It is to be noted that R_{design} should take into account the performance of the connectors used. If the connectors and their surrounding regions have stiffness, strength, energy absorption and energy dissipation capacities that ensure a performance compatible to that of the lateral force resisting system of the structure, R_{design} values can be as much as the values recommended for monolithic construction. Otherwise, more conservative values should be used.

(v) Calculate the design base shear V_{base} by dividing the elastic strength demand by the selected design response modification factor, R_{design}.

(vi) Determine the displacement ductility demand for the selected value of R_{design} and the calculated fundamental period using the provided charts for displacement ductility demand spectra. It is noted that according to Clough's method, ductility demand is determined based on the relation $\mu=(R^2+1)/2$.

(vii) Calculate the maximum global plastic displacement, $\delta_p=\delta_e(\mu-1)/R_{design}$.

(viii) Perform kinematic analysis of the assumed failure mechanism to determine local plastic hinge rotations in terms of the maximum global plastic displacement, δ_p. It should be noted that the main members of the lateral load resisting systems are to proportioned with sufficient stiffness and strength such that plastic deformation will only take place at predefined hinge locations during an ultimate limit state earthquake.

(ix) Specify the demand force and deformation capacities for the connections and at plastic hinge locations. The demand deformations will be used for effective detailing of the connections such that during a damaging earthquake the joints can provide the necessary rotations imposed by the desirable failure mechanism.

The procedure just described is next applied to a 10-story precast concrete building with large panel coupled wall system. The plan and elevation of the building are shown in Fig. 12. The building has a height of 3250 cm and is assumed to have fixed foundation, thus neglecting soil-structure interaction in this study. Each panel has a length, height, and thickness of 720 cm, 300 cm, and 200 cm, respectively. The panels are connected to each other and to floor slabs through platform type connection (horizontal joints) with 25 cm thickness. The two panel walls are coupled through 180 cm long and 68 cm deep precast panel coupling beams, as shwon in Fig. 12. Vertical continuity between wall panels is to be provided by post-tensioning steel extending from the roof level to the foundation. It is further assumed that the flexural resistance in the walls (resistance to joint opening) results from contributions of gravity loads and post-tensioning force. In general, there

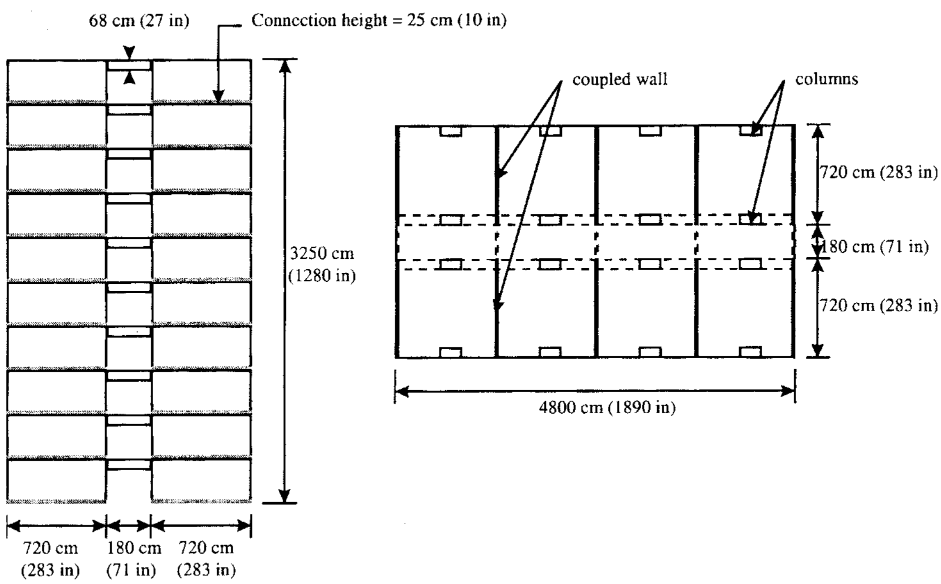

Fig. 12 Plan and elevation of the building

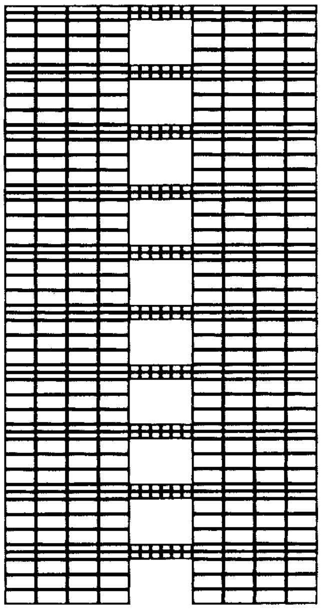

Fig. 13 Finite element model

could be some early nonlinear behavior of the structure due to slight opening of joints (base isolation mechanism) even when steel and concrete still show elastic behavior. With increased seismic input, the joint openings increase and may lead to the yielding of the connection, while the members remain elastic. Under severe earthquakes, wall panel corners can be subjected to severe distress (e.g., crushing) as a result of reduced bearing area when the joints are wide open. In addition to these modes of behavior resulting from rocking, shear slip could also lead to some panel movements. As those kinds of response are not typical of monolithic construction, using this method, the designer can choose R values appropriate for the precast concrete structure considering the attributes of the lateral load resisting system.

The transverse direction fundamental period of the building is determined as a result of a dynamic analysis of the structure, although it could be estimated using a code empirical equation. In this study, the analysis of the finite element model of an interior coupled wall with tributary loading is carried out using DRAIN-2DX (Prakash and Powell 1993). Fig. 13 shows the finife element model developed for the structure. The wall panels, horizontal joint regions, and the coupling beams are all modeled as four node rectangular plane stress elements which are assumed to have LE behavior throughout the analysis. Gross section properties are used for the concrete which is assumed as monolithic for LE analysis. The masses are assumed to be concentrated at floor levels. The fundamental period is determined to be 0.41 sec. For comparison purposes, the fundamental period is also determined using BOCA's empirical equation,

$$T_a = C_T(h_n)^{3/4} \tag{3}$$

where, h_n is the height of the building and C_T a coefficient based on seismic resisting system taken hear as 0.02. This results in a value of 0.66 second for the fundamental period, which in this particular case is not conservative (larger period results in a smaller design force) compared to the value obtained from dynamic analysis. In general, since code empirical equation primarily depends on the structural dimensions and neglects relative member proportions and properties and the details of the configuration, the results in most cases do not agree with those of a computer analysis, although they are generally on conservative side.

For the elastic strength demand calculation, the method suggested by BOCA National Building Code (BOCA 1996) is used in this example. According to this code, in addition to the structure's self weight, partition loads, mechanical equipment weight and snow load, 25% of the floor live loads should also be considered in calculation of the total weight effective to induce lateral seismic forces. The result is a total effective seismic dead load W=12476 kN. For convenience, before the calculation of the actual base shear, an arbitrary base shear V=1000 kN is distributed over the height of the structure according to BOCA provisions using the following equation:

$$F_x = C_{vx} V; \; C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{4}$$

where, F_x is the story lateral force, C_{vx} is the distribution factor, w_i and w_x are the weight of story i and x, and h_i and h_x are the height above the base to story i or x.

Parameter k is taken as 1 for $T \leq 0.5$ second, 2 for $T \geq 2.5$ seconds, and determined by linear interpolation for 0.5 second $\leq T \leq 2.5$ seconds. The results are tabulated in Table 2.

The static analysis of the finite element model is also carried out by using DRAIN-2DX. The arbitrary 1000 kN base shear distributed over the height produced a maximum top displacement of 0.73 cm.

According to BOCA (1996), the seismic base shear V_{base} is calculated based on the following equation:

$$V_{base} = C_s W; \; C_s = \frac{1.2 A_v S}{R T^{2/3}} \tag{5}$$

Table 2 Distribution of 1000 kN base shear over the height of the structure

Level	C_{vx}	F_x (kN)
Roof	0.185	184.6570
9	0.163	163.0685
8	0.145	144.9498
7	0.127	126.8311
6	0.109	108.7123
5	0.091	90.5936
4	0.072	72.4749
3	0.054	54.3562
2	0.036	36.2375
1	0.018	18.1187
Total	1.000	1000.0000

Table 3 Factors selected for calculation of *ESD*

Effective peak velocity related acceleration, A_v	0.15
Site coefficient for soil characteristics, S	1.2
Response modification factor, R	1.0
Fundamental period, T	0.40 second
Total seismic weight, W	12476 kN

where W is the total effective seismic dead load, C_s seismic coefficient, A_v is the coefficient representing effective peak velocity-related acceleration, and S is the coefficient for the soil profile characteristics of the site. The values for the parameters used are listed in Table 3. The resulting value of C_s is 0.40, which gives the elastic strength demand (*ESD*) base shear V_{base}=4990 kN. Using the result of linearly elastic analysis with 1000 kN base shear, the maximum elastic roof level displacement due to the distribution of V_{base}=4990 kN over the height of the structure can be obtained by proportion as δ_e=4990/1000 (0.73 cm)=3.66 cm.

In this example, the same code (BOCA) is used for calculation of both *ESD* and F_y. The R value according to BOCA is 4.5, which results in F_y value of 1109 kN. This value of R resulting from the ratio of *ESD* to F_y or a different value more suitable for the specific precast lateral load resisting system under study shall be used to determine maximum global plastic displacements and local plastic rotations under the ultimate seismic loading condition so that a rational detailing of the connections can eventually be done. The maximum global plastic lateral displacement is to be determined using the EEC (according to the original Clough's method) or through the use of IDRS as suggested here.

Although the R value for monolithic reinforced concrete bearing wall construction is given as 4.5 by both BOCA and NEHRP Recommended Provisions (BSSC 1994), Chapter 6 of NEHRP suggests restricted response modification factors for precast construction which can be as low as 1/2 the value suggested for monolithic construction. Such recommendations are based on research results referenced in the commentary of NEHRP and can be used even if another code is being considered. In this example, an R_{design} value of 3 is used. This results in the required design ultimate capacity $F_{y\ design}$ of 1663 kN. Based on the value of R_{design}=3 and T=0.4 second, the displacement ductility demand μ is obtained from the chart with 5% damping and no strain hardening. With the resulting ductility demand of 2.75, the maximum plastic displacement is determined to be 2.13 cm. Fig. 14 shows the plot of base shear versus displacement for this example. It is of interest to compare the result just obtained with that based on the original Clough's method using the same analytical model. Since the period T=0.4 second is between 1/8 second and 1/2 second, the EEC applies. It is noted, of course, that Clough's method is applicable for periods greater than 1/8 sec. In this case, with R value of 3, the ductility demand can be calculated as $\mu=(R^2+1)/2=5.0$. This results in the maximum plastic displacement of 4.88 cm. This value is more than twice the value 2.13 cm obtained using the charts for displacement ductility spectra. It can be seen that for such periods, the original Chough's mothod is overly conservative.

The maximum plastic displacement (assumed to be at the roof level) calculated can then be used to perform a kinematic analysis with prescribed joint openings to determine corresponding deformations at plastic hinges. The number of horizontal joint openings depends on the distribution of cracking at these locations over the height of the structure. Analytical studies (Schricker and Powell 1980) have shown that such cracking is concentrated in the lower levels for precast walls.

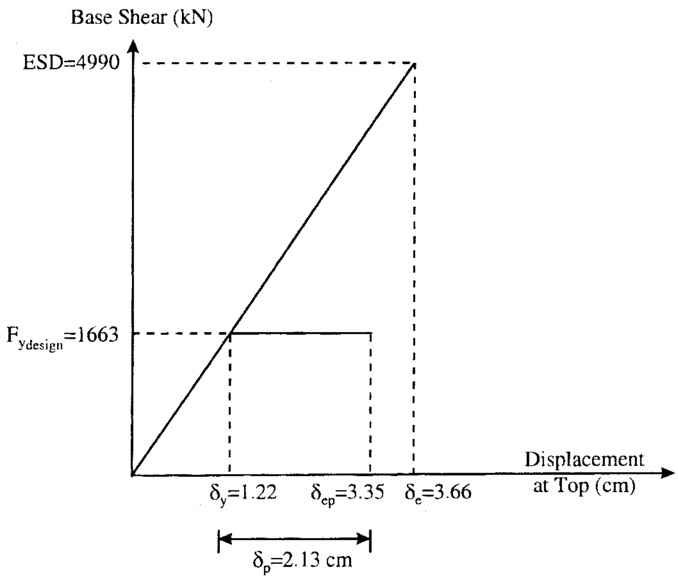

Fig. 14 Prediction of global inelastic displacement

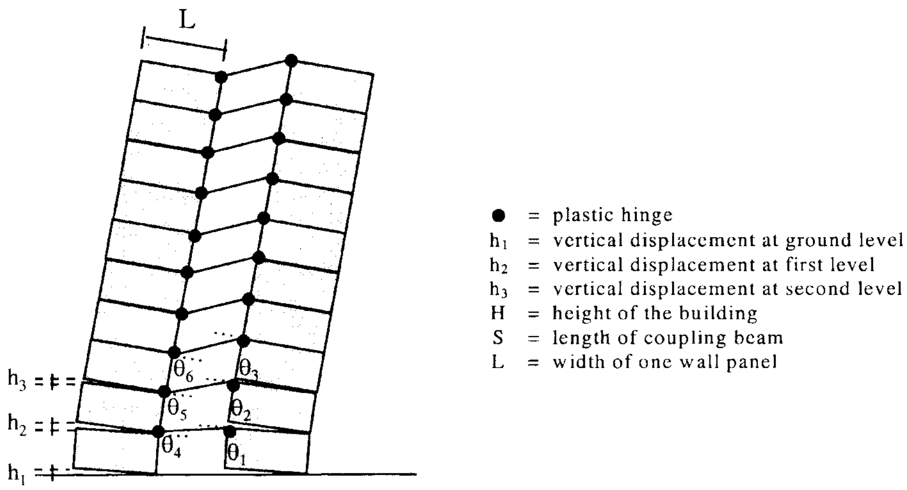

Fig. 15 Kinematic analysis with 3 joint openings

The plastic rotation of wall panels corresponds to plastic strains in steels crossing the joints. Based on kinematic analysis with three joint openings for the failure mechanism illustrated in Fig. 15, the equation for rotation angles shown in Table 4 can be derived (Yu 1992). The parameters used in Table 4 are explained in Fig. 15.

The total vertical plastic displacement for the three joint opening case determined approximately as w δ_p/H is 0.48 cm. Assuming 50%, 30%, and 20% of the total vertical displacement to occur, respectively, at the first, second, and third floor levels, we get h_1=0.24 cm, h_2=0.14 cm, and h_3=0.10 cm for the example building under consideration. The resulting plastic hinge rotations obtained using the equations in Table 4 for the modified and the original Clough's method are tabulated in Table 5.

Table 4 Equations of rotation angles for three joint openings (Yu 1992)

Rotation	Equation	Rotation	Equation
θ_1	$\frac{h_1}{L}$	θ_4	$\theta_1\left(1+\frac{L}{S}\right)$
θ_2	$\frac{b}{L}-\theta_1$	θ_5	$(\theta_1+\theta_2)\left(1+\frac{L}{S}\right)$
θ_3	$\frac{\delta_p-a-b}{0.8H}-(\theta_1-\theta_2)$	θ_6	$\theta_1+\theta_2+\theta_3\left(1+\frac{L}{S}\right)$
$a=\frac{0.1Hh_1}{L}$	$b=\frac{h_1(L-h_2)}{L}+h_2$	$c=\frac{0.1Hb}{L}$	

Table 5 Hinge rotations

Rotation	Value (rad)	Value from EEC (rad)	Rotation	Value (rad)	Value from EEC (rad)
θ_1	0.0003534	0.0007420	θ_4	0.0017618	0.0036999
θ_2	0.0002119	0.0004590	θ_5	0.0028185	0.0059884
θ_3	0.0001402	0.0004310	θ_6	0.0035175	0.0081376

It can be seen that the plastic rotations obtained using the original Clough's method are between 2.1 and 3.1 (with an average of 2.3) times the corresponding values based on the modified method suggested here.

To account for the oscillatory seismic response, Clough's method recommends that connections be designed for a number of reversed cycles of loading and deformation depending on seismic code zonation, the natural period, and the response modification factor. Design of connectors to perform satisfactorily during severe earthquakes, accordingly, is based on deformation capacities with the consideration of load reversals and satisfaction of strength requirements. Such recommendations are the essence of the proposed rational seismic performance criteria for effective detailing of connectors and are specifically suggested for precast concrete structures. These steps will provide the designer with a clear insight into the desirable deformation behavior of the connector to be designed. The connectors, however, are not unique and depending on the member types, suitable connector types can be designed and tested for performance verification.

6. Conclusions

In this study, two alternate methods based on collapse mechanism analysis to estimate plastic hinge rotations suitable for seismic assessment/design of structural concrete have been reviewed, one initially proposed for seismic assessment of existing reinforced concrete buildings to be extended later with some modifications for design of precast concrete buildings and another for design of new precast jointed wall panel buildings. Similarities and differences of the methods have been discussed. The deficiency with original Clough's and Park and Paulay's methods is their reliance primarily on EEC to obtain a relation between displacement ductility demand and response modification factor for relatively short period ranges. It has been shown that by developing charts in the form of displacement ductility demand more reliable values for ductility demand can be obtained. The proposed modification is thus applicable to ranges considered by Park and Paulay's method as well as Clough's method. Parametric studies performed on the derived ductility spectra show that for stiff structures (e.g. foundamental period less than 0.4 sec), the ductility demands increase very rapidly as R values increase. For such stiff structures, the ductility demands approach numerical values of response modification factors as period increases, which indicates the range where the EDC applies. Further parametric studies showed that post yield strain hardening results in reduced ductility demand compared to elastic perfectly plastic models. The effect of damping was found to be particularly small on displacement ductility demand in the developed spectra.

The example worked out showed that the result obtained by the original suggested method is highly conservative compared to that obtained based on the modified approach suggested in this paper. The study has contributed to the efforts in formulating approaches for quantifying deformation demand for seismic resistant design of connections in jointed precast construction. The methods reviewed along with modifications suggested here are suitable for practical seismic assessment of existing buildings or design of new structures.

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