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Influence of irregularity on the q factor of RC frames

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ABSTRACT: A parametric study of the strength reduction factor (q) of vertically irregular plane reinforced concrete frames is presented. Frames considered have different first story heights and are designed according to the Eurocode EC8. For the estimation of q a computer design - nonlinear analysis algorithm is used that adopts collapse criteria of global or interstory drift and local curvature ductility comparisons at all the Critical regions.

1 INTRODUCTION AND STATEMENT OF THE PROBLEM

Modern seismic provisions recognize that a certain amount of damage can be tolerated under moderate seismic response of buildings. Following this philosophy internal forces obtained typically from a regional or site dependent response spectrum are reduced using a response reduction factor, denoted q, following the notation of the EC8 [CEC 1988] investigated herein.

Since ductility has been taken advantage of in seismic design [Newmark and Riddell 1979], the value of such a response reduction factor, has been determined primarily from single degree of freedom studies of inelastic systems and evaluation of actual or model building response during past earthquakes [Bertero 1986, Uang 1990, Wood 1992].

Single (or two) degree of freedom models emphasize the base excitation content, intensity and duration [Al Sulaimani 1985, Hadjian 1989, Fajfar et al 1990, Fischinger et al 1990, Zeris 1990]. In these studies the structure is idealized as a simple oscillator of relatively few parameters. This idealization offers significant information primarily on drift and drift ductility associated indices, is, however, unable to address other collapse criteria such as local damage and energy dissipation under inelastic cyclic response.

When local damage criteria are considered, the interaction of all influencing factors (form and mechanical characteristics of the resisting system, design rules in effect and type of base excitation) need to be quantified through multidegree of freedom analyses. At this level of idealization, the study of q is not adequately substantiated; isolated parametric studies [Tassios 1986, Kapos 1991] have been considered for relatively regular frames. The objective of the present study is to investigate analytically the q factor of plane frames having irregularities with height, setbacks or other changes in topology.

For the evaluation of the q factor, two methods have been proposed and are both possible to follow with the algorithmic procedure described: in the direct method, a set of collapse criteria is adopted together with a design q and peak ground acceleration. The structure is repetitively designed for the initially assumed and subsequently decreasing or increasing q values. Each design is then analyzed in the time domain for the design earthquake (or a collection of earthquakes whose elastic response spectrum is represented by the design one) to the point that a collapse criterion is just satisfied. This limit q is referred to as the actual response reduction factor of this system.

In the indirect method, a set of collapse criteria, a design q and a peak ground motion are chosen. The structure is designed and subsequently analyzed under increasing or decreasing ground acceleration to the point of satisfying a collapse criterion in the limit. The actual response reduction factor of the system is equal to q scaled by the ratio of collapse to design level acceleration.

Both methods yield a q factor dependent on the implicit assumptions made: the assumed elastic design response spectrum, regulations of the specific code used in design (such as for instance, method of analysis, load factors for design, capacity design requirements, reinforcement detailing), assumed base excitation(s) and material strength reduction factors at the design and analysis stages of the process.

Furthermore, the direct method has the drawback that it may not converge to a definitive result. Such a lack of convergence has been observed in cases where the relative influence of gravity loads in the system is high or when the requirements for capacity design and minimum reinforcement limits set by the code govern the inelastic performance of the system for increasing q.

2 EVALUATION OF THE RESPONSE REDUCTION FACTOR

As part of a broader investigation of the response reduction factor of irregular frames designed by the Eurocode 8 [CEC 1988], two research projects are pursued at the NTU Athens. In the first study the response reduction factor for 1/5.5 scale plane reinforced concrete frames is determined through shake table testing. The objective of the second study is to investigate several of the problem parameters for q estimation, through analytical studies of inelastic response of plane frames, results of which are described herein.

In order to perform a reliable parameter study that is based on a sample of objective designs satisfying the code, a q estimation algorithm is used as described herein. Both the direct and the indirect methods [CEC 1988] are possible to follow: in the former method, successive (re)designs of a system are investigated for collapse under a specified excitation of design level intensity. In the latter method, following initial design of the structure the base intensity is iteratively scaled until collapse, yielding an estimate of the q factor. During the design phase, common to both methods, different capacity design measures for weak beam-strong column response can be considered for comparison.

2.1 Description of the algorithm

The evaluation procedure encompasses several different modules for the design, linear and nonlinear analysis and collapse evaluation of reinforced concrete plane frames. Only frame members are considered; infill masonry or reinforced concrete walls cannot be considered in the model yet. The process is fully automated once the type of resisting system and key design parameters are defined. Input parameters are the basic building configuration and member sizes, material properties and load information (namely load factors for dead and live load combination, density, live load and tributary slab widths at each floor, additional weight of partitions).

1. Elastic analysis. The structure is initially analyzed by a linear analysis program of wide applicability [Wilson et al. 1973] for prediction of periods, mode shapes and design force resultants for the members. All applicable load cases are considered, gravity and seismic (both the equivalent triangular inertia load distribution in two directions and spectral combination). A first pass check of the building is performed to verify that the structure satisfies the code for allowable drift limits and possible influence of second order effects.

2. Section design and capacity evaluation. A minimum longitudinal reinforcement design is performed for each critical section, with the inherent assumption that beams at midspan do not yield: all critical regions monitored are at the member ends.

Material properties used assume a user input capacity reduction factor and the stress-strain characteristics and limit points adopted in the Eurocode. In a typical design procedure the section steel ratios p and p′ (=ap) are varied within code limits keeping a fixed to 1.00 for columns and variable for beams starting from an initial value of 0.5.

Following selection of steel area the capacity of each critical section is evaluated assuming possible material strength amplification factors that may be assumed (e.g. material overstrength from characteristic to design or mean value). The capacity of the beams is evaluated prior to design of the columns so that joint overstrength coefficients can be determined for these members.

3. Critical section inelastic analysis. A fiber section representation [Zeris and Mahin 1986] is used to evaluate the yield and ultimate curvature and the ratio of maximum to yield moment in positive and negative bending. For the ultimate curvature supply either the tension steel fails (assumed at 18% elongation) or the concrete compression strain exceeds the maximum specified, as defined by the transverse volumetric steel ratio of the member [CEC 1988]. For columns, the above section characteristics are established at five axial load levels: 80% of ultimate tensile capacity, zero axial load, 25%, 50% and 80% of the balanced load. In between values are linearly interpolated. Axial loads outside this region, if encountered, imply collapse.

4. Structural inelastic analysis. Using the above mechanical characteristics a nonlinear computer model of the structure is formulated and analyzed in the time domain to iteratively establish the peak ground

acceleration leading to collapse. The program DRAIN2D [Kanaan et al. 1973] is used, with extended output capabilities. All members are modelled with lumped plasticity models accounting for axial load bending capacity interaction and second order effects according to program conventions.

5. Checks. At the end of each iteration, collapse is checked for exceedence of the following criteria:

a) absolute drift limits (total roof drift and interstory drifts at each floor) set by the user; a value equal to 2.5% of the total or clear height, respectively, is assumed.

b) local curvature ductility exceedence at critical regions of monitored members.

Curvature ductility is defined as the ratio of demanded curvature to the yield curvature; for columns, this is a function of the acting axial load. The curvature demand is evaluated from the time histories of inelastic rotations, taking into account the current plastic hinge length; this length is determined at each step from the equilibrium bending moment diagram of the member, the ratio of ultimate to yield moment for the acting axial load and the clear member length. However, to avoid unusually high demands, a lower bound of half the effective depth of the member is imposed.

2.2 Iterative procedure and improvements

A repetitive application of steps (1) to (5) above constitutes the direct method, with the design q being the traced parameter. A single application of steps (1) to (3) followed by an iterative application of steps (4) and (5) constitutes the indirect method, with the ground acceleration as the traced parameter. In either case the traced parameter is (de)incremented according to the collapse status of the structure until two bounds are established, with subsequent iterative refinement.

Improvements of the procedure under implementation include a) the definition of an additional monitoring point at midspan of all beams, for low rise frames, b) a critical section check accounting for number of cycles.

3 INVESTIGATION OF THE Q FACTOR OF THREE IRREGULAR FRAMES

The algorithmic procedure described above is demonstrated in the evaluation of the q factor of three six story, three bay reinforced concrete frames. Additional accelerogram inputs and building irregularities are given elsewhere [Zeris 1991]. Due to space limitations, the indirect approach is presented herein. All frames have a center span 3.0m long and two side spans 5.0m long [Fig.1]. They differ only at their ground story, being 5.0,

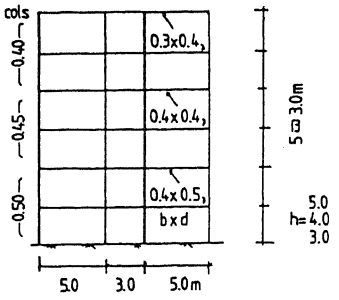

Figure 1. Frame geometry.

4.0 and 3.0 m tall (B5, B4 and B3). Only planar response is considered. All frames, designed as ductility class II structures for a peak ground acceleration of 30% acceleration due to gravity, are excited at the base by the transverse component of the 1986 Kalamata accelerogram; this record has a similar peak ground acceleration and is chosen herein because, despite its relatively short strong motion duration, it exhibits the characteristic pulse content of near field motions.

The inelastic response spectra of the record used are compared with the design spectrum of the EC8 in Fig.2. The spectra

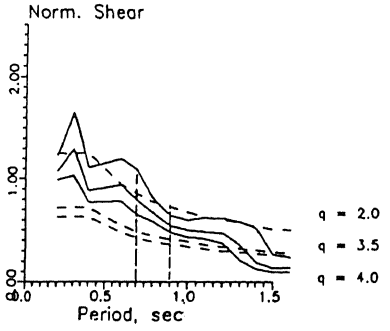

Figure 2. Inelastic spectra for the base input used.

indicate the dependence on demanded resistance (normalized by the peak ground acceleration) in order for the oscillator to exhibit a specific drift ductility; spectra for ductilities 2., 3.5 and 4. are shown. Single degree of freedom inelastic analyses have been performed using an analysis program by Mahin and Lin [1983]; the program was extended to evaluate resistance spectra

for specified ductility. The oscillator is assumed to be bilinear with no hardening, while damping is assumed to be 5% critical.

The first mode periods of the frames are from 0.70 to 0.90 seconds. In this period range, the more flexible structures exhibit base shear resistance lower than the normalized elastic design spectrum demand. The inelastic response spectra of this earthquake are consistently above the design ones, indicating that the normalized resistances required to achieve a given ductility consistently lie above the design requirement for the same design q.

The buildings are designed with a design q of 3.5, observing all code regulations for capacity design in the beam-column joints, allowing for a 20% increase in the beam capacity due to steel hardening. All section inelastic characteristics are evaluated assuming design material properties. Curvature ductility lower bound limits set by the code are checked at the critical regions, assuming that the plastic hinge length is half the effective depth of the member. A constant time step of 0.005 sec is used in all inelastic analyses, spanning strong motion portion of the Kalamata transverse accelerogram. Mass and stiffness proportional damping coefficients are selected such that the structural damping is 5% critical in the first two modes.

The base shear - roof displacement characteristics of each frame are initially compared in Fig.3. In each case, frames are loaded under a monotonically increasing

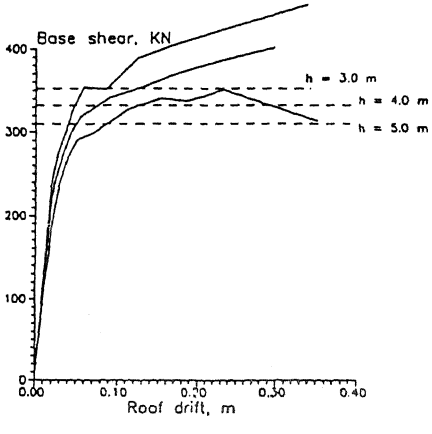

Figure 3. Inelastic base shear-roof displacement characteristics of the three frames.

triangular load pattern; for comparison, the design load levels for each frame are also shown. As expected, the influence of second order effects is more pronounced for the tall first story structure, which, being the most flexible, also exhibits the lowest design base shear and ultimate strength.

Beams are assumed to have a post yield stiffness 5% of the initial, while columns do not exhibit any post yield hardening. The capacity design procedure used ensures delayed yielding of the columns under static lateral loading. Columns at the base yield at around 7 cm roof drift well after the beams. Frame B5 exhibits higher interstory drifts at ground level and hence earlier yielding of the ground columns; thus, relatively higher column rotational demands are observed in this structure for increasing drift. As an indication, at 20 cm roof displacement, the demanded plastic rotations are between 1.55% and 1.30% rads for the three structures.

The dynamic inelastic response of each frame, under increasing base excitation intensity, is influenced by the extent of irregularity present. In Fig. 4 are shown the interstory drift envelope profiles exhibited by each frame under increasing excitation intensities until a maximum value of 2.5% interstory drift is attained. Excitation intensities are not proportional but follow the collapse status of each structure.

The variation of maximum interstory drift (between all floors) and peak inelastic rotational demands of interior first floor beams and exterior ground column (base) with base acceleration is shown in Fig.5; base excitations are normalized by the design input intensity. Frame B3 attains its peak interstory drift consistently at the third floor. The structure exhibits a desirable type of response by which lateral drifts are fairly uniformly spread over the entire height, with significant deviations only at the top story. Frame B4, though localizing drifts at the base does distribute somewhat inelastic deformations to the higher stories; on the contrary, in frame B5 all local inelastic deformations concentrate at the ground story [Fig.4]. The consequences from the formation of a soft story at the base in frame B5 are clearly evident considering the rate of increase in inelastic rotations at the beam and column: under increasing excitation intensity, the maximum rotational demand in the beams is reduced while all column rotations at the ground increase at an increasing rate.

Frame B5 will be unable to reach 2.5% drift unless a high ductility supply is ensured at the base column. For an estimated yield curvature of 0.6% 1/m, uniform over the range of axial loads encountered herein, the corresponding maximum supplied inelastic rotation at the base is 0.7% and 1.4% rads for curvature ductilities of 5 and 10 (a value in excess of the minimum specified for type II structures) respectively. As evidenced in Fig.5, under the curvature ductility of 5 the structure is unable to provide the assumed q factor of 3.5.

Increased confinement allows for an increase of the q factor for this building to 4.7.

Frame B3, exhibits a fairly constant rate of increase of all inelastic demands under increasing intensity. Assuming the previous limits of inelastic rotations, the corresponding q factors for this frame are 4.4 and 7.1. Finally, for frame B4, the design q of 3.5 seems to be sufficient if a curvature ductility of 5 is ensured in the columns. However, this structure is able to reach a maximum value of 4.6 only prior to developing significant interstory drifts.

4 CONCLUSIONS

A computer algorithm is presented for the evaluation of the strength reduction factor, q, of plane reinforced concrete frames designed by the Eurocode for Seismic Regions. Iterative nonlinear time history analyses following an automatic linear analysis and design procedure are employed to establish the inelastic demands in the critical regions under the imposed base excitation. Adopted criteria for estimating q include conventional drift limits as well as local curvature ductility checks in all critical regions.

The method is demonstrated for three reinforced concrete frames having similar geometry and different first story heights. For those specific frames, the controlling criteria for defining q are local demanded ductilities or global instability rather than drift. The estimated response reduction factors are higher than those assumed for design but for the frame with a relatively tall first story, for which tighter local detailing restrictions are needed in order to achieve the design reduction factor assumed in the design.

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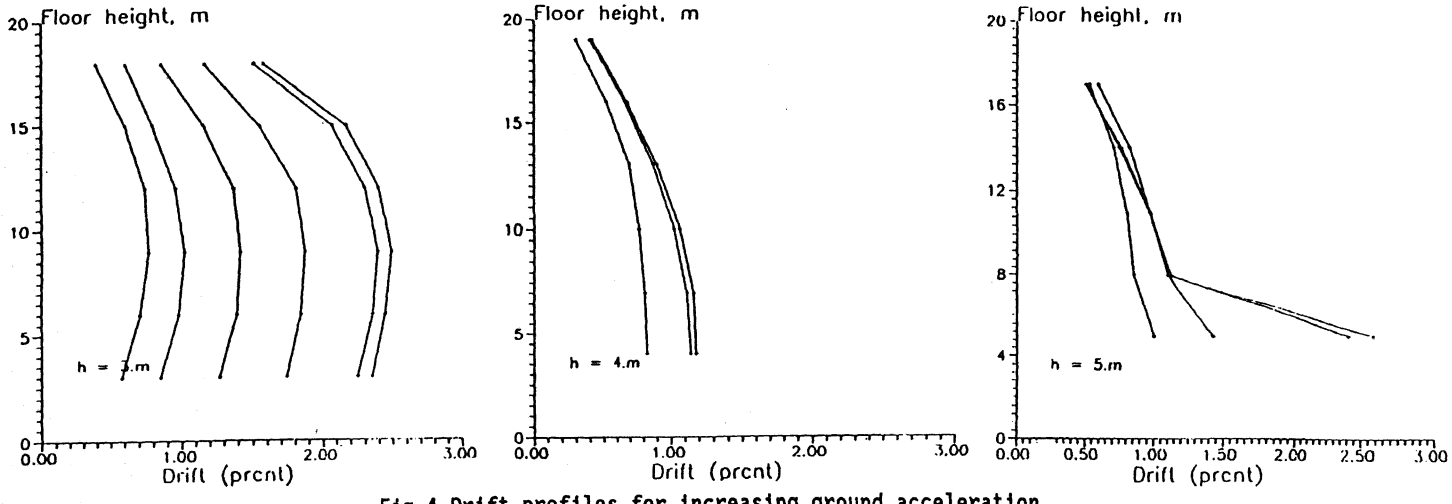

Fig.4 Drift profiles for increasing ground acceleration.

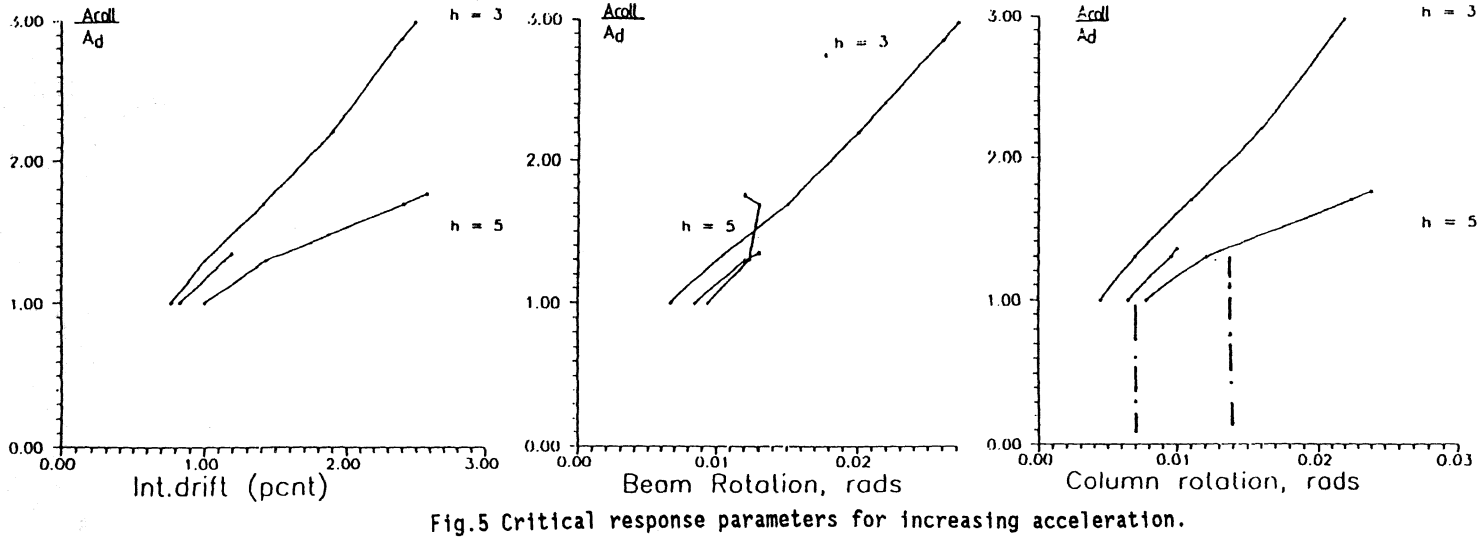

Fig.5 Critical response parameters for increasing acceleration.