ASIAN JOURNAL OF CIVIL ENGINEERING (BUILDING AND HOUSING) VOL. 8, NO. 4 (2007)
PAGES 405-421

CONSEQUENCE OF STRENGTH-DEPENDENT STIFFNESS ON TRADITIONAL SEISMIC DESIGN APPROACH

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ABSTRACT

Traditional seismic design approach uses initially assessed natural periods as fixed design parameters for any structural system ignoring its dependency on original strength provided to meet elastic strength demand computed on the basis of such periods, divided by the response reduction factor. This implies the consideration of a constant stiffness obtained due to initial period while a varying yield displacement inversely proportional with response reduction factors. However, a series of recent studies shows that the reality being completely otherwise, the yield displacement of any structural system remains almost constant making assigned strength and hence, resulting stiffness both varying inversely with response reduction factor considered in the design. This reality puts a question mark on the validity of the existing approach. The present paper is a limited effort to resolve this issue. To achieve this end, the strength demand, inelastic displacement demand and ductility demand of a large category of systems encompassing all feasible combinations of lateral period and response reduction factors are computed through both the approaches and compared. While computing the inelastic response, elasto-plastic as well as various feasible degrading hysteresis behaviours, have been used. The study indicates that the traditional approach yields a safer estimation of the strength and ductility demand except for system with very stiff periods. On the other hand, inelastic displacement demand is underestimated by this traditional approach, which can be by and large compensated by multiplying this displacement demand by response reduction factor.

Keywords: Seismic behaviour, Strength-dependent stiffness, inelastic response, traditional seismic design approach.

1. INTRODUCTION

For striking a balance between economy and safety, currently practiced seismic design strategy limits the strength of the structural members such that the structure undergoes vibration in the post-elastic range in the event of strong ground shaking, without exceeding

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ultimate deformation. On the other hand, the systems are designed to behave elastically only under minor to moderate earthquake. Such a design philosophy often referred as dual-design approach [1] is expected to lead to a satisfactory performance for systems having appropriate distribution of strength among the lateral load-resisting structural elements.

To this end, in the conventional practice, design starts with the premise that the stiffness of the structure and hence the period of the same can be reasonably estimated. Elastic strength demand of the system is then derived based on the elastic design spectra. Strength demand so estimated is generally reduced by a factor R_μ known as response reduction factor depending on the level of inelastic excursion it is allowed to undergo during severe shaking to assess the design strength of the system. However, in this process, it is assumed that the stiffness or the period of the system remains unaffected due to such strength assignment philosophy. However, such fundamental behaviour of lateral load resisting structural elements (LLSE), which is the basis of implementing seismic design provisions embodied in most of the current codes [2], is perceived to be grossly erroneous in a few recent studies [3-6].

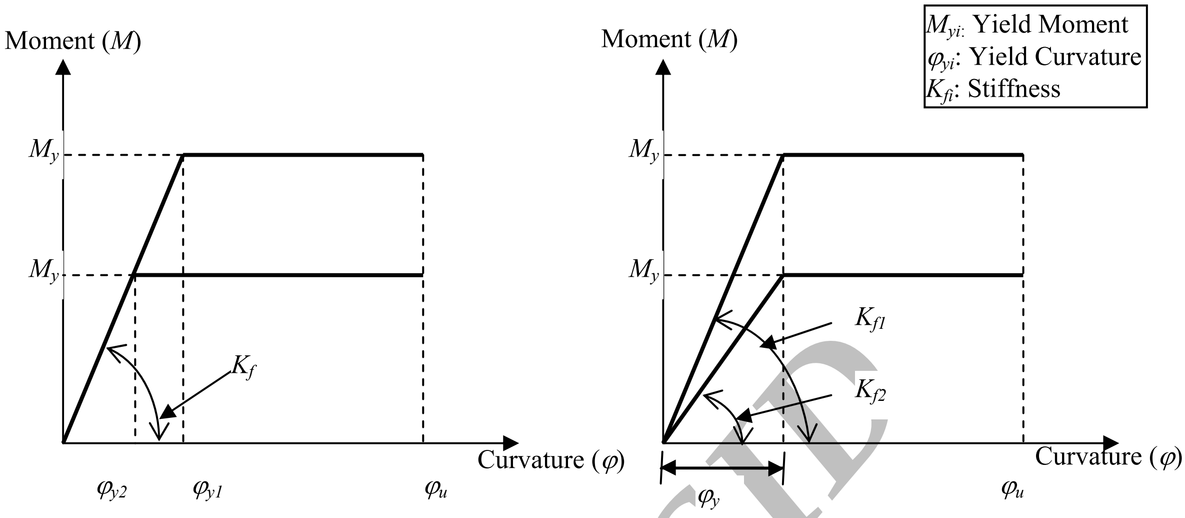
It is established that, for lateral load-resisting structural elements (LLSE) such as piers, flexural walls and moment-resisting frames, the stiffness is essentially dependent on the strength assigned. Traditionally assumed behaviour of LLSE in contrast to the realistic one is illustrated in Figure 1. Figure shows that the yield deformation of structural members remains almost invariant resulting in a proportional change in stiffness with the yield strength as opposed to the traditional constant stiffness assumption independent of strength.

Thus, this understanding clearly indicates that the period on the basis of which the structure is designed and the actual period of the structure as dictated by the strength attributed to it, are different. This may bring about a sea change in the strength demand as well as the inelastic demand as compared to the same estimated in the conventional design process. In this context, the objective of the present paper is to compare the seismic response obtained through these two different approaches and to asses the applicability and limitations of the conventional design. The study may prove useful for the purpose of practical design highlighting on the deficiencies of rudimentary design philosophy.

2. BEHAVIOUR OF LLSE

Traditionally assumed Behaviour of Lateral Load-resisting Structural Elements (LLSE) is described in Figure 1(a). It is evident from such an assumption that the strength and stiffness are two independent characteristics, which are of doubtful validity for many structural materials.

Idealized response of the typical structural wall subjected to the lateral force at the top shows that the walls with similar geometry approximately yield at the same curvature regardless of the strength of the wall [3,7]. Such yield curvature (φ_y) may reasonably be approximated as



(a) (b)

Figure 1. Moment-curvature relationships considered: (a) Traditionally assumed relationship and (b) moment-curvature relationship as per recent studies [4,5]

$$\phi_y = \frac{1.56 f_y}{E_s l} \tag{1}$$

where, f_y = yield strength of steel; E_s = Modulus of elasticity of reinforcement; l = Overall height of the structural element. This moment-curvature relationship may be transferred to force-displacement relationship using a yield displacement (Δ_y) given by Eq. (2)

$$\Delta_y = \frac{\phi_y l^2}{3} \tag{2}$$

Such a load-displacement relationship may be assumed to remain unaltered due to the influence of moderate axial compression expected in multistory buildings [4]. The essence emerged from this discussion leads to the understanding that the yield displacement, for the purpose of seismic design, may be considered as nearly invariant. It is, thus, evident that a very realistic approximation for stiffness (K_i) is [5],

$$K_i = \frac{V_{ni}}{\Delta_{yi}} \tag{3}$$

where, V_{ni} is the nominal element strength. This leads to the realization that stiffness of LLSE is dependent on strength, while the yield displacement is nearly a constant [Figure 1(b)]. Similar observations along with experimental evidences and its applicability for beams, columns and moment-resisting frames are available elsewhere [8,9].

3. AIM OF THE STUDY

The present paper makes a limited effort to arrive at a conclusive end regarding the relevance of traditional seismic design in the light of the recently perceived behaviour of LLSE, i.e. stiffness proportional to strength. A large number of single degree of freedom systems with lateral period varying over a feasible range of about 0.1sec to 4.0sec, are considered covering the fundamental periods of most of the practical structures. Each of these systems are designed under representative earthquake histories with various values of response reduction factors (R) namely R = 1, 2, 4, 6 to achieve various levels of inelasticity. The inelastic displacement and ductility demand of these systems are obtained on the basis of initial stiffness corresponding to initial period of the system calculated from code specified period formulas. Further the strength of these systems are provided as the strength demand obtained through an elastic time history analysis divided by the respective response reduction factor.

The inelastic seismic response of these systems is then studied under simulated ground motion and noted. Further the system stiffness is reduced by dividing the initial stiffness by response reduction factor, R, to make the stiffness proportional to the provided strength, keeping yield displacement constant as per the philosophy of strength dependent stiffness. The strength is assigned after the recent version of Indian Earthquake code [10] based on the initial periods. In this code, depending on the choice of combinations of various factors like zone factors, response reduction factors and importance factor, a possible maximum and minimum values of strength can be arrived at. Both of these two possible strengths, referred as possible maximum assigned strength and possible minimum assigned strength, are considered for any system. This consideration is made to see the impact of the codal constant on strength dependent stiffness and the resulting post-elastic range seismic response of the system.

3.1 System, Analysis and Ground Motion

The details of the inelastic systems as specified through the choice of hysteresis models, the methodology of analysis adopted and the earthquake records used are discussed in the following subsections with adequate reasonings.

3.2 Hysteresis model and Damping

A simplified hysteresis model proposed in literature [11] to represent the behaviour of reinforced concrete (RC) member under cyclic loading is considered in the present investigation. The stiffness and strength deterioration characteristics are incorporated in this model with a bilinear backbone curve. In absence of any well-established value for the strain-hardening ratio, backbone curve is chosen as elastic-perfectly plastic. The model involves three input parameters, viz., initial yield strength (F_y), initial loading stiffness (K) and strength deterioration factor (δ) which is assumed to be a constant during the entire load history. Experimental studies reveal that the average fractional strength drop (δ) due to each inelastic excursion for most of the normally detailed RC structures may be as high as about 10% while very often it exhibits a drop of around 5% [11]. Hence, the present study considers $\delta = 0.05$ in each yield excursion. However, sometimes steel structural members or some particular variety of concrete structural members predominantly exhibits stiffness degradation without much of strength deterioration. The results are also obtained with $\delta = 0.0$ to simulate only stiffness degrading behaviour. The model computes the degraded strength by deteriorating the yield strength by a fixed fraction equal to average strength deterioration factor (δ) computed by dividing the total drop in strength by the number of yielding, expressed as a percentage of initial yield strength. The stiffness of the elastic loading portion of the load-displacement curve is obtained using the principle similar to Takeda's model [12]. The elastic loading branch targets the previous point of unloading on the same side (either positive or negative) of the load history; and thus deteriorated loading stiffness is automatically calculated. Unloading elastic stiffness remains same as initial stiffness before and after yielding. Steel framed structures are very often idealized to exhibit a pure elasto-plastic behaviour. Another hysteresis model prescribed in the same literature [12] is suitably used to simulate elasto-plastic and only strength-degrading behaviour. The responses of all the cases are obtained for each of these two idealized hysteresis behaviours also to provide a better insight into the behaviour of steel framed structures and to understand the impact of stiffness and strength-degrading features separately. Damping is considered as 2% and 5% of critical damping for steel and concrete structures, respectively in elastic condition and kept as constant during the inelastic range behaviour.

3.3 Ground Motions

Inelastic seismic response may be sensitive to many factors, e.g., frequency content, pattern of pulses and number of records used [13]. Thus, to perceive the response of the system in the inelastic range, the present paper, in its limited scope, primarily considers records compatible with design spectrum. In this context, design spectrum of old Indian earthquake code [14], derived from that of Housner [15], is chosen. Two uncorrelated artificial ground motions compatible with this design spectrum are generated using a procedure outlined in the literature [16]. These synthetic earthquake acceleration histories are also used in some other studies [11,17]. The target design spectrum as well as the response spectrum regenerated from one of these time histories is presented elsewhere [11]. The same reveals a close resemblance between these two spectra. In this context, mean response of the systems under these two time histories is depicted in the present paper to achieve insight into the overall behaviour.

Furthermore, acceleration time histories consisting of two ground acceleration data in two mutually orthogonal directions, collected during Imperial Valley Elcentro earthquake in 1940, are used to obtain the response of the systems under this ground excitations. The average response obtained under these two ground acceleration-time histories in two mutually orthogonal directions is illustrated. Such averaging may help at least to eliminate the sporadic effect of the individual time history to some extent.

3.4 Method of Analysis

For arriving at the representative heights for buildings with various fundamental lateral periods, empirical relations provided in recent version of Indian Seismic Code [10] has been used. Since yield displacement of these structures are found to be a constant percentage of this representative height [9], hence this plays an important role to arrive at the correct value of nearly constant yield displacement. Though such empirical relationships slightly differ in various codes, using a particular code cannot be a major source of variation in understanding. Following Indian Seismic Code [10], the height of a building (*H*) without brick infill can be expressed in terms of the fundamental period through the following relationship.

$$H = \left(\frac{T}{0.075}\right)^{1/0.75} \tag{4}$$

The similar relationship for all other buildings, including moment-resisting frame with brick infill panels can be expressed as follows

$$H = \frac{T \times \sqrt{d}}{0.09} \tag{5}$$

where, *d* is the base dimension of the building frame. The yield displacement being a geometric property of a cantilever element is inversely proportional to the length of the element and thus does not bear any relationship to flexural rigidity EI [4]. For A36 grade steel, the yield displacements are seen to be about 0.75% of the height [9]. Hence, for the purpose of the analysis, the yield displacement (Δ_y) is calculated as $\Delta_y = (0.0075)H$, *H* being calculated as equations (4) and (5) derived after the recent version of Indian code [10]. Then, the design horizontal seismic coefficient (A_h) and base shear (V_b) can be computed as $A_h = (ZI/2R).(S_a/g)$ and $V_b = A_h.W$ respectively following the same standard [10]. If this strength is attributed to the lateral load-resisting element of the system then the resulting element stiffness (K_{mod}) and corresponding modified natural period (T_{mod}) can then be quantified in terms of mass *M*, weight *W* and yield displacement Δ_y as follows,

$$K_{\text{mod}} = \frac{V_b}{\Delta_y} = \frac{A_h W}{\Delta_y} = \frac{A_h M g}{\Delta_y} \tag{6}$$

and

$$T_{\text{mod}} = 2\pi\sqrt{\frac{M}{K_{\text{mod}}}} = 2\pi\sqrt{\frac{\Delta_y M}{A_h M g}} = 2\pi\sqrt{\frac{\Delta_y}{A_h g}} \tag{7}$$

Corresponding to T_{mod}, modified Sa/g, denoted herein by $(Sa/g)_{mod}$ from spectral curve is determined again. After this, the ratios of $(Sa/g)_{mod}$ to *Sa/g* obtained initially and the T_{mod} to *T* are determined and studied. The first ratio can be regarded as the ratio of the modified strength demand to the strength provided and can be denoted by $r_{strength}$ while the second ratio is denoted by r_{period}, respectively.

$$r_{strength} = \frac{(S_a/g)_{\text{mod}}}{(S_a/g)} \tag{8}$$

and

$$r_{period} = \frac{T_{\text{mod}}}{T} \tag{9}$$

Further, the inelastic response of the systems with initial stiffness, period and strength and the same with modified stiffness and lateral period with same attributed strength (as this itself remains unaltered and causes change to stiffness) are obtained for comparison. The nonlinear equation of motion resulting from the used hysteresis behaviour for spring force is numerically solved in the time domain following step-by-step integration method, likewise the previous studies [1,18]. Newmark's β-γ scheme with constant average acceleration over each incremental time step is used for the purpose. Newmark's parameters $\gamma = 0.5$ and $\beta = 0.25$ are considered to achieve unconditional stability condition. For better accuracy, the present study performs iterations in each incremental time step with modified Newton-Raphson technique. The time step of integration is taken as less than $T/800$ second, where T is the lateral natural period of the system. This time step is found to be sufficiently small from sample convergence study.

From the inelastic analyses, Inelastic Displacement Demands considering yield displacement constant philosophy (Δ_{iydc}) and that considering traditional stiffness constant approach (Δ_{isc}) are determined. The ratio, $r_{idd} = \Delta_{iydc} / \Delta_{isc}$ is obtained and its variation is studied to see that how the actual displacement demand due to attributed strength is varying because of actual stiffness imparted due to strength as compared to the demand which is evaluated by the traditional approach with initially considered stiffness.

If Δ_{yydc} and Δ_{ysc} are the yield displacements due to the considerations of modified strength-dependent stiffness implying constant yield displacement (independent of R) and the traditional approach with constant stiffness, respectively, then, for R=1.0, $\Delta_{yydc} = \Delta_{ysc} = \Delta_y$ (say) and for $R > 1.0$, $\Delta_{yydc} = \Delta_y$ and the $\Delta_{ysc} = (\Delta_{yydc} / R) = (\Delta_y / R)$. The ratio of the ductility demands obtained by these two approaches ($r_{\square}$) can then be determined as,

$$r_{\mu} = \frac{\mu_{ydc}}{\mu_{sc}} = \frac{\Delta_{iydc} / \Delta_{yydc}}{\Delta_{isc} / \Delta_{ysc}} = \left(\frac{\Delta_{iydc}}{\Delta_{isc}}\right) \text{x} \left(\frac{\Delta_{ysc}}{\Delta_{yydc}}\right) = \left(\frac{\Delta_{iydc}}{\Delta_{isc}}\right)\frac{1}{R} = \frac{r_{idd}}{R} \tag{10}$$

where, μ_{ydc} and μ_{sc} are ductility demands corresponding to the new and conventional methods respectively. The variation of this quantity may also be indicative of difference between the actual ductility demand due to effective modified stiffness corresponding to attributed strength and the conventionally calculated ductility demand with initially considered stiffness.

3.5 Parametric Study

The fundamental natural period of a wide variety of structures including buildings, water tanks etc., are generally not more than 4.0sec. Thus, the initially chosen lateral period (T) is considered to vary over a wide range of practical interest. Prima facie, in the elastic range, such period range is considered to vary from 0.01 sec. to 4.0 sec. with a very close interval. However, while examining inelastic range response, the lowest fundamental lateral natural period is considered to be 0.1 sec. as the same appears to be realistic. Furthermore, a close scrutiny of the elastic range response presented later reveals that the traditional method is expected to yield conservative strength demand in general barring a few abnormally stiff systems, consideration of which does not have much of practical implication. Hence, in the inelastic range, systems with periods 0.1 sec., 0.4 sec., 1.0 sec., 1.5 sec., 2.0 sec., 2.5 sec., 3.0 sec., 3.5 sec. and 4.0 sec. are only studied with the exclusion of systems with intermediate period range, which is expected to be reflective of the overall trend. Four values of the response reduction factor (R), namely, 1, 2, 4 and 6 are considered both for frames with brick infill panels (with base dimension d = 10m which participates in the formulae of natural period for this type of buildings in Indian Seismic code [10]) and without brick infill panels. The horizontal seismic coefficient (A_h) varies as the factor ($ZI/2R$) varies in Indian Seismic Code [10]. Z, being the zone factor, is constant at a particular location. The ratio (I/R) of importance factor (I) and response reduction factor (R), is varied in such a way that the maximum and the minimum values for ($ZI/2R$) are attained, maintaining the condition that the ratio I/R does not exceed 1.0 as prescribed in Indian Seismic code [10]. Such a variation may lead to a wide range of variation in strength and subsequently in strength dependent stiffness. Accordingly, for each of the values of R = 1,2,4 and 6, the maximum and minimum values of the seismic coefficient A_h are considered to arrive at the stipulated maximum and minimum strength, as per Indian Seismic code [10].

3.6 Results and Discussion

The results are presented in two subsections depicting the comparisons of the strength demand obtained by two approaches in the first subsection while the same of inelastic demands (i.e., inelastic displacement and ductility demand) is presented in the next subsection.

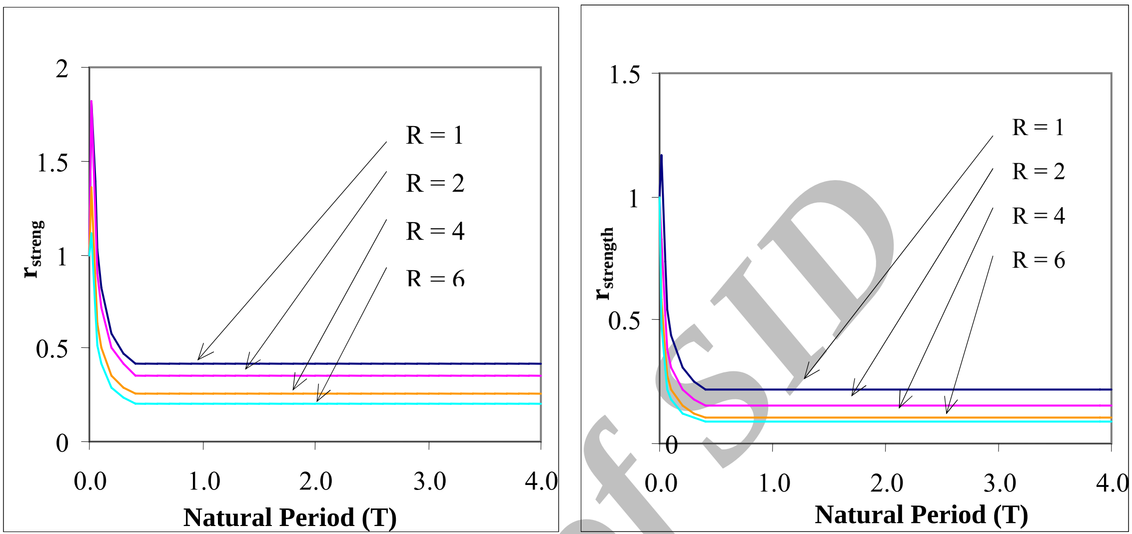
3.7 Strength demand

The comparison of strength demands is studied for buildings with brick infill and that without brick infill separately.

3.8 Buildings with Brick infill panels

As mentioned earlier since the natural period formula in Indian seismic code is dependent on base dimension, this dimension is considered to be 10 m. as this is a reasonable dimension for moderate size domestic buildings. However, the trends observed may be believed to be applicable for general class of such buildings.

Figure 2 presents the curves plotted for ratio, $r_{strength}$, against natural period (T). It may be noted that Z, I and R remain constants even if T changes. Thus, from the curves, it can be inferred that as long as the ratio $r_{strength} \leq 1.0$, $(Sa/g)_{mod}$ is always less than or equal to initial (Sa/g). This means that the modified strength demand of the structure does not exceed the initial strength provided on the basis of the traditional design approach based on the initially calculated period and the structure is not endangered. Whereas, when $r_{strength} > 1.0$, the modified strength demand becomes more than the initial strength provided and thus the structure becomes vulnerable to earthquake as per the existing design procedure.



Ratio for maximum value of *(ZI/2R)* Ratio for minimum value of *(ZI/2R)*

Figure 2a. Ratio of the modified strength demand to the strength provided ($r_{strength}$) in buildings with brick infill panels

When the maximum possible strength is attributed through consideration of maximum possible value of *(ZI/2R)* as per Indian Standard code, then following observation can be made. For R=1.0, structures having $T \leq 0.075$ sec exhibits $r_{strength} > 1.0$, implying an unsafe condition. On the other hand, systems with $T > 0.075$ have $r_{strength} < 1.0$, implying that the structure will be safe even due to the stiffness modification.

Similar observations can be made when the minimum possible strength is attributed through consideration of minimum possible value of *(ZI/2R)*. For R=1.0, structures having $T \leq 0.025$ sec exhibits $r_{strength} \geq 1.0$, implying an unsafe condition and systems with $T >$ 0.025 have $r_{strength} < 1.0$, implying that the structure will perform safely even due to the modified stiffness.

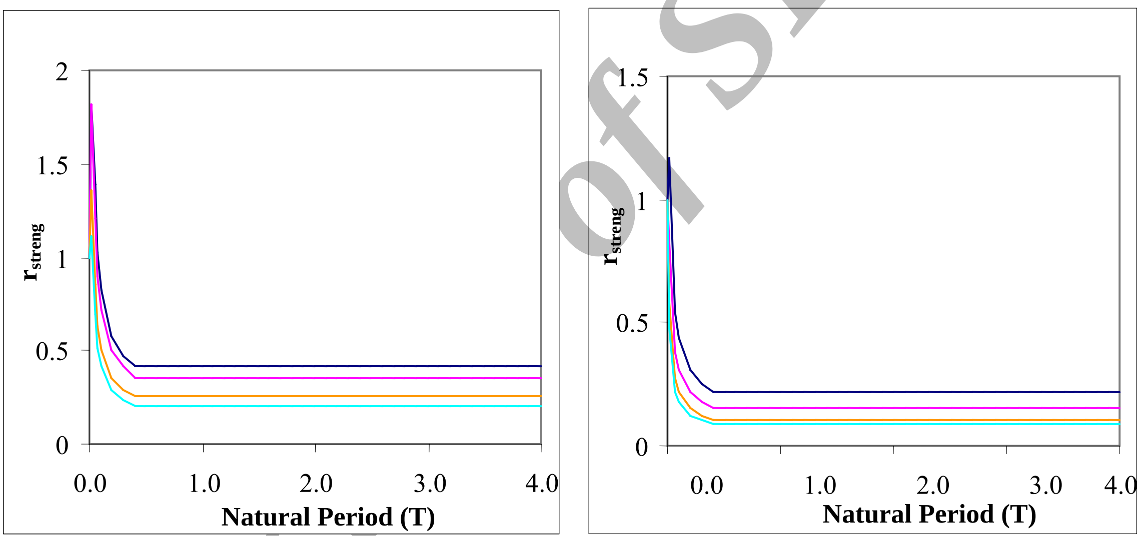
For other values of response reduction factors (R), namely 2, 4 and 6, the ratio $r_{strength}$ exhibits a value greater than 1 for a period range of only upto about 0.05sec, 0.025sec and 0.025sec, respectively. Thus, beyond the range of such unrealistically stiff periods, most of such systems seem not to be endangered because of period modification as far as the strength demand exhibited through elastic range response is considered.

The variation of changed lateral period due to change in strength dependent stiffness has been studied through plotting r_{period} as a function of the initially assessed period T. Such plots, though not presented for the sake of brevity, shows that the modified period becomes more than about twice the initially assessed period. Because of such flexibility attributed through strength dependent stiffness, as expected strength demand of the modified actual system reduces as observed from the variation of $r_{strength}$.

5.1.1 Inelastic demand

The inelastic response of the system shown in Figures 3-10 presents the variation of the ratio (r_{idd}) of inelastic displacement demand obtained from stiffness-modified actual system to that of initially considered system. Such response is further assessed in terms of the parameter r_{μ}, expressed as the ratio of the ductility demands obtained from strength dependent stiffness consideration and traditional approach.

Initially, the strength demand of such systems is obtained from the elastic analysis under the stipulated time history and is then divided by the appropriate response reduction factor. Hence, the codal spectrum does not have any participation in the process of the strength assignment while it becomes particular time-history dependent to exactly assign the appropriate strength demand from theoretical point of view.



Ratio for maximum value of *(ZI/2R)* Ratio for minimum value of *(ZI/2R)*

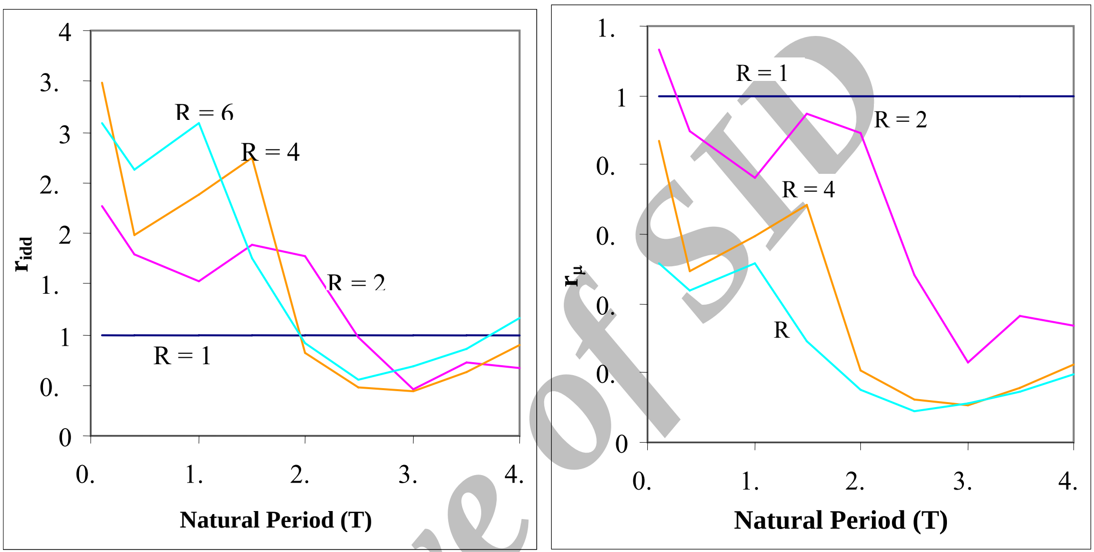
Figure 2(a) Ratio of the modified strength demand to the strength provided ($r_{strength}$) in buildings with brick infill panels

5.2.1 Response under El Centro earthquake

5.2.1.1 Elasto-plastic system

Figure 3(a) represents the variation of r_{idd} for elasto-plastic system as a function of initial natural period. It is seen that, for *R*=2, the displacement demand of the stiffness modified system exceeds the displacement demand of the initially considered system to the extent of 50% to 130%. For *R*=4 and 6, such increase has been observed over a period range of 0.1 to 1.5sec and the extents of increase are 100% to 250% and 75% to 200% respectively.

Figure 3(b) represents the variation of r_{μ} for elasto-plastic system corresponding to the variation of initial lateral natural period *T*. The figure shows that the ductility demand of the system with modified stiffness resulting from the strength design based on initially assessed period is about 15% higher than that of the system with initially assumed stiffness when the initial lateral period is upto about 0.2sec. For rest of the entire period range, the ductility demand of the systems with modified stiffness is lower than that exhibited by the initial ones. Thus, the existing approach of considering an initially assumed stiffness obtained on the basis of empirical period formula proposed in codes may result in an overestimation of ductility demands and hence, may be safe on this aspect. On the other hand, displacement demand may be underestimated by these systems at least over stiffer period range.



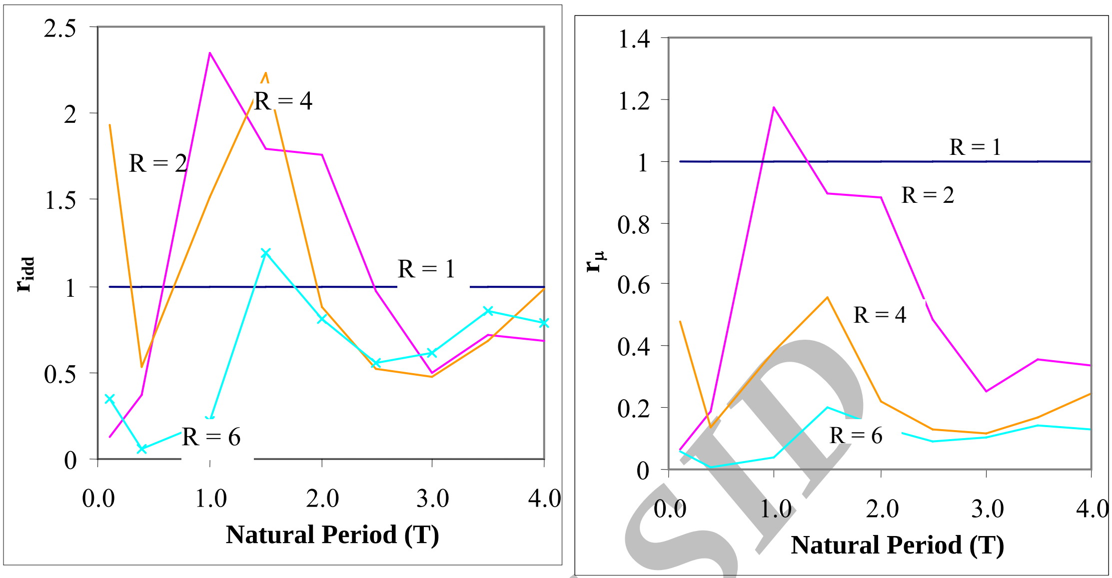
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 3. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with elasto-plastic hysteresis behaviour

5.2.1.2 Degrading systems

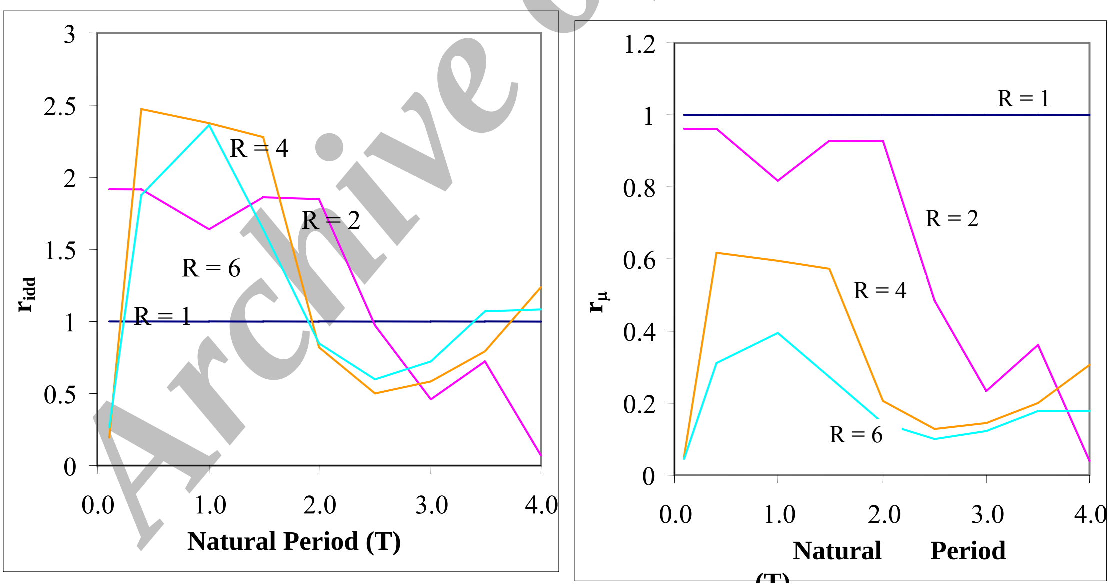
Figures 4(a) and 4(b) present the similar variation for the ratio of inelastic displacement demands (r_{idd}) and the ratio of inelastic ductility demands (r_{μ}), considering an only strength degrading hysteresis model. Likewise Figure 3, Figure 4 also shows that the inelastic displacement demand of the modified system may exceed that of the initially assumed system over various small domains of lateral period for various response reduction factors, generally over a range of upto 2 sec of lateral periods. On the other hand, such exceedance of ductility demand of stiffness modified system over that of the initially assumed systems are only marginal, being not more than about 20%.

Figures 5 and 6 present the variations of similar quantities considering only stiffness degrading and simultaneously strength and stiffness degrading hysteresis behaviours, respectively. Both the Figures exhibit similar trend as exhibited by Figures 3 and 4.



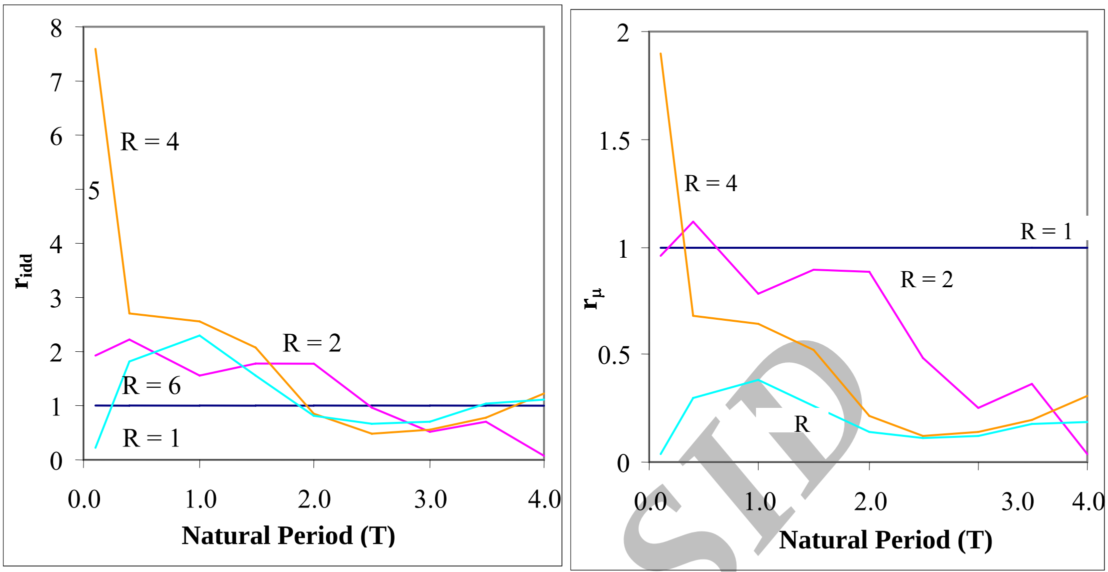
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 4. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with only strength degrading hysteresis behaviour



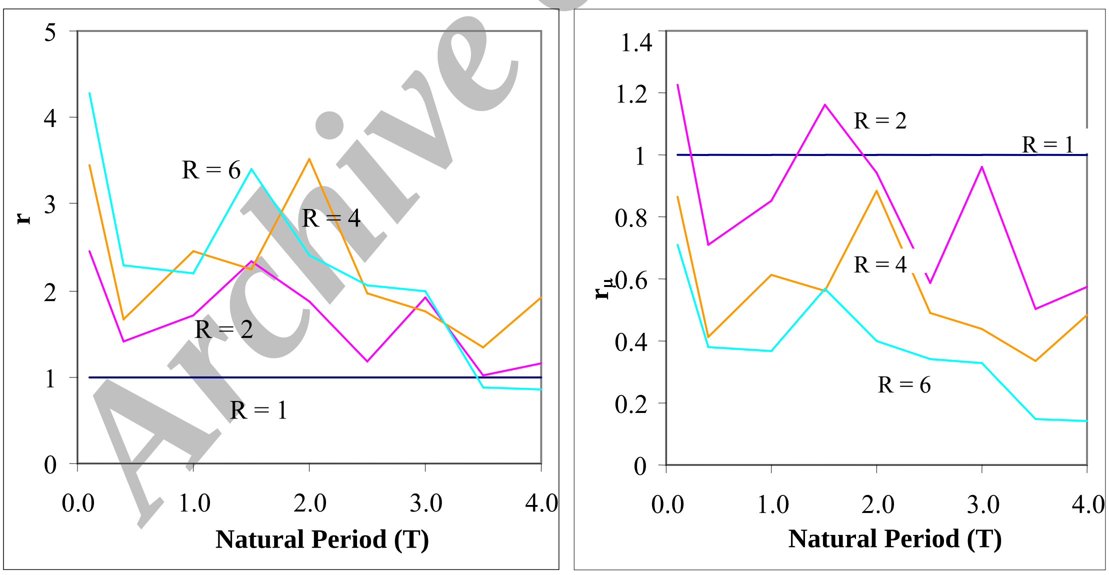
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 5. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with only stiffness deteriorating hysteresis behaviour



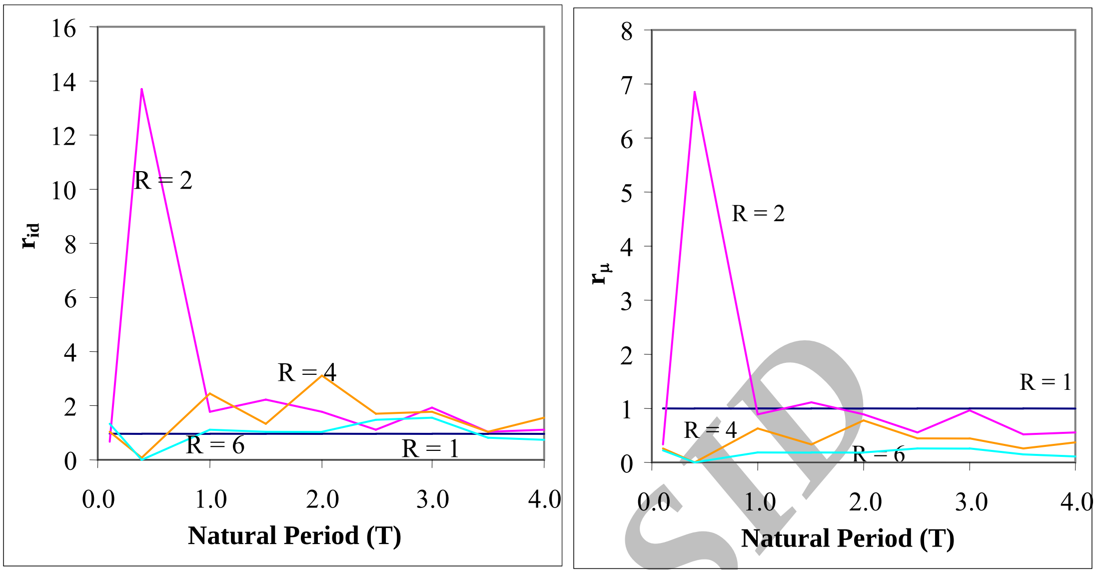
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 6. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with both stiffness and strength degrading hysteresis behaviour



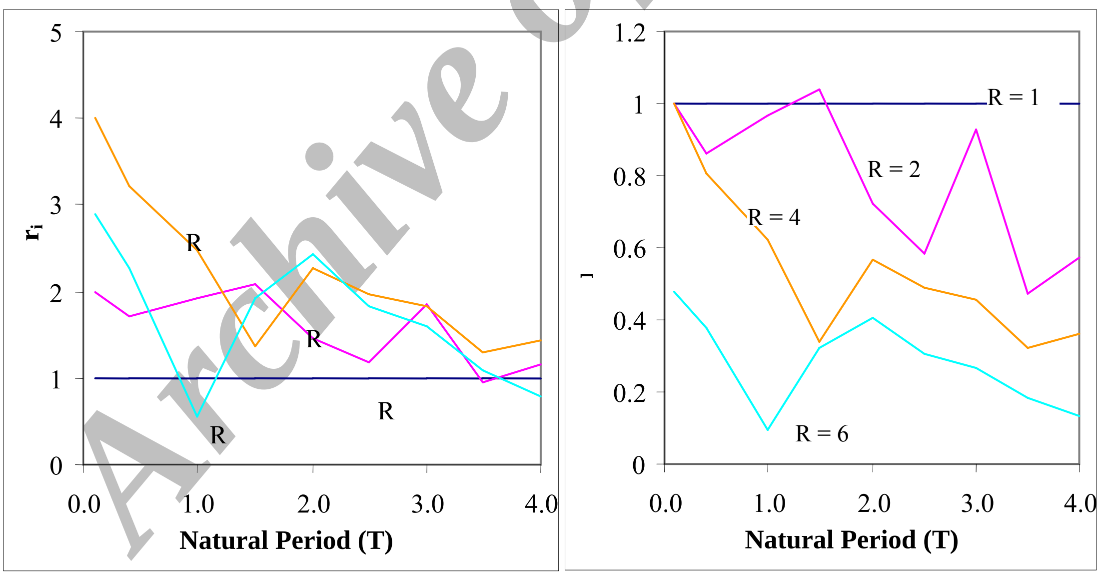
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 7. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with elasto-plastic hysteresis behaviour



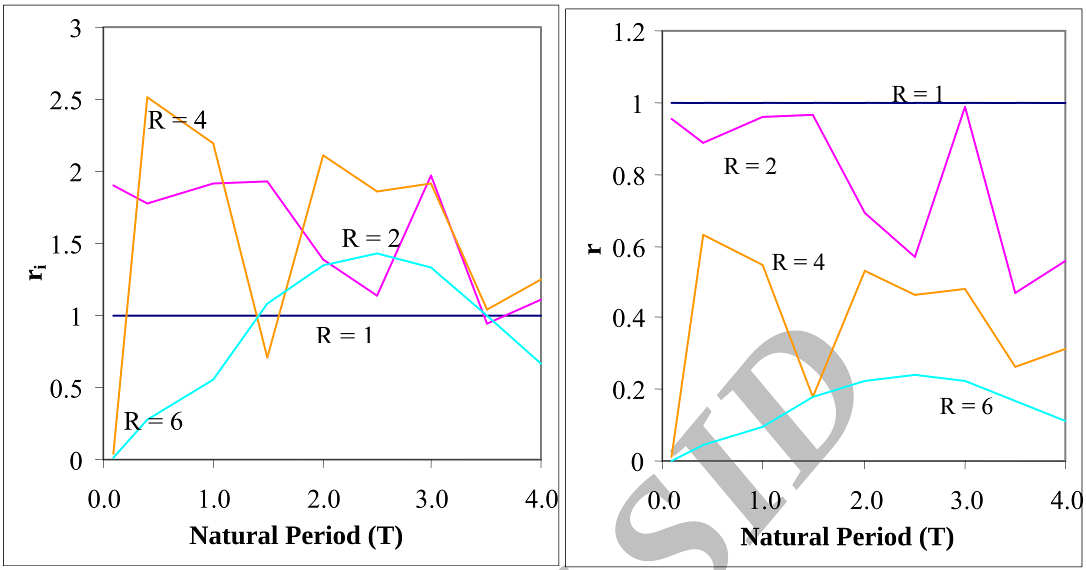
(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 8. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with only strength degrading hysteresis behaviour



(a) Ratio, r_{idd} , of displacement demands (b) Ratio, r_{μ} , of ductility demands

Figure 9. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with only stiffness deteriorating hysteresis behaviour



(a) Ratio, r_{idd}, of displacement demands (b) Ratio, r_{μ}, of ductility demands

Figure 10. Ratio of the displacement demands and ductility demands of initially assumed and stiffness modified system with both stiffness and strength degrading hysteresis behaviour

4. CONCLUSIONS

The limited study presented in the paper helps to judge the sanctity of the traditional seismic design methodology based on lateral period as prime input. The following broad conclusions can be made.

1. The strength demand calculated by structural systems based on their initially assumed period appears to be generally greater than the strength demand exhibited by the actual system with its stiffness and lateral period adjusted according to its strength provided based on such initial period, except abnormally stiff systems (period less than about 0.05 sec). Hence, such strength assessment based on traditional period based design approach seems to perform satisfactorily to ensure a safe elastic level response under stipulated moderate seismic shaking.
2. The inelastic displacement demand of the systems based on its initially assessed period is found to be lower than what is predicted due to its modified stiffness and period arrived at due to the strength assigned based on initially assessed period for many period ranges. Hence, the seismic design methodology based on such initially assessed stiffness seems not to be reliable for prediction of inelastic displacement demand. Knowing such demands helps to control the damage of non-structural elements.
3. However the ductility demand predicted by such initially assumed system seems to be higher than that predicted by modified actual systems, except in a few sporadic cases. In fact, it has been discussed that the yield displacement of such initially assumed system is lowered by dividing the yield displacement of systems with response reduction

factor, *R*, keeping the stiffness constant. Such modification in yield displacement magnifies the ductility demand of the initially assumed system as compared to their stiffness modified counterparts with constant yield displacements. This leads to a slightly upper bound prediction of ductility demand for the first category of systems, which is desirable from the design point of view. This suggests that inelastic displacements of the systems with initially assessed period may be multiplied by response reduction factors to yield a value comparable with that exhibited by the actual systems with modified stiffness but unchanged yield displacements. Furthermore, these observations in second and third conclusions seem to be generally valid irrespective of types of hysteresis behaviour exhibited by the structure in the inelastic range.

Traditional period based seismic design approach considers initially assessed lateral period as the primary input parameter. This approach considers stiffness to remain constant while reduces yield displacement proportionally with reduction in strength, i.e., increase in response reduction factor. However, the recent literature [5, 8, 9] shows that stiffness almost proportionally changes with strength while yield displacement remains unchanged.

The present study summarily shows that such ignorance of actual phenomenon does not hamper the safety in evaluation of strength and ductility demand generally, except for abnormally stiff structures, very rarely existing in practice. On the other hand, inelastic displacement demand obtained with this traditionally derived system should be multiplied with response reduction factor to obtain a safer estimate of the same. This explains why and how the traditional period based seismic design approach with a constant stiffness consideration which is widely popular because of its simplicity can still be relevant and can be used for safe performance based seismic design.

REFERENCES

1. Goel, R.K., and Chopra, A.K., Dual-level approach for seismic design of asymmetric-plan buildings, *Journal of Structural Engineering, ASCE*. No. 1, **120**(1994) 161-179.
2. International Association for Earthquake Engineering. Regulations for Seismic Design. A World List, IAEE, Tokyo 1996.
3. Paulay, T., Are existing seismic torsional provisions achieving the design aims? *Earthquake Spectra*. No. 2, 13(1997) 259-279.
4. Paulay, T., A simple seismic design strategy based on displacement and ductility compatibility, *Earthquake Engineering and Engineering Seismology*. **1**(1999), 51-67.
5. Paulay, T.: 2000, A simple displacement compatibility based seismic design strategy for reinforced concrete buildings, Proceedings of the 12th World Conference on Earthquake Engineering, Auckland. No. 0062, 1-8.
6. Paulay, T., A re-definition of the stiffness of reinforced concrete elements and its implications in seismic design, *Structural Engineering International*. **1**(2001) 36-41.
7. Smith, R.S.H. and Tso, W.K., Inconsistency of force-based design procedure, *Journal of Engineering Seismology. JSEE*. No. 1, **4**(2002) 47-54.
8. Priestley, M.J.N.: 2000, Performance based seismic design, 12 World Conference of Earthquake Engineering, Vancouver, Canada. paper No 2831.

9. Aschheim, M., Seismic Design based on the yield displacement, Journal of *Earthquake Spectra*, Earthquake Engineering Research Institute. No. 4, **18** (2002)581-600.
10. IS-1893(Part 1): Indian Standard Criteria for Earthquake Resistant Design of Structures (Part 1- General Provisions and Buildings), New Delhi, India: Bureau of Indian Standards, 2002.
11. Dutta, S.C. and Das, P.K., Inelastic seismic response of code-designed reinforced concrete asymmetric buildings with strength degradation, *Engineering Structures*. **24**(2002) 1295-1314.
12. Takeda, T., Sozen, M.A. and Nielsen, N.N.: 1970, Reinforced concrete response to simulated earthquake, *Journal of Structural Engineering, ASCE*. **96**(ST12), 2557-2573.
13. Chandler, A.M., Duan X.N. and Rutenberg, A., Seismic torsional response: assumptions, controversies and research progress, *European Earthquake Engineering*. No. 1, **10**(1996) 37-50.
14. IS-1893 (Part 1): 1984, Indian Standard Criteria for Earthquake Resistant Design of Structures, New Delhi, India: Bureau of Indian Standards.
15. Housner, G.W.: 1959, Behaviour of structures during earthquakes, Proc. *Engineering Mechanics Division, ASCE*. **85**(EM-4) 109-129.
16. Khan, M.R., Improved method of generation of artificial time-histories, rich in all frequencies, *Earthquake Engineering and Structural Dynamics*. No. 8, **15**(1987) 985-92.
17. Dutta, S.C., Effect of strength deterioration on inelastic seismic torsional behaviour of asymmetric RC buildings, Building and Environment. No. 10, **36**(2001) 1109-1118.
18. Goel, R.K., Seismic response of asymmetric systems: energy-based approach, *Journal of Structural Engineering, ASCE*. No. 11, **123**(1997) 1444-1453.