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Seismic safety of structures: Influence of soil-flexibility, asymmetry and ground motion characteristics

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Abstract

Structures may experience degradation in strength in the event of strong seismic shaking. A rational estimation of the reserve strength of the structures is often desired in the process of retrofitting or strengthening the same. To achieve this end, the present paper confirms the suitability of an existing hysteresis model in reproducing experimental load–displacement characteristics for reinforced concrete (R/C) structural members. Attempt has also been made for rational and realistic estimation of the degradation parameter required for the model in absence of any case-specific calibrated value. Subsequently, post-earthquake behaviour of the low-rise symmetric structures is assessed with and without accounting for the effect of soil–structure interaction. Such response for low-rise multistorey systems with regular asymmetry has also been investigated in the sample form. To develop insight into the behaviour of asymmetric (uni-directional and bi-directional) systems, detailed investigation has been made on idealized single-storey asymmetric systems under simulated and real ground motions with different phase difference or time lag variation. This suggests a serious implication of occurrence of peaks of the ground motions on the seismic performance of bi-directionally eccentric structures and indicates a relatively higher torsional vulnerability of bi-directionally eccentric system compared to equivalent uni-directional counterpart. The results along with the endeavour toward measuring the ductility capacity for R/C structural members based on the systematic observation and interpretation of the available experimental results, made in the paper, may prove useful in evaluating the seismic safety of low-rise R/C structures.

1. Introduction

Strategy relevant to earthquake resistant design of structures allows damage of the structures without collapse during strong earthquake. However, such structures are expected to experience little or no damage in moderate or minor ground shaking. This philosophy of controlling damage to acceptable levels points out the immediate need of a reliable estimate of the post-earthquake response scenario of structures for an

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appropriate retrofitting measure. In this context, a realistic assessment of the structural capacity after damage (residual strength), in terms of the traditionally used design parameters, may be of ample help.

At this backdrop, the present paper attempts to identify a simple yet reasonable existing hysteresis model [1], a prerequisite of such damage assessment and verifies the applicability of the same through case studies. Subsequently, the same is utilized to evaluate the degraded load-carrying capacity of low-rise structures, symmetric in configuration. Importance of soil–structure interaction (SSI) to assess the seismic response of short-period systems has been highlighted in the literature (e.g. Refs. [2,3]). Hence, SSI is also incorporated in assessing the deteriorated strength of the buildings considered in the present study. Outcome of this study may also offer insight to assess as to what extent a structure may experience damage due to some anticipated earthquake highlighting on the level of safety of the structure. Reduced load-carrying capacity of the structural members may also be gauged offering valuable input in the process of retrofitting.

However, in practice, buildings are seldom found to be symmetric in configuration and as a result seismic response may largely be amplified due to lateral-torsional coupling. Such possibility of intensified seismic vulnerability of asymmetric structures has been exemplified during several earthquakes in the past [4–7] and also recognized in the literature [8]. Buildings located at the side of the road may often have large openings in the street facing sides (flexible side) to fulfil various commercial requirements in order to accommodate shops, garages, etc., while the other sides are generally filled with masonry (stiff side). This asymmetric distribution of the load-resisting elements leads to the generation of stiffness eccentricity, which is defined as distance between centre of mass (CM) and centre of stiffness (CS) in one direction. On the other hand, for buildings located typically at the street corners, similar eccentricity may arise in two mutually orthogonal directions. The first category of systems, in the present paper, is referred to as uni-directionally asymmetric system, while the second as bi-directionally asymmetric one. In such systems, the response of flexible and stiff side elements is expected to significantly differ and the collapse may be triggered due to the failure of either of the two sides. Thus the deteriorated capacity of such structures as a whole after a seismic event may not reflect the subsequent performance of the system and the evaluation of the capacity of the individual element seems imperative.

At this background, the present investigation attempts to address and resolve a critical aspect relevant to the seismic behaviour of bi-directionally asymmetric systems in which residual strength of such structural elements may be dependent. Response of such systems under bi-directional ground motions may largely be regulated by the relative sense of eccentricities with respect to the ground motion as decided by the quadrant-wise location of the CS. This may attribute to the variation in response of similar bi-directionally asymmetric systems situated at two opposite sides of a street corner. This issue has been comprehensively investigated in a recent study [9].

Having resolved the impact of this issue through exhaustive response study under harmonic pulses and subsequently due to synthetic and real ground motions both in the elastic and inelastic range, residual capacity of the asymmetric system (both uni-directionally and bi-directionally) is evaluated in the sample form to have a preliminary idea about the influence of torsion in deteriorating the overall performance of R/C structures. The analysis, though considers multistorey asymmetric system, excludes the consideration of base-flexibility in the limited scope of the single paper. Further, these results cannot recognize the possibility of local failure of the load-resisting structural elements and hence further investigation on the same is urged.

2. Idealization of member

Experimentally observed load–displacement behaviour of R/C member under cyclic loading as available in the literature (e.g. Refs. [10–19]) clearly exhibits three basic characteristics, viz., stiffness degradation, strength deterioration and pinching. A critical examination of the available load–displacement history for R/C member under cyclic loading reveals the sensitivity of the load–displacement response on various factors, such as grade of concrete, reinforcement percentage both in the transverse and longitudinal directions, reinforcement detailing, etc. However, development of a numerical scheme accounting for all identified factors, though physically appealing, may be too tedious to use in practice. At bottom, the objective of idealizing the hysteresis behaviour is to predict the overall seismic performance of structures. Hence, a computationally intensive scheme accounting for all three basic features of R/C member under cyclic loading as was developed earlier

[20] and included in a computer program IDARC [20,21], may be of limited use. In this context, the present investigation seeks for a mathematically simple and physically close numerical scheme to represent the hysteresis behaviour of R/C members under cyclic loading.

2.1. Details of hysteresis model chosen

A simplified hysteresis model proposed elsewhere [1] to represent the behaviour of R/C member under cyclic loading is considered in the present investigation. The stiffness and strength deterioration characteristics are incorporated with a bilinear backbone curve in this model. The model involves three input parameters, viz., initial yield strength (F_y), initial loading stiffness (k) and strength deterioration factor (δ), which are assumed to be constant during the entire load history. The model computes the degraded strength by deteriorating the yield strength by a fixed fraction equal to average strength deterioration factor. The stiffness of the elastic loading portion of the load–displacement curve is obtained using the principle similar to Takeda's model [22]. 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 and pinching is not incorporated in explicit sense. However, the model is found to reproduce the hysteresis curves with strength and stiffness degradation as well as pinching with reasonable closeness as discussed later. With these postulations, hysteresis model is developed to simulate the behaviour of R/C structural members under cyclic loading. Further details of the same are available elsewhere [1].

2.2. Calibration of input parameters

Performance of any computational scheme to be used for practical purpose not only depends on the sound conceptual understanding of the behaviour and modeling the corresponding mechanics in minute details, but to a large extent, on the precision of the input parameters. The hysteresis model considered in the present study needs three input parameters, of which strength deterioration factor (δ) seems difficult to assess. However, such factor δ, is observed to be more sensitive to the longitudinal and transverse reinforcement [23]. In this context, the present study makes a limited effort to investigate the dependence of such deterioration factor on various longitudinal and transverse reinforcement quantities. To achieve this end, a number of load–displacement curves under cyclic loading and the reinforcement details of the test specimens are collected from the literature (e.g. Refs. [10–19]) and carefully examined. Average strength deterioration δ per yield is calculated by dividing the total drop in strength by the number of yielding experienced by the reinforced concrete members during the vibration under reversible type loading. From a careful examination of δ so obtained for member of various concrete grades and amount of steel reinforcement, it seems that primarily the amount and type of reinforcement detailing of R/C structural member regulate strength degradation under cyclic loading. In this context, pending the availability of adequate experimental results expounding the effect of concrete grade explicitly, degradation parameters are correlated with the reinforcement only. Therefore, average drop in strength δ per yield excursion is plotted as a function of the percentage of transverse reinforcement (p_t) for various ranges of the percentage of longitudinal reinforcement (p_l). These ranges of longitudinal reinforcement are, namely, 0.0–0.5%, 1.5–2.0%, 2.0–2.5%, and 3.0–3.5%, respectively. However, on the basis of about hundred load–displacement curves only, it appears very difficult to conceive any trend. Further, some experimental data, when plotted graphically, are found to be scattered. Hence, with the limited data at hand, an upper bound envelope curve as shown in Fig. 1 is considered to have conservative estimate for δ. The envelope curve is found to be decreasing with increase in transverse reinforcement in most of the cases. This clearly indicates that the maximum possible deterioration per yielding decreases with increase in confining reinforcement. It is an observation supported by proposition in many literatures (e.g. Ref. [23]). However, a statistical analysis based on series of tests of R/C members with various reinforcement percentages and detailing are needed to be conducted to arrive at a more definitive estimate of the same.

Values of δ obtained from these curves are adopted in the present paper while investigating the performance of the selected model to reproduce the specific experimental load–displacement behaviour of R/C member under cyclic loading. On the other hand, such quantity is observed to vary in the range of about 2–9%

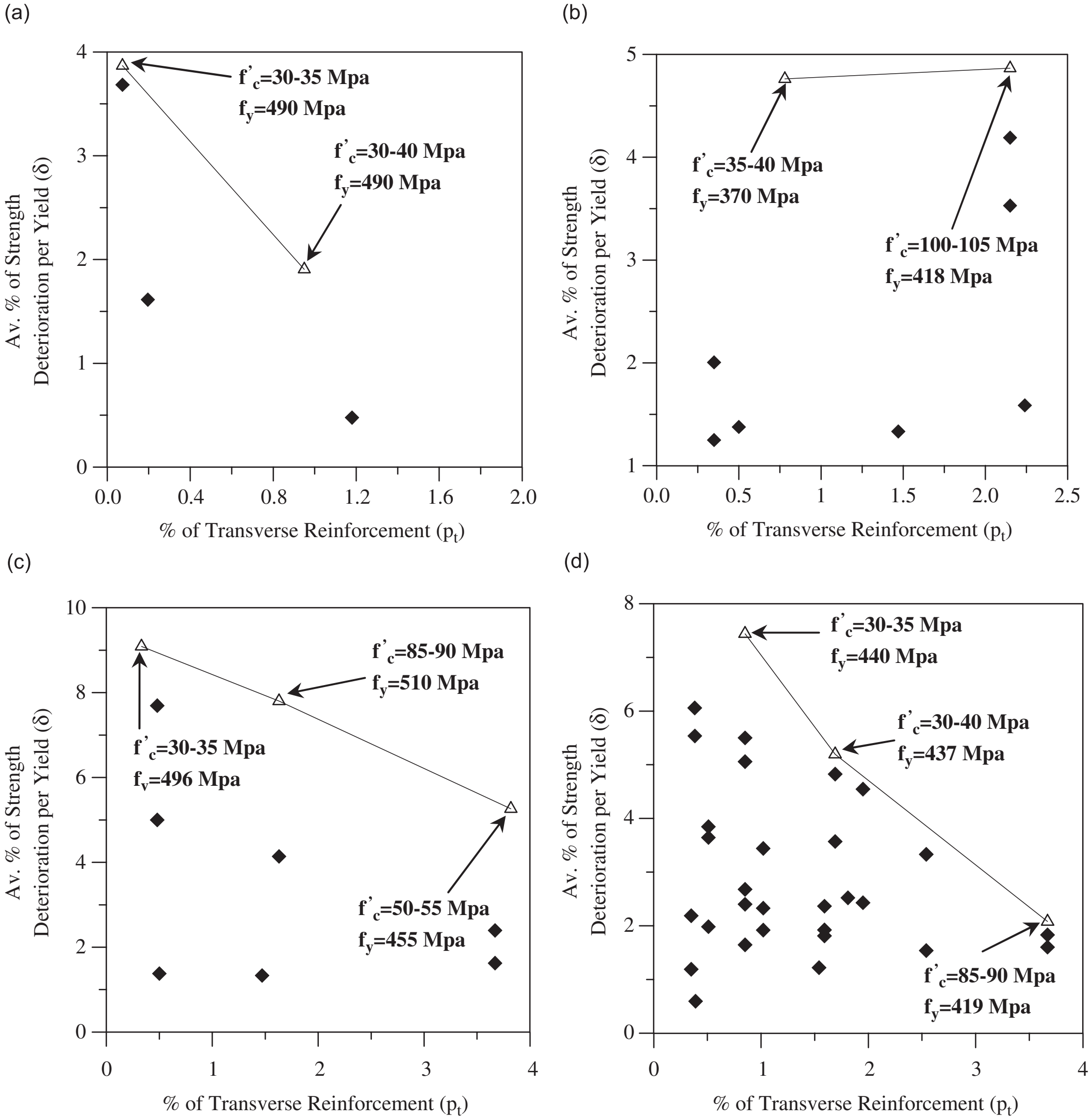


Fig. 1. Variation of strength deterioration (δ) with percentage of transverse reinforcement (p_t) for percentage range of longitudinal reinforcement (p_1): (a) 0.0–0.5%, (b) 1.5–2.0%, (c) 2.0–2.5% and (d) 3.0–3.5%.

and hence an intermediate value of 5% is assumed in assessing the safety level of R/C structure in the rest of the study.

2.3. Performance of hysteresis model

Satisfactory performance of the model to reproduce a number of experimentally observed load–displacement curves of R/C members with reasonable accuracy utilizing this scheme is already demonstrated earlier in the literature [1]. However, those experimental curves are not found to have considerable pinching. Hence, in the present paper, the applicability of the hysteresis model is further judged through the prediction of the load–displacement behaviour of R/C member exhibiting reasonable pinching as reported in earlier studies. Using the upper-bound envelope (refer to Fig. 1) δ is chosen as 8% for the specimen used by Ehasani and Wight [24], while the same is considered as 1.3%, 4.8% and 4.8% for specimens IIB, IIC and IID, respectively chosen from another literature [25]. All these curves exhibit considerable pinching.

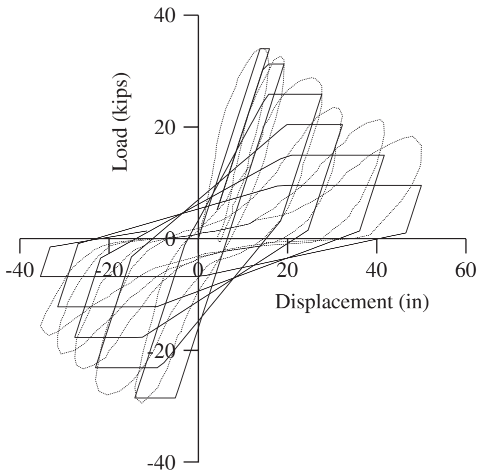


Fig. 2. Computationally reproduced hysteresis curves by model used, from experimental curves presented in literature [24].

Computationally reproduced curves for various samples are presented in Figs. 2 and 3(a–c). Experimentally observed curves are also superposed with dotted line on the corresponding curves obtained computationally. The figures reveal a close resemblance between the experimental and computational hysteresis curves. The proximity of the total hysteretic energy dissipated through the whole displacement history obtained from the computational model to the energy actually dissipated in experiment is another important quantitative index of accuracy of the hysteresis rule used. Hence, such quantity is also computed on the basis of the load–displacement curves obtained through the experiment and the computational scheme. Energy dissipation by a purely elasto-plastic model, by model incorporating only stiffness degradation and by only strength deterioration is also computed for the sake of comparison. Such quantity together with the deviation of the computationally estimated energy dissipation utilizing each model from that of the experimental one is furnished in Table 1.

A maximum deviation even in the order of 250% in such quantity is observed due to elasto-plastic model and the same drastically reduces to the tune of about 140% due to the incorporation of only stiffness degradation or only strength degradation. However, such error reduces to the order of about only 30% due to the simultaneous inclusion of stiffness and strength degradation as is exhibited through the response observed using present model. Due to such an improved accuracy, the present study uses this model in analyzing the inelastic range behaviour of reinforced concrete structures.

3. System idealization

3.1. Idealization of structure

Buildings have been idealized as rigid diaphragm model with three degrees of freedom at each floor level, two translations in two mutually orthogonal horizontal directions and one in-plane rotation. Masses are assumed to be lumped at the floor levels. In fact, owing to the considerable strength attributed to beam due to floor slabs, such rigid diaphragm idealization is believed to be adequate for the analysis of inelastic seismic behaviour of buildings [26,27]. Vertical distribution of stiffness is assumed uniform and hence the total lateral storey stiffness is considered to be same for each storey. Thus the total storey stiffness can be evaluated from standard eigenvalue problem knowing the lateral period and mass matrix of the system. Generally, in the residential or office buildings, lateral load-resisting structural members are found to be uniformly distributed over its plan. To represent such plan-wise distribution of the load-resisting elements (say, columns); in the present investigation, the structure is modelled to consist of six such elements, three in each orthogonal direction. The locations of the outer edge elements and subsequently the intermediate distance D may vary depending on the various characterizing parameters unlike several previous studies. Fifty per cent of the total

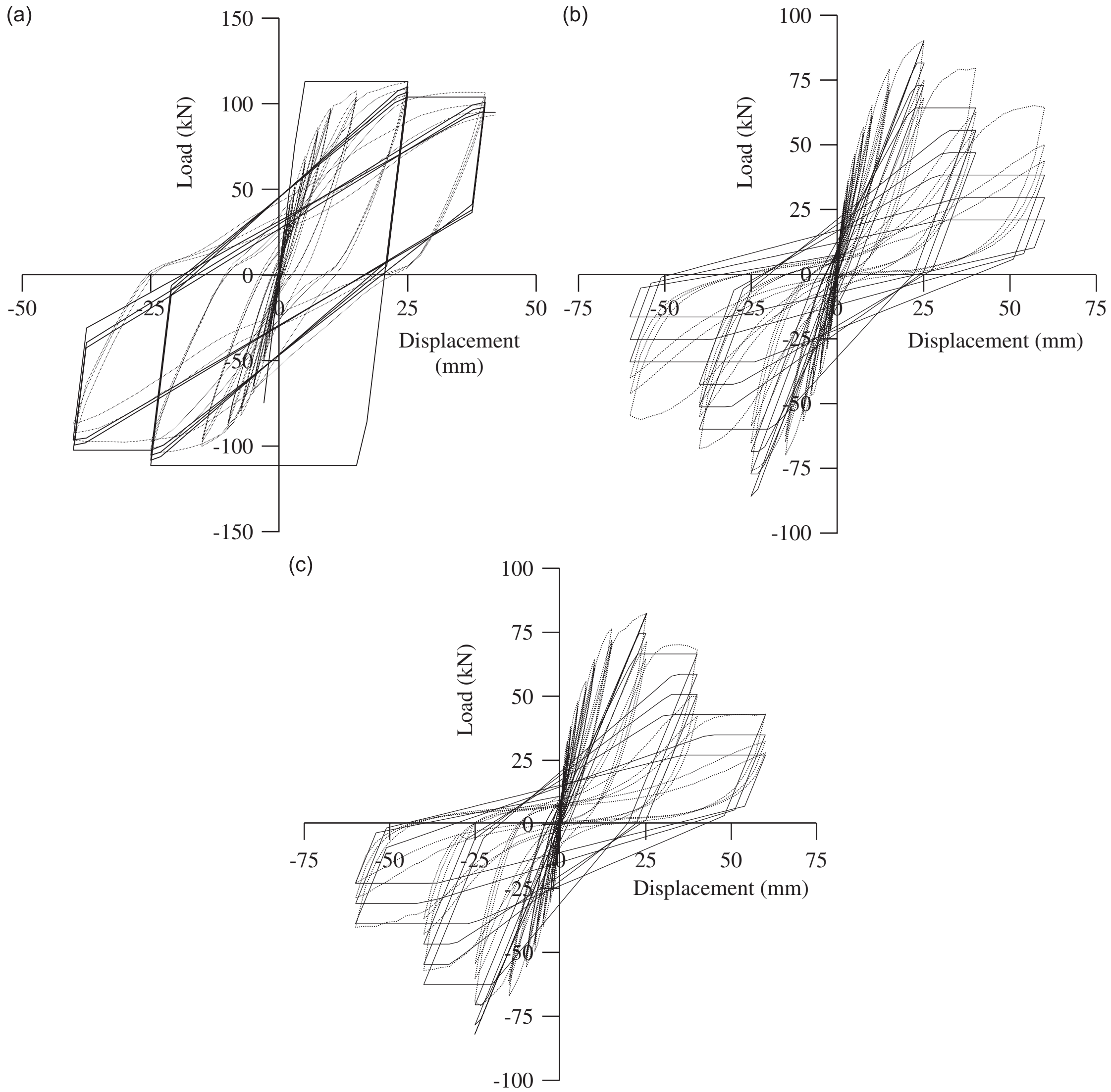


Fig. 3. Computationally reproduced hysteresis curves by model used, from experimental curves presented in literature [25]: (a) specimen II-B, (b) specimen II-C and (c) specimen II-D.

lateral stiffness is attributed to the middle element and the rest 50% is equally distributed between two edge elements.

Behaviour of asymmetric system is examined through equivalent single storey model likewise many other studies (e.g. Refs. [1,9,28–31]). This is expected to offer useful insight into the behaviour with relative ease. However, the limited number of case studies on multistoried asymmetric buildings is also presented for the sake of completeness. The stipulated amount of eccentricity in each storey level is introduced by increasing the stiffness of the lateral load-resisting element of one edge by a calculated amount and decreasing the same of the opposite edge element by the equal amount. This does not cause any change in the overall lateral and torsional stiffness of the idealized system. The lateral load-resisting edge elements, which have lesser stiffnesses, are designated as flexible elements and the opposite edge elements that have greater stiffnesses are designated as stiff elements. The asymmetric systems with eccentricities along two principal axes, as shown in Fig. 4b, are primarily considered in the present study. Systems with eccentricity in one principal direction (Fig. 4c), referred to as uni-directionally asymmetric systems or mono-symmetric systems due to their

Table 1
Comparison of hysteretic energy dissipation obtained through experimental observations and different computational models

Source of experimental curves	Model type	Energy value (kN mm)	Percentage deviation from experimental values
Ehsani and Wight, 1985 [24]	Experimental	3193.802	–
	Elasto-plastic	10516.43	229.276
	Stiffness degrading	5787.309	81.204
	Strength deterioration	6147.194	92.473
	Present model	4205.68	31.7
Jain and Murty, December 1999 [25]			
II-B	Experimental	30614.20	–
	Elasto-plastic	107620.50	251.538
	Stiffness degrading	45403.26	48.308
	Strength deterioration	74066.73	141.936
	Present model	33804.00	10.4
II-C	Experimental	16070.50	–
	Elasto-plastic	45434.46	182.72
	Stiffness degrading	24958.25	55.305
	Strength deterioration	39793.24	147.62
	Present model	19985.8	24.36
II-D	Experimental	14987.63	–
	Elasto-plastic	48044.98	220.564
	Stiffness degrading	26395.09	76.113
	Strength deterioration	34011.25	126.93
	Present model	20061.89	33.86

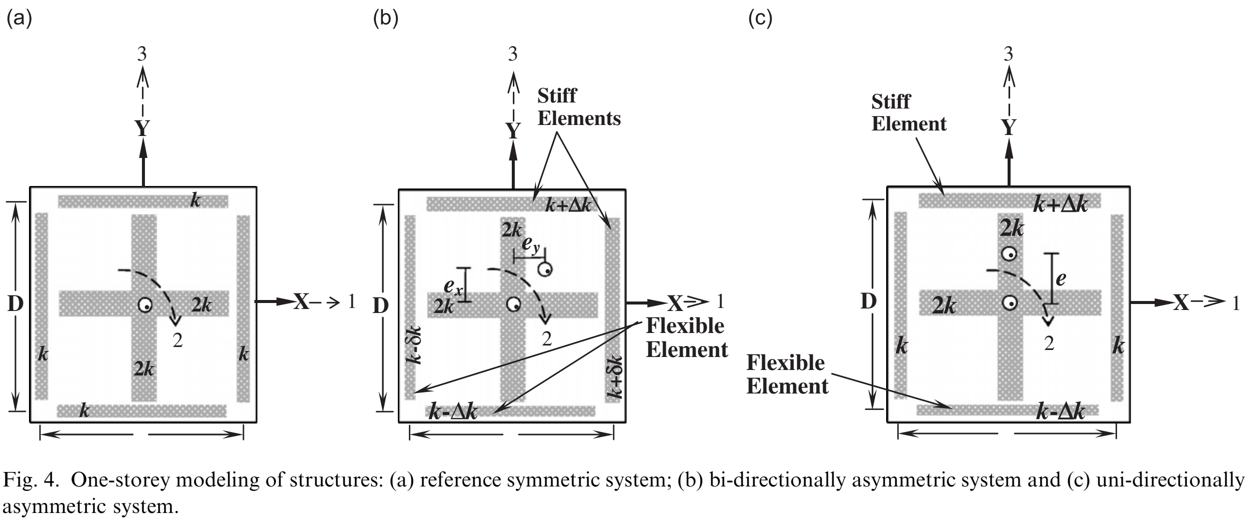


Fig. 4. One-storey modeling of structures: (a) reference symmetric system; (b) bi-directionally asymmetric system and (c) uni-directionally asymmetric system.

symmetry in one direction, are also considered in the present study for the sake of comparison. e_x and e_y denote the eccentricities between CM and CS, with respect to x and y axes, respectively, as shown in Fig. 4. Similar type of idealized structural systems was also considered in a few recent studies (e.g. Refs. [9,30]). Multistoried asymmetric structures, in the present paper, are assumed to have the CM of various stories to lie on one vertical axis passing through the geometric centroid of the floor decks, which may often be the case with buildings in practice except the ones with setbacks [26]. Location of the CS is also considered to lie along a particular line separated by a distance equal to the eccentricities from the CM axis.

Table 2
Details of soil parameters chosen from literature [41] and used elsewhere [37]

Type of clay	N value	C (kN/m^2)	ϕ (deg.)	γ_{sat} (kN/m^3)	C_c	e_0
Soft	3	18.5	0.0	17.0	0.189	0.90
Medium	6	36.8	0.0	18.5	0.135	0.72
Stiff	12	73.5	0.0	19.4	0.12	0.67

N. B. N, C, ϕ, γ_{sat}, C_c and e_0 denote N value obtained from SPT, cohesion value, internal friction angle, density in the saturated condition, compression index and initial void ratio of soil, respectively.

Table 3
Expressions for stiffnesses of equivalent springs along various degrees of freedom as available in the literature [32] and used elsewhere [37]

Degrees of freedom	Stiffness of equivalent soil spring
Vertical	$[2GL/(1-v)](0.73+1.54\chi^{0.75})$ with $\chi = A_b/4L^2$
Horizontal (lateral direction)	$[2GL/(2-v)](2+2.50\chi^{0.85})$ with $\chi = A_b/4L^2$
Horizontal (longitudinal direction)	$[2GL/(2-v)](2+2.50\chi^{0.85})$–$[0.2/(0.75-v)]GL[1-(B/L)]$
Rocking (about the longitudinal)	$[G/(1-v)]I_{bx}{}^{0.75}(L/B)^{0.25}[2.4+0.5(B/L)]$
Rocking (about the lateral)	$[3G/(1-v)]\ I_{by}{}^{0.75}\ (L/B)^{0.15}$
Torsion	$3.5G\,I_{bz}{}^{0.75}(B/L)^{0.4}(I_{bz}/B^4)^{0.2}$

N. B. A_b—AREA of the foundation considered; B and L—half width and half length of a rectangular foundation, respectively; I_{bx}, I_{by}, and I_{bz}—moment of inertia of the foundation area with respect to longitudinal, lateral and vertical axes, respectively.

3.2. Idealization of soil

The effect of SSI to enhance the flexibility of the system and reduce the vibration energy absorption by the structural system may be modelled by incorporating such effect through equivalent soil springs representing the action of the soil medium. Stiffness of such equivalent soil springs may be computed following the explicit expression furnished in the literature [32] after detailed computational and experimental studies [33,34]. Similar expressions have also been recommended in the recent version of many codes (e.g. Refs. [2,35]). Traditionally, stiffness of such equivalent springs representing the soil medium is expressed as frequency dependent to conveniently account for the variation of the inertia force under seismic loading. However, in the normal period range of the low-rise structures, influence of such frequency dependence, particularly in the context of seismic response analysis, may not be significant [2,36]. In fact, a detailed analysis on the impact of such frequency dependent soil impedances on the seismic response of low-rise buildings reveals that such response is marginally sensitive to such frequency dependent soil flexibility [37,38]. Thus, the effect of such frequency dependence has not been considered in the present investigation relevant to low-rise systems.

The reasonable estimation of the stiffness of the equivalent soil springs needs appropriate evaluation of the foundation dimension. These have been arrived at on the basis of the allowable bearing capacity determined as per the recommendation of the relevant standards [39,40]. The same has been computed assuming depth of foundation as 1.5 m at full submergence condition based on the soil-parameters mentioned in Tables 2 as suggested in the literature [41] and used in some other recent studies [37,38]. Expressions for assessing stiffness of equivalent soil springs are reproduced in Table 3 for further convenience. Mass of the foundation so designed has also been properly incorporated in the analysis through the consideration of consistent mass matrix.

4. Ground motions

To achieve insight into the behaviour of the asymmetric system, harmonic pulses of various periods are applied simultaneously along two principal axes of the idealized structural systems, with various phase differences. The preliminary understanding of the system response is convenient under harmonic ground motion, as the effects of various frequencies are not superimposed. Moreover, absence of the sporadic nature of the seismic ground motion helps to have a better insight into the physical behaviour of the system.

These ground motions also have similarity with the nature of the near-fault seismic ground motion. Hence, these types of ground motions are previously employed in a number of studies including those of seismic behaviour of asymmetric structural systems (e.g. Refs. [42,43]). These harmonic single frequency ground motions have been characterized by frequency ratio β. Frequency ratio can be expressed as $\beta = \omega/\omega_l$, where ω and ω_l are frequency of the harmonic ground motion and the uncoupled lateral natural frequency of the systems, respectively. Frequency ratio β of the single frequency harmonic motion is varied over a feasible range of 0.5–1.5 likewise some other studies (e.g. Ref. [9]) with an interval of 0.25. For the sake of convenience, uncoupled lateral natural period of idealized system in lateral vibration along two principal axes is maintained as 1.0 s, during variation of frequency of the harmonic pulse. The ground motions $\ddot{u}_{gx}(t)$ and $\ddot{u}_{gy}(t)$ can be expressed as $\ddot{u}_{gx}(t) = a_g \sin(\omega t)$ and $\ddot{u}_{gy}(t) = a_g \sin(\omega t + \theta)$ such that the phase difference between these two orthogonal components of ground motions becomes θ. It is to be noted that $\ddot{u}_{gy}(t)$ changes its sign if $\theta = \pi$. Thus, a variation of θ ranging from $0 < \theta < \pi$ takes care of the effect of phase difference and implicitly accounts for the effect of time difference of peaks in two mutually orthogonal components of ground motion. A number of such combinations of harmonic pulses with peak ground acceleration of $0.1g$ are considered in the analysis where g refers to the acceleration due to gravity. However, the trend indicating response corresponding to $\theta = 0$ and $\pi/2$ is only included in the present paper.

Moreover, two uncorrelated acceleration histories, consistent with design spectrum of Indian Standard Code of Practice for Earthquake Resistant Design of Structures [44], generated following the well accepted literature [45], are also considered in the present investigation. Time-acceleration histories of these two synthetic seismic excitations (viz., ACCN1and ACCN2) shown in Fig. 5 clearly indicate that the instant of occurrence of peaks of the same is different. The system behaviour is studied under two possible combinations of such spectrum consistent earthquake histories. The first combination contains ACCN1 and ACCN1 applied along both the principal directions. The response obtained under such a combination is the response due to simultaneous occurrence of peaks along both the principal directions. Such a combination may be little unrealistic. However, this is expected to yield response on the conservative side and also be considered in practice to account for the effect of the close occurrence of peaks of the orthogonal components of the ground motions. On the other hand, the response obtained due to a combination of ground motions ACCN1 and ACCN2 clearly indicates the response, which is likely to occur when peaks in two orthogonal histories are sufficiently away from each other. The distinction in response describes the effect of time difference of occurrence of peaks as the peak ground acceleration and the frequency contents are closely similar for both the acceleration time-histories.

Besides these, seismic behaviour of bi-directionally asymmetric structural system is also explored using two pairs of orthogonal acceleration histories recorded during 1994 Northridge Earthquake. The acceleration histories are collected at two different instrument stations, namely, 24207-Pacoima Dam (upper left) (indicated as PUL104 and PUL194 in Fig. 6) and 24436 Tarzana Cedar Hill (indicated as TAR090 and TAR360 in Fig. 6). The ground motion histories in the event of a single earthquake and recorded in stations with similar geological feature are generally expected to have similar frequency content. However, orthogonal pairs of acceleration histories collected at different locations may have different phase difference, i.e. different time gaps between occurrences of peaks. Such time gaps are estimated to be 0.035 and 2.1 s between the major peaks of the orthogonal components of ground motions collected at Pacoima Dam and Tarzana Cedal Hill sites, respectively. The time acceleration histories and corresponding response spectra of these two pairs of acceleration histories as presented in Fig. 6 also affirm such speculations. The idealized structural systems are investigated under each pair of acceleration histories initially scaled to 0.1 g to maintain consistency in response.

However, residual strength of multistorey symmetric structures are evaluated for two different uncorrelated spectrum compatible ground motions, viz., ACCN1, ACCN2 and average response is plotted in the present paper. The same is also made under NS and EW components of 1940 ElCentro Earthquake record.

5. Variation of system parameters

5.1. Ductility reduction factor

The extent of inelasticity in seismic behaviour exhibited by a structure depends on the ratio of the elastic force demand and the actual lateral strength provided. Such parameter termed as ductility reduction factor

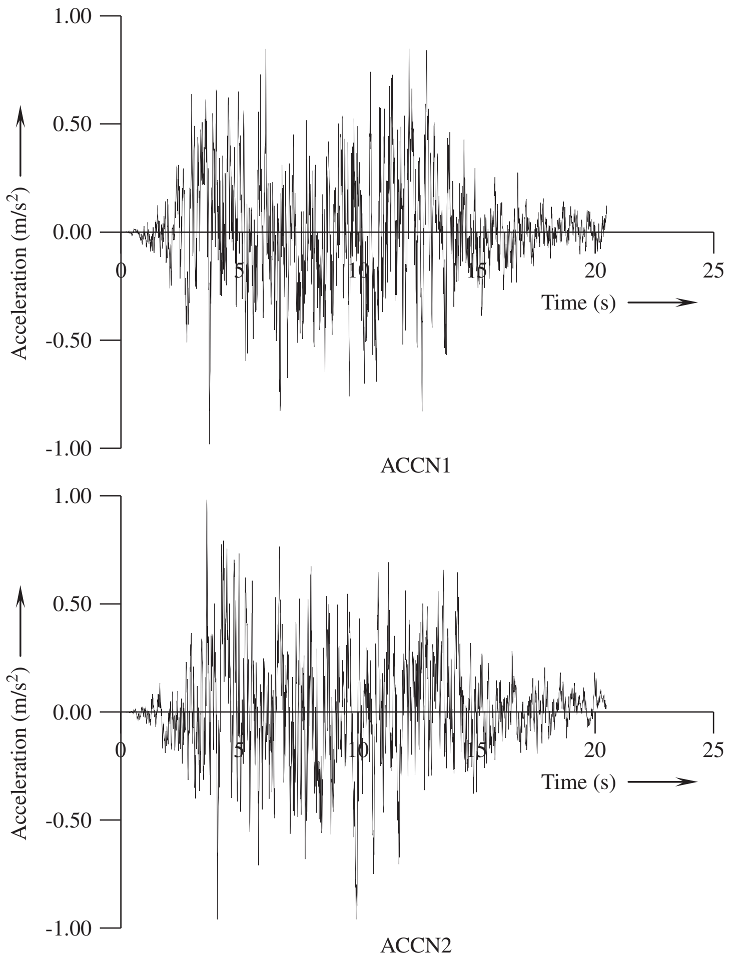


Fig. 5. Two uncorrelated synthetic ground motions consistent with the spectrum in IS 1893:1984 [44].

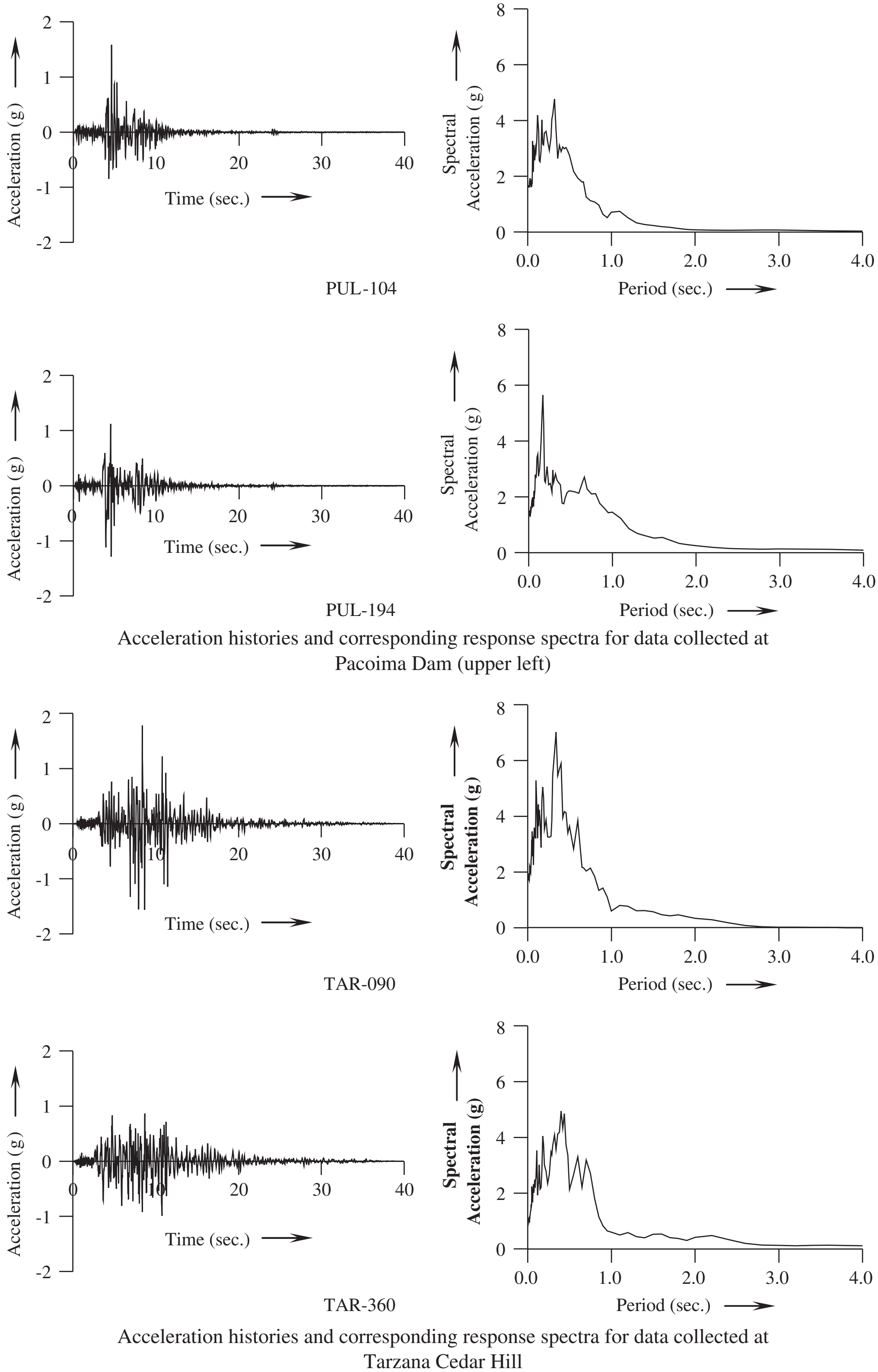
(R_μ) is considered to vary over a wide range of 1–8. However, to comprehend the response of bi-directionally eccentric system, results for ductility reduction factor, $R_\mu = 2$ are only included as the representative one in the limited scope of the present paper.

5.2. Dynamic characteristics

The effect of variation of uncoupled lateral natural frequency of the system (ω_1) and the frequency of the harmonic excitation (ω) are incorporated in a single parameter β as mentioned earlier. The torsional-to-lateral natural period ratio (τ) is varied over a range of 0.25–2.0 with an interval of 0.05, following previous studies [9,30,31] as it likely to cover the range of interest expected for the real buildings.

5.3. Nature of eccentricity

Two types of stiffness eccentricities, namely, small eccentricity ($e/D = 0.05$) and large eccentricity ($e/D = 0.2$) are chosen as the two representative magnitudes of eccentricities of the asymmetric systems considered in the present investigation. All possible combinations of these two eccentricities along two principal directions (X and Y as shown in Fig. 4) are considered for response study of bi-directionally asymmetric structural systems. Thus, such systems with four combinations of eccentricities, namely,



Acceleration histories and corresponding response spectra for data collected at Pacoima Dam (upper left)

Acceleration histories and corresponding response spectra for data collected at Tarzana Cedar Hill

Fig. 6. Characteristic features of seismic excitation data collected at two different instrument stations during 1994 Northridge Earthquake.

$e_x/D = 0.05$, $e_y/D = 0.05$; $e_x/D = 0.05$, $e_y/D = 0.2$; $e_x/D = 0.2$, $e_y/D = 0.05$, and $e_x/D = 0.2$, $e_y/D = 0.2$ are studied under harmonic ground motions. Responses of uni-directionally asymmetric systems with two distinct magnitudes of eccentricities, namely, $e_x/D = 0.05$ and 0.2 are also considered in the present investigation for the sake of comparison. However, the response under synthetic and real ground motions has been studied for bi-directionally eccentric system having small and medium eccentricities, viz., $0.05D$ and $0.1D$, respectively. Response of the systems with medium stiffness eccentricity equal to $0.1D$ is also studied in the limited form. The strength eccentricities of all types of asymmetric systems are maintained identical to the corresponding stiffness eccentricities.

6. Methodology

The nonlinear equations of motion due to earthquake loading is numerically solved in the time domain following step-by-step integration method likewise the previous studies (e.g. Refs. [29,46]), by Newmark's β–γ scheme which considers constant average acceleration over each incremental time step. Newmark's parameters $\gamma = 0.5$ and $\beta = 0.25$ are considered to achieve unconditional stability condition. Iterations are performed in each incremental time step with modified Newton–Raphson technique to improve the accuracy. The time step of integration is taken as not less than $T_1/800$ s, where T_1 is the lateral natural period of the system. The time step of integration is found to be sufficiently small from sample convergence study.

7. Damping

Realistic estimation of damping, particularly for an integral soil–structure-foundation system, is quite complicated. While it is believed that 5% of critical damping in each mode of vibration is reasonable for R/C structure, the same may be as high as 20% for the subgrade medium [41,47,48]. However, such a significant order of soil damping is expected at high strain level which is associated with degradation in stiffness and hence the energy dissipation due to the coupled influence of stiffness and damping may be treated as complementary to each other. In general, clay with moderate to high plasticity is likely to behave linearly even at high strain level and the corresponding damping is not so high [47]. Effective damping of the soil may further reduce depending on the aspect ratio of the structure [49]. It is already recognized that the inelastic range response is insensitive to damping of the system. In this context, the present study considers 5% of critical damping in each mode of vibration to constitute damping matrix, which is hardly a source of variance in the inelastic range response study. However, an equivalent damping of the overall soil–structure-foundation system may also be adopted for this purpose [49].

8. Assessment of residual strength for symmetric structures

In the seismic retrofitting strategy, the primary approach adopted is to improve the probable seismic performance of the structures or otherwise reduce the existing risk to an acceptable level. Thus for the effective implementation of the seismic risk reduction strategy, capacity of the structures after damage needs to be evaluated first. Such capacity is referred to as 'residual strength' of the system and may be expressed as fraction of lateral load-resisting capacity of undamaged system at yield. This may also be indicative of the expected behaviour of structures due to some anticipated earthquake.

The study primarily restricts its focus on low-rise buildings and hence three regular buildings, one with single storey and the rest with two and three stories, respectively, are considered in the first phase of work. Such systems are considered to rest on soft, medium and stiff soil to investigate the effect of SSI. For three-storey system, cases corresponding to soft soil are excluded, as the same with shallow foundation may not be practically viable. The study primarily attempts to judge the consequence of the assumption made in conventional design to consider the structures fixed at base. Hence, strength design for the lateral load-resisting members located at different storey level has been made on the basis of the corresponding storey shear obtained from the elastic analysis of the structures fixed at base. Then, inelastic analysis is carried out both with fixed base as well as flexible base assumption. The residual strength estimated with fixed base assumption reflects the traditionally estimated one, while the same quantified with flexible base assumption are

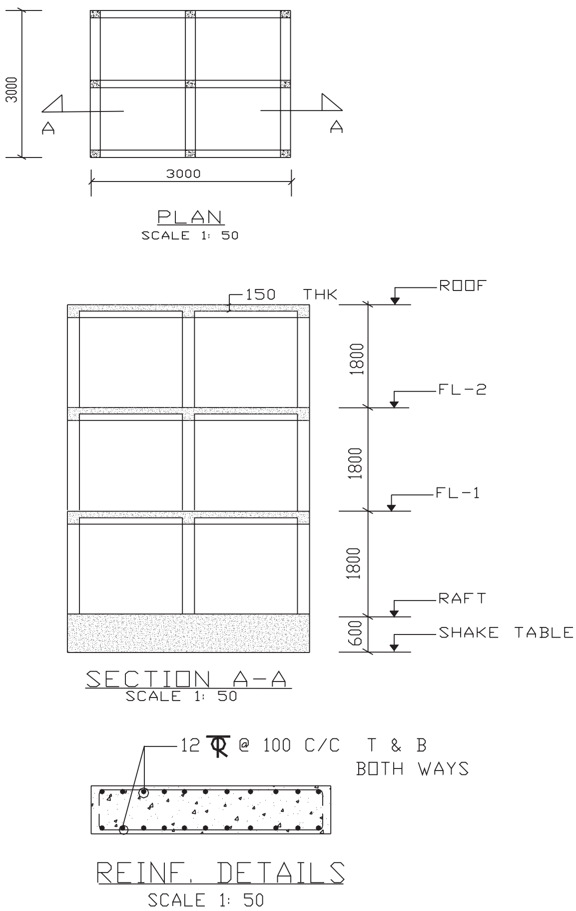


Fig. 7. Structural details of model building frame (supplied by Board of Research in Nuclear Science, India).

the ones expected to be exhibited by the structures in reality. Comparison of such residual strength obtained in these two ways helps to understand the consequence of neglecting the effect of SSI in the process of capacity design. This study may throw light on the post-earthquake damage scenario in the true sense and hence may be valuable input in the process of retrofitting. Two model buildings, one with two and the other having three stories, details of which are supplied by Board of Nuclear Research as available in Fig. 7, have also been analysed at fixed base condition.

8.1. Results and discussions

Inelastic seismic response of low-rise R/C buildings are studied due to synthetic accelerograms compatible with the design spectrum furnished in Indian Standard [44] as specified earlier as well as under 1940 ElCentro ground motion. Residual strength of the load-resisting elements after the seismic shock is expressed as a percentage of the initial strength of the same before experiencing the shock. Variation in the residual strength normalized to the initial design strength of the system, expressed in percentage, is plotted as a function of response reduction factor R_μ. Such quantity exhibited by the system with and without accounting for the effect of soil structure interaction is superimposed in the same figure to facilitate comparison.

Fig. 8 presents the variation of such quantity for one, two and three storey systems due to spectrum compatible ground motions. Fig. 8a exhibits a maximum damage for single storey system and the residual strength in the same is found to be on the order of about 35% at $R_\mu = 8$ assuming the system fixed at base. However, the same is further reduced to the order of about 28% while resting on soft soil. This observation is physically intuitive. Spectral acceleration ordinate sharply increases with little increase in period in short

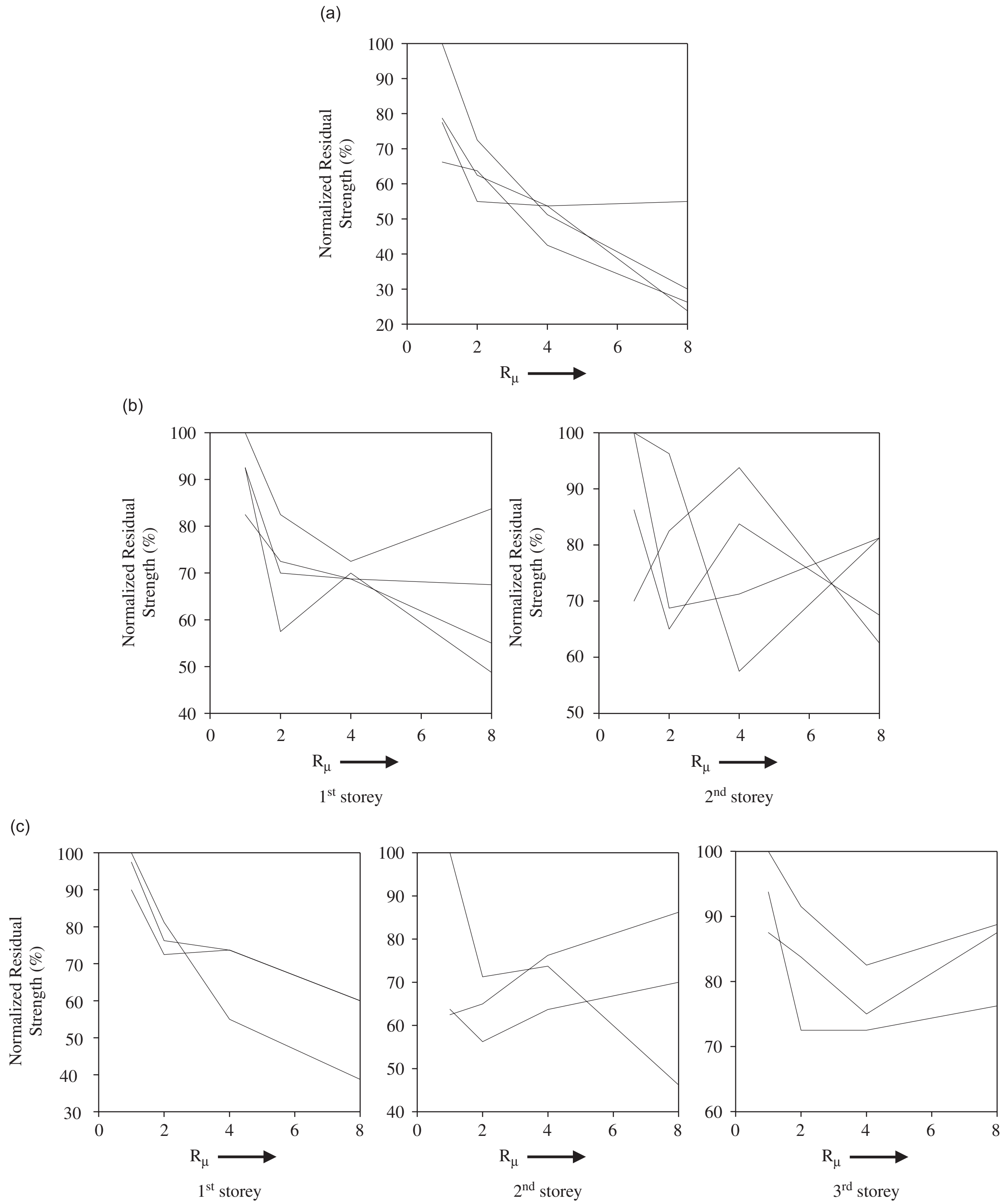


Fig. 8. Variation of residual strength of load-resisting structural members due to spectrum consistent ground motion: (a) single storey, (b) two storey and (c) three storey. —— fixed; –··–··– medium; ··············· soft; – – – – – – stiff.

period range as depicted through any standard response spectrum curve. Period of the low-rise buildings generally lies in the short-period range and the effect of soil-flexibility may lengthen the same. This effectively increases the elastic force demand of the system. Thus the strength provided on the basis of the elastic force demand estimated through the analysis of fixed-base system results in an increase of design R_μ and hence inelastic damage due to the incorporation of soil flexibility. However, for multistorey systems, response exhibited by elements of individual stories may not uniformly follow the identical trend perhaps due to the combined participation of higher modes. Fig. 8 shows overall diminishing trend in such residual strength with increase in R_μ. Results presented for two and three storey systems in Fig. 8b and c, respectively, also exhibit concordant trend. Similar response for two storey system with and without accounting for the influence of SSI due to 1940 ElCentro ground motion presented in Fig. 9 corroborates earlier observation. The limited study, though does not offer any systematic trend, clearly manifests that the residual strength may considerably be reduced due to the impact of soil-flexibility. These variation curves demonstrate that the influence of soil-flexibility on the residual strength of the structural elements located at the ground storey level of the buildings is not so significant in the lower range of R_μ. On the contrary, such influence of SSI appears considerable for higher response reduction factor in the load-resisting elements of ground storey level. This observation is interesting from the standpoint of practical design. In low-rise buildings, in practice, many a time lateral load-resisting columns are designed with uniform strength in all storey levels. Under such circumstances, column hinging is likely to occur in the ground storey elements because of the highest base shear at the corresponding level, while the elements of other storeys may vibrate in the rigid body mode. Thus, the consideration of SSI is crucial at least for systems so designed at higher level of R_μ.

Fig. 10 depicts the variation of such quantity for the load-resisting elements of two and three storey model building system. Such systems are analysed at fixed base condition under spectrum consistent ground motions only. These systems have uniform strength of the load resisting elements in all the storey levels. This may often be the case in conventional design practice for low-rise systems as opposed to the capacity design principle. In such cases, load-resisting elements, of the upper stories experience elastic vibration as, due to the concentration of the highest base shear, plastic hinge develops at the bottom most storey level. Effectively, the structure vibrates like an inverted pendulum with a soft bottom storey. This leads to exhibit no reduction in strength of the load-resisting elements of the higher stories for these two and three storey systems and hence the response of ground storey level is only included in Fig. 10. The figure shows that the residual strength in the bottommost storey level decreases with increase in R_μ . Such decrease may be in the order of about 70% and 38% at $R_\mu = 8$ for two and three storey systems, respectively.

Thus the study discerns the fact that the residual strength of the structures may be significantly influenced due to soil flexibility. However, the limited study does not indicate any definitive trend in response.

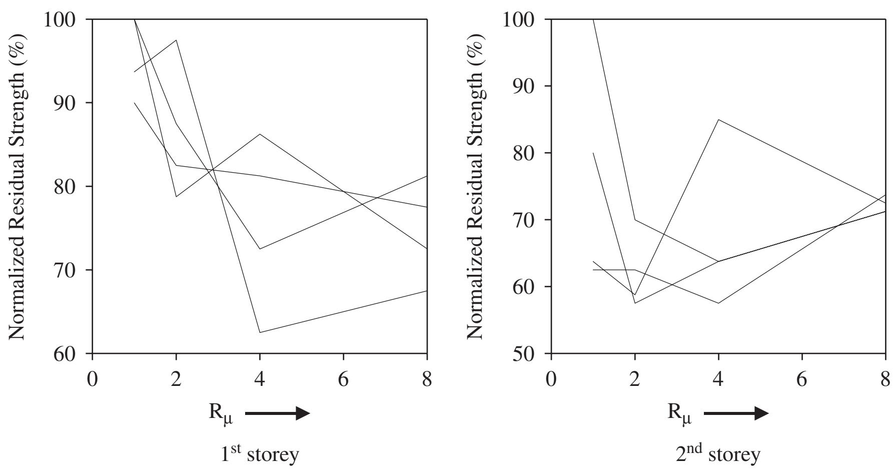


Fig. 9. Variation of residual strength of load-resisting structural members due to 1940 ElCentro earthquake. —— fixed; —··—··— medium; ················ soft; - - - - - - - stiff.

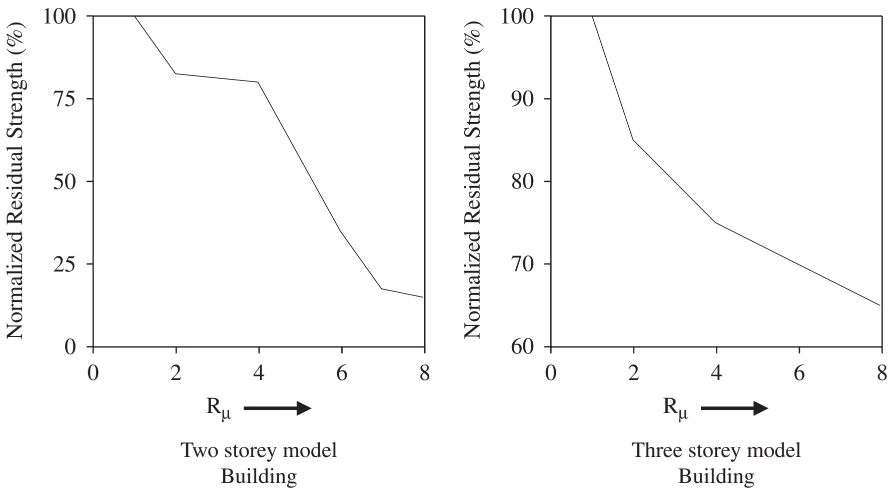


Fig. 10. Variation of residual strength of load-resisting structural members of the model building (Fig. 7) at fixed base condition due to spectrum consistent ground motion.

9. Understanding response of asymmetric structures

To develop insight into the behaviour of bi-directionally eccentric systems, response due to harmonic pulses is carried out in details. Prima facie, elastic range response of such systems is conducted in the limited form to perceive the behaviour easily. Response of similar systems with eccentricity along one principal direction is also included for comparison. Response quantities exhibited by the asymmetric systems are normalized by the similar symmetric systems and presented as a function of τ. Variation of similar quantities under spectrum consistent synthetic ground motions and some recorded ground motions as mentioned earlier are also included to verify the trend.

9.1. Response under single frequency ground motion

Elastic and inelastic range response of the edge elements of bi-directionally eccentric systems with different combinations of eccentricities along two mutually orthogonal directions is studied under harmonic ground motions as detailed earlier. Limited results for uni-directionally eccentric structures is included corresponding to phase difference $\theta = 0$ between two components of ground motion, as responses of such systems do not change with variation of phase difference between orthogonal components of input ground motions. Each of the sets of curves in every figure contains five different curves for the responses of asymmetric systems under five different single frequency harmonic motion corresponding to five different values of frequency ratio β.

9.1.1. Elastic response

Figs. 11 and 12 present the maximum elastic range responses of edge elements of asymmetric systems having equal small and large eccentricity, respectively along two mutually orthogonal directions. Out of several possible combinations studied, results corresponding to $\theta = 0$ and $\pi/2$ are included as representative to exhibit the effect arising due to time lag between the peaks of the ground motions along two principal axes. Results show that the maximum normalized displacement response may increase from about 1.0 to 1.4 in systems with small eccentricity, while such quantity may increase even about 10 times in systems with large eccentricity as θ changes from 0 to $\pi/2$. Thus it appears that the influence of proximity of the peaks of ground motions may play a crucial role to determine the response of bi-directionally asymmetric systems, at least, for systems with large eccentricity. Impact of the same on bi-directionally asymmetric systems having large eccentricity ($e_y/D = 0.2$) in one principal direction and small eccentricity ($e_x/D = 0.05$) in the other, though not included, also seems significant. Similar response of mono-symmetric systems (Fig. 13) shows that maximum displacement for systems with large eccentricity may be about 2.8 times that of the reference symmetric

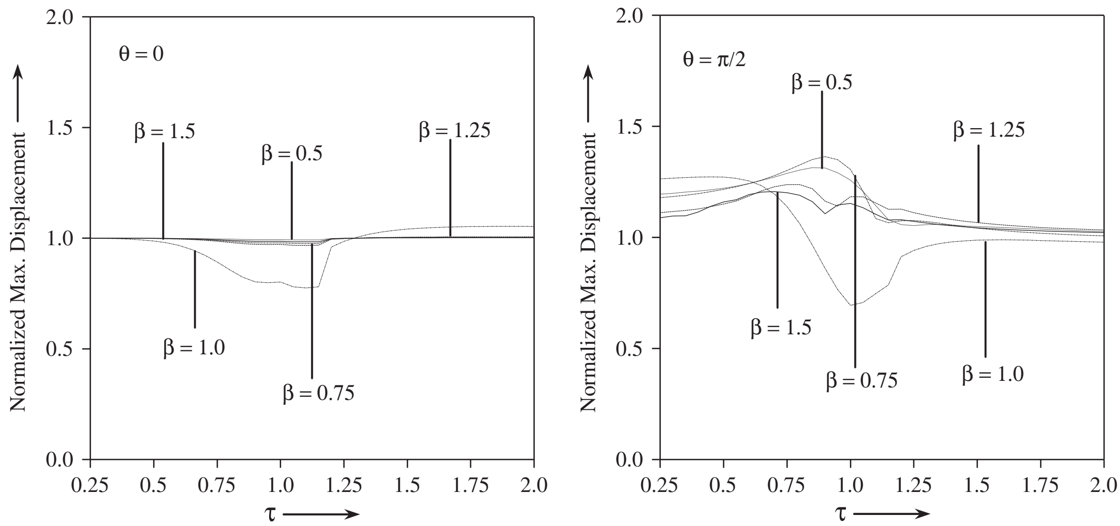


Fig. 11. Maximum elastic displacement response of edge elements of bi-directionally eccentric systems with equal small eccentricity ($e_x/D = e_y/D = 0.05$) in two principal directions.

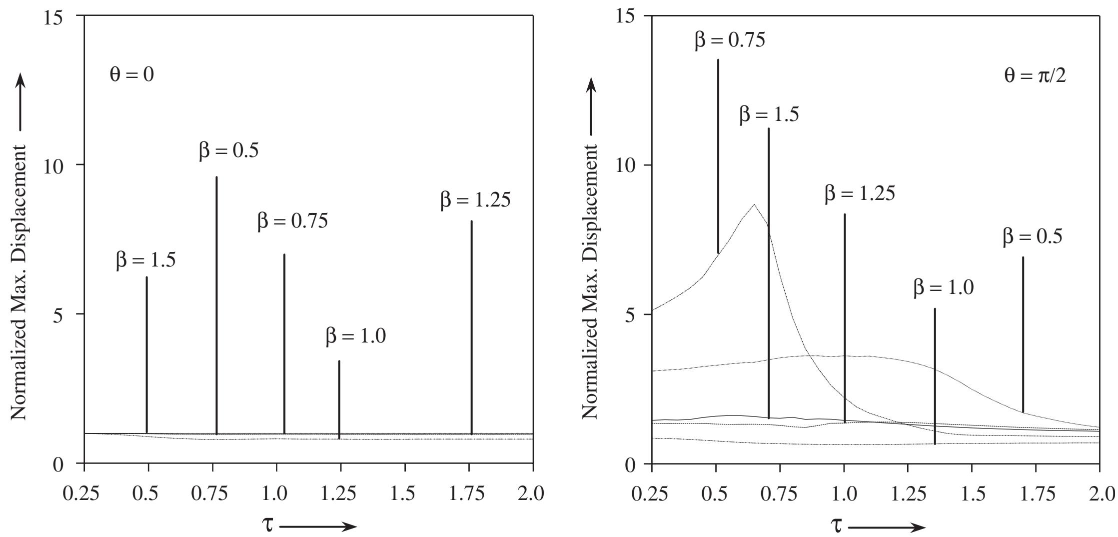


Fig. 12. Maximum elastic displacement response of edge elements of bi-directionally eccentric systems with equal large eccentricity ($e_x/D = e_y/D = 0.2$) in two principal directions.

systems. A comparison of the responses of these mono-symmetric systems with that of the bi-directionally eccentric ones clearly indicates the relatively stronger vulnerability of bi-directionally eccentric systems.

9.1.2. Inelastic response

Inelastic displacement responses of edge elements of bi-directionally asymmetric systems are investigated for both degrading and elasto-plastic hysteresis behaviours of lateral load-resisting elements. Fig. 14 presents normalized maximum inelastic displacement responses of flexible elements of bi-directionally eccentric systems having equal and small eccentricity ($e_x/D = e_y/D = 0.05$) due to degrading hysteresis behaviour. Similar quantities for the stiff sides of such systems are depicted in Fig. 15. Similar response quantities for flexible and stiff sides of systems with small eccentricity along one axis and large eccentricity along other ($e_x/D = 0.05$,

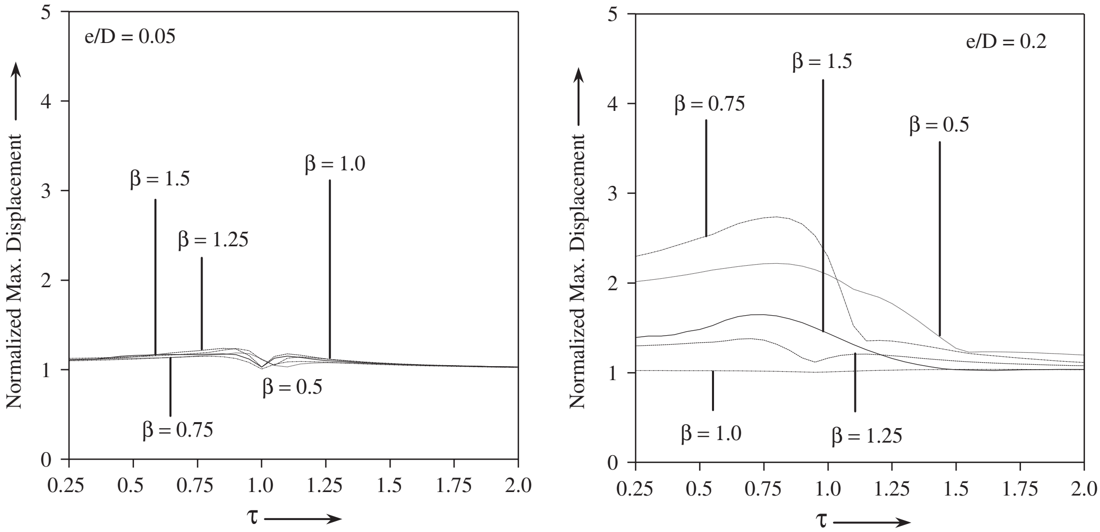


Fig. 13. Maximum elastic displacement response of edge elements of mono-symmetric systems for phase difference $\theta = 0$ between two components of ground motions in two principal directions.

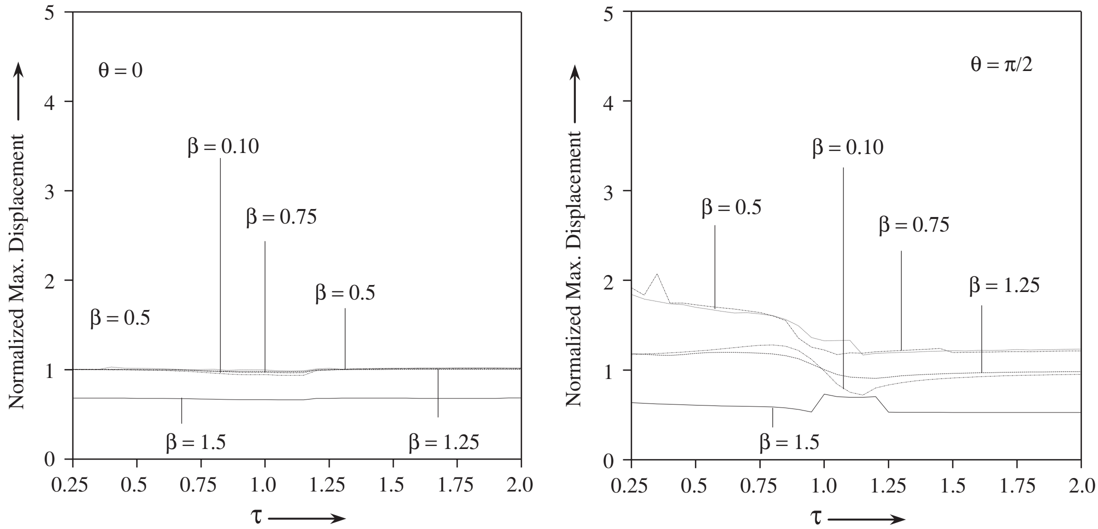


Fig. 14. Maximum inelastic displacement response of flexible element of bi-directionally eccentric systems with equal small eccentricity ($e_x/D = e_y/D = 0.05$) in two principal directions, using maximum degradation.

$e_y/D = 0.2$) are presented in Figs. 16 and 17, respectively. A careful observation of the results discerns that the response may be magnified by about two times in both flexible and stiff sides of systems with equal and small eccentricity (Figs. 14 and 15) depending on the proximity of the peaks of the ground motion. However, such influence may not be so significant in systems with large eccentricity along one axis ($e_x/D = 0.05$, $e_y/D = 0.2$), though the increase in response compared to the reference symmetric system is substantial (Figs. 16 and 17). This observation is physically intuitive. A large eccentricity triggers earlier yielding of one edge element with respect to the other. This leads to the progressive increase of the stiffness and strength eccentricity due to reduction in stiffness and strength of the edge element undergoing earlier yielding, resulting from degrading feature incorporated in the hysteresis behaviour. This results in considerable torsional vulnerability and the difference in response due to time lag between the peaks of the orthogonal components of ground motion gets

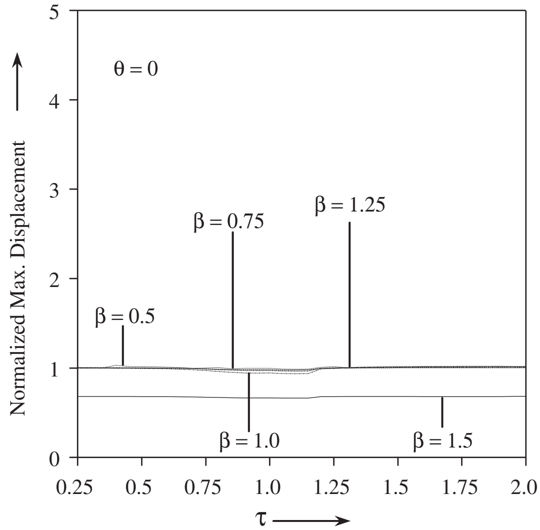


Fig. 15. Maximum inelastic displacement response of stiff element of bi-directionally eccentric systems with equal small eccentricity ($e_x/D = e_y/D = 0.05$) in two principal directions, using maximum degradation.

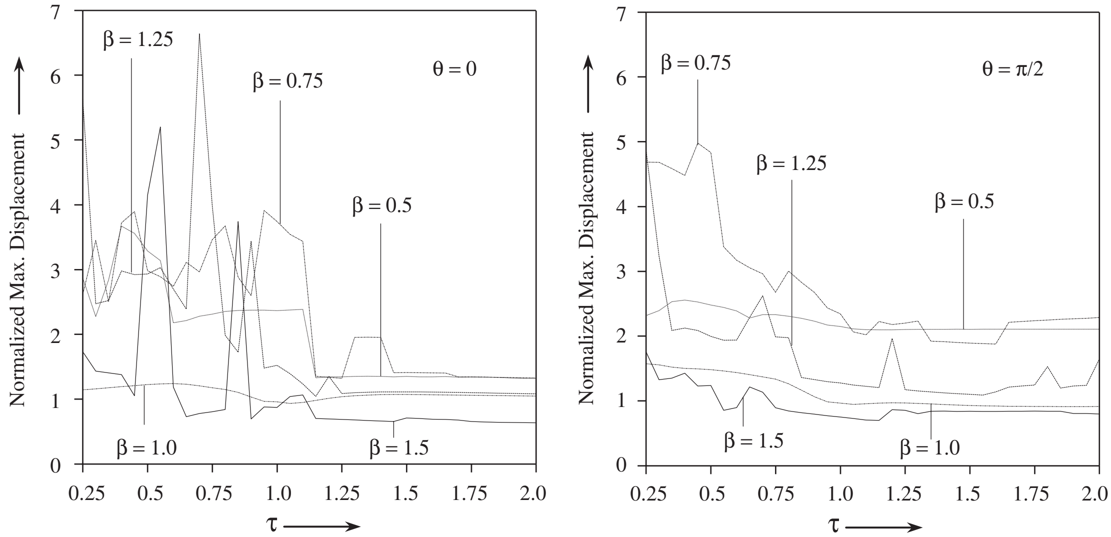


Fig. 16. Maximum inelastic displacement response of flexible element of bi-directionally eccentric systems with unequal eccentricity ($e_x/D = 0.05$, $e_y/D = 0.2$) in two principal directions, using maximum degradation.

subdued. On the other hand, influence of proximity of peak on the overall seismic response does not appear to be significant for systems with elasto-plastic hysteresis behaviour (reference Figs. 18 and 19). Response due to torsion may, however, increase by about 2 times in such systems with large eccentricity.

9.2. Response under spectrum consistent ground excitation

Maximum displacement responses of edge elements of bi-directionally eccentric structural systems with $T_1 = 1.0$ s. and $R_\mu = 2$ are studied in the limited form. Response quantities are computed considering both degrading and elasto-plastic hysteresis behaviour for various combinations of eccentricities and for two different combinations of synthetic acceleration histories, namely, ACCN1, ACCN2 and ACCN1, ACCN1,

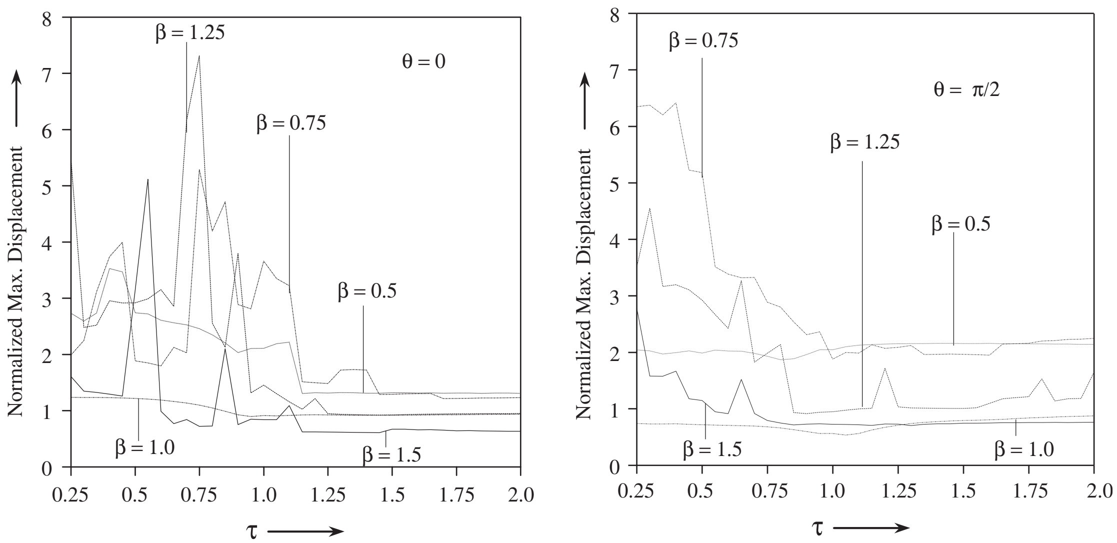


Fig. 17. Maximum inelastic displacement response of stiff element of bi-directionally eccentric systems with unequal eccentricity ($e_x/D = 0.05$, $e_y/D = 0.2$) in two principal directions, using maximum degradation.

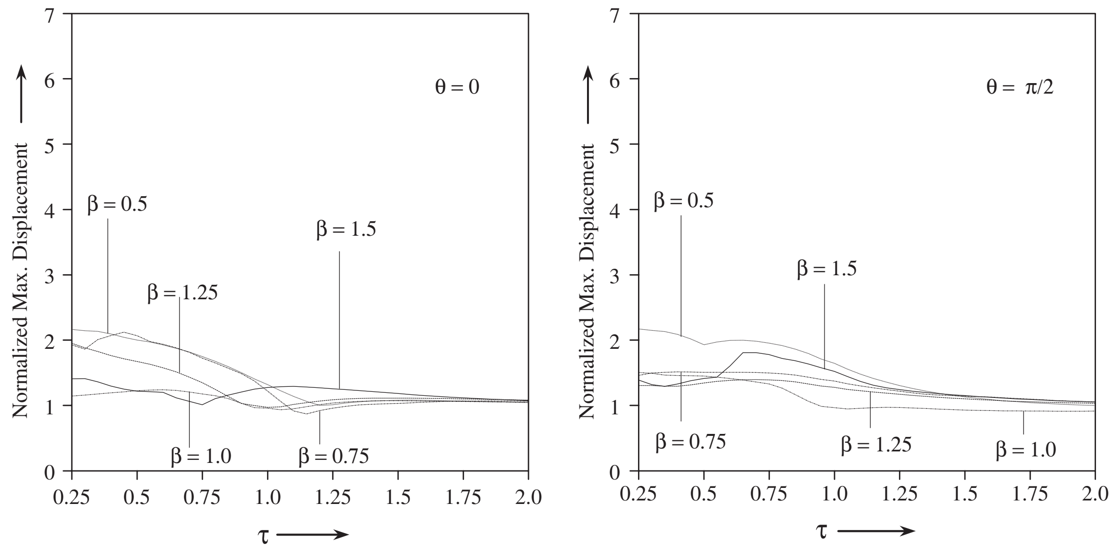


Fig. 18. Maximum inelastic displacement response of flexible element of bi-directionally eccentric systems with unequal eccentricity ($e_x/D = 0.05$, $e_y/D = 0.2$) in two principal directions, using elasto-plastic behaviour.

respectively. The first combination of the ground motions represents the situation when the peaks of the orthogonal components of ground motions are away from each other, while the second is a representative combination having identical time of occurrence (very close in real situation) of peaks of the orthogonal components. In case of a bi-directionally eccentric system, due to the eccentricities along two orthogonal directions, two time-varying torsional moments may generate simultaneously in contrast to single torsional moment arising in case of uni-directionally eccentric system. These two torsional moments generated in a bi-directionally eccentric system may be of additive or cancelling nature depending upon the relative sense of eccentricities of the system with respect to the directions of the ground motions. This may result in different response scenario for similar buildings located on either sides of a street under a similar ground motion. This issue has been comprehensively examined elsewhere [9]. In this context, response under each pair of ground

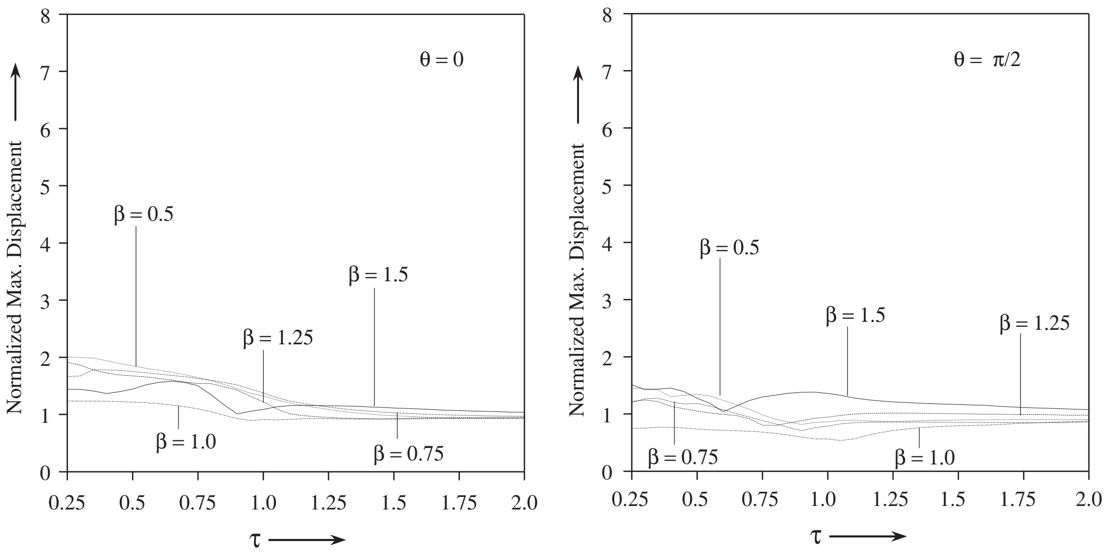


Fig. 19. Maximum inelastic displacement response of stiff element of bi-directionally eccentric systems with unequal eccentricity ($e_x/D = 0.05$, $e_y/D = 0.2$) in two principal directions, using elasto-plastic behaviour.

motions is studied for two different senses of eccentricities to gauge the worst possible response. Maximum element deformation normalized to the same exhibited by the corresponding symmetric counterparts has been plotted in Figs. 20a and b for degrading and elasto-plastic systems, respectively. Careful observation of the results reveals that displacement responses of edge elements of strength and stiffness degrading bi-directionally eccentric systems may considerably be influenced due to the change of occurrence of peaks of the ground motion. Such magnification in response compared to its symmetric counterpart, for the cases considered, seems to vary in the range of about 2–45, 2.5–10 and 2–40 for systems with equal medium eccentricities, equal small eccentricities and a combination of medium and small eccentricity, respectively. Results also exhibit considerable influence of sense of eccentricity to alter the overall seismic response of such systems. For example, response under ACCN1 and ACCN2 seems to vary in a range of about 3–40 and 3–7 in the flexible side of the system with medium and small eccentricities, respectively due to change of sense of eccentricity. This observation is in line with the same made in a recent study [9]. However, such increase or variation is not so significant for similar but elasto-plastic bi-directionally asymmetric system.

In this context, this seems interesting to examine the vulnerability of a bi-directionally eccentric system as compared to its uni-directional counterpart having eccentricities equal to the summation of the eccentricities in both the principal directions of bi-directional ones. To this end, maximum normalized element displacements of such uni-directional counterparts are also superimposed in Figs. 20a and b. Such variation curves manifest a higher torsional vulnerability of bi-directionally eccentric system. This observation is physically intuitive. In a bi-directionally eccentric system, torsional moments may cause an early yielding of the load-resisting flexible elements along both the principal axes. On the other hand, for uni-directionally eccentric systems, such reduction is likely to occur in the single flexible element. This leads to a relatively higher reduction of torsional resistance in bi-directionally eccentric systems. However, the implication of such issues seems marginal in elasto-plastic systems as evident from Fig. 20b.

To be more conclusive, similar systems are considered for investigation under two orthogonal pairs of acceleration histories collected during 1994 Northridge Earthquake. Response of the edge elements observed under the orthogonal pair of ground motion collected at two different stations in the event of a single earthquake is presented in Fig. 21 considering both senses of eccentricities. Response of the corresponding uni-directionally eccentric counterpart is also superimposed on the same figures. A comparison of responses under ground motion collected at Pacoima Dam and Tarzana Cedar Hill as presented in Fig. 21 indicates that maximum normalized displacement responses of flexible elements of bi-directionally asymmetric systems may

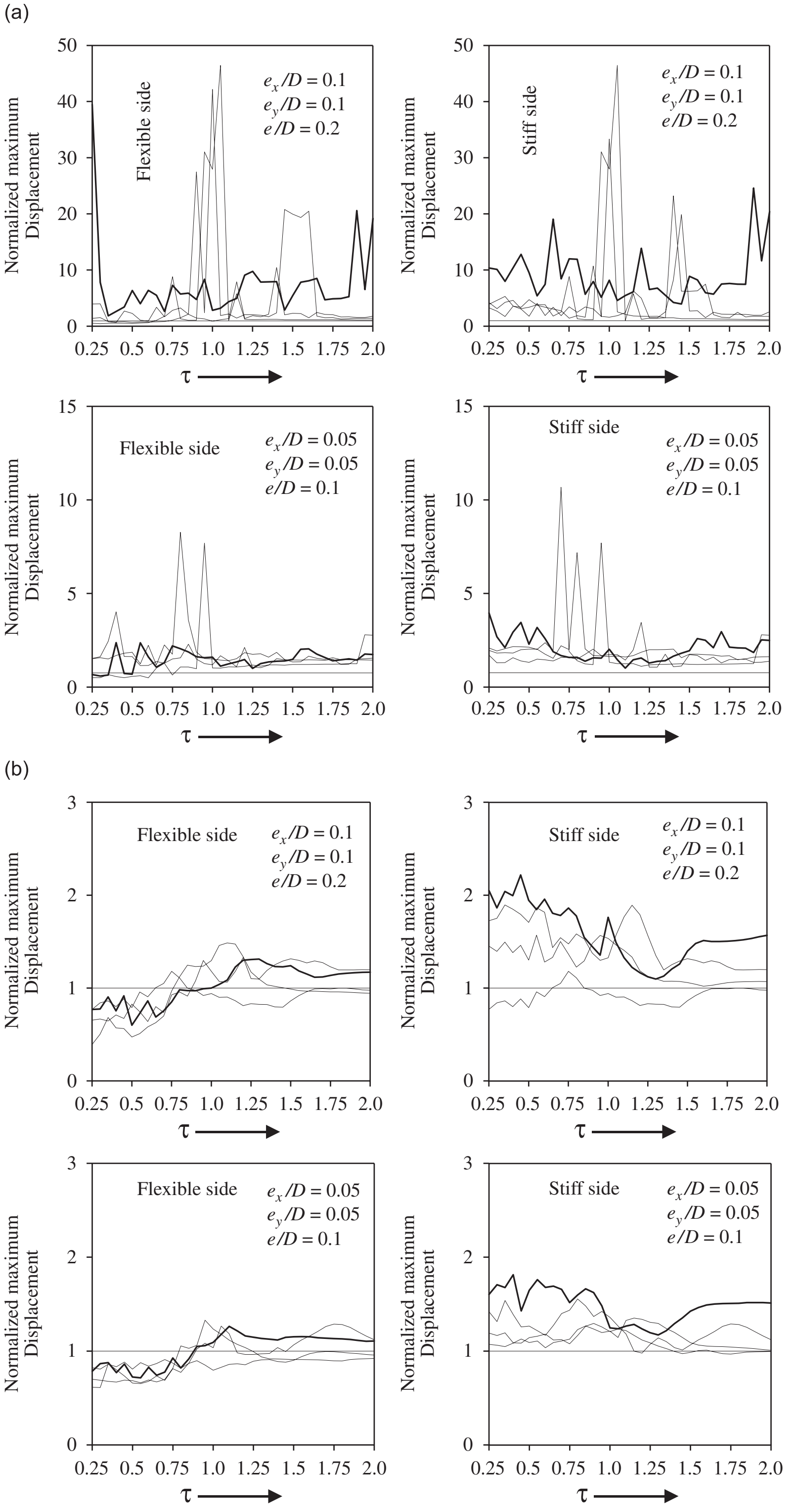


Fig. 20. Maximum inelastic displacement response using (a) degrading and (b) elasto-plastic hysteresis behaviour under spectrum consistent ground motion. For ACCN1, ACCN2 combination: —— e_x/D +ve, e_y/D +ve; - - - - - - - e_x/D +ve, e_y/D −ve. For ACCN1, ACCN1 combination: —·—·—·- e_x/D +ve, e_y/D +ve; —··—··— e_x/D +ve, e_y/D −ve. Uni-directionally eccentric system ━━━.

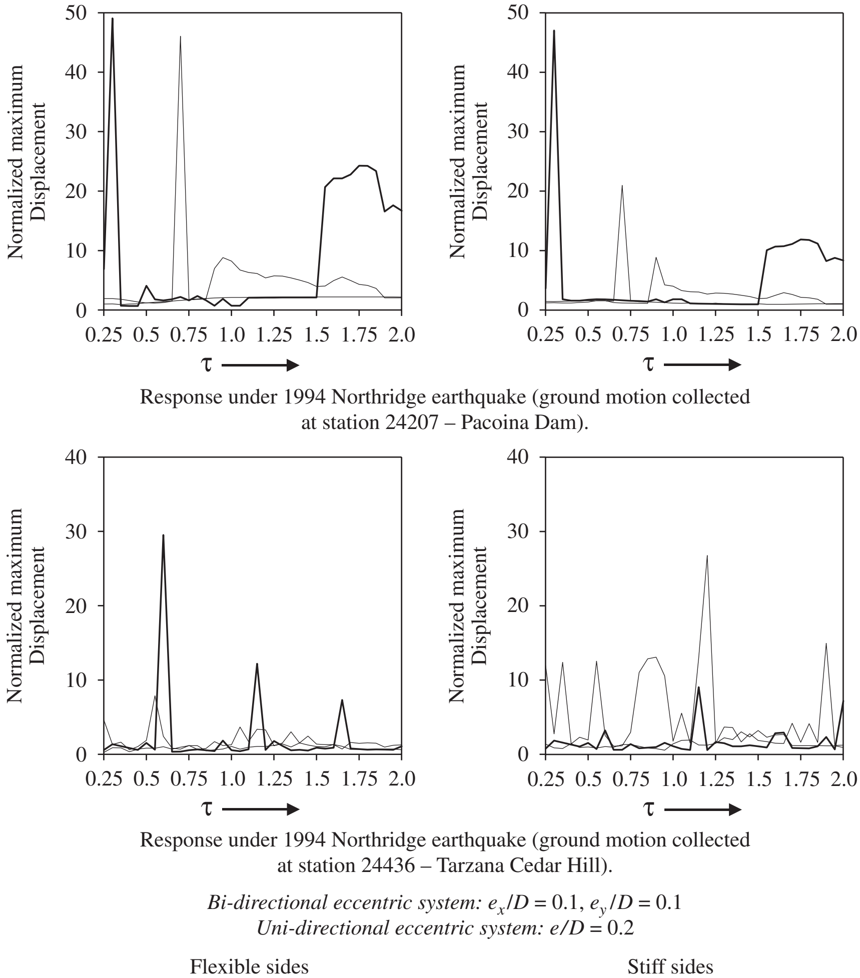


Fig. 21. Maximum inelastic displacement response using degrading behaviour under real ground motion. —— e_x/D +ve, e_y/D +ve; ------- e_x/D +ve, e_y/D −ve; ▬▬ uni-directionally eccentric system.

considerably change due to change in acceleration histories even if their response spectra and PGAs are very similar. Furthermore, positions of occurrence of peak displacements also vary considerably between the responses obtained under two pairs of orthogonal acceleration histories collected at two different instrument stations during a single earthquake. These two figures clearly indicate that responses of bi-directionally asymmetric systems can significantly change under seismic excitations of a single earthquake due to small change in time of occurrence of acceleration peaks resulting from spatial variation. Impact of sense of eccentricity on the overall seismic response in bi-directionally eccentric system also seems considerable as observed earlier and in a recent study [9]. Results superimposed in Fig. 21 for equivalent uni-directional systems confirm the trend of additional vulnerability of bi-directionally eccentric system.

10. Assessment of residual strength for asymmetric structures

In order to assess the seismic performance of the low-rise buildings with asymmetric distribution of stiffness and strength in plan, limited analyses have also been carried out. A typically large eccentricity of $0.2D$ and $0.05D$, a representative of small eccentricity, are chosen for this purpose, where D is the plan dimension of the building. The limited study depicts the reduced strength of load-resisting structural elements of asymmetric systems corresponding to torsional-to-lateral period ratio (τ) equal to 1.0, representative of moderate torsional stiffness, with a strength degradation parameter δ equal to 5%. Response due to code compatible simulated ground motions are studied assuming the structures fixed at base.

Residual strengths for load-resisting members of two storey plan asymmetric (uni-directional and bi-directional) systems due to spectrum consistent ground motions (ACCN1 and ACCN2) have been presented in Fig. 22. Response exhibited by similar symmetric system at fixed base condition as presented in Fig. 8b is also superimposed on the same to gauge the extent of seismic vulnerability attributed due to asymmetry. A comparative analysis of the response exhibited by two storey symmetric, asymmetric (both in uni-direction and bi-direction) system clearly indicates the possibility of the considerable magnification in damage of the load-resisting elements in plan-asymmetric system as compared to its symmetric counterparts. For systems with large eccentricity, normalized residual strength is observed to be on the order of 82.5% in the first storey level of two storey symmetric system at $R_\mu = 2$ whereas the same reduces to the order of about 30% in both uni-directional and bi-directional asymmetric systems. Similar quantity for load-resisting flexible element at the upper storey level is observed to be around 65% and 40% in case of uni-directional and bi-directional asymmetric systems, respectively in contrast to a residual strength of about 96% in the similar symmetric system. Limited investigation reveals that residual strength may sharply come down to about 25% of the original strength due to asymmetry, whereas such quantity is observed to be always 50% or more in the

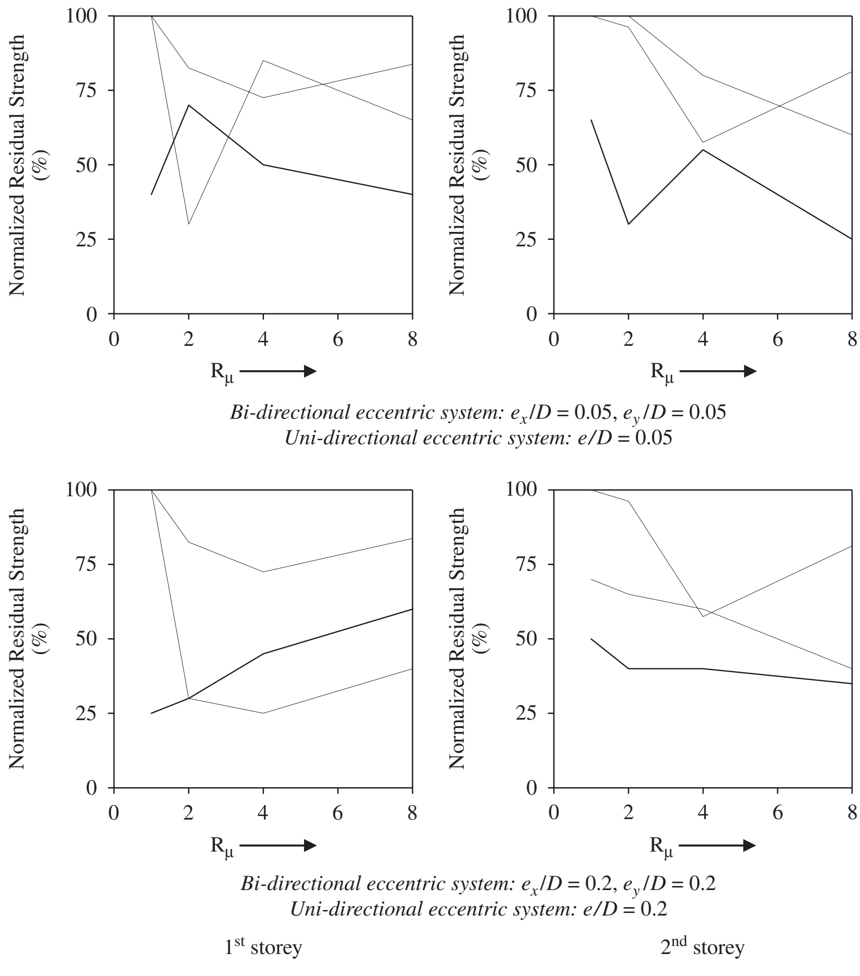


Fig. 22. Variation of residual strength of load-resisting structural members of two-storey system at fixed base condition due to spectrum consistent ground motion. ················ symmetric system; – – – – – – uni-directionally asymmetric system; ——— bi-directionally asymmetric system.

corresponding symmetric system. In structures with asymmetry, coupled lateral-torsional vibration may result in an early yielding of the flexible side load-resisting elements and thus the assessment of the reserve strength of the individual element rather than the system is needed to be made in details. These issues deserve further investigation with and without accounting for SSI. Consideration of relative sense of eccentricities established in a previous study [9] and time lag between the occurrences of the peaks of the ground motions as elucidated in the present paper is also urged in such investigation relevant to bi-directionally asymmetric system.

11. Utility/applicability in design

Provisions for designing and detailing of reinforced concrete frames are relatively new, particularly in many developing countries likewise IS: 13920-1993 [50] in India. Thus, the vulnerability of a considerable portion of the reinforced concrete buildings, particularly of seismic regions needs to be addressed as an important step of seismic preparedness. In this context, the assessment of the safety level of the existing structures and the required strengthening seem to offer confidence to combat seismic hazard. The present study may prepare the background for the same in case of the existing structures providing a primary database for such safety assessment.

The assessment of the structural behaviour made in the current investigation may also lead to important insight to the safety level of the structures survived after the event of an earthquake. A structure designed with $R_\mu = n$ after an earthquake is expected to exhibit a behaviour similar to a system originally designed with response reduction factor $R_\mu = n/p \times 100$, where p is the residual strength of the structure expressed as a percentage of initial yield strength provided (as presented in the figures). Knowing the increased reduction factor of the load-resisting elements after experiencing damage during an earthquake, ductility demand of the same under a similar event may approximately be estimated through using the relationship for ductility equal to R_μ or $1/2[R_\mu{}^2 + 1]$ [23]. Similarly, based on the desired response reduction factor for the structures proposed to be constructed, ductility demand expected to be exhibited by the same may also be estimated.

This is to be noted that the seismic demand and post-earthquake reserve strength of the structures as estimated in the present study are essentially dependent on the fidelity of the hysteresis model used for reinforced concrete structural elements. The maximum lateral load-carrying capacity of reinforced concrete column section can be readily evaluated from the relevant interaction diagram for a known axial load. It is well known that, in case of a typical column, the compressive axial load increases the plastic moment carrying capacity of a section and hence the yield lateral load, as it neutralizes the tensile stress developed due to lateral loading up to the limit of simultaneous yielding of concrete and steel. This implies that the consideration of axial load may bring about a marginal change between the section details of the members to maintain the same yield lateral load capacity, which becomes an implicit assumption of the present study. Hence, most of the existing models as detailed elsewhere [51] to predict hysteresis behaviour of reinforced concrete under cyclic loading do not directly include the effect of axial load unless the model attempts to predict the behaviour from the specifications of explicit section details. However, it may be interesting to examine the effect of axial load on hysteresis model when section details of a structure are specified. Attempt has been made to incorporate the effect of constant axial forces induced due to dead and/live load only in very few studies (e.g. Ref. [52]). However, a limited study [52] depicts that the influence of the same on the variation of ductility demand is not appreciable at least up to a response reduction factor of 4, since the effect of axial force is percolated to the hysteresis behaviour only through changing the yield lateral force carrying capacity of the lateral load-resisting members to a limited extent.

A really accurate incorporation of the effect of total axial force varying due to the consideration of the fluctuating seismic loading is an extremely complicated phenomenon and computational problem to the realm of structural dynamics deserving a separate study beyond the scope of the present paper. In fact, very few models (e.g. Refs. [53,54]) are available till date capable of accounting for the effect of such complex interaction between the fluctuating axial load and varying lateral force produced due to lateral seismic shaking and are so much computationally intensive that it becomes really difficult to be applied for any realistic three-dimensional structure. However, the demand evaluated in the present study at the exclusion of such effect may be at least sufficiently trend-indicative, pending such comprehensive studies.

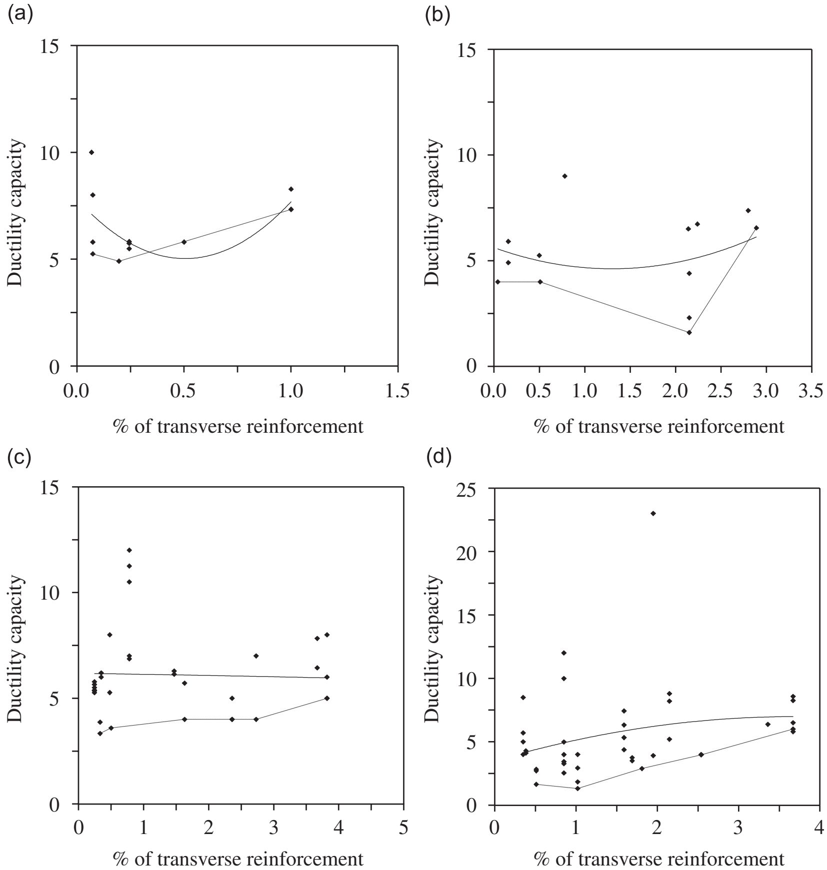


Fig. 23. Curves showing variation of ductility capacity for various reinforcement percentages. (a) Longitudinal reinforcement = 0–1%, (b) longitudinal reinforcement = 1–2%, (c) longitudinal reinforcement = 2–3% and (d) longitudinal reinforcement = 3–3.5%.

In this context, a preliminary idea of member ductility capacity, i.e. ductility demand that may safely be accommodated by the load-resisting members, expressed as a function of reinforcement percentage, may be of some help to the designers to assess the safety level of various structural elements from the discrepancies between ductility demand and capacity. To this end, a number of experimentally observed load–deformation curves of reinforced concrete members under cyclic loading available in the literature (e.g. Refs. [10–19]) have been critically examined. Ductility observed for reinforced concrete members has been plotted with change in transverse reinforcement percentage for different percentages of longitudinal reinforcement. These plots have been presented here as Fig. 23. From the limited available data presented, points are found to be dispersed. However, a best-fit polynomial of second degree has been plotted with a firm line in Fig. 23 to comprehend the general trend. The lower-bound variation has also been attempted to be investigated in the same to have an idea about the minimum expected ductility capacity corresponding to a particular combination of the percentage of longitudinal and transverse reinforcement. The curves may be useful to have a broad idea about the ductility capacity available with reinforced concrete structural members.

12. Summary and conclusions

The present investigation analyses the inelastic seismic behaviour of low-rise structures under strong ground motion incorporating the effect of SSI to gauge the reduced strength of the structures after experiencing seismic damage. These results can, therefore, help to evaluate the additional strength demand for retrofitting. Influence of ground motion characteristics on the inelastic seismic response of bi-directionally asymmetric systems is systematically examined to achieve a fair insight into the behaviour of such systems. Capacity of such asymmetric systems after the seismic event is also studied in the limited form. The observations made in the present investigation may be summarized as follows.

(a) The present investigation reiterates the acceptability of an existing hysteresis model through validating its performance in predicting degrading as well as pinching load–displacement behaviour of R/C members due to cyclic loading. Subsequently, a database is prepared for the strength degradation parameter, a critical input for the model. This may facilitate the use of the model for reinforced concrete structural members with known reinforcement details. However, such database needs be further supplemented with more experimental results.
(b) The variation of residual strength exhibits a decreasing trend with increase in design R_μ. Such quantity may reduce even to the order of about 35–50% of the original strength for low-rise building system. These results may be of help to assess the effective response reduction factor R_μ after the event of seismic shaking. This may indirectly help to assess the ductility demand as well.
(c) Residual strength of the structures may be significantly overestimated if the same is computed assuming the structure fixed at base. Limited investigation on the impact of SSI on residual strength of the structures does not reveal any consistent trend. Thus the case-specific analysis accounting for such effect is emphasized to realistically assess the extent of such damage for important structures. However, in case of low-rise buildings designed with uniform cross-section and reinforcement details throughout, the influence of soil-flexibility may be ignored in the assessment of such reserve strength if the design R_μ is low. On the other hand, effect of soil-flexibility appears significant for such system if the design R_μ is on the higher side.
(d) Such residual strength of the structures may be further deteriorated in asymmetric system due to the effect of torsion. Typically, in a two-storey system, the same may be on the order of about 25% lesser than the corresponding symmetric counterpart.
(e) Response of bi-directionally eccentric systems is sensitive to the time lag between the peaks of two components of ground motion acting along two principal axes of the system. Such influence seems to be stronger in degrading system as compared to their elasto-plastic counterpart. Such response is also found to be regulated by the sense of eccentricities as decided by the quadrant-wise location of the stiffness centre relative to the direction of the ground motion [9]. However, the effect of time lag between the peaks should be considered in addition to this effect to arrive at the worst vulnerable scenario.
(f) Residual strength of bi-directionally eccentric system should, therefore, be evaluated accounting for the above-mentioned factors. To this end, a particular ground motion component having identical occurrence of peaks may be simultaneously considered for two different senses of eccentricity. Such a consideration, though little hypothetical, may be used for the purpose of prediction on the conservative side. Besides, assessment of residual strength of individual element rather than that of the system is deemed essential for such system.
(g) Torsional vulnerability seems to be more serious in bi-directionally eccentric system as compared to a similar uni-directionally eccentric one with eccentricity equal to the summation of eccentricities in both the directions of the bi-directional system. Thus, in the planning of the building, if asymmetry is unavoidable, it should be attempted to be limited in one direction rather than get distributed in two mutually orthogonal principal directions.
(h) The study is also augmented by prediction of ductility capacity through the plots of this quantity at member level for various combinations of transverse and longitudinal reinforcement. These variation curves may be useful to the process of retrofitting after evaluating enhanced ductility demand on the basis of increased R_μ value as explained in the paper. These may also be used as preliminary design guideline for new structures.

Thus the present paper may be of help in the process of damage analysis of the built or to-be-built structures in the event of any anticipated earthquake. Safety level of the structures undergoing seismic shock without collapse may also be assessed to plan for the post-earthquake strategy. In practice, structures are generally found to have asymmetric stiffness distribution to serve various functional and architectural requirements. Such an asymmetric stiffness distribution over the plan area leads to the additional vulnerability of the system as addressed in the present paper in the sample form. Furthermore, the sensitivity of the bi-directionally eccentric *R/C* structures to the possible time lag among the peaks of the ground motion should be appropriately addressed in order to assess the seismic vulnerability of such systems. The present paper may prove useful to provide broad guidelines to address all these issues together and to emphasize the needs of investigating the same in further details.

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