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ESTIMATING SEISMIC SHEAR IN COLUMNS OF RC SPECIAL MOMENT FRAMES

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Abstract

In the United States, state-of-practice methods for approximating seismic shear in special moment frames (SMRFs) may underestimate the magnitude forces that SMRF columns need to sustain during strong ground shaking. Overloading of reinforced concrete columns in shear is likely to reduce the flexural deformation capacities and, in extreme cases, may cause shear failure. This, in turn, may lead to localized damage and failure of the structural system which is why a conservative estimate of seismic shear is imperative to an adequate overall design. In this paper, the results of a series of numerical analyses of four archetype buildings conforming to ASCE 7-10 and ACI 318-14 are presented in order to: a) illustrate the varying estimates of seismic shear in SMRF columns based on several common ACI 318 interpretations and b) develop an approach that better estimates these forces and that can be used as the more appropriate lower bound design seismic shear than the code-based elastic forces from procedures such as the modal response spectrum analysis. It is shown that consideration of system overstrength, dynamic amplifications, and statistical distribution of peak responses leads to a method that produces the most consistent estimate of column shear demand with satisfactory level of conservatism for the buildings studied.

Keywords: Seismic shear; Reinforced concrete; Special Moment Frames; Columns

1. Introduction

Avoiding shear failure in reinforced concrete (RC) frame members intended to resist earthquake loads is accomplished through a design for shear resistance based on capacity-design principles. Generally, the elements are first designed for flexure and axial forces following the appropriate code-based procedures and the flexural yielding regions are identified. Then, the design shear is found by equilibrium assuming that the flexural members develop probable moment strengths in these plastic hinge regions. While this approach is widely applied in beam design, in columns it can produce design shear values requiring large amount of transverse reinforcement that is often difficult, and sometimes impossible to construct.

In a typical special moment resisting frame (SMRF), the actual shear demands on columns generated under earthquake loads are often much lower than the theoretical upper bound values computed based on full development of flexural hinging at the ends of the columns. This is partly a result of the strong column-weak beam design philosophy that favors the development of plastic hinging in the beams along the building height, rather than in the columns. This is implicitly recognized in current design standards [1] and thus, most often, SMRF columns are detailed with less shear reinforcement than is required to sustain the development of probable moment strengths at the column ends. However, the way to determine seismic shear in columns is somewhat left up to the discretion of the engineer, which can produce design values with large variations of conservatism depending on which approach he or she undertakes. In extreme cases, the actual shear demands on columns may be underestimated by as much as a factor of two. This paper presents an alternative method of estimating the column seismic shear demand and is recommended for establishing the minimum design values.

2. Available Methods for Estimating Seismic Design Shear, V_e

Currently, two different publications used in the U.S. address the design seismic column shear in reinforced concrete SMRFs: the ACI 318 design code [1] and the NIST GCR 8-917-1 [2] design guide. Both recognize that designing the SMRF columns for fully developed plastic hinges at the ends may be overly conservative and offer different approaches to quantify the level of seismic shear to be used in the design. The following sections summarize the design procedures recommended, which are then evaluated in Section 6 via nonlinear dynamic analysis of four prototype buildings.

2.1 V_e According to ACI 318-14

According to ACI 318, the design seismic shear for SMRF columns, V_e, shall in no case be less than the factored shear determined from the analysis of the structure (such as the modal response spectrum analysis or the equivalent lateral force analysis set forth in ASCE 7 [3]). Shear strength requirements then require the design shear force V_e to be determined from consideration of the maximum forces that can be developed at the faces of the joints at each end of the column. The joint forces are determined using the maximum probable moment strengths, $M_{pr,c}$, at each end of the column associated with the range of factored axial loads acting on the column. The ACI 318-14 implicitly recognizes that this approach, termed here A, could largely overestimate column shears and could potentially produce the amount of transverse reinforcement that is not constructible. Hence the maximum column shear that needs to be considered in the design is defined as the column shear at the instance when the beams framing into the joint above and below reach their respective probable moment strengths, $M_{pr,b}$. However, the ACI 318 leaves up to the engineer to decide how to distribute the moment unbalance to adjoining columns (Fig. 1).

In practice, it is not uncommon to assume the resisting moment $\sum M_{pr,b}$ at a given joint to be divided between the columns in proportion to their flexural rigidities. In most cases, this implies an even distribution of the moment to the column above and below the joint, which would roughly correspond to the point of contra-flexure being located at the story mid-height. This approach, termed here B1, produces a design seismic shear at floor $i > 1$ equal to $V_{e,i} = (\sum M_{pr,b,i} + \sum M_{pr,b,i} - 1) / 2l_{u,i}$, where $l_{u,i}$ is the clear span length of the column in question. For clarity, the contribution of beam shear at the face of the column to joint moment equilibrium was ignored in this expression, but it is included in the design shear envelopes presented in the corresponding plots in this paper. In the first story, column design shear is obtained by replacing the $\sum M_{pr,b,i}$-1 values by the column $M_{pr,c}$ at level i - 1, that is, at the base of the building.

For comparison, an alternative way to interpret ACI 318 is considered also, termed here Method B2. It is essentially an upper-bound of Method B1 and conservatively assumes that column at level i resists all the probable moments from beams framing into floors above and below the column, that is, $V_{e,i} = (\sum M_{pr,b,i} + \sum M_{pr,b,i} - 1) / l_{u,i}$. This approach doubles the values of shear forces found by Method B1, except at the bottom story, because both methods consider the development of $M_{pr,c}$ at the base of the column, which is typically much larger than $M_{pr,b}$ of the beams and thus controls the value of $V_{e,1}$. In practice, this interpretation is deemed overly conservative, and thus most designers opt for moment distribution that falls somewhere between those obtained by Methods B1 and B2, but fall closer to a less conservative Method B1.

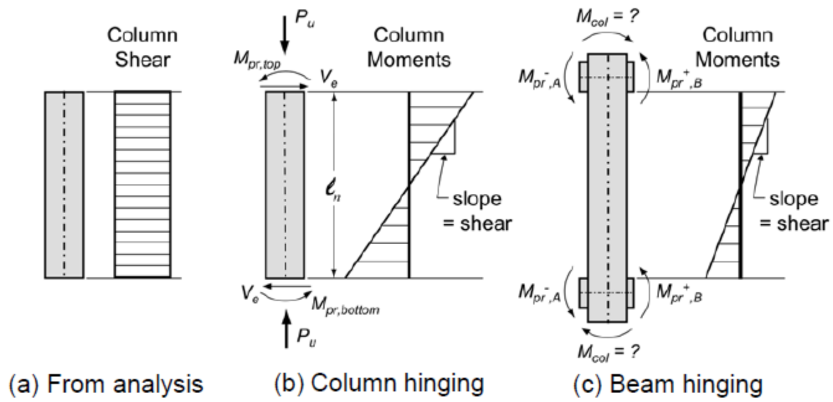


Fig.1 – Design shear in column members, according to ACI 318 (source: NIST GCR 8-917-1)

2.2 V_e According to NIST GCR 8-917-1

In 2008, National Earthquake Hazard Reduction Program (NEHRP) released a NIST GCR 8-917-1 publication [2] intended to serve as a guide to practicing structural engineers in applying the ACI 318 requirements in reinforced concrete SMRF design. This NIST Design Guide addresses the key SMRF design aspects such as flexural and shear design of beams and columns, joint design, and anchorage requirements.

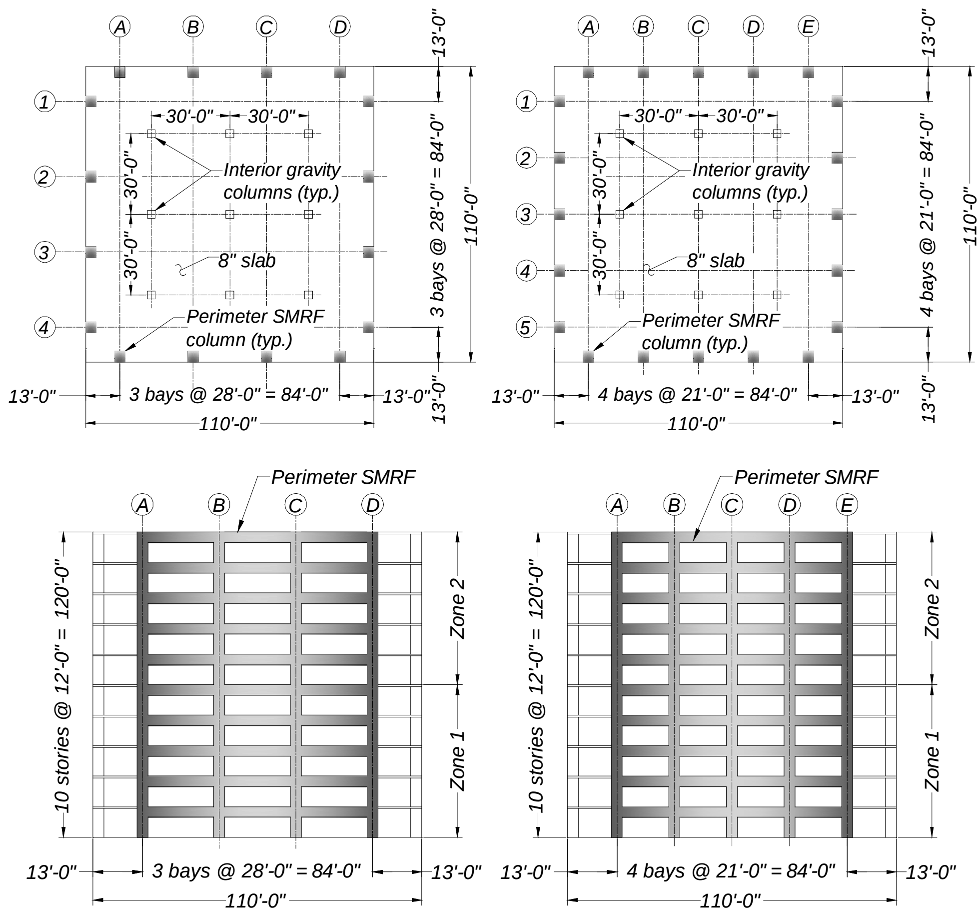
NIST Design Guide [2] points to a tendency for Method B1 to produce grossly unconservative column shear estimates and recommends using Method A whenever feasible. As an alternative to Method A, NIST recommends computing seismic shear based on the envelopes from a code-based elastic lateral analysis and amplified to account for the beam flexural overstrength. In present study, the code-based shear envelopes are those obtained from the modal response spectrum analysis (MRSA) as defined in the ASCE 7 and are denoted as V_{MRSA}. This is also the minimum design shear required by the ACI 318 provisions. Then, according to NIST Design Guide, the V_{MRSA} should be amplified by a factor corresponding to an average flexural overstrength of all beams framing into the column, Φ_m = average($M_{pr,b,i,j}/M_{u,i,j}$), where $M_{pr,b,i,j}$ signifies probable moment strength of beam i (where more than one beam frames into the column at a given floor) of story j and $M_{u,i,j}$ signifies the seismic moment from the elastic lateral analysis for that same beam.

3. Archetype Buildings

Four buildings with varying SMRF geometries and all having perimeter SMRF arrangement (Fig. 2) are numerically investigated in this paper. Two buildings are 10- and the other two 20-stories tall, with each story being 3.65m high. For each building height, a three 8.5m-bay and a four 6.4m-bay SMRF configuration is considered, as shown in Fig. 2. The buildings are labeled “AXX-Y,” where “XX” denotes number of stories and “Y” number of bays (e.g. A20-3 is a 20-story building with a 3-bay SMRF configuration).

The four buildings conform to ACI 318-14 and ASCE 7-10 provisions. According to ASCE 7 classification, each building belongs to risk category II with seismic importance factor I_e = 1.0. All four buildings classify under seismic design category D, according ASCE 7. Gravity loads include the self-weight of structure and permanent non-structural components and contents. Total seismic weight of the four buildings, W_t, includes 100% of dead load and 25% of the live load (which is taken as 2.87 kN/m^2). Because each principal building direction has two special moment frames, the seismic weight per frame is $W = W_t/2$.

Design forces were determined with the code-prescribed MRSA procedure using square root of sum of squares (SRSS) modal combination rule with a response modification factor R = 8. The first five modes were included in the elastic analysis of 10-story buildings, which accounted for more than 97% of the modal mass. The first ten modes were included in the elastic analysis of 20-story buildings, which accounted for more than 98% of the modal mass. The effective flexural rigidities for columns and beams used in the elastic MRSA analysis were $0.5E_cI_g$ and $0.35E_cI_g$, respectively [2], where I_g = gross section moment of inertia and E_c = elastic modulus of concrete, which was calculated as 31,690 MPa based on a nominal concrete strength of 41.4 MPa. Nominal strength of steel was taken as 413 MPa. All detailing of SMRF complies with ACI 318-14. More details on the design of the buildings can be found in [4].



(a) (b)

Fig.2 – 10-story archetype buildings (20-story buildings similar) in (a) three-bay SMRF configuration and (b) four-bay SMRF configuration (3.28’ = 1m)

4. Seismic Hazard

Four archetype buildings are located on a hypothetical site in San Francisco, CA, on stiff soil corresponding to site class D, according to ASCE 7 (shear wave velocity ranging between 182 and 366m/s in top 30m of soil). The analyses presented in this paper correspond to a design earthquake (DE) shaking intensity having 10% probability of exceedance in 50 years. Ground motions were selected using the PEER Ground Motion Database selection tool [5] to match the 5% damping ratio smoothed design spectrum of ASCE 7. The acceleration records were linearly scaled so that the geometric mean of the FN and FP components approximately matches the smoothed design spectrum prescribed by the ASCE 7 code for a given site, between the specified periods of interest; namely between T_2 and $1.5T_1$ (T_1, T_2 = first and second mode of vibration periods, respectively), as shown in Fig. 3. Because of differences in vibration periods between the 10- and the 20-story buildings, different sets of ground motions (bins) were used for different building heights. Each bin contains thirty different ground motions, each of which includes both fault-normal and fault-parallel ground acceleration components. All of the motions selected occurred in events on strike-slip fault types and some ground motions include distinct velocity pulses due to directivity effects. Details on individual ground motions are found in [4].

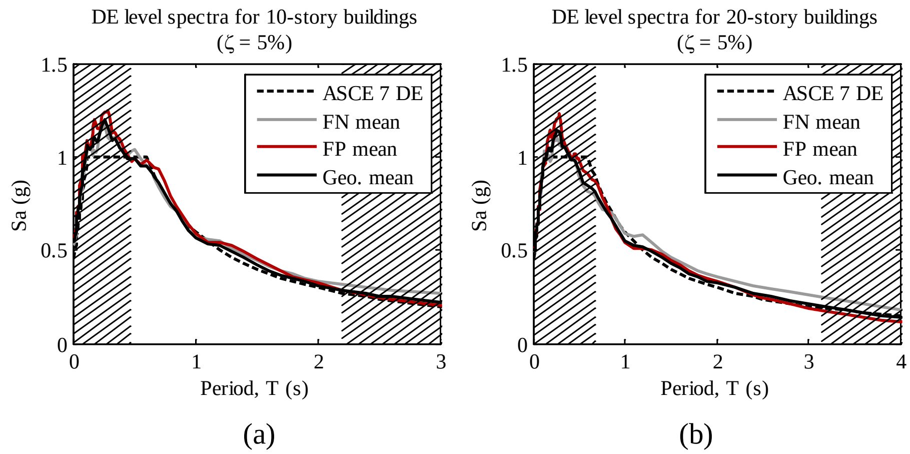


Fig.3 – Pseudo-acceleration spectra for ground motions used to analyze (a) 10-story (Bin 1) and (b) 20-story (Bin 2) buildings (unhatched area shows the period range used for matching the design spectra)

5. Numerical Model

A two-dimensional mathematical model consisting of a single SMRF with lumped mass and vertical load applied at the joints was implemented on OpenSees platform [6]. Both beams and columns were represented by the force-based Euler-Bernoulli nonlinear fiber-section frame elements with P-Δ geometric transformation. This modeling approach includes axial force - bending moment interaction, but also allows for the adequate representation of the beam elongation effects, which has been shown to have significant effect on the nonlinear response of RC frames [7, 8, 9, 10].

Beam-column joints were modeled with rigid frame elements connecting between the centerline of columns to the ends of beams. The length of a rigid frame element on either side of the joint is equivalent to half of the dimension of the column in that direction (h/2), so that the total rigid zone has the same width as the column width in the plane of the frame. Numerical models accounted for strain penetration of beam longitudinal reinforcement into joints and column longitudinal reinforcement into the foundation by means of rotational springs based on zero-length elements with fiber sections, as described in [4]. Because the aim of this study was computing the levels of shear in column members that would need to be accommodated during nonlinear response, it was assumed that shear yielding mechanism does not occur. Thus, the shear forces computed are not bounded by some hypothetical capacity, but were rather allowed to reach any level necessary to balance the moments developing in beams and columns during the dynamic response of the buildings. Slab effects were not considered in the numerical model. Damping matrix was defined based on initial stiffness Rayleigh damping with 2% damping ratio in modes 1 and 3.

The models of buildings of the same height have very similar properties, with 10-story buildings having fundamental period T_1 = 1.4-1.47s, and 20-story buildings having T_1 = 2.12-2.24s. For all four buildings, the ratio of first to second mode period T_1/T_2 is approximately 3.1, while the ratio of the first to third mode period T_1/T_3 ranges between 5.4-5.7. The effective modal mass of the first mode normalized with the total mass M_1/M is around 0.68 for all four buildings, while the corresponding range for the second mode is 0.10 for 10- story and 0.13 for the 20-story buildings.

6. Column Shears Observed in Nonlinear Dynamic Response

Fig. 4 shows column shear envelopes obtained from the nonlinear response history analyses (NRHA) for two example buildings- a ten story, three-bay (A10-3) and a twenty story, four-bay (A20-4) building. For each building, the plots are shown separately for the exterior and the interior columns. All shear values are normalized by the $A_g\sqrt{f'_c}$ where A_g is the gross area of the column and f'_c is the compressive strength of concrete, expressed MPa. Grey lines represent the maximum shears computed during each analysis run. The red lines are the average values for each fault-normal (FN) and fault-parallel (FP) component group. The plots in Fig. 4 also show the design envelopes based on the methods described in Section 2.

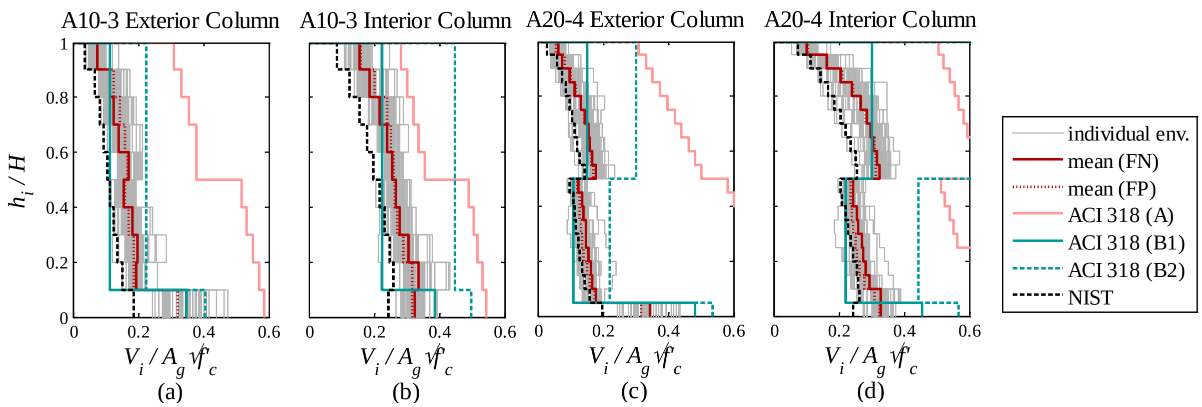


Fig. 4 – Shear response envelopes for columns of A10-3 and A20-4 buildings (h_i = story height, H = overall building height, V_i = computed column shear)

Based on the results presented in Fig. 4, it is evident that most available design methods can largely underestimate the ultimate shear forces developed in the columns under the DE level ground motions. The exception is the ACI 318 Method A, which corresponds to the columns hinging at both ends and is the upper bound for the design shear. This method seems to provide the best estimate of ultimate shear in interior columns of 10-story buildings (Fig. 4[b]), but produces overly conservative design values in most of the lower half of the exterior column (Fig. 4[a]). In interior columns of 20-story buildings (Fig. 4[d] and [e]), ACI 318 method A tends to overestimate design shear by a factor of 4 or more. This factor is further augmented in exterior columns, because the exterior columns tend to resist less shear than the interior columns in typical design, while developing larger flexural capacities under transient compressive loads due to overturning.

Commonly used ACI 318 Method B1 underestimates the ultimate column shear along at least 70% of the building height in all four archetype buildings. Thus, this method should be avoided in design, as previously suggested in the NIST Design Guide [2]. ACI 318 Method B2 provides conservative estimates for most ground motions in all buildings, but the shear forces reached in the lower third story in the exterior column of A10-3 (Fig. 4[a]) exceed those estimated for four ground motions. For the four archetype buildings where beam flexural capacity was kept uniform along the height, this Method B2 overestimates the design shear by more than a factor of three in the upper half of each building. This is especially evident in the interior columns of 20-story buildings (Fig. 4[d] and [e]). It is also evident that the design envelopes produced by Methods B1 and B2 for the buildings studied do not represent the spatial distribution of the shear forces. Despite the apparent conservatism of Method B2, it is shown elsewhere [4] that this method can largely underestimate seismic shears in frames that have disproportionately sized columns. For example, a parametric study of frames with increasingly larger exterior column cross sections has shown that the actual seismic shear demands reached in the exterior columns exceed those estimated by Method B2 in over half of the building. Refer to [4], Chapter 5 for more details.

Method suggested by NIST Design Guide underestimated the mean column shear in all cases by at least 25%. However, because this design method was based on MRSA, it resulted in an ultimate shear envelope that most closely followed the mean column shear envelopes from NRHA.

7. Proposed Method to Estimate Seismic Column Shear, V_e

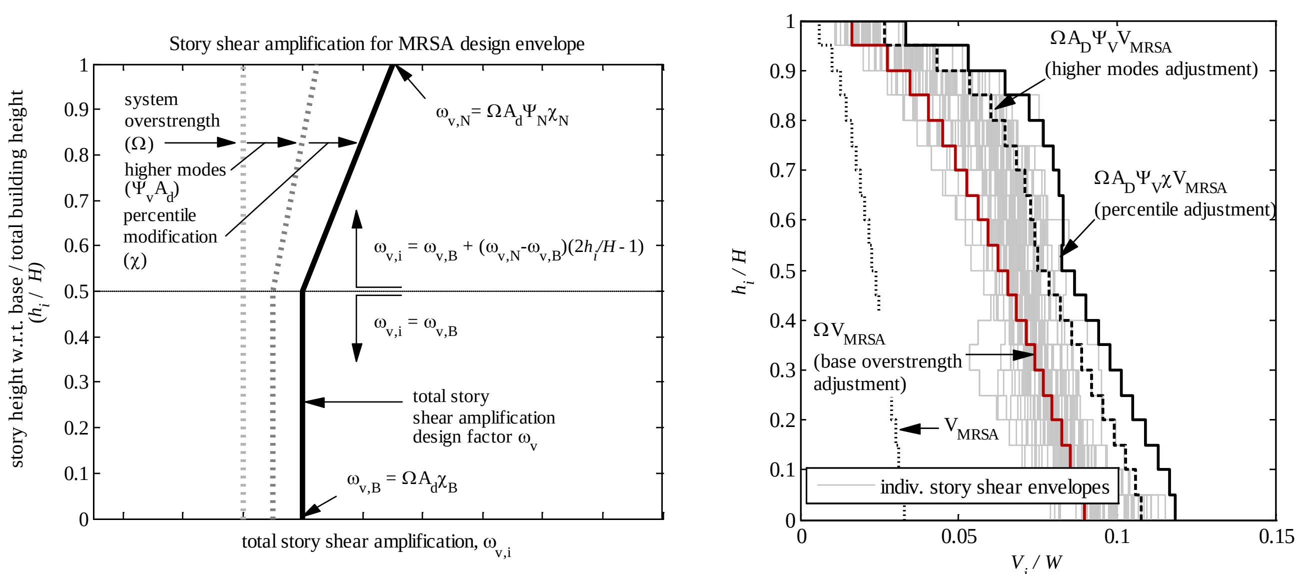
Similar to the NIST Design Guide procedure, the proposed method of estimating the design seismic shear in columns is based on amplifying the MRSA-obtained shear envelopes by the appropriate factors to obtain the shear forces comparable to those observed during the NRHA. Instead of considering only the flexural overstrength of SMRF elements as recommended by [2], additional factors are applied to account for dynamic effects from higher modes and also a statistical variation of the individual responses during the NRHA.

The proposed expression to estimate the seismic column shear V_e at story i is:

$$V_{e,i} = (\kappa_v)(\omega_{v,i})V_{eMRSA.i} = (\kappa_v)(\,\Omega\, A_D \Psi_{v,i}\, \chi_{,i})V_{eMRSA.i} \tag{1}$$

where $V_{eMRSA,i}$ is the column shear at story i obtained from the elastic MRSA analysis, $\omega_v = f(\Omega, A_D, \Psi_v, \chi)$ is the story shear amplification factor and κ_v accounts for the difference between the exterior and interior column shear amplification.

As will be shown in the subsequent sections, the largest amplification of shear forces compared to those obtained in the MRSA procedure is due to design overstrength inherent to the system, which originates from both design overstrength and the differences between the nominal and expected material strengths. Higher mode effects also contribute to increased shear forces, particularly in the upper levels of the building. The percentile adjustment factor is intended to capture the response variability under different design level ground motions. Fig. 5(b) illustrates the result of amplifying the MRSA-based *story* shear envelope, plotted alongside the response envelopes from NRHA for a sample building, A20-4. These results are for DE level of shaking and account for 84th percentile of ground motions. Each factor used to calculate V_e is discussed separately in the subsequent sections. With the exception of κ_v, all factors were derived based on the story shear amplification.



(a) (b)

Fig. 5 – Proposed story shear amplification factors: (a) variation of amplification factor along the height, (b) calculation of story shear envelopes using the proposed equations and factors (The plot includes both FN and FP peak shear envelopes, plotted in light grey.)

7.1 System Base Overstrength Factor: Ω

The system base overstrength factor, Ω, is similar to the overstrength factor presented in NIST Design Guide [2]. While the overstrength factor in the NIST Design Guide accounts for the flexural overstrength of all beams framing into a column for which design shear is to be calculated, the presented method defines a single overstrength factor for the entire SMRF based on the average flexural overstrength of all hinges in an idealized strong column – weak beam mechanism. That is, Ω is calculated considering two plastic hinges in each beam and also plastic hinges at the base of the SMRF columns. In the equation form:

$$\Omega = \frac{\sum_{k=1}^{NCol} M_{pr,c,k} + \sum_{k=1}^{NBm}\left(M_{pr,b,i} + M_{pr,b,j}\right)}{\sum_{k=1}^{NCol} M_{E,c} + \sum_{k=1}^{NBm}\left(M_{E,b,i} + M_{E,b,j}\right)} \tag{2}$$

The numerator in Eq. 2 represents the sum of the probable moment strengths of all plastic hinges, i.e., the probable moment strengths of each column at the base $M_{pr,c,k}$, and probable moment strengths of each beam $M_{pr,b,i}$ and $M_{pr,b,j}$, at ends *i* and *j*,respectively. Terms *NCol* and *NBm* denote the number of columns and beams, respectively. The denominator in Eq. 2 is the sum of the corresponding plastic hinge seismic moment demands calculated from the elastic code-based procedure. Table 1 lists the factors Ω calculated with Eq. 2 written as exterior/interior/middle (where applicable) column. Middle column refers to column on grid line C in four-bay frame configurations (Fig. 2), while the interior columns are on grid lines B and C, and B and D, for three- and four-bay frame configurations, respectively. Included for comparison are the overstrength factors Φ_m from the NIST Design Guide [2]. As can be seen, Ω is approximately 10% larger than Φ_m.

Table 1 – Calculated amplification factors for archetype buildings.

	A10-3	A10-4	A20-3	A20-4
Φ_m (NIST)	2.58/2.62	2.45/2.5/2.54	2.23/2.20	2.62/2.55/2.48
Ω	2.81	2.72	2.45	2.80
A_D	1.16	1.17	1.17	1.07

7.2 Higher Modes Amplification Factors: A_D and Ψ_v

Amplifying the elastic MRSA-based story shear with the system overstrength factor Ω brings the estimated forces closer to those computed in NRHA (Fig.5[b]). However, it is clear that the spatial distribution of shears calculated from the individual NRHA runs (marked in light grey), tends to deviate from that computed by the elastic analysis. This phenomenon is caused by the differences in the higher modes contribution to the total response during the nonlinear analyses and is most apparent in the upper half of the building (Fig.5[b]). Higher modes effect on moment and shear distribution in multistory RC frame and also wall buildings has been studied by others [11] and is explicitly addressed in the New Zealand concrete design code [12].

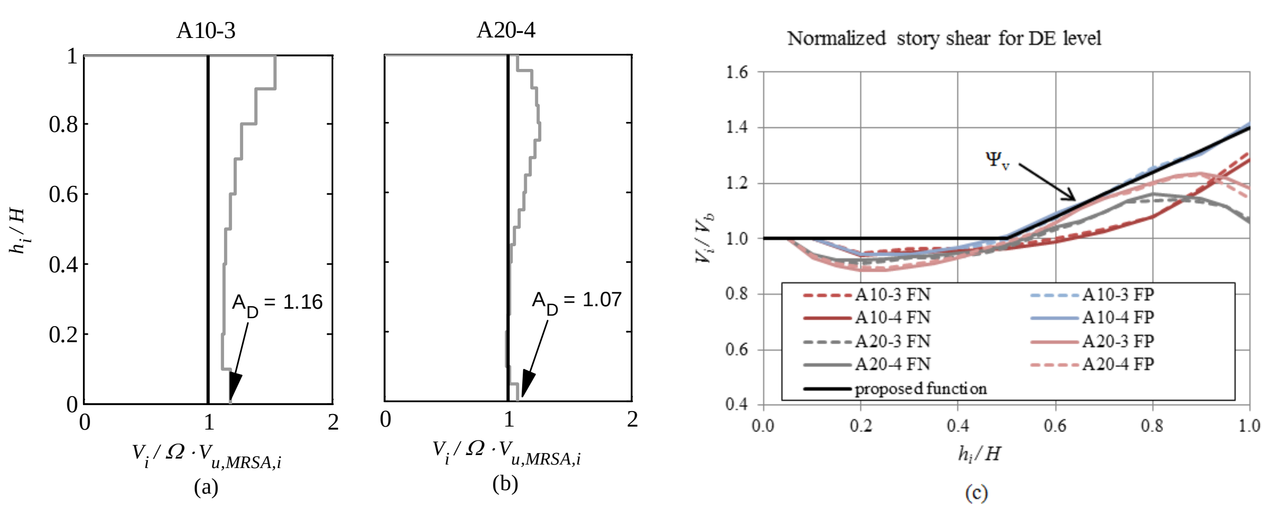


Fig. 6 – Higher mode effects on story shear distribution: (a) & (b) ratio of mean shear to MRSA-based shear amplified with flexural overstrength factor only in A10-3 and A20-4 buildings, respectively; (c) definition of Ψ_v for all archetype buildings.

Higher modes contribution to the response is better visible in Figs. 6(a) and (b) which plot the ratio of the mean story shear to the MRSA-based story shear amplified only to account for the system overstrength, as defined in Section 7.1. From the responses of sample 10- and 20- story buildings (Figs. 6[a] and [b]), it can be seen that higher modes amplify shear forces both in the upper portions of the building and at the base. Factor A_D is defined here such that its product with ΩV_{MRSA} gives the mean base shear computed for the 30 ground motion runs for a given building. For the archetype buildings, A_D fluctuates between 1.07-1.17 (Table 1). For all practical purposes, A_D = 1.2-1.25 is recommended for design.

Fig. 6(c) plots a vertical distribution of story shears for all archetype buildings normalized with their respective base shears V_b. It is clear that amplifying the code-based elastic story shears (V_{MRSA}) by Ω and A_D will produce adequate shear estimates only in the lower half of the building. Thus, a multiplication factor Ψ_v is introduced to account for the higher mode effects in the upper half of the building. Fig. 6(c) suggests that a simplified, piecewise linear variation of Ψ_v along the building height is in a reasonable agreement with the shear distribution trends observed in the NRHA envelopes. Thus, Ψ_v is taken as unity in the bottom half of the building, implying that the story shear higher mode amplification for the stories located between ground and 0.5*H* is equal to the base shear higher mode amplification, A_D. For the stories between 0.5*H*-1.0*H*, Ψ_v is assumed to linearly increase with height until it reaches the maximum value at the top of the building (Ψ_N in Fig. 5[a]). Thus in these levels, the higher modes amplification factor is the product of A_D and a $\Psi_v \geq 1$. For DE level of shaking and the archetype buildings, Ψ_N = 1.4 appears adequate.

7.3 Percentile Modification Factor: χ

The statistical dispersion of shear forces computed is accounted for with a percentile modification factor χ, which is introduced in order to increase the conservatism in estimating the design story shear against larger percentile of the ground motions at a given seismic hazard level. The percentile modification (adjustment) factor χ is related to the coefficient of variation of story shear, c_v, as follows:

$$\chi^{(m)} = 1+m\cdot c_v \tag{4}$$

The term (*m*), appearing both in the superscript on the left hand side and as a multiplication coefficient on the right hand side expression of Eq. 6.11, signifies the number of standard deviations considered above the mean design value. For example, $\chi^{(1)}$ represents the percentile modification factor used when one standard deviation above mean is set as the upper limit on the design quantity, corresponding to the 84th percentile.

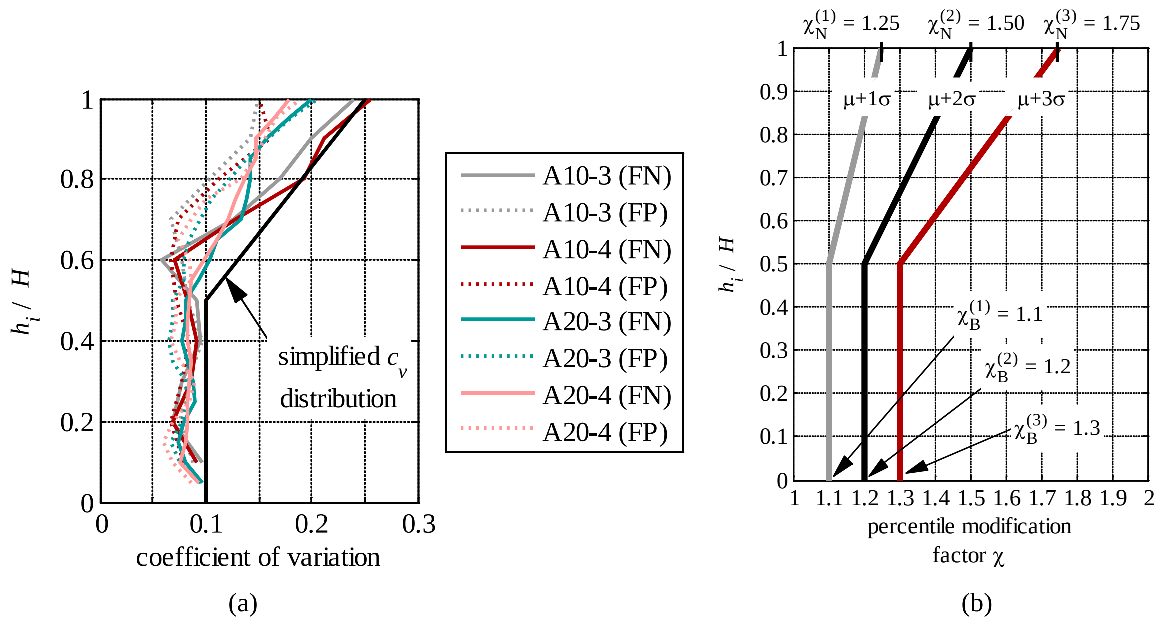


Fig. 7 – (a) Coefficient of variation in story shears; (b) proposed percentile modification factor

Fig. 7(a) plots the coefficient of variation c_v in story shear for the four buildings under DE hazard level. As can be seen, c_v is almost uniform in the lower 60% of the building height for all archetype buildings, tending to a peak value of c_v = 0.10. The c_v increases in an almost linear fashion to a maximum value at the very top level, which varies between 0.20-0.30, dependent on the building and orientation of the ground motion component with respect to the source fault. For simplicity, it is proposed to use c_v = 0.10 in the bottom 0.5H of the buildings, and linearly interpolate between c_v = 0.10 and $c_{v,max}$ = 0.25 for the stories between 0.5H-1.0H, as illustrated in Fig. 7(a). Adopting a piecewise linear function to represent the variation in the c_v along the height of the building, the percentile modification factor χ also follows a piecewise linear function, as shown in Fig. 7(b). The plots in this figure represent the $\chi^{(1)}$, $\chi^{(2)}$, and $\chi^{(3)}$, for the DE levels of shaking, and show the values of those percentile modification factors for a given building as a function of the story height. The shears at the bottom half of the building height are amplified with the constant factor χ_B, while the shears in the upper half of the stories are amplified by factors linearly increasing up the height until they reach the maximum value χ_N.

7.4 Considerations for Exterior Columns: κ_v and Beam Elongation

The ratio of the shear forces in the exterior column to those in the interior column at a given floor depends on the relative stiffness of frame components. It has been shown that the amplification of MRSA-based shears during NRHA is higher in exterior than interior columns [4]. This is partly a result of the uneven variation in stiffness among the columns during the dynamic response, which in a typical building affects mostly the exterior columns, due to overturning. Therefore, exterior column shears need to be amplified by a factor to account for these differences in addition to amplification factors presented in Sections 7.1-7.3.

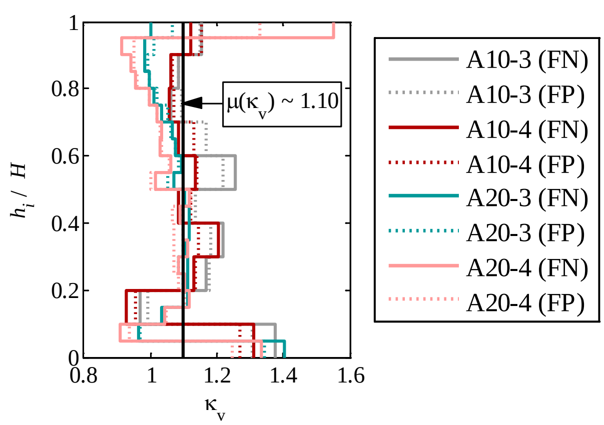


Fig. 8 – Ratio of exterior column shear to interior column shear for the archetype buildings at DE hazard level.

Fig. 8 plots the ratio of the maximum exterior column shear to maximum interior column shear, denoted here κ_v, computed during the NRHA along the height of the four buildings. For DE level of shaking, the average value of κ_v is about 1.10. Note that κ_v at first floor reaches amplitudes 30 to 60% higher than those in the second floor columns. This is a result of the kinematic interaction of the elongating 1st story SMRF beam with the columns, which leads to an increased exterior column shear [7,8,9,10]. Beam elongation impact worsens with more intense shaking, directly owing to the larger amount of deformation demands on the beam which produces more elongation [4]. Because the increase in column shear is a kinematic effect, amplification of elastic shear forces suggested in Sections 7.1-7.3 does not apply for the first story exterior columns. NZS3101 [12] accounts for beam elongation effects and requires that affected columns be designed for the shear demand occurring when the plastic hinges form at both ends of the column, analogous to Method A of ACI 318 (see Section 2.1). Thus, ACI 318 Method A is recommended to use in the base stories of exterior columns. For all other stories, an exterior column shear magnification factor κ_v = 1.2 is recommended in design. For interior columns, κ_v = 1.

7.5 Summary of Proposed Method to Estimate Seismic Shear

This section provides a step-by-step summary in implementing the proposed method of estimating seismic column shear using the factor values for a DE level of shaking (for MCE values, refer to [4]). Prior to implementing this method, beams and columns should be completely detailed for flexure and axial load, and beam transverse reinforcement should be finalized prior to designing for column shear. Based on the column shears obtained from the elastic code-based MRSA procedure, $V_{eMRSA.i}$, the proposed method is as follows:

1. Find system overstrength factor Ω using Eq. 1.
2. Select higher mode amplification factor A_D = 1.25, and select $\Psi_{v,N}$ = 1.4 for DE hazard level.
4. Select a shear response percentile bracket and find the corresponding percentile modification factor at the base (χ_B) and at the top floor (χ_N) using Fig. 7(b).
7. Determine $\omega_{v,b}$ and $\omega_{v,N}$ values based on Fig. 5(a) and calculate the amplification factor at each story.
8. For interior columns, multiply the individual column shears, $V_{eMRSA.i}$, with the corresponding shear amplification factor $\omega_{v,i}$. This is the column design shear, $V_{e,i} = (\omega_{v,i})V_{eMRSA.i}$.
9. For exterior columns, multiply the individual column shears from MRSA, $V_{eMRSA.i}$, with both the corresponding shear amplification factor $\omega_{v,i}$ and exterior column magnification factor κ_v = 1.2. The exterior column seismic design shear is $V_{e,i} = (\kappa_v)(\omega_{v,i})V_{eMRSA.i} = 1.2(\omega_{v,i})V_{eMRSA.i}$

8. Comparison of Design Shear Envelopes

Using the design procedure proposed, the design column shear envelopes were computed for the archetype buildings at DE level shaking intensity and 98th percentile of ground motions. These have been plotted in Fig. 7 along with individual peak column shear envelopes for all analysis runs. For reference, the envelopes developed with methods ACI 318 B1 and B2 and NIST GCR 8-917-1 are also plotted. In addition, the figure shows the design shear envelope calculated with NIST GCR 8-917-1, which has been adjusted for higher mode amplification and percentile modification factors for a more consistent comparison with the proposed method.

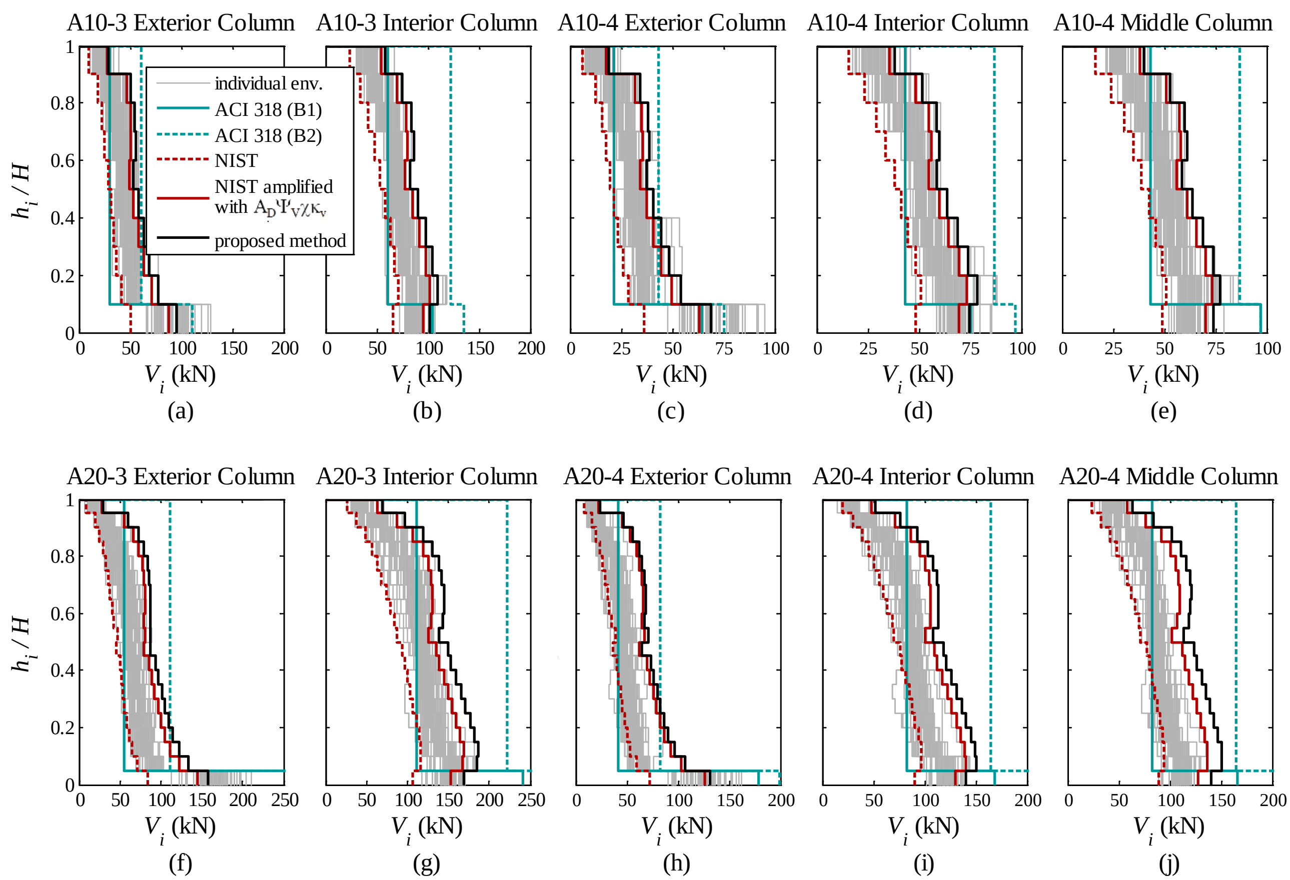


Fig. 7 - Comparison of column design shear envelopes for the buildings investigated.

As can be seen in all plots, the proposed method provides conservative estimate of seismic column shear in most cases. Modified NIST GCR 8-917-1 procedure shows comparable level of conservatism. Both the proposed and the modified NIST methods underestimate the exterior column base shear because they do not account for the impact of beam elongation. As mentioned in Section 7.4, it is recommended to calculate the design column shears in exterior columns at the base using ACI 318 Method A.

9. Conclusions

Seismic shears in columns of SMRF buildings estimated using the common ACI 318 methods based on distribution of the moment imbalance to columns under the assumption that plastic hinging occurs in the beams, does not represent either the spatial distribution or the level of the shear forces that the columns may be experiencing during the earthquake response. The results presented for four archetype buildings show that commonly used and code-compliant design methods can lead to a deficient shear design. Seismic shear forces in columns calculated in a nonlinear response history analyses are better estimated with the methods based on magnifying the shears from the elastic code-prescribed analysis such as the modal response spectrum analysis. It is thus recommended that regardless of the existing code method used in design, the minimum design seismic shear forces in columns not be lower than those calculated by the method summarized in Section 7.5.

It is important to keep in mind that the discussions and the design recommendations presented in this paper are based on the numerical studies of two-dimensional SMRF buildings with regular geometry subjected to unidirectional ground motion acting in the plane of the analyzed SMRFs. The effects of multidirectional ground motion (including vertical) and orientation of the building with respect to fault were not considered in the present dissertation but comprise a topic that calls for further investigation. In addition, the analysis of buildings with irregular elevation and floor plan is strongly recommended. This includes, but is not limited to, the buildings with setbacks, varied bay lengths within a single SMRF, and buildings with torsional eccentricities. A 3-dimensional analysis of SMRF buildings, especially having perimeter frames joined at the corner (i.e., having common corner columns in two orthogonal SMRFs) also warrants further examination.

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