DESIGN ORIENTED SEISMIC PERFORMANCE EVALUATION OF THREE DIMENSIONAL MOMENT RESISTING STEEL FRAMES

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Abstract

A simple proposal for ultimate seismic demand evaluation for two dimensional (2D) moment resisting (MR) steel frames was presented in ref [1]. This paper focus on ultimate seismic demand evaluation for three dimensional (3D) MR steel frames.

The restoring force characteristics are represented by yield polyhedron model, simulating global elastic-plastic behaviour of the frame, instead of a set of member hysteresis based models usually adopted in the inelastic structural analysis. Yield polyhedron is a convex region bounded on all sides by the failure planes and it enables a quick identification of important design conditions in the seismic analysis. For 2D frames, failure modes are enumerated [1] based on engineering judgement.

For 3D frames, a generalised approach for identification of the failure modes is proposed in this paper using stochastic compact procedure. This approach can identify failure modes for irregular and complex structures and assumes no prior knowledge for envisaging the failure modes. The input to the compact procedure is equilibrium equations of the structure.

Random pushover approach [1] is used to choose important failure modes. It is based on FORM and a simple equivalent-static model of the seismic load effects.

Pushover in the direction of the dominant vibration mode (s) is performed and the reliability index of the failure modes represented by equilibrium equations in each step of the compact procedure is computed. The failure modes having reliability index within a range from minimum reliability index are selected and these are further unified by PNET. The design point search to unified failure modes is performed and only those are retained which are traced at a lower or same reliability index. Using these failure modes, yield polyhedron is constructed and the inelastic dynamic analysis is performed.

The validity of the proposed concepts is judged by performing the inelastic time history analysis of a five storey 3D frame to an earthquake ground motion. It is seen that proposed method provide good results compared with member hysteresis based detailed analysis and can capture essential features affecting performance.

References: [1] Khandelwal P, Ohi K, Fang P, A Simple Proposal for Ultimate Seismic Demand Evaluation of Moment Resisting Steel Frames, Journal of Structural and Construction Engineering, Architectural Institute of Japan, No. 545, July 2001

Keywords: Three dimensional, Compact procedure, Random pushover, Seismic performance, Yield polyhedron

1. Introduction

A simple proposal for ultimate seismic demand evaluation for two dimensional (2D) moment resisting (MR) steel frames was presented in ref [1]. This paper focuses on ultimate seismic demand evaluation for three dimensional (3D) MR steel frames.

The restoring force characteristics are represented by yield polyhedron model, simulating global elastic-plastic behaviour of the frame, instead of a set of member hysteresis based models usually adopted in the inelastic structural analysis [1]. Yield polyhedron is a convex region bounded on all sides by the failure planes and it enables a quick identification of important design conditions in the seismic analysis. For 2D frames, failure modes are enumerated based on engineering judgement.

For 3D frames, a generalised approach for identification of the failure modes is proposed in this paper using stochastic compact procedure. This approach is mathematical in nature and assumes no prior knowledge for envisaging the failure modes. The input to the compact procedure is equilibrium equations of the structure. Failure modes are selected using random pushover approach [1] for construction of yield polyhedron and inelastic dynamic analysis is performed in this region.

The validity of the proposed concepts is judged by studying response of a five-storey 3D frame to an earthquake ground motion and results are compared with a member hysteresis based detailed dynamic analysis.

2. Generation of Yield Polyhedron Model

In order to explain construction of yield polyhedron, a two storey single bay frame as shown in Fig. 1 is taken. The failure/collapse modes are shown in Fig. 2. These include local storey collapse mechanisms and combined mechanism. Locations of the plastic hinges are assumed at the top and bottom ends of the columns, which have undergone side sway, and at the floor beams end connected with the side columns. Sequence of yielding before mechanism formation i.e. partial yielding is ignored in the yield polyhedron model. The likely failure modes can be drawn based on the experience or stochastic limit analysis using compact procedure [2, 3,4] may be used for complex situations.

Equation of Motion for MDOF structures for seismic excitation can be written as:

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+\{f\}=-[M]\{1\}\{\ddot{y}\} \tag{1}$$

where, {x}, {y} and {f} are vectors of relative displacement, ground displacement and the restoring force, respectively in original coordinates. Matrices [M] and [C] denotes the mass and damping matrix, respectively. Eq. (1) can be uncoupled by the following classical elastic modal transformation equations:

$$\{x\}=[\phi]\{q\} \tag{2}$$

$$\{f\}=[\phi^T]^{-1}\{r\}=[\psi]\{r\}=\{\psi^1\}r^1+\{\psi^2\}r^2+...+\{\psi^n\}r^n \tag{3}$$

where, {q}, {r} and [ϕ] are modal displacement vector, modal restoring force vector and modal participation matrix, respectively. Matrix [ϕ] is normalised such that the participation factor for any mode is unity. Eq. (2) and (3) represent the displacement and force transformation, respectively. Equation of motion in modal coordinates can be written as:

$$\ddot{q}^j+2h^j\omega^j\dot{q}^j+\frac{r^j}{m^{j*}}=-\ddot{y}, \quad (j=1,2,...,n) \tag{4}$$

where, vectors {h}, {ω} and {m*} represent the modal damping ratio, circular frequency and effective modal mass, respectively. The number of total degrees-of-freedom (d.o.f.) is denoted by *n*.

The f1 and f2 represent the restoring forces (in original space) at the second floor and roof level, respectively. Fig. 3 shows the global yield polyhedron in the original restoring force space. Each column vector of $[\phi]^{T-1}$ represents the lateral loading pattern of each vibration mode and remains constant during the analysis. Equation of motion for seismic excitation is integrated by numerical methods. The locus of restoring forces can move within or tangentially on the boundary of the yield polyhedron, as shown in Fig. 4 for the two-storey frame. The r1 and r2 are components of {r} and represent the restoring forces for the first and second mode, respectively.

Yield polyhedron approach enables a quick examination of the seismic response in terms of the failure modes of the structure, which is quite informative and helpful in understanding critical/ important design situations.

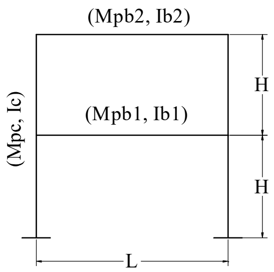
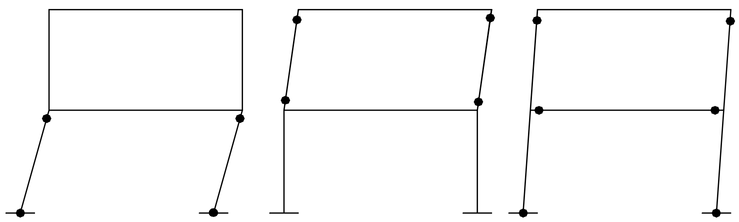


Fig. 1 Two Storey Single Bay Frame



1st Storey Local Collapse 2nd Storey Local Collapse Combined Mechanism

Fig. 2 - Failure Modes of the two-storey frame

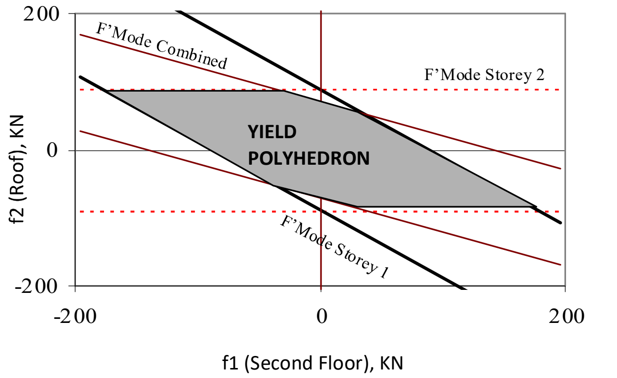


Fig.3 Global Yield Polyhedron in Original Restoring Force Space

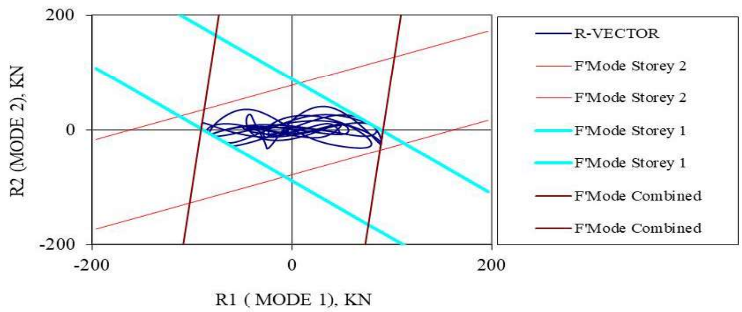


Fig. 4 Locus of Restoring Force in Modal Space

3. Random Pushover Approach

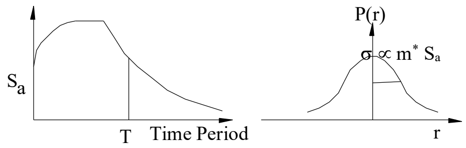
Random pushover approach [1] is proposed to choose important failure modes. It is based on FORM and a simple equivalent-static model of seismic load effects.

3.1 Equivalent random seismic loading model

An equivalent static model of seismic load effects as given by Eq. (3) is used. The {ψ} vector represents the lateral loading pattern and remains constant in the analysis. As the ground motion varies randomly in time, expected value of modal restoring forces is taken as zero and its standard deviation (σ_r^j) is proportional to effective modal mass ($m^{*(j)}$) times the spectral values (S_a^j) read from the acceleration response spectra. The vibration modes are considered independent.

$$E\left(r^j\right)=0\,,\quad E\left(r^i r^j\right)=0\quad (i\neq j)\,,\quad \sigma_r^{\,j}=c\,m^{*(j)}S_a^j \tag{5}$$

where, c is a constant and S_a^j is dependent on period and damping of the jth vibration mode (refer Fig. 5). The value of the constant in Eq. (5) is taken as one-third [5,6] for all the modes.



Acceleration response spectra

Definition of standard deviation

Fig. 5.Random Loading Model

3.2 Reduction of failure modes

Stochastic limit analysis is applied for identification of important failure modes. It is assumed that the loads and resistances are independent of each other. The performance function for a failure mode can be written as:

$$g = g(X) = \sum_{i=1}^{m} M_{pi}\left|\theta_{pi}\right| - \sum_{j=1}^{n} a_j r^j = g_0 - \sum_{j=1}^{n} a_j r^j \tag{6}$$

where, m and n are number of element resistance (moment) capacity and number of vibration modes respectively. M_{pi} is the basic variable of element moment capacity, $\left|\theta_{pi}\right|$ represents the rotation of plastic hinge and a_j is the coefficient of the jth modal restoring force r^j for the failure mode considered. Mean value (μ_g) and standard deviation (σ_g) are given as:

$$\mu_g = \sum_{i=1}^{m} \bar{M}_{pi}\left|\theta_{pi}\right| - \sum_{j=1}^{n} \bar{r}^j a_j \,, \quad \sigma_g = \sqrt{\sum_{i=1}^{m} \sigma_{mpi}^2 \theta_{pi}^2 + \sum_{j=1}^{n} \left(\sigma_r^j\right)^2 \left(a_j\right)^2} \tag{7}$$

where, $\bar{M}_{pi}, \bar{r}^j$ are mean values and σ_{mpi}, σ_r^j are standard deviations of load and resistance capacity, respectively. Eq. (7) represents the uncertainties in load effects and resistance capacity. In the analysis, no uncertainty in the global elastic-plastic behaviour is considered i.e. σ_{mp} is taken as zero. Reliability index (β) is given as μ_g/σ_g.

A lower β indicates higher probability of failure. In our view, failure modes with probability less than about 10% of the most likely (β_{min}) failure mode can be excluded from further analysis. Alternatively, failure modes with $\beta \geq \beta_{min} + \Delta$ can be neglected.

Mutual correlation between the failure modes is considered by PNET [7]. In this approach, the failure modes with correlation coefficients more than a demarcating correlation ρ_0 are unified. The maximum number of failure modes for the dynamic analysis should be limited to the number of vibration degrees of freedom.

4. Identification of failure modes

In the case of two-dimensional frames, the failure modes were enumerated considering that engineering judgement exists for the same [1]. The inelastic behaviour of three- dimensional frames may be relatively complex to understand, compared with two dimensional frames. A designer may not possess adequate experience for enumeration of failure modes. For such situations, a generalised approach for identification of the failure modes is proposed by using the stochastic compact procedure. This approach assumes no prior knowledge for envisaging the failure modes.

The compact procedure was originally proposed [8] for the limit analysis of steel frames or trusses. The stochastic limit analysis of steel frames using compact procedure was proposed in ref. [2,3,4] for static condition. In this study, the methodology of stochastic compact procedure is extended for the identification of important failure modes for the seismic response analysis. The procedure is summarised as follows:

The input to the compact procedure is equilibrium equations of the structure. The interaction effects of the resistance capacity are included in the equilibrium equations by changing the element resistance variables. Pushover in the direction of the dominant vibration mode(s) is performed and the reliability index of the failure modes represented by each line of the equilibrium equations in each step (iteration) of the compact procedure is computed. Once the last iteration of the compact procedure is finished, the failure modes having reliability index $\beta \le \beta_{\min} + \Delta$ are selected and these are further unified by PNET. The design point search to unified failure modes is performed and only those are retained which are traced at a lower or same reliability index. Using these failure modes, global yield polyhedron is constructed and dynamic analysis by the proposed method, referred to as reduced analysis (RA) hereafter, is performed.

5. Illustration of basic concepts – Three dimensional single storey frame

5.1 Description of frame model

A single storey three-dimensional is taken to illustrate the basic principles. The isometric view of the frame is shown in Fig 6. The columns are pinned at base and the floor is considered as rigid. The yielding is considered only for the columns. The storey height (h) and bay width (l) is taken as 4m.

The axial force in the columns is very small compared with the capacity and its effect is neglected in the interaction equation. For the El-Centro (NS), 1940 ground motion, the first and third mode appear dominant. By selecting $\Delta = 1$ and PNET coefficient of 0.85, three failure modes appear. All of the three failure modes are retained for performing reduced analysis (RA). The yield polyhedron model is shown in Fig. 7.

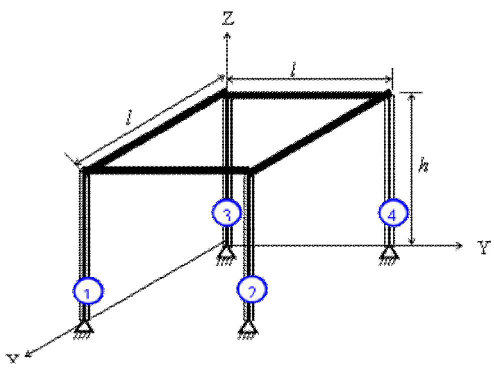


Fig. 6 Single-Storey Three Dimensional Frame

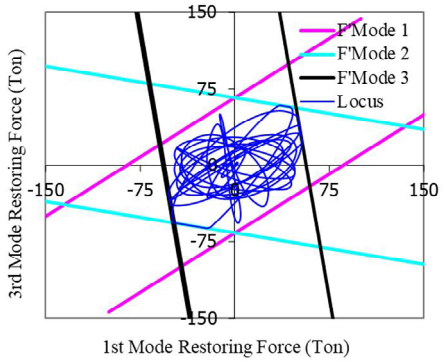


Fig. 7 Global Yield Polyhedron Model

5.2 Verification scheme

In order to validate the results of RA, a member hsyteresis based detailed analysis has been performed using a sophisticated computer program ROMEO [9]. This program uses multi-spring joint to represent the inelastic behaviour of the steel members subjected to axial load and bi-axial bending moments. The major differences in the modelling of inelastic behaviour response by simplified RA and sophisticated ROMEO can be considered as follows:

The partial yielding before mechanism formation is ignored in RA whereas it is accounted in the member based analysis by ROMEO. Online interaction of the axial load and bi-axial moments is simulated by multi-spring joint in ROMEO whereas for simplicity RA doesn't account for online interaction though static interaction effects are accounted.

Fig. 8 represents the modelling of steel member in Romeo wherein each member can be divided into inelastic (multi-spring) zone and elastic zone. The control points for comparison of results have taken as the nodes at the top of the frame as shown in Fig.9.

5.3 Discussion of Results

The single-storey frame is analysed for El Centro (NS), 1940 ground motion with peak ground acceleration of 0.5m/sec2in the Y direction. The results of the reduce analysis are referred as RA whereas the results obtained by sophisticated member based analysis are referred as ROMEO. Fig. 11 and 12 compare the response history of Y and X translation at the top four corners, respectively. Fig. 13 and 14 shows the Y direction base shear v/s average floor displacement for RA and ROMEO, respectively. By studying Figs. 11 through 14, it can be seen that simplified RA is able to trace all major trends of the dynamic response compared with sophisticated ROMEO analysis. The time variations as well as the peak response quantities are well simulated for practical considerations. Similar pattern of the major yielding is traced by RA compared with ROMEO as shown in Figs. 13 and 14. For the practical purpose of performance evaluation, RA provides fairly good simulation of the response quantities.

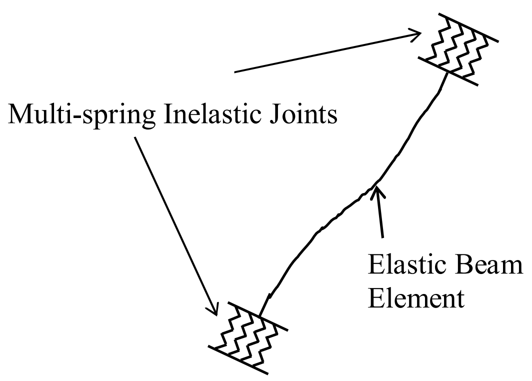


Fig. 9 Modelling of steel member in Romeo

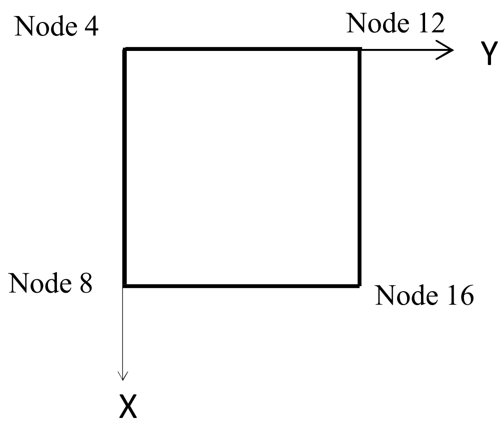


Fig. 10 Nodes at the top of the frame

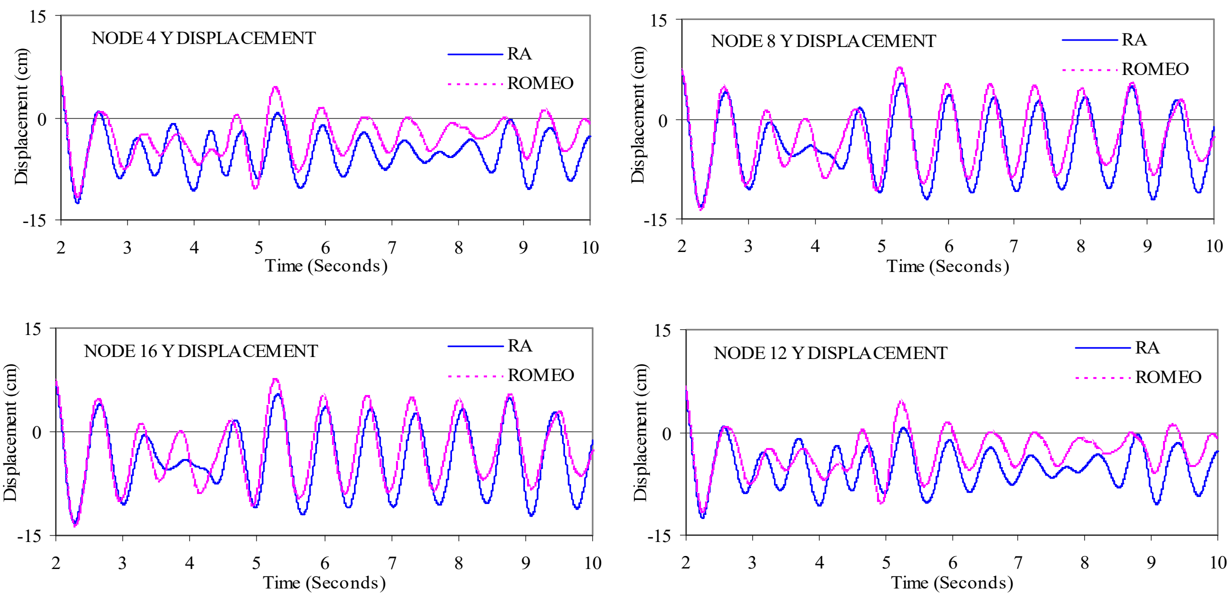


Fig. 11 Response history of Y translation of corner nodes of single-storey frame

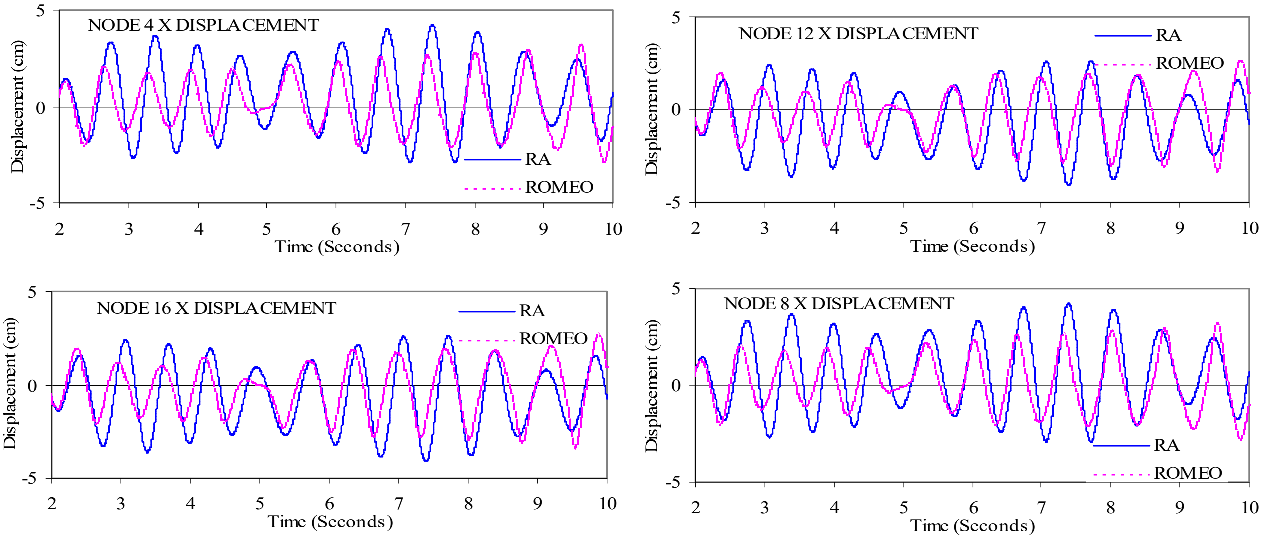


Fig. 12 Response history of X translation of corner nodes of single-storey frame

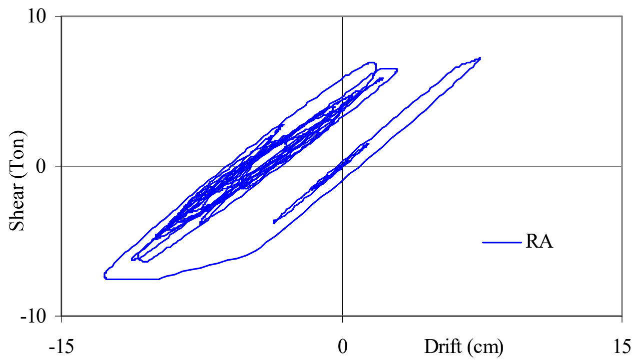


Fig. 13 Y direction base shear v/s average drift (RA)

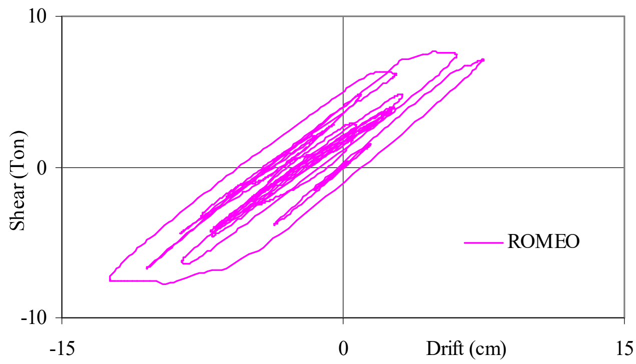


Fig. 14 Y direction base shear v/s average drift (ROMEO)

6. Five storey frame example

6.1 Description of frame model

As a further verification of the proposed reduced analysis method, a five-storey frame is considered. The frame example is taken from ref. [10]. In order to create irregularity, one of the corner columns is removed from the original configuration. The elevation and plan of the frame are shown in Fig.15 and Fig. 16, respectively. The columns and beams are composed of box sections and H sections, respectively. The vibration modes with frequencies less than 10Hz are selected for dynamic analysis. The vibration periods for mode 1 to mode 9 vary from 0.838 secs to 0.123 secs, respectively.

6.2 Identification of failure modes

The stochastic compact procedure, explained earlier, is applied for identification of important failure modes. The plastic moment capacity of the columns is reduced considering the axial force due to dead load and by assuming linear interaction surface. Average level of the axial force at the base of columns is observed about 10% of the axial load carrying capacity of the columns.

For writing equilibrium equations, two translations and one rotation degree of freedom have been considered for each floor. The contribution of column torsion in the floor torsional moment equilibrium is neglected. The lowest reliability index $(\beta_{\min})$ is 1.436. The failure mode equations observed with the reliability index ranging from $\beta_{\min}$ to $\beta_{\min}+1$ have been selected for further filtering/ unification by PNET. A demarcating correlation coefficient ρ_0 of 0.999 was used. Adoption of ρ_0 was done considering that number of failure modes after unification should be of the order similar to the number of the vibration modes. The selected failure mode patterns include combined collapse mechanisms and local collapse mechanisms. Some of the selected failure modes are shown in Figs. 17 through 18.

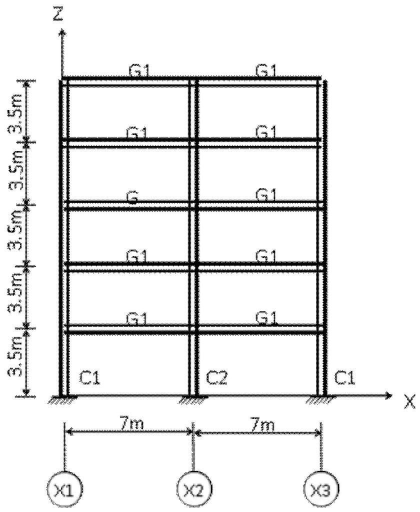


Fig. 15 Elevation of the five-storey frame

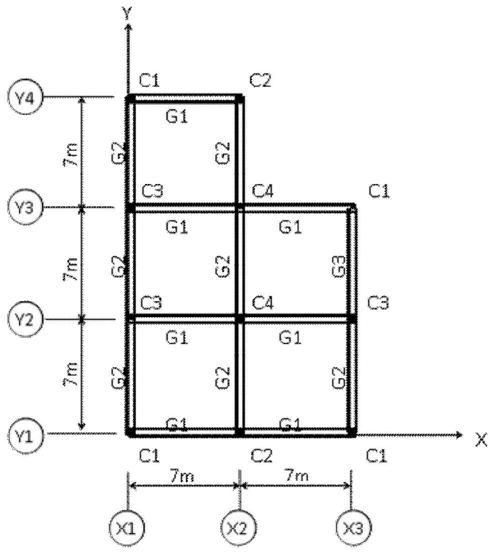


Fig. 16 Plan of the five-storey frame

7. Comparison of results

The five-storey frame is analysed for El Centro (NS), 1940 ground motion with peak ground acceleration of 0.5m/sec2in the Y direction. The results of the reduced analysis are referred as RA whereas the results obtained by sophisticated member based analysis are referred as ROMEO. A response analysis using only the elastic model of the frame has also been carried out and the results have been referred as ELASTIC.

Nodes 16 and 64 are the corner nodes at X = 0.0m, whereas nodes 204 and 306 are the corner nodes at X=14m. Fig 19 compares the response history of Y translation at the four corner nodes of roof. Fig. 20 compares the average Y direction drift for the various storeys. Fig. 21 compares the Y direction overturning moments at the bottom level of each storey.

By studying Figs. 19 through 21, it is observed that the response is primarily translational in Y direction and elastic response is more than inelastic response. It can be seen that simplified RA is able to trace all major trends of the dynamic response compared with sophisticated ROMEO analysis. The peak to peak displacement/ drift response of RA is close to peak to peak of ROMEO, with little shifts appearing in some cases. Fig. 21 shows nearly same pattern and order for Y direction overturning moments for RA and ROMEO. In general, the time variations as well as the peak response quantities are fairly simulated by RA. For the practical purpose of performance evaluation, RA provides fairly good estimation of the response quantities.

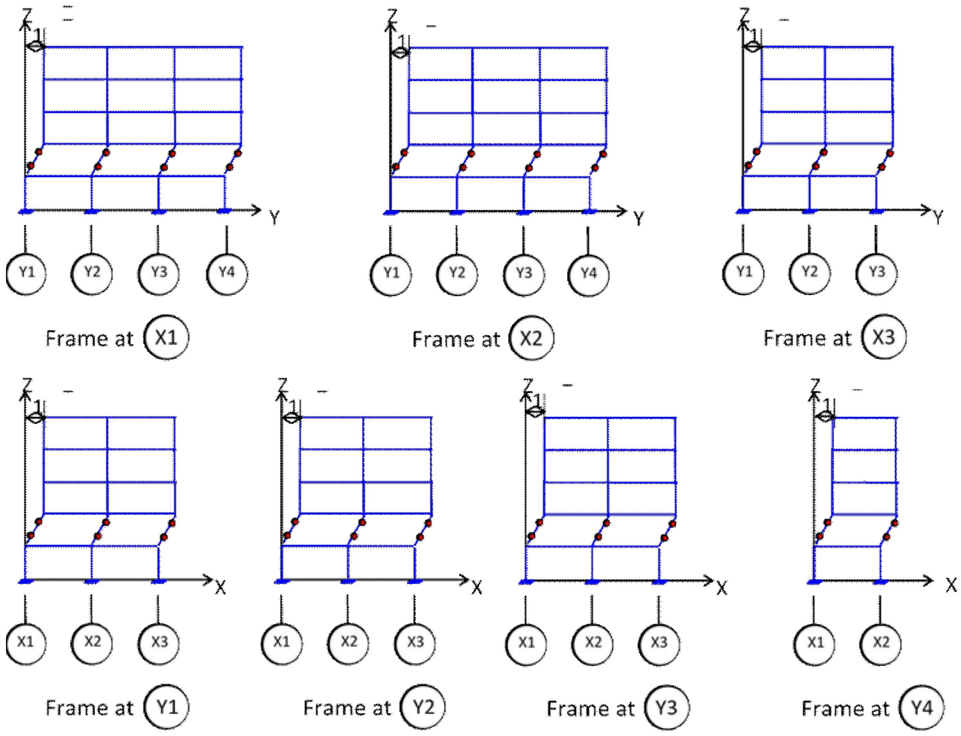


Fig. 17 Failure Mode $\beta_4 = 1.910$

Second Storey Local Collapse Mode - Translational in Y and X direction

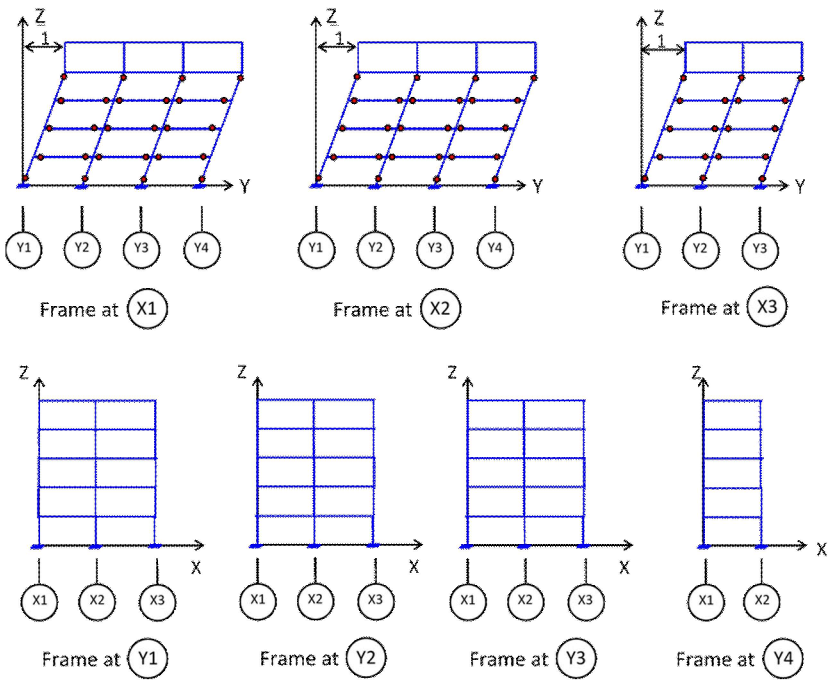


Fig. 18 Failure Mode $\beta_1 = 1.4359$

Collapse Mode - Translational in Y direction

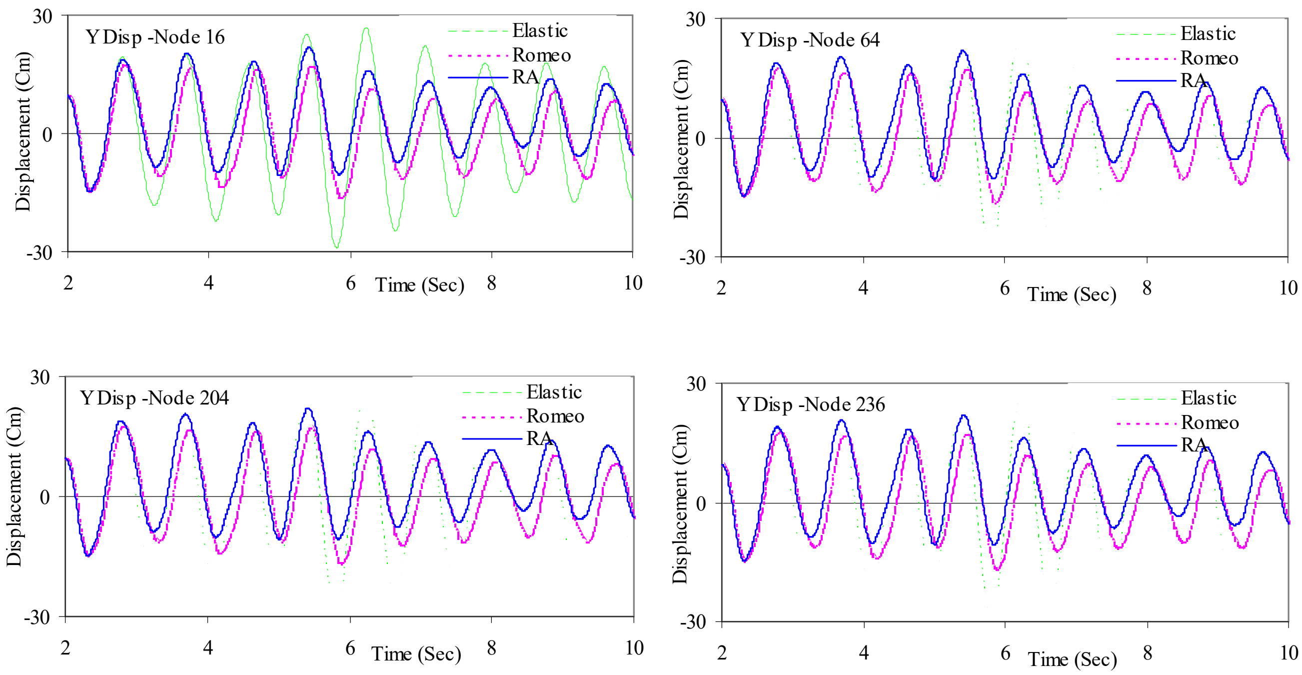


Fig. 19 Y translation at corner nodes of roof

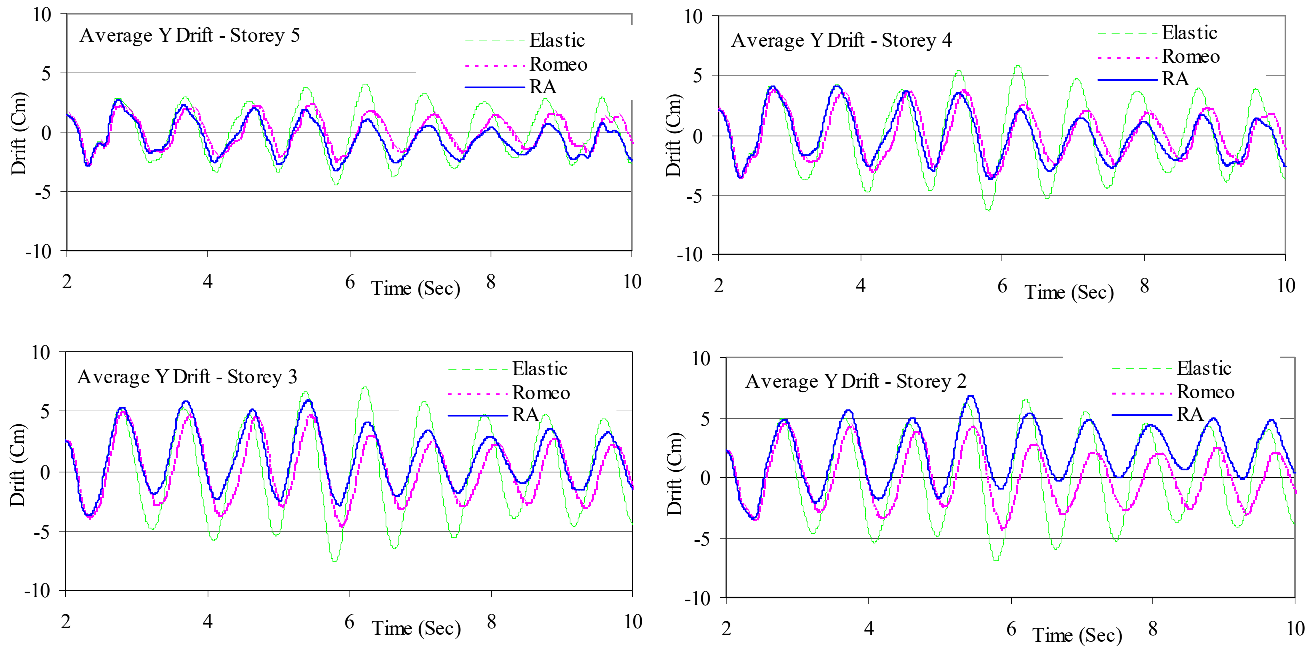


Fig. 20 Average Y drift for various storeys

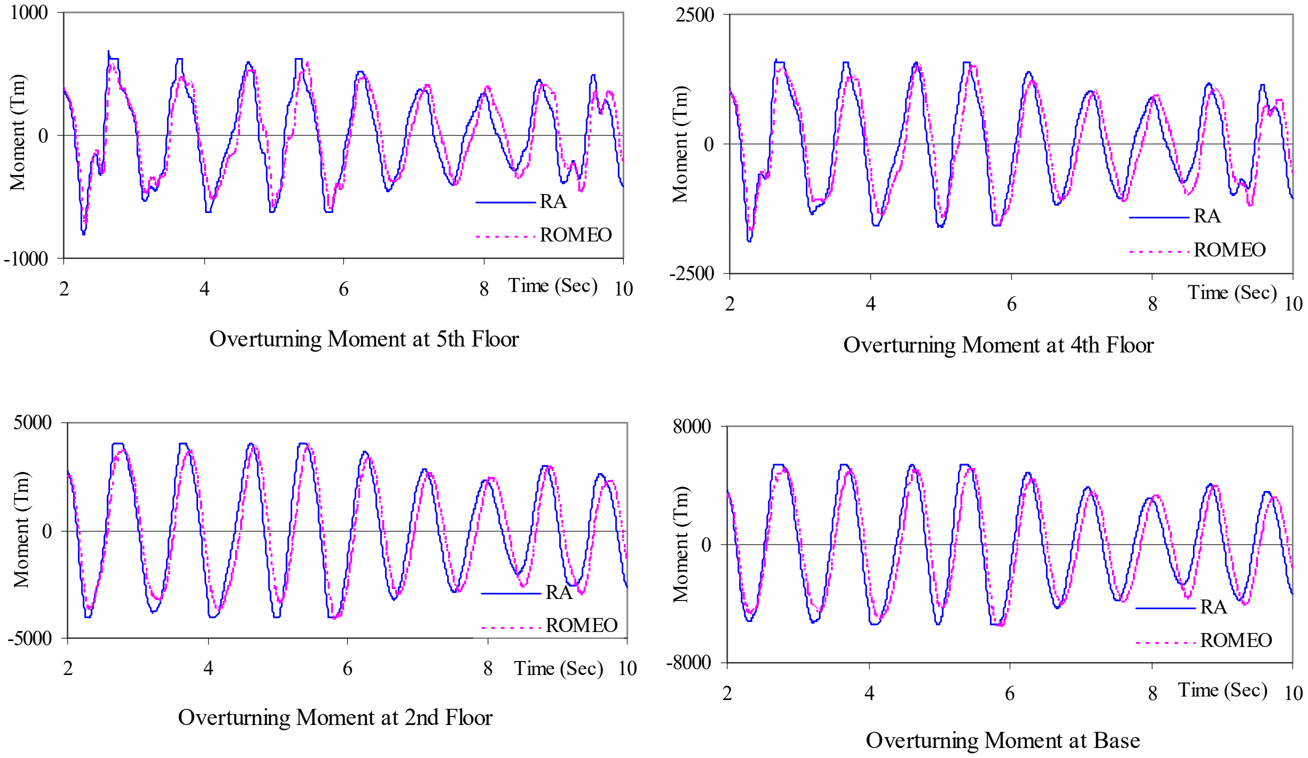


Fig. 21 Y direction overturning moments at bottom level of storeys

8. Conclusions

In this paper, the feasibility of applying the reduced analysis for seismic-demand evaluation of three-dimensional moment resisting steel frames is examined. The identification of the important failure modes is done by applying stochastic compact procedure to seismic analysis. The stochastic compact procedure begins from the equilibrium equations of the structure and uses linear programming techniques. As the proposed approach is mechanical in nature and assumes no prior knowledge for envisaging the failure modes, design engineer may find it attractive to apply for irregular and complex structures.

The basic principles of three-dimensional reduced analysis have been explained with a single-storey frame example and the application has been studied for a five-storey frame. Prediction of the response quantities is seen as fairly good, for practical applications, compared with a sophisticated member based detailed analysis.

It may be pertinent to mention that the magnitude and complexity of a three-dimensional member based detailed analysis is many fold compared with the reduced analysis. Comparatively, the reduced analysis provides a simple and fast techno-economical solution, which can capture the essential features affecting the performance. It is expected that the proposed ideas will find favour with the structural designers.

9. References

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