Energy Formulation for Seismic Collapse Assessment of RCC Structures: Improvements in Performance Design

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Abstract

Quantification of structural collapse is one of the key components of performance based earthquake engineering (PBEE) design. The point of dynamic structural collapse due to earthquakes is defined by FEMA (Federal Emergency Management Authority) guidelines, which is based on parameters approaching subjective threshold values. It is argued that these thresholds may vary on case-by-case basis and therefore they do not sufficiently characterize the dynamic instability and subsequent collapse. Alternatively, energy-based formulation has surfaced as a more robust and generic approach that tracks variation between the incident seismic energy and the energy dissipated by the structural system.
In this paper, energy based formulation has been employed to predict the dynamic structural collapse for reinforced concrete structures, which are especially prone to gravity-load collapse mechanism. A computational frame model is developed in OpenSees (open system for earthquake engineering simulation) and incremental dynamic analysis (IDA) is performed to establish the response parameters at structural collapse. The response parameters found based on the energy-based description of collapse are then compared to the response parameters obtained using the conventional approach. It has been found that the energy-based formulation is more robust even in the prediction of the gravity-load collapse.

Keywords: Performance based earthquake engineering, dynamic structural collapse, energy formulation.

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1. INTRODUCTION

Prediction of structural collapse has been a problem of interest in last two decades especially after performance-based-earthquake-engineering (PBEE) was introduced (Deierlein et al., 2003, Ghobarah, 2001, Cornell and Krawinkler, 2000, Bozorgnia and Bertero, 2004, Zareian et al., 2010, Deierlein, 2004). Structural systems subjected to dynamic forces such as seismic excitation need designs based on robust understanding of their response. This is imperative as an appropriate margin of safety is to be ensured against collapse. Unfortunately, it is extremely difficult to predict the accurate point of collapse and the corresponding structural collapse capacity owing to the fact that it is a highly complex nonlinear phenomenon. As per the guidelines in practice, structural collapse is defined as the point when the structural system is unable to sustain its gravity loads (Engineers, 2014, Venture and Committee, 2000b). These design guidelines have a ‘physical’ parameter-based approach to quantify this point of collapse, which targets exceedance of threshold values that are defined subjectively (Deniz et al., 2017). Therefore, there is a huge amount of uncertainty in terms of the collapse definition itself, especially when it takes place through different mechanisms. This is because the assumed threshold values are based on experimental observations, which represent a particular type of collapse mechanism. Further, understanding of this highly nonlinear phenomenon in a variety of cases need thorough experimental investigations of the near collapse stage. Although, there exist some benchmark experiments in the literature (Elwood and Moehle, 2003, Lignos, 2008, Kanvinde, 2003), they are still far from being enough to characterise every structural collapse mechanism effectively (Deniz, 2015, Wu et al., 2009). It is therefore essential to establish a generalized framework which explains the dynamic structural collapse for different types of collapse mechanisms. Recently, an energy-based approach has been introduced that tracks the input energy of the system (due to seismic excitation) with respect to the gravitational energy which appears due the vertical displacements (Deniz et al., 2017). The collapse is defined based on the point when the gravitational energy of the system exceeds the input seismic energy. This approach seems intriguing, as it does not involve any physical parameter based definition of collapse, which is likely to change on case-by-case basis. However, the energy-based assessment of structural collapse has only been identified to work when sidesway collapse takes place by ductile mechanism in steel structures (Deniz et al., 2017). It is therefore necessary to understand the credibility of the method when structural collapse takes place by brittle gravity load collapse mechanism in reinforced concrete structures.

Gravity load collapse is a rarely studied mechanism, however, there are a number of studies and experiments that provide insight of the phenomenon which can possibly cause the onset of such collapse mechanism. Gravity load collapse usually takes place when the columns or the vertical supporting members fail in shear followed by the axial failure (Elwood and Moehle, 2003). Therefore, the phenomenon of shear failure of vertical supports can lead to the onset of gravity load collapse. Several studies have been carried out to understand the shear and axial failure of the columns (Elwood and Moehle, 2003, Sezen and Moehle, 2004) and empirical models based on these studies aim to quantify drift at which the failure takes place. The models are particularly useful to simulate the structural damage and carryout the collapse simulations. Nevertheless, although these models may appear extremely useful to determine a local failure, they do necessarily represent partial or total collapse of a structure. On the other hand, in order to carryout performance based design of structures, it is essential to quantify performance measures such as ‘prevention of collapse’ (FEMA-356, 2000) which are represented by horizontal drift under seismic excitations. Simultaneously, it can also be understood that such parameters (such as drift capacity) are not generic and are bound to change as the structural configuration varies. Therefore, it is imperative to understand the collapse based on something which can explain every structural phenomenon irrespective of the structural system, structural configuration or the material with which a structure is built. Structural energy can be regarded as the global indicator of the state (Deniz et al., 2017). However, there are very few studies that are based on energy parameters for the identification of collapse. Hence, the current research focuses to provide a greater insight into the method of predicting structural collapse using energy based formulation by exemplifying gravity load collapse mechanism.

2. MATHEMATICAL BASIS

The energy-based formulation for assessing the collapse capacity tracks the variation of the input seismic energy into the structural system with respect to the energy dissipated by the gravitational forces acting on the structural components (Deniz et al., 2017). Mathematically, the energy balance equation (Uang and Bertero, 1990) needs to hold when the structure is under motion due to the seismic forces. However, when the structure begins to collapse, the energy balance is violated (Deniz et al., 2017). To enunciate this mathematically, the energy balance equation can be derived from the basic structural dynamics equation of motion (consolidating the motion in all the degrees of freedom in one equation), which is given as:

$$M\ddot{u}(t) + C\dot{u}(t) + K(t) \cdot u(t) = MT\ddot{u}_{ext}(t) \qquad (1)$$

where, M is the mass matrix, C is the damping matrix, K is the equivalent stiffness matrix, T is the transformation matrix mapping all the ground accelerations $\ddot{u}_{ext}(t)$ to the masses; $u(t)$, $\dot{u}(t)$ and $\ddot{u}(t)$ are the relative nodal displacement, velocity and acceleration respectively with reference to the fixed base of the structure. Since the equation (1) consolidates all the degrees of freedom in one equation, the dimension of the matrices M, C and K are rNxrN, where r is the total number of lumped mass nodes and N is the number of degrees of freedom. Equation (1) can then be integrated as the structure displaces from $u(0)$ to $u(t)$:

$$\int_{u(0)}^{u(t)} \big(M\ddot{u}(t)\big) \cdot du + \int_{u(0)}^{u(t)} \big(C\dot{u}(t)\big) \cdot du + \int_{u(0)}^{u(t)} \big(K(t) \cdot u(t)\big) \cdot du = -\int_{u(0)}^{u(t)} \big(MT\ddot{u}_{ext}(t)\big) \cdot du \qquad (2)$$

Equation (2) represents relativistic energy balance equation of the structure under motion (Uang and Bertero, 1990). This can also be represented as:

$$E_K + E_D + E_S = E_I \qquad (3)$$

where, the relative kinetic energy, $E_K = \int_{u(0)}^{u(t)} \big(M\ddot{u}(t)\big) \cdot du$. The damping energy, $E_D = \int_{u(0)}^{u(t)} \big(C\dot{u}(t)\big) \cdot du$. The strain energy, E_S which is the sum to elastic strain energy E_E and the hysteresis or the plastic strain energy E_P, $E_S = E_E + E_P = \int_{u(0)}^{u(t)} \big(K(t) \cdot u(t)\big) \cdot du$. The input energy emerging from the inertial forces due to seismic acceleration and gravity (Deniz et al., 2017), $E_I = -\int_{u(0)}^{u(t)} \big(MT\ddot{u}_{ext}(t)\big) \cdot du$. The input energy is the relative energy which is fed into the structural system by the external force which is an inertial force due to seismic acceleration $\ddot{u}_{EQ}$ and gravity. Therefore, the acceleration vector $\ddot{u}_{ext}(t)$ can be represented as:

$$\ddot{u}_{ext}(t) = \begin{bmatrix} \ddot{u}_{EQ}(t) \\ g \end{bmatrix}$$

where, $\ddot{u}_{EQ}(t)$ is the horizontal acceleration due to seismic excitation and g is the constant acceleration due to gravity. The transformation vector is given as (Deniz et al., 2017):

$$T = \begin{bmatrix} 100100100\ldots \\ 010010010\ldots \end{bmatrix}^T$$

Since, the acceleration is acting in two directions, the dimensions of the transformation vector is rNx2 and each column of the transformational vector represent a direction of action of acceleration (earthquake in horizontal and gravity in vertical). Therefore, by multiplying the transformation matrix to the acceleration vector $\ddot{u}_{ext}(t)$, the input energy can be segregated into $E_I = E_{EQ} + E_G$, where the earthquake input energy is given by, E_{EQ}=$-\int_{u(0)}^{u(t)} \left(MT\ddot{u}_{EQ}(t)\right) \cdot du$ and the gravitational energy is given by, $E_G = -\int_{u(0)}^{u(t)}(MTg) \cdot du$

Now, rearranging equation (3), it takes form (Deniz et al., 2017):

$$E_K + E_D + E_S - E_G = E_{EQ} \tag{4}$$

Or,

$$E_{struct} - E_G = E_{EQ} \tag{5}$$

where, E_{struct} is the combination of energy possessed and dissipated by the structure. Initially, when the P-Δ effects are minimum, the input energy fed into the system by earthquake acceleration is balanced by the energy possessed and dissipated by the structural system. At this point, the energy balance equation takes the form:

$$E_{struct} \approx E_{EQ} \tag{6}$$

However, when the structure begins to deform significantly and starts deteriorating, P-Δ effects become significant, thereby increasing the destabilising effect or the gravitational energy. The energy equation then takes the form of equation (5). The dynamic structural collapse due to the instability of the structure can then be identified when the energy balance given by equation (5) is violated. In other words, the gravitational energy exceeds the earthquake input energy (E_{EQ}), discounting the energy possessed by the structure given by equation (7). To simplify this, collapse can be defined when the gravitational energy exceeds the earthquake input energy, given by equation (8).

$$-E_G > E_{EQ} - E_{struct} \tag{7}$$

$$\left| E_G \right| > \left| E_{EQ} \right| \tag{8}$$

This can also be interpreted as the work done by the gravity forces is more than the energy fed into the system. Nevertheless, it should be noted that this type of failure is caused by sidesway collapse mechanism where the P-Δ effects play a vital role. On the other hand, gravity load collapse takes place when a section of a structural element fails in shear or fails axially. It is only after this point that the P-Δ effects come into picture. Moreover, this method defines global collapse of a structural system and cannot necessarily identify partial or local collapse. Since, in the current study a partial gravity load collapse is investigated, a new collapse criterion is established which caters to the identification of partial or local collapse.

A partial collapse within a structural system can be identified by comparing the rate at which the input seismic energy is fed into the system versus the rate at which the gravitational energy builds up. This is given by equation (9):

$$\frac{\partial}{\partial \mathrm{t}} \left| E_G \right| \gg \frac{\partial}{\partial \mathrm{t}} \left| E_{EQ} \right| \tag{9}$$

The above equation can be interpreted as a necessary condition for the occurrence of instability and a subsequent collapse, which means that the destabilizing energy builds up much faster than the energy fed into the system. Thus, partial collapse can be quantified once this *necessary criterion* is met. However, this equation cannot explain global collapse. A global collapse would take place when the criterion given by equation (8) being met. Hence, equation (8) can be regarded as the *sufficient criterion* for collapse.

3. COMPUTATIONAL MODELLING

The computational model employed for the analysis is based on the shake table experiments performed by Elwood and Moehle (2003). This is a 2-bay single story reinforced concrete frame model. The central column is detailed to be vulnerable in shear failure and a subsequent gravity load failure which thereby facilitates the mechanism of gravity load collapse. Figure 1 shows the isometric view of the computational model developed in ETABS, a commercial structural engineering program, to understand the preliminary behaviour of the structure. Figure 2 shows a detailed engineering sketch of the experimental model which is adapted from Elwood and Moehle (2003). Since, the ETABS program is not capable of carrying out sophisticated nonlinear collapse simulation, a nonlinear model is developed in OpenSees (open system for earthquake engineering simulation) program (McKenna et al., 2007) as a 'tcl' programming language code (Mazzoni et al., 2006). It should be noted that the dimension of the structure is close to a 'real-life' building structure, therefore, it becomes overly computational intensive to carry-out the collapse simulations using traditional continuum finite element analysis. It is for this reason, OpenSees program was chosen as it supports formulation of nonlinear fiber-beam-column 'macro' finite elements (Spacone et al., 1996) which facilitate efficient computations with a reasonable accuracy. Nevertheless, these fiber based nonlinear beam-column elements are incapable of capturing the highly nonlinear dynamic collapse phenomenon. To mitigate this limitation, 'critical' constitutive relationships that defines nonlinearity and damage in the reinforced concrete section were incorporated in the computational model as analytical springs. These springs were calibrated to an empirical model (see section 3.3) proposed by Elwood (2004) using a database of results from several other shake table experiments. This empirical model predicts shear and axial failure when the reinforced concrete element attains a particular threshold drift given by the failure surface. Hence, the final model is a hybrid analytical-computational model.

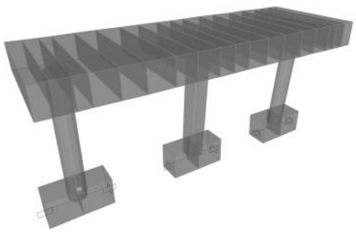


Figure 1: Isometric view of the computational model developed in ETABS

Initially, the computational model developed is verified by comparison with results from the shake table experiments. The time period of the model is 0.28s which is quite close to 0.3s observed from the experiment. Although the dynamics of the computational model are close enough to that observed in the shake table experiments, there are some differences in the simulation results and the computational model does not exactly behave as observed in the experiment. This may be attributed to the complexity in the shear and axial failure in the reinforced concrete section and the empirical model used to model springs may not be highly accurate. Nevertheless, since this study aims to comment on the collapse capacity of the structure, a reasonable validation of the computational model suffices for the understanding of the episodes of collapse.

3.1 Fiber based finite element model with analytical springs

The collapse was identified by the failure of the columns under the seismic ground motion as emulated by the shake table experiment. To address the modelling of damage, the computational model incorporates analytical springs at the ends of the column elements where the damage is localised (Elwood and Moehle, 2003). The analytical springs can be regarded as empirical damage models. These damage models were developed by Elwood (2004) based on the observations from a database of results from several shake table tests. Since, the central column is detailed to fail in shear and to subsequently fail axially, the top end spring is modelled as a combination of horizontal and vertical springs which are based on shear and axial failure models respectively. These models identify the failure when the structure at a particular loading attains a threshold of seismic drift defined by the failure surface of the empirical model (see section 3.3). It is for this reason, the springs are required to be calibrated again as the section geometry changes. Apart from this top end spring, at the bottom end of the central column and at either ends of the outer columns, a rotational spring is used which captures the reinforcement slip in the columns. All of the analytical springs or the spring elements are modelled as zero length elements between the two coinciding nodes. Therefore, all nodes where the springs are located are basically two coincident nodes connected by these zero length springs.

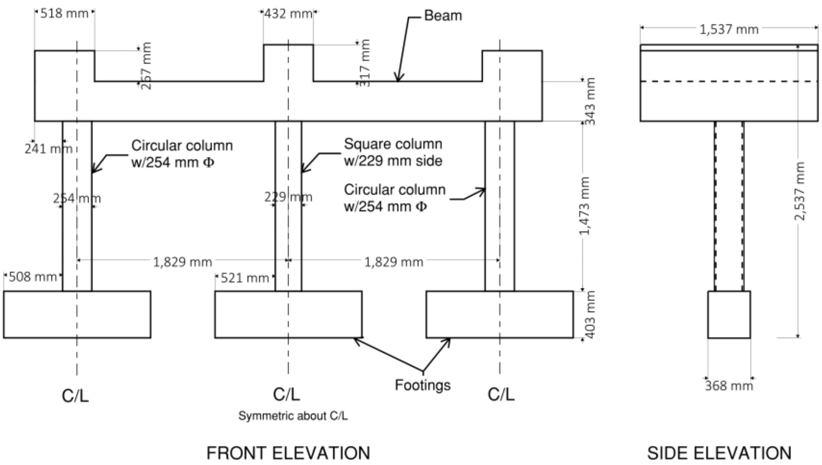


Figure 2: Engineering sketch of the structure (Elwood and Moehle (2003))

3.2 Gravity loads on the structure

The structure is lumped with masses on the beam nodes shown in the sketch of the computational model (figure 3). The total load on all of the beam nodes is 298.03 kN. Further, to ensure the axial failure of the column section, an additional 177.92 kN of prestress was applied over the central column. Therefore, a total load of 475.95 kN was applied on the structure.

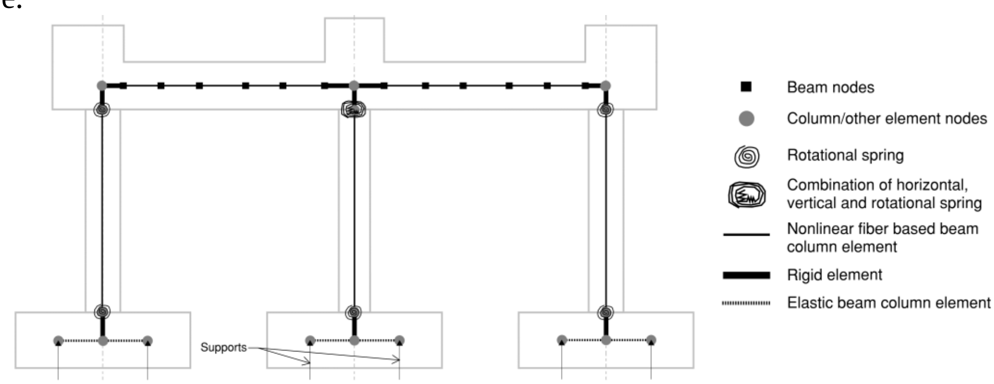


Figure 3: Sketch of computational model in OpenSees (Elwood and Moehle (2003))

3.3 Material characteristics, constitutive model & empirical failure model for springs

The material characteristics are exactly the same as used for the shake table experiment and can be referred from Elwood and Moehle (2003). The 'crtitical' constitutive models used for analytical springs are moment curvature relationships which are obtained by section yielding analysis of the reinforced concrete section by Elwood and Moehle (2003). The empirical models which emulates the damage (failure at thresold drift) can be given as (Elwood, 2004):

The shear failure model defines shear failure at a thresold drift given by expression (Elwood, 2004):

$$\frac{\Delta_s}{L} = \frac{3}{100} + 4\rho'' - \frac{1}{40}\frac{\upsilon}{\sqrt{f_c'}} - \frac{1}{40}\frac{P}{A_g f_c'} \geq \frac{1}{100}$$

where, Δ_s is the drift at shear failure, L is the length of the reinforced concrete element, ρ'' is the transverse reinforcement ratio, f_c' is the characteristic compressive strength of the concrete (in MPa), P is the axial load on the column, υ is the nominal shear stress (in MPa) and A_g is the gross cross sectional area of the reinforced concrete section. The plot of axial load (P) and thresold shear drift ratio ($\frac{\Delta_s}{L}$) will form the empirical shear failure surface.

The axial load failure model defines axial failure at a thresold drift given by expression (Elwood, 2004):

$$\frac{\Delta_a}{L} = \frac{4}{100}\frac{1 + (\tan\theta)^2}{\tan\theta + P(\frac{s}{A_{st} f_{yt} d_c \tan\theta})}$$

where, Δ_a is the drift at axial failure, θ is the crtical crack angle from the horizontal (assumed 65º), d_c is the distance from the centerline of the core of the column to the centerline of th ties (transverse reinforcement), s is the spacing of the ties (transverse reinforcement) f_{yt} is the is the yield strength of the ties (transverse reinforcement), P is the axial load on the column and A_{st} is the area of the transverse reinforcement. The plot of axial load (P) and thresold axial drift ratio ($\frac{\Delta_a}{L}$) will form the empirical axial load failure surface.

4. GROUND MOTION CHARACTERISTICS

The structure was subjected to a time history of the shake table response, which was in turn obtained by subjecting shake table to Chile Valparaiso 1985 earthquake *(Elwood and Moehle, 2003)*. The ground record was taken from the station at Vina del Mar, Chile. The shake table motion time history has peak ground acceleration of 0.797g. Figure 4 shows the time history of shake table response as a fraction of 'g' (acceleration due to gravity). The computational model of the structure is exposed to this time history for a duration of 60 seconds.

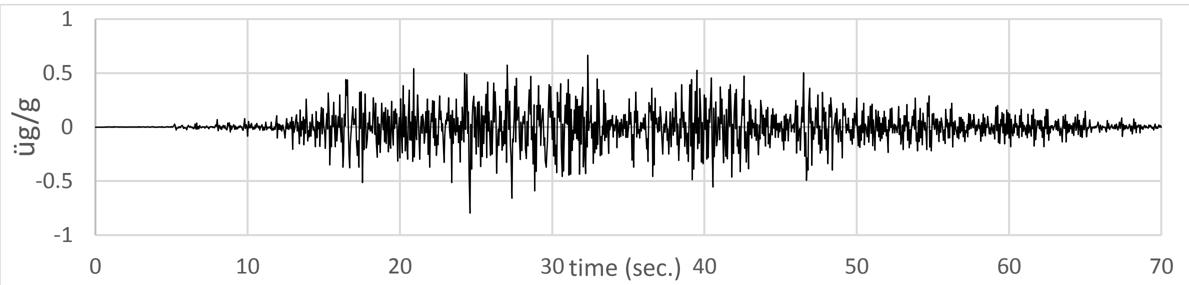


Figure 4: Shake table response history to VinadelMar station record (1985, Chile) (adapted from Elwood and Moehle (2003))

5. INCREMENTAL DYNAMIC ANALYSIS (IDA) & PERFORMANCE MEASURES

Incremental dynamic analysis (Vamvatsikos and Cornell, 2002) is the current state-of-the-art method to quantify the collapse capacity of a structural system subjected to the earthquake ground motions (Venture and Committee, 2000a, Venture and Committee, 2000b) and US Federal Emergency Management Agency (FEMA) has established it as a state of art method of analysis for global structural collapse. It is a parametric analysis where the ground motion records are scaled to different intensity levels and are then used to perform a series of typical nonlinear dynamic analysis. The point of collapse is determined by plotting intensity measure (IM) parameter against a response parameter which is the damage measure (DM). Global collapse is defined when this plot attains a near flat shape (Venture and Committee, 2000a).

Vamvatsikos and Cornell (2002) have demonstrated how IDA can be used within the PBEE (performance based earthquake engineering) design framework proposed by PEER (Pacific Earthquake Engineering Centre) (Cornell and Krawinkler, 2000, Moehle and Deierlein, 2004). Since, PBEE framework demands quantification of IM and DM, IDA serves as the best-suited method for performance evaluation of structures (Krawinkler and Miranda, 2004, Vamvatsikos and Fragiadakis, 2010). However, researchers have still questioned if IDA represents the 'exact' collapse stage, since the point of structural collapse is subjectively defined in the current guidelines which utilises IDA (FEMA-356, 2000, Venture and Committee, 2000a, Venture and Committee, 2000b). Very recently, Deniz et al. (2017) have argued that there is a reasonable difference in the collapse stage as predicted by the subjective FEMA guidelines with respect to the 'reality'. As per the guidelines of FEMA-356 (2000), the structural performance level of collapse prevention for a concrete structural frame is subjectively prescribed as 4% of transient or permanent drift. This is bound to vary as the structural configuration of the frame changes and this is exactly where the energy based formulation comes into picture. Quantifying the IM and DM based on the observation of energy transitions in the structure provides an effective way to establish such parameters which thereby objectively defines collapse capacity of the structure. In this study, a single record IDA is performed based on the ground motion described in section 4, so that a comparison can be made between the two approaches of defining structural collapse.

6. RESULTS

Structural collapse is investigated by two approaches as mentioned above:

6.1 FEMA guidelines based evaluation of capacity

The time history as showin in figure 4 is scaled to intensity levels from 0.1 to 1.2 and 1.4 with the incremental scaling factor being 0.1. This way a series of thirteen nonlinear dynamic analysis were performed and the peak drifts were observed. Figure 5 shows the IDA curve with y-axis being acceleration or the IM (intensity measure) and x-axis is the peak drift ratio or the damage measure (DM). The peak ground motion recorded from the original time history is 0.797g and with max intensity of 1.4, the peak ground acceleration is 1.115g. The dashed line is critierion for collaspe prevention for a framed concrete structure, that is, when the structure attains 4% drift (FEMA-356, 2000). However, first it can be seen from the IDA curve that the structure can perform upto large drifts of about 9%. Secondly, there was no failure observed in the structure at 4% drift which correspond to an intensity of about 0.6 times the actual earthquake intensity. Certainly, this means that the structure has reserve capacity which is not accounted for and 4% value for drift to prevent collaspe is a conservative estimate.

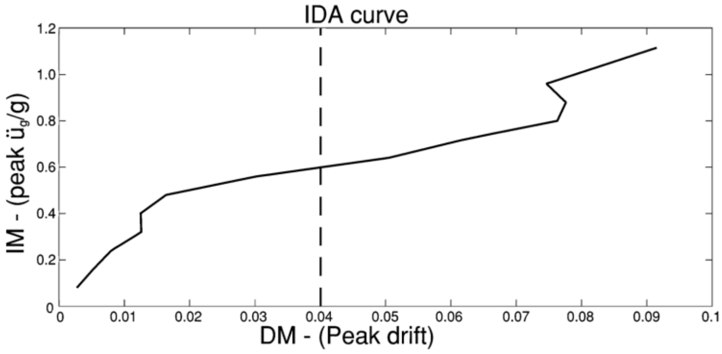
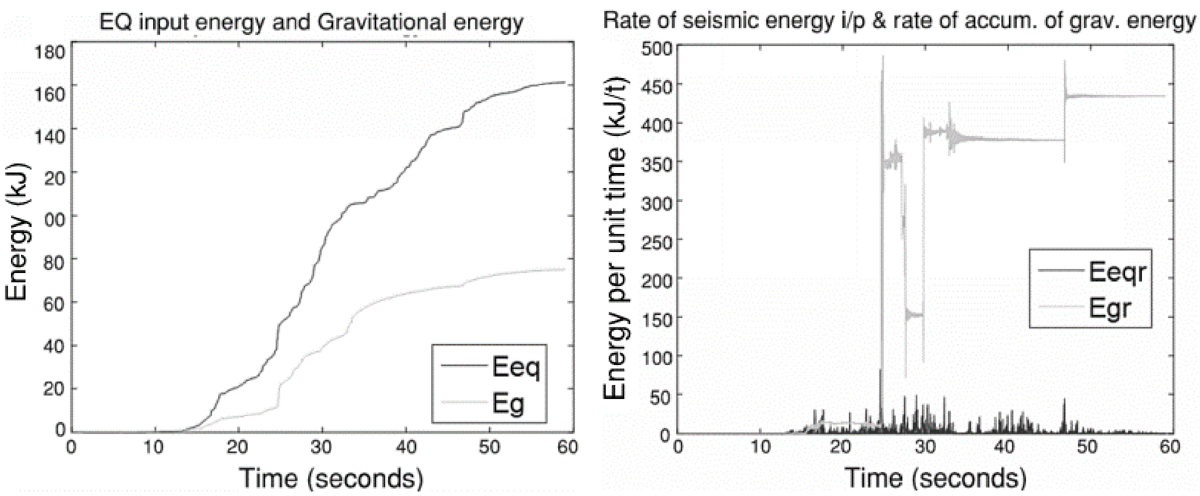


Figure 5: Incremental Dynamic Analysis curve

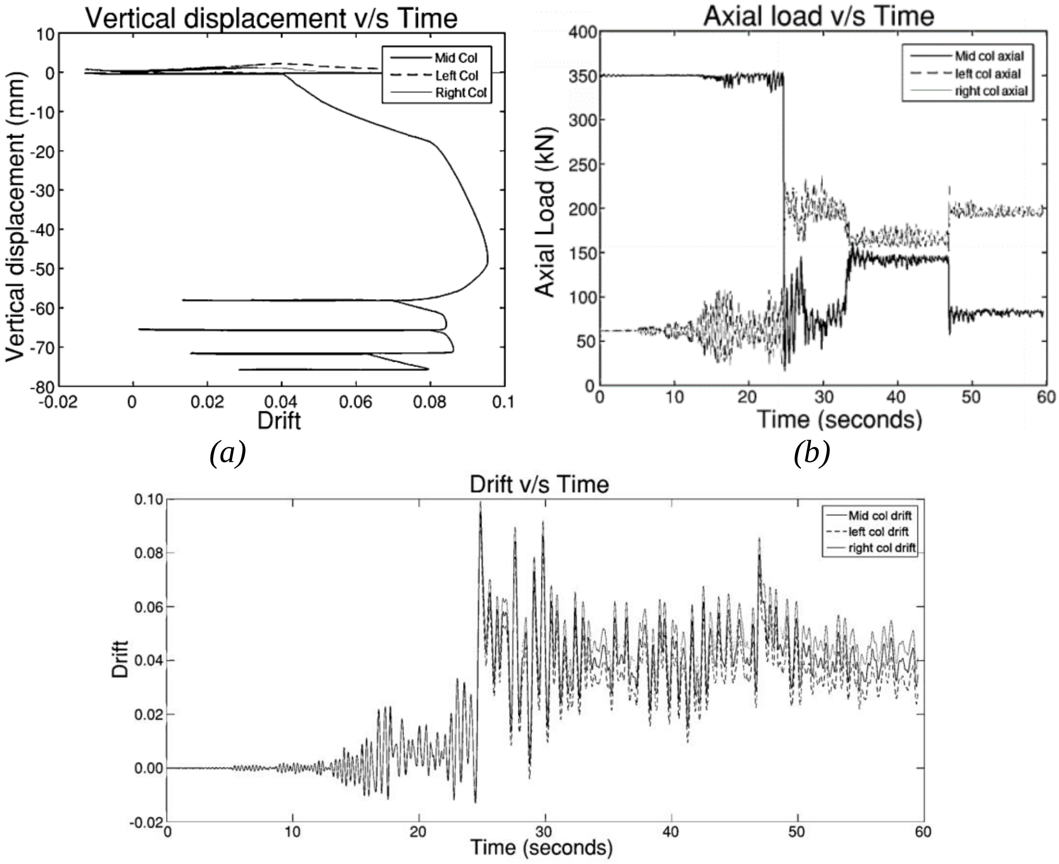
6.2 Energy based evaluation of capacity

The collapse indicator based on the energy approach is the instability triggered in the structure which is defined by equation (8). From figure 6(a) it can be seen that no global collapse has taken place as the gravitational energy is always less than the input seismic energy. On the other hand from figure 7(a), it is evident that significant deformations has taken place. However, from figure 6(b) a sharp spike in the rate of accumulation of the gravitational energy ($Egr = \frac{\partial}{\partial \mathrm{t}} \left| E_G \right|$) is observed which is much larger than the rate of input seismic energy ($Eeqr = \frac{\partial}{\partial \mathrm{t}} \left| E_{EQ} \right|$). This happens exactly at the point where the axial load in the central column is seen to drop sharply (figure 7(b)). Therefore, at time t=24.65s, partial collapse takes place where the central column loses its axial load carrying capacity. The damage measure (DM) or the drift is quantified at the time step just before equation (9) is satisfied, that is, when the rate of input earthquake energy is almost equal or comparable to the rate of gravitational energy released. At this point drift of the structure is as high as up to 10%. Therefore, the partial collapse or simply the collapse capacity of the structure, in terms of damage measure (DM) is around 10%. This is almost 250% higher than the collapse capacity specified in the code as a performance measure for collapse prevention (FEMA-356, 2000).



(a) Eeq (seismic input) v/s Eg (grav. Input) *(b) Rate of change of energies*

Figure 6: Comparison of earthquake input energy (Eeq) and gravitational energy (Eg)



(c)

Figure 7: Structural response

7. DISCUSSION

From the above results it is evident that global collapse does not take place as the structure still sustains the initial gravity loads applied. However, a partial collapse can be seen as the middle column losses its axial load carrying capacity. As the force redistributes, the outer columns experience increased loading (figure 7(b)). The redistribution of forces cannot be explained by a subjective definition of collapse (such as 4% of drift) which is prescribed as a performance measure. On the other hand, employing the energy based method for quantifying performance measure explains the redistribution of forces and partial collapse. Therefore, this method predicts more drift at collapse for the same structure as compared to the conventional methods (such as IDA). Also, the rate of change of gravitational energy can be used as a 'new' performance measure for collapse prevention. This way, in an effort to adhere to a particular value of drift, an engineer may design a structure 'leaner' thereby saving the material. This is possible since by using the conventional methods (such as IDA), the drift at collapse is underestimated which is not the case with the proposed energy method in this study. Hence, for a subjective value of drift at collapse of say 4%, an engineer instead of using a bigger section (stiffer section), s/he can use a smaller or 'leaner' section for which at actual drift at collapse is the same value of 4%. Ultimately, this will result in the development of more economical designs. Moreover, since the structural section can be designed smaller in size, it will also help save architectural space which in turn can be capitalised as an increase in the carpet area of the floor.

Evidently, energy based quantification of structural collapse can help develop more robust designs. Nevertheless, it can be argued that the collapse capacity indicated in this study may vary when the uncertainties in the empirical model used to define shear and axial failure are taken into account. Therefore, in order to establish collapse capacity so identified from the newly proposed method as a performance measure, a statistical analysis is imperative. Furthermore, one can also argue that proposed definition of partial/local collapse does not completely objectify the intended phenomenon. This is because, the proposed criterion is still subjective in terms of its inability to explain that by how much the rate of accumulation of gravitational energy should exceed the rate of input energy so as to categorise partial collapse.

8. CONCLUSIONS

It can be inferred that energy based collapse criterions are more efficient in quantifying the collapse capacity and hence employing energy based rules to quantify the performance measures will aid in the performance based earthquake engineering. Two collapse criterions are introduced for partial and global collapse which are basically *necessary and sufficient* conditions for structural collapse respectively. Therefore, using the first criterion, partial structural collapse can be characterised which is exemplified by gravity load collapse simulation in this study. Hence, economical designs can be expected when energy based rules are used. However, owing to several sources of uncertainties, it is important to understand the variation of the collapse capacity predicted by this method. Moreover, credibility of this method still needs to be established in different failure scenarios since, as discussed, the criterion of partial collapse is still quite subjective in nature. Thus, in the line of this thought, further research in the future is desired.

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