Predicting the Collapse Potential of Structures

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SUMMARY:

A new simplified expression to predict the collapse potential of moment-resisting steel frames structures subjected to seismic excitation is proposed. This expression will answer the question whether it is possible to estimate the collapse load of the structures without recourse to nonlinear analysis. This is an important parameter in design of low order computational models and the construction of realistic nonlinear response spectra. Nonlinear pushover analysis and incremental dynamic analysis are used with a range of heuristic building models to validate the proposed expression. Nonlinear static and dynamic analyses exhibit considerable scatter in their predictions of collapse potential. This is due to uncertainties in inertial loading patterns and ground motion timehistories. The proposed collapse potential expression is reasonably well correlated with the mean prediction of nonlinear incremental dynamic analysis. This expression would be extremely useful in the risk assessment of a large population of existing buildings where it is not realistic to perform very large number of nonlinear finite element analyses.

Keywords: Collapse potential, Incremental Dynamic Analysis, Pushover Analysis

1. INTRODUCTION

Generally, the current seismic design codes rely on the force-based design approach (SEAOC 1974; SEAOC 1988). The force-based approach considers the use of the behaviour factor, q and enforces capacity design principles. Indeed the use of the behaviour factor, q is only a rough estimation of the nonlinear structural response. Its specified design code values are dependant upon the material of construction and the type of structural system used. Hence the problems with the force-based approach are (i) a rough estimation of q, (ii) the highly smoothed design response spectrum (iii) the generalized approximation of the natural period(s) of the structure which is based on linear elastic behaviour, (iv) the modal summation of linear modes and (v) its inability to estimate inelastic displacements accurately. This makes the evaluation of the design base shear only a crude empirical formulation for engineers to use when designing buildings (Caglar, Pala et al. 2009; Crowley and Pinho 2010). From the experience of past earthquakes it is indicated that the force-based approach is not sufficiently subtle to control levels of damage in structures because it does not contain a detailed description of the inelastic behaviour that occurs during an earthquake (Li and Jirsa 1998).

For that reason, the engineering community has moved towards performance-based design (PBD) (Vision 2000 1995; FEMA-349 2000; SEAOC 2009). PBD is a concept that allows the designer to specify and predict the building's behaviour (namely the level of damage) to this range of increasingly less likely but more severe events (Pennucci, Sullivan et al. 2011). However, PBD is not without its problems. While the general philosophy is admirable, the proposed implementation suggests a very onerous computational effort on the part of the engineer. Attempts to quantify levels of damage have spawned the introduction of damage measures (DM), e.g. inter-storey drift etc., that are a proxy for actual damage (i.e. Vision 2000 limit states). The definition of these limit-states introduces uncertainty and ambiguity due to the use of damage measures rather than actual physical damage. This suggests the need to move towards a more comprehensive probabilistic approach, e.g. employing a fragility curve approach (Jaiswal, Wald et al. 2011).

Currently, nonlinear time-history analysis (NTHA) is the most rigorous analysis that is available and is used to obtain fragility curves. However, to use this type of analysis is not an easy undertaking, as it requires nonlinear analyses of complex structural systems to a set of unknown future seismic events. Besides problems with DM, there are many uncertainties in the definition of an appropriate set of ground motions to use in these analyses. Intensity measures (IM) seek to quantify ground motion intensity and there are many suggestions in the literature (Akkar and Özen 2005; Luco and Cornell 2007; Katsanos, Sextos et al. 2010; Bradley 2011; Grigoriu 2011). However, it is still not entirely clear what constitutes a dangerous ground motion for a particular structural system and this is especially the case for an inelastic structural system. Finally, the time and expertise required to conduct these analyses, which are pivotal for an engineer, is not insignificant.

Therefore, as an alternative treatment to nonlinear dynamic analysis, Fajfar and colleagues (Fajfar and Fischinger 1988; Fajfar and Dolšek 2011) suggested the N2 method, which is a fusion of nonlinear static analysis (i.e. MDOF pushover analysis (POA)) in conjunction with inelastic spectrum analysis of equivalent SDOF system. The presentation of the response spectrum in the acceleration vs. displacement format and the lateral force/mass vs. displacement from the pushover analysis (POA) enable the plotting of these two graphs on the same axes. Thus, capacity (from POA) and demand (from the response spectrum) can be clearly visualised. This provides an overview of the basic parameters that account for the seismic response of the structure. The method is considered more realistic in estimating the vulnerability of buildings during earthquakes than the force-based procedures commonly contained in current codes.

Despite this, one of the drawbacks of the N2 method is that from the pushover analysis it is strongly influenced by the assumed lateral load pattern. This is because without a complete nonlinear time-history analysis the inertial load applied to the structure can only be estimated very approximately. In Eurocode 8, there are two types of lateral load patterns that have been suggested, these are the triangular and the uniform one. It is known that these types of lateral load pattern are not reliable in capturing the dynamic behaviour (Medina 2004). This is because it is difficult to determine the force profile (up the structure) caused by a particular ground motion, i.e. every single earthquake record produces different behaviour. Moreover, this method neglects duration and cyclic effects, which account for the progressive changes in the dynamic properties that take place in a structure as it experiences the yielding and unloading during the earthquake.

From the discussion above each of the approaches considered (consisting of the force-based approach, NTHA and N2 method using POA) estimate the capacity of the structure. This capacity is an important parameter for developing the nonlinear response spectrum and the equivalent SDOF system. Each of the approaches highlighted have their own advantages and disadvantages. One of the main concerns is the time that is required to perform the robust and credible structural analyses, i.e. the specific modelling demand: for POA how many different load patterns should be explored? For NTHA how many acceleration records should be used? Thus, the main purpose of this study is to put forward a simplified, yet accurate, expression for predicting the capacity of structural systems.

In this study, the capacity which is the maximum force a structure can sustain is termed collapse load. Collapse load has alternatively been termed in the literature, collapse prediction, ultimate collapse load or the ultimate base shear, F_{ult}, etc (Zareian and Krawinkler 2010; Shafei, Zareian et al. 2011). However, the process of determining the collapse load is not an easy task. Each of the analytical methods has its own problems and this is true even for the most sophisticated of them, the nonlinear dynamic analysis.

Our proposed expression is more realistic and can be easily used by practitioners. Although it is impossible to fully replace the nonlinear analysis, such an expression can be used to tackle the problem with the nonlinear static analysis, i.e. the lateral load pattern. Moreover, this simplified expression can be used as an alternative to the dynamic analysis as it requires less computational time. The expression also could be used as an alternative approach for engineers to replace the current estimate of base shear that is proposed by Eurocode 8.

1.1. Aims and scope of this study

The main aims of this study are to present an analytical equation to predict the collapse load of moment-resisting steel frame (MRSF) of 2 and 4-storey structure. This expression is already published in the authors' previous work (Nazri and Alexander 2012) as shown in equation (1.1). From the previous study, the expression only validated using pushover analysis (POA). However, in this study, this expression also will be validated with the POA but with different lateral load pattern, namely uniform and triangular load pattern that suggested by Eurocode 8 (BSI 2004). Thus we extent the study by examining the expression using Incremental Dynamic Analysis (IDA). In IDA, at least three ground motion records were selected and used for the analysis (BSI 2004).

$$\max|F_s| = \chi\bar{M}\left(\frac{\alpha}{6n_s}g + \frac{3n_s^2+3n_s-2}{2n_s\left(n_s+1\right)}\lambda S_d(T_1)\right) \tag{1.1}$$

$\max|F_s|$ is a collapse load that have a relationship between the base shear and the participation factor. $\bar{M}$ is the total mass of the building. α is a ratio of the length of beam and column. n_s is a number of storeys. $S_d(T_1)$ is the ordinate of the design spectrum at period T_1, see Clause 3.2.2.5 (BSI 2004). Damage parameter χ is a one of the main parameters in this paper. It describes the proportion of the building that will damage at failure. χ is estimated computationally using POA. χ are shown below and details discussion about this expression can be referred in author's previous work as stated earlier.

$$\chi = \begin{cases} 1 & : n_s \le 2 \\ \dfrac{0.1n_s+0.8}{0.206n_s+0.657} & : n_s > 2 \end{cases} \tag{1.2}$$

2. DATA COLLECTION

2.1. A generic MRSF structures

A set of 2-D, generic MRSF buildings, (with 2- and 4storeys) are individually and economically designed according to the Eurocodes, EC3 (BSI 2005) and EC8 (BSI 2004) as shown in **Table 1**. Each building case has 3 bays (of 6m span) in each lateral direction and an identical storey height of 3.3m. The buildings are uniform and regular in plan and elevation. The most economical form is sought; hence, the designs represent the minimum acceptable forms of the framework, according to Eurocodes. Uniform (and identical) beam sections and identical columns are used throughout these frames. The prototype buildings are analyzed for seismic loads using SeismoStruct and Eurocodes are used to stimulate the seismic loads on the buildings.

Table 1. Details of MRSF

n_s	n_b	L_b [m]	L_c [m]	Beam section (UB)	Column section (UC)	T_1 [s]
2	3	6	3.3	457 x 191 x 98	356 x 406 x 287	0.35
4	3	6	3.3	533 x 210 x 101	356 x 406 x 235	0.59
n_s = number of storeys; n_b = number of bays; L_b = length of beam; L_c = length of column;						

All the structural frameworks are designed on soil type A and with peak ground acceleration (PGA) of 0.5g. The behaviour factor, q = 4 for MRF buildings with medium ductility class were used as suggested in EC8 (clause 6.3.2). The design of columns is in accordance with EC3 (BSI 2005). The column sections must be sufficiently large to avoid a soft storey failure. This was confirmed by

nonlinear pushover analysis.
In addition, the design is based on the strong-column weak beam philosophy, with the plastic hinges confined to beam ends. The global P-Δ effects are included in FEA the frames system as this can influence sway-collapse mechanics, (Bernal 1987; Gupta and Krawinkler 2000; Gupta and Krawinkler 2000; Mwafy and Elnashai 2001; Adam, Ibarra et al. 2004). The linearly elastic fundamental period for the first mode, T_1 for all storeys is stated in the figure for all frames (the period based on the linear modal analysis in GSA software). Furthermore, for the dynamic time-history analysis, 5% elastic viscous damping, ξ is assigned to the first mode.

2.2. The ground motion records

Selecting the accelerograms is a vital step in numerical simulations because the outcome of this analysis is markedly affected by this choice (Pagliaroli and Lanzo 2008). There are three types of accelerograms, which is artificial, synthetic and real accelerograms in terms of acceleration time-histories. Among these possible options, real accelerograms are nowadays emerging as the most attractive input for dynamic analysis mainly because they genuinely reflect the main factors (source, path and site effect). However, the question about which criteria should be used to select the records and the amount of records that are required for the analysis is still a controversial issue. Nevertheless, in the selection of the ground motion there a few parameters we should consider, such as the event magnitude, peak ground acceleration (PGA), distance, and soil type (Kramer 1996). Bommer and Ruggeri (2002) suggested that, the criteria for selection should match characteristics of the design earthquake; including magnitude, distance and site condition. For the number of records, it is controlled by the degree of scatter amongst the characteristics of the selected accelerograms. However, EC8 (2004) states, *"seismic motion may also be represented in terms of ground acceleration time-histories and depending on the nature of the application and on the information actually available, the descriptions of the seismic motion may be made by using artificial accelerograms and recorded or simulated accelerograms"*. In EC8 (BSI 2004), at least three ground motion time-histories should be employed regardless of their nature. Similar suggestion is made in FEMA 450 as stated *"as a minimum, the Provisions require that suites of ground motion include at least three different records"*

Table 2. Selected corrected accelerograms

Earthquake event	Host	Date (Time)	Magnitude (M_L)	Epicenter [km]	PGA [g]	Source
South Iceland	Iceland	21-06-2000 (00:51:48)	6.4	14	0.176	ISESD
Irafoss	Iceland	21-06-2000 (00:51:48)	6.4	9.3	0.103	ISESD
Ljosafos	Iceland	21-06-2000 (00:51:48)	6.4	20	0.106	ISESD

In this study, at least three accelerograms, recorded from past events has been taken from ISESD (Internet Site for European Strong-Motion Databases), to use in this study. The criteria for selection of these accelerograms are: (i) within a magnitude range 5.5-6.5 M_L; (ii) distance within 30km from epicenter; (iii) free-field records; and (iv) with a soil class A (rock) because this study will not consider soil-structure interaction. In addition, only accelerograms in horizontal components will be considered. All the corrected records from PEER and ISESD databases that fell within the range of these selection criteria were selected; this resulted in 17 records, of which 3 were selected for this paper. Further analysis shall be undertaken in future publications with the full set of records. Beyond these characteristics, no additional attempts have been made to categorize the records selected to this study. All the records are listed in Table 2 and **Figure 1** .

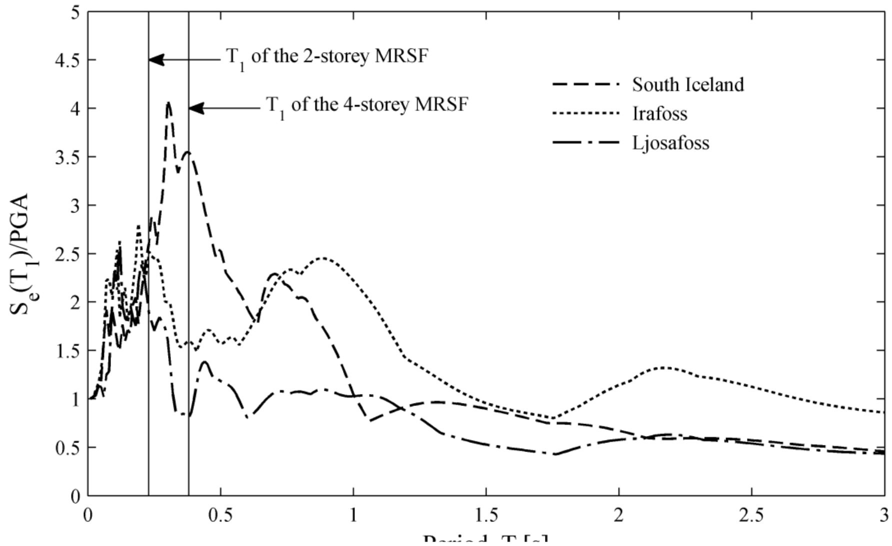


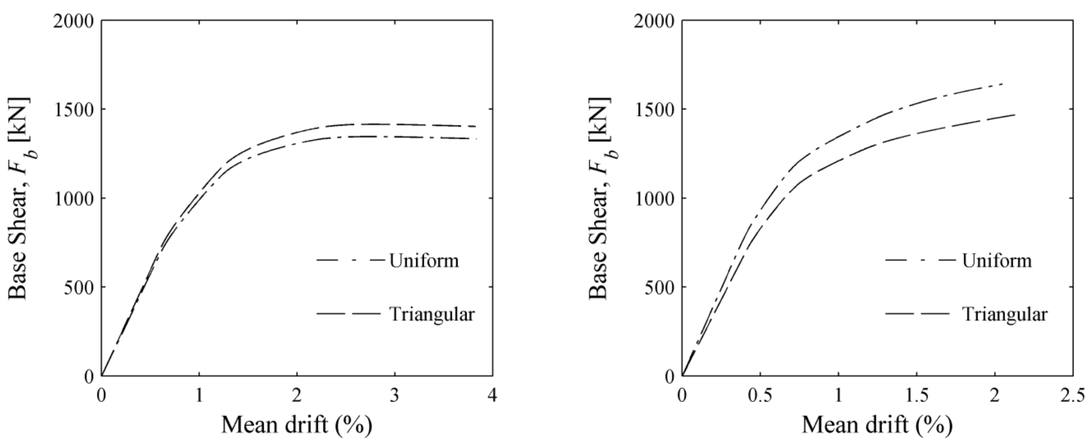
Figure 1. Elastic total acceleration response spectra for records

3. RESULTS AND DISCUSSION

3.1. Pushover analysis (POA)

Figure 2 shows the capacity curve with a correlation between the base shear, F_b and the mean drift (%). The capacity curves in the figure have been plotted with a different lateral load pattern. Each lateral load patterns produces different curves because of the plastic hinges formation in the structures.

The mean drift in this study was calculated by taking the maximum displacement of the top storey and dividing by the total number of stories. Drift can be defined as the storey drift at which incipient global collapse caused by P-Δ effects has been reached (Huang and Foutch 2009). Drift is important for a structural stability and human comfort during and after the building experiences the motions (Naeim 1989). Moreover for statistical simplicity mean storey drift is used rather than maximum storey drift.



(a) 2-storey (b) 4-storey

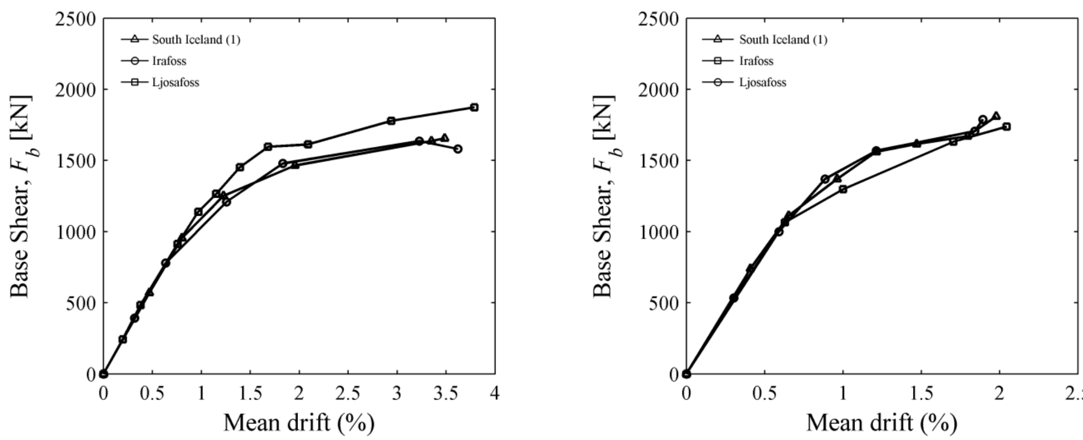
Figure 2. Capacity curve with different lateral load pattern for 2- and 4-storey

The capacity curve in the figures is based on the failures of the beam section. A beam section typically fail in one of four ways, either failure from the plastic hinge formation, shear failure at a section (which is not a problem here), lateral-torsional buckling along the length of the beam, or local buckling of the beam cross section. However, this study only focused on the formation of the plastic hinges. Theoretically, the plastic hinges start to occur when the bending stress reaches the material yield. In collapse due to the formation of the plastic hinge, the stress in the beam reaches the yield stress. The bending moment cannot be increased and the beam collapses as though a hinge has been inserted into the beam.

From the results, the triangular lateral load pattern for 2-storey produce slightly higher predicted capacity compared to triangular load pattern. Conversely, in 4-storey case, the uniform lateral load pattern produce the highest capacity rather than triangular. It can be concluded that, the lateral load pattern plays an important roles in the predicting the capacity of the structure for a static case. There is no strong evidence saying which lateral load pattern should be used for the analysis of the structure. This is because, the behaviour of the capacity curve is depends on the formation of the plastic hinges, geometry of the structure, material used and the section geometry. The question is which lateral load pattern compare best with the predicted capacity from dynamic analysis (IDA)?.

3.1. Incremental Dynamic Analysis (IDA)

The results of IDA for 2- and 4-storey are displayed in Figure 3. Ground motion records are scaled to a PGA and full MDOF analyses are undertaken for all 3 three records. Then, PGA is incremented and the analyses are repeated until it collapses the structure, which requires several hours of computations per record. Figure 3 shows the earthquake records that can cause collapse of a structure with a PGA of less than 3g. These values are compared with the capacity curve in Figure 2 and proposed analytical expression.



(c) 2-storey (d) 4-storey

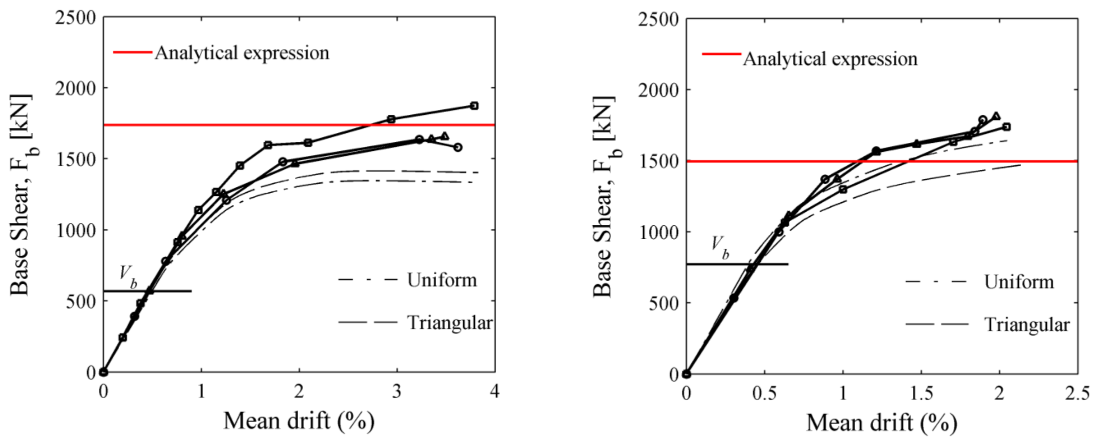
Figure 3. IDA curve results

3.2. POA-IDA-Analytical expression

Figure **4** shows the comparison results from the numerical estimation using equation (1.1) with IDA curve and POA with a uniform and triangular load pattern. The capacity curve is a valuable tool for the study of global behaviour of the structures, i.e the base shear and displacement. For evaluation of the accuracy and performance in estimating the seismic behaviour of the structures, the capacity curve obtained from the pushover analysis has been depicted and compared to the IDA curves.

Other than that, the uniform and triangular load pattern suggested by Eurocode is a bit conservative in order to capture the demand capacity from the dynamic analysis. Therefore, from this observation, a

current lateral load pattern that has been suggested in Eurocode needs to be revised.
For another comparison, the design base shear, V_b has also been plotted in the figures. As defined in Eurocode 8 (BSI 2004), V_b is an estimation of the maximum expected lateral force that will occur due to seismic ground motion at the base of the structure. The calculation of the V_b is dependent on the soil condition at the site, the level of ductility, mass of the building and the fundamental period (linear) of the vibration of the structure when subjected to earthquake loading. Therefore, even though it is simple to use, it is crude empirical formulae (Shafei, Zareian et al. 2011). Based on our results, it shows that V_b far underestimates the collapse load for both static and dynamic analyses. Again, a new expression is needed to introduce the replacement of a current expression proposed by Eurocode in order to calculate the maximum expected lateral force. As an alternative procedure, the proposed equation (1.1) can be used as an indicator to a designer.



(a) 2-storey (b) 4-storey

Figure 4. The estimation of numerical FEA by using equation (1.1) with POA and IDA results

3. CONCLUSIONS

This study was an extended discussion about the proposed equation (1.1) to predict the collapse potential or collapse load, F_s from the authors' previous work. The study continued by comparing the F_s with the IDA and the pushover analysis (POA). Results show that our analytical expression quite promising for a low and medium rise buildings. A uniform and triangular lateral load pattern were used since this characteristic has a major influence on the ensuing analysis in order to determine the capacity demands of the structure. Results indicate that both type of lateral load pattern that suggested by Eurocode underestimate the IDA predicted capacity. Therefore, from this observation, a current lateral load pattern that has been suggested in Eurocode needs to be revised.

An additional comment based on the findings from this section is that, the design base shear, V_b is a serious underestimate of the collapse load obtained from both static and dynamic analyses, i.e. the design base shear doesn't equal the collapse load.

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