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Assessing collapse capacity and risk in subsequent ground motions using SHM results for incremental dynamic analysis

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ABSTRACT

Predicting building collapse due to seismic motion is critical in design and more so after a major event. Damaged structures can appear sound, but collapse under following major events. There can thus be significant risk in decision making after a major seismic event concerning the safe occupation of a building or surrounding areas, versus the unknown impact of unknown major aftershocks. Model-based pushover analyses are effective if the structural properties are well understood, which is not valid post-event when this risk information is most useful.

This research combines Hysteresis Loop Analysis (HLA) structural health monitoring (SHM) and Incremental Dynamic Analysis (IDA) methods to determine collapse capacity and probability of collapse for a specific structure, at any time, a range of earthquake excitations to ensure robustness. The nonlinear dynamic analysis method presented enables constant updating of building performance predictions using post-event SHM results. The resulting combined methods provide near real-time updating of collapse fragility curves as events progress, quantifying the change of collapse probability or seismic induced losses for decision-making - a novel, higher resolution risk analysis than previously available. The methods are not computationally expensive and there is no requirement for a validated numerical model. Results show significant potential benefits and a clear evolution of risk. They also show clear need for extending SHM toward creating improved predictive models for analysis of subsequent events, where the Christchurch series of 2010-2011 had significant post-event aftershocks after each main event. Finally, the overall method is generalisable to any typical engineering demand parameter.

1 INTRODUCTION

Structural health monitoring (SHM) arose from the need to accurately detect, localise and quantify damage after major seismic or environmental loads. Significant damage can be exacerbated in later large aftershocks, increasing the risk to occupants and the structure. SHM thus addresses the immediate needs of responders and leadership. However, it does not provide a ready, quantified means of assessing, the ongoing safety of a structure or the, potentially now modified, risk of collapse. This outcome requires a model made from SHM results. It would enable further nonlinear analyses to assess the risk of collapse and thus more optimal decision making for occupants and owners.

Collapse prediction under seismic loading is one of the principal objectives of earthquake engineering (Adam and Jager, 2012b, Han and Chopra, 2006). In particular, large lateral displacements in tall structures can lead to *P*-delta effects (Fenwick and al, 1992). *P*-delta effects increase overturning moments, generating effectively negative post-yield stiffnesses, and thus increase the likelihood of global collapse (Adam and Jager, 2012a). Degradation in strength and stiffness are also a related important consideration for buildings in seismic zones. Hence, there is a need to know the structural stiffness degradation and inelastic net deformation to better estimate collapse risk.

Stiffness degradation is dependent on structural characteristics, such as material properties, geometry and connection types (Adam and Jager, 2012b), and may be more significant for structures built on soft soil (Miranda and Ruiz-Garcia, 2002, Moghaddasi et al., 2015, Moghaddasi et al., 2012). Hence, degradation and *P*-delta effects must be considered in SHM and risk analysis (Zhou et al., 2017a, Zhou et al., 2017c).

There is thus a need to link the results of SHM to prediction of further structural capacity, including *P*-delta effects, to enable better prediction of subsequent structural behaviour and thus risk. Incremental Dynamic Analysis (IDA) linked to SHM identified models offers a solution. IDA of practical structures is computationally demanding and, as a result, simplified methods approximating structures as single degree of freedom systems and using static pushover analysis have been developed to provide approximations (Han and Chopra, 2006). However, this approach can lack resolution and lead to inaccuracy.

This study is a proof of concept numerical analysis for dynamic IDA linked to a proven SHM method to provide potentially valuable risk assessment relatively immediately post-event. Hysteresis Loop Analysis (HLA) is an accurate, proven, real-time dynamic SHM method (Xu et al., 2014, Xu et al., 2015, Zhou et al., 2015a, Zhou et al., 2015b, Zhou et al., 2017a, Zhou et al., 2017c, Zhou et al., 2017b), providing realistic structural parameters for subsequent collapse prediction analyses. It is the first to utilise HLA, or any SHM method, in combination with nonlinear IDA to identify collapse capacity and probability of collapse.

2 METHODS

The proof of concept system is modelled as a single degree of freedom (SDOF) structure using an elasto-plastic bilinear hysteric model, and analyses cases with and without stiffness degradation. HLA SHM (Zhou et al., 2015a) is used to determine the current damaged or undamaged structural parameters of the system. IDA methods (Vamvatsikos and Cornell, 2002) using a suite of earthquake records with incremented intensity are utilized with nonlinear dynamic analyses to generate collapse capacity spectra quantifying the probability of collapse for the SHM identified post-event structure under this range of input seismic loading.

2.1 Hysteresis Loop Analysis (HLA)

Hysteresis Loop Analysis (HLA) (Zhou et al., 2015a) is used to identify stability and hardening coefficient, elastic and plastic stiffness, and yielding displacement. It removes the need for the typically used static pushover analysis, which requires assuming first mode collapse, where assuming a structural model in SHM can lead to errors if it is not accurate (Zhou et al., 2017c). The HLA method extracts significant half cycles to identify elastic and plastic stiffness values from the hysteretic force-displacement response. The equations used to determine these values are defined:

$$\theta = \frac{P}{Kh} = \frac{mg}{Kh} \tag{1}$$

$$K_e = (1 - \theta)K \tag{2}$$

$$K_p = (\alpha - \theta)K \tag{3}$$

Where θ and α are the stability and hardening coefficients, respectively, K is the original structural stiffness, identified using HLA, excluding P-delta effects, P is the weight (kN) of the building, h is the height of the building, and K_e and K_p are the values for elastic and plastic stiffness from the HLA identification.

Thus, the stability coefficient θ can be rewritten as a function of identified K_e and K_p values:

$$\theta = \frac{mg}{K_e h + mg} = \frac{mg\alpha}{K_P h + mg} \tag{4}$$

2.2 Incremental Dynamic Analysis (IDA)

Incremental Dynamic Analysis (IDA) is a parametric analysis method used to more thoroughly estimate building performance under seismic loading to quantify the relationship between seismic capacity and demand, potentially leading to evaluation of financial risk (Vamvatsikos and Cornell, 2002, Mander et al., 2007). This relationship is found by subjecting a structural model to varying ground motion records scaling the intensity of the records in increments until, in this case, collapse is predicted. This analysis results in multiple curves of any intensity measure (IM), such as peak ground acceleration, plotted against any engineering demand parameter (EDP), such as maximum deflection or drift. (Adam and Jager, 2012a, Mander et al., 2007). It quantifies the largest earthquake intensity at which a structure maintains, in this case, stability, for each particular ground motion and is used to determine the global collapse capacity.

There is no unique definition to characterize the intensity of an earthquake record (Adam and Jager, 2012b, Vamvatsikos and Cornell, 2002). An accepted method is to use the normalized spectral acceleration damped at ξ=5% at the structures fundamental period ($S_a(T_1, 5\%)$) as a fraction of the product of acceleration of gravity, g, and the base shear coefficient, γ**.** In this study, spectral acceleration at the structures first mode period has been used as the intensity measure, (Asgarian et al., 2010).

IDA is thus highly dependent on the earthquake record used. Thus, the analysis presented is performed for a range of 20 design level ground motion inputs from the SAC medium suite (Sommerville et al., 1997).

The classification of the collapse capacity for multiple earthquake events, CC_i, is defined:

$$\boldsymbol{CC_i = \frac{S_{a,i}}{g\gamma} \qquad for\ i = 1, \ldots, n} \tag{5}$$

Where g is gravitational constant and γ is defined:

$$\gamma = \frac{f_y}{mg} \tag{6}$$

Where m is the total mass of the structure and f_y is defined:

$$f_y = d_y K \quad (7)$$

Where d_y is the yielding displacement and K is the initial building stiffness, assumed or identified using HLA or another SHM method.

This analysis utilized peak ductility, μ, as the damage measure or EDP, which is defined:

$$\boldsymbol{\mu = \frac{d_{max}}{d_y}} \quad (8)$$

Where d_{max} is the maximum displacement response, and d_y is the yielding displacement, which can be found from the response data from a prior event.

2.3 Degradation Analysis

Stiffness degradation was incorporated in a second analysis based on the total energy dissipation for each cycle and calculated from the total area enclosed by the hysteresis loop for each half cycle. A stiffness degradation parameter, η, was defined:

$$\boldsymbol{\eta = 1 + \delta_\eta \epsilon} \quad (9)$$

Where δ_η is the stiffness degradation constant, and ϵ is the energy dissipation for half cycles including plastic deformation. Therefore, the changing stiffness $K_\eta(t)$ can be defined:

$$K_\eta(t) = \frac{1}{\eta} \times K \quad (10)$$

This method allows continuous updating of the stiffness values throughout the analysis if degradation due to prior events is to be included.

Note that degradation, as defined in Equations 9-10, can also be defined to mimic a known degradation measured during SHM for a given building or class of buildings.

3 CASE STUDY/ PROOF OF CONCEPT ANALYSIS

This methodology is used in a proof of concept case study to determine the probability of collapse for a 7-storey steel moment resisting frame (SMRF) structure with known properties in Table 1 (Christopoulos et al., 2002). The initial stability coefficient, θ, is calculated to be 0.1014 using Equation (4). The effect of stiffness degradation is included, as variation of θ, α and T_0 would change the collapse capacity

Table 1. Structural properties of 7 story steel MRF structure for proof of concept analysis

Total mass (m)	4	kN s^2/mm
Initial stiffness (K) without P-Δ effect	157.9	kN/mm
Initial period (T_0)	1	s
Height (h)	24.5	m
Hardening coefficient (α)	0.02	
Yielding displacement (d_y)	24.85	mm

The structural response of the modelled structure is first simulated using the Newmark-β integration. Results are sampled with 10% RMS noise added at 1000Hz for acceleration and 1Hz for displacement, representing realistic sensor measurements, sensor noise, and sampling rates (Zhou et al., 2015a, Zhou et al., 2017c). The

HLA method is then applied to the modelled response with added noise to identify the structural parameters for input to the IDA procedure, thus simulating the overall SHM of a “real” structure with known behaviours in this case, followed by IDA using the identified structural parameters to assess the risk of collapse.

The IDA curves and the probability of collapse for the modelled structure is assessed for each ground motion. From this curve the exact intensity of collapse and its likelihood of occurrence are calculated. Ground truth, direct simulation results to provide the “truth” for comparison at each event and on average to assess the performance of these two methods in combination, and thus their potential for use in real-time monitoring and decision making after an event. All analyses include *P*-Delta effects, first excluding stiffness degradation and then again with degradation to assess its impact in increasing collapse risk using Equations 9-10. The overall analysis quantifies the potential of mixing proven SHM and IDA methodologies to rapidly provide collapse risk assessments right after a major event.

4 RESULTS AND DISCUSSION

4.1 Structural Parameter Identification

HLA identified structural mean values for the 20 earthquake events are compared with the ground truth results from direct numerical simulation in Table 2. The identified values are sufficiently close to the numerical model ground truth, despite the 10% RMS added noise, matching prior analytical and experimental results (Zhou et al., 2017a, Zhou et al., 2017b, Zhou et al., 2015a, Zhou et al., 2017c, Zhou et al., 2015b). Table 3 shows collapse capacity found using IDA and stability coefficients from HLA results for each event. Stability coefficients are very consistent, while median and one lognormal standard deviation (16^{th} and 84^{th} percentiles) of collapse capacity values shows significant variability.

Table 2. Mean values from analysis compared with simulation inputs

	Elastic stiffness K_e (N/m)	**Plastic stiffness K_p (N/m)**	**Stability coefficient θ**	**Yielding Displacement d_y (m)**
Mean HLA Values	1.42E+07	-1.21E+06	0.1017	0.0244
Known Ground Truth	1.42E+07	-1.29E+06	0.1014	0.0249

Most earthquake ground motions lead to a softening structural response, with gradual degradation towards collapse. Some earthquakes lead to more severe hardening and non-monotomic behaviour (Vamvatsikos and Cornell, 2002)**.** This variation illustrates the varying demand structures are subject to during seismic excitation depending on the specific ground motion , and validates using multiple events to generate a robust risk profile.

4.2 Stiffness Degradation Analysis

Including stiffness degradation per Equations 9-10 assesses the impact of stiffness degradation, where the structure is assumed to be further damaged during a subsequent event from the initial event used to find the HLA identified structural parameters. In general, it would be assumed a structure would thus collapse at a lower intensity as a result of continuing structural degradation. Figure 1 compares the changes of elastic stiffness over time between the HLA identified values and the simulated true values found using Equations 9-10. The overall results validate the ability of the HLA method to accurately estimate structural parameters for the IDA procedure including degradation, and thus the IDA results shown above without degradation.

Table 3. Stability coefficient and collapse capacity for 7-storey SMRF structure under 20 ground motion excitations, where stability coefficients are very consistent and median and 1 lognormal standard deviation (16th and 84th percentiles) are shown for the IDA resulting collapse capacity.

EQ	Stability coefficient	Collapse Capacity
1	0.10147	23.56
2	0.10782	4.71
3	0.10139	48.22
4	0.10178	14.62
5	0.10147	7.59
6	0.10132	53.58
7	0.10134	13.77
8	0.10103	11.65
9	0.10148	48.54
10	0.10132	17.60
11	0.10113	18.01
12	0.10147	12.41
13	0.10321	11.91
14	0.10143	27.52
15	0.10112	7.93
16	0.10141	4.85
17	0.10051	7.23
18	0.10116	17.71
19	0.10137	14.27
20	0.10102	9.45
Median		12.55
16th Percentile		27.01
84th Percentile		6.97

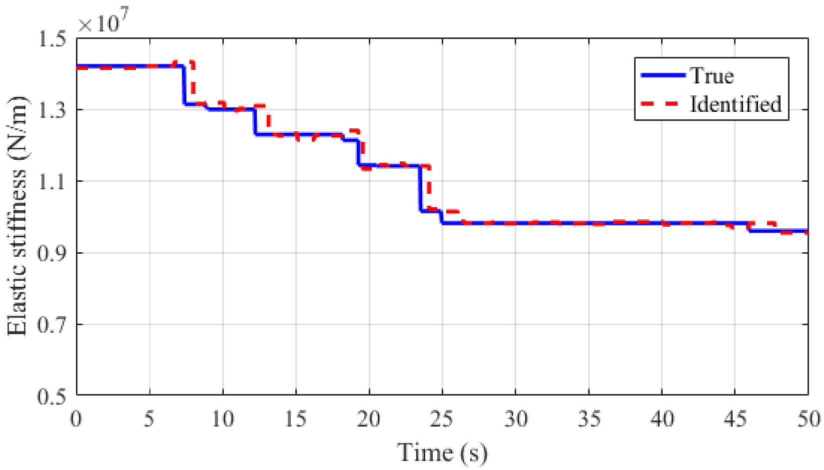

Figure 1.Comparision of stiffness degradation between the identified and true values.

The probability of collapse is then re-evaluated considering the continuing degradation of stiffness combined with *P*-delta effects. Figure 2 compares the new collapse fragility curve considering structural degradation with the one found from Table 3 without degradation. There is notable change in risk, as expected.

More importantly, the change of the collapse safety is also quantified over events due to the use of the HLA method, which is not the case for the static pushover methods. In particular, collapse capacity in 10% probability of collapse after degradation dropped to 4.9 from the original value of 6.4, and can be compared to any designed tolerable probability of collapse such as 10% probability of collapse at the 2% probability in 50 years with 90% confidence (Zareian and Krawinkler, 2007). It thus provides more intuitive and accurate estimates of potential earthquake induced losses, enabling improved decision-making for hazard mitigation.

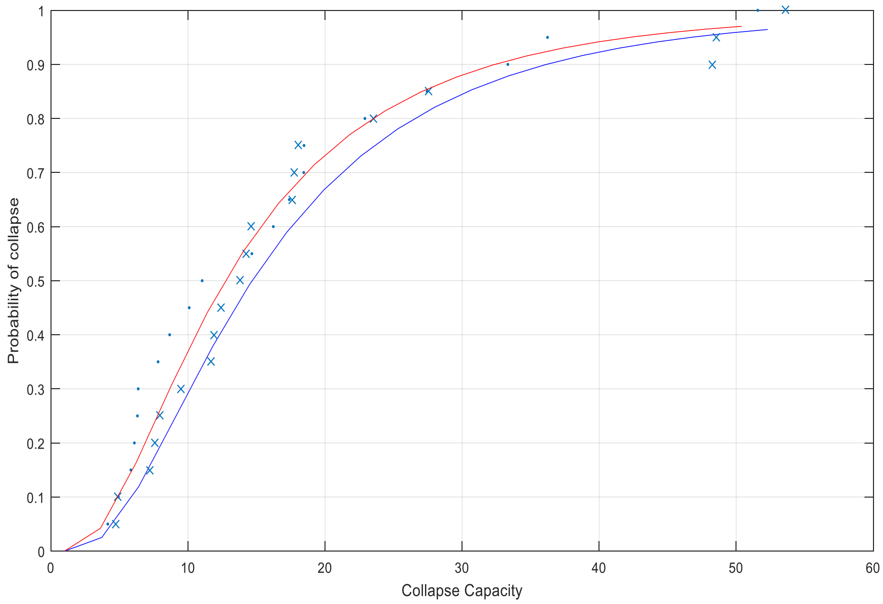

Figure 2. Comparison of collapse fragility curves before and after a significant earthquake event (EQ19), where the blue line represents the original non-damaged collapse fragility curve with data points (x), and the red line represents the updated collapse fragility curve with data points (dots).

This analysis clearly shows the combination of HLA and IDA analysis can provide the same results as a nonlinear analysis, validating the concept. The main advantage is the robustness implied by the ability of the HLA method to accurately identify structural parameters through a major event. Thus, there is no need to assume model parameters for an already damaged building for the IDA analysis, if the desired result is the current collapse capacity curve and consequent risk of collapse.

5 CONCLUSIONS

This paper presents a novel combined SHM and IDA methodology to assess collapse risk for a structure after a major event, a major need in providing better post-event information for decision making on building occupation and the safety of surrounding areas. It quantifies the increased risk when structures realistically degrade further under large post-event aftershocks, such as those seen in the Christchurch series of events in 2010-11. This latter result thus highlights the needs for not only implemented SHM to assess current capability, but also the ability to utilise these results in predictive models, which currently do not exist.

The analysis presented also clearly shows the potential of the methods presented to accurately assess collapse risk by combining SHM and nonlinear dynamic analysis methods. Importantly, the results were obtained with relatively very low computational effort, so the results would be available very soon post-event. Finally, the overall method is generalisable to other forms of degradation, and thus can be extended to assess the risk associated with any typical engineering demand parameter, as well as to thus utilise these methods with existing methodologies to extend them to financial risks and annualised estimated costs.

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