PRELIMINARY EVALUATION OF SEISMIC CAPACITY OF CONCRETE HIGHWAY BRIDGES CONSIDERING DAMAGE ACCUMULATION

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ABSTRACT

A practical approach to manage and reduce earthquake risk of structure is to conduct seismic performance assessment. In this approach, the structural performance levels are described by the damage limit-states which are specified in the form of peak responses by the current seismic codes. However, due to the dynamic nature of seismic load, peak responses cannot capture the actual effects of damage accumulation. Therefore, in order to accurately quantify the seismic risk to structures, it is important to re-evaluate and examine the seismic capacities of structures in the form of cumulative damage indices. In this paper, a typical RC supporting structure of highway bridges is examined. Two nonlinear FE models of the supporting structure are established using the discrete and smeared cracking models for concrete fracture. Each model is then analysed under increased quasi-static, cyclic, and seismic loading individually up to the near-collapse performance level. To identify the level of seismic capacities of both models, seismic damage is evaluated in the form of both the peak and cumulative responses. The results of this study can be used to further develop the seismic fragility curves for the supporting structure, when damage accumulation is concerned.

KEYWORDS

Performance-based seismic design, damage accumulation, record duration, fundamental period, concrete highway bridges

INTRODUCTION

Natural hazards have always had undesirable influences on infrastructure, human life, economies, and communities. Recent advances in computational science and technology have resulted in higher safety levels against natural hazards for the benefit of human societies. However, earthquakes remain one of the most unpredictable of natural hazards, due to the randomness of the intensity of ground motion and of the frequency of occurrence. Initial academic and scientific attempts for mitigating fatalities and financial losses of earthquakes started in the early 1900s and have resulted in several structural seismic codes and building standards (ASCE 41-06, 2007; ASCE 7-10, 2010). Since this achievement has been accomplished only in the past few decades, a large number of structures remain which are unable to satisfy the requirements of the well-accepted seismic standards. Consequently, while the new earthquake-resistant structures are designed to have an appropriate balance of stiffness and ductility, a lack of this balance causes the existing aged structures to be susceptible to earthquakes. Therefore, seismic performance assessment of the structures is a key element in the process of achieving the overall safety of infrastructure against seismic hazard. Using the traditional force-controlled earthquake engineering and structural design methods does not provide the possibility to assess the performance of the structures under seismic forces (Wakabayashi, 1986). Subsequently, an effort has been made by structural engineers over the past two decades towards developing Performance-Based Seismic Design (PBSD) methods (Bazzurro & Cornell, 1994). This is especially efficient for investigating the performance of the structure during seismic events. The structural performance levels are explained by descriptive damage levels as shown in Table 1. For the practicality in representing these performance levels, a structural-element-specific damage measure is assigned to each level, namely the limit-states. Consequently, to achieve the above performance levels, unlike force-controlled earthquake engineering, the seismic responses are controlled to not exceed the prescribed limit-states.

Table 1. Damage control and building performance levels (ASCE 41-06, 2007)

Target Building Performance Levels	Overall Damage	General Remarks
Collapse Prevention (CP)	Severe (Extensive)	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. In-fills and un-braced parapets failed or at incipient failure. Building is near collapse.
Life Safety (LS)	Moderate	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.
Immediate Occupancy (IO)	Slight	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.
Operational (OP)	Very Light	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional

As understood from above, PBSD is an approach in which the consequences are highly sensitive to the defined structural performance limit-states. This means that the type and measure of a selected damage indicator as the limit-states affect the seismic risk to the structure, wherein it may differ from one damage index to another. Conventionally, the storey drift ratio is used to explain the limit-states and structural damage. Using lateral displacement as a damage index has the advantage of simplicity in application as well as providing the possibility to investigate P-Δ effects. However, a perfect structural damage indicator must be capable of addressing the details and location of seismic damage as well as the residual capacity and sustainability of structures in aftershocks. In addition, the nature of seismic forces denotes dynamic loadings. This means that the use of displacement-based damage indices is not appropriate to capture the effects of damage accumulation. Therefore, in order to accurately quantify the performance limit-states, it is important to re-evaluate and examine the seismic capacity of structures considering alternative damage indices. This study specifically focuses on investigating the seismic capacity of reinforced concrete highway bridges by developing a new cumulative damage index. In regard to this, a new damage index is proposed based on the variation of fundamental period of structure which can capture the effects of both the damage accumulation of the nonlinear seismic load cycles, as well as the ground motion duration.

SEISMIC DAMAGE ACCUMULATION

Cumulative Damage Factors

It is possible to reflect the cyclic effect of seismic loads by defining a cumulative damage factor. Such a damage factor seems to be more appropriate than structural deformations especially when a previously damaged structure imposed by a sequence of earthquakes is considered (Mander & Rodgers, 2013). In regard to this, the phenomena which signifies the importance of damage accumulation are the effects of low-cycle fatigue, produced by the nonlinear cycles of the seismic loads (Miner, 1945), as well as an energy concept in which the dissipated hysteretic energy of seismic load cycles are taken into account as the indicator of produced and accumulated damage (Housner, 1956). An ideal cumulative damage measure is defined as a normalised index ranging from 0 to 1, in which 0 denotes no damage and 1 indicates failure. In regard to this, the famous Park-Ang damage index (Park & Ang, 1985) is one of the most recognised cumulative damage indices for concrete structures:

$$D_i = \frac{\delta_M}{\delta_u} + \frac{\gamma}{Q_y \delta_u} \int dE \tag{1}$$

where δ_M and δ_u are the maximum and ultimate deformations, Q_y is the yield strength, dE is the incremental absorbed hysteretic energy, and γ is a non-negative parameter (γ is usually taken as 0.15). Although the Park-Ang damage index is widely accepted as a reliable cumulative damage index, it is incapable of addressing the effects of ground motion duration. Experiences from multiple different earthquakes have confirmed that the duration of ground motion influences the level of structural damage. For example, records with large acceleration and spectral values may produce slight damage if the duration is short (e.g. the Ancona earthquake in 1972), whereas records with low acceleration and long duration can be very destructive (e.g. the Mexico earthquake in 1985). A damage factor was proposed by Cosenza & Manfredi (1997), I_D, which is established on the "*Arias Intensity*" and number of plastic loading cycles, and therefore; it is related to the ground motion duration and energy content of the earthquake:

$$I_D = \frac{\int_0^{t_E} a_g^2(t)\, d(t)}{PGA \cdot PGV} \tag{2}$$

In the equations above, g, t_E, $a_g(t)$, PGA, and PGV are the gravitational acceleration, total duration of the earthquake, ground motion acceleration, peak ground acceleration, and peak ground velocity, respectively. Although many different definitions have been proposed for strong-motion duration, defining the earthquake duration in relation to the main energy content remains a problem. This is because duration is a secondary parameter of a ground motion in which it is not possible to establish a direct correlation between the ground motion duration and seismic damage.

A New Damage Factor

The elongation of fundamental structural period is also a factor which demonstrates the structural damage. To investigate this, the instantaneous fundamental period, T_i, is calculated and plotted at input record acceleration time-steps, t_E. A regression analysis of these data is then performed to probe the structural damage pattern. In Figure 1 a schematic tri-linear regression is shown which was assumed to fit to the changes of T_i. In the first segment, A-B, T_i is considered to be constant and equal to the elastic fundamental period, T_E. This denotes that the linear loading cycles do not affect structural strength and stiffness. Any increases in T_i at this stage will be diminished whenever loading is terminated. The next segment, B-C, asserts that during the nonlinear loading cycles, T_i increases linearly until it reaches the ultimate value, T_U. $t_{0,P}$ and $t_{U,P}$ are the first and last time steps corresponding to the increase of T_i, and α is the slope of linear variation in this stage.

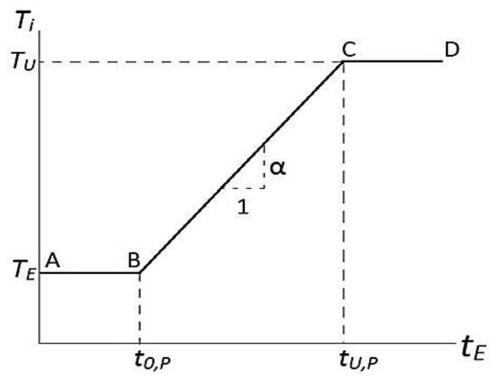

Figure 1. Linear regression analysis of the variation of fundamental period versus the ground motion record time

Finally, the last segment, C-D, represents the minor loading cycles after the main plastic loading cycles, which do not significantly affect the changes of fundamental period. Therefore, it remains almost constant and equal to T_U. In the damage model presented above, the ratio of the fundamental periods, T_U/T_E, is an index of the structural ductility, while the growth of fundamental period, T_U-T_E, highlights the effects of damage accumulation. In addition, the length of time interval, $t_{U,P}$-$t_{0,P}$, and slope of changes of T_i during plastic cycles, α, can be taken into account to capture the influences of narrow-banded ground motions (long-duration rapidly deteriorating ground motions) in the target damage index. To establish a damage index based on the fundamental period, the sensitivity of these parameters to seismic damage measurements is then analysed through the acknowledged cumulative damage indices to assign a weighting factor to each parameter.

RESEARCH METHODOLOGY

Structural Model Description

Seismic performance assessment is particularly important for bridge management and maintenance in order for transportation networks to continue functioning during post-event operations. An essential requirement for this purpose is to attain an overview of the probable seismic damage to highway bridges in order to decide on an effective rehabilitation strategy. In this study, a typical configuration of Multi-Span Simply Supported (MSSS) concrete highway bridges is considered which is shown in Figure 2a. This is because this type of bridges is very common worldwide and the failure of such kind of bridges during seismic events may result in a long suspension of the whole transportation network.

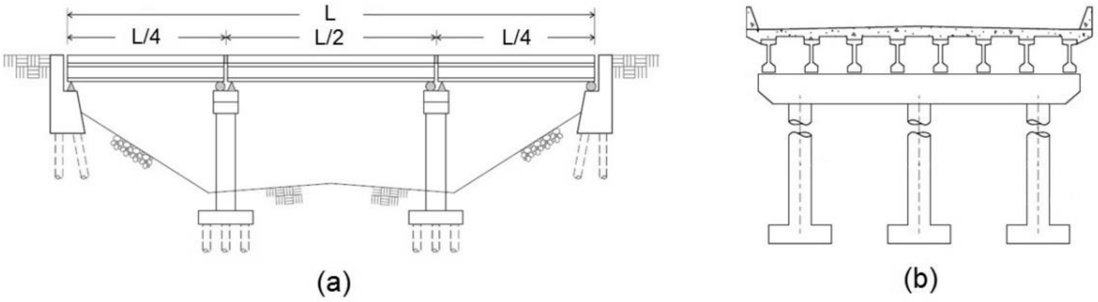

Figure 2. (a) The general overview of the MSSS concrete highway bridge (b) Configuration of the MCSS with girders and deck over the beam (not to scale)

These bridges are usually constructed in three spans in which the length of the middle span is twice of the end ones. The middle span is supported by two Multi-Column Supporting Structures (MCSS) while the end spans bear on a pile type abutment at one end and a MCSS at the other end. The MCSS of these bridges, shown in Figure 2b, is considered for the further analysis in this study. Therefore, several full scale Finite Element (FE) models of the MCSS have been built where the variable parameters are the dimensions, reinforcing details, and the concrete strength. A plastic distribution was assumed for the stress-strain distribution of concrete while the reinforcing steel behaves elastic-perfectly-plastic. Static loads consist of the distributed mass gravity forces and the concentrated vertical loads at the place of bearings over the beam. Also, as for the boundary condition, it is assumed that the lower and surrounding sides of the single footings are fixed in which no displacement and/or rotation happens at these surfaces.

Concrete Fracture Models

In addition to the above-mentioned parameters and modelling variables, the considered concrete fracture mechanism is believed to be a significant factor in the calculated structural capacity. To investigate this issue, two different concrete cracking models are tested in this study. The first one is the discrete crack model for RC structures. This model is aimed to simulate the initiation and propagation of dominant cracks which will cause the failure at later and higher stress levels. This method is theoretically more suitable to capture the failure locations however, propagation of the cracks along the element boundaries causes a *mesh bias* error in which an adaptive re-meshing technique is required (Jendele et al., 2001). The other concrete fracture model is the smeared crack model. It is based on the idea that in concrete, due to its heterogeneity and the presence of reinforcement, many small cracks nucleate which only in a later stage of loading process they link up to form one or more dominant cracks. Since each individual crack is not numerically resolved, the smeared crack model captures the deterioration process through a constitutive relation. Therefore, this model was built on an equivalent continuum concept of elastic degradation and/or softening plasticity within the fixed meshing (de Borst et al., 2004). Both of these concrete fracture models are tested to investigate the seismic capacity of the MCSS of concrete highway bridges.

Nonlinear Dynamic Analysis

A promising technique that has recently risen to estimate the structural responses under seismic loads is Incremental Dynamic Analysis (IDA) (Vamvatsikos & Cornell, 2002). It involves performing nonlinear dynamic analyses of the structural model under a suite of ground motion records, each scaled to several intensity levels, in which the structure is forced all the way from elasticity to final global dynamic instability. Consequently, a single IDA curve of the structural response is generated corresponding to each individual ground motion record which shows the increasing of the damage measure (DM) versus the growing ground motion intensity (IM).

RESULTS AND DISCUSSIONS

Figure 3 illustrates the nonlinear dynamic analysis result of the FE model of MCSS, using a single unscaled ground motion record.

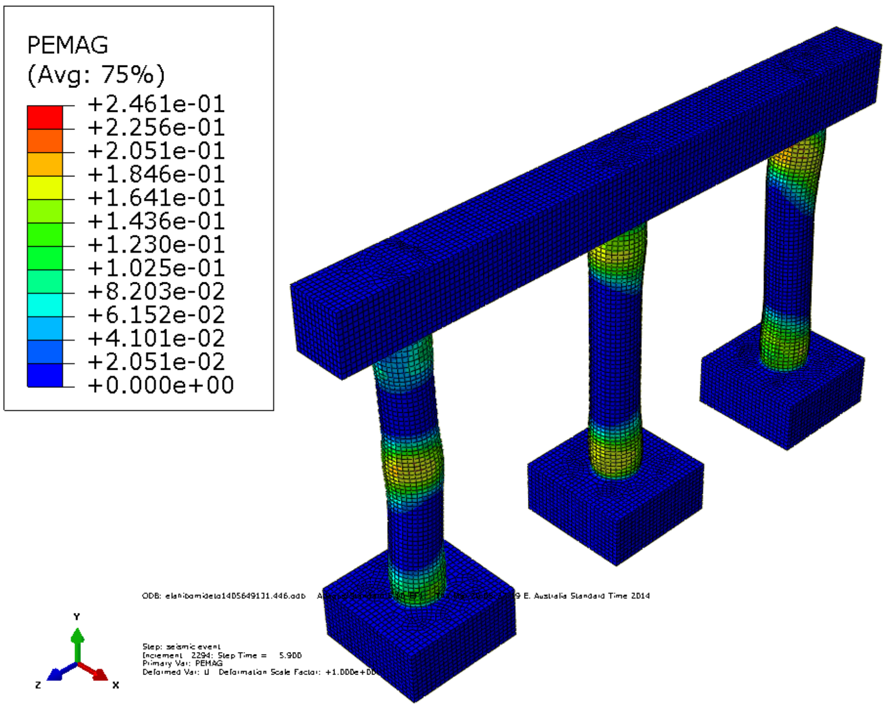

Figure 3. Distribution of the plastic strain magnitude over the FE model

It demonstrates the magnitude of the plastic strain as well as the places and deformation of plastic hinges. The nonlinear dynamic analyses of the MCSS are then performed at increasing levels of IM until numerical the non-convergence is encountered. This exactly is when the structure signals the global dynamic instability or alternatively it reaches to its seismic capacity. Vamvatsikos & Cornell (2002) reported that the corresponding DM usually happens when the slope of the IDA curve decreases to the 20% of the elastic slope, as shown by Figure 4.

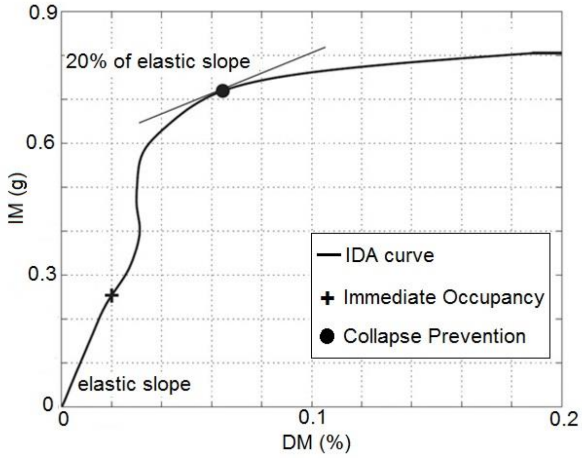

Figure 4. Limit-states, as defined on IDA curves by Vamvatsikos and Cornell (2002)

CONCLUSION

Previous attempts to capture the effects of nonlinear damage accumulation and ground motion duration in structural seismic responses were either inconsistent or accompanied by simplifying assumptions. In this paper, a damage index based on the variation of fundamental period ratio is proposed to be considered. In regard to this, a FE model of a typical MCSS of concrete highway bridges was built for nonlinear dynamic analysis. Further research is in progress to propose a new damage factor. It is anticipated that an innovation could be delivered for seismic design and risk analysis of concrete structures by developing this research approach in future

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