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SEISMIC RELIABILITY OF REINFORCED CONCRETE BRIDGE COLUMNS BASED ON DAMAGE APPROACHES

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Abstract. *The seismic energy response of a reinforced concrete bridge column is dissipated through the initiation of plastic hinges, which represent the resulting damage in the column member. The seismic design criterion SDC that is used by many building codes employs the demand/capacity ductility principle for a performance-based design, which assumes a low damage state in the member, allowing for a better serviceability of the structure after the seismic event. The damage in the plastic hinge, according to the SDC, is limited to spalling and yielding of the longitudinal reinforcement bars, but the crack growth in the column core can lead to excessive strength degradation in the hysteresis behaviour of the structure.*

In this paper, a dynamic non-linear analysis using the Fibre Element Method is performed to obtain the hysteretic analysis for a designed column subjected to different loading rates. A stress-based method is used to determine the local damages in the core and cover of the reinforced column. Different damage states are obtained under different loading rates. The Discrete Element/Finite Element/ method, DE/FEM, with Rotating cracke model is used as a small-scale approach, to investigate the crack growth in the critical zones with softening strains and fractured elements across the concrete column's core and cover. Despite its excessive computation time, the DE/FEM model provides reliable information about the local damage state of the RC column core, which enhances understanding the seismic performance of the structural member.

In this RC column, designed according to the SDC of the California Department of Transportation (Caltrans 2001), a large area of the column core zone at the plastic hinge is damaged when subjected to Lexington Dam, Loma Prieta, 1989, and the column loses its integrity in the plastic zone, which affects the bar/concrete bond substantially. This could lead to total collapse especially if the reinforcement bars are quite exposed and start to buckle.

1 INTRODUCTION

The principle of PBSD performance-based seismic design has played a vital role in Earthquake Engineering. Its significance is to assure that the constructed buildings will resist the effects of earthquake ground motions of different severities within acceptable limiting levels of damage. This implies that the structure will not be damaged beyond certain limit states [1]. In general, the PBSE performance-based seismic engineering has a broad concept which includes the evaluation of damage in structural members, non-structural facilities and also floor contents. In terms of structural members, PBSE is concerned with all aspects of the building process, such as the design criteria, selection of a structural system, layout proportions, detailing of the structural members, construction quality control and long-term maintenance. Therefore, the majority of research work in this concern is associated with determining the different levels of reliability that a building can act under specified levels of excitations [1].

Several levels of reliability could be investigated; such as the material, section, member and the overall structure behaviour. One of the most important is the reliability of reinforced concrete RC bridge columns, which are seismically designed according to the Demand/Capacity principle of seismic design criteria, SDC.

It is a general concept in Eurocode 8 and also other codes that “the bridge should retain its structural integrity and adequate residual resistance after the seismic event”[2]. However, there are structural parts in RC bridges that are susceptible to damage by their contribution to energy dissipation during the seismic event, but the structure should still sustain emergency traffic [2].

Therefore, one of the design principles in bridge engineering is to allow local minor damages in the bridge columns, considering the initiation of PH plastic hinge zones. The concept of a plastic hinge in the design methodology presumes the loss of the concrete cover only, known as spalling, and the initiation of non-linear straining of the longitudinal bars along the PH zone. However, this may not be the case during severe earthquakes. A sever local damage may destroy the concrete core of the column section, and could lead to a total collapse of the structure, especially when the longitudinal reinforcement bars are severely deformed or buckled. The main goal of this research is to investigate the damaged plastic hinges at the core of RC bridge columns when subject to earthquake loading.

Seismic analytical models are used to perform non-linear dynamic analyses, but they may not necessarily predict the damage growth in the elements and its effect on the adjacent elements. However the damage growth mechanism in quasi-brittle 3D continuum is still a complex subject in Material Mechanics Engineering.

In relevance to damage, several earthquake records show that relatively long duration impulses with low frequency have the potential to cause further damage, more than those records having similar PGA’s but with relatively short duration impulses and higher frequency. The phenomenon of long duration and low frequency is known as the Acceleration Pulse, or Fling [3], which increases the seismic hazard and brings more challenges to PBSE in the field of RC bridges design and assessment. Singh [3] considers that having PGA as an Intensity Measure IM alone is a poor parameter for evaluating the damage potential.

In addition to damage, two other important issues are also crucial to PBS design for RC bridge columns, and should be taken into consideration; the residual displacement after an earthquake, and the structure’s displacement exceeding the allowable lateral displacement stated by building codes. These two issues, if not considered, can also reduce the seismic performance of the structure, even in case of low damage levels. These issues are not within the scope of this work.

In this paper, using two different methods for non-linear analysis, the local damage of plastic hinges PH is investigated analytically in a RC bridge column under seismic loading. They are; the Fibre Element Method and the combined Discrete Element/Finite Element Method.

2 LOADING RATE

Different loading rates can have significant effects on the performance of a RC structure. The uncertainty of intensity and rate of earthquake loading increases the challenge to predict responses of high risk excitements. In general, seismic loads can be relatively classified by the acceleration rate into different acceleration pulses.

2.1 Acceleration Pulse

Two useful classifications for ground acceleration records are relevant to the expected damage in the structures. The first is the phenomenon of long duration impulses with low frequency in ground acceleration records, which is known as the Acceleration Pulse, or Fling. The second is the acceleration peak associated with short duration impulses of high frequency, and known as the Acceleration Spike [3]. It has been found that an Acceleration Spike is not as severely damaging to the RC bridge columns as an Acceleration Pulse [3].

The reason behind having such a high potential damage in a long duration impulse is that it allows for a higher velocity and thus higher displacement response. However, short duration impulses in a record of high frequencies, i.e. acceleration spikes, can also be very damaging if their high frequencies are within the range of the structure's natural frequencies. However, PGA of high frequencies can seldom initiate resonance or produce large scale damage since most structures are not within the range of high frequencies records [3]. Therefore, large PGA alone can seldom initiate resonance or produce large scale damage.

Many researchers have shown that the frequency, duration, incremental velocity and incremental displacement can have profound effects on the structural response more than the PGA, especially in the inelastic range [3]. E Cosenza and G Manfredi stated that the PGA is a basic measure of earthquake potential but is not totally reliable [4]. Examinations of recorded seismic events have shown that earthquakes with a very large PGA could not produce appreciable structural damage, while earthquakes with very low PGA's produced an unexpectedly high level of destruction [4]. Instead, the PGV seems to be a more representative measure of earthquake intensity as it is directly connected with energy demand[3].

3 DAMAGE APPROACHES

3.1 Stress-based Damage

Damage can be estimated by measuring the loss of stresses at the critical zones in a plastic hinge. In inverted pendulum problems, such as bridge column problems, most of the damage is due to excessive axial compressive and tensile strains. Thus, classified as a flexural damage. However, a very limited portion of the damage is caused by shear failure in these problems, especially in relatively small diameter members, therefore, no shear failure is expected.

The elasto-plastic constitutive relationship for a selected element can be used to indicate the damage state at that zone approximately. The Local compressive damage index for concrete fibres is based on the ratio between axial compressive stresses $\sigma_{i,fibre}$ and the ultimate strength of concrete σ_{ult}, and can be obtained during the strain softening of the analysed fibres, as in the following equation:

$$D_i = 1 - \frac{\sigma_{i,fibre}}{\sigma_{ult}} \tag{1}$$

where i is the time-step, or pseudo time in case of quasi-static analysis. When D_i equals 1, the fibre has lost its strength and is not capable to resist any more axial compressive stresses, indicating a local totally damaged state under compression.

This index is sufficiently expressive, but it is mostly used for elements under compressive stresses only since that concrete elements with tensile stresses are considered fully damaged due to their very limited strength to resist tension.

Using the Fibre Element Method, the non-linear analysis for the RC columns is performed, by using the SeismoStruct dynamic solver [5], which is capable of plotting the constitutive curves of the stressed fibre elements. The fibre elements are designed to compute the non-linear axial forces with the flexible failure mode. However, shear forces are also obtained from the coupled stiffness matrix, but their corresponding shear stresses are not calculated since the shear failure mode in these problems is not dominant.

The following example for RC single-cell box-girder bridge columns, shown in figure 1, has a damping ratio of 5% and subjected to artificial ground accelerations applied at the base of the structure. The relative change in the duration of acceleration pulses of ground acceleration is conducted in 3 different slope rates; 1.414 g/s, 1.880 g/s and 2.801 g/s, where g is the gravity constant, as shown in figure 2. These loading rates have been taken based on the PGA of Lexington Dam record from the Loma Prieta earthquake 1989, which reaches approximately 6.0 m/s^2.

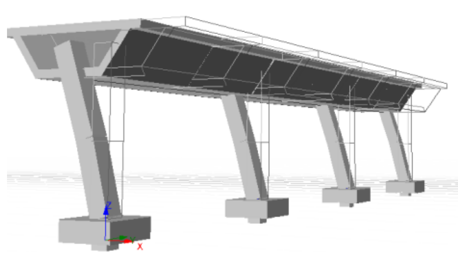


Figure 1, Displacement in RC bridge columns

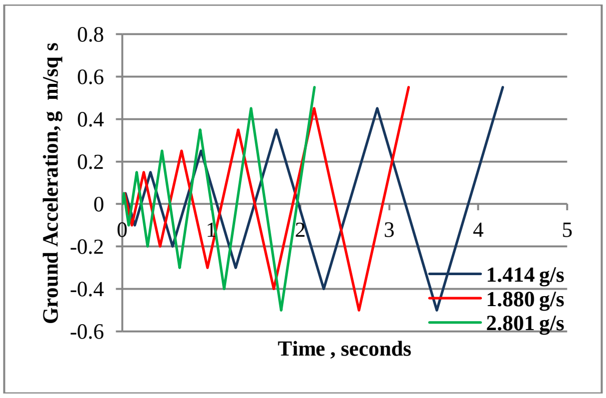
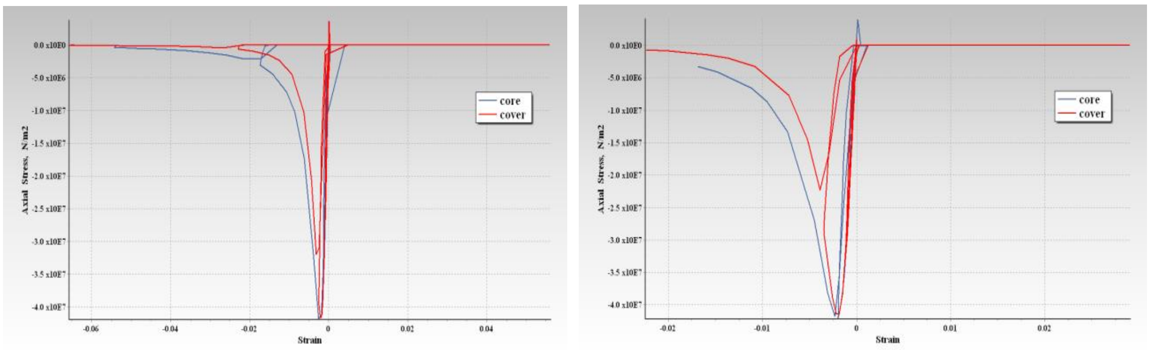


Figure 2, Loading rates

As a nonlinear response to the effect of different loading rates, the constitutive curves shown in figures 3,4 and 5 show that for longer durations of an acceleration pulse the response tend to have more plastic stresses, and for shorter durations the response tends to have less plastic stresses. The corresponding damage can be then determined for the stressed fibres at selected points on the cover and core of the column's section, using equation (1). The loading rate of 1.414 g/s, (longer duration loading), showed an extended constitutive curve with large plastic strains and degraded strength on the core and cover in figure 3. In figure 4 less plasticity is expressed on the core and cover with 1.880 g/s loading rate, and almost linear constitutive curves are found with 2.801 g/s loading rate, (shorter duration loading).

This indicates that much less damage occurs with larger loading rates, and more damage occurs as the loading rate decreases.



Figures 3&4 stress-strain curves at 1.414 g/s and 1.88 g/s loading rates

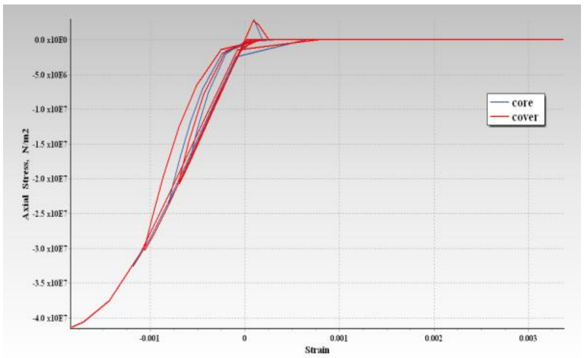


Figure 5 stress-strain curves at 2.801 g/s loading rate

3.2 Fracture-based Damage

This method is based on modelling the fractured elements of the model by using a DE/FE Explicit Dynamic solver. The Explicit-Elfen code is used to perform the non-linear dynamic analysis for a limited time of applied loading, since fracture analysis takes a relatively large computational time to attain the analysis of few seconds of loading.

3.2.1 Failure and Fracture Model

The non-linear dynamic analysis in this approach is governed by Mohr-Coulomb/Rankin with tension cut-off model, covering both tensile and shear failure modes, Mode (I) and Mode (II), respectively. The failure model is characterised by shear strength, angle of friction, angle of dilation and tensile strength. The fracture model is characterized by tensile strength and fracture energy, to simulate the tensile cracking mode Mode(I) only, and is known as the Rotating Crack model. Mode (I) is suitable to represent the cracks in the column's dynamic oscillation motion, since the fracture in the column base is mostly due to tensile cracking mode.

The applied dead load in the proposed example is 4.5MN, which is only 5% of the column's capacity for axial load, and thus, the confinement reached by the transverse reinforcement stirrups is found to be only 4.5% of the steel yield stress f_y. This leads to less confinement, and thus, the principal stresses of the concrete become closer to the failure envelope, and concrete is more vulnerable to fail.

In this FE analysis the bond effect is not simulated since that 2D steel bar elements are fully conjugated with the edges of the tetrahedral 3D concrete elements. In general, bond friction could have some effect on the fracture mechanism and crack growth, but its existence could also increase the computational effort significantly.

3.2.2 Fracture Energy

The fracture energy is defined as the amount of energy needed to create a continuous crack on a unit area, and it is the equivalent alternative to the softening law. The fracture energy for a controlled volume, often chosen to be the finite element, is the area under the softening curve, as shown in figure 6. Modelling wise, if the stresses have not dropped to zero in the softening stage, the area under the softening curve is less than the assigned fracture energy G_f, and the material is partially damaged, i.e. the Failure Factor is assigned between 0 and 1, and the controlled volume is under micro-cracks but no cracks are initiated yet. If this area is equal to the fracture energy, the material is totally damaged, i.e. Failure Factor=1, and cracks start to initiate.

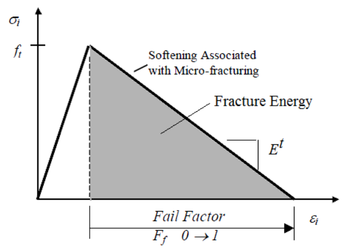


Figure 6 Fracture energy under softening curve

The release of the fracture energy rate ∂G_f is dependent on the degree of damage caused during the softening stage, which is defined as:

$$\partial G_f = \sigma . du \tag{2}$$

$$G_f = \int \sigma . du = \int \sigma . \varepsilon(s) . ds \tag{2'}$$

Where σ is the stress, u is the displacement and $\varepsilon(s) = \frac{du}{ds}$ is the softening strain in the direction s of the principal plane. Integrating over a localization band l_c for a constant slope softening model, this gives:

$$E^t = \frac{\partial \sigma}{\partial \varepsilon} = -\frac{f_t^2\, l_c}{2G_f} \tag{3}$$

where, l_c is a function of an element area and f_t is the concrete tensile stress, and the negative sign is for the modulus slope. The fracture energy is used to define the softening curve E^t, and the resulting area under the curve is either less than or equal to the energy fracture G_f. Once the area under the curve reaches G_f, which is usually 100 to 150 N/m for concrete, the fracture energy of that point is said to be released, i.e. work of "softening strains" is completed during the softening stage at that plastic zone, and a crack initiates.

Fracture is now introduced using an algorithm that updates the topology of the mesh through insertion of discrete fractures in the “failed” regions. A visible crack is then allowed to initiate, and also propagate.

3.2.3 Problem Set-up for the FE Analysis

As equivalent to the PGA motion of Lexington Dam record, from the Loma Prieta earthquake 1989, loading is applied on the RC structure, which has a damping ratio of 5%. This force is applied at the side surface of the top mass, and its rate of loading should be similar to that of the ground acceleration. The required axial loading is due to the dead load which is modelled by having an artificial mass structure with density and volume producing an equivalent loading effect. For less computation efforts in the analysis, the following procedures have been taken:

1- Applying only half structure since both geometry and loading are symmetric about the xy vertical plane.
2- Excluding modelling of reinforcement stirrups apart from the PH zone, since the confinement of concrete core is more important in that zone.
3- Out of the total record time of 40 seconds, only partial loading with the PGA value is selected from the Lexington Dam record of Loma Prieta earthquake 1989. The maximum loading lateral force is approximately 2.2e6 N, and the corresponding time is from 3.48 seconds up to 5.48 seconds, lasting for 2.0 seconds only. This applied peak forces vary in rate, from 0.70 to 2.0g per second, as shown in figure 7.

Another dynamic loading rate is applied, with a loading rate of 2.27g per second on the same example, to compare its analysis with the previous one, and discover the influence of rate change.

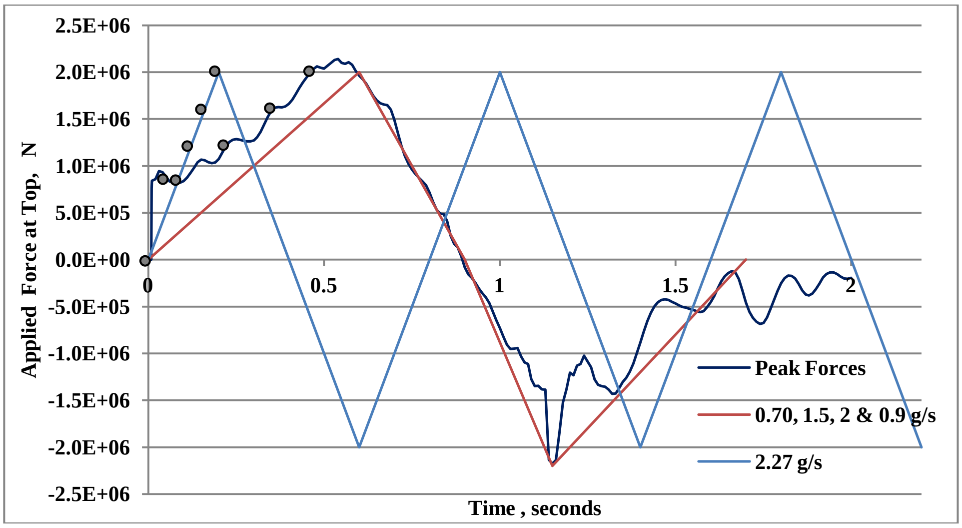


Figure 7: Different rates of applied forces at top of structure.

3.2.4 Results and Discussion

Prior to comparing between the problem responses of different loading rates, it is useful to compare the performance of the two dynamic solvers; the DE/FE fracture analysis using the Explicit-Elfen compiler and the SeismoStruct analyser. Figure 8 shows the load-deflection curves by SeismosStruct, for the quasi-static and dynamic nonlinear analyses for the RC column problem, subjected to Lexington Dam record of the Loma Prieta earthquake 1989 in the dynamic case. It also shows the load-deflection curve by the DE/FE fracture analysis for the same problem, but subjected to only 2 seconds of the peak forces of the Loma Prieta earthquake. Due to severity of damage, the structure is deflecting towards an unstable position as the plastic hinge PH becomes severely fractured. Comparing this with the response of SeismoStruct analysis for the same problem, the linear stiffness of the SeismoStruct curves have a good agreement with the Elfen response, but differences are noticeable in the non-linear behaviour and during the hysteretic stages. This concludes that under strong ground motion, with peaks such as PGA reaching approximately 6.0 m/s^2, the damage could be very severe and leads to a total collapse of the structure.

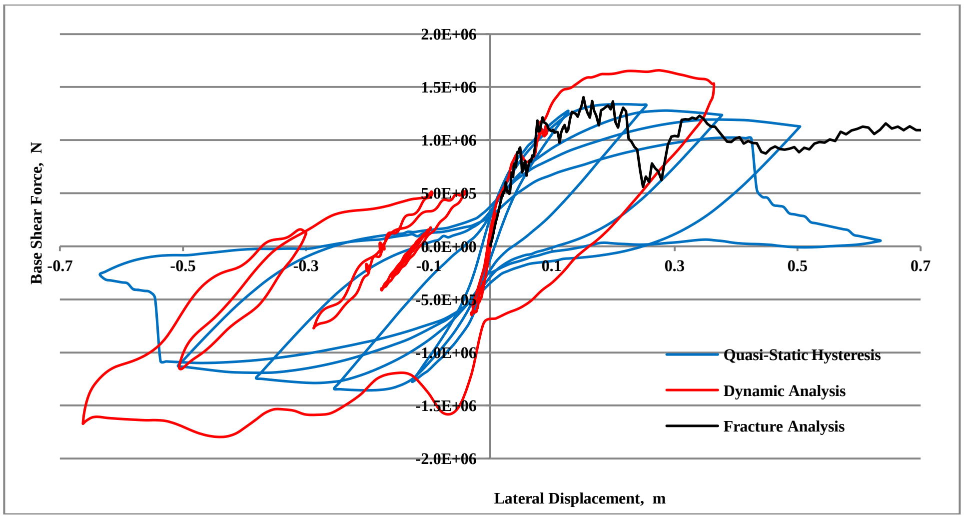
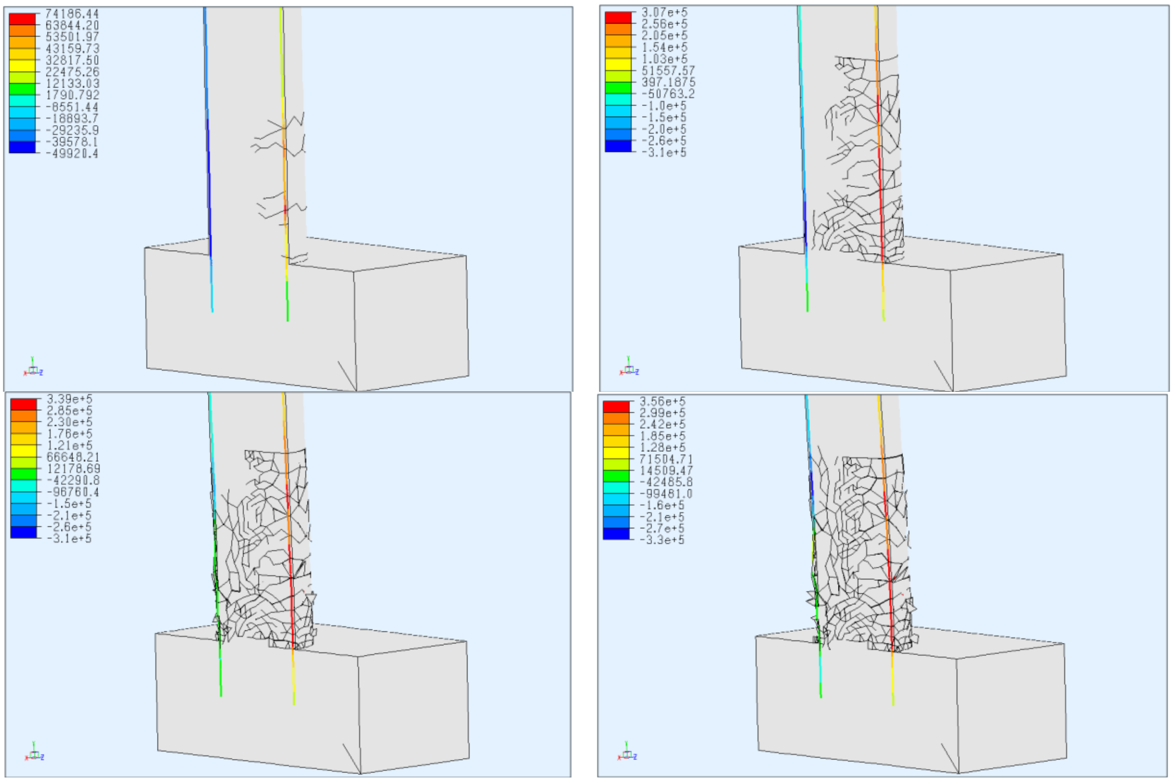


Figure 8, Load-Deflection curves by different analyses for the structure under Loma Prieta earthquake.

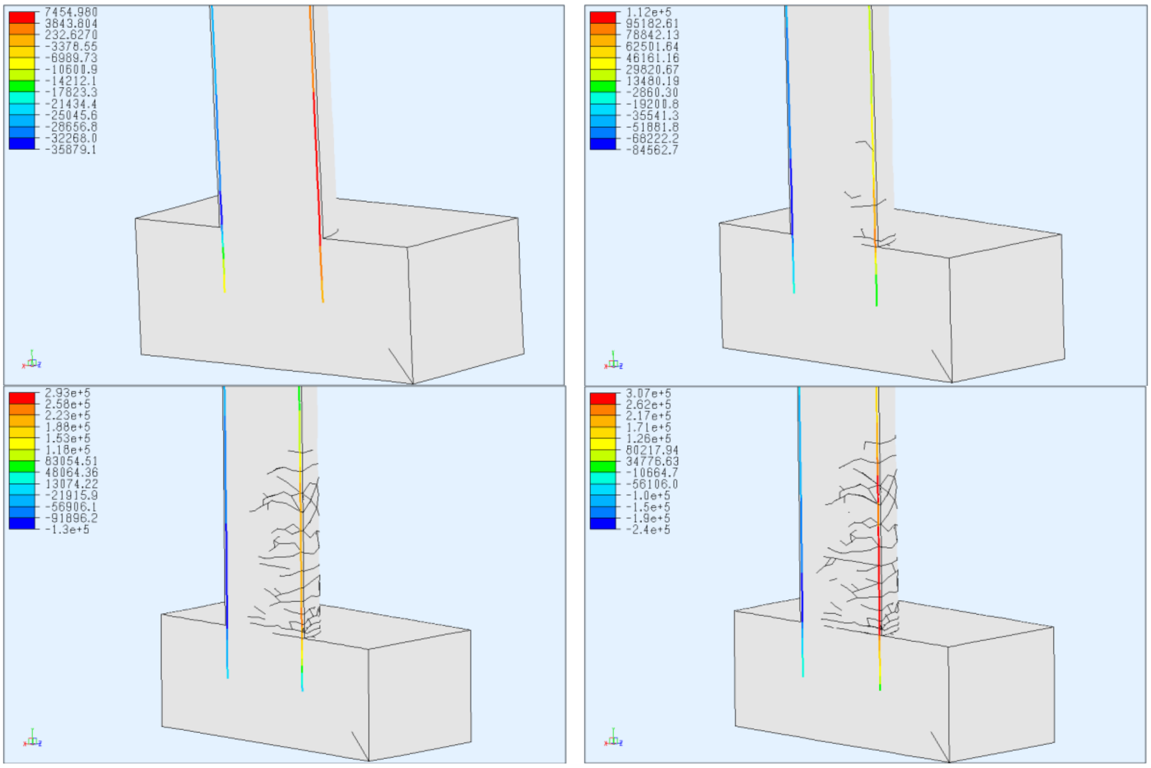
Under different rates of loading, different responses have been obtained for the same lateral loads that are applied on each problem, and marked by points on the in figure 7. Table 1 shows the responses of time, displacements, degree of damage and bar tensile stresses for long and short duration lateral loads. The 4 applied loads; 0.821, 1.208, 1.619 and 2.03 MN, care selected to show the damage that's pictured in figures 9, 10, 11 and 12 for the problem with case(A) of longer duration.

Applied Lateral Load (MN)	**Case (A): Long Duration, Low Frequency Acceleration Pulse, 0.70g per second.**			**Case (B): Short Duration, High Frequency Acceleration Spike, 2.27g per second.**		
	Time (s)	**Lateral Displacement (m)**	**Bar Tensile Stress at PH, MPa**	**Time (s)**	**Lateral Displacement (m)**	**Bar Tensile Stress at PH, MPa**
0.76	0.01	0.0163	-9.92	0.076	0.002	1.9
0.821	0.09	0.04093	98.7	0.082	0.007	11.5
1.04	0.14	0.10242	269.0	0.104	0.0141	94.7
1.208	0.22	0.24028	474.8	0.121	0.022	121.9
1.44	0.33	0.29027	491.8	0.144	0.0365	221.1
1.619	0.36	0.43260	524.3	0.162	0.0517	399.0
1.83	0.43	0.43260	524.3	0.183	0.074	475.0
2.03	0.47	0.53106	550.6	0.2	0.095	475.0

Table 1: Responses of RC column under long and short duration applied lateral loads.



Figures: 9, 10, 11 and 12; Concrete fracture and steel tensile forces due to applied loads of 0.821, 1.208, 1.619 and 2.03 MN, respectively, for longer duration case (A).



Figures: 13, 14, 15 and 16; Concrete fracture and steel tensile forces due to applied loads of 0.821, 1.208, 1.619 and 2.03 MN, respectively, for shorter duration case (B).

The states of damage caused by the same 4 applied loads but with shorter duration of loading, case (B), are pictured in figures 13, 14, 15 and 16. It is obvious that cracks tend to grow densely in the long duration case more than in the short duration case. As the structure is symmetrically modelled and loaded, the pictures show an obvious grow of cracks inside the column core itself, especially with at large loading. Such cracks are obtained for the peak loads of that record only, but more cracks could accumulate if the rest of the loads of the record are added.

Damage is the result of lateral displacements, and the maximum displacement in case (B) is approximately 20% of that in case (A) even though both are subjected to the same load value, but with different loading rates.

In respect of tensile stresses in the reinforcement bars, stresses are consequently less in case (B), and their ductility is less consumed than in case (A). However, bar tensile stresses in case (B) rise fast as loading rises, and then reach 86% of the tensile stresses in case (A). This shows that a huge part of the seismic energy is dissipated by the steel reinforcement bars, causing less damage to the concrete body. At displacement of 0.095m, the tensile stresses in case (B) reaches 475MPa, while a displacement of 0.102m, the tensile stresses in case (A) does not exceed 269MPa. This is because that less damage is found in case (B), and therefore, more concrete-steel bond exists in the context of the plastic hinge, while in case (A) more crack growth formation with less concrete-steel bond exist, and thus, less tensile stresses may produce. This indicates that ductility is affected and the member does not follow the demand/capacity principle sufficiently in the nonlinear stage. Therefore, the SDC seismic design criterion based on this principle could fail due to lack of ductility.

4 CONCLUSION

- The damage of quasi-brittle materials such as concrete is very sensitive to the rate of loading, and the inconsistency of loading rates of earthquake motion makes the damage pattern in such RC columns with low confinement unpredictable and difficult to generalize. Therefore, it is very much recommended to analyse each loading case independently for an approximated fracture simulation.
- At responses of approximately similar displacements, the bar tensile stresses reach 475MPa and 269MPa in the two nonlinear analyses; with short duration and longer duration loads, respectively. This is because that less damage is found in the short duration loading case, and therefore, more concrete-steel bond functions in the context of the plastic hinge, while in the longer duration loading case more crack growth is formed with less concrete-steel bonding, and thus, less tensile stresses is produced.
- The SDC seismic design criterion that’s used by many building codes is based on the principle of seismic demand/seismic capacity balance. This principle functions effectively in both linear and non-linear cases, but requires an effective ductility of the members to function properly during the non-linear stage. However, in quasi-brittle materials this principle lacks to sufficient ductility of the members, since concrete cracks cause less concrete/steel bond, and thus, the steel bars do not reach sufficient ductility. In such case the seismic demand/capacity principle is not sufficiently fulfilled.

- Confinement action could prevent much of crack penetration inside the concrete column core, however, the formation of confinement stresses around the core is a function of the axial load on the section, and it produces the balance between outward strains of the concrete and inward stresses of the steel hoops. If the axial loads are not sufficient, very low confinement is produced, and therefore, more cracks may penetrate inside the column core.
- RC columns supporting single-cell box-girder bridges are vulnerable to high risk damage at their plastic-hinge zones, since they have less confinement action due to low axial stresses and are subject to strong ground motion or long duration ground acceleration.

- For such structural systems to function according to a performance-based seismic design when subjected to strong ground motion, they must be provided by one of the energy dissipative devices such as seismic isolation systems or damping braces. Alternatively, they should be provided by some sufficient means that enhance their confinement action such as carbon-fibre confinement sheets.

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