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Evaluation of effect of confinement on the collapse probability of reinforced concrete frames subjected to earthquakes

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Abstract

Confinement of reinforced concrete members has repeatedly proven crucial when structures are exposed to strong ground motions. Yet, unconfined concrete is still widely used in developing countries with significant seismic hazard. To quantify the effect of lack of confinement on collapse probability of reinforced concrete frames, incremental dynamic analyses were conducted on two 4-story reinforced concrete moment resisting frame buildings using 5 ground motions. The structures are identical and comply to Eurocode demands except for the fact that one is completely unconfined whereas the other is confined according to the code. The analyses were conducted in OpenSEES using Mander's material model for confined and unconfined concrete. Based on the results of the analyses, collapse fragility curves were created for each structure. The results show statistically significant effect of confinement on the collapse probability of the reinforced concrete frame. The prabability of collapse is 1.2 and 12 %, respectively, for the code complying and the unconfined structure exposed to a design level earthquake.

Keywords: Confinement; incremental dynamic analysis; collapse fragility curves

1. Introduction

Confinement, i.e. closely spaced stirrups in concrete elements, serve several purposes in seismic design. Besides increasing the shear capacity, it keeps both longitudinal reinforcement and core concrete in place during severe deformations. It thus increases the ductility of the member, and prevents collapse due to crushing of the concrete core and subsequent buckling of the longitudinal rebars [1]. However, lack of confinement is a typical structural deficiency in earthquake prone regions. The aim of this paper is to investigate how lack of confinement affects the probability of collapse for a building. This is done through incremental dynamic analysis (IDA), which is a set of nonlinear response history analyses, where the response from one or multiple records, scaled up and down, is plotted in the same graph [2]. The use of multiple records enables statistical evaluation of the performance of the structure through fragility curves.

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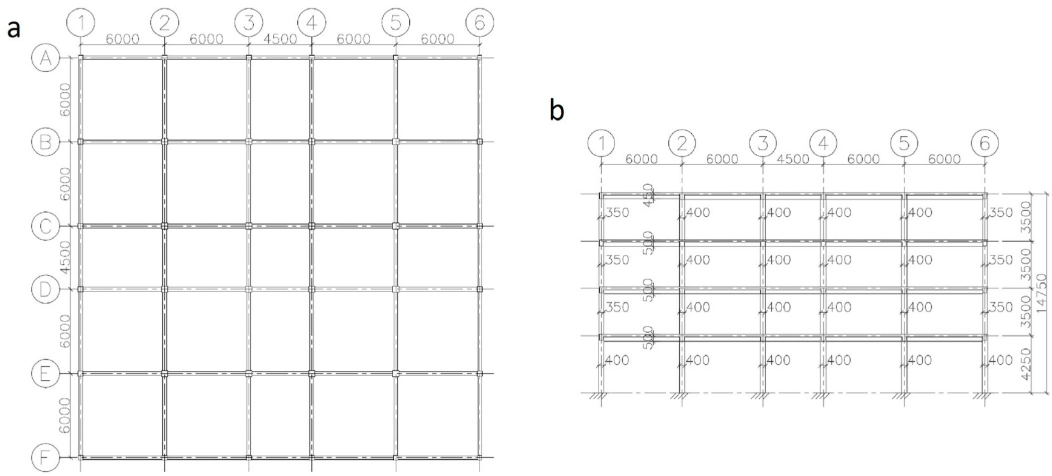

Fig. 1. (a) Plan view of the structure; (b) Vertical projection of the structure. All columns are square

The IDA plot consists of an intensity measure (IM) and an engineering demand parameter (EDP). The IM is a non-negative scalar or vector that depends on an accelerogram, and is increased until collapse is reached [2]. Several different intensity measures have been proposed during the past decades. Peak Ground Acceleration, Peak Ground Velocity, 5% damped Spectral Acceleration at the buildings fundamental period ($S_a(T_1, 5\%)$), and the spectral deformation at the fundamental period, $S_D(T_1)$ are examples of scalar IMs. Researchers have found excellent results with the application of the vector ($S_a(T_1)$, ϵ) as IM, where ϵ is a measure of the difference between $S_a(T_1)$ and the median of spectral acceleration predicted by the attenuation equation at T_1 [3,4]. The spectral acceleration of the first mode was chosen due to the fact that it is an efficient IM, and simple to compare for different structures [5].

Fragility curves illustrate the probability of violating a certain limit state, in this study the probability of collapse, as a function of the selected IM. They are based on the results of an IDA which provide the IM at collapse when a structure is exposed to different ground motions. In order to determine the fragility curve, the shape of the curve must first be assumed. A lognormal distribution has been shown to provide best predictions for IDA [5,6]. The fragility curve is thus determined as the lognormal curve with the standard deviation of the collapse intensities found in IDA, prescribing 50 % probability of collapse for the median observation in the IDA. In this study, fragility curves will be determined for the probability of collapse of two 4-story reinforced concrete moment resisting frame buildings. One is designed according to the DCM provisions in EN 1998-1-1. The other is identical but lacks confinement. Comparison of these fragility curves is used to quantify the effect of confinement on the investigated structure.

2. Investigated structures

The investigated structural system is illustrated in Figure 1. It is a residential building assumed to be situated in southern Europe. All columns are square. The thickness of the slabs is 150 mm, and the beam width is 300 mm except for the roof level where it is 260 mm. The structure is double symmetric, and was modeled in two, rather than three, dimensions. This reduces computational time.

During an earthquake, the frame with the largest tributary load will be the critical part of the structure because of larger axial loads in the columns. For the structure in Figure 1, the critical frame is hence that in axis B (which is identical to those in axis E, 2 and 5). Thus, only the frame in axis B is investigated.

The vertical beam loads in the seismic situation were determined according to EN 1990 [7]. The peak design ground acceleration (PGA) was chosen as 0.35 g, i.e. 3.5 m/s^2. This is representative for southern Europe according to the probabilistic seismic hazard assessment for Euro-Mediterranean region [8]. The ground type chosen is C in EN 1998-1 [9]. The corresponding shear wave velocity is in the range of 180-360 m/s. Only the response spectrum Type 1 in EN-1998-1 is applied since the structure is situated in a high seismicity region.

Concrete quality C30/37 with 25 ± 10 mm cover is chosen. The concrete strength is thus larger than the minimum requirement for residential buildings according to EN 1992-1-1 [10]. However, it was chosen to avoid excessive column dimensions. Steel quality B500C is applied for the reinforcement.

The code complying structure is designed for medium ductility, denoted DCM in EN 1998-1. This is the standard ductility class in the code. The other structure is identical except that it lacks confinement. Confinement does not affect the stiffness of the concrete members, as the natural periods are identical for the two structures. The fundamental period is 0.8 s, which means that the elastic spectral acceleration is 5.5 m/s^2 according to EN 1998-1.

3. Numerical model

The effect of confinement on the reinforced concrete stress-strain relationship was determined according to Mander et al. (1988) [11]. The mean concrete strength is 38 MPa according to EN 1992-1-1. However, the concrete strength is affected by the strain rate, set to $0.0167s^{-1}$ which is representative of seismic loading [12]. Thus, the unconfined concrete strength is 44.6 MPa. The tensile strength of concrete was determined according to the equation provided in EN 1992-1-1 Table 3.1. The ultimate strain of unconfined concrete was set to 0.006, which is a typical value found in tests on reinforced concrete frame members [13]. The stress strain relationship of reinforcing steel was idealized as bilinear with yield strength and strain hardening ratio equal to 500 MPa and 5 %, respectively, in accordance with EN 1992-1-1.

Fiber sections were used in the IDA. The number of integration points is set to at least three for all members, to ensure sufficient accuracy and computational efficiency. It was however increased for members with larger axial load, up to six points for the columns in the first floor. Gauss-Lobatto quadrature integration is applied, as the placement of integration points at the element ends decreases the number of integration points needed to provide accurate results at the joints [14,15].

Rayleigh damping was applied with 5 % damping of the first mode and $T = 0.2\,s$, using the tangential stiffness matrix as recommended in [16].

4. Ground motion selection

The selection of representative ground motions is a crucial part of the non-linear response history analysis as each record will lead to different collapse probabilities. The goal is to choose sufficient ground motions to represent this variety. However, each record should be realistic for the site, i.e. be recorded on site conditions that are similar to those of the analyzed structure, since this affects the ground motion characteristics [13,17,18]. The intensity level of an earthquake likewise affects the characteristics of the ground motion, and each chosen event should therefore be representative of the limit state considered [18]. The investigated intensity in the current study is collapse, and the selected earthquakes should therefore be intense enough to make a modern, code complying structure approach collapse. Such events are extremely rare, so it is necessary to choose records from less intense ground motions and scale them up to collapse intensity. This is an unavoidable approximation that leads to uncertainties in the results [19].

Thus, the selection was based on the following criteria:

- Solely horizontal far-fault recordings are considered, i.e. the distance from the site to the fault rupture should be greater than or equal to 20 km, as prescribed in FEMA P695.
- PGA $\geq 2.00\,m/s^2$. This is chosen to represent the threshold of structural damage for new buildings in accordance with the recommendations in FEMA P695.
- $180\,m/s \leq V_{s30} \leq 360\,m/s$. This is the range which the code compliant structure is designed for.
- Moment magnitude larger than 6.5. Events with magnitudes lower than this are not likely to collapse new structures [18].
- To prevent bias, only one recording from each event was used. Wherever more than one record satisfied the demands listed above, the record with largest peak ground velocity was chosen. This is in accordance with FEMA P695.

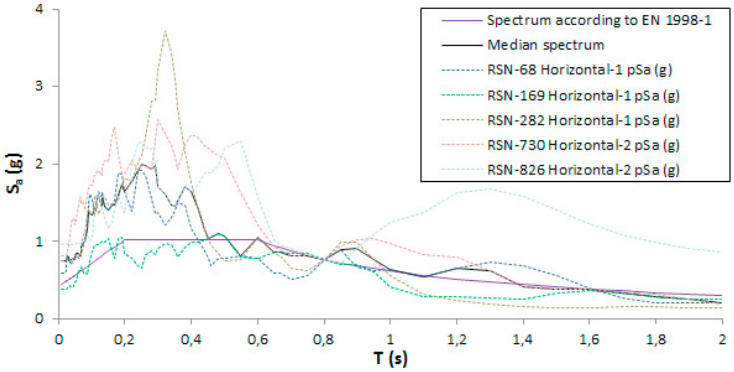

Fig. 2. Scaled response spectra of the selected ground motions as well as the median spectrum and the elastic response spectrum according to EN 1998-1.

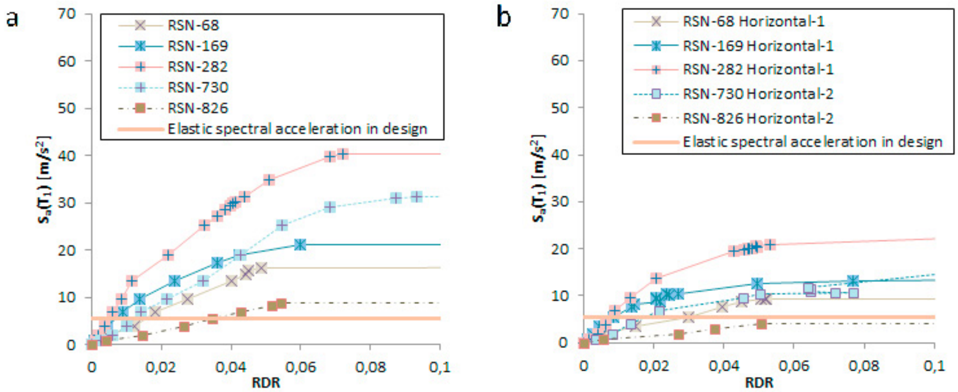

Fig. 3. (a) IDA for code complying structure; (b) IDA for unconfined structure.

In this study, the earthquake ground motion records are taken from the NGA-West2 database at PEER [20]. Table 1 lists the chosen ground motions.

Table 1. Selected ground motions.

RSN	Earthquake Name	Year	Mag.	Rrup (*km*)	Vs30 (*m/sec*)	Unscaled PGA (*g*)	Hor. component
68	”San Fernando”	1971	6.6	23	316	0.31	RSN68_SFERN_PEL090
169	”Imperial Valley-06”	1979	6.5	22	242	0.43	RSN169_IMPVALL_H-DLT262
282	”Trinidad”	1980	7.2	76	312	0.22	RSN282_TRINIDAD_B-RDW000
730	”Spitak Armenia”	1988	6.8	24	344	0.27	RSN730_SPITAK_GUK090
826	”Cape Mendocino”	1992	7.0	42	337	0.24	RSN826_CAPEMEND_EUR090

The scaling method used in this study is that recommended in FEMA P695, i.e. scaling to the spectral acceleration at the fundamental period. Figure 2 shows response spectra scaled to match the design spectrum at the 1^{st} natural period, $T_1 \approx 0.8s$. Large scatter is evident. The variance between the records affects the results both due to period elongation and higher mode effects.

5. IDA

The Hunt and Fill algorithm was applied in the IDA [5]. The resulting curves for the roof drift ratio (RDR) are illustrated in Figure 3. The effect of confinement is clear from the figure, as all five ground motions result in lower

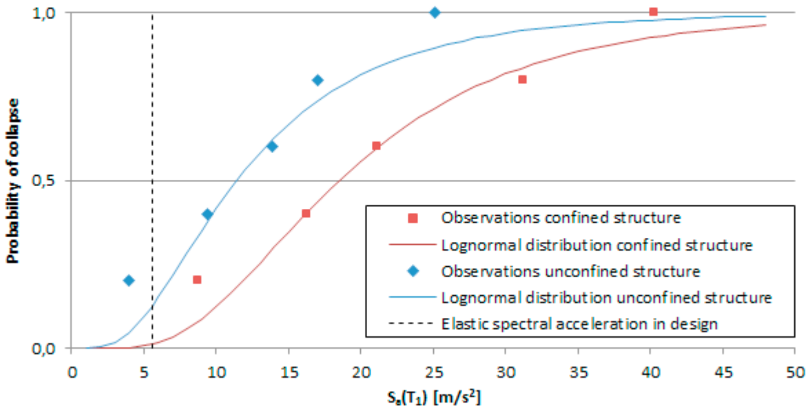

Fig. 4. Fragility curves for code complying and unconfined structure.

collapse capacity with unconfined sections.

The large scatter in the response spectra for periods larger than 0.8 s, ref. Figure 2, results in similar scatter for the IDA curves. Thus, ground motion RSN-826, which have largest spectral acceleration for periods longer than the 1^{st} natural period leads to collapse at lowest spectral acceleration for both structures. Notice that the collapse capacity for this ground motion is lower than the elastic spectral acceleration according to EN 1998-1 for the unconfined structure (Figure 3b). Similarly, ground motion RSN-282 have lowest spectral acceleration for natural periods larger than 1.2 s, and results in largest spectral acceleration at collapse for both structures.

The order in which the ground motions collapse the structure is identical in Figure 3a and b, but the variance in the collapse spectral acceleration is larger for the code complying structure compared than with unconfined sections. This is especially clear for the RSN 169 and 282 records, which have practically identical collapse capacity in Figure 3b, but differ by a factor of 1.5 in Figure 3a.

6. Fragility curves

Based on the results of the IDA, fragility curves were created for collapse probability of the structure based on the median spectral acceleration and standard deviation from results. E.g., it was found from the IDA that the probability of collapse is 40 % at $S_a(T_1) = 9.40\, m/s^2$ and 60 % at $13.9\, m/s^2$ for the unconfined structure, and the fragility curve thus indicate 50 % risk of collapse at $S_a(T_1) = e^{ln(9.40+13.9)} = 11.5\, m/s^2$. The curves are illustrated in Figure 4, and can be used to compare the collapse probability of the two structures.

As is evident from the figure, the probability of collapse is only 1.2 % for the code complying structure exposed to a design level earthquake. The low probability of collapse is partly a result of the fact that the cross section sizes, concrete quality and reinforcement ratios of the structural members were to a large extend determined due to minimum requirements in the design code, and the overstrength of the structure is thus large. However, the probability of collapse of an identical structure with unconfined sections is 12 %.

The above observations imply substantial effect of confinement, but the calculations are based on relatively few observations. Thus, a paired t-test was performed to investigate if the results are statistically significant. In this test the difference in collapse probability for the two structures excited by the same ground motion is evaluated. This is done for each of the five ground motions, and the probability that confinement does not affect the collapse capacity of the structure is calculated. It was conducted on the logarithmic values of the spectral accelerations at collapse for each of the structures. The logarithmic values were applied due to the fact that t-tests are based on theory for normally distributed outcomes [21]. The two-sided P-value is 0.00085, e.i. the probability that confinement does not affect the collapse capacity of the structure is 0.085 %.

The ground motions with lowest and largest spectral acceleration for both structures provide observations to the left of the fragility curves. This indicate that the shape of the lognormal distribution is not ideal for the observations, and it is noteworthy as lognormal distributions have previously been shown to provide good agreement with observations in IDA. This peculiarity is probably a result of the large spectral accelerations for the RSN-826 ground motion for periods larger than the 1^{st} natural period of the structures in the elastic range. It is thus expected that the fit between the shape of the fragility curve and the extreme observations would be better if more ground motions had been applied in the structure such that the impact of RSN-826 would be smaller.

7. Summary and conclusion

IDA was performed for five ground motion records on two moment resisting reinforced concrete frames in order to investigate the effect of confinement. All five investigated ground motions gave lower collapse capacity with unconfined sections compared to confined sections. Through determination of fragility curves, the probability of collapse for the unconfined structure was found to be ten times that of the code complying structure when subjected to the design level ground motion intensity. A paired t-test showed that the probability that confinement does not affect the collapse capacity of the structure is 0.085 %. It is thus concluded that confinement increases the collapse capacity of the investigated structure.

The scatter in the results is large, and the lognormal shape of the fragility curve does not represent the results well. Analyses with more ground motion records should thus be completed.

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