

STATISTICAL DISTRIBUTION OF FLOOR RESPONSE SPECTRA DUE TO DIFFERENT TYPES OF EARTHQUAKES GROUND MOTIONS

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ABSTRACT:

Recent earthquakes have demonstrated that damage associated with Non-Structural Components (NSC) constitutes a substantial amount of total loss. Following this, the study of non-structural component has constituted an important component of Performance-based Earthquake Engineering (PBEE). Earthquake loading standards NZ 1170.5:2004 has introduced new provisions for the design of NSCs and building parts. The current provisions include factors for peak floor acceleration along the height, response amplification for different periods of component, ductility of the component itself and its fixings and a risk factor to reflect the failure consequence which is quite different from overseas counter parts. In this study, the response of a ductile low-rise reinforced concrete frame building located in high seismicity area with shallow soil has been presented. The building model is subjected to different sources and types of earthquakes records with two levels of annual probability of exceedence representing life safety or ultimate limit state (ULS) and operational or serviceability limit state (SLS). The statistical distribution of floor response spectra and its amplification peak floor acceleration along the height of the building are observed under ULS and SLS conditions. It is noted that the peak acceleration demand with respect to ground is deamplified up the height of the building. The floor response spectra up the height of the building have shown peaks near the modal periods which are not reflected by code provisions. This effect is observed both in SLS and ULS conditions, but more pronounced in SLS. Hence, different envelopes for component response amplification may be suggested for ULS and SLS conditions.

KEYWORDS: Floor response spectra, Earthquake ground motions, Ultimate limit state, Serviceability limit state

1. INTRODUCTION

The aftermath lessons from recent earthquakes have shown that damage associated with Non-Structural Components (NSC) constitutes a substantial amount of total loss. Following this, the study of non-structural component has constituted an important component of Performance-based Earthquake Engineering (PBEE). Previous studies have shown that the loss due to non-structural components and contents could be in the range of 70% of the total loss (Taghavi and Miranda, 2003) for a typical office building. ATC-58 (2007) document presents methodology to assess the loss due to damage of non-structural components using appropriate fragility functions developed for various performance groups.

Over the last decade, a number of research studies have attempted to assess the behaviour of NSCs and the influence of various parameters on their response using simplified structural models. The design force for seismic restraints of non-structural components depends on the peak component acceleration (PCA) demands. In general, the perception is that the effect of inelasticity on the primary structure reduces the peak acceleration demands. Recent studies on inelastic moment resisting frames have compared inelastic behaviour of building on PCA with the usually assumed elastic behaviour of building and have quantified the parameters that contribute to the amplification or decrease of inelastic floor response spectra with respect to elastic floor response spectra (Medina et al., 2006). The parameters included location of the NSC in the building and damping ratio of components. The research has shown that components with periods closer to building modal periods experience accelerations larger than the values given by SEI/ASCE-02 Provisions.

Earthquake loading standards NZ 1170.5:2004 has introduced new provisions for the design of NSCs and building parts. In the current provisions, an approach similar to that international XΧ has standards been adopted by including factors for peak floor acceleration along the

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Figure 1 Building model for reinforced concrete moment resisting frame

height, component response amplification, ductility of the component itself and a risk factor. These provisions are the outcome of numerous time-history analytical studies on 3-dimensional models of a suite of buildings designed to meet the latest code requirements with different ductility and located on different soil sites subjected to a small number of earthquake records (Roger Shelton, 2004). It should be noted that the distribution of floor height coefficient over the height of the building is quite different from overseas counterparts. In this paper, attention has been focussed to study the floor response spectra of low-rise concrete ductile moment resisting frames under different types of earthquakes ground motions with respect to the source, distance and its

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occurrence rate expressed as annual probability of exceedence.

Ground motions generated by shallow crustal events, interface events and slab events are considered. Typical floor response spectra are generated at various height of the building and compared for ground motion types considered. Further, the building is analysed for two sets of ground motions generated by crustal events with intensity corresponding to annual probability of exceedences of 95% in 50 years referring to Ultimate Limit State (ULS) and 63% in 50 years referring to Serviceability Limit State (SLS) there by avoiding the likely error in downscaling of amplitudes of ULS records. This study demonstrates that current seismic provisions do not always provide adequate design forces, especially when the period of the component is close to one of the building modal periods and this effect is more pronounced under SLS conditions.

SA 0.10 0.01 0.01 0.1 1 Period (s) Figure 2 Illustration of record scaling and the different

frequency contents between a shallow crustal record (El Centro 1940 and a subduction intra-slab record

Target spectra El Centro 1940 N-S Subduction slab record

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ANALYTICAL MODELS 2.

The structural model considered is a 3 storey reinforced concrete moment resisting frame with a roof designed in accordance with NZS 1170.5:2004 to have a ductility factor equal to 6. The building is assumed to be located in Wellington region with shallow soil conditions. The building is designed using a capacity design approach to ensure ductile behaviour and inelastic hinges will be formed in beams near the face of the columns and at the base of the ground storey columns. The nonlinear dynamic simulations are performed on 2-dimensional model of the building in SAP2000 (version 11.8) platform and care has been taken to account for Takeda hysteresis in beams using non-linear link element. Column elements have been modelled with fibre hinges to account for axial load-moment-interaction. P-Delta effects have been considered in the analyses by including a column pinned at floor levels.



The height of the ground story is 4.5m and other stories are 3.65m high. The bay length is 7.5m. The first and second modal periods of the structure are 1.1s and 0.33s. The seismic loads are considered as dead load plus 40% of live load.

3. EARTHQUAKE GROUND MOTIONS

We selected 8 horizontal components from shallow crustal events and 2 records each from subduction interface and slab events. The earthquake magnitude and the closest source distances to the rupture plane for crustal events are given in Table The ranges of magnitude and 1. source distance for the selected records were determined by the deaggregation results from the probabilistic seismic hazard analyses. These records were scaled to match the 500-year design spectra for site class C by a procedure stipulated in the current design standards (NZS

Table 1 Parameters for selected crustal event records		
$M_{\rm w}$	Dist.(km)	Earthquake name
50 year return period		
5.8	24	1986 Chalfant Valley-01
6.2	18	1987 Superstition Hills-01
6.5	17	1987 Superstition Hills-02
7.3	36	1992 Landers
6.5	12	1979 Imperial Valley-06
500 year return period		
7.0	7	1940 El Centro
7.4	2	1978 Tabas, Iran
8.1	121	1985 Michoacan
7.3	14	1999 Kocaeli, Turkey
7.3	15	1999 Kocaeli, Turkey
6.9	11	1989 Loma Prieta
6.9	18	1989 Loma Prieta
6.9	31	1989 Loma Prieta

1170.5, 2004). Figure 2 shows the scaled spectra that match the design spectra used for the design of the building model considered in the present study. We also selected 5 records from shallow crustal earthquakes that are scaled to match the spectra for a return period of 50 years, again using the deaggregation results from the probabilistic seismic hazard study for the assumed building site. The relevant information for the records selected for representing ground motions with a return period of 50 years is presented in Table 1.

The selection of strong motion records from different type of earthquakes, i.e., shallow crustal, subduction interface and subduction slab events, is to check the effect of frequency contents on the response of non-structural elements. Earthquake motions from subduction interface are expected to produce smaller response spectra at periods over 1.0s (Zhao et al, 2006) than shallow crustal events and subduction slab events produce very high short period ground motion compared with other types of earthquakes, see Figure 2 for the comparison between records from crustal and subduction slab records. However, the selection of records using the method stipulated in the current design standards (NZS 1170.5, 2004) diminishes the effect of frequency contents and so scaling and matching to the design spectra is done only for short period range for slab event records. In this study, two strong motion records from the subduction earthquake have been used for the purpose of showing the effect of frequency contents in floor response spectra.

4. RESPONSE CHARACTERISTICS OF NONSTRUCTURAL COMPONENTS – CURRENT PROVISIONS

The response of non-structural components mounted on the floors is dependent on floor acceleration demands which vary over the building height. In a particular floor, the spectral response of components S_ac can be obtained from the respective floor response spectra. So, the provisions in NZS 1170.5 include two factors to determine the design forces: (i) a factor namely to Floor Height Coefficient (FHC) to represent the variation of Peak Floor Acceleration (PFA) along the height with reference to the ground parameter, usually Peak Ground Acceleration (PGA) (ii) component amplification factor to represent the spectral amplification of component with reference to PFA. The acceleration demand for stiff systems with period of vibration close to zero is equal to the Peak Floor Acceleration (PFA). In NZS 1170.5, the FHC is related to the building's site hazard



coefficient at zero period C(0). Further, the response of the system may be reduced by 'response reduction factor' to account for the system flexibility which includes ductility of connections.

In common with other standards worldwide (SEI/ASCE-02) and (ICC 2003), NZS 1170.5 (Section 8) has adopted conventional force-based procedure to determine earthquake design action on parts and a force is expressed by means of a multi-factor equation. The proposed equation is as in Eqn. 4.1:

$$F_{ph} = C_p \left(T_p\right) C_{ph} R_p W_p \le 3.6 W_p \tag{4.1}$$

where

 $C_p(T_p)$ = coefficient obtained for component with period T_p from Eqn. 4.2

 C_{ph} = Response reduction factor

 R_p = Risk factor

 W_p = Weight of the component

$$C_p(T_p) = C(0)C_{Hi}T_pC_i(T_p)$$
(4.2)

where

C(0) = site hazard coefficient for building period, $T_B = 0$ for given soil conditions

 C_{Hi} = Floor height coefficient as represented in Fig.

 $C_i(T_p)$ = Component amplification factor as represented in Fig.

Figure 2 shows envelop of floor height coefficient (FHC) derived for the building considered in this study with reference to NZS 1170.5: 2004 where two segments (linear and a constant) depending on the component floor to total height ratio (h/H) is evident. This was considered necessary to reflect higher mode effects in high rise buildings. Other provisions such as ICC and ASCE-02 suggest linear variation up the height of the building. Figure 3 shows component amplification factor presented with reference to component period (T_p) . Where as ASCE-02 provisions express them with reference to the ratio of component period to the building period (T_p/T_B) and attempt to capture the amplification near the resonant period.

In this study, the response results will be discussed and compared with NZS 1170.5:2004 provisions only.



Figure 2 Variation of floor height coefficient

Figure 3 Component amplification factor



5. FLOOR RESPONSE UNDER TYPES OF GROUND MOTIONS

As discussed in section 3.0, earthquake records are chosen from crustal, interface and slab events to study the floor response characteristics. First the response due to crustal event records will be discussed. Then, the interface event and slab event will be discussed. The effect of damping of NSC on floor responses is not included within the scope of the present study. The responses have been derived for 5% critical damping.

5.1 Response under ULS and SLS crustal event records

For crustal events, two individual sets of earthquake records are chosen; a set with 8 records representing 500 year return period motions for ULS condition and another with 5 records representing 50 year return period motions for SLS conditions. Both set of records were scaled to match the respective design spectrum. Two main characteristics namely FHC and component amplification factors are observed from floor responses.

The distribution of floor height coefficient (FHC) is compared with the values suggested by the NZS provisions as shown in Figure 4. Acceleration response of non-structural components with periods close to zero and to the building modal periods are plotted after normalising with respect to C(0). In ULS conditions, the median curve for all three periods (zero, 1^{st} and 2^{nd} mode) are less than envelope suggested by the NZS provisions. FHC is less than unity for zero period and first modal period denoting deamplification of peak floor acceleration. The reason is generally attributed to "softening effect" due to inelastic behaviour.



(a) Response with ULS records (b) Response with SLS records

Figure 4 Response under crustal events

In SLS conditions, the FHC corresponding to zero period components is close to unity; for components with fundamental period (1.1s) is close to the NZS provisions which is based on the fundamental mode shape. For components with second mode period, FHC is exceeding well above the NZS limits. So, the components with period close to higher mode period of the building are expected to experience higher acceleration demands than at ULS conditions.

Figure 5 shows floor response spectra derived for all the floors above the ground for ULS and SLS records and median, 84th and 16th percentile responses are given. It is observed that spectral response of components with periods close to building modal period is amplified in both ULS and SLS records and the peaks near the higher modes exceed the limits given by the NZS provisions. These limits can be considered as adequate on the following basis: The higher mode peaks are generated by high floor acceleration pulses of very short duration and they are associated with very small displacements. Hence, they do not necessarily result in damage of building parts (Roger Shelton, 2004). However, the peak responses of 2nd floor to roof exceeds well above the limits by NZS provisions near the first mode period of the building and this effect is more pronounced in SLS records





Figure 5 Statistical distribution of component amplification

Summarising the observed responses of FHC and component amplification factor, for short period components, the combination of reduced FHC and increased component amplification factor could result in a value closer to the limits within the NZS provisions, under ULS and SLS conditions. But, when (T_p/T_B) is close to 1, the NZS

provisions could very much underestimate the design forces both in ULS and SLS. According to the observed responses, at certain floor levels (say, the third floor) the SLS conditions might govern the design requirements for component forces rather than ULS, because of the increased component amplification factor and floor height coefficient at that level near the first mode period.



5.2. Interface and Slab Events

In this section, the effect of earthquakes generated by interface and slab events on floor response will be discussed. The intension is to show typical response characteristics for both events and compare with the envelope by NZS provisions. The number of records chosen for interface event is 6 and the records are scaled to match ULS design spectrum of NZS 1170.5 at the fundamental period of building. Figure 6 shows typical median response for floor response spectra at roof level and floor height coefficient up the height of the building. With reference to response due to crustal events, interface events have produced larger values but the distribution patterns are similar. This is a result of the spectra matching in the selected subduction interface records.







For slab event, the scale factors at the period range suitable for the structure presented here will lead to unrealistically high short-period ground motion. Here we used the scale factor that match the short period spectra only as a probable ground shaking from a subduction slab event. Figure 7 shows that the floor response spectra has a very large peak at second modal period. The PFAs at all levels are larger than those of shallow crustal and subduction interface events corresponding to ULS. At 0.3s, the floor acceleration spectra all levels exceed the distribution factor in the current NZ design standards. At the fundamental period of the structure, the floor acceleration response spectra are considerably less than those of the records from shallow crustal and subduction interface records.

6. SUMMARY

The floor response characteristics of a ductile, low-rise reinforced concrete moment resisting frame building are derived and the effects of earthquake records from crustal, interface and slab events are discussed. For crustal



events, the responses under ULS and SLS records were studied. General observations are as follows: In ULS conditions, the amplification of component response is obvious around the building periods and above the limits suggested by the NZ standards. The peaks showed 'softening effect' due to inelastic building response. The floor responses under SLS level of excitation have shown distinct spikes near building mode periods well exceeding the limits by the NZS provisions. Under SLS level of excitation, not much inelasticity is expected within the building. It is suggested that the NZS provisions may need to be revised to include the amplification near the region where (T_p/T_B) is equal to 1.0.

It is the general practice that the design of parts is generally carried out under ULS conditions with the assumption of satisfying SLS criteria. However, from the limited analytical results, it appears that the design for non –structural components may be governed by SLS conditions.

The present study was intended to appreciate the effect of ground motions with respect to the type and return period on floor response spectra. However, the results are the response of one building model. General recommendations for design of non-structural components will be suggested from the on going research in similar lines for different types of buildings, for example, reinforced concrete wall and eccentric braced frame structures with different levels of ductility.

Design force for parts has been dealt in NZS 1170.5 within section 8. However, preparation of a draft for the design of seismic restraints for various types of non-structural components is underway (DZ 4219).

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