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Abstract

Near-source (NS) ground motions are receiving increasing attention in modern seismic engineering practice and research, because they can carry seismic demand systematically different and larger than that of so-called ordinary records. This is due to phenomena such as rupture forward directivity (FD), which can lead to distinct pulses appearing in the velocity time-history of the ground motion. The objective of the present study is to apply, investigate and evaluate the framework necessary for taking FD into account in probabilistic seismic hazard analysis (PSHA) and to subsequently discuss the extension of non-linear static procedures with respect to the inelastic demand associated with FD. In this context, a methodology is presented for the implementation of the DCM towards estimating NS seismic demand, by making use of the results of NS-PSHA and a semi-empirical equation for NS-FD inelastic displacement ratio. Illustrative applications of NS-PSHA and of the DCM (displacement coefficient method), are presented. Additionally, non-linear dynamic analysis results are obtained and compared to the static procedure estimates. Conclusions drawn from the results help to assess the importance of incorporating NS effects in PBSD (performance-based seismic design).

Keywords: directivity, probabilistic hazard analysis, coefficient method, pushover

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1 Introduction

In has been known for some time now, in both the field of seismology and that of earthquake engineering, that earthquake ground motions recorded at sites located in proximity to seismic faults exhibit particular characteristics which duly affect structural response. These exceptional ground motions are said to be subject to phenomena collectively termed near-source (NS) effects. Perhaps the most important among these, is forward rupture directivity (FD): during fault rupture, shear dislocation may propagate at velocities similar to the shear wave velocity and as a result at sites aligned along the direction of rupture propagation, shear wave-fronts generated at different points along the fault may arrive at the same time, delivering most of the seismic energy in a single double-sided pulse registered early in the velocity recording. Such impulsive behavior, which is actually the result of constructive interference of horizontally polarized waves, is typically most prominent in the fault-normal component of ground motion (Somerville et al., 1997).

As engineering interest in this type of impulsive ground motions continues to grow, it is the objective of this research task to study the effects of NS ground motions on the response of structures, with the intention of being able to better define seismic actions and structural design rules for regions situated near known seismic sources.

Therefore, this study is focused on both the definition of NS design seismic actions and the development of design tools appropriate for NS conditions. In this context, presented research follows three principal directions: firstly, existing semi-empirical models are put to use in order to define NS elastic design spectra by means of well-established probabilistic seismic hazard procedures. Secondly, modification factors to derive inelastic structural demand from the elastic one are developed, appropriate for NS pulse-like ground motions; such modification factors are potentially applicable in the implementation of nonlinear static procedures of structural assessment in NS environments. Thirdly, a methodology is presented for the implementation of the displacement coefficient method (DCM) towards estimating NS seismic demand, by making use of the results of NS-PSHA and a semiempirical equation for NS-FD inelastic displacement ratio. Illustrative applications of NS-PSHA and of the DCM, with explicit inclusion of NS-pulse-like effects, are presented. Different design scenarios are considered in these applications and non-linear dynamic analysis results are obtained and discussed with respect to the static procedure estimates. Conclusions drawn from the results help to assess the importance of incorporating NS effects **PSHA** and PBSD in in general.

2 Near-source seismic hazard and design scenarios

Most advanced seismic codes worldwide define structural design actions based on probabilistic seismic hazard analysis or PSHA (Cornell 1968, McGuire 2004), which allows the building of hazard curves starting from seismic source models and ground motion prediction equations (GMPEs). PSHA is, to date, a consolidated procedure; however, the need for adjustments for sites close to a seismic fault, is emerging. In fact, in NS conditions, both ground motions and seismic structural response, may show systematic spatial variability which classical PSHA is not able to explicitly capture.

Recent attempts to explicitly assess directivity effects in probabilistic hazard assessment are aimed at modifying classical PSHA to highlight the pulse-like features of ground motion rather than formulating a new procedure (e.g., Abrahamson 2000, Tothong et al. 2007, lervolino and Cornell 2008).

2.1 NEAR-SOURCE PSHA

Standard approach for computing the mean annual frequency (MAF, λ) of exceeding a ground motion intensity measure (IM) threshold is shown in Equation (2.1) for a single seismic source. The chosen IM is the elastic spectral acceleration (*Sa*) at a fixed spectral period (T^*) exceeding an intensity level, $Sa(T^*) = s_a^*$:

$$\lambda_{Sa}\left(s_{a}^{*}\right) = \nu \cdot \iint_{m,r} G_{Sa|M,R}\left(s_{a}^{*}|m,r\right) \cdot f_{M,R}\left(m,r\right) \cdot dm \cdot dr$$
(2.1)

where *M* is the magnitude and *R* is the source-to-site distance, v is the mean annual rate of occurrence of earthquakes on the source within a magnitude range of interest, $f_{M,R}$ is the joint probability density function (PDF) of *M* and *R*, and $G_{Sa|M,R}$ is the complementary cumulative distribution function (CCDF) of *Sa* (usually lognormal if obtained by a GMPE).

NS-PSHA requires the MAF to be a linear combination of two hazard terms which account for the absence or the occurrence of the pulse, $\lambda_{Sa,NoPulse}$ and $\lambda_{Sa,Pulse}$ respectively, as reported in Equation (2.2). In fact, the problem of estimating seismic hazard in near-source conditions may be posed as if two faults are present at the same location: one producing ordinary ground motions, and one producing pulse-like records.

$$\lambda_{Sa}(s_a^*) = \lambda_{Sa,NoPulse}(s_a^*) + \lambda_{Sa,Pulse}(s_a^*)$$
(2.2)

The two terms of Equation (2.2) are implicitly weighted by the pulse occurrence probability. Moreover, two other tasks, which are not faced in traditional hazard analysis, appear: (i) pulse period prediction, and (ii) pulse amplitude prediction. In Equation (2.3) and Equation (2.4), which expand Equation (2.2) in the case of a single fault with undefined rupture mechanism, \underline{Z} is a vector with all the required information (to follow) about the relative position between the seismic source and the site (e.g., Tothong et al. 2007, lervolino and Cornell 2008).

$$\lambda_{Sa,NoPulse}\left(s_{a}^{*}\right) = \nu \cdot \iint_{m \underline{z}} P\left[NoPulse|m,\underline{z}\right] \cdot G_{Sa|M,\underline{z}}\left(s_{a}^{*}|m,\underline{z}\right) \cdot f_{M,\underline{z}}\left(m,\underline{z}\right) \cdot dm \cdot d\underline{z}$$

$$\lambda_{Sa,Pulse}\left(s_{a}^{*}\right) = \nu \cdot \iiint_{m \underline{z} \ t_{p}} P\left[Pulse|m,\underline{z}\right] \cdot G_{Sa,mod|M,\underline{z},Tp}\left(s_{a}^{*}|m,\underline{z},t_{p}\right) \cdot f_{Tp|M,\underline{z}}\left(t_{p}|m,\underline{z}\right) \times$$

$$f_{M,\underline{z}}\left(m,\underline{z}\right) \cdot dm \cdot d\underline{z} \cdot dt_{p}$$

$$(2.4)$$

Equation (3) refers to the case of pulse absence and it is weighted for the corresponding probability, $P[NoPulse|m,\underline{z}]$. All other terms are equal to those of Equation (2.1) in which pulse-like effects are not considered. Conversely, Equation (2.4) refers to the case of pulse occurrence as indicated by the pulse occurrence probability, $P[Pulse|m,\underline{z}]$. To account for the peculiar spectral shape of pulse-like records, it is possible to specifically calibrate a new GMPE or to modify an existing one: the latter is considered herein, thus the $G_{Sa,mod|M,\underline{z},Tp}$ symbol. Because modification of ordinary GMPEs depends on the pulse period, the $f_{Tp|M,\underline{z}}$ distribution is required in the analysis. Finally, $f_{M,\underline{z}}$ is the joint distribution (of magnitude and geometrical parameters) similar to the ordinary PSHA, but with a more detailed description

(by means of \underline{Z}) of relative source-to-site position, with respect to the simple distance

variable of Equation (2.1). The complexity of rupture and wave propagation phenomena makes directivity prediction difficult if based only on physical parameters: in fact it is not always observed in the sites where it is expected, and may also occur at sites apparently not prone to pulse-like ground motion (e.g., Bray and Rodriguez-Marek 2004). Thus, stochastic models for the prediction of the pulse occurrence probability, P[pulse], were developed (e.g., lervolino and Cornell 2008). These models depend only on geometrical parameters depicted in Figure 2.1, which are slightly different in the case of strike-slip or dip-slip faults, see Somerville et al. (1997). Such parameters in SS [DS] case are: (i) distance (s) from the epicenter to the site [d, from hypocenter to the site] measured along the rupture direction, (ii) θ angle between the fault strike and the path from epicenter to the site [ϕ angle between the fault plane and the path from hypocenter to the site], and (iii) minimum distance R between the rupture and the site. (For SS, some additional parameters, are shown in Figure 2.1.) Equation (2.5) and (2.6) report the two models used in this study; note that the geometrical variables for DS were used in lervolino and Cornell (2008) to fit generic non-strike-slip (NSS) data.

$$P[Pulse|R,s,\theta] = \frac{e^{0.859-0.111\cdot R+0.019\cdot s-0.044\cdot\theta}}{1+e^{0.859-0.111\cdot R+0.019\cdot s-0.044\cdot\theta}}$$
(2.5)
$$P[Pulse|R,d,\phi] = \frac{e^{0.553-0.055\cdot R-0.027\cdot d-0.027\cdot\phi}}{1+e^{0.553-0.055\cdot R-0.027\cdot d-0.027\cdot\phi}}$$
(2.6)

Equation (2.5) and Equation (2.6) are defined for R (SS), R (DS), s, d, θ or ϕ varying in the intervals of [0 km, 30 km], [5 km, 30 km], [0 km, 40 km], [0 km, 20 km], [0°, 90°], and [0°, 90°], respectively.



Figure 2.1 SS (a) and DS (b) geometrical parameters.

An example to illustrate the pulse occurrence probability model of lervolino and Cornell (2008) is given in Figure 2.2 with reference to the rupture characteristics of the L' Aquila 2009 event (normal faulting). It is to point out that the occurrence probability is never larger than 0.5; this is because the model was developed generically for non-strike-slip earthquakes, which are often complex and in which it is not easy to identify rupture directivity effects. Nevertheless, it may be used to highlight sites comparatively more likely to be affected by velocity pulses given the source geometry. From this point of view, results of pulse occurrence probability model are in general agreement (except ORC) with the results of algorithm for the identification of pulse-like ground motions.



Figure 2.2 Contours of occurrence probability and accelerometric stations with pulse-like signals (blue).

In the case of an individual SS source of fixed dimensions and geographical location, the necessary geometrical parameters to compute hazard are the rupture length (L, assumed to be lower or equal to the fault length), the position of the rupture on the fault (P), and the epicenter location (E) as reported in Figure 2.1a. A deterministic relationship between these parameters and {R,s, θ } vector exists. Thus, the <u>Z</u> vector in Equation (2.3) and Equation (2.4) can be replaced by a vector comprised of L, P, and E.

In the analysis, L can be considered as a function of magnitude (e.g., Wells and Coppersmith 1994), while P and E may be associated to a uniform probability distribution on the fault and on the rupture length, respectively. Given these assumptions, Equation (2.3) and Equation (2.4) are specialized for the SS particular case, obtaining Equation (2.7) and Equation (2.8).

$$\lambda_{Sa,NoPulse}\left(s_{a}^{*}\right) = \nu \cdot \iiint_{m \ l \ p \ e} P\left[NoPulse\left|m,l,p,e\right] \cdot G_{Sa|M,L,P,E}\left(s_{a}^{*}\left|m,l,p,e\right]\right) \times f_{P,E|M,L}\left(p,e\left|m,l\right\right) \cdot f_{L|M}\left(l\left|m\right) \cdot f_{M}\left(m\right) \cdot dm \cdot dl \cdot dp \cdot de$$

$$(2.7)$$

$$\lambda_{Sa,Pulse}\left(s_{a}^{*}\right) = \nu \cdot \iiint_{t_{p}\ m\ l\ p\ e} P\left[Pulse\left|t_{p}, m, l, p, e\right] \cdot G_{Sa, \text{mod}|Tp, M, L, P, E}\left(s_{a}^{*}\left|t_{p}, m, l, p, e\right) \times f_{Tp|M, L, P, E}\left(t_{p}\ |m, l, p, e\right) \cdot f_{P, E|M, L}\left(p, e|m, l\right) \cdot f_{L|M}\left(l\ |m\right) \cdot f_{M}\left(m\right) \cdot dt_{p} \cdot dm \cdot dl \cdot dp \cdot de$$

$$(2.8)$$

It appears that in order to account for all the geometrical parameters, a fifth order integral is necessary, leading to a computational effort significantly higher than ordinary PSHA. In fact, Equations (2.7) and (2.8) can be simplified neglecting some stochastic dependencies. More specifically:

- pulse probability is a function of geometrical parameters and is independent on magnitude and pulse period, that is, $P[Pulse|t_p, m, l, p, e] = P[Pulse|l, p, e]$;
- pulse period PDF is a function of magnitude only, that is, $f_{T_{p|M,L,P,E}}(t_p|m,l,p,e) = f_{T_{p|M}}(t_p|m);$
- PDF of rupture and epicenter's position depends on the rupture length, that is, $f_{P,E|M,L}(p,e|m,l) = f_{P,E|L}(p,e|l);$
- $G_{Sa,mod|Tp,M,L,P,E}$ is the modified GMPE, and $G_{Sa|M,L,P,E}$ is the original GMPE. Using R_{jb}, both functions are independent on the location of the epicenter, E;
- Pulse occurrence and absence are complementary: P[NoPulse|l, p, e] = 1 - P[Pulse|l, p, e].

Regarding the DS case, pulse occurrence probability model considers such a rupture in its two-dimensional representation, which is easy to render analogous to the SS case (Figure 2.1). In fact, dip-slip three-dimensional representation is given in Figure 2.3 by two orthogonal sections, which may be useful to identify all the geometrical variables needed, and those that can be neglected in the hazard integrals.

For simplicity, the hypotheses of rectangular fault and rupture are taken here. B and L variables represent rupture surface sides (Figure 2.3a and Figure 2.3b). As for SS, they can be computed using Wells and Coppersmith (1994), which provide relationships as functions of M for each of the two linear dimensions and the total rupture area. A possible option (used in the following applications) is to assume only one of the three mentioned parameters as a variable (dependent on event magnitude) and forcing a constant ratio for B and L.



Figure 2.3 Dip-slip rupture representation.

 X_r and X_h are the distances of the rupture and of the hypocenter from the fault bounds, respectively. The former is necessary for the identification of R_{jb} of the site for which hazard is computed, and it has to appear in the hazard integral, while the latter can be neglected. In

fact, the hypocenter's position is important for parameters represented by section AA in Figure 2.3, where X_h does not appear.

Equations (2.7) and (2.8) can be adapted to the dip-slip case, that is, Equation (2.9) and Equation (2.10), considering the following differences:

- o position of the epicenter, E, has to be replaced by the position of the hypocenter (H);
- the rupture's geometrical parameter dependent on event magnitude is assumed to be the rupture length L, in analogy with the SS case (in principle it should be the area of the rupture from which B and L are assumed to be deterministically dependent in the hypothesis of a constant B/L ratio);
- $\circ~$ to compute pulse occurrence probability X_r is also necessary (for computation of R, along L, P, and H);
- the introduction of X_r requires knowledge of its PDF conditional to L, $f_{X_r|L}(x_r|l)$. Such distribution is, in principle, also conditional on the dimension and position of the fault, however they are considered as known.

$$\lambda_{Sa,NoPulse}\left(s_{a}^{*}\right) = \nu \cdot \iiint_{m \ l \ p \ h \ x_{r}} P\left[NoPulse\left|l, p, h, x_{r}\right] \cdot G_{Sa|M, X_{r}, L, P}\left(s_{a}^{*}\left|m, l, p, x_{r}\right) \times f_{P, H|L}\left(p, h|l\right) \cdot f_{X_{r}|L}\left(x_{r}\left|l\right) \cdot f_{L|M}\left(l|m\right) \cdot f_{M}\left(m\right) \cdot dm \cdot dl \cdot dp \cdot dh \cdot dx_{r}$$

$$\lambda_{Sa,Pulse}\left(s_{a}^{*}\right) = \nu \cdot \iiint_{l_{p} \ m \ l \ p \ h \ x_{r}} P\left[Pulse\left|l, p, h, x_{r}\right] \cdot G_{Sa, \text{mod}|Tp, M, X_{r}, L, P}\left(s_{a}^{*}\left|t_{p}, m, l, p, x_{r}\right) \times f_{Tp|M}\left(t_{p}\left|m\right) \cdot f_{P, H|L}\left(p, h|l\right) \cdot f_{X_{r}|L}\left(x_{r}\left|l\right) \cdot f_{L|M}\left(l|m\right) \cdot f_{M}\left(m\right) \cdot dt_{p} \cdot dm \cdot dl \cdot dp \cdot dh \cdot dx_{r}$$

$$(2.10)$$

2.2 NS-PSHA APPLICATIONS

Because the marginal pulse occurrence probability is generally fairly low according to lervolino and Cornell (2008), some applications of the proposed approach are required to quantitatively assess the effects of these modifications on seismic hazard estimates, and whether NS-PSHA is able to represent the pulse-like directivity threat adequately. To this aim, some applications are developed in terms of hazard, disaggregation and definition of design spectra. The geometrical configuration of the examples is analyzed in detail what follows, yet there are some common assumptions and working hypotheses that can be given beforehand:

- because modification factor of GMPE accounting for pulse-like effects was fitted on the model of Boore and Atkinson (2008), this GMPE is used;
- chosen IMs are the elastic spectral accelerations at all the spectral periods provided by used GMPE (from 0 sec, that is, Peak Ground Acceleration or PGA, to 10 sec); thus, results are first represented in terms of elastic response spectrum characterized by the same exceedance probability in a fixed time window for all ordinates (i.e., the uniform hazard spectrum, UHS);
- $\circ~$ a return period (T_r) equal to 475 years is assumed: in other words, computed intensity measures have an exceedance probability in 50 years equal to 10% (assuming a

homogeneous Poisson process for earthquake occurrence; Cornell 1968, McGuire 2004);

• the annual rate of earthquake occurrence (v) on each fault is assumed to be equal to 0.05.

2.2.1 Applications

In the first application a SS fault and two different construction sites are considered: site S₁ is aligned with fault direction and located five kilometers far from its upper edge, while site S₂ is on the center of the fault (Figure 2.5a). Both sites are expected to be prone to directivity, having a θ angle equal to zero (e.g.,Somerville at al. 1997, lervolino and Cornell 2008). Fault length is assumed equal to 200 km while rupture length (L) and rupture location on the fault (P) are considered as random variables. The distribution of the former, conditional to M, is lognormal (e.g., Wells and Coppersmith 1994), while that of the latter is uniform and limited by the fault dimension and the rupture length itself. In fact, for a given size, the rupture can be located in all the possible positions with a uniform probability distribution constrained by the fault limits. R_{jb} of the site is univocally defined once the rupture, but in order to reduce the numerical effort of the illustrative analyses, only three possible positions of the epicenter were considered, that is, in the middle of the rupture or located at 30% of the total rupture length, measured from each of the two rupture extremities. Once the epicenter location is defined, s-parameter is known.

As a first case, the assumption that all the earthquakes generated by the source have a fixed (*characteristic*) magnitude equal to 7, was considered, and analyses were performed for both sites. Then, referring only to S_1 , a Gutenberg-Richter-like distribution (Gutenberg and Richter 1944) for M was assumed with a negative slope equal to 1, and minimum and maximum M equal to 4.5 and 7.5, respectively. In order to reduce the computational effort, magnitude distribution was lumped by three discrete values of 5, 6 and 7; the corresponding associated discrete probabilities are respectively 0.9, 0.09 and 0.01.

Before discussing results, it may be useful to plot PDFs of pulse period and rupture length conditional on those magnitudes considered. Figure 2.4a shows that, if generated earthquakes have M equal to five, it is very unlikely that forward directivity effects affect spectral periods higher than 3 sec, while a M 7 earthquake may influence a very large range of structural periods. This is because, according to the considered model, standard deviation of the logarithms is 0.59, which means about 60% coefficient of variation of Tp. Moreover, a M 5 earthquake has an associated rupture length significantly lower than the total length of the considered fault (Figure 2.4b), thus its probability of being located near the considered sites is lower than that associated to a M 7 event.



Figure 2.4 Probability density function of T_p (a); and rupture length L (b) conditional to M equal to 5, 6 and 7.

Results of NS-PSHA were compared to the corresponding ordinary hazard estimates; i.e., Equation (2.1). In Figure 2.5c, UHS are reported for ordinary (Sa_{PSHA}) and modified $(Sa_{NS-PSHA})$ analyses, and for characteristic and multiple M cases (magnitude distribution is applied only for S₁ and hereafter results for S₂, with characteristic magnitude, will be indicated with a cross superscript: Sa_{PSHA}^+ and $Sa_{NS-PSHA}^+$). In Figure 2.5d the increments due to forward directivity effects, with respect to the ordinary case, are reported for each case.

Figure 2.5c shows that three analyzed cases yield different results: referring to PGA (for which directivity effects are negligible according to the assumed framework), it is apparent and expected that characteristic earthquakes generate higher accelerations for S_2 because of the lower distance from the rupture. The lower response spectrum for S_1 , in the case of multiple-magnitude distribution, with respect to the case of characteristic magnitude, is of less intuitive understanding, but can be explained recalling that frequency of occurrence of M 5 is much higher than those of M 6 and M 7 (see Figure 2.4b).

Because of the aim of the study and working hypotheses, more attention is given to hazard increments with respect to the ordinary case, rather than on absolute acceleration values, and from Figure 2.5d the following may be pointed out.

- Hazard increments vary from about 25% to about 100% depending on the characteristics of the investigated case. This is mainly because of the applicability range of pulse occurrence probability model. For site S₁, a zero pulse probability is associated to a number of rupture's positions larger than S₂ (directivity effects are likely to occur more frequently for S₂ rather than for S₁ site). It can also be inferred that, under the hypotheses of uniform distribution of rupture position on the fault, geometry and magnitude occurrence may have significant effects on increments values.
- 2. The shape of hazard increments, the range of spectral periods in which increments are significant, and the period corresponding to the maximum directivity effect, can be directly derived from the model of magnitude occurrence on the fault. Such a dependency derives from the pulse period prediction model; e.g., the PDF of Tp conditional to M 7, has a median value equal to 3.7 sec, which is a good approximation of the period corresponding to maximum increment.

3. Similarly, because cases with multiple magnitudes are mostly affected by smaller and more frequent events, the period of maximum increments for the multiple-magnitude distribution is well correlated with the median value of Tp distribution for M 5 (0.43 sec), while increments are negligible for spectral periods higher than 4 sec.



Figure 2.5. Rupture-site configuration of analyzed cases (a); mass probability distribution of magnitude occurrence (b); 475 yr UHS (c); and hazard increments (d).

A second example application, this time considering a DS rupture is also performed and similar to the SS examples, fixed planar dimensions (4000 km²) and position are assumed for the seismic source. In fact, the fault is identified by the following angles: 50° and 90° for dip and rake, respectively; i.e., a *normal* fault. Both fault and rupture areas are assumed to be rectangular with the B/L ratio equal to two. An individual possible event magnitude, M 7, is assumed.

A site (S) placed within the surface projection of the rupture is considered and reported in Figure 6. In the same figure, size and position of the fault are represented by dotted lines. The rupture's area (A) (or its sides B and L) and rupture location (identified by distance to fault sides, X_r and P_r), are random variables. A-area is assumed to be a lognormally distributed conditional to M (Wells and Coppersmith 1994), and it is limited by the fault dimensions. For a given A-value, the rupture can be located in all the possible positions with a uniform PDF, yet constrained by the fault boundaries. R_{jb} for the site is univocally defined once the rupture is known. Ideally, the hypocenter can also be located everywhere on the rupture, but in order to reduce the computational demand, only three possible positions on the diagonal of the rupture were assumed. In Figure 2.3, the generic position of the

hypocenters is identified as H. Once the hypocenter is also known, the d-parameter can be determined.

The hazard integral in case of dip-slip ruptures is reported in Equation (2.9) and Equation (2.10). UHS computed with ordinary and modified PSHA are reported in Figure 2.6a, while increments are shown in Figure 2.6b. The shape of hazard increments is analogous to SS, because it depends only on distribution of T_p given magnitude, which is the same in the two cases. Conversely, values of such increments are different because of the different geometrical configuration.



Figure 2.6 475 yr UHS with modified and classical PSHA (a); and increments due to directivity effects (b).

2.2.2 Pulse-like disaggregation

Disaggregation is complementary to hazard analysis (McGuire 1995), and it is useful to identify probability-based design scenarios (e.g., *design earthquakes*) and to provide a rational basis for the selection of representative seismic input (e.g., ground motions) to be used in dynamic analyses of structures. In fact, it is typically used to compute the distribution of magnitudes, distances, and ε values contributing to occurrence or exceedance of some ground motion intensity level (s_a^*) . This issue is especially important in near-source conditions in which criteria for design scenarios and ground motion record selection criteria are not well established yet. In fact, classical disaggregation equations can be modified in accordance with the expressions of NS-PSHA, to provide contribution to hazard of the main variables; i.e., the probability that a ground motion intensity level is caused by a pulse-like ground motion, and the distribution of pulse periods associated to it, or the probability that a set of geometrical parameters determines the exceedance of a hazard threshold.

Referring to the hypotheses of a single fault, and comprising site-source geometrical parameters with the Z vector, disaggregation's most synthetic result is:

$$f\left(m,\underline{z},\varepsilon \left|Sa > s_{a}^{*}\right) = \frac{\nu \cdot P\left[Sa > s_{a}^{*} \left|m,\underline{z},\varepsilon\right] \cdot f\left(m,\underline{z},\varepsilon\right)}{\lambda_{s_{a}^{*}}}$$
(2.11)

in which $P[Sa > s_a^* | m, \underline{z}, \varepsilon]$ is the probability of exceeding the hazard level s_a^* given magnitude, ε and all the geometrical variables of the problem.

Marginal disaggregation distribution of pulse period can also be obtained considering only the case of pulse occurrence as reported in Equation (2.12):

$$f\left(t_{p}\left|Sa > s_{a}^{*}, Pulse\right) = \frac{\nu \cdot P\left[Sa > s_{a}^{*}\left|t_{p}, Pulse\right] \cdot f\left(t_{p}\left|Pulse\right)\right]}{\lambda_{s_{a}^{*}|Pulse}}$$
(2.12)

where $\lambda_{s_a^*|Pulse}$ is the MAF of exceeding the s_a^* value given that the pulse occurs, while $\lambda_{s_a^*,Pulse}$ of Equation (2.2) is the MAF of the joint event of exceeding s_a^* and occurrence of pulse.

As a final result, probabilities of observing pulse occurrence or absence given the exceedance of s_a^* can be computed. They give information about how likely exceedance is due to forward directivity effects. The two terms are mutually exclusive and complementary to one; analytical expression of the former is reported in Equation (2.13).

$$P\left[Pulse \left| Sa > s_a^* \right] = \frac{\nu \cdot P\left[Sa > s_a^* \left| Pulse \right] \cdot P\left[Pulse\right]}{\lambda_{s_a^*}}$$
(2.13)

Referring to the hazard result of the SS case with a multiple-magnitude distribution, the disaggregation distributions in Equation (2.14) and Equation (2.15) were computed for Sa(1sec):

$$f(m,r,\varepsilon,t_p|Sa > s_a^*,Pulse) = \frac{\nu \cdot P[Sa > s_a^*|Pulse,m,r,\varepsilon,t_p] \cdot f(m,r,\varepsilon,t_p|Pulse)}{\lambda_{s_a|Pulse}}$$
(2.14)

$$f(m,r,\varepsilon|Sa > s_a^*, NoPulse) = \frac{\nu \cdot P[Sa > s_a^*|NoPulse, m, r,\varepsilon] \cdot f(m,r,\varepsilon|NoPulse)}{\lambda_{s_a^*|NoPulse}}$$
(2.15)

where $f(m,r,\varepsilon,t_p|Sa > s_a^*,Pulse)$ is the probability of $\{m,r,\varepsilon,t_p\}$ being the causative vector for s_a^* in the case of pulse occurrence, and $f(m,r,\varepsilon|Sa > s_a^*,NoPulse)$ is the probability of $\{m,r,\varepsilon\}$ being the causative vector in the case of no pulse occurrence in ground motion.

Because some of the considered PDFs cannot be clearly represented (being defined in spaces of order larger than \Re^3) numerical integration was used in order to obtain marginal distribution of easier graphical handling; e.g., Equation (2.16).

$$f\left(m,r\middle|Sa>s_{a}^{*},Pulse\right) = \iint_{t_{p}\varepsilon} f\left(m,r,\varepsilon,t_{p}\middle|Sa>s_{a}^{*},Pulse\right) d\varepsilon \cdot dt_{p}$$
(2.16)

In Figure 2.7 the following disaggregation PDFs are reported: magnitude and distance conditional to pulse (a), and no-pulse (b) occurrence; ϵ values conditional to pulse (c), and no-pulse (d) occurrence; pulse period in case of pulse (e). All PDFs refer to Tr = 475 yr.

Distance disaggregation, conditional to pulse occurrence, is limited by the definition domain of the pulse probability model. Conversely, the same disaggregation plot, but conditional to absence of pulse, shows non-negligible hazard contributions for larger distances (however, data for distances larger than 50 km are not reported).

Mean ε , conditional to pulse occurrence, is lower than ε conditional to pulse absence (0.5 and 1.0, respectively), because the first disaggregation is computed by the modified GMPE in which Tp effects are applied on the modified predicted median. Finally, the Tp disaggregation distribution has a similar shape of the PDF of Tp conditional to M 5; however, because disaggregating hazard refers to 1 sec spectral acceleration, the mean is moved from 0.5 sec to 0.9 sec.

2.2.3 Design scenarios

It is well known that the uniform hazard spectrum (UHS) does not account for correlation of spectral ordinates. In fact, UHS is an *envelope* spectrum, which may not be representative of any specific ground motion. The problem was studied for ordinary conditions by Baker and Cornell (2006). The proposed solution consists of the *conditional-mean spectrum considering* ε (CMS- ε); i.e., a spectrum in which the spectral ordinate associated to the structural period of interest has a defined exceedance probability and all the others are computed from disaggregation for the considered structural period, and account for correlation.

In NS-PSHA, resulting UHS is the envelope of many seismic scenarios in which cases of pulse occurrence are combined with cases of pulse absence weighted by the pulse probabilities. Moreover, terms characterized by pulse occurrence account for many different pulse periods, having the PDF of Tp, conditional on event magnitude, a quite flat shape. As a consequence, the effect of pulses on the final spectrum is spread over a large range of periods as shown in the previous examples. Thus, the well-known limitations of UHS may be even worse in NS cases. On the other hand, CMS- ϵ is not of easy direct application in the NS-PSHA framework because, to date, an assessment of correlation of spectral ordinates fitted on pulse-like records is not yet available (if not indirectly via the GMPE modification factor), and because it is not univocally defined how to account for Tp in the CMS- ϵ procedure.

Referring to the hazard result of the SS case with a multiple-magnitude, all the mean values of disaggregation distributions necessary to compute discussed design spectra have been

computed for three values of spectral periods representative of short (0.5 sec), medium (1.0 sec) and long (2.0 sec) structural periods, as reported in Table 2.1.

	$M_{p,\mu}$	$R_{p,\mu}$	Tp_{μ}	$M_{np,\mu}$	$R_{np,\mu}$	$P(Pulse Sa > s_a^*)$	$P(NoPulse Sa > s_a^*)$
T=0.5	5.2	10.9	0.71	5.8	27.9	0.54	0.46
T=1.0	5.3	10.5	0.88	5.9	37.0	0.42	0.58
T=2.0	5.4	10.2	1.12	6.1	48.7	0.27	0.73

Table 2.1 Average disaggregation values for application 1.

Here a procedure for identification of design spectra is preliminarily proposed. It consists of the identification of two different spectra representative of the ordinary scenario (i.e., in which no forward directivity effects occur) and the pulse-like scenario (i.e., which accounts entirely for the effects of forward directivity). The latter will be referred to as *close-impulsive spectrum* or CIS.

In Figure 2.8 UHS is compared with the two computed spectra defined before for pulse-like, $Sa_p(M_{p,\mu}, R_{p,\mu}, Tp_{\mu})$, and non-pulse-like, $Sa_{np}(M_{np,\mu}, R_{np,\mu})$, scenarios. Comparisons are reported in terms of pseudo-accelerations (in logarithmic scale) and displacements (*Sd*) spectra. It is noted that Sa_p and Sa_{np} have significantly different causative magnitude and distance, therefore they are representative of different earthquakes. In order to underline the characteristic shape of CIS, a spectrum computed from ordinary GMPE with magnitude and distance values of pulse-like scenario $Sa_{np}(M_{p,\mu}, R_{p,\mu})$, and scaled to the same PGA value of CIS, is also reported in each plot.

Significant ranges of periods are affected by differences between $Sa_p(M_{p,\mu}, R_{p,\mu}, Tp_{\mu})$ and $Sa_{np}(M_{p,\mu}, R_{p,\mu})$ spectra, this is because of the shape of modification factor. Moreover, for spectral periods different to the one disaggregation refers to, differences between the proposed spectra and UHS can be significant.

These near-source scenarios may also help in assessing structural performance by means of non-linear dynamic analysis. In fact, record selection for near-source sites should account for pulse-like and non-pulse-like records. The more straightforward way to address this issue would be to select records with a required M, R and T_D , from disaggregation.



Figure 2.7. Hazard disaggregation, for Tr = 475 yr and 1 sec spectral period, in terms of: magnitude and distance conditional to pulse occurrence (a) and pulse absence (b); ϵ values conditional to pulse occurrence (c) and absence (d); pulse period (e).



Figure 2.8 UHS and proposed conditional mean spectra for T equal to 0.5 (a and b); 1.0 (c and d); and 2.0 sec (e and f), in terms of accelerations and displacements.

2.3 SENSITIVITY ANALYSIS OF DIRECTIVITY EFFECTS ON PSHA

In the preceding sections, the subject of *hazard increments* (HIs) due to NS-PSHA with respect to the ordinary PSHA was broached. Results therein show that numerical values of such increments are dependent on geometrical parameters determining the source-to-site configuration, such as fault dimensions and site location. In order to deepen this issue, additional illustrative applications are presented retaining the hypothesis of known fault geometry which is common to all the previous works regarding this topic.

As already discussed, rupture length and rupture position are, in principle, random variables; however, these applications are implemented under the simplifying hypothesis of fixed rupture dimension and position. Such hypothesis appears to be acceptable if a single magnitude can be generated by the considered fault (as assumed in the following). The hypothesis of uniform distribution of the epicentre position on the rupture is retained.

Two cases of single generated magnitude, equal to 5.0 and 6.0, are considered. Starting from the Wells and Coppersmith (1994) prediction model which assumes a lognormal distribution (base 10) of rupture dimension, two fixed source length are associated to each magnitude scenario. More specifically, fifth and fiftieth percentiles of rupture length distributions are chosen: 2.0 and 3.4 km SS lengths are associated to M 5.0, while 8 and 14 km are obtained for the M 6.0 scenarios.

In the following, seismic hazard is estimated via the UHS for a return period (*Tr*) equal to 475 years; used GMPE is again that of Boore and Atkinson (2008); mean annual rate of earthquake occurrence on the fault is 0.05. Attention is focused on *HI*s if ordinary PSHA is replaced by its modified version for NS sites. In most of the advanced seismic codes worldwide, seismic action is characterized by UHS computed by ordinary PSHA and quantification of HIs in NS conditions is useful to understand the consequences of neglecting pulse-like directivity effects in hazard analysis. For each spectral period *T* and for the chosen *Tr* = 475, hazard increments HI(T) are analytically defined by the percentage factor in Eq. (2.17).

$$HI(T) = \frac{S_a(T)_{NS-PSHA} - S_a(T)_{PSHA}}{S_a(T)_{PSHA}} \cdot 100$$
(2.17)

More specifically, contours of *HI*s in a wide area around the seismic sources are studied, in order to avoid being constrained to site-specific results. In fact, such contours are also useful for identification of the distance from the fault beyond which NS-PSHA provides negligible differences with respect to PSHA.

UHS and HIs were computer for 64 sites influenced by the 8 km SS fault. Studied sites were identified by a 5 km by 10 km lattice over a 40 km by 60 km area. HIs are represented in terms of contours of maximum values at each site (Figure 2.9a). Similarly, contours of maximum HIs due to a 14 km fault generating M 6.0 events are reported in Figure 2.9b. In both cases, maximum HIs correspond to 1 s spectral period.

The case of M 5.0 generating fault is also studied with the fault length of 2.0 and 3.4 km (Figures 9c and 9d, respectively) chosen from Wells and Coppersmith (1994). In these cases, maximum HIs are computed for 0.4 s spectral period.



Figure 2.9 Contours of HI(1s) for the 8 (a) and 14 km (b) faults generating M 6.0 events and contours of HI (0.4s) for the 2.0 (c) and 3.4 km (d) faults generating M 5.0 events.

In all the plots, sites with distance from the rupture higher than 30 km are shaded because a zero pulse probability is associated to them (ordinary and NS-PSHA are formally equivalent).

It is also to note that contours of HIs are similar even if different generated magnitudes are considered. The reason is in the absence of magnitude dependency in the used pulse occurrence probability model. Two symmetry axes are apparent for the contours, while the elongated shape of HI contours comes from the dependency of pulse occurrence probability on the θ parameter. The minor differences between Figs. 2.8a, 2.8b, 2.8c and 2.8d depend only on the different rupture lengths. In all cases, the red dotted lines approximate an area near the fault with HIs higher than 10%: these lines are 15 km far from the fault and are parallel to it. This means that, at least in these applications, directivity effects can be considered *negligible* for all the sites external to the selected zone (the threshold is arbitrarily chosen).

Such results apply, without any additional computation, to all SS faults with length between 2 and 14 km. Furthermore, these seem to allow to identify the zone in which directivity effects are relevant, according to the considered NS-PSHA procedure, via a preliminary analysis, and independently of the specific characteristics of the considered fault.

2.4 DISCUSSION ON NS-PSHA AND DESIGN SCENARIOS

NS-PSHA was applied to cases of sites subjected to single seismic sources with SS or DS rupture mechanisms, and different magnitude distributions. It was found that the range of spectral periods in which hazard increments due to forward directivity effects are significant, shapes of such increments, and periods corresponding to the maximum increment, can be directly derived from the model of magnitude occurrence on the fault. Such a dependency derives from the relationship between pulse period and event magnitude. Thus, if earthquakes are generated with a Gutenberg-Richter relationship, lower structural periods are those most influenced by directivity effects, due to the higher recurrence frequency of smaller magnitudes.

The amount of hazard increments seems to be largely dependent on the characteristics of the studied cases (geometry above all). Because the pulse period prediction model depends on the event magnitude with a significant heterogeneity, it was also shown that HIs often affect a large range of periods.

HIs are found to be strongly dependent on the considered spectral ordinate. Periods to which maximum *HI*s are associated depend on the magnitude of generated events. Thus, more *hazardous* spectral periods can be predicted with the knowledge of the fault's characteristics in terms of generated magnitudes. Shapes of contours are similar even if different generated magnitudes are considered and minor differences depend only on the different rupture lengths. For investigated cases, it was also possible to geometrically identify the area affected by relevant directivity effects, independently of the specific characteristics of the considered fault in terms of event magnitude. Results suggest that, increasing the number of analyses, more general rules can be identified.

Regarding design scenarios in near-source conditions, known limits of UHS were found of larger importance in near-source conditions with respect to ordinary PSHA, and a procedure was explored based on NS hazard disaggregation. A *close-impulsive spectrum* was preliminarily discussed for the pulse-like hazard part, to complement an *ordinary* spectrum for the non-pulse-like case. These attempts may be helpful in the research for the identification of engineering ground motion characteristics at near-source sites.

3 The displacement coefficient method in nearsource conditions

3.1 THE DISPLACEMENT COEFFICIENT METHOD

Performance-based seismic design of new structures – or assessment of existing ones – requires that the engineer be able to obtain estimates of structural response well into the inelastic range. Traditional methods based on linear-elastic analysis may be inadequate, while fully non-linear dynamic analysis can present the engineer with a task of daunting effort demand. The development of approximate procedures, based on static non-linear analysis of structures, thus emerged as a compromise, offering relative simplicity, while still explicitly treading beyond the elastic limit.

The key concept underlying static non-linear analysis procedures is to represent the structure by a substitute yielding single degree of freedom (SDOF) system and to subsequently use the inelastic spectral response of this system (for given elastic demand at each performance level) as a proxy for the inelastic demand of the original structure. Typically, a *capacity* or *pushover* force versus displacement curve is derived starting from a non-linear model of the structure. This curve is then approximated by a simpler (typically bilinear) relation, which is in turn used to derive the characteristics of the substitute (or equivalent) yielding SDOF system representing the structure. It is well known that this representation has limitations, depending primarily on the structure of interest. The interested reader is referred to Krawinkler and Seneviratna (1998) for a more thorough discussion.

The transition from elastic demand (e.g., determined by seismic hazard) to inelastic displacement at the SDOF level, is generally achieved by employing inelastic response spectra (Miranda, 2001). The required inelastic spectra are traditionally derived via semi-empirical models based on the response of yielding SDOF oscillators subjected to a sample of recorded ground motions. These can be presented in the form of constant-strength (C_R) or constant-ductility inelastic displacement ratios.

As far as the DCM in particular is concerned, the conceptual foundations were developed by Seneviratna and Krawinkler (1997). It was widely introduced to engineers with its adoption by the publications on seismic rehabilitation by FEMA (FEMA-273, FEMA-356). Improvements to the method were subsequently suggested in FEMA-440 and are also considered here. The DCM attempts to estimate the inelastic displacement demand of the structure, which corresponds to a reference degree of freedom and is termed the target displacement, δ_t , by applying a succession of modification factors upon the elastic spectral response of the *corresponding* infinite-strength linear SDOF system, Equation (3.1).

$$\delta_{t} = C_{0} \cdot C_{1} \cdot C_{2} \cdot C_{3} \cdot S_{a} \cdot \frac{T^{2}}{4\pi^{2}}$$
(3.1)

In Equation (3.1), S_a is chosen to represent elastic demand and forms the basis for design. It is derived from seismic hazard provided in the form of a pseudo-acceleration design spectrum corresponding to the performance level considered. Thus, $S_a \cdot (T^2/4\pi^2)$ represents elastic spectral displacement, $S_{d,e}$, of the corresponding SDOF system having a period of natural vibration equal to T. Coefficients C_0, C_1, C_2, C_3 are intended to transform this elastic SDOF response to inelastic structural response. More specifically, C_0 converts the displacement of the equivalent SDOF system into that of the original multiple degree of freedom (MDOF) structure. C_1 is termed the (constant strength) inelastic displacement ratio and is defined as the peak displacement response $S_{d,inel}$ of an inelastic SDOF system divided by the displacement of the corresponding indefinitely elastic SDOF oscillator with period T, $S_{d,e}$; see also the next section.

 $\rm C_2$ is intended to account for the effect of hysteretic behavior on maximum inelastic displacement, in the case of cyclic stiffness and/or strength degradation. This implies that for the derivation of $\rm C_1$ non-evolutionary hysteretic relationships are used, as originally envisioned by Seneviratna and Krawinkler (1997). An alternative approach can be to evaluate inelastic displacement ratios for degrading SDOF systems directly, as was the case in Chenouda and Ayoub (2008) and also in Dimakopoulou et al. (2013) for NS-FD ground motions. The effect of cyclic structural degradation on inelastic displacement ratios for pulse-like ground motions was also studied by Ruiz-Garcìa (2011) but no relation applicable for $\rm C_2$ in NS conditions was suggested . Another study by Erduran and Kunnath (2010),

proposed an improved relation for C_2 , having also investigated the effect of degradation on the inelastic response to pulse-like NS records. According to Akkar and Metin (2007), implementing *moderate* stiffness degradation during response history analysis (RHA) of several generic frames, led to an average increase of peak roof displacement of the order of 7%, when compared to corresponding analyses with bilinear behavior. While following one of the aforementioned approaches to also incorporate a modified coefficient C_2 in this adaptation of the DCM for NS conditions appears feasible, the added complexity could hinder the objective evaluation of the resulting demand estimates. With this in mind, in the applications presented later on in this paper, exclusively modern code-conforming buildings are considered, exhibiting a beam-sway mechanism at collapse, for which it is assumed that only limited degradation occurs. Therefore, C_2 coefficient is constrained to unity in what follows.

Last, coefficient C_3 was aimed at accounting for increased inelastic displacements in cases where second order (or P- Δ) effects become an important factor resulting in negative postyield stiffness for the equivalent SDOF approximation. It was suggested in 0 that instead of a displacement modification coefficient, an upper limit on strength reduction factor (see below) should be considered, beyond which dynamic instability is likely to occur. Alavi and Krawinkler (2004) reported that pulse-like ground motions may be more sensitive to phenomena of dynamic instability due to P- Δ effects than non-pulse-like ground motions. However, the issue of whether or not the C_3 coefficient should be maintained remains outside the scope of the present study and C_3 is also taken as unity hereafter.

3.2 DISPLACEMENT RATIOS OF ORDINARY AND PULSE-LIKE RECORDS

In FEMA-440 it was recommended that inelastic displacement ratio C_1 be estimated from Equation (3.2), depending on strength reduction factor R and a site-subsoil-dependent parameter α (T is the period of vibration).

$$C_{1} = C_{R|nopulse} = \begin{cases} 1 + (R - 1)/(0.04 \cdot \alpha) & T < 0.20s \\ 1 + (R - 1)/(\alpha \cdot T^{2}) & 0.20s \le T < 1.00s \\ 1.00 & T \ge 1.00s \end{cases}$$
(3.2)

The strength reduction factor R appearing in Equation (3.2), is the reciprocal of SDOF yield strength, F_y , normalized with respect to the maximum elastic force induced by the ground motion on an infinitely elastic SDOF structure, F_e , (Equation 3.3).

$$R = F_e / F_v$$
(3.3)

In fact, inelastic displacement ratios of NS pulse-like ground motions, systematically differ, both in amplitude and shape, from those obtained for ordinary ground motions, and it was discussed by Ruiz-Garcia (2011) that C₁, as given by Equation (3.2), is not explicitly representative of the particular spectral shape associated with impulsive records. Hence the notation $C_{R|nopulse}$ for C_1 , which indicates that Equation (3.2) is hereafter only used when ordinary (non-impulsive) ground motions are considered.

Equation (3.4) was proposed by lervolino et al. (2012) for the (constant-strength) inelastic displacement ratio, $C_{R|pulse}$, based on a dataset of pulse-like FD ground motions identified as such in previous works (Baker, 2008 and Chioccarelli and lervolino, 2010). Using non-linear regression, estimates were obtained for the parameters $\theta_i \{i = 1, 2, 3, 4, 5\}$ and are given in Table 3.1. A graphical representation of Equation (3.4) is provided in Figure 3.1. The most important feature of this analytical model for $C_{R|pulse}$, is the use of normalized period T/T_p as a predictor variable in order to capture the spectral regions of inelastic response amplification.

$$C_{R|pulse} = \frac{S_{d,inel}}{S_{a} \cdot (T^{2}/4\pi^{2})} = 1 + \theta_{1} \cdot (T_{p}/T)^{2} \cdot (R-1) + \theta_{2} \cdot (T_{p}/T) \cdot \exp\left\{\theta_{3} \cdot \left[\ln(T/T_{p}-0.08)\right]^{2}\right\} + \theta_{4} \cdot (T_{p}/T) \cdot \exp\left\{\theta_{5} \cdot \left[\ln(T/T_{p}+0.5+0.02 \cdot R)\right]^{2}\right\}$$
(3.4)



Table 3.1 Coefficient estimates for Equation (3.4).

Figure 3.1 Inelastic displacement ratio of near-source pulse-like ground motions according to lervolino et al. (2012).

3.3 HAZARD DISAGGREGATION AND NEAR-SOURCE INELASTIC DEMAND

Disaggregation of NS seismic hazard can be performed once NS-PSHA results are available. Given, for example, the exceedance of an IM threshold of interest, it serves to obtain the probabilities (or probability functions) of some variables appearing in Equations (2.7,2.8) being causative for such an event (Chioccarelli and lervolino, 2013). In fact, hazard may be disaggregated given either the exceedance or the occurrence of a fixed level of the IM and therefore all directly obtainable results are conditional on either $S_a(T) > s_a$ or $S_a(T) = s_a$.

The probability density of pulse period $f_{T_p|S_a(T)=s_a,pulse}$ conditional on occurrence of a given design hazard threshold, $S_a(T) = s_a$, is relevant in the implementation of the DCM in NS

conditions, as it is required in order to directly compute the expected value of C_R given the hazard level, according to Equation (3.5).

$$E[C_{R} | S_{a}(T) = s_{a}, pulse] = \int_{t_{p}} E[C_{R} | S_{a}(T) = s_{a}, T_{P} = t_{p}, pulse] \cdot f_{T_{P}|S_{a}(T) = s_{a}, pulse}(t_{p}) \cdot dt_{p}$$
(3.5)

Note that the conditional expectation $E[C_R | S_a(T) = s_a, T_P = t_p, pulse]$ appearing in Equation (3.5) corresponds to Equation (3.4) herein.

Some attention should be drawn to the occurrence of the given hazard level, rather than its exceedance, as the conditioning event. One interpretation may be that even if the design elastic demand is usually determined on the basis of the exceedance probability of an IM within a time-frame at the site of interest (i.e., from the hazard curve), the subsequent structural analysis may be seen as conditional to that IM level (e.g., given the occurrence of the design spectral value). Indeed, in modern seismic code approaches, the structure is not required to be safe for the occurrence of IMs larger than that considered for design. In fact, assuming zero failure probability for IMs lower than that used for design and disregarding additional safety factors, the probability of the design IM being exceeded virtually coincides with the, implicitly accepted, risk of the structure overstepping a performance level, up to – and including – collapse.

Apart from PDFs of pulse period, another useful result can be obtained from disaggregation of NS hazard, namely, the conditional probability of pulse occurrence, $P[pulse | S_a(T) = s_a]$. This can be alternatively expressed as the probability that a pulse-like ground motion will be causative for the given hazard level.

The latter probability may in turn be used to estimate NS inelastic demand δ_{t-NS} , via the conditional expectation theorem, as an average of two separate contributions: target displacement given pulse occurrence $\delta_{t|pulse}$ and absence thereof $\delta_{t|no\,pulse}$. These two terms are weighted by their probability of occurrence conditional to the scenario of interest, Equation (3.6).

$$\delta_{t-NS} = \delta_{t|pulse} \cdot P[pulse | S_a = s_a] + \delta_{t|no pulse} \cdot (1 - P[pulse | S_a = s_a])$$
(3.6)

3.4 DESIGN SCENARIOS AND BUILDING MODELS

3.4.1 Probabilistic hazard with and without pulse like effects

Three design scenarios were considered to evaluate the impact of adjusting the DCM to near-source conditions. All of them refer to a hypothetical 200 km long strike-slip seismic source and two possible construction sites (Figure 3.2). Site A is aligned with the fault's strike and is located at a distance of 5 km off the tip. Site B is at 9 km from the same extremity, but in a direction normal to the fault's strike.

The main criterion for selecting these specific positions relative to the fault was for the two sites to exhibit the same level of design hazard (i.e., elastic spectrum ordinates) over a period range of interest $(T = 0.50s \div 1.00s)$, when said hazard is estimated by means of *classical* PSHA (i.e., where NS effects are not explicitly considered – see for example Reiter, 19900) for a return period of 975 yr. This was to ensure that similar structures located at either of these sites would be designed to resist the same base shear. Thus, observed differences in terms of strength reduction factors R will be attributable to NS effects, as will be elaborated later on. In order to also exclude potential soft soil site effects, subsoil conditions at both sites were taken to correspond to stiff soil deposits with a shear wave velocity averaged over the first 30 m of terrain, V_{s,30}, equal to 400 m/s.



Figure 3.2 Schematic representation of site-source configuration for the design scenarios considered.

The first two design scenarios correspond to these two sites when seismicity on the fault is (arbitrarily) assumed to follow a Gutenberg-Richter (G-R) (Gutenberg and Richter, 1944) relationship bounded between magnitude (M) 4.5 and M 7.5, with unit negative slope and a mean annual rate of event recurrence v = 0.20. A third design scenario, the choice of which will be clear later on, was also considered with reference to Site A. In this case, source seismicity was assumed to correspond to a simplified characteristic earthquake (CE) model; i.e., a single magnitude M 7.0 is assumed. Annual rate of earthquake recurrence for the third scenario was assumed to be 1 event/200 yr (v = 0.005) which was selected on the basis that classical hazard in the $T = 0.50s \div 1.00s$ range be approximately equal to the one resulting from the G-R model assumption. This extends the premise of shared design spectral values among all considered scenarios.

Recalling the assumption that earthquake recurrence follows a homogeneous Poisson process, uniform hazard spectra (UHS) were computed for two return periods T_R = 975 yr and 2475 yr (5% and 2% probability of $S_a(T) > s_a$ in 50 yr respectively) for all three scenarios. The UHS from classical hazard calculations are shown in Figure 3.3a.

Regarding NS-PSHA, point A and point B were intentionally selected to correspond to site-to-source configurations both prone to FD effects, yet to a different extent; e.g., the probability that the 2475 yr return period $S_a(T = 0.50s)$ will be exceeded due to an impulsive - rather than an ordinary - record was computed to be 76% for Site A, while for Site B the same probability was found to be 32% (assumptions underlying these calculations to follow).

In all three scenarios, seismic hazard was calculated through NS-PSHA (as outlined in Section 2). For this computation, a uniform distribution of potential epicenters along the fault was assumed.



Figure 3.3 Uniform hazard spectra computed for the various design scenarios by either performing classical PSHA calculations (a) or by considering NS-FD effects in the hazard computation (b).

UHS were computed for the same two return periods of 975 and 2475 yr as in the classical hazard case above. In Figure 3.3b, the NS spectra for the three cases are presented. Note that in the G-R scenario there is visible spectral amplification due to FD - with respect to the classical (Figure 3.3a) case - mostly affecting periods around T=0.50s. This is a consequence of T_p dependence on causal magnitude combined with the narrowband amplification scheme of Baker (2008) adopted in the NS-PSHA calculations (note that the exponential magnitude distribution of G-R seismicity leads to a preponderance of lower magnitudes in the determination of hazard at nearby sites while median T_p for M 5.0 is 0.43s). On the other hand, FD in the CE case mostly affects a range of longer spectral periods beyond those represented in the figure, which explains the proximity of the classical and NS-UHS (median T_p for M 7.0 being 3.67s).

In Table 3.2, $S_a(T)$ values defining NS seismic hazard are reported for the three

design scenarios described above, two return periods corresponding to design performance levels and three spectral periods (T equal to 0.50s, 0.75s and 1.00s), which correspond to the fundamental periods of the structures considered in the following. The lower spectral ordinates encountered at Site B in comparison with Site A are attributable to the different orientation of the two sites with respect to the fault, which, as mentioned, makes the former less prone to FD (i.e., lower conditional pulse occurrence probability) than the latter (see also Chioccarelli and lervolino, 2014).

	Т	_R =2475yr						
	SITE	A	SITE B	SIT	ΕA	SITE B	T _R =975yr	
	G-R CE		G-R	G-R CE		G-R	hazard, equal in all cases	
$S_a(T = 0.50s)$	0.612 g	0.466 g	0.456 g	0.418 g	0.296 g	0.309 g	0.293 g	
$S_a(T = 0.75s)$	0.458 g	0.382 g	0.352 g	0.294 g	0.221 g	0.229 g	0.215 g	
$S_a(T=1.00s)$	0.348 g	0.303 g	0.271 g	0.213 g	0.167 g	0.172 g	0.161 g	

 Table 3.2 Spectral acceleration values at periods of interest.

3.4.2 Disaggregation results

Disaggregation of NS hazard was performed conditional on *occurrence* of $S_a(T) = s_a$, at the three periods of vibration in Table 3.2, and for both return periods considered. The PDFs of T_p for the 2475 yr return period are shown in Figure 3.4.



Figure 3.4 PDFs of pulse period T_p , resulting from disaggregation of NS hazard, conditional on pulse occurrence and $S_a(T) = s_a$, referring to 2745yr return period for each scenario (histograms normalized to unit area). Dashed lines indicate the location

of the mean, $E \left[T_p \left| S_a \right| = s_a \right]$, whose value is also shown along with standard deviation

 $\sigma_{T_p|S_a=S_a}$.

3.4.3 Structural models

The chosen set of structures consists of three reinforced concrete (R/C) plane frames: a 4storey, a 5-storey, and a 6-storey frame (Figure 3.5). These frames were chosen to correspond to the internal frames of perfectly symmetric buildings without in-fills. Furthermore, structure geometry was selected so that all frames would exhibit first-mode dominated dynamic elastic response (first mode participating mass ratios in excess of 80%), with first-mode periods of natural vibration T₁ equal to 0.50s, 0.75s and 1.00s respectively, which justify the period range discussed above. The consideration of similar structures – bar first mode period – was a conscious choice, the objective being to evaluate the potentially different effects of FD at various spectral ordinates, whilst remaining within the DCM applicability domain.

All three structures were designed against gravity loads and seismic actions according to modern codes (eurocodes 2 and 8), in a manner that ensures flexure-dominated inelastic response when subjected to increasing lateral forces. More specifically, each frame was designed for inelastic response corresponding to a *behavior factor* \approx 4.0 under the actions of

the 975 yr return period site-specific, classical UHS (Figure 3.3a). Design values of $S_a(T)$

are given in the last column of Table 3.2. These acceleration values are divided by the behavior factor to determine the actions under which the structures are expected to remain elastic. Material qualities assumed for design were C20/25 for concrete and S500/550 for reinforcing steel. A summary of final detailing is given in Figure 3.5.

All three frames were considered in the context of each of the three design scenarios described above, in the direction normal to the fault's strike (Figure 3.2), leading to eighteen cases because of the two return periods. Inelastic displacement demands were estimated using the DCM at two performance levels: *significant damage*, assumed to correspond to seismic action with a 5% probability of exceedance in 50 yr (T_R = 975 yr), and *near collapse*, corresponding to seismic action with a 2% probability of exceedance in 50 yr (T_R = 2475 yr).

Initially, pushover (base shear versus roof displacement) curves were obtained for all three structures (also shown in Figure 3.5). The non-linear structural models built for these inelastic static analyses, adopted a lumped plasticity approach, using a multi-linear moment-plastic rotation relation. The elastic stiffness of R/C members was modeled using a smeared crack approach. Moment-rotation relationships for each member were estimated using mean strength and stiffness properties for confined concrete and reinforcing steel. The bilinear approximations of the resulting relations used the *collapse prevention* limiting values recommended in 0356 for ultimate chord rotation capacity.

The static non-linear (pushover) analyses were carried out by applying a gradually increasing lateral force profile which remained unchanged throughout each analysis and corresponds to each structure's first mode excitation to base acceleration (first mode eigenvectors shown in Figure 3.5). Second order (P- Δ) effects were incorporated into the analyses on all accounts, yet collapse mechanisms were characterized by plasticization at the beam ends and the bases of ground floor columns (beam-sway mechanisms), as a consequence of conformity to capacity design rules leading to positive post-yield stiffness of the equivalent SDOF systems.

3.5 IMPLEMENTING THE DCM IN NS CONDITIONS

Once the pushover curves were obtained, the constituent terms of the right-hand-side of Equation (3.6) had to be estimated separately. For the estimate of the elastic demand, which is needed to compute both $\delta_{t|nopulse}$ and $\delta_{t|pulse}$, the NS-UHS computed for each design scenario and performance level was used (shown in Figure 3.3b, in addition to which S_a values are given in Table 3.2). Then, the non-impulsive contribution $\delta_{t|nopulse}$ was obtained by simple implementation of the DCM in its traditional form using $C_{R|nopulse}$ from Equation (3.2), in which subsoil coefficient α was set equal to 90, corresponding to $V_{s,30}$ =400 m/s (NEHRP class C subsoil). For the estimation of the impulsive contribution $\delta_{t|pulse}$, Equations (3.4) and (3.5) were used to compute the mean inelastic displacement ratio for FD ground motions, $C_{R|pulse} = E[C_R | S_a(T) = s_a, pulse]$.

It is to recall that these target displacements, in the DCM, require a bilinear approximation of the pushover curve, which was constructed via the methodology suggested in FEMA-356. This method requires that the bilinear approximation intersect the pushover curve at the target displacement δ_t thus resulting in some positive (in this case) post-yield stiffness. This

hardening behavior is typically ignored when estimating $C_{R|nopulse}$ via Equation (3.2). However, this matter will not be discussed here. What should be mentioned is that this method of selecting the equivalent bilinear system, implies that the base shear corresponding to conventional yield, V_y , is dependent on target displacement δ_t , thus the evaluation of both the impulsive and non-impulsive contributions requires some iteration for the estimation of strength reduction factor (see also Baltzopoulos et al., 2013).

A graphical representation (corresponding to the converged iteration) for each of the two inelastic displacement contributions considered in Equation (3.6), is given in Figure 3.6 for the 4-storey frame situated at Site A, under the assumption of G-R seismicity and for the near collapse performance level.

Given that, under these conditions, a 74% probability was computed for pulse occurrence conditional to the hazard threshold (i.e., from disaggregation of NS hazard), applying Equation (7) one obtains the result in Equation (3.7).

$$\delta_{t}^{NS} = \delta_{t|pulse} \cdot 0.74 + \delta_{t|nopulse} \cdot 0.26 = 7.1 \cdot 0.74 + 5.6 \cdot 0.26 = 6.7 \text{ cm}$$
(3.7)

So as to better appreciate this result, it is useful to also obtain a target displacement without explicitly accounting for FD effects, hereafter termed ordinary target displacement, δ_t^{ord} . In order to evaluate δ_t^{ord} one simply has to use the classical DCM (Equation 3.1) and the classical PSHA uniform hazard spectrum corresponding to each design scenario (Figure 3.3a), to represent elastic demand. For the case to which Equation (3.7) refers (4-storey frame at Site A, G-R seismicity, near collapse), one obtains $\delta_t^{ord} = 3.8 \text{ cm}$, which means that accounting for FD lead to a 77% increase in target displacement. It may be worthwhile to

underline that both target displacements $\delta_{t|nopulse}$ (ordinary component of NS demand) and δ_t^{ord} (no consideration of NS effects) are derived by applying coefficient $C_{R|nopulse}$ (Equation 3.2), valid for ordinary ground motions, yet using different spectral values (from NS-PSHA and classical PSHA, respectively).



Figure 3.5 Geometry, detailing (flexural reinforcement), modal information and pushover curves for the three R/C frames used in the application.



Figure 3.6 Graphical representation of application of the DCM for a 4-storey R/C frame (T=0.50s) at Site A under G-R seismicity. Target displacement estimates for near collapse performance level (TR=2475 yr) considering impulsive (a) and non-impulsive (ordinary) contributions (b).

The results of the application of the DCM to all cases presented in the previous section are summarized in Table 3 to facilitate comparisons. It can be observed that the effect of FD on inelastic displacement demand was more pronounced for lower performance levels, which correspond to longer T_R .

Table 3.3 Summary of target displacement estimates resulting from application of the DCM. Two different performance levels per design scenario, per structure considered.

Column C_{Ripulse} reports mean inelastic displacement ratio conditional on pulse

occurrence while	$C_{R nopulse}$	denotes	mean	inelastic	displacement	conditional	on no	pulse
			C	occurring				

	T _R	First mode period	$\boldsymbol{C}_{R \text{pulse}}$	$C_{R\mid nopulse}$	$\delta_{t pulse}$ (mm)	$\delta_{t nopulse}$ (mm)	$P[pulse S_a = S_a]$	$\delta_t^{ m NS}$ (mm)	$\delta_{t}^{\mathrm{ord}}$ (mm)	$\frac{\boldsymbol{\delta}_{t}^{NS}-\boldsymbol{\delta}_{t}^{ord}}{\boldsymbol{\delta}_{t}^{ord}}$
ty		0.50s	1.44	1.12	71	56	0.741	67	38	77%
eismici	2475 yı	0.75s	1.31	1.06	111	90	0.673	104	63	65%
E A hter se del	Z	1.00s	1.21	1.04	137	118	0.629	130	83	57%
SIT erg-Ric mo		0.50s	1.17	1.08	40	37	0.687	39	24	63%
utenbe	975 yr	0.75s	1.09	1.04	60	56	0.602	58	40	46%
Ō		1.00s	1.04	1.02	72	70	0.513	71	53	34%
odel	L	0.50s	3.77	1.07	143	40	0.170	58	39	47%
lake mo	2475 y	0.75s	3.10	1.04	218	73	0.166	97	71	37%
'E A arthqu		1.00s	2.51	1.03	248	101	0.165	125	96	30%
SIT istic E		0.50s	3.13	1.05	75	25	0.100	30	24	25%
aracter	975 yı	0.75s	2.03	1.02	83	42	0.073	45	40	12%
Cha		1.00s	1.72	1.01	94	55	0.060	57	53	8%
ty	<u>ب</u>	0.50s	1.62	1.09	60	40	0.280	46	36	27%
eismici	2475 y	0.75s	1.46	1.05	95	68	0.245	75	61	22%
E B hter se del		1.00s	1.28	1.03	113	91	0.243	96	81	19%
SIT erg-Ric mo		0.50s	1.17	1.05	29	26	0.225	27	24	11%
utenbe	975 yr	0.75s	1.12	1.03	47	43	0.181	44	40	9%
U		1.00s	1.06	1.02	60	57	0.150	57	53	8%

A number of observations can also be made, by comparing the DCM estimates of inelastic displacement demand among the design scenarios considered herein. A comparison between Site A and Site B, under the working assumption that seismic hazard at both sites is dictated by the same single source following a G-R law, must necessarily focus on the fact that the position and orientation of Site A relative to the fault, is decidedly more unfavourable than that of Site B, when potential FD effects are concerned. Although this was in part expected beforehand (given existing empirical models such as lervolino and Cornell, 2008 and also recent investigations such as Chioccarelli and lervolino, 20140) it is also confirmed in a most emphatic manner by the results of NS-PSHA and hazard disaggregation; probabilities of pulse occurrence given the hazard threshold computed at Site A are more than twice the ones computed for Site B and the amplification of spectral ordinates at Site A due to FD is accordingly more pronounced (Table 3.2).

Given the occurrence of hazard levels associated with near collapse performance, both sites appear most likely to be affected by pulse-like ground motions characterized by T_p between 0.50s and 1.00s, with the modal value for each case corresponding to a ratio of $T/T_p \approx 1$. This effect can be affirmed from the left-skewed probability densities of T_p (Figure 3.4) and can be attributed to the exponential distribution of magnitude associated with the G-R model. As a result, the realization of T/T_p ratios belonging in the range of high inelastic amplification (as such ranges were documented by Ruiz Garcia, 2011, lervolino et al., 2012 and Akkar et al., 2004) is associated with low probability, conditional on the hazard. Thus, the difference between NS and ordinary structural response, at both sites, is primarily influenced by the elastic component, which is duly amplified by the more frequently occurring, shorter duration pulses.

A comparison, regarding FD effects, between the two different seismicity models considered at Site A comes in stark contrast with the one directly above. The CE model is associated with events of lower rate, yet greater average magnitude and consequently longer expected pulse duration, which leave the elastic spectral ordinates in the range considered largely unaffected (Figure 3 is particularly eloquent to this effect). Furthermore, the conditional probabilities of pulse occurrence from hazard disaggregation are lower than either of the two G-R cases; loosely speaking, the expected long-period pulses, are less likely to be responsible for reaching the hazard threshold at $T = 0.50s \div 1.00s$ than ordinary ground motions are. However, due to the fact that the higher mean T_p corresponds to a T/T_p ratio,

which translates into potentially aggressive pulse-like ground motions, expected inelastic demand is almost as large as under the G-R model scenario. In other words, the CE seismicity model, presents a case where, *for a given range of periods*, the NS elastic response spectrum hardly departs from the traditional case and yet expected inelastic demand greatly supersedes that of the classical case, resulting as a weighted average between the more frequent, benign ground motions and some rare pulse-like ground motions, which can cause larger excursions into inelasticity.

3.6 DCM VERSUS NON-LINEAR DYNAMIC ANALYSIS

Even though validating the results of non-linear static procedures is an open issue in earthquake engineering (e.g., Kalkan and Kunnath, 20070) and remains beyond the immediate purposes of the work presented herein, which acknowledges the DCM as an established procedure, it may be useful to ensure that dynamic RHA using recorded ground

motions, consistent with the models above, provide comparable design targets. With this aim, out of the various cases addressed in the preceding sections, two were selected: the five- and six-storey frames (T_1 =0.75s and 1.00s, respectively) subjected to the 975 yr return period seismic hazard at site A in the M 7.0 CE scenario.

3.6.1 Selection of ordinary records

In this exercise, the pulse-like and non-pulse-like cases were treated separately with regard to the selection of real ground motions. For the non-pulse-like case (indicated above by the *nopulse* notation), a suite of 20 ordinary records was selected to match a target spectrum using the methodology proposed by Jarayam et al. (2011). Said target spectrum is a *conditional mean spectrum* (CMS), whose computation requires the average causal magnitude and Joyner-Boore distance, $\left(\overline{M}, \overline{R}_{\rm JB}\right)$, given absence of a directivity pulse. These

values are obtainable from disaggregation of the 975 yr NS seismic hazard, at the two considered structural periods and are reported in Table 3.4, along with the number of standard deviations (in log-space) that separate the design value of $S_a(T)$ from the median

– a parameter known as epsilon (ϵ). Having obtained $(\overline{M}, \overline{R}_{JB}, \epsilon)$, the conditional mean spectral values at other periods and their conditional variances could be calculated, using the ground motion prediction equation of Boore and Atkinson (2008) and the correlation model of Baker and Jarayam (2008), for each of the two cases.

Table 3.4 Results from disaggregation of NS hazard (given absence of directivity pulse and occurrence of S_a) used for the selection of the ordinary ground motion record set.

T ₁	$S_{a}(T_{1})$	$\overline{\mathbf{M}}$	\overline{R}_{JB}	$\epsilon(S_a)$
0.75s	0.221g	7.0	48.5 km	0.865
1.00s	0.167g	7.0	52.6 km	0.897

As can be seen in Table 3.4, the values assumed by the conditioning parameters differ only slightly between the two cases, leading to similar shapes of conditional mean spectra. For this reason, a single suite of records was chosen to represent the ordinary component of seismic hazard at both periods (naturally with differing scale factor). The selected records (Table A.1) are from a subset of the NGA database (see Jarayam et al., 2011) from which pulse-like ground motions were excluded and each was linearly scaled to exhibit the design $S_a(T)$. This ground motion selection strategy is summarized in terms of response spectra in Figure 3.7a, where the target CMS can be seen and where each individual record has been scaled at a common $S_a(0.75s) = 0.221g$.

3.6.2 Selection of pulse-like records

For the pulse-like case, a different record selection strategy had to be followed, due to the fact that $S_a(T)$ is not a sufficient IM when pulse-like ground motions are concerned as shown by Tothong and Cornell (2008). For this reason, some methodologies for the selection and scaling of pulse-like records have been proposed based on advanced IMs by Baker and Cornell (2008) and Tothong and Luco (2007); be that as it may, compatibility with current design practice and the DCM, requires that reference to the design spectrum – and therefore use of S_a as IM – be maintained.

The problem that the directivity case poses for record selection can be summarized as follows: for a specific structure with given strength, some pulse-like ground motions are particularly aggressive, resulting in high ductility demand while others prove relatively benign, leading to structural behavior reminiscent of ordinary records. Inclusion of arbitrary numbers of either type of record will thus lead to biased estimates of NS inelastic demand (see Tothong and Cornell, 2008). Ideally, assembling a set of pulse-like records that closely reflects hazard at a NS site in terms of pulse period, should address the aforementioned problem, since it is known that T_n plays an important role in determining SDOF and MDOF inelastic demand. However, this is not the case due to the small number of registered directivity ground motions. Indeed, if one attempts to closely match the marginal density of T_{p} from disaggregation – such as the one presented in Figure 3.8a – he is faced with the problem that in some T_n intervals there may be very few records to choose from – if any. Since it is unlikely that a sample as small as a couple of records will reproduce the average trend of inelastic response for some interval of T_p , this can lead to biased estimates of NS inelastic demand. In order to address this problem posed by the relative scarcity of available pulse-like records within some specific T_{p} range restrictions, the following steps were taken: first, the cumulative distribution function (CDF) of T_n was used to divide the available dataset of pulse-like ground motions (which consists of the impulsive records used by lervolino et al. (2012) with the addition of some records from more recent events, as will be discussed in the next section), into 5 bins of equal probability (Figure 3.8b).



Figure 3.7 Response spectra of the ordinary (a) and pulse-like (b) scaled records selected for the non-linear dynamic analysis of the 5-storey R/C frame (T=0.75s). Also shown is the NS uniform hazard (design) spectrum of the considered scenario and – in the case of the ordinary record set – the target conditional mean spectrum.

Given a target number of 20 pulse-like ground motions for the selection, this entails extracting four records from each bin. This strategy effectively relaxes the requisite of closely reflecting the distribution of T_p predicted by NS hazard yet – as an offset – provides more densely populated record bins from which to choose. This procedure is analogous to that employed by Almufti et al. (2013). The second step consisted is calculating the average pulse period $\overline{T_p}$ for each bin, deriving the corresponding inelastic displacement ratio $C_{R|pulse}\left(T_1/\overline{T_p}\right)$ from Equation (3.4) and finally selecting four records from within each bin whose inter-bin average inelastic spectra match this $C_{R|pulse}$ as closely as possible. Thus,

even when a bin spans a range of rare pulse periods, such as the one denoted on Figure 3.8b, the selection is guided towards the average trend exhibited by the entire dataset of impulsive ground motions in an effort to avoid bias due to the scarcity of records within the bin.

This record selection strategy resulted in two sets of pulse-like ground motions being assembled, one for each of the two cases considered. All pulse-like records were scaled to a common spectral ordinate at the first mode period of each structure. In the case of ordinary ground motions, it has been shown that, to some extent, this type of scaling does not introduce bias to inelastic response (see Shome et al., 1999). This approach was maintained for the pulse-like directivity case as well (see for example Figure 3.7b), since the target distributions of T_p were obtained from disaggregation conditional on occurrence of these

 $S_a(T)$ values. In Figure 3.9, it can be seen the degree with which these distributions were matched by the selected record sets, despite having relaxed this criterion due to the binning strategy adopted. The suites of design ground motions obtained (Table A.2) can be said to reflect the impulsive portion of NS seismic hazard for the considered cases.



Figure 3.8 (a) PDF of pulse period from disaggregation of NS hazard (T=1.00s, TR=975 yr) and (b) corresponding CDF multiplied by intended number of pulse-like records to be selected and divided into five bins of equal probability for the calculation of inelastic displacement ratio corresponding to the average pulse period of each bin.



Figure 3.9 Comparison of target densities of pulse period with T_p histograms of the selected pulse-like ground motion sets for the T=0.75s five-storey frame (a) and the T=1.00s six-storey frame (b). The probability densities have been scaled in order for their areas to coincide with those of the histograms. Relevant statistics also are shown.

3.6.3 Non-linear response history analyses

Having obtained these record sets, non-linear models of the two frames were finally each subjected to the two suites of scaled ordinary and pulse-like ground motions. Results in terms of peak roof displacement for each individual record can be found in Tables A.1-2 of the appendix. Note that in the case of the six-storey frame, the El Centro Array #10 record of the Imperial Valley earthquake (California, 1979) and the Lucerne record of the Landers earthquake (California, 1992) both caused collapse of the structure, even though the level of seismic hazard under consideration corresponds to a damage limitation performance level; thus, the roof displacement values reported in Table A.2 are the maximum values attained prior to the onset of dynamic instability. A summary of the dynamic RHA is given in Figure 3.10, where relevant response statistics and corresponding DCM estimates, carried over from Table 3.3, are also reported.

It can be observed that dynamic RHA results indicate an overestimation of inelastic demand due to directivity by the DCM adaptation to NS conditions, of the order of 12%. This can be partly attributed to the fact that the continuous lognormal model for T_p employed during NS-

PSHA cannot be effectively reproduced by recorded ground motions due to the rarity of very long duration directivity pulses, in excess of 10s. Furthermore, the RHA confirms the premise that NS inelastic demand due to potential directivity effects can supersede ordinary demand enough to merit special consideration; this, is in agreement with the findings of the previous studies of Akkar and Metin (2007) and Champion and Liel (2012) (note that the latter study dealt with the effect of FD on collapse probability, while the present work deals with its effect on mean demand, rather than probability of exceeding capacity).



Figure 3.10 Histograms of maximum inelastic roof displacement resulting from nonlinear dynamic RHA for the five-storey (T_1 =0.75s) frame subjected to the pulse-like (a) and ordinary (b) excitation suite as well as the respective results for the six-storey (T_1 =1.00s) frame (c) and (d).

4 Conclusions

The presented study dealt with the implementation of various procedures intended to estimate seismic structural response in NS conditions, namely NS-PSHA and the DCM. While the former methodology was implemented with view of estimating elastic demand, the latter was intended for estimating inelastic design demand in a simplified manner. A set of illustrative applications was also provided, where single-fault NS design scenarios, assuming different site-to-source configurations and source seismicity were considered in order to represent a variety of cases with respect to expected forward directivity effects.

The range of spectral periods where hazard increments due to forward directivity effects are significant, as well as the form and amplitude of said increments, can be directly derived from the model of magnitude occurrence on the fault. This dependence stems from the relationship between pulse period and event magnitude. These hazard increments were found to be strongly dependent on the considered spectral ordinate. Thus, more *hazardous* spectral periods can be predicted given some knowledge of the fault's characteristics in terms of generated magnitudes. For investigated cases, shapes of hazard increments contours are similar even if different generated magnitudes are considered and minor differences depend only on the different rupture lengths; it was also possible to geometrically identify the area affected by relevant directivity effects, independently of the specific characteristics of the considered fault in terms of event magnitude. These results suggest that, some general rules could be identified, given a sufficient number of parametric analyses.

The DCM was implemented for modern-code-conforming R/C frames and compared to design for classical hazard and inelastic demand. The results may help to quantify the significance of accounting for NS-FD in structural design and assessment. Inasmuch as the DCM can provide a useful estimate of structural seismic performance in the inelastic range, FD was shown to induce appreciable increase – in an engineering sense – in displacement demand, particularly when longer return period performance levels are considered. This behaviour was further confirmed when dynamic RHA was performed using suites of ground motions carefully selected in order to reflect NS demand for such a design scenario.

Depending on the distribution of causal event magnitudes most likely to characterize a given source, potential directivity may be manifest by means of relatively short duration pulses, comparable with the periods of natural vibration of typical building structures. This type of impulsive records would mostly affect the elastic response of such structures; that being the case, computing design spectra by means of NS-PSHA should constitute the key step towards estimating NS inelastic response, combined with use of inelastic spectra for NS-FD. However, it was also shown that there are cases where NS effects have small-to-negligible influence on seismic hazard (expressed in elastic response IMs) around a specific spectral region, and yet produce more pronounced increase in mean inelastic demand for structures whose fundamental period places them in that portion of the elastic response spectrum. The non-linear dynamic analyses carried out corroborate this finding. It was shown that this effect can be explicitly accounted for in structural analysis by use of NS hazard disaggregation results, which provide additional information with respect to the design spectrum.

The amount of hazard increments seems to be largely dependent on the characteristics of the studied cases (geometry above all). Because the pulse period prediction model depends

The displacement coefficient method in near-source conditions

on the event magnitude with a significant heterogeneity, it was also shown that hazard increments often affect a large range of periods.

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Appendix A

Table A.1: Set of ordinary ground motion records used for the RHA of the 5- and 6-storey frames and results for maximum roof displacement.

No	Earthquake Name	Station Name	Year	М	Mech.	R _{JB} km	ClstD km	V _{s,30} m/s	PGV cm/s	SF 0.75s	SF 1.00s	u _{roof,max} 5st. mm	u _{roof,max} 6st. mm
1	Borrego Mountain	San Onofre - So Cal Edison	1968	6.6	S-S	129	129	443	3.7	3.602	3.022	37	49
2	Cape Mendocino	Fortuna - Fortuna Blvd	1992	7.0	R	16	20	457	24.7	0.975	0.907	45	54
3	Chi-Chi, Taiwan	CHY090	1999	7.6	R-0	58	58	215	17.0	1.339	1.154	62	57
4	Chi-Chi, Taiwan	CHY104	1999	7.6	R-0	18	18	223	54.9	0.324	0.544	43	60
5	Chi-Chi, Taiwan	HWA049	1999	7.6	R-0	47	51	273	22.4	0.666	0.710	30	41
6	Chi-Chi, Taiwan	ILA010	1999	7.6	R-0	78	80	474	8.3	1.683	2.812	40	46
7	Chi-Chi, Taiwan	ILA067	1999	7.6	R-0	33	39	553	14.3	0.680	0.643	26	35
8	Chi-Chi, Taiwan	TCU085	1999	7.6	R-0	55	58	1000	7.5	2.787	2.869	32	38
9	Chi-Chi, Taiwan	TTN033	1999	7.6	R-0	56	59	273	6.7	2.323	2.205	73	62
10	Chi-Chi, Taiwan	WNT	1999	7.6	R-0	2	2	664	55.4	0.295	0.317	33	52
11	Chi-Chi, Taiwan-04	CHY035	1999	6.2	S-S	25	25	474	12.3	1.328	0.629	23	48
12	Hector Mine	Little Rock Post Office	1999	7.1	S-S	147	147	442	4.6	1.705	2.361	27	37
13	Hector Mine	Valyermo Forest Fire Station	1999	7.1	S-S	136	136	345	6.2	2.675	3.921	28	60
14	Hector Mine	Wrightwood - Nielson Ranch	1999	7.1	S-S	113	113	345	5.4	2.776	6.346	31	65
15	Kocaeli, Turkey	Tekirdag	1999	7.5	S-S	164	165	660	3.8	2.460	3.161	29	48
16	Landers	Baker Fire Station	1992	7.3	S-S	88	88	271	9.7	1.990	1.522	39	67
17	Landers	La Crescenta - New York	1992	7.3	S-S	148	148	446	3.6	3.557	2.218	62	57
18	Loma Prieta	Bear Valley #10, Webb Res.	1989	6.9	R-0	67	68	304	8.5	1.720	1.850	36	64
19	Northridge-01	West Covina - S Orange Av	1994	6.7	R	51	52	309	5.8	2.390	1.788	34	43
20	Tabas, Iran	Dayhook	1978	7.4	R	0	14	660	28.2	0.511	0.489	46	47

R_{JB}: Closest distance to horizontal projection of the fault plane.

ClstD: Closest distance to the fault plane.

Rupture mechanisms S-S: Strike-Slip, R: Reverse, R-O: Reverse-Oblique

PGV: Peak Ground Velocity

SF: Scale Factor

No	Earthquake Name	Station Name	Year	М	Mech.	R _{JB} km	ClstD km	V _{s,30} m/s	PGV cm/s	T _p s	SF 0.75s	SF 1.00s	u _{roof,max} 5st. mm	u _{roof,max} 6st. mm
1	Coyote Lake	Coyote Lake Dam	1979	5.7	S-S	5.3	6.1	597	19.8	0.56	0.809	-	47	-
2	San Fernando	Pacoima Dam (up. left abut)	1971	6.6	R	0.0	1.8	2016	116.5	1.60	0.212	-	62	-
3	Northridge-01	LA Dam	1994	6.7	R	0.0	5.9	629	77.1	1.65	0.225	0.205	55	69
4	Loma Prieta	Gilroy Array #2	1989	6.9	R-O	10.4	11.1	271	45.7	1.72	0.321	0.357	49	67
5	Imperial Valley-06	Agrarias	1979	6.5	S-S	0.0	0.7	275	54.4	2.30	0.533	0.494	41	76
6	Northridge-01	Newhall - W Pico Canyon Rd.	1994	6.7	R	2.1	5.5	286	87.8	2.41	0.290	0.203	81	78
7	Northridge-01	Sylmar - Olive View Med FF	1994	6.7	R	1.7	5.3	441	122.7	3.11	0.277	-	49	-
8	Cape Mendocino	Fortuna - Fortuna Blvd	1992	7.0	R	16.0	20.0	457	22.3	3.14	0.942	-	43	-
9	Imperial Valley-06	EC Meloland Overpass FF	1979	6.5	S-S	0.1	0.1	186	115.0	3.35	0.554	0.315	149	94
10	Northridge-01	Sylmar - Converter Sta	1994	6.7	R	0.0	5.4	251	130.3	3.48	-	0.121	-	62
11	Northridge-01	Sylmar - Converter Sta East	1994	6.7	R	0.0	5.2	371	116.6	3.49	-	0.229	-	61
12	Westmorland	Parachute Test Site	1981	5.9	S-S	16.5	16.7	349	35.8	3.58	0.645	-	58	-
13	Imperial Valley-06	El Centro Array #6	1979	6.5	S-S	0.0	1.4	203	111.9	3.84	-	0.389	-	82
14	Imperial Valley-06	Brawley Airport	1979	6.5	S-S	8.5	10.4	209	36.1	4.03	1.785	0.785	146	78
15	Imperial Valley-06	El Centro Array #5	1979	6.5	S-S	1.8	4.0	206	91.5	4.05	0.404	-	65	-
16	Imperial Valley-06	El Centro Array #10	1979	6.5	S-S	6.2	6.2	203	46.9	4.49	1.576	0.898	195	127*
17	Imperial Valley-06	El Centro Array #4	1979	6.5	S-S	4.9	7.1	209	77.9	4.61	-	0.335	-	73
18	Landers	Lucerne	1992	7.3	S-S	2.2	2.2	685	140.3	5.10	0.327	0.384	49	102*
19	Imperial Valley-06	El Centro Array #3	1979	6.5	S-S	10.8	12.9	163	41.1	5.24	-	1.321	-	89
20	Imperial Valley-06	El Centro Array #8	1979	6.5	S-S	3.9	3.9	206	48.6	5.39	0.551	-	40	-
21	Kocaeli, Turkey	Gebze	1999	7.5	S-S	7.6	10.9	792	52.0	5.87	0.905	0.752	62	79
22	Chi-Chi, Taiwan	TCU101	1999	7.6	R-O	2.0	2.0	273	61.5	6.00	-	0.552	-	91
23	Northridge-01	Lake Hughes #9	1994	6.7	R	24.9	25.4	671	7.3	6.33	4.309	5.287	55	69
24	Imperial Valley-06	El Centro Array #11	1979	6.5	S-S	12.5	12.5	196	41.1	7.36	-	0.783	-	101
25	Darfield, N. Zealand	Templeton School (TPLC)	2010	7.0	S-S	6.0	6.0	250	64.4	8.93	0.920	-	71	-
26	Chi-Chi, Taiwan	TCU128	1999	7.6	R-O	13.0	13.0	600	71.0	9.01	-	0.646	-	92
27	Duzce, Turkey	Lamont 1060	1999	7.1	S-S	25.8	25.9	782	11.3	9.63	6.217	4.191	50	78
28	Chi-Chi, Taiwan	TCU051	1999	7.6	R-O	8.0	8.0	273	41.2	10.39	0.754	-	72	-
29	Chi-Chi, Taiwan	TCU087	1999	7.6	R-O	7.0	7.0	474	41.3	10.40	-	1.093	-	70

Table A.2: Set of pulse-like ground motion records used for the RHA of the 5- and 6-storey frames and results for maximum roof displacement.

* Record caused collapse of the structure; reported roof displacement corresponds to the maximum reliable value from the analysis (maximum roof displacement attained prior to the onset of dynamic instability).