Contemporary issues in earthquake engineering research: processing of accelerometric data, modelling of inelastic structural response, and quantification of near-fault effects

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A dissertation submitted in partial fulfillment of the requirements for the degree of doctor of philosophy

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Abstract

This study focuses on three contemporary issues related to the fields of Earthquake Engineering and Engineering Seismology: (1) processing accelerometric data and assessing permanent ground displacements, (2) modelling the response spectra of inelastic structures, and (3) quantifying near-fault ground-motion effects relevant to engineering structures.

Most of the noise in accelerometric data can be removed by standard signal processing, for example, low-pass and high-pass filters. High-pass filters, if used to adjust baseline errors, can result in inadvertent loss of long-period signal due to, for example, permanent ground displacements in the near-fault area. Despite several efforts, a consistent and well-defined method to adjust baseline errors without losing a part or the whole of permanent displacements (if any) is not available in the literature. This study develops and thoroughly tests such a method by using several near-fault accelerograms.

An inelastic response spectrum is required in the design of earthquake-resistant structures. In the current practice, it is obtained by scaling an elastic response spectrum by some ad-hoc factors, such as, structural behaviour factor, or force reduction factor. This approach lacks a rational basis and is highly uncertain. This study shows that inelastic response spectra can be modelled in a manner similar to elastic response spectra by calibrating Ground-Motion Prediction Equations (GMPEs) for inelastic structures. This is achieved by regressing peak responses of inelastic structures against earthquake parameters (such as, size, faulting mechanism, etc.), path parameters (such as, source-to-site distance), site parameters (such as soil conditions), and structural parameters (such as, undamped natural period of vibration, damping ratio, displacement ductility, yield strength, etc.). The resulting GMPEs can be applied in Probabilistic Seismic Hazard Assessment (PSHA) and Probabilistic Seismic Demand Assessment (PSDA) to estimate uniform-hazard representations of design forces and ductility demands with greater reliability and accuracy than by traditional methods.

Near-fault ground motions affected by forward-directivity effects, which are on the focus of this study, contain most of their energy in a narrow frequency band, and therefore affect certain structures more severely than others. To ensure reliable design/evaluation of structures, important properties of near-fault ground motions, such as their amplitude and frequency content, need to be properly quantified. In this study, peak ground velocity (PGV) and predominant period ($T_d$) of ground motion are used to characterize near-fault ground motions. Mathematical models relating PGV and $T_d$ to earthquake size, source-to-site distance, etc, are calibrated using a large set of near-fault ground-motion data. These parameters, along with structural properties, such as, undamped natural period and damping ratio of an oscillator, are used to quantify the elastic as well as inelastic response spectra of pulse-like ground motions in the near-fault area more adequately than was possible using earlier approaches.
Ágrip

Þessi rannsókn beinist að þremur viðfangsefnum sem nú eru efst á baugi á sviði járðskjálftaverkfræði: (1) úrvinnslu hröðunarmæligagna og ákvörðun varanlegrar færslu yfirborðs, (2) gerð stærðfræðilíkaka af svörunarrófi ólínulegra ólínulegra kerfa og (3) ákvörðun nárvídshlíkaka sem skipta máli fyrir mannvirkjagerð.


Ólínuleg járðskjálftasvörunaróf eru notað við hónum járðskjálftafolinna mannvirkja. Þau eru ákvörðuð samkvæmt nýgildandi reglum með því að kvarða línulegt svörunaróf með þar til grunnlínuskekkju getur leitt til villu í lágtíðnimerki, t.d. vegna varanleggur færslu yfirborðs nálægt upptökum. Þátt fyrir margar tiltraumin hefur hingað að ekki tekist að setja fram áreiðanlegar aðferðir til þess að leidiðetta grunnlínuskekkjum og trygga að ekki glatist hluti af þeirri varanlegu færslu sem kann að hafa átt sér stað. Í þessari rannsókn hefur tekist að þróa síka aðferð og prófa hana ítarlega með því að nota nárvíðshröðunarráðir.

Ölínuleg járðskjálftasvörunaróf eru notað við hónum járðskjálftafolinna mannvirkja. Þau eru ákvörðuð samkvæmt nýgildandi reglum með því að kvarða línulegt svörunaróf með þar til grunnlínuskekkju getur leitt til villu í lágtíðnimerki, t.d. vegna varanleggur færslu yfirborðs nálægt upptökum. Þátt fyrir margar tiltraumin hefur hingað að ekki tekist að setja fram áreiðanlegar aðferðir til þess að leidiðetta grunnlínuskekkjum og trygga að ekki glatist hluti af þeirri varanlegu færslu sem kann að hafa átt sér stað. Í þessari rannsókn hefur tekist að þróa síka aðferð og prófa hana ítarlega með því að nota nárvíðshröðunarráðir.
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1 Introduction

1.1 Background

This dissertation reports on the various research studies conducted during my doctoral study. The study focuses on three contemporary issues in the field of Earthquake Engineering: (1) processing of strong-motion accelerograms, (2) modelling of inelastic response spectra of earthquake ground motions, and (3) characterization of near-fault ground motions and their response spectra.

‘Processing of strong-motion accelerogram’ is used in this dissertation to refer to the various signal processing techniques applied to separate noise, insofar as possible, noise from a strong-motion accelerogram. Most of the noise in an accelerogram can be effectively handled by using standard signal processing techniques. However, a type of noise, known as baseline errors, is not well-understood. Despite several efforts, a consistent and well-defined method to handle baseline errors is not available in the literature. Such a method is developed and tested in this study.

Inelastic response spectra are required in the preliminary design of structures for seismic resistance. The commonly used method of constructing inelastic response spectra by reducing their elastic counterparts involves large uncertainties in its application. A new method of predicting inelastic spectral demands of both constant-ductility and constant-strength systems is proposed. The proposed method is simple, and it can be easily applied in probabilistic applications to calculate uniform-hazard representation of design forces as well as the ductility demands of engineering structures.

Strong motion in the near-fault area can cause severe damage to engineering structures. The special properties of near-fault ground motions and their effects on structures need to be properly characterized to ensure reliable design and/or evaluation of structures located close to an earthquake fault. Most of the work done in this direc-
tion has concentrated on modelling near-fault ground motions with simple waveforms. While simple waveforms are useful in representing recorded near-fault accelerograms, they do not provide consistent and unambiguous characterization of ground-motion amplitude and frequency content in the near-fault area. It is therefore proposed that the amplitude and frequency content of near-fault ground motions be characterized by peak ground velocity and predominant period of ground motion, both of which can be derived from the accelerograms recorded in the near-fault area. Using this characterization, elastic response spectra of near-fault ground motions is modelled as a function of earthquake size, source-to-site distance and the properties of structures; namely, undamped natural period of vibration and viscous damping ratio.

1.2 Dissertation’s organization

The contents of this dissertation are developed around the three issues mentioned above. Most of the results of this doctoral study have been published in peer-reviewed journals in the field of earthquake engineering and engineering seismology; these materials are referred to as ‘papers’. The part related to near-fault ground motions has been submitted to a journal for publication but has not yet been published. All of the published papers and the submitted manuscript are contained in this dissertation. The papers and manuscript are included as self-contained sections. The background information related to these papers and manuscript, literature review related to the issues addressed in them, and examples of their practical applications are contained in the four chapters preceding them. A summary of the papers and manuscript and the dissertation chapters related to them is presented below.

1.2.1 Scientific papers


1.2. Dissertation’s organization


1.2.2 Dissertation chapters

1.2.2.1 Chapter 1

This chapter provides an overview of the dissertation and outlines its organization.

1.2.2.2 Chapter 2

Chapter 2 is related to the processing of strong-motion accelerograms. Common types of noise in strong-motion accelerograms are described with special emphasis on baseline errors. This chapter provides background and supporting information to Paper 3 and Paper 4 listed above. In Paper 3, a new method to correct baseline errors is developed; its results are compared with those of other methods found in the literature, and several illustrations of the proposed correction scheme are provided, using data from two earthquakes; namely, the 1999 Chi-Chi, Taiwan, earthquake and the 2008 Ölfus, South Iceland, earthquake. In Paper 4, the proposed method is used to retrieve permanent ground displacements caused by the 6 April 2009 L’Aquila earthquake in Italy. In Chapter 2, the method is used to correct the accelerograms obtained from the earthquakes occurring in June 2000 in South Iceland. Several near-fault accelerograms are corrected to obtain estimates of permanent ground displacements, which are compared with published independent results obtained from GPS measurements. The usefulness of the proposed baseline correction method is illustrated with several examples.
1.2.2.3 Chapter 3

This chapter provides background information related to Paper 1 and Paper 2. Different methods existing in the literature to compute inelastic response spectra and displacement demands of structures are discussed. The limitations of using the so-called structural behaviour factors (or force reduction factors) are illustrated with practical examples, and the rationale behind the models applied in Paper 1 and Paper 2 is discussed.

1.2.2.4 Chapter 4

This chapter is related to strong ground motions in the near-fault area and their effects on structures; it provides information supplementing Paper 5. While the details of the proposed characterization of near-fault ground motions and their elastic as well as inelastic response spectra are provided in Paper 5, Chapter 4 reviews several related studies found in the literature.

1.2.2.5 Chapter 5

Chapter 5 summarizes the results of this study and mentions areas in which the methods applied in this study can be extended.

The relationships between the papers and the chapters contained in this dissertation are summarized in the following table.

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2 Strong-motion accelerograms: Record processing issues

2.1 Introduction

Strong-motion accelerographs are the instruments deployed to record acceleration of the ground induced by an earthquake. The time evolution of ground acceleration recorded by them is called an accelerogram. Accelerograms are valuable sources of information for seismologists as well as earthquake engineers. In earthquake engineering, information embedded in accelerograms is used to assess seismic demand—the potential of ground shaking to damage engineering structures.

Apart from ground-motion signal, accelerograms also contain some noise. Signal is the portion of an accelerogram corresponding to the actual motion of the ground, whereas other extraneous motions of different origin that also get recorded are called noise. It is important to separate noise and the actual signal from an accelerogram so that reliable information about ground shaking can be derived. Due to the random nature of noise and the lack of information related to the processes generating them, it is often very difficult to precisely distinguish noise from the actual signal. Nevertheless, most of the noise can be identified by carefully analyzing accelerograms in the time and the frequency domains. Any noise identified in a record (accelerogram) should be removed before the record is used in engineering or seismological applications. Various techniques are available to efficiently identify and remove noise from an accelerogram, thereby obtaining a clean signal insofar as possible. Such techniques are often referred to as record processing or strong-motion data processing in the literature.
In this chapter, a brief overview of different types of noise commonly encountered in strong-motion accelerograms is presented. The nature and the origin of these noises is briefly discussed, and some processing techniques to deal with them are mentioned. Following this, a detailed treatment of a special type of noise—known as baseline errors—is presented. A novel baseline adjustment technique designed during this doctoral study is discussed in Rupakhet et al. (2010a). In this chapter, some examples of the usefulness of the proposed technique are presented.

2.2 Effects of recording instruments

Most of the strong-motion instruments installed in the past—and still in use today—are of the analog type. They are built with optical and mechanical devices, integrated in an instrument, to produce traces of ground acceleration on light-sensitive film (or paper). Due to the non-reusable nature of the media on which analog instruments record, they are set to start recording only when a threshold acceleration is exceeded. This results in the loss of pre-event information, which could provide useful clues about any ambient noise present at the time of recording. The transducers used in analog instruments have a limited dynamic range, which is generally limited to 25 Hz. This means that any information about the signal at frequencies higher than 25 Hz is not reliably recorded. Analog instruments also have the disadvantage that the traces recorded by them need to be digitized before they can be used in numerical computations. Digitization is generally time-consuming and error-prone. Errors in digitization can lead to high-frequency, as well as low-frequency, noise in the final result. During digitization, a random error can occur due to the imperfect marking of the exact mid-point of the film (or paper) trace. Such errors result in high-frequency noise in the digitized record. If a long analog trace is digitized in parts and later joined together, then errors in aligning the parts cause the baselines of these different sections to be different. These effects produce long-period noises in the form of baseline variations. Trifunac et al. (1973) provide a detailed analysis of the types and sources of noise commonly encountered in digitized accelerograms.

Digital instruments are equipped with built-in analog-to-digital converters. This eliminates the introduction of noise due to errors during digitization. Digital instruments also possess a high dynamic range (50–100 Hz) and provide more reliable information at frequencies higher than their analog counterparts. In addition, they provide higher resolution in time. Since digital memory devices are reusable, these instruments can be set to record continuously, and pre-event data can often be retrieved. In spite of these improvements, digital accelerograms are not free from noise. Digital records, like the digitized analog ones, have unknown baselines in many cases. Unlike the digitized analog records—where variations in baseline occur due to imperfect splicing
of parts, warping of the recording media, or lateral movement of the same during recording—baseline variations in digital records can occur due to instrument operation or the process of analog-to-digital conversion in the instrument itself. The pre- and the post-event memory of digital records can serve to identify a noise model; however, any noise generated by the signal itself is difficult to model.

The nature, the source, and the amount of noise present in a strong-motion record depend on the characteristics of the instruments recording them. Digital instruments provide a higher signal-to-noise ratio than analog instruments. Nevertheless, most strong-motion records, whether digitally recorded or digitized from an analog trace, are contaminated by various noises, and proper processing is required. In the following, commonly used processing techniques are discussed with special emphasis on adjustments for baseline errors.

### 2.3 Easily identifiable noise

Some noise can be identified by careful examination of an accelerogram. Spurious spikes are an example of such noise (Boore and Bommer, 2005). Spurious spikes have been identified in the accelerogram recorded at the Selfoss Hospital during the 29 May 2008 Ólfus earthquake in South Iceland. Small, but unusual, spikes are present in all three components of the ground acceleration recorded on a digital cassette by a Terra technology DCA 333 instrument. The vertical component of the acceleration is plotted in Fig. 2.1 for illustration. The pre-event mean has been subtracted from the whole record in the accelerogram shown in the figure.

The unusual spikes, marked by red circles in Fig. 2.1, have an amplitude of 12.75 cm/s² and are almost uniformly spaced at 1 s intervals. It is easier to identify the spikes in the pre- and the post-event part of the record, where the signal is relatively weak. When the signal is much larger than the spikes, they are difficult to locate. The first spike, occurring at 0.47 s, is followed by two more at 1.46 s and 2.46 s in the pre-event portion. Other spikes, identified by careful examination of the record, are indicated with red circles in the figure. When the amplitude of the spikes is small compared with the main signal—as is the case here—they can usually be ignored. In some cases, as reported by Boore and Bommer (2005), these spikes can be larger than the signal itself. In such cases, it is essential to identify and remove them from the record. They can be removed by locating their positions and replacing the spike ordinate by the average value of ordinates immediately before and after the spike location. The accelerogram of Fig. 2.1, after removing the spurious spikes, is shown in Fig. 2.2.
Spurious spikes are broad-band in nature and can impart long-period noise to an accelerogram, apart from high-frequency contamination. When an accelerogram containing spurious spikes is integrated to obtain a displacement waveform, the effects of long-period noise become visible. In Fig. 2.3, a comparison between the displacements obtained from the spiky and the spike-free accelerograms of Fig. 2.1 and Fig. 2.2, respectively, is presented. The difference between the two displacement waveforms is due to the long-period noise imparted by the spikes. It is noted here that none of these displacements are physical-looking because they display a trend towards the end. From physical considerations, displacement at the end of a record should be equal to zero if there is no static offset of the ground. In case static offsets occur, the displacement waveform should remain constant at a certain offset towards the end of the record. The trends in displacement waveforms plotted in Fig. 2.3 are caused by variations in the baseline of the accelerogram. Adjustment techniques to deal with these effects are discussed later in Section 2.7.
2.4 Instrument response

Accelerographs make use of built-in pendulums to record ground acceleration and work on the assumption that the pendulum displacement is proportional to ground acceleration. This assumption is valid only if the natural frequency of the pendulum, also known as transducer frequency, is much higher than the frequency of ground motion being recorded. Most analog instruments have their transducer frequencies below or close to 25 Hz. This implies that the phase, as well as the amplitude, of any ground-motion component having a frequency close to or greater than the transducer frequency is distorted (Trifunac et al., 1973). This problem is less severe in digital instruments because they have maximally flat instrument response up to ∼100 Hz. If reliable information at frequencies close to or greater than the transducer frequency is required, appropriate corrections for instrument response should be performed. Significant ground-motion amplitudes at high frequencies can be expected at stiff sites located close to a strong earthquake. Soft sites filter out the high-frequency components. In addition, high-frequency motion attenuates rapidly with distance. Instrument correction is therefore generally not necessary for accelerograms recorded at soft sites far from the source. Corrections for transducer response can be performed in the frequency domain (see Converse and Brady, 1992; Shyam Sunder and Connor, 1982). Instrument correction tends to amplify high-frequency noise. If high-frequency noise from digitization is present, instrument correction will further amplify it. Therefore, instrument correction should be used carefully and be avoided unless it is essential.
2.5 High-frequency noise

High-frequency ambient noise is present in digital as well as analog accelerograms operating in noisy environments. In addition, random errors during digitization procedure—such as imperfect marking of the exact midpoint on the trace with the digitizer cross-hair, discretization errors, and truncation errors—manifest themselves as high-frequency noise in the final result. In case of digital instruments, high-frequency noise can be caused by low resolutions of their analog-to-digital converters. 10- or 12-bit converters provide noisier results than 24-bit converters. High-frequency noise is significant in the records where the high-frequency part of the signal is weak, due to the small size of the earthquake, distance attenuation, or filtering effects of soft soils. However, high-frequency noise rarely poses a problem in records with large amplitudes, which are generally caused by large earthquakes and at stations close to the source. If high-frequency noise is identified in the record, high-cut (or low-pass) filters can be used in either the time or the frequency domain. The corner frequency of the filter should be chosen below the Nyquist frequency—the highest frequency at which ground motion is correctly recorded—which is equal to half the sampling frequency. When pre-event memory is available, a noise model can be constructed, and the filter corner frequency should be selected so that any part of the record with signal-to-noise ratio less than a certain threshold is filtered out.

2.6 Low-frequency noise

Low-frequency noise is most commonly encountered in digitized analog records. Trifunac et al. (1973) present a very detailed description of the source and nature of long-period noise in analog records. In accelerograms recorded by digital instruments, low-frequency noise is mainly caused by baseline variations. Baseline variations are commonly associated with the hysteresis occurring in the transducers during strong shaking (Iwan et al., 1985). Tilts, occurring at the base of the instrument, can also introduce low-frequency artifacts in the accelerogram. Boore (2001) attributed ground tilts as a source of baseline variation at the TUC129 station recording the 1999 Chi-Chi, Taiwan, earthquake. Graizer (2006) provides a detailed description of tilts in strong-motion records, and describes methods to handle them. Apart from tilts, problems associated with analog-to-digital converters can also induce baseline shifts in accelerograms. A commonly used solution for removing low-frequency noise from an accelerogram is the low-cut (high-pass) filter. See Boore and Bommer (2005) for a summary of different types of filters that can be used, and how their parameters—such as the cut-off frequency and the roll-off—can be selected.
2.7 Baseline adjustment methods

Baseline variations are the unknown distortions and/or shifts of the zero-level of accelerograms. They are near-instantaneous and usually of small amplitude—hardly noticeable in the acceleration trace. Their effects are amplified in the integrated velocity and displacement waveforms, where they appear as linear and quadratic trends, respectively. Although low-cut filters can effectively remove or hide these variations, they also remove permanent displacements—which might occur close to the fault, either due to plastic response of the ground or its elastic deformation due to co-seismic slip on the source. An estimate of permanent deformation is useful in designing structures crossing active faults—for example, the Bolu viaduct in Turkey (see Priestley and Calvi, 2002), which crosses the rupture of the 1999 Düzce earthquake.

If baseline variations result from imperfect splicing of two or more sections of an analog trace, they can be identified in the integrated velocity as linear trends starting at the points where the splices are joined. In these situations, and similar cases of multiple linear trends in the velocity, piece-wise sequential removal of linear trends can be used to remove baseline shifts. For a practical example of these situations, readers are referred to Boore and Bommer (2005).

In many digital records, baseline variations are of random nature, and their amplitude and location on time axis cannot be clearly identified. To deal with baseline variations, Graizer (1979) proposed a scheme of fitting a series of progressively higher-order polynomials to the integrated velocity. In the past researchers have applied other modifications of this scheme (see, for instance, Douglas, 2002). A simplified form of the scheme has also been by fitting a quadratic curve to the velocity (see Boore and Bommer, 2005). The more recent methods of baseline adjustment schemes generally adopt a noise model proposed by Iwan et al. (1985). Some examples of the methods based on this noise model are Akkar and Boore (2009); Boore (2001); Boore et al. (2002); Wang et al. (2003); Wu and Wu (2007); and Rupakhety et al. (2010a).

As a part of this doctoral study, a new method of baseline adjustment was developed. The details of this method are described in Rupakhety et al. (2010a). Unlike other similar methods, the proposed method is less subjective in selecting the parameters of the adjustment scheme. The paper also reviews other available schemes of baseline adjustments, and discusses their shortcomings. The usefulness of the proposed method is demonstrated by comparing its results with those of other methods available in the literature. Several examples of the method, using strong-motion accelerograms from the 29 May 2008 Ölfus earthquake, and the 1999 Chi-Chi, Taiwan, earthquake are discussed in the paper.
2.8 Practical applications

The method developed in (Rupakhety et al., 2010a) has been successfully applied to recover permanent displacements from several records of the 29 May 2008 Ölfus earthquake, and the 1999 Chi-Chi earthquake. In addition, the method has been used in Rupakhety and Sigbjörnsson (2010) to recover permanent displacements at near-fault stations recording the 2009 L’Aquila earthquake in Italy. Below, the method is used to obtain permanent displacements from near-fault strong-motion records obtained from the June 2000 earthquakes in South Iceland.

2.8.1 Baseline shifts in the accelerograms of June 2000 South Iceland earthquakes

A sequence of earthquakes occurred in June 2000 in the southern part of Iceland. The sequence started on 17 June with a $M_w$ 6.5 event occurring just north of Hella. A second major earthquake with $M_w$ 6.4 occurred on 21 June. The epicentre of this event was located approximately 17 km west of the epicentre of the 17 June event (Árnadóttir et al., 2001; Stefánsson, 2006). More details about the earthquakes and their effects can be found in Sigbjörnsson et al. (2007) and Sigbjörnsson and Ólafsson (2004). Several accelerograms of the two events were recorded by the Icelandic Strong Motion Network (Sigbjörnsson, 1990). Useful information regarding the near-fault stations that recorded these earthquakes can be found in Halldórsson et al. (2007).

Some of the near-fault accelerograms from these two events display signs of permanent ground displacement. Due to baseline variations, and possibly other long-period noise in the raw data, the integrated displacement waveforms display quadratic trends. By applying the baseline adjustment procedure described in Rupakhety et al. (2010a), these trends are removed, and an estimate of permanent ground displacement is obtained. The details of the records from which permanent ground displacements were retrieved are listed in Table 1.1. For each record, permanent displacements estimated in the N–S, E–W, and the vertical directions are also shown in Table 1.1. Positive values indicate northwards, eastwards and upwards displacements.

The first six rows of Table 1.1 correspond to the 17 June event, while the other seven rows correspond to the 21 June event. Stations recording both of these events are repeated. Raw data of three-component accelerograms were obtained from the ISESD (Internet Site for European Strong-Motion Data) online database (Ambraseys et al., 2004). As an initial treatment, a zero-order correction (see Boore and Bommer, 2005) is performed, which involves removing the pre-event mean—or the mean of the whole
record when pre-event data are not available—from each of the accelerograms. The accelerograms obtained after applying zero-order correction are hereafter referred to as ‘raw acceleration’. Integration of raw acceleration once and twice results in raw velocity and raw displacement, respectively.

Table 2.1: Near-fault stations of June 2000 South Iceland earthquake.

<table>
<thead>
<tr>
<th>Station</th>
<th>SID</th>
<th>WID</th>
<th>° N</th>
<th>° W</th>
<th>N–S</th>
<th>E–W</th>
<th>U–D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flagbarnarholt</td>
<td>106</td>
<td>4674</td>
<td>63.991</td>
<td>20.264</td>
<td>−23.28</td>
<td>19.50</td>
<td>3.10</td>
</tr>
<tr>
<td>Kaldarholt</td>
<td>103</td>
<td>6263</td>
<td>64.004</td>
<td>20.474</td>
<td>28.11</td>
<td>−25.06</td>
<td>−6.13</td>
</tr>
<tr>
<td>Minni–Nupur</td>
<td>108</td>
<td>4675</td>
<td>64.050</td>
<td>20.160</td>
<td>−14.74</td>
<td>−0.67</td>
<td>1.17</td>
</tr>
<tr>
<td>Hella</td>
<td>105</td>
<td>4673</td>
<td>63.840</td>
<td>20.390</td>
<td>17.46</td>
<td>−3.05</td>
<td>−2.78</td>
</tr>
<tr>
<td>Thorsarbru</td>
<td>502</td>
<td>6277</td>
<td>63.931</td>
<td>20.649</td>
<td>−8.23</td>
<td>−6.03</td>
<td>−0.61</td>
</tr>
<tr>
<td>Solheimar</td>
<td>109</td>
<td>4676</td>
<td>64.065</td>
<td>20.642</td>
<td>4.64</td>
<td>−9.46</td>
<td>−0.92</td>
</tr>
<tr>
<td>Thorsarbru</td>
<td>502</td>
<td>6349</td>
<td>63.931</td>
<td>20.649</td>
<td>−16.00</td>
<td>46.91</td>
<td>−13.13</td>
</tr>
<tr>
<td>Thorsartun</td>
<td>107</td>
<td>6332</td>
<td>63.928</td>
<td>20.648</td>
<td>−6.57</td>
<td>48.41</td>
<td>−6.63</td>
</tr>
<tr>
<td>Solheimar</td>
<td>109</td>
<td>6334</td>
<td>64.065</td>
<td>20.642</td>
<td>−24.59</td>
<td>−22.60</td>
<td>5.65</td>
</tr>
<tr>
<td>Kaldarholt</td>
<td>103</td>
<td>6328</td>
<td>64.004</td>
<td>20.474</td>
<td>−5.71</td>
<td>12.25</td>
<td>5.04</td>
</tr>
<tr>
<td>Selfoss Hospital</td>
<td>101</td>
<td>6326</td>
<td>63.940</td>
<td>20.987</td>
<td>6.76</td>
<td>6.76</td>
<td>−1.26</td>
</tr>
<tr>
<td>Selfoss Town Hall</td>
<td>112</td>
<td>6335</td>
<td>63.937</td>
<td>21.002</td>
<td>4.27</td>
<td>4.27</td>
<td>−0.81</td>
</tr>
<tr>
<td>Hella</td>
<td>105</td>
<td>6330</td>
<td>63.840</td>
<td>20.390</td>
<td>3.75</td>
<td>4.10</td>
<td>4.71</td>
</tr>
</tbody>
</table>

1Station identification number of the Icelandic Strong-motion Network
2ISESD Waveform identification number (Ambraseys et al., 2004)

An example of the nature of baseline shifts in the raw data is presented. In Fig. 2.4, the zero-order-corrected longitudinal component of acceleration at Kaldarholt station from the 17 June event is plotted in the top panel. The middle and the bottom panels of the figure display the corresponding velocity and displacement, respectively. Baseline shifts in the accelerogram cannot be identified by looking at the acceleration trace. However it is clearly seen in the velocity, and the displacement traces. The velocity trace shows a linear trend towards the end of the record. This is physically impossible because, after the earthquake ends, ground velocity should return to zero. Similarly, the displacement trace in Fig. 2.4 shows a quadratic trend and steadily increases even after the strong shaking has ended. This example illustrates how baseline offsets—which are hardly visible in the acceleration trace—affect the velocity and the displacement waveforms obtained from the acceleration trace. To obtain a plausible estimate of velocity and displacement waveforms, it is essential to make adjustments for baseline shifts in the raw data.

The results obtained after applying the baseline correction scheme of Rupakhety
et al. (2010a) to the accelerogram shown in Fig. 2.4 are shown in Fig. 2.5. Raw acceleration is plotted in red, and corrected acceleration is plotted in blue in the top panel. The difference between the raw and the baseline-corrected acceleration is hardly visible. However, the velocity and the displacement waveforms obtained after baseline correction are more plausible than the raw ones. The velocity waveform after correction (middle panel of Fig. 2.5) effectively comes to zero towards the end. The displacement waveform (bottom panel of Fig. 2.5), on the other hand, does not return to zero. This is possibly due to permanent ground displacement at the station caused by co-seismic slip on the fault. Similar corrections were applied to all the acceleration traces of the records listed in Table 2.1.

Figure 2.4: Longitudinal component of Kaldarholt station recording during the 17 June 2000 South Iceland earthquake—acceleration (top), velocity (middle), and displacement (bottom) with zero-order correction.

In all cases, some permanent displacement has been retrieved. The permanent displacements obtained here are only approximate. A true estimate of velocity and displacement cannot be obtained from three translational components of acceleration data alone. True displacement and velocity can only be obtained if 6-components (three translational and three rotational) of acceleration are available (Graizer, 1979). Unknown initial conditions at the beginning of the record also mean that an accurate estimate of displacement and velocity waveforms cannot be obtained by integrating the acceleration data. Nevertheless, a reasonable estimate can be obtained by a carefully performed baseline correction scheme. Comparing of the results of a baseline correction scheme with independent measures of ground displacement can be useful to judge the accuracy of the method. As shown in Rupakhety et al. (2010a) and Rupakhety and Sigbjörnsson (2010), the results of the proposed scheme match well
with the information available from GPS and InSar data.

2.8.2 Permanent displacements

Permanent displacements obtained from the three components of ground motion at several stations recording the June 2000 earthquakes are presented in Table 2.1. First, permanent displacements due to the 17 June event alone are considered (records 1–6 in Table 2.1). These stations are plotted as red triangles on the map shown in Fig. 2.6. The red stars on the map indicate the epicentres of the 17 June and 21 June events. Their locations are taken from Stefánsson (2006). Both of these events follow a right-lateral strike-slip mechanism, as shown by the beach balls in Fig. 2.6. The 17 June event occurred in the vicinity of the Holt fault. The 21 June event occurred west of the 17 June event and near the Hestvatn fault. Approximate locations of these faults are shown with black lines. The faults shown in Fig. 2.6 reflect the locations of aftershocks on several mapped faults and do not correspond to a single clear fault. Displacement vectors at the stations (see Table 2.1 for station names) are shown with arrows originating from them. Permanent displacements caused only by the 17 June event are plotted in the figure. At each station, N–S displacement is shown by a red arrow, and E–W displacement is shown by a green one. The resultant displacement is shown with the black arrows. Largest displacements are observed at 106 and 103, which are also the closet ones to the fault. The sense of displacements at these
stations is opposite, with 106 moving south-east and 103 moving north-west. This is consistent with the right-lateral strike-slip mechanism of the earthquake. These two stations got displaced from each other by 51 cm in the fault-parallel direction. This displacement gives some indication of the co-seismic slip on the fault plane. Several studies have estimated co-seismic slip on the fault plane for this event; a summary is in Table 1 of Stefánsson et al. (2005). The value of co-seismic slip obtained from a uniform slip model was reported as 1.7 m. Using a distributed slip model, a slip in the range 0–2.6 m was estimated (Stefánsson et al., 2005). Árnadóttir et al. (2001) estimated a slip of 2.0 m, and Pedersen et al. (2003) estimated a slip of 0.3–2.4 m. Station 103 is about 3 km north of the June 17 epicentre. Around this area, the slip on the fault computed by Dubois et al. (2008) is about 80 cm. Considering the attenuation of co-seismic displacement with distance from the fault, our estimate of 51 cm relative displacement across 106 and 103 is in good agreement with the slip model of Dubois et al. (2008).

Figure 2.6: Permanent ground displacements in the near-fault region of the 17 June 2000 South Iceland earthquake. Triangles indicate the strong-motion recording stations; epicentres of the 17 June and the 21 June events are marked with red stars. Green, red and black arrows originating at the stations indicate eastwards, northwards, and resultant displacements, respectively. Vertical displacements are indicated by texts placed next to each station.
Stations 502 and 107 (see Table 2.1) are located very close to each other. Station 502 is located on a bridge pier on the west bank of Thjorsa River. Station 107 is located on a farm called Thorsartun on top of a small hill, on the eastern bank of the river. The western bank of the river is formed on lava rock overlaying soft sediments, and evidence of soil amplification has been reported at this site (Bessason and Kaynia, 2002). The eastern bank, on the other hand, consists of bed rock. The study of Bessason and Kaynia (2002) indicates amplification of ground motion in the western bank relative to the eastern bank for frequencies above 1.25 Hz. Since permanent displacements are related to smaller frequencies, large differences in permanent displacement in these closely located sites is not likely. From Table 2.1, it can be seen that the permanent displacements at these stations due to the 21 June event are comparable. A similar comparison could not be made for the 17 June event as station 107 did not record this event.

Another issue worth investigating here is the possible contamination of the recording at Station 502 being contaminated by structural response of the bridge where the instrument is located. Numerical studies were performed to simulate the behaviour of the structure modelled as a SDOF (Single Degree Of Freedom) system. A thorough study of the response of the bridge has been done by Bessason and Haffidason (2004). Their study reported a transverse natural period of the first mode of vibration of the bridge to be 0.83 s from field measurements, and 1.29 s from finite element analysis. Three different SDOF systems with undamped natural periods in the range 0.8–2 s and damping ratio equal to 5% of critical damping were considered. Assuming linear elastic behaviour, these SDOF models are subjected to the transverse component of raw acceleration recorded at Station 502 during the 21 June event, and their total displacement responses were computed. The results are presented in Fig. 2.7. The coloured lines in the figure represent the total displacement response of the bridge modelled as SDOF having different natural periods. The solid black line is the ground displacement computed from raw ground acceleration. Ground displacement obtained from the ground acceleration corrected by the proposed method is shown with the dashed line. It is evident from the figure that the effect of bridge response is higher for more flexible systems with longer natural periods. It can also be said that the effect of bridge response is more pronounced on peak displacement than on final displacement. Permanent displacement is controlled by much smaller frequencies than the natural frequency of the bridge. The close resemblance of displacements obtained at 502 and 107 also indicates that the bridge response, if any, does not play an important role in the computation of permanent displacements.
Figure 2.7: Permanent ground displacement at the Thorsarbru Bridge site during the 21 June 2000 South Iceland earthquake. The transverse direction is considered. The solid and the dashed black lines correspond to the raw and corrected ground displacements, while the coloured lines are the total displacement response of the bridge modelled as SDOF with three different natural periods, as indicated in the legend.

2.8.3 Comparison of the results with GPS measurements

In order to check the validity of the permanent ground displacements computed by the proposed scheme, it is useful to compare the results with estimates from other independent studies. If there were collocated GPS stations with a continuous measurement system, GPS data could be used as a reference for validating the results of the baseline correction method. Unfortunately, there are not any collocated or nearby continuous GPS stations in the area of interest. However, GPS surveys were conducted in the area in 1986, 1989, 1992, 1995, 1999, and 2000. Árnadóttir et al. (2001) used GPS measurements from the 1995, 1999, and 2000 surveys to compute coseismic displacements caused by the June 2000 earthquakes. They made adjustments for inter-seismic displacements between 1995–2000 in order to isolate the displacements caused by the earthquakes in 2000. Because survey data between the 17 June and the 21 June events are not available, their contributions to the total displacements could not be isolated, and the total displacements due to both these events were reported in Árnadóttir et al. (2001). In order to compare the results of this study with their estimated permanent displacements, the displacements computed at stations recording both these events were summed, and total displacements were computed.

In Fig. 2.8, the total displacement vectors computed from baseline-corrected accelerograms are plotted as black arrows. The GPS measurements reported in Árnadóttir et al. (2001) are shown with red arrows. It should be noted that station 107 did not record the 17 June event, and therefore the total displacement at this station shows the contribution of only the 21 June event. Although GPS stations and strong-motion stations are not collocated, GPS results from nearby stations display a pattern similar to the results of this study. Permanent displacement computed at the
Thorsarbru Bridge, denoted as 502 in Fig. 2.8, is in very close agreement with the results obtained from GPS measurements. The displacement computed at Thorsar-tun, marked as 107 in Fig. 2.8, shows some deviation from the one computed at a nearby station 502. However, from Table 2.1, it is clear that the displacements at 502 and 107 due to June 21 event are almost the same. The total displacements at these stations appear different in Fig. 2.8 because the 107 displacement lacks contribution from the 17 June event. If displacements at 107 and 502 due to the 17 June event are assumed to be almost equal, then the total displacements at these stations match each other very closely.

Figure 2.8: Permanent ground displacements in the near-fault region of June 2000 South Iceland earthquakes. Triangles indicate the strong-motion recording stations; epicentres of the 17 June and 21 June events are marked with red stars. The black and the red arrows indicate displacement vectors computed in this study and that of Árnadóttir et al. (2001), respectively.
Displacement time histories at stations close to the fault can provide information regarding the evolution of displacement on the fault. Careful study of displacement time histories at stations lying on the opposite sides of the fault can indicate the faulting mechanism. An example of such application is presented in this section. The 17 June event was recorded by strong-motion stations at Flagbjarnarholt and Kaldarholt among others. Both of these stations are within 5 km of the fault (see Fig. 2.6). Flagbjarnarholt lies to the east, and Kaldarholt lies to the west of the Holt fault, where the 17 June event occurred.

In Fig. 2.9, the particle motion at these stations is projected on the horizontal plane. The black and the red traces represent motion at Flagbjarnarholt and Kaldarholt, respectively. Fault-parallel and fault-normal displacements are plotted in the vertical and the horizontal axes, respectively. The displacement evolution evident from this figure displays remarkable symmetry. The overall motion at these two stations clearly display a right-lateral mechanism. Initially, Flagbjarnarholt moves south-west, and Kaldarholt moves north-east. Then the fault-normal movement changes, with Flagbjarnarholt going east and Kaldarholt going west. This sense of south-east movement at Flagbjarnarholt and north-west movement at Kaldarholt remains steady for a major part of ground shaking. Towards the end, there are small oscillations at both of the stations in the NW–SE direction. Finally, Kaldarholt is permanently displaced to the north-west of its initial position, and Flagbjarnarholt is permanently displaced to the south-east of its initial position.

Similarly, the particle motion projected on a vertical plane parallel to the fault is shown in Fig. 2.10. Upwards and northwards displacements are plotted as positive. The opposite sense and symmetry of motion at these stations is clearly visible in the figure. The motion at Flagbjarnarholt shows a simple pattern. Initially it moves upwards and then the motion is mainly southwards, followed by a downwards motion almost returning to its original level. At the end, a small northwards reconciliation is observed, during which it also moves upwards, thus retaining a permanent upwards displacement. Kaldarholt shows a similar pattern in the opposite direction, but with a more oscillatory character. At the end of ground shaking, Kaldarholt is permanently displaced below its original position.
2.9 Effects on response spectra

For engineering applications, it is important to ensure that any correction scheme being applied does not result in artificial and undesirable effects in the response spectra of an accelerogram. Different schemes of baseline adjustments result in different displacement waveforms, and which one is the most reliable is often difficult to judge (see Rupakheti et al., 2010a). This creates confusion as to which scheme should be used if the corrected accelerograms are to be used in engineering applications. It is therefore useful to understand the sensitivity of response spectra to different schemes of baseline adjustments. Baseline adjustments affect the long-period components of ground motion. Therefore, acceleration and velocity response spectra of an accelerogram, whose baseline is adjusted by different schemes, show very small to no differences at all, provided that all these schemes result in stable displacements towards the end of the record. Displacement response spectra at long periods might be slightly sensitive to the method of baseline adjustment used to correct the origi-
inal accelerogram. Boore (2001) observed that the elastic displacement spectra of accelerograms obtained by different baseline correction procedures started to display small differences at natural periods as large as 20 s. Rupakhety et al. (2010a) observed similar results for the accelerograms obtained from the 29 May 2008 Ölfus earthquake in South Iceland. Akkar and Boore (2009) argued that inelastic response spectra could be more sensitive than elastic response spectra to different baseline adjustment schemes due to higher effective periods of inelastic systems.

The nature of the accelerogram being processed is the most important factor determining whether or not response spectra will be dependent on the baseline adjustment schemes. In most cases, the displacement waveform looks like a simple ramp with small oscillations super-imposed on it. In some cases, a ramp-like displacement also contains large oscillatory pulses super-imposed on it. In the first case, the peak ground displacement is almost equal to the permanent displacement. In the second case, the peak ground displacement is controlled by the oscillatory pulse and is larger than the final displacement. Most of the records obtained from Icelandic earthquakes are of the second type. For these types of ground motions, peak ground displacements obtained by different baseline adjustment schemes are almost the same (see Rupakhety et al., 2010a). Since the peak relative displacement at long periods is controlled by the peak ground displacement, the response spectra of these types of records are not sensitive to the method of baseline correction being used. On the other hand, if peak ground displacement is controlled by the permanent displacement, and if different schemes of baseline adjustment result in different estimates of permanent displacement, then the response spectra of corrected accelerograms can be different at long periods. The accelerograms from the 1999 Chi-Chi earthquake used in Boore (2001) are of this type, and their spectra are sensitive to baseline adjustment methods. In most cases, the effect of a baseline adjustment procedure on response spectra is visible only at very long periods. Therefore, it can be said that a carefully designed procedure does not produce any side effects in terms of response spectra in the frequency band of common engineering interest.
Inelastic response modelling

3.1 The importance of inelastic design

The current philosophy of seismic design, which is implemented in regulatory codes, is based on forces. Force-based seismic design uses an approach that is similar to the primary load design, which is used for gravity loads. A force-based design procedure starts with an estimate of the forces applied to a numerical model of a structure. Unlike gravity loads, seismic excitations are dynamic in nature, and their equivalent static representation is therefore considered as design action. Then, structural members are dimensioned to carry these actions without exceeding their plastic capacity. Finally, a check on structural displacements is made to satisfy code requirements.

Seismic design codes, such as Eurocode 8 (CEN, 2004), specify seismic action on structures in terms of elastic design spectra. Elastic design spectra represent peak actions on structures responding in the elastic zone and possessing a certain level of viscous damping, which is usually taken as 5% of its critical value. The strong-motion intensity level associated with the design spectra is severe—commonly characterized by a mean return period of 475 years.

In areas of high seismic activity, the code-specified levels of design seismic actions can be very severe. For a structure to remain elastic under such severe actions, very strong structural members are required, which result in bulky, uneconomical design. To overcome this, seismic codes permit a designer to reduce the elastic design forces. This reduction is interpreted in terms of the capacity of structural members to dissipate seismic energy by means of inelastic deformations. Reduction in design forces due to hysteretic energy dissipation in structural members is quantified in terms of a strength reduction factor. This factor is used to scale down the elastic design force,
thereby permitting structures to undergo inelastic deformations. Strength reduction, in some cases, is related to structural over-strength. Over-strength is said to be effective when the actual plastic capacity of structural members is higher than their values considered in design. A good overview of structural over-strength is provided in Bertero et al. (1991). Besides hysteresis and over-strength, damping mechanisms also contribute to strength reduction. The combined effect of these three factors—hysteretic energy dissipation, energy dissipation due to damping, and structural over-strength—is quantified in terms of the so-called structural behaviour factor \((q)\) in Eurocode 8. In American codes, the same quantity is known as strength reduction factor \((R)\). If only hysteretic energy dissipation is considered, the symbol \(R_\mu\) is used, which implies that strength reduction is related to inelastic displacement ductility \((\mu)\).

Although force reduction factors specified in seismic design codes are intended to account for the combined effects of hysteresis, damping, and structural over-strength, their values are mainly based on the performance of different structural systems during strong past earthquakes. Due to this, a wide-spread concern about the lack of rational criteria for selecting strength reduction factors is prevalent among researchers (see, for example, Bertero, 1986; Fischinger and Fajfar, 1990). To improve the reliability of earthquake-resistant design of structures, methods for reliably estimating strength reduction factors are required. An overview of the different methods available in the literature is presented in the following section.

### 3.2 Force reduction factors

Before describing different methods for estimating force reduction factors, some definitions related to SDOF systems are presented. The level of inelastic deformation of a system caused by a given ground motion is quantified by its displacement ductility ratio \((\mu)\):

\[
\mu = \frac{\max |u(t)|}{u_y}
\]

(3.1)

where \(u_y\) is the yield displacement, and \(u(t)\) is the relative displacement expressed as a function of time \((t)\). The undamped natural period of the system is denoted by \(T_n\) and is given by:

\[
T_n = 2\pi \sqrt{\frac{m}{k}} = 2\pi \sqrt{\frac{mu_y}{F_y}}
\]

(3.2)

where \(m, k,\) and \(F_y\) are the mass, the initial stiffness, and the yield strength of the system, respectively. The maximum force developed in a linearly elastic system is denoted by \(F_e\). The strength reduction factor \((R_\mu)\) is the ratio of \(F_e\) to the yield strength required to limit the displacement ductility to a pre-determined value, which
is known as target ductility ($\mu_t$).

\[
R_\mu = \frac{F_e}{F_y(\mu \leq \mu_t)} = \frac{F_y (\mu = 1)}{F_y (\mu \leq \mu_t)} 
\] (3.3)

### 3.2.1 Newmark and Hall 1973

The first systematic study of inelastic behaviour of SDOF systems subjected to earthquake ground motion was performed by Veletsos and Newmark (Veletsos and Newmark, 1960). Five years later, Veletsos et al. (1965) put forward a method to construct inelastic response spectra of SDOF systems that are subjected to earthquake ground motion. Based on these pioneering works—and the elastic as well as inelastic response spectra of the N–S component of the 1940 El Centro earthquake record—Newmark and Hall (1973) proposed a set of equations relating $R_\mu$ to $T_n$ and $\mu$. It was assumed that $R_\mu$ is equal to $\mu$ in the long-period range. This observation is known as the equal displacement rule. At short periods of vibration, the elastic and the inelastic systems experience the same force, and $R_\mu = 1$. At intermediate periods of vibration, they proposed the so-called equal energy rule, which states that the energies associated with the force corresponding to the maximum displacements reached by an elastic and an inelastic systems are equal. For such systems, the force reduction factor is given by the following equation.

\[
R_\mu = \sqrt{2\mu - 1} 
\] (3.4)

### 3.2.2 Lai and Biggs 1980

Based on an analysis of 20 synthetic ground-motion accelerograms, Lai and Biggs (1980) proposed the following equation to compute strength reduction factors:

\[
R_\mu = \alpha + \beta (\log T) 
\] (3.5)

where $\alpha$ and $\beta$ are coefficients depending on $T_n$ and $\mu$.

### 3.2.3 Riddell and Newmark 1979

Riddell and Newmark (1979) were the first to perform a statistical study of inelastic response spectra of recorded accelerograms. They used ten accelerograms that were recorded on rock or alluvium sites to derive force reduction factors for bilinear as
well as stiffness degrading systems. Their equations are similar to the ones proposed by Newmark and Hall (1973). Riddell and Newmark (1979) concluded that strength reduction factors are not greatly influenced by the hysteretic model that is used in the analysis, and elasto-plastic idealization is generally conservative.

### 3.2.4 Elghadamsi and Mohraz 1987

Elghadamsi and Mohraz (1987) proposed a new approach to computing inelastic response spectra. They computed an inelastic response spectrum by interpolating two smoothed response spectra, namely, an elastic response spectrum and an inelastic response spectrum, corresponding to a yield level of 0.05 in. Their study, which was based on 50 accelerograms recorded on alluvium and 26 accelerograms recorded on rock, showed that force reduction factors are not significantly influenced by soil conditions.

### 3.2.5 Hidalgo and Arias 1990

Unlike other researchers, who used piecewise-linear functions to relate $R_\mu$ with $\mu$, Hidalgo and Arias (1990) put forward an equation expressing strength reduction factors as a continuous function of $T_n$:

$$R_{\mu} = 1 + \frac{T_n}{kT_{no} + \frac{T_n}{\mu-1}}$$

(3.6)

where $kT_{no}$ takes different values for different groups of ground motions.

### 3.2.6 Nassar and Krawinkler 1991

Nassar and Krawinkler (1991) were the first to study the effects of epicentral distance and structural properties, such as natural period of vibration, yield level, strain-hardening ratio, and stiffness degradation, on strength reduction factors. Their study showed that epicentral distance and stiffness degradation do not influence strength reduction factors significantly. Their equations for mean strength reduction factors are

$$R_{\mu} = [c(\mu - 1) + 1]^\frac{1}{2}$$

(3.7)

$$c(T_n, \alpha) = \frac{T_{na}}{1 + T_{na}} + \frac{b}{T_n}$$

(3.8)
where $\alpha$ is the post-yield stiffness ratio of a bilinear system, and $a$ and $b$ are coefficients depending on $\alpha$.

### 3.2.7 Vidic, Fajfar, and Fischinger 1994

Vidic et al. (1994) proposed simple equations to estimate mean strength reduction factors that were derived from the elastic and inelastic response spectra of 20 accelerograms. They idealized the variation of $R_\mu$ with $T_n$ as two straight lines. $R_\mu$ is equal to one at zero period, and increases linearly with $T_n$ in the short-period range. At the end of the increasing branch, $R_\mu$ is fixed to a constant value. Mathematically, they are expressed as:

$$
R_\mu = \begin{cases} 
  c_1 (\mu - 1)^{c_R} \frac{T_n}{T_{n0}} + 1 & \text{when } T_n < T_{n0}, \\
  c_1 (\mu - 1)^{c_R} + 1 & \text{otherwise}
\end{cases}
$$  \quad (3.9)

where $T_{n0}$ is the period where the increasing branch and the constant branch of the $R_\mu - T_n$ relation intersect. $T_{n0}$ is related to the predominant period of ground motion ($T_g$) and is given by the following equation.

$$
T_{n0} = c_2 \mu^{c_T} T_g
$$  \quad (3.10)

In equations (3.9) and (3.10), $c_1$, $c_2$, $c_R$, and $c_T$ are the model coefficients that depend on viscous damping and the hysteretic characteristics of a system. The structural behaviour factors recommended in Eurocode 8 are based on this model.

### 3.2.8 Miranda and Bertero 1990

(Miranda and Bertero, 1994) presented an excellent review of different equations that are used to compute strength reduction factors. Theirs was also the first study that used a large number of recorded accelerograms—124 in total. They presented mean strength reduction factors for different soil conditions, namely, rock, alluvium, and very soft soil. For 5%-damped elasto-plastic systems the strength reduction factors proposed by them are given by

$$
R_\mu = \frac{\mu - 1}{\Phi} + 1
$$  \quad (3.11)
where $\Phi$ depends on $T_n$, $\mu$, and soil condition, and is given by

\[
\Phi = \begin{cases} 
1 + \frac{1}{10 T_n - \mu T_n} - \frac{1}{2 T_n} \exp \left[ -\frac{3}{2} \left( \ln T_n - \frac{3}{5} \right)^2 \right] & \text{for rock sites} \\
1 + \frac{1}{12 T_n - \mu T_n} - \frac{2}{5 T_n} \exp \left[ -2 \left( \ln T_n - \frac{1}{5} \right)^2 \right] & \text{for alluvium} \\
1 + \frac{T_g}{3 T_n} - \frac{3 T_g}{4 T_n} \exp \left[ -3 \left( \ln \frac{T_n}{T_g} - \frac{1}{4} \right)^2 \right] & \text{for soft soils}
\end{cases}
\] (3.12)

where $T_g$ is the predominant period of ground motion, which is defined as the period at which the pseudo-spectral velocity of a 5%-damped elastic oscillator is the maximum.

### 3.2.9 Borzi and Elnashai 2000

(Borzi and Elnashai, 2000) used 364 accelerograms to study the statistical characteristics of strength reduction factors. They presented a tri-linear representation describing the variation of $R_\mu$ with $T_n$. Their equations were shown to display uniform reliability design, meaning that the probability of exceedance of the mean strength reduction factors is the same throughout the range of $T_n$ considered in the study. They also observed that ground motion properties affect the elastic and the inelastic response spectra in a similar manner, and $R_\mu$ depends mainly on structural properties, namely, $\mu$ and $T_n$. In addition, they reported only mild influence of hysteretic models on strength reduction factors.
3.3 50 years of research on strength reduction factors

As is evident from the previous section, starting from the pioneering work of Newmark and Veletsos in 1960, more than 50 years of research has been conducted in an effort to better characterize strength reduction factors. Some general conclusions that can be extracted from the studies mentioned above are listed below.

1. Strength reduction factors depend primarily on $T_n$ and $\mu$. Although the equal-displacement rule and equal-energy rule are generally conservative, they are not strictly valid for individual accelerograms.
2. The effects of ground-motion characteristics on elastic and inelastic spectra are similar, whereas their effects on strength reduction factors are not significant. For example, (Nassar and Krawinkler, 1991), (Miranda and Bertero, 1994), and (Chopra and Chintanapakdee, 2004) all conclude that earthquake magnitude and epicentral distance do not affect force reduction factors.
3. There is no definite evidence for either confirming or denying the dependence of $R_\mu$ on soil conditions. Whereas Miranda and Bertero (1994) found dependence of $R_\mu$ on site conditions, Elghadamsi and Mohraz (1987), and Borzi and Elnashai (2000) did not observe such dependence.
4. The type of hysteretic relationship does not have greatly affect strength reduction factors, as concluded by (Nassar and Krawinkler, 1991), (Borzi and Elnashai, 2000), and (Chakraborti and Gupta, 2005).

These observations indicate that the various models of force reduction factors in the literature are similar, and there are slight modifications in the recommendations of Newmark and Hall (1973). Different authors used different sets of ground-motion data, made different assumptions, and proposed different $R_\mu - \mu - T_n$ equations. Most of these equations are based on the equal-displacement and the equal-energy rules. The different forms of the equations proposed by different authors reflect the differences in the data used in their analyses. Whereas one model might fit a certain data set closely, a different model might be desirable for a different data set. Nevertheless, the mean values of $R_\mu$ predicted by these models are similar.

During this time, when a new model for estimating $R_\mu$ was being proposed almost every other year, its statistical distribution received very little attention. Several efforts were made to develop empirical equations matching the mean strength reduction factors of an ensemble of ground motions. However, the variations in $R_\mu$ of individual accelerograms from the mean $R_\mu$ of an ensemble of accelerograms have often been neglected. Apart from modifying the models that were used to estimate mean strength
reduction factors, their reliability was not studied in depth. For example, for a given 
ensemble of accelerograms, what are the confidence intervals of the mean strength 
reduction factors being reported? What kind of scatter is present around the mean, 
and what is its statistical nature? The lack of concern for these questions led to a 
uni-directional research activity—designing a better model for mean prediction. No 
serious attempts were made to study the uncertainties associated with these models, 
and how they would propagate to inelastic spectral ordinates that are obtained by 
reducing elastic spectral ordinates.

3.4 Disadvantage of the two-step procedure

The two-step procedure in computing inelastic spectra, in this work, is meant to 
represent the conventional method of creating inelastic spectra. This is the method 
widely used today and requires the use of mean strength reduction factors. In the 
first step of this procedure, elastic response spectra are computed. This can be 
accomplished in the following ways.

- A pre-defined spectral shape is scaled with a design peak ground acceleration 
  (PGA) to obtain elastic response spectra.
- An attenuation equation, also known as Ground-Motion Prediction Equation 
  (GMPE), is used to compute elastic response spectra. An example of such an 
  equation is in (Ambraseys et al., 2005). Douglas (2003) provides a comprehen-
  sive review of several GMPEs.
- Mean elastic spectrum of an ensemble of ground motion accelerograms is com-
  puted.

In the following, it is assumed that mean elastic spectral ordinates are computed from 
a GMPE. GMPEs are calibrated by performing regression analysis of elastic spectral 
accelerations that are computed from time history analyses of recorded accelerograms. 
These computed values, which are used in regression analysis, are called the observed 
values of elastic spectral acceleration ($S_E$). The model predictions, on the other hand, 
are called the predicted values ($\hat{S}_E$). The difference between a predicted value and 
an observed value is called the residual or error term. If large samples of data are 
used in regression, the residuals can be modelled as a normally distributed random 
variable having a mean value equal to zero and a standard deviation equal to $\sigma_{\text{log } S_E}$.

In the second step of the two-step procedure, the predicted mean of an elastic spec-
tral ordinate is divided by $R_\mu$ to obtain an inelastic spectral ordinate. $R_\mu$ can be
3.5 Bias in inelastic response spectra

computed from equations like those presented in Section 3.2. Or, they can be estimated by performing elastic and inelastic time history analyses of an ensemble of ground-motion records. In either case, the mean value of $R_\mu$ is used, assuming it to be ‘deterministic’. However, it has been observed that $R_\mu$ computed for an individual accelerogram can differ significantly from its mean value that is computed from an ensemble of accelerograms. Ignoring this variability might result in biased estimates of inelastic spectral ordinates as discussed in the following section.

3.5 Bias in inelastic response spectra

To illustrate the bias introduced in inelastic response spectra by the two-step procedure discussed in the previous section, an example is presented here. For this purpose, 93 ground motion records obtained from shallow strike-slip earthquakes in continental Europe and the Middle East are considered. Further description of the data can be found in Rupakheti and Sigbjörnsson (2009b). For each record, spectral accelerations for elastic as well as constant-ductility systems are computed at several values of $T_n$; the damping ratio is assumed to be equal to 5% of the critical level. Out of the two horizontal components of each of these 93 records, the component responsible for larger spectral acceleration is selected. Spectral acceleration of a linear elastic system is denoted by $S_E$, and that of an elasto-plastic system is denoted by $S_{IE}$. In the following, the two steps as discussed above, are illustrated.

3.5.1 GMPE for linear-elastic spectral acceleration

First, the elastic spectral ordinates are modeled by a prediction equation of the form

$$\log S_E = b_1 + b_2 M_w + b_3 \log \sqrt{R^2 + b_4^2} + \epsilon \quad (3.13)$$

where $\log$ represents base-10 logarithm, $M_w$ is the moment magnitude, $R$ is the distance from source to site, $b_i$ ($i = 1 \ldots 4$) are the regression coefficients, and $\epsilon$ represents the residual. Residuals are defined as the differences between model predictions and observed values. A hat (‘$\hat{\cdot}$’) over a term indicates model prediction. For example, $\log \hat{S}_E$ represents the value of the logarithm of spectral acceleration predicted by the model, while $\log S_E$ is that of spectral acceleration computed from time history analysis. The error term, $\epsilon$, is assumed to be a normally distributed random variable with zero mean and standard deviation $\sigma_{\log S_E}$. For simplicity, $b_4$ is set equal to 4.5, and this makes the model linear. Combining all of the 93 ground motions, the model can
be written in the form of a matrix equation:

$$\log S_E = Xb + \epsilon$$  \hspace{1cm} (3.14)

where \( S_E \) is a vector of size \( n \)—the number of observations available, which is 93 in this example. Similarly \( X \) is a matrix that has \( n \) rows, each row corresponding to one of the records. The columns of each row are 1, \( M_w \), and \( \log \sqrt{R^2 + 4.5^2} \). The vector of model coefficients is denoted as \( b \), whose entries are \( b_1 \), \( b_2 \), and \( b_3 \). The model coefficients estimated from regression are denoted by \( \hat{b} \) and the predicted spectral accelerations are given by \( X\hat{b} \). The estimated values of errors are given by \( \log S_E - X\hat{b} \). The model is evaluated at different values of \( T_n \). An example of the results is shown in Fig. 3.1. It can be seen that the individual observations and the model predictions are different, whereas their mean values are the same. The differences between the two give an estimate of the error term, which has zero mean. The right panel of Fig. 3.1 shows that the errors are approximately normally distributed. This example shows the results at \( T_n = 0.2 \) s. To create an elastic response spectrum, this process has to be repeated at several values of \( T_n \).

![Figure 3.1](image.png)

**Figure 3.1**: Left: observed and predicted values of spectral acceleration at \( T_n = 0.2 \) s for the larger horizontal components of 93 ground-motion records. Right: Distribution of regression residuals compared with a normal distribution.

### 3.5.2 Force reduction factor

To estimate an inelastic spectral ordinate from its elastic counterpart, \( R_\mu \) is commonly used. It is usually obtained from empirical equations like the ones mentioned in Section 3.2. A more reliable estimate might be obtained by performing inelastic time history analyses of the ground-motion records that are being considered. The latter approach is followed here to compute \( R_\mu \) for a target ductility value of 4 and \( T_n = 0.2 \) s. \( R_\mu \) is computed for each record by dividing the elastic spectral
3.5. Bias in inelastic response spectra

acceleration (larger of the two horizontal components) by the inelastic one (larger of the two horizontal components). The computed values of $R_\mu$ are shown in Fig. 3.2. A circle in the figure represents the $R_\mu$ of an individual record, while the horizontal line represents the mean value. It is obvious that there are considerable variations of $R_\mu$ from the mean value; which, in this case, is 3.1. It should be noted that force reduction factors given by empirical equations, like the ones mentioned in Section 3.2, can be different from the value computed here. For example, its value computed by using Eq. 3.11 of Section 3.2.8, proposed by Miranda and Bertero (1994), is equal to 2.6.

![Figure 3.2: Force reduction factors computed for the larger horizontal component of 93 ground-motion records to limit the displacement ductility at 4, and for a 5%-damped elasto-plastic SDOF having $T_n = 0.2$ s.](image)

**3.5.3 Inelastic spectral acceleration**

If the conventional two-step procedure is used, inelastic spectral acceleration is given by:

$$\log S_{IE} = Xb - \log (R_\mu) + \epsilon$$

(3.15)

where $b$ is the vector of model coefficients derived for elastic spectral acceleration, as discussed in Section 3.5.1. Inelastic spectral accelerations computed with this approach are compared with their observed values (obtained from time history analyses) in Fig. 3.3. In the left panel, the observed values ($\log S_{IE}$) are compared with the predicted ones ($\log S_{IE}^\ast$). The comparison shows that the model predictions are close to the observed values; however, there is a subtle difference in the statistical characteristics of the two. This difference is related to their mean values. Whereas mean values of predicted and observed elastic spectral accelerations shown in Fig. 3.1 were the same, the mean of the predicted inelastic spectral accelerations is different from the mean of the observed inelastic spectral accelerations shown in Fig. 3.3. This means that the residuals have a non-zero mean. The distribution of the residuals is shown in the right panel of Fig. 3.3, which seems skewed rather than being symmetric.
This illustrates that $\hat{S}_{IE}$ and $S_{IE}$ do not have the same probability distribution. Their mean values are different. In addition, the residuals of predicted inelastic spectral acceleration, i.e., $\hat{S}_{IE} - S_{IE}$, display different characteristics than those of elastic spectral accelerations presented in the right panel of Fig. 3.1. Therefore, the elastic and inelastic spectral accelerations are not compatible to each other in a probabilistic sense. It is also important to note that the mean reduction factor used in this example was derived from the same ground motion data used to calibrate the elastic model. If empirical equations, such as the ones discussed in Section 3.2, are used to compute strength reduction factors, additional bias is likely to be introduced in the final result.

![Figure 3.3: Left: observed and predicted inelastic spectral accelerations at $T_n = 0.2s$ and target ductility of 4 for maximum horizontal components of 93 ground-motion records. Right: Distribution of residuals compared with a normal distribution.](image)

### 3.6 GMPE for inelastic spectral acceleration

The problems discussed in the preceding section can be resolved by developing GMPEs for inelastic response spectral ordinates. For inelastic systems, GMPE models that are similar to the ones used for elastic systems can be used. However, the parameters of a GMPE model for inelastic response are calibrated by performing regression analysis on the inelastic response of SDOF systems computed by time history analyses. A simple example of such a model can be expressed as:

$$\log S_{IE} = Xb_{IE} + \epsilon_{IE}$$  \hspace{1cm} (3.16)

where $S_{IE}$ represents the vector of spectral ordinates computed from time history analyses of inelastic SDOF systems; $b_{IE}$ is the vector of model coefficients, and $\epsilon_{IE}$
is the vector of residuals. The estimated values of model coefficients and residuals are denoted as $\hat{b}^{\text{IE}}$ and $\hat{\epsilon}^{\text{IE}}$, respectively. Using this approach, it is guaranteed that $\hat{S}^{\text{IE}}$ and $S^{\text{IE}}$ have the same mean, and the error has a mean of zero. If $S^{\text{IE}}$ is log-normally distributed, $\epsilon^{\text{IE}}$ is normally distributed, like in the regression model of elastic response. By following this approach, elastic and inelastic spectral ordinates are consistently defined. In addition, the model can be used in probabilistic seismic hazard assessment (PSHA) computations to create uniform hazard spectra that are applicable to inelastic systems. One potential disadvantage of the model is that it has to be calibrated for different values of displacement ductilities, and model coefficients have to be provided for each case. However, this is not a major problem as digital computers are routinely used in storing and processing these models.

The results obtained by using this approach are shown in Fig. 3.4. In the left panel, the observed values of inelastic spectral acceleration at a period of 0.2 s, and with a target ductility of 4, are compared with the predicted values of the model represented by Eq. 3.16. In the right panel, the distribution of $\epsilon^{\text{IE}}$ is compared with a normal distribution. In this case, both the observed and the predicted values have the same mean, and the residuals have a mean of zero.

![Figure 3.4: Left: observed and predicted spectral acceleration at $T_n = 0.2 \text{s}$ and target ductility of 4, for maximum horizontal components of 93 ground-motion records. Right: Distribution of residuals compared with a normal distribution.](image)

In order to test the normality of the residuals shown in Fig. 3.3 and Fig. 3.4, Jarque-Bera test of the null hypothesis that the residuals come from a normal distribution with an unknown mean and variance was performed. At a 5% significance level, the null hypothesis was found to be invalid for both of the residuals. The null hypothesis for the residuals of Fig. 3.4 was found invalid up to a significance level of 0.48, which is very close to the maximum value of 0.5. On the other hand, the null hypothesis that the residuals of Fig. 3.3 come from a normal distribution is valid for a significance level above 19%. These results show that when mean strength reduction factors
Chapter 3. Inelastic response modelling

computed from the same data that are used in calibrating the regression model for elastic spectral acceleration are used to obtain inelastic spectral acceleration, the residuals may be assumed to be normally distributed. However, the mean of the residuals may not be equal to zero. If force reduction factors are computed from empirical equations like the ones mentioned in Section 3.2, the residuals can have a mean significantly different than zero, and their distribution is possibly skewed.

It is therefore desirable to model and calibrate GMPEs for inelastic response of structures. Doing so eliminates the need to use force reduction factors. Such GMPEs can then be used in PSHA applications to construct uniform hazard spectra for different target ductility levels. This approach will provide a rational basis for specifying design forces applicable to inelastic systems in design codes. An application of this method is presented in Rupakhety and Sigbjörnsson (2009b), which illustrates how such an approach can be implemented. Due to the limited number of data considered in the paper, the results presented there are not intended for immediate use. However, the concept presented in the paper can be used with more extensive data available from different parts of the world. Bozorgnia et al. (2010) have employed this approach by utilizing 3100 horizontal ground-motion records, corresponding to 64 earthquakes. The method put forward in Rupakhety and Sigbjörnsson (2009b), if used with extensive data sets of ground motions recorded in the European region, possibly supplemented by records from other similar regions, will be useful in developing inelastic GMPEs that can be used in hazard analysis in the region. This will facilitate the specification of design seismic actions in Eurocode 8.

3.7 Inelastic displacement demand

This chapter has so far discussed the lateral strength of an inelastic system that is required to maintain a certain target ductility level, which is a basic requirement in preliminary design. Nevertheless, there are situations making displacement demands of inelastic systems of primary interest to an engineer. One such situation is the preliminary design of structures using the displacement-based concept (see, for example, Calvi and Kingsley, 1995; Kowalsky et al., 2006; Priestley et al., 1996). Deformation-controlled design procedures, proposed by Panagiotakos and Fardis (1999), also require inelastic displacement demands in preliminary design of structures. In addition, the evaluation of existing structures' response to earthquake ground motions also requires estimates of inelastic displacement demands.

Most of the structural, and some of the non-structural, damage sustained by engineering structures are related to their lateral displacements caused by earthquake-induced ground motion. In performance-based seismic design, earthquake-induced damage to
3.7. Inelastic displacement demand

3.7.1 Methods based on equivalent linearization

In this approach, an inelastic SDOF system is replaced by an equivalent elastic SDOF system. The equivalent system has less lateral stiffness and a higher damping ratio than the inelastic one. Peak displacement of the equivalent elastic system is then obtained from elastic design spectra. The stiffness and the damping ratio of the equivalent system are derived from the initial elastic stiffness, damping ratio, and maximum displacement ductility of the inelastic system. Different methods are available in the literature for obtaining the properties of the equivalent elastic system. Some of the methods are briefly discussed below.

3.7.1.1 Rosenblueth and Herrera 1964

Rosenblueth and Herrera (1964) were the first to put forward a method for estimating the secant stiffness and equivalent damping of an elastic SDOF system representing an inelastic SDOF system. In their method, based on harmonic excitation, the equivalent period of a substitute elastic SDOF system ($T_E$) is related to the undamped natural period of the inelastic SDOF system ($T_n$) by the equation

$$T_E = T_n \sqrt{\frac{\mu}{1 - \alpha + \alpha \mu}}$$  \hspace{1cm} (3.17)

where $\alpha$ is the post-yield stiffness ratio of the inelastic system. The equivalent damping ratio ($\zeta_E$) of the substitute elastic system is related to the viscous damping ratio...
(\(\zeta\)) of the original inelastic system by the following equation.

\[
\zeta_E = \zeta + \frac{2}{\pi} \left[ \frac{(1 - \alpha)(\mu - 1)}{\mu - \alpha \mu + \alpha \mu^2} \right]
\] (3.18)

### 3.7.1.2 Gülkan and Sozen 1974

Unlike harmonic excitations, the displacement response of an SDOF system to earthquake excitation is less than its peak value for a large portion of its response time. Therefore, an equivalent damping ratio based on harmonic excitation (Eq. 3.18) would be larger than one corresponding to earthquake ground motions. This leads to underestimation of the peak response of the equivalent elastic system. This observation was first made by Gülkan and Sozen (1974), who conducted shake-table tests on reinforced concrete frames, and proposed the following empirical equation for estimating an equivalent damping ratio.

\[
\zeta_E = \zeta + 0.2 \left( 1 - \frac{1}{\sqrt{\mu}} \right)
\] (3.19)

This work led to the concept of substitute structure, which is an extension of equivalent linearization to Multi-Degree-Of-Freedom (MDOF) structures, and was first presented by Shibata and Sozen (1976). This concept formed the backbone of the displacement-based design philosophy and was used exclusively in displacement-based design. More recently, it has been shown that a substitute elastic structure is not strictly required in displacement-based design if inelastic design spectra are available (see, for example, Chopra and Goel, 2001; Xue, 2001).

### 3.7.1.3 Iwan 1980

Based on time history analyses of 12 accelerograms, Iwan (1980) presented the following empirical equations to estimate the equivalent period and equivalent damping ratio.

\[
T_E = 1 + 0.121 (\mu - 1)^{0.939}
\] (3.20)

\[
\zeta_E = \zeta + 0.0587 (\mu - 1)^{0.371}
\] (3.21)

### 3.7.1.4 Kowalsky 1994

Based on secant stiffness and Takeda’s model of hysteresis (Takeda et al., 1970) with an unloading stiffness factor of 0.5, Kowalsky (1994) proposed the following equation.
3.7. Inelastic displacement demand

\[ \zeta_E = \zeta + \frac{1}{\pi} \left( 1 - \frac{1 - \alpha}{\sqrt{\mu}} - \alpha \sqrt{\mu} \right) \]  

(3.22)

3.7.1.5 Limitations of equivalent linearization methods

The methods of equivalent linearization, which are mentioned above, are all based on secant stiffness at maximum displacement, except for the method of Iwan (1980). However, they differ in their equations that estimate equivalent damping ratios. Miranda and Ruiz-García (2002) compared these different methods and concluded that the method of Gülkan and Sozen (1974) is likely to estimate largest peak displacements of the equivalent SDOF. The methods of Kowalsky (1994) and Iwan (1980) were found to give similar results for ductility values below 6, while the method of Rosenblueth and Herrera (1964) was found to estimate the largest equivalent damping, and therefore smallest peak displacements.

In the preliminary design of structures, displacement ductility is known to the designer (based on the ductility capacity of the structure), and the concept of equivalent linearization can be used. However, when existing structures are to be evaluated, displacement ductility is not known \textit{a priori}. Instead, the lateral strength of the system is known, and the peak displacement is to be estimated. This means that equivalent linearization methods, which require displacement ductility, cannot be directly used in such situations. To tackle this problem, an iterative procedure has to be followed (see, for example, ATC, 1996). However, neither the convergence nor a unique solution of the iterative procedure is guaranteed (see Chopra and Goel, 1999, 2000; Miranda and Akkar, 2003).

3.7.2 Displacement modification methods

These methods do not require an equivalent elastic SDOF system. They are based on displacement modification factors, which multiply the peak displacements of elastic SDOF systems to obtain peak displacements of inelastic SDOF systems. Both the systems have the same initial elastic stiffness and damping ratio. The displacement modification factors are a function of the period of vibration of the system and its displacement ductility ratio.
3.7.2.1 Newmark and Hall 1982

Newmark and Hall (1982) proposed that peak inelastic displacement can be derived from peak elastic displacement by using the equation

$$\Delta_{IE} = \left(\frac{\mu}{\hat{R}}\right) \Delta_E$$  \hspace{1cm} (3.23)

where $\hat{R}$ is the mean strength reduction factor estimated from an $R - \mu - T_n$ relationship, which, according to Newmark and Hall (1982) is

$$\hat{R} = \begin{cases} 
1, & T_n < T_a \\
(2\mu - 1)^\beta, & T_a \leq T_n \leq T_b \\
\sqrt{2\mu - 1}, & T_b \leq T_n < T_{c'} \\
\mu \frac{T_n}{T_{c'}}, & T_{c'} \leq T_n < T_c \\
\mu, & T_n \geq T_c
\end{cases}$$  \hspace{1cm} (3.24)

where

$$\beta = \frac{\log(T_n/T_a)}{2 \log(T_b/T_a)}$$  \hspace{1cm} (3.25)

$$T_{c'} = \frac{\sqrt{2\mu - 1}}{\mu} T_c$$  \hspace{1cm} (3.26)

In Equations (3.24)–(3.26), $T_a = 0.03$ s, $T_b = 0.125$ s, and $T_c$ is the period where constant-acceleration and constant-velocity spectral regions intersect.

3.7.2.2 Chopra and co-workers

When ductility demand is not known in advance, the displacement modification method using Equations (3.24)–(3.24) requires an iterative procedure. To avoid iteration, Chopra and his co-workers (Chopra and Chintanapakdee, 2001; Chopra and Goel, 2000) used Eq. 3.23 and Eq. 3.24 to estimate inelastic displacements in terms...
of known force reduction factors, and derived the following equations.

\[
\Delta_{IE} = \begin{cases} 
\infty, & T_n < T_a \\
\frac{R^{1/\beta}}{2\beta} \Delta_E, & T_a \leq T_n \leq T_b \\
\frac{R^2+1}{2R} \Delta_E, & T_b \leq T_n < T_{c'} \\
\frac{T_n}{T_c'} \Delta_E, & T_{c'} \leq T_n < T_c \\
\Delta_E, & T_n \geq T_c 
\end{cases}
\]  

(3.27)

Miranda (2001) argued against the validity of these equations and opined that these equations provide only the first order approximation of inelastic displacement demands.

### 3.7.2.3 Miranda 2001

Miranda (2001) performed a statistical study of inelastic and elastic displacement demands of SDOF systems subjected to ground-motion accelerograms that were recorded on firm sites and proposed the following equation.

\[
\frac{\Delta_{IE}}{\Delta_E} = \left[ 1 + \left( \frac{1}{\mu} - 1 \right) \exp \left( -12T_n\mu^{-0.8} \right) \right]^{-1}
\]

(3.28)

Miranda (2001) also compared the different approximate methods mentioned in the preceding sections. The study concluded that even though the approximate methods used to estimate inelastic displacement yield small mean errors compared with results of time history analyses of an ensemble of ground-motion records, dispersion of the results is significant.

### 3.7.2.4 Ruiz-García and Miranda method 2003

Ruiz-García and Miranda (2003) noted that the constant-ductility inelastic displacement ratio, such as the one given by Eq. 3.28, cannot be used in evaluating existing structures, where their ductility demand is not known, but the strength is known. They also demonstrated that using such equations in an iterative manner underestimates the maximum displacements of systems with known lateral strengths. This is expected because in the constant-ductility approach, the strength of a system changes
from one accelerogram to another, whereas the strength of an existing system remains the same—its ductility demand varies from one accelerogram to another. For systems with known lateral strengths, they presented displacement modification factors in terms of $R$. The equation they proposed is

$$\frac{\Delta_{IE}}{\Delta_E} = 1 + \left[ \frac{1}{a \left( \frac{T_n}{T_s} \right)^b} - \frac{1}{c} \right] (R - 1) \quad (3.29)$$

where $T_s$ is the characteristic period of the site, and $a$, $b$, and $c$ are site-dependent constants given in Ruiz-García and Miranda (2003). Akkar and Miranda (2005) evaluated different approximate methods of estimating inelastic displacement demands. The main conclusions of their study are summarized below.

- At periods longer than 1.0s, the approximate methods (including both equivalent linearization and displacement modification approaches) are associated with small bias, usually smaller than 15%. The method of Kowalsky (1994) overestimates mean deformation demands by 20–30%.
- At short periods, i.e., $T_n < 0.5$ s, equivalent linearization method proposed by Kowalsky (1994) overestimates displacement demands, whereas the method proposed by Iwan (1980) has a tendency to underestimate displacement demands if $R < 3$, and overestimate the same if $R > 4$.
- On average, the method proposed by Ruiz-García and Miranda (2003), which is based on a statistical study of time history analysis results of elastic and inelastic systems, has the smallest bias compared with other methods and is also associated with smaller dispersion.
- The errors produced by any of these approximate methods can be large when $R > 4$.
- Finally the authors of Akkar and Miranda (2005) write:

The results presented in this study suggest that caution should be exercised when employing approximate procedures that estimate maximum elastic displacement demands of systems with known lateral strength from the maximum deformation of linear systems. Users of nonlinear static procedures in which target displacements are computed using equivalent linear methods or displacement modification factors should be aware of the limited accuracy offered by these approximate methods, particularly for weak systems relative to the strength required to remain elastic and/or for systems with periods of vibration shorter than 0.6 s.
3.7 Inelastic displacement demand

3.7.3 Disadvantages of the two-step procedure again

The main conclusion of the research on computing inelastic spectral demand is that the best results are obtained from statistical properties of time-history-analysis results of recorded accelerograms. Even when such an approach is used, which is done in Ruiz-García and Miranda (2003), the predicted values of mean inelastic displacement demands were found to be biased. This is caused by the dispersion of displacement modification factors of individual ground-motion records from the mean of an ensemble of ground-motion records. This was inevitable because, in all these studies, the focus was to obtain a procedure that can modify the peak elastic displacement to estimate the peak inelastic displacement. Therefore, the problem lies in the two-step procedure, wherein the elastic displacement is estimated first and modification factors are applied in the second stage. The uncertainties involved with such a procedure, which were discussed in Section 3.5 in relation to inelastic strength demands, apply also to inelastic displacement demands. Therefore, to have an unbiased estimate of peak inelastic deformations, models that can directly estimate inelastic displacement demands—without relying on elastic displacement demands—are required. Tothong and Cornell (2006) quantified the uncertainties involved with displacement modification factors. Their study showed, as is evident from their Equation 8, that the uncertainties related to displacement modification factors are high, more so for weaker systems than for stronger ones.

3.7.4 Direct estimation of inelastic displacement demands

If the constant-ductility approach is used, inelastic displacement demand can be directly estimated from inelastic spectral acceleration. The methods that were mentioned in Section 3.7.1 and Section 3.7.2 are required only when a proper model that can estimate inelastic spectral acceleration is not available. If models that can estimate inelastic spectral accelerations corresponding to a certain target ductility are available, such as the ones described in Rupakheti and Sigbjörnsson (2009b) or in Bozorgnia et al. (2010), inelastic displacement demands can be readily estimated from such models. For example, in Section 3.6, the prediction equation for $S_{IE}$, which is the inelastic spectral acceleration corresponding to a target ductility of $\mu$, was presented. With this information, inelastic displacement demand for the same level of target ductility can be computed by:

$$
\Delta_{IE} = \mu \Delta_y = \mu \frac{F_y}{k} = \mu \frac{S_{IE}}{mk} = \mu \frac{\tilde{S}_{IE} T_n^2}{4\pi^2}
$$

(3.30)
where $\Delta_y$ is the yield displacement; $F_y$ is the yield force; $m$ is the mass; $k$ is the initial elastic stiffness of the system, and $\hat{S}_{IE}$ is the estimated value of spectral acceleration from a GMPE model. In addition, if the residuals of $\log(S_{IE})$ are approximately normally distributed—with mean value equal to zero and standard deviation equal to $\sigma_{\log S_{IE}}$ (see, Section 3.6 and Rupakhety and Sigbjörnsson (2009b))—the residuals of $\Delta_{IE}$ follow the same distribution. This equivalence exists because all the parameters, barring $S_{IE}$, appearing in Eq. 3.30 are independent of ground-motion characteristics; they are ‘deterministic’. This provides a simple, ready-to-use approach that can be implemented in probabilistic applications; for example, in probabilistic seismic demand assessment.

In other situations, where inelastic displacement demands of systems with known strengths are required, the constant-ductility approach cannot be used. Bozorgnia et al. (2006) put forward GMPEs that can be used to compute inelastic displacement demands directly, i.e., without relying on elastic displacement demands and displacement modification factors. However, their model is valid for constant-ductility systems only, i.e., it might not be conservative for systems with known lateral strengths. For systems whose lateral strengths are known, GMPEs calibrated for peak ductility demands of constant-strength systems are more reliable than those calibrated for peak displacement demands of constant-ductility systems. This idea is implemented in Rupakhety and Sigbjörnsson (2009a), where empirical equations relating ductility demands of 5% damped elasto-plastic SDOF systems to earthquake magnitude, source-to-site-distance, local site conditions, undamped natural period of vibration, and mass-normalized yield strength ($c_y$) are put forward. Once ductility demand is computed from the GMPE, peak inelastic displacement can be computed as

$$\Delta_{IE} = \hat{\mu} \Delta_y = \frac{\hat{\mu} F_y}{k} = \frac{\hat{\mu} c_y T_n^2}{4\pi^2}$$

(3.31)

where $\hat{\mu}$ denotes the ductility demand estimated from the GMPE. The strength of the system, denoted as $c_y$, is either known to the engineer or is assessed using non-linear procedures, for example, pushover analysis. This model provides a simple and rational basis that can be implemented in probabilistic seismic demand analysis (PSDA)(Cornell, 1996; Luco, 2001). Finally, it is emphasized that the results presented in Rupakhety and Sigbjörnsson (2009a) are not derived from an extensive database. Therefore, they are intended more to illustrate the concept than to provide ready-to-use results. Implementation of the model with large sets of ground-motion data available from the European region—and possibly supplemented by data from other similar tectonic environments—will provide valuable information about the seismic demand of existing structures in the event of a future earthquake.
4 Near-fault ground motions

4.1 Introduction

Near-fault ground motions are known to affect certain types of engineering structures more severely than far-fault ones. Several studies describing the important characteristics of near-fault ground motions and their effects on engineering structures are reported in the literature. A detailed treatment of near-fault issues in engineering design is presented in Rupakhety et al. (2010b), which this chapter complements by providing some relevant background information and several practical examples.

4.2 Historical background

The potential of near-fault ground motions to severely damage engineering structures became evident after strong earthquakes in the past, for example, the 1994 Northridge, the 1995 Kobe, and the 1999 Chi-Chi earthquakes. This potential problem was however known to seismologists long before these earthquakes occurred. Benioff (1955), for example, had reported strong seismological evidence of near-fault phenomena while explaining the intensity patterns observed after the 1952 Kern County, California, earthquake.

The first accelerogram with a clearly visible near-fault effect was obtained at Station No.2 (Coalinga 2, C02) during the 1966 Parkfield, California, earthquake. This record contained a distinct long-period velocity pulse, not common in typical far-fault ground-motion records. This long period pulse was first discovered by Housner and Trifunac (1967). Aki (1968), in computing synthetic seismograms at this station using a moving dislocation model, found excellent agreement between the simulated
and recorded motion and concluded that the dislocation is the most dominant source parameter controlling seismic motion near an earthquake fault.

The severe effects of near-fault ground motions on engineering structures became clearer after the 1971 San Fernando, California, earthquake. The Pacoima Dam recording of this earthquake contained a distinct pulse in the fault-normal component of ground velocity. The severe damage sustained by the Olive View Hospital Medical Treatment and Care Facility during this earthquake was studied in detail by Mahin et al. (1976). Based on this study, Bertero et al. (1978) associated the structural damage sustained by the hospital building with the long-duration acceleration pulses observed in the near-fault accelerograms of the earthquake. This was the first study to point out that near-fault ground motions may result in very large displacement ductility demands in buildings designed according to the code provisions of the time. The study also revealed that near-fault ground motions, which are associated with few large displacement cycles, are more severe to structures than far-fault ones, which contain many small amplitude oscillations.

Anderson and Bertero (1987) studied the response of steel frames subjected to representative ground motions recorded during the 1979 Imperial Valley earthquake. Their study showed that the nonlinear response of a structure is sensitive to the duration of acceleration pulses relative to the fundamental period of the structure and its amplitude relative to the yield strength of the structure. Their study recommended that design spectra used at the time should be increased in the long period range to account for near-fault effects.

Hall et al. (1995) studied the response of building structures using artificially generated pulse-like ground motions. Their analysis showed that displacement pulses occurring in near-fault ground motions impose severe displacement demands on high-rise and base-isolated buildings designed according to code provisions. Iwan (1997) observed that ground motion pulses in the near-fault area travel through the height of the building as waves. He argued that the modal superposition method using response spectra may not be be appropriate to analyze structures responding to near-fault pulses. He proposed using drift spectrum in such cases. However, Chopra and Chintanapakdee (1998) argued that the response spectrum analysis is accurate for near-fault ground motions and should be preferred over the drift spectrum proposed by Iwan (1997). Malhotra (1999) used recorded as well as simulated accelerograms to study the effects of near-fault ground motions on building structures. He concluded that pulses in ground motion do not always result in significantly higher mode contributions in high-rise buildings. Therefore, wave propagation analyses as advocated by Iwan (1997) are not necessary for ground motions with relatively high ratios of peak ground velocity to peak ground acceleration, but might be suitable for those with relatively low ratios of the same.

Hall and Ryan (2000) and Makris and Chang (2000a,b), among others, studied base-
isolation and energy dissipation systems to reduce near-fault effects in engineering structures. Anderson et al. (1999) found that retrofitting flexible structures by increasing their stiffness is not desirable in the near-fault region. Chai and Loh (1999) used three types of velocity pulses to calculate strength reduction factors of inelastic structures and found that strength demands depend on the pulse period of ground motion relative to the fundamental period of vibration of structures. Nakashima et al. (2000) studied the response of steel moment-resisting frames subjected to near-fault ground motions in Japan, Taiwan and the U.S.A. and concluded that ground motions in these regions caused similar story drift demands in the frames.

Alavi and Krawinkler (2004) used generic frames as models of building structures and studied their response to near-fault ground motions and their equivalent pulse representations. Their study highlighted some important features of the response of building structures to near-fault ground motions. First, the spectral response of structures to near-fault ground motions severely exceeds the values recommended by design codes. Second, elastic story shear demands are highest at upper stories if the building’s fundamental period is larger than the period of predominant velocity pulse. This causes early yielding in higher stories of weak systems, and yielding migrates towards the base, where it grows with further reduction in strength. When the structural period is shorter than the pulse period, yielding mainly occurs at the bottom portions of the building. This indicated that the load patterns—related to equivalent static loads—recommended in design codes might not be appropriate for near-fault ground motions. Mollaioli et al. (2006) studied the response of SDOF systems to near-fault ground motions and equivalent pulses. They observed that long-period pulses impose very high energy and displacement demands, which may be several times larger than the limits specified by design codes. Phan et al. (2007) performed shake table tests on reinforced concrete bridge columns subjected to near-fault ground motions and found that they suffered large residual displacements even under moderate motions.

4.3 Characteristics of near-fault ground motion

From a structural engineering point of view, the impulsive nature of near-fault ground motions is its most important characteristic. Structures designed for non-impulse ground motions can sustain severe damage when exposed to strong impulsive ground motions, whose energy is concentrated in a short duration. The amplitude and frequency content of pulse-like ground motions are controlled by various seismological processes, such as, directivity effects, permanent displacement effects, hanging wall effects, and surface or interface P-wave effects. A detailed description of these effects is outside the scope of this study. For conditions leading to different types of directivity effects and the resulting ground motion characteristics, readers are referred to Somerville (1997) and Somerville et al. (1997). Abrahamson (2000) provides a description of permanent translation effects. Hanging wall effects are described in Abrahamson and Somerville (1996) and Mavroeidis and Papageorgiou (2002). Descriptions of surface or interface P-wave effects can be found in Bouchon (1978), Kawase and Aki (1990), and Mavroeidis and Papageorgiou (2000).

4.4 Analytical models of near-fault pulses

Simple waveforms representing typical ground-motion pulses in the near-fault area can be found in the literature; some of them are briefly described below.

4.4.1 Hall et al. 1995

The use of simple waveforms to study the response of engineering structures to near-fault ground motions first appeared in the the works of Hall et al. (1995), who used triangular velocity pulses of two types, namely, Type A and Type B pulses. The Type A pulse, called ‘forward-motion only’ by the authors, is triangular in shape with a duration of $T_p/2$ and an amplitude of $v_{gA}$. The type B pulse, called ‘forward and back motion’ by the authors, consists of two Type A pulses joined in opposite directions. Type A and Type B pulses are shown schematically in Fig. 4.1.
4.4. Analytical models of near-fault pulses

4.4.2 Makris 1997

Makris (1997) used cycloidal fronts to represent the forward and forward-backward motion pulses; the expressions for Type A and Type B velocity pulses used by him are:

\[ v_{gA} = \frac{V_p}{2} - \frac{V_p}{2} \cos(\omega_p t), \quad 0 \leq t \leq T_p \]  

(4.1)

\[ v_{gB} = -V_p \cos(\omega_p t + \frac{\pi}{2}), \quad 0 \leq t \leq T_p \]  

(4.2)

where \( V_p \) and \( T_p = \frac{2\pi}{\omega_p} \) are the amplitude and duration of the pulse, respectively.

4.4.3 Makris and Chang 2000

Makris and Chang (2000b) used Type C pulses to represent near-fault ground motions with one or more long-duration cycles of displacement. A Type C pulse with \( n \) displacement cycles is called a Type \( C_n \) pulse and its displacement is given by:

\[ u_{gC_n}(t) = \frac{V_p}{\omega_p} \left[ -\cos(\omega_p t + \varphi) + \cos \varphi - t\omega_p \sin \varphi \right], \quad 0 \leq t \leq \left( n + \frac{1}{2} - \frac{\varphi}{\pi} \right) T_p \]  

(4.3)

where the phase angle, \( \varphi \), is set to have zero ground displacement at the end of the pulse. Its value depends on \( n \) and is given by the solution of the following
transcendental equation:

\[
\cos[(2n + 1)\pi - \varphi] + [(2n + 1)\pi - 2\varphi] \sin \varphi - \cos \varphi = 0
\]  

(4.4)

The Type A, Type B, and Type C\(C\) pulses are shown in Fig. 4.2, taking \(V_p = 1\), and \(T_p = 2\).

![Graph showing Type A, Type B, and Type C velocity pulses](image)

*Figure 4.2: Type A, Type B, and Type C\(C\) velocity pulses used by Makris and Chang (2000b).*

### 4.4.4 Alavi and Krawinkler 2001

Alavi and Krawinkler (2001) used three types of pulses, namely, P1, P2, and P3. The P1 and P2 pulses are similar to the Type A and Type B pulses of Hall et al. (1995). The P1, P2, and P3 velocity pulses normalized by their peak values are shown in Fig. 4.3; the time axis is normalized by pulse period \(T_p\).
4.4. Analytical models of near-fault pulses

4.4.5 Menum and Fu 2002

Menum and Fu (2002) put forward an analytical equation representing fault-normal components of near-fault ground velocity. The velocity pulse is given by

\[
v_g = \begin{cases} 
V_p \exp \left[ -n_1 \left( \frac{3}{4} T_p - t + t_0 \right) \right] \sin \left( \frac{2\pi}{T_p} (t - t_0) \right), & t_0 < t \leq t_0 + \frac{3}{4} T_p \\
V_p \exp \left[ -n_2 \left( t - t_0 - \frac{3}{4} T_p \right) \right] \sin \left( \frac{2\pi}{T_p} (t - t_0) \right), & t_0 + \frac{3}{4} T_p \leq t \leq t_0 + 2T_p \\
0, & \text{otherwise}
\end{cases}
\]

where \( V_p \) is the pulse amplitude; \( T_p \) is the pulse period; \( t_0 \) is the time when the pulse starts, and \( n_1 \) and \( n_2 \) are related to the shape of the pulse. The authors found that \( V_p \) is strongly related to peak ground velocity, and \( T_p \) is strongly related to the predominant period of ground-motion (the period where 5%-damped elastic pseudo-velocity spectral amplitude is the maximum).

The fault-normal components of ground velocities recorded at the PCD station during the 1971 San Fernando earthquake and Fault Zone 1 station during the 2004 Parkfield earthquake are compared with the ones simulated using Eq. 4.5 in Fig. 4.4. The
parameters selected for the PCD record are $V_p = 115$, $T_p = 1.47$, $n_1 = 1.4$, and $n_2 = 1.4$; while $V_p = 60$, $T_p = 1.3$, $n_1 = 3.5$, and $n_2 = 0.5$ are selected for the Fault Zone 1 record.

![Figure 4.4: Velocity pulses used by Menum and Fu (2002) compared with recorded velocities. The grey and dark lines represent the recorded and simulated velocities, respectively.](image)

**4.4.6 Mavroeidis and Papageorgiou 2003**

Mavroeidis and Papageorgiou (2003) modified the Gabor wavelet to find an analytical expression for near-fault pulses. According to their model, the velocity waveform is given by

$$v_g = \begin{cases} \frac{4}{2} \left[ 1 + \cos \left( \frac{2\pi f_p}{\gamma} (t - t_0) \right) \right] \cos \left[ 2\pi f_p (t - t_0) + \nu \right] , & t_0 - \frac{\gamma}{2f_p} \leq t \leq t_0 + \frac{\gamma}{2f_p} \\ 0 , & \text{otherwise} \end{cases}$$

(4.6)

where $A$ is the amplitude of the pulse; $f_p = 1/T_p$ is the frequency of the pulse; $\nu$ is the phase angle; $\gamma > 1$ controls the zero-crossings of the pulse, and $t_0$ is related to the location of the pulse. This model, like that of Menum and Fu (2002), has five parameters. The two parameters related to the amplitude and period of the pulse assume similar values when either of the two models is calibrated with a recorded
ground velocity. Whereas the number of half-cycles and the phase angle of the pulse are controlled, respectively, by $\gamma$ and $\nu$ in the Mavroeidis and Papageorgiou (2003) model, they are controlled by $n_1$ and $n_2$ in the Menum and Fu (2002) model. These two models often provide similar results. In Fig. 4.5, ground velocities shown in Fig. 4.4 are plotted with the velocity pulses simulated from the analytical equations of Menum and Fu (2002) and Mavroeidis and Papageorgiou (2003). $A = 115$, $T_p = 1.47$, $\gamma = 1.6$, and $\nu = 180$ are used to simulate ground velocity at the PCD station; $A = 60$, $T_p = 1.3$, $\gamma = 2.1$, and $\nu = 150$ are used to simulate ground velocity at the Fault Zone 1 station. Note that $A$ and $V_p$—which represent pulse amplitudes in the models of Mavroeidis and Papageorgiou (2003) and Menum and Fu (2002), respectively—are equal for the two records shown in Fig. 4.4. In addition, $T_p$ values corresponding to these two models are also the same. Fig. 4.5 shows that these two models produce similar pulses. In addition, 5%-damped elastic pseudo-velocity spectra (PSV) of the two records are compared with those of the simulated pulses in Fig. 4.6. The comparison shows that the pulses generated by both of the models have similar response spectra; they match well with those of recorded motion at long periods but lack high frequencies.

Figure 4.5: Velocity pulses from the models of Menum and Fu (2002)(red) and Mavroeidis and Papageorgiou (2003)(blue) compared with recorded ground velocities (grey).
4.4.7 He and Agrawal 2008

He and Agrawal (2008) proposed amplitude-modulated sinusoidal waveforms to represent near-fault ground-motion pulses. The velocity pulse proposed by them is by

\[
v_g = \begin{cases} 
  C (t - t_0)^n \exp\left[-a (t - t_0)\right] \sin \left[\omega_p (t - t_0) + \nu\right], & t \geq t_0 \\
  0, & \text{otherwise}
\end{cases}
\]  

(4.7)

where \(n\) is a non-negative shape parameter; \(\omega_p = 2\pi/T_p\) is the pulse frequency; \(C\) is the amplitude scaling parameter; \(a\) is the decay factor; \(t_0\) is the time when the pulse starts, and \(\nu\) is the phase angle. A comparison between recorded velocity and the pulse models of He and Agrawal (2008), Mavroeidis and Papageorgiou (2003), and Menum and Fu (2002) is presented on the top panel of Fig. 4.7. Strike-normal component of the El Centro Array Number 6 record of the 1979 Imperial Valley earthquake is considered. The bottom panel of the figure compares the 5%-damped elastic PSV of the recorded motion with those of the pulses. The parameters of the pulse corresponding to Mavroeidis and Papageorgiou (2003) and He and Agrawal (2008) are taken from the respective papers, while the parameters for the model of Menum and Fu (2002) are selected as \(V_p = 115\), \(T_p = 3.85\), \(n_1 = 0.8\), and \(n_2 = 0.35\). It is observed that the velocity pulses generated by all of these three models match the recorded ground velocity reasonably well. In addition, their response spectra match well with the response spectra of the recorded motion at long periods, but all of the pulses lack high frequencies.
4.4. Analytical models of near-fault pulses

Apart from analytical functions discussed above, other approaches of extracting velocity pulses from near-fault ground motions are also available in the literature. Baker (2007) proposed a wavelet-based technique to extract dominant velocity pulses from near-fault ground-motion records. His method has well-defined quantitative criteria to determine whether a given ground-motion is pulse-like or not. This provides consistently reproducible results, while the analytical functions described above involve subjective decisions in selecting their parameters. The top panel of Fig. 4.8 compares the wavelet (using the method of Baker (2007)) extracted from the strike-normal component of the PCD record of the 1971 San Fernando earthquake (red line) with the recorded ground velocity (grey line). For comparison, the pulse model of Mavroeidis and Papageorgiou (2003) is also shown (blue line). The bottom panel of the figure shows their 5%-damped elastic PSV. The model of Mavroeidis and Papageorgiou (2003) matches the ground velocity and its corresponding PSV better than the wavelet extracted by Baker (2007). In fact, the main purpose of the wavelet technique described in Baker (2007) is to rapidly identify pulse-like ground motions from a large database, which it does with high efficiency. More recently, Xu and Agrawal (2010) have shown that empirical mode decomposition can be used to identify and isolate dominant pulses from near-fault ground-motion records.
Figure 4.8: Comparison of velocity and PSV of the strike-normal ground-motion record (grey) at the PCD station during the 1971 San Fernando earthquake with the wavelet extracted by Baker (2007)(red) and the pulse model of Mavroeidis and Papageorgiou (2003)(blue).

### 4.5 Which pulse model?

The use of simple pulses in studying the dynamic response of structures has a long history, for example, Biggs (1964) used one-sided pulses of rectangular, triangular and ramp-like shapes to evaluate elastic as well as inelastic responses of SDOF systems. The purpose of the study was to quantify the impact of impulsive forces, such as those caused by bomb blasts, on engineering structures. This study showed that the amplitude of the pulse and the period of the pulse relative to the period of vibration of an SDOF system are the two important parameters controlling the maximum elastic response of the SDOF. Since near-fault ground motions exhibit impulse character, it was natural that researchers followed the approach of Biggs (1964), i.e., they used simple pulses to model near-fault ground motions. Starting with the simple pulse models of Hall et al. (1995), many different types of pulses can be found in the literature—some of them discussed in the previous section. The search for pulse models better than the ones used by Biggs (1964) started with the observation that, unlike blast loads, earthquake ground motions are not purely impulsive because they are related to a dynamic process. To incorporate the oscillatory character of near-fault ground motions, Hall et al. (1995) used two different types of pulses; namely, Type A and Type B. The type C pulses proposed by Makris and Chang (2000b) introduced further improvements in controlling the phase and number of oscillations of the pulse. Then, Menum and Fu (2002) used continuous parameters, such as $n_1$ and $n_2$, to control the oscillatory behaviour and relative amplitudes of different
4.6 Why pulse models?

While more and more efforts are being made to create better pulse models by adding additional parameters to the model, the usefulness of these models has received little attention. First, it is important to realize why these pulse models are needed. Then, it must be evaluated whether or not they fulfil their objectives. The main reasons why a simple pulse is used to represent near-fault ground motions are summarized below.

- To cope with the lack of near-fault ground-motion records, simple pulses were used in studying the response of structures to such motions. Today, several records of near-fault ground motions from earthquakes of different size are available, and using simple pulses is not essential.
- To save the computational cost of dynamic analyses of structures, simple pulse models were useful because closed-form solutions of the elastic response of SDOF systems subjected to these pulses were available in many cases. This was very important when computers were slow and had costly memory. Today, dynamic analyses of even the most complicated models of structures can be performed relatively fast on modern computers. Simple pulses are therefore no longer needed for studying the dynamic response of structures to near-fault ground motions. On the contrary, using simple pulses in time history analyses of structures responding in multiple vibrational modes can lead to serious underestimation of their drift demands. A thorough description of this limitation of simple pulses is presented later in Section 4.8.
- Another objective of using simple pulses, as mentioned in Mavroeidis and Papageorgiou (2003), is quoted here:

Furthermore, such a development would enable engineers to study the response of conventional, non-conventional (e.g., base-isolated), and special structures (e.g., fluid-storage tanks, suspension bridges)
to near-fault seismic excitations more effectively as a function of the input parameters of the analytical models and thus, ultimately, of the earthquake size.

This is a valid argument if the relationship between all the input parameters of a pulse model and the size of an earthquake are reliably established. However, only two parameters, namely, pulse period and pulse amplitude, can be related to earthquake size, while others are arbitrary. In fact, pulse amplitude is related to the peak ground velocity, and pulse period is related to the predominant period of ground motion (see Rupakheti et al., 2010b). These quantities can be obtained from the ground-motion record itself; a pulse model is not required to define them.

• Halldorsson et al. (2010) used a simple pulse model to simulate long-period components of ground-motion time series, which, according to the authors, can be used in non-linear time history analyses of structures. The reliability of such simulations largely depends on the confidence with which the parameters of the pulse model can be estimated. As mentioned earlier, only pulse amplitude and pulse period can be reliably estimated from earthquake size and source-to-site distance, while the others are arbitrary; they are nevertheless very important as demonstrated below.

4.7 Sensitivity analysis of pulse parameters

Ground motions simulated from simple pulse models are sensitive to the model parameters, for example, $\gamma$ in the model of Mavroeidis and Papageorgiou (2003). In this section, a demonstration of the sensitivity of simulated ground motions to $\gamma$ is presented. Before presenting the results, some assumptions made in the simulation are summarized.

4.7.1 Pulse amplitude

Pulse amplitude is related to peak ground velocity which can be predicted from attenuation equations, such as the one proposed in Rupakheti et al. (2010b).
4.7. Sensitivity analysis of pulse parameters

4.7.2 Pulse period

Pulse period is related to the earthquake size and can be predicted from scaling relations, such as the one presented in Mavroeidis and Papageorgiou (2003).

4.7.3 Oscillatory character

The number of half-cycles of the pulse represented by $\gamma$ in Mavroeidis and Papageorgiou (2003) is not predictable, and is treated as a random variable. The probability distribution of this variable is not clear, and a discussion regarding it is deemed necessary. While Halldorsson et al. (2010) assume $\gamma$ to be normally distributed, there are certain problems associated with this assumption. In the left panel of Fig. 4.9, a histogram plot of $\gamma$ as reported in Mavroeidis and Papageorgiou (2003) is shown. The black line in the figure represents a normal distribution fitted to the data which has a mean value of 1.8 and standard deviation of 0.5. A Jarque-Bera test at 5% significance showed that the null hypothesis that $\gamma$ is normally distributed is invalid. Assuming that $\gamma$ is normally distributed with a mean value of 1.8 and standard deviation of 0.5, 1000 random samples of $\gamma$ were obtained from a Monte Carlo simulation. The histogram of $\gamma$ simulated in this manner is shown in the right panel of Fig. 4.9. The figure reveals that, out of 1000 simulations, about 120 realizations of $\gamma$ are less than one. The pulse model of Mavroeidis and Papageorgiou (2003), however, requires $\gamma$ to be greater than one. Therefore, $\gamma$ cannot simply be represented by a normally distributed random variable with the mean and standard deviation taken from the data of Mavroeidis and Papageorgiou (2003), as was done in Halldorsson et al. (2010). The histogram shown in the left panel of Fig. 4.9 shows that $\gamma$ is approximately uniform except for values greater than 2.5, of which there are four occurrences. It is therefore assumed that $\gamma$ is uniformly distributed in the interval $[1.02 \quad 2.5]$.

Figure 4.9: Left: distribution of $\gamma$ plotted from the data reported in Mavroeidis and Papageorgiou (2003) compared with a normal curve fitted to the data, Right: randomly simulated values of $\gamma$ assuming it to be a normal random variable with a mean of 1.8 and standard deviation of 0.5.
4.7.4 Phase angle

Mavroeidis et al. (2004) showed the effect of phase angle ($\nu$) on elastic response spectra to be minimal, and it is therefore assigned a constant value of 90°.

4.7.5 Simulation

Let us assume a scenario where long-period ground-motion time series are required for time history analyses of structures. The magnitude of the design earthquake is 6.52, and the controlling source capable of generating pulse-like ground motions at the site of interest is 3.76 km away. Based on these assumptions, peak ground velocity is computed from the attenuation relation presented in Rupakhety et al. (2010b). Assuming log-normal distribution, 1000 samples of PGV are generated for this magnitude and distance combination. The mean value of the simulated samples of PGV is 71 cm/s². It is assumed that $A$ is 90% of PGV. Then, 1000 realizations of $T_p$ are simulated using the mean and standard deviation of $T_p$ reported in Halldorsson et al. (2010). The mean value of the simulated $T_p$ samples is 2.44 s. Then, 1000 realizations of $\gamma$ are simulated assuming it to be uniformly distributed in the interval $[1.02, 2.5]$. With these parameters, 1000 different pulses are simulated using Eq. 4.6. The simulated velocity time series are presented in Fig. 4.10. Grey lines in the figure show the realizations, while the red line is the mean value of all the realizations. The coloured lines represent some samples of the simulated velocity time series. It is noticeable how the mean pulse lacks oscillatory character.

Figure 4.10: 1000 samples of simulated velocity pulses for a moment magnitude of 6.5 and source-to-site distance of 3.76 km.

After simulating these pulses, an engineer has to decide whether to use the mean pulse as a representative ground motion or to use all of these pulses and compute the statistics of the response of the structure being analyzed. Both these approaches
4.7. Sensitivity analysis of pulse parameters

are demonstrated here, using an SDOF system. The damping ratio of the SDOF system is fixed at 5% of its critical value, and undamped natural periods from 0.05 s to 5 s are considered to create elastic response spectra (expressed in terms of PSV). The results are shown in Fig. 4.11. In the left panel, grey lines represent the PSV of the 1000 simulated velocity pulses, while the green line is the median value. The red line represents the PSV of the mean pulse indicated by the red line in Fig. 4.10. The large variability of the response spectra is evident from the scatter of grey lines plotted in the left panel of Fig. 4.11. The standard deviation of log-transformed (base 10) PSV is shown in the right panel of Fig. 4.11; the dashed line represents the mean pulse period corresponding to the earthquake being considered. The higher standard deviations at long periods, where near-fault ground motions severely affect engineering structures, implies that near-fault ground-motion time series simulated from simple pulse models result in large uncertainties in the computed structural response. A large proportion of this uncertainty is related to the unpredictable nature of \( \gamma \), which affects the response spectral ordinates at periods close to and greater than the pulse period. Even if we consider this uncertainty to be realistic, whether the results are in accordance with available records of near-fault ground motions needs to be checked.

![Figure 4.11: PSV of simulated velocity pulses shown in Fig. 4.10.](image)

To compare the results obtained above with available records of near-fault ground motions, nine accelerograms corresponding to earthquake magnitudes in the range of 6.49 to 6.57 and a mean value of 6.52 are used. Note that the magnitude of the simulated pulses was intentionally selected to be equal to 6.52 in order to make a valid comparison. In addition, the average distance between the stations recording these accelerograms and their corresponding sources is equal to 3.76 km, which is also the distance considered for simulated pulses. The mean value of the recorded PGV of these records is equal to 76 cm/s, which is fairly close to the mean PGV of the simulated pulses; any bias from the attenuation relationship of PGV is therefore avoided. A comparison between the median PSV of these records and that of the simulated pulses is shown in Fig. 4.12. The difference between the two is alarming.
The median PSV of the simulated pulses is significantly smaller than that of the recorded accelerograms. In addition, the median spectrum of the simulated pulses (black line in Fig. 4.12) shows a broadband character different from the narrow-banded nature of the median spectrum of the recorded accelerograms (red line in Fig. 4.12). This effect is partly due to the uncertainties related to pulse period. Using the regression equation of Halldorsson et al. (2010), the simulated pulse periods for the magnitude being considered vary from 1 s to 6 s with a mean value of 2.44 s. Such a wide range of pulse periods means that the PSV of the simulated pulses will have their peaks in a wide range of periods. When these spectra are averaged, the narrow-banded nature of PSV is lost. In addition, the variability of \( \gamma \) adds to these effects because the period at which the PSV of a simulated pulse is maximum depends not only on \( T_p \) but also on \( \gamma \). In this sense, \( T_p \) and \( \gamma \) cannot be considered as independent variables. If all the parameters of any pulse model cannot be reliably estimated independently from the parameters of an earthquake source and/or wave propagation path, it is questionable whether these models are useful to simulate ground-motion time series for design or evaluation of structures in a given earthquake scenario.

\[ \text{Figure 4.12: Median PSV of simulated velocity pulses compared with that of recorded ground motions.} \]

4.8 Pulses and tall buildings

It is commonly assumed that if the fundamental period of a structure is greater than the pulse period, then an equivalent pulse (instead of recorded accelerogram) can be used in dynamic analysis of the structure. This argument is based on the observation that the response spectra of equivalent pulses and recorded motions are similar for \( T_1 > T_p \), where \( T_1 \) represents the period of the first mode of vibration of a
4.8. Pulses and tall buildings

building. While this assumption might be satisfactory for short buildings whose total response is dominated by their first mode of vibration, it might not be valid for tall buildings where higher modes are important. Higher modes are associated with higher frequencies, and since simple pulses lack high-frequency components, they cannot simulate the response of a tall building to actual ground motions, which contain high frequency components in addition to long-period pulses. Although higher modes have small contributions to base shear demands, their contributions to story shear forces of tall buildings are significant. Story shear forces control drifts between two adjacent floors. Since inter-story drifts are related to building damage, how accurately they can be predicted by using simple pulses needs to be examined. In the following sections, some examples illustrating the inadequacy of simple pulses in predicting inter-story drift demands in tall buildings are presented.

4.8.1 Structural model

The structure being considered is an 18-story plane steel moment-resisting frame (SMRF). It is a single bay frame with bay-width 7.32 m, and all the stories have equal heights of 3.66 m. More details about the design of the frame can be obtained from Chintanapakdee and Chopra (2003). It should be noted that the frame is designed to have a regular distribution of stiffness. The period of the first mode of vibration of the frame is 3.32 s. Rayleigh damping with 5% of critical value in the first and the 18th mode of vibration is considered, and it is assumed that the structural components remain elastic. The first few mode shapes and modal participation factors of the frame are reported in Rupakhety (2008).

4.8.2 Structural analysis

Maximum drifts of the SMRF are computed from the three procedures described below.

4.8.2.1 Dynamic time history analysis

The frame is modelled and analyzed with the computer program Ruauomoko (Carr, 2008a,b,c). Nodal displacement time histories obtained from the program are used to calculate inter-story drift time histories. Inter-story drift demand \( \delta \) is then taken as the maximum value of inter-story drift normalized by story height. Its values corresponding to an actual ground-motion record and its equivalent pulse are denoted by \( \delta_{Rr} \) and \( \delta_{Rp} \), respectively.
4.8.2.2 Model superposition

Displacement time histories at all the stories are computed by modal superposition of the 18 modes of vibration of the SMRF. Modal displacements are computed from the numerical integration of the equation of motion of a SDOF system characterized by the period of vibration, damping ratio, and participation factor of each mode. Modal displacements are then converted to structural displacements by using the corresponding mode shapes. The contributions of all the modes are summed, and the total displacement time history is obtained, from which $\delta$ is calculated as described in Section 4.8.2.1. Inter-story drift ratios computed in this manner are denoted by $\delta_{MSr}$ when an actual ground-motion record is used and $\delta_{MSp}$ when its equivalent pulse is used.

4.8.2.3 SRSS combination of peak modal response

In this method, displacement time histories of different modes are converted to $\delta$ time histories. The maximum values of these $\delta$ time histories are computed separately for all 18 modes of vibration. Finally, the maximum values are combined, using the square-root-of-sum-of-squares (SRSS) rule. It should be noted that this procedure is slightly different from response spectrum analysis, in which maximum drifts are computed from maximum displacements. Inter-story drift demands computed in this manner are denoted by $\delta_{SRSSr}$ when an actual ground-motion record is used and $\delta_{SRSSp}$ when its equivalent pulse is used.

4.8.3 Illustration 1

In this illustration, the strike-normal component of the accelerogram recorded at the SCG (see Rupakheti et al. (2010b)) station during the 1994 Northridge earthquake ($M_w 6.7$) is used. This record is selected because the velocity pulse contained in it has a period of 2.94 s, which is smaller than the fundamental period of the SMRF. In the left panel of Fig. 4.13, recorded ground velocity and the simulated pulse using the model of Mavroeidis and Papageorgiou (2003) are plotted in blue and red, respectively. The right panel shows the corresponding 5%-damped elastic PSV. The periods of the first three modes of vibration of the SMRF are indicated by the vertical lines. The simulated and the recorded PSV match very well in the long-period range, while the pulse lacks high frequencies.

The inter-story drift demands of the SMRF computed from the procedures described in Section 4.8.2.1 and Section 4.8.2.2 are presented in Fig. 4.14. The results obtained
from Ruaumoko and modal superposition overlap each other, which is expected for an elastic system. However, the equivalent pulse significantly underestimates inter-story drift demands of the SMRF. At the roof level, the inter-story drift ratio obtained by using the pulse is 3 times smaller than that obtained by using the actual record. The differences between the two are greatest on the top stories. It is also noticeable how the pulse predicts almost constant drift along the height of the frame. It is worth mentioning here that the frame is designed to have almost constant drift along its height in the first mode of vibration. It is therefore evident that the pulse, which has a dominant frequency comparable to the first mode frequency of the frame, is capturing only the first mode of vibration of the system. On the other hand, the recorded ground motion contains significant energies at high frequencies, and the distribution of drift demands along the height of the frame is influenced by its higher mode responses.

To further clarify the effects of the pulse lacking in high frequencies, an examination...
of the contributions of different modes of vibration to the peak inter-story drifts of the frame would be useful. Since the peaks of all the modes do not occur at the same time, the actual contribution of each mode to the total response is difficult to evaluate. However, drift demands due to individual modes relative to the total drift demand computed from SRSS combination provide a measure of the relative importance of the different modes. In Fig. 4.15, peak inter-story drift ratios computed from the SRSS combination of modal response are presented; the left and the right panels of the figure correspond to the actual accelerogram and its equivalent pulse, respectively. The blue and red lines represent the total values of $\delta_{SRSS_r}$ and $\delta_{SRSS_p}$, respectively. Modal contributions are indicated by horizontal bars of different colours; the colours representing the first four modes of vibration are identified in the colour bar near the right edge of the figure. The left panel of the figure shows that higher mode contributions to $\delta_{SRSS_r}$ are significant at the upper stories of the SMRF. Because the pulse lacks high frequencies, their contributions are missing in $\delta_{SRSS_p}$, which is almost entirely contributed by the first mode. It should be noted that the drifts computed by the SRSS rule are slightly smaller than those shown in Fig. 4.15.

Figure 4.15: Inter-story drift demands of the 18-story SMRF computed by SRSS combination of modal response; the strike-normal component of the SCG record of the 1994 Northridge earthquake and its equivalent pulse are considered.

4.8.4 Illustration 2

The poor performance of the pulse discussed above is partly due to the presence of multiple peaks in the PSV of the ground motion to which it was fitted. These peaks are clearly visible in Fig. 4.12. Although near-fault ground motions might contain more than one pulses resulting in multiple peaks in their PSVs, they are not very common. In this illustration, the strike-normal component of the El Centro Array number 6 record of the 1979 Imperial Valley earthquake is selected. The velocity time series and 5%-damped elastic PSV of this record and the corresponding pulse are shown in Fig. 4.16. This ground motion has a clear peak in the PSV, and the PSVs of the record and its equivalent pulse match very well at long periods. The vertical lines in the right panel of the figure indicate the periods of the first three
4.8. Pulses and tall buildings

modes of vibration of the SMRF.

Figure 4.16: Ground velocity, equivalent pulse, and the corresponding elastic response spectra of the strike-normal component of the El Centro Array number 6 record of the 1979 Imperial Valley earthquake.

The SMRF was analyzed using the recorded motion and its equivalent pulse. The results are presented in Fig. 4.17. The red line corresponds to the maximum inter-story drift ratio corresponding to the actual record, while the black line corresponds to its equivalent pulse. The pulse is found to underestimate drift demands in the upper quarter of the frame. Drift demand computed from the SRSS combination of modal responses (using the actual record) is shown with the blue line; the coloured bars indicate modal contributions. It is apparent that the SRSS combination does not give accurate results at the upper stories. These effects were also observed by Iwan (1997) and Alavi and Krawinkler (2001). The results presented here support the argument of Iwan (1997) that the modal superposition method might not be appropriate for pulse-like ground motions.

Figure 4.17: Inter-story drift demands of the 18-story SMRF, computed by Ruaumoko and SRSS combination; the strike-normal component of the El Centro Array number 6 record of the 1979 Imperial Valley earthquake and its equivalent pulse are considered.

4.8.5 Illustration 3

Finally, a case where the pulse period is larger than the fundamental period of the frame is considered. For this purpose, the strike-normal component of the Lucrene Valley record of the 1992 Landers earthquake is selected. The predominant pulse contained in this ground motion has a period of 5.88 s. The ground velocity of this
record is compared with the simulated pulse in the left panel of Fig. 4.19; their 5% damped elastic PSV are compared in the right panel of the figure. The resulting drift demands are presented in Fig. 4.19. These results are consistent with the results obtained for other ground-motion records discussed above. It is therefore highlighted that using simple pulses in time history analysis of tall buildings can lead to unreliable conclusions regarding drift demands and their heightwise distribution. These results correspond to regular and elastic structures. Irregular structures responding in the inelastic domain should be considered even more seriously before a simple pulse is used to estimate their response to near-fault ground motions.

Figure 4.18: Ground velocity, equivalent pulse, and the corresponding elastic response spectra of the strike-normal component of the Lucrene Valley record of the 1992 Landers earthquake.

Figure 4.19: Inter-story drift demands of the 18-story SMRF computed by Ruaumoko and SRSS combination; the strike-normal component of the Lucrene Valley record of the 1992 Landers earthquake is considered.

4.8.6 Statistical evaluation

4.8.6.1 Ground-motion records

In the preceding section, equivalent pulses were evaluated against three different ground-motion records in their capacity to estimate maximum inter-story drift ratios along the height of a 18-story SMRF. The equivalent pulses were found to significantly underestimate the drift demands at the upper quarter of the SMRF, a definite
4.8. Pulses and tall buildings

conclusion, however, cannot be derived from the results of a few records. A statistical study is therefore necessary to obtain a more reliable assessment of the pulse performance. To perform such a study, 28 ground-motion records are selected, all of which have been analyzed and fitted with equivalent pulses by Mavroeidis and Papageorgiou (2003). Each of these records satisfies the criterion that $T_1 > 0.7T_p$, where $T_1$ is the fundamental period of the SMRF. This criterion is selected because it is generally believed that the pulse performance is acceptable when it is met. The parameters of the pulse model are taken from Mavroeidis and Papageorgiou (2003). More details of these records can be obtained from Rupakhety et al. (2010b); the corresponding Waveform Identification numbers (WID) of the selected records are 1, 2, 7, 10, 11, 12, 13, 26, 28, 30, 34, 38, 42, 44, 45, 46, 70, 71, 74, 75, 76, 77, 78, 79, 80, 81, 83, and 87. A comparison between the mean 5%-damped elastic PSV of these 28 records and their corresponding pulses is shown in Fig. 4.20; the periods of the first three modes of vibration of the SMRF are shown with vertical lines.

![Figure 4.20: 5%-damped mean elastic PSV of the 28 ground-motion records (blue) and their corresponding pulses(red); the vertical lines indicate the periods of the first three modes of vibration of the SMRF being analyzed with these ground motions.](image)

### 4.8.6.2 Error statistics

The 18-story SMRF described in Section 4.8.1 is subjected to each of the 28 ground motion records and their equivalent pulses, and $\delta_{Rr}$ and $\delta_{Rp}$ are computed as described in Section 4.8.2.1. The performance of an equivalent pulse against the actual ground motion is quantified by an error term defined as

$$E = \frac{\delta_{Rr}}{\delta_{Rp}}$$

(4.8)
where $\delta_{R_r}$, $\delta_{R_p}$, and $E$ are computed at each story of the 18-story SMRF. A value of $E$ greater than 1 implies that the pulse is underestimating inter-story drift demand, and vice versa. Error terms corresponding to each of the 28 ground-motion records are evaluated separately, and their statistic is quantified as the mean error $\bar{E}$, which is the average value of $E$ at a given story. In addition, 15$^{th}$ and 85$^{th}$ percentile errors are also computed; they are denoted as $E_{15}$ and $E_{85}$, respectively.

### 4.8.6.3 Results

The error statistics are shown in Fig. 4.21. The vertical axis in the figure represents the relative height of the frame, while error term is plotted on the horizontal axis. The blue line corresponds to the mean error quantifying the bias introduced by the use of equivalent pulses instead of actual records. It is observed that the bias is minimum in the lower 75\% percent of the height, where the mean error is close to 1. However, the bias is high in the top 25\% of the height of the frame. At the roof, the average error is 2—meaning that the pulses underestimate the drift demands by a factor of 2. The 15 and 85 percentile errors indicate that the error scatter is small and almost constant in the lower 75\% of the height of the frame. The scatter is larger at the upper stories, where the mean error is closer to the 85$^{th}$ percentile value than the 15$^{th}$ percentile value.

![Figure 4.21: 15$^{th}$, 50$^{th}$, and 85$^{th}$ percentile errors in the prediction of inter-story drift ratios of the 18-story SMRF.](image-url)
Near-fault ground-motion records with forward-directivity pulses exhibit high spectral accelerations in a structural period band close to the pulse period. Since most attenuation equations of spectral acceleration are calibrated by using sets of ground-motion records dominated by far-fault observations, the equations blur the near-fault effects. A reliable calculation of ground-motion hazard at near-fault sites requires spectral acceleration estimates that account for near-fault effects. The methods so far used to model near-fault response spectra have not been entirely satisfactory. In this section, some of these methods are discussed, and a new model developed in Rupakheti et al. (2010b) is discussed.

### 4.9.1 Broad-band directivity model

Somerville et al. (1997) developed the first spectral attenuation model that accounted for forward-directivity effects. Based on a statistical study of the response spectra of near-fault ground motions, they proposed modification factors to decrease or increase the amplitudes of spectral acceleration obtained from attenuation models that do not account for forward-directivity effects. This model was modified later by Abrahamson (2000). This model applies modification factors monotonically with respect to increasing period and therefore fails to capture the narrow-banded nature of near-fault ground motions. The modification factors were derived from both pulse-like and non-pulse-like records, and the characteristics of the former were blurred.

### 4.9.2 Narrow-band directivity model

Tothong et al. (2007) used narrow-band modification factors to modify the spectral attenuation equations of Abrahamson and Silva (1997). Although this model is capable of capturing the narrow-banded nature of near-fault ground motions, the modification factors are not applicable in the strict sense because the equation of Abrahamson and Silva (1997) was not exclusively based on non-pulse-like records. To form a rational framework that can be used in Probabilistic Seismic Hazard Assessment (PSHA) in the near-fault area, two response spectral models—one for non-pulse-like ground motions and the other for pulse-like ground motions—are necessary. A response spectral model applicable to pulse-like ground motions in the near-fault area is presented in Rupakheti et al. (2010b). A brief discussion of the background information related to the model is presented below.
4.9.3 Ground motion characterization

In order to develop a model of the elastic response spectra of near-fault ground motions, which is simple, reliable, and easy to use in probabilistic applications, a new approach is used in Rupakhety et al. (2010b). The results and discussions presented so far in this chapter have clearly demonstrated that the pulse amplitude and pulse period are the most important characteristics of near-fault ground motions. Although simple pulse models are useful to define these parameters, such models cannot provide unique values for these parameters. Estimates of the pulse amplitude and pulse period differ from one model to another and from one user to another applying the same model. These two parameters are related to the amplitude of ground velocity and frequency content of ground motion. A unique and unambiguous measure of the amplitude and frequency content of ground motions is necessary for engineering characterization of near-fault ground motions.

4.9.3.1 Amplitude characterization

To characterize the amplitude or strength of velocity pulse, peak ground velocity (PGV) can be used. It can be directly estimated from an accelerogram and does not require a pulse model to be fitted to it. This approach was followed in Rupakhety et al. (2010b), and an attenuation model of PGV as a function of earthquake magnitude and source-to-site distance, among other variables, is also presented in the paper.

4.9.3.2 Frequency content characterization

To characterize the frequency content of near-fault ground motion, predominant period of ground motion \( T_d \) is used. It is defined as the value of \( T_n \), where the 5%-damped PSV has a clear and dominant peak. It should be noted that \( T_d \) depends on \( T_p \) and the number of zero-crossings of the velocity pulse. Since the number of zero-crossings is arbitrary to some extent and varies from one ground motion to another, \( T_d \) instead of \( T_p \) provides a more reliable estimate of frequency content. It has the additional benefit that it can be directly estimated from the elastic response spectra of a ground-motion record, which many engineers are familiar with. In this sense, \( T_d \) is more attractive to engineers as it does not require them to fit several pulses with recorded ground motions. Using this definition, it is shown in Rupakhety et al. (2010b) that \( T_d \) is proportional to the seismic moment. An equation describing the conditional probability distribution of \( T_d \) for a given value of seismic moment is also presented in Rupakhety et al. (2010b).
4.9.4 Spectral shapes

The dependence of spectral ordinates on source-to-site distance in the near-fault area is mainly due to PGV attenuation. The distance dependence can therefore be neutralized by normalizing PSV with PGV. Spectral ordinates obtained by normalizing PSV with PGV represent only the shape of a spectrum (not the amplitude), and are therefore referred to as spectral shapes ($PSV_n$) in Rupakhety et al. (2010b). To account for the magnitude dependence of $T_d$ and its effects on spectral shapes, magnitude-dependent spectral shapes are required. The spectral shapes presented in Rupakhety et al. (2010b) incorporate the effects of earthquake magnitude. In addition, viscous damping ratio is incorporated in the model as a continuous parameter. More details of the model, its important features, and its calibration are available in Rupakhety et al. (2010b).
5 Conclusions and future research

The main conclusions of the doctoral study described in this dissertation are discussed in the papers that follow; namely, Paper 1, Paper 2, Paper 3, Paper 4, and Paper 5. The methods and applications discussed in this dissertation can potentially be extended in many ways; some of the most relevant extensions are listed below.

- The baseline adjustment scheme developed during this study was specially designed for near-fault accelerograms potentially containing permanent displacements. While baseline adjustment is essential in these applications, far-fault accelerograms might also contain baseline errors and other long-period noises, that should be separated from the signal insofar as possible. The proposed method is not directly applicable in such situations due to the underlying assumption of the model: the displacement waveform resembles a ramp function. However, the proposed method can be adjusted to make baseline adjustments when permanent displacements do not occur. In particular, the selection criterion of $t_2$ needs to be modified so that the corrected displacement waveform returns to zero at the end of the record. To achieve this, different options can be tested; for instance, selecting $t_2$ to minimize the final displacement or selecting it to minimize the slope of the Fourier Amplitude Spectra of corrected velocity. These options, and possibly others, need to be studied in detail and tested with accelerograms recorded in the far-fault area.

- The GMPE models presented in Paper 1 and Paper 2 are for illustration only as they do not utilize an extensive ground-motion database. These models can be extended for use in Europe; for example, by calibrating the model with ground-motion records available from Europe, the Middle East and possibly other regions with a similar tectonic environment. The extended models can then be used in PSHA for mapping uniform-hazard inelastic response in the
The near-fault ground-motion database used in Paper 5 needs to be updated by adding more records to it. For instance, several near-fault records from the Icelandic earthquakes are not included in it. Other possible sources of near-fault ground motions, such as, New Zealand, Iran, Turkey, Canada, etc., should also be considered. The expanded database will possibly result in more robust models of PGV attenuation, pulse period, and spectral shapes. With additional data, GMPEs like the ones described in Paper 1 and Paper 2, but for inelastic response of exclusively pulse-like ground motions, can be developed. This will provide important input required for reliably mapping uniform-hazard inelastic response in the near-fault area. It will also be useful to investigate whether ground-motion time histories compatible with the spectral shapes presented in Paper 5 can be simulated. In fact, simulation of ground-motion time histories compatible with a smooth response spectrum is common in earthquake engineering practice. Nevertheless, two important issues related to it need to be investigated: (1) whether the simulated ground-motion time histories are statistically compatible with recorded near-fault ground motions and (2) whether a set of simulated and recorded near-fault ground-motion time histories results in statistically comparable responses of structures vibrating in multiple modes.

In most seismic design codes, elastic story shears are assumed to follow an inverted-triangular distribution with additional shear at the topmost story. The propagation of ground-motion pulses along the building height, however, results in unusually high shear forces in the top quarter of a tall building. It is therefore necessary to develop better representations of elastic story shear distribution in tall buildings responding to pulse-like ground motions.
A Appendix

A.1 Raw and corrected velocity time series

The raw and corrected velocity time series at different stations recording the June 2000 earthquakes in South Iceland are presented in the following pages.
Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Flagbjarnarholt
Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Kaldarholt
Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Minni–Nupur
A.1. Raw and corrected velocity time series

Component: Longitudinal

Component: Transverse

Component: Vertical

Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Hella
Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Thjorsarbru
Velocity time series; raw (grey) and corrected (dark)
Date: 17.6.2000; Station: Solheimar
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Thjorsarbru
A.1. Raw and corrected velocity time series

Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Thjorsartun
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Solheimar
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Kaldarholt
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Selfoss–Hospital
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Selfoss–City–Hall
Velocity time series; raw (grey) and corrected (dark)
Date: 21.6.2000; Station: Hella
A.2 Corrected displacement time series

The corrected displacement time series at different stations recording the June 2000 earthquakes in South Iceland are presented in the following pages.
Component: Longitudinal

Component: Transverse

Component: Vertical

Displacement time series;
Date: 17.6.2000; Station: Flagbjarnarholt
Displacement time series;
Date: 17.6.2000; Station: Kalderholt
Displacement time series;
Date: 17.6.2000; Station: Minni–Nupur
Displacement time series;
Date: 17.6.2000; Station: Hella
Component: Longitudinal

Component: Transverse

Component: Vertical

Displacement time series;
Date: 17.6.2000; Station: Thjorsarbru
Displacement time series;
Date: 17.6.2000; Station: Solheimar
Displacement time series;
Date: 21.6.2000; Station: Thjorsarbru
Displacement time series;
Date: 21.6.2000; Station: Thjorsartun
Displacement time series;
Date: 21.6.2000; Station: Solheimar
A.2. Corrected displacement time series

Displacement time series;
Date: 21.6.2000; Station: Kaldarholt
Displacement time series;
Date: 21.6.2000; Station: Selfoss–Hospital
A.2. Corrected displacement time series

Displacement time series;
Date: 21.6.2000; Station: Selfoss–City–Hall
Displacement time series;
Date: 21.6.2000; Station: Hella
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Ground-motion prediction equations (GMPEs) for inelastic response and structural behaviour factors

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Abstract The objective of this paper is to present ground-motion prediction equations describing constant-ductility inelastic spectral ordinates and structural behaviour factors. These equations are intended for application within the framework of Eurocode 8. Most of the strong-motion data used in the present work is obtained from the ISESD (Internet Site for European Strong-motion Data) databank. Present analysis includes ground motion records from significant Icelandic earthquakes, which are augmented by records obtained from continental Europe and the Middle East. In all cases the selected ground motion records are generated during shallow earthquakes within a distance of 100 km from the recording station. The classification of site conditions in the present work is based on the Eurocode 8 definition.

Keywords Eurocode 8 · Ground-motion prediction · Attenuation relationship · Structural behaviour factor · Inelastic response · Constant-ductility response spectrum

1 Introduction

Within the framework of Eurocode 8 (European Committee for Standardization 2003), the earthquake motion at a given point on the Earth’s free surface is represented by a linear elastic acceleration response spectrum popularly known as elastic response spectrum. Such a spectrum is constructed by scaling a predefined spectral shape with the peak ground acceleration corresponding to the hazard at the given site. Therefore, it is required that probabilistic seismic zoning maps displaying peak ground acceleration for an appropriate mean return period, usually taken as 475 years, are available. Such constant hazard maps are produced by performing probabilistic seismic hazard assessment (Cornell 1968).

For quantifying the seismic hazard at a given site, or for producing a seismic zoning map of a region for generalized applications, it is necessary to establish equations to calculate
various strong-motion parameters as a function of earthquake size, distance to source and the geological conditions. Acceleration time-histories, which provide the most comprehensive information about the ground motion, are not feasible to be modelled by empirical equations because their amplitude is stochastic in nature. In the past the usual practice has been to develop an attenuation relation for the peak ground acceleration, which is used to scale a normalized standard spectral shape (e.g. Biot 1942; Housner 1959; Newmark and Hall 1969; Seed et al. 1976; Mohraz 1976). McGuire (1977) pointed out the inadequacy of peak ground acceleration and a fixed spectral shape as a measure of strong motion intensity. Instead he used the maximum response of linear elastic oscillators for defining a ground motion parameter proportional to the maximum response of 1 Hz, 2% damped linear elastic oscillator which in conjunction with the peak ground acceleration was demonstrated to lead to a risk-consistent design spectra. Trifunac (1992) discussed the various factors such as the size of the earthquake source, geologic site conditions, local soil conditions and frequency dependent attenuation that actually influence the variations in spectral shapes. To avoid the use of peak ground acceleration scaled predefined spectral shape several attenuation relations have been developed, for maximum response of linear elastic oscillators at various natural periods, by many investigators in the past (Douglas 2003; Bommer 1998; Ambraseys et al. 2005; Bindi et al. 2006; Berge-Thierry et al. 2003; Cauzzi and Faccioli 2008 to mention a few).

The basis of design according to Eurocode 8 is the inelastic response spectrum of a single degree-of-freedom system exhibiting an elastic-perfectly-plastic force-deformation relation under monotonically increasing loading. Such a system is completely defined by its undamped natural period of vibration \( T \), damping ratio \( \zeta \) expressed as a percentage of critical value and the yield force \( F_y \).

Once the elastic response spectrum is developed, Eurocode 8 permits to derive the inelastic response spectrum by applying the so-called structural behaviour factor, \( q \), defined by the following ratio

\[
q = \frac{\text{max} |F_{\text{elastic}}|}{F_y} \tag{1}
\]

Here, \( F_{\text{elastic}} \) is the force that would develop if the system behaved as linear-elastic and \( F_y \) is the yield force of an inelastic system. The peak displacement demand of the structure is then expressed in terms of the ductility factor defined as follows

\[
\mu = \frac{\text{max} |u|}{u_y} \tag{2}
\]

where, \( u_y \) is the yield displacement of the system and \( u \) is the induced displacement. The use of behaviour factor permits the calculation of seismic internal forces required to design the members of structures through linear elastic analysis. In such a procedure Eurocode 8 requires that the structure possesses the capacity to sustain a peak global displacement demand at least equal to its global yield displacement multiplied by the displacement ductility factor corresponding to the \( q \) value that was used to reduce the elastic spectral ordinates when deriving the inelastic spectral values.

This procedure could be applied with higher confidence if, instead of reducing the elastic response spectrum with the so-called \( q \) factor, the designer had access to uniform hazard spectrum of inelastic response. To construct such uniform hazard spectrum within the framework of probabilistic seismic hazard assessment (PSHA), proper ground-motion prediction equations for inelastic systems need to be developed.

One of the most recent works that have been performed regarding maximum response of inelastic oscillators is that by Tothong and Cornell (2006). They developed attenuation
relationships for the ratio of inelastic spectral displacement to its elastic counterpart. These ratios were used with existing attenuation relationships for elastic spectral displacement (e.g. Abrahamson and Silva 1997) to produce the inelastic displacement spectra. Borzi et al. (2001) established some equations for computing the ratio of elastic spectral displacement to inelastic spectral displacement single degree-of-freedom oscillators. This ratio, termed as the displacement modification factor is, in displacement based design (Moehle 1992; Kowalsky et al. 1994; Priestley et al. 2007) a coefficient equivalent to the behaviour factor in force based design.

As mentioned earlier, a number of attempts have been made to develop inelastic displacement spectra. Such spectra could be easily converted into the pseudo-acceleration spectra simply by multiplying with the square of the natural frequency of vibration. Displacement spectra are useful in performance based design (Priestley 2000; Bertero 1997), structural demand analysis and displacement based design. Within the framework of Eurocode 8, acceleration spectra are the primary requirement and displacement spectra are derived from them. Additionally, we believe that deriving the inelastic spectra from the elastic spectra by using the displacement modification factors or the inelastic displacement ratios as mentioned earlier involves higher uncertainty due to the fact that apart from the uncertainty that is present in the estimation of the elastic response spectrum, an additional uncertainty is added in the final product which is a result of the regression analysis for the empirical equations of the displacement modification factors or the inelastic displacement ratios. The second source of uncertainty could be easily avoided by directly developing empirical equations for the maximum response of inelastic oscillators. This is justified by the results we present later which show that the standard deviation of residuals associated with the prediction of the elastic response and the inelastic response are comparable. In this study we focus on the task of developing GMPEs for constant-ductility response spectral ordinates.

1.1 The study area

The study area of the present work is Iceland, in particular the South Iceland Seismic Zone. The earthquakes in this zone are characterised as shallow, moderate to strong, with a predominant strike-slip faulting mechanism. The fault planes of the largest earthquakes are in all cases close to vertical and the rupture typically propagates to the surface (Sigbjörnsson et al. 2008).

Iceland lies on the diverging boundary of the North American and Eurasian plates, also known as the Mid-Atlantic Ridge. This boundary can be distinguished into regions of active tectonic extension and transformation, respectively. The transform zone of the Mid-Atlantic Ridge is associated with earthquake hazards in Iceland. Regions of the greatest earthquake hazard in Iceland can be broadly divided into the South Iceland Seismic Zone (SISZ) in the south and the Tjornes Fracture Zone (TFZ) in the north. The SISZ covers the largest agricultural area in Iceland and is among its most densely populated regions (Sigbjörnsson et al. 2008). This area has witnessed considerable earthquakes in the past. Here it is especially worth mentioning the following four events: The 6 May 1912 $M_w$ 7 earthquake, the 17 June 2000 $M_w$ 6.5 earthquake, the 21 June 2000 $M_w$ 6.4 earthquake and the 29 May 2008 $M_w$ 6.3 earthquake. The latest earthquake caused widespread damage in the densely populated region near the western border of the SISZ, including the towns of Hveragerdi and Selfoss (Sigbjörnsson et al. 2008).

The SISZ is situated between two sections of the Mid-Atlantic Ridge, i.e. the Reykjanes Peninsula, which is the inland extension of the Reykjanes Ridge, and the Eastern Volcanic
Zone. It is not an ideal transform fault, as it does not connect both rifts at right angles and as the earthquakes do not occur on EW-trending left-lateral shear faults but on their conjugate, NS-oriented right-lateral, rupture planes. This is indicated by surface fault traces and aftershock distributions.

2 Methodologies

2.1 Functional form of the model

The GMPE model for linear elastic as well as constant ductility spectral ordinates used in the present study is the one suggested by Ambraseys et al. (1996) for linear elastic system, which is represented as:

$$\log_{10}(S_a) = b_1 + b_2 M_w + b_3 \log_{10} \sqrt{d^2 + b_4^2} + b_5 S$$  (3)

Here, $S_a$ is the spectral ordinate; $b_1, b_2, b_3, b_4$ and $b_5$ are the model parameters obtained by regression analysis; $M_w$ is the moment magnitude; $d$ is the distance from the site to the surface trace of the causative fault measured in km; and $S$ is the site factor, which is taken as 0 for rock sites and 1 for stiff soil conditions. Rock sites are classified as Site Class A and stiff soil as Site Classes B and C in the Eurocode 8 provisions. The site classification is based on the average shear wave velocity in the upper 30 m of soil profile at a strain of $10^{-5}$ or less represented by the symbol $v_{s,30}$ in Eurocode 8. Site Class A is characterized by $v_{s,30} > 800$ m/s; site class B by $360 < v_{s,30} < 800$ m/s and site class C by $180 < v_{s,30} < 360$ m/s.

Many researchers have discussed the magnitude dependence of the far-field decay rate. It has been pointed out that response spectral ordinates induced by larger earthquakes decay slower than those by smaller ones and the decay rate, represented by $b_3$ in Eq. 3, of smaller sized earthquakes is faster than the commonly assumed $-1$ (Ambraseys et al. 2005). For a comprehensive review of the possible causes of decay rates faster than $-1$, readers are referred to Frankel et al. (1990). Ambraseys et al. (2005) observed that the geometric decay rate is magnitude dependent based on their analysis of records from ten earthquakes. Due to the limitation of number of data available, they assumed a linear relationship. As the present study is confined to a much smaller geographic region, less data are available and an attempt to determine the accurate dependence of the decay rate on the magnitude is not believed to produce conclusive results. Therefore, we have assumed a decay rate that is magnitude independent. Boore et al. (1997) introduced a term with quadratic dependence on magnitude in their model to account for the fact that the scaling of ground motion parameters with magnitude is different for events that rupture the entire seismogenic zone. However, Ambraseys et al. (2005) concluded that for shallow crustal earthquakes investigated by them, such an inclusion did not produce significant differences. In this work we do not consider the quadratic dependence of ground motion parameters. Furthermore, due to limitation of data it has been assumed that the anelastic decay is effectively represented by the geometrical spreading parameter, $b_3$ in Eq. 3.

The structural behaviour factor can be computed as follows

$$q(\mu, T, \zeta) = \frac{S_{\text{elastic}}(T, \zeta)}{S_{\text{inelastic}}(\mu, T, \zeta)}$$  (4)
The numerator in the right hand side of Eq. 4 is calculated from the appropriate GMPE (see Eq. 3) where the model parameters are obtained by performing regression analysis on the elastic response spectral ordinates. The denominator is computed by using the GMPE (see Eq. 3) where the model parameters are fitted to the corresponding constant-ductility response spectral ordinates.

2.2 Regression method

Equation 3 suggests that the model being used is non-linear in parameter $b_4$. Simple multiple linear regression following the method of ordinary least squares, which can be effectively used for linear models (Ambraseys and Bommer 1991), cannot be applied for non-linear models. However, it is clearly understood from the functional form of the model that for a constant value of the model parameter $b_4$, the model becomes linear and can easily be solved by standard procedure of ordinary least squares regression. The question now arises: Which value of the model parameter $b_4$ is realistic for the dataset being used? To answer this question we adopt an iterative procedure. Since the parameter $b_4$ represents a depth parameter, we know that it has a finite positive value. Therefore we start with a value of $b_4$ equal to 0 and make the model linear. The least square solution of such a model will result in a certain value of the standard deviation of the residuals. Then the value of parameter $b_4$ is increased in small increments and standard deviation of residuals calculated for each corresponding value of the parameter $b_4$. Since we are basically dealing with shallow earthquakes we increase the value of $b_4$ to 10 km. The value of $b_4$ that results in the lowest value of standard deviation of the residuals is adopted. It can easily be seen that minimizing the standard deviation of the residuals is actually equivalent to minimizing the sum of squares of the deviation of predicted response from its observed value.

3 Data applied

The strong-motion records used in the present work are listed in the Appendix, Table 1. The records are primarily those obtained from South Iceland but are augmented by data from continental Europe and the Middle East. The data for the first 76 recordings in Table 1 are obtained from the CD entitled European Strong-Motion Database, Vol. 2 (Ambraseys et al. 2004b). The data for recordings 76–82 in Table 1 are obtained from the Internet Site for European Strong-Motion Data (ISESD) databank (Ambraseys et al. 2004a). Records 83–93 in Table 1 were generated during the 29 May 2008 Iceland earthquake. The corrected acceleration time histories for these records were obtained from the ICEARRAY strong-motion array. ICEARRAY is a small-aperture strong-motion array in South Iceland consisting of 15 strong-motion recording stations situated in the town of Hveragerdi (Sigbjörnsson et al. 2008). A total of 186 strong-motion acceleration time histories (two horizontal components for each of the 93 recording stations listed in Table 1) have been used in the present work.

3.1 Magnitude

The magnitude scale used is the moment magnitude $M_w$ proposed by Kanamori (1977). The records are generated by earthquakes varying in magnitude from 5 to 7.7.
3.2 Source-to-site distance

The distance parameter used in the present work is the distance to the surface projection of the fault (Joyner and Boore 1981), commonly known as fault distance. In cases where the fault distance is not available, epicentral distance is used instead. These distances were obtained from Ambraseys et al. (2004a,b) except for the ICEARRAY recordings. For the ICEARRAY recordings, epicentral distances were computed based on the macroseismic epicentre of 63.98°N and 21.13°W (Sigbjörnsson et al. 2008). The records show a variation in fault distance from 1 to 97 km. Recordings further away from the fault are excluded as they are believed to be of low engineering significance. Such an approach also reduces the difference in anelastic decay in different regions of Europe and the Middle East (Ambraseys et al. 2005).

3.3 Faulting mechanism

Most of the records are generated during strike slip earthquakes except for the recordings from the 1998 Iceland earthquake (records 70–76 in Table 1). Although this earthquake is characterized with oblique faulting mechanism, the fault plane is nearly vertical and the relative motion of the fault planes is predominantly in the direction of the strike.

3.4 Building type

It is believed that the ground motion characteristics of strong-motion records are altered by properties of large buildings, where the recording instruments are commonly stored. Due to limited availability of data, such records were not omitted. However, records from the June 2000 Iceland earthquake, and 29 May 2008 Iceland earthquake which were recorded at the Thjorsarbru Bridge, have been omitted because they show distinct structural influences and site dependent conditions not characteristic of the study area as a whole.

3.5 Site conditions

The criteria based on classifying the site are within the framework of Eurocode 8. Only those sites that are classified as Site Class A, B or C are included in the analysis. Soft soil data have been neglected because they are not characteristic of the study area. Although the recordings from the December 1990 Armenia earthquake satisfy all of the criterion for record selection discussed above, they have been excluded due to the fact that they are characterized by non-extensional region which is not characteristic of the study area.

3.6 Data correction

The acceleration time histories available in Ambraseys et al. (2004a,b) were obtained in corrected form. Data obtained from the ICEARRAY recordings were corrected by using band pass filters with individually chosen cut-off frequencies.

3.7 Spectral amplitudes

The elastic response spectra are computed from the corrected time histories corresponding to 66 log-spaced undamped natural periods in the range 0.04–2.5 s. Inelastic spectra
are computed assuming an elastic-perfectly-plastic force-displacement relation of a single degree-of-freedom oscillator characterized by an undamped natural period, ductility factor and a damping ratio of 5% of the critical. The computations are performed using the software Bispec version 1.61 (Hachem 2000, 2008) and reconfirmed with the results obtained from the software SeismoSignal (SeismoSignal 2007). The time step of integration was set to the minimum of the sampling time of the acceleration time history and the natural period divided by 20. Out of the spectral acceleration corresponding to the two horizontal components, the larger value is selected.

4 Results

The results of the regression analysis are displayed in Fig. 1. The values of the model parameters and the standard deviation of the residuals are listed in Appendix Tables 2, 3, 4, 5 for selected natural periods. It is seen that the estimated model parameters are not smooth functions of the undamped natural period, but apparently possess some irregularities or randomness. The standard deviation of the residuals obtained for the model is comparable to what has been reported in the literature for elastic response spectral ordinates (see Douglas 2003).

Figure 2 shows the comparison of the model with the observed peak ground acceleration (PGA). Figures 3 and 4 show the comparison of the capacity demand for elastic as well as inelastic structures computed by the proposed model to the observed values for natural periods of 0.2 and 1.0 s respectively. Careful examination of Figs. 2 and 3 shows that the scatter of the observed capacity demand around the predicted values is comparable for linear
Fig. 2  Comparison of the proposed model with the observed PGA values. The solid blue line corresponds to the result of the proposed GMPE and the red circles are the observed values. Note that the vertical axis has been normalized.

\[ b_3 \log_{10} \sqrt{d^2 + b_4^2} \]

\[ S_a \] -b_1-b_2 M_w-b_5 S

Fig. 3  Comparison of the capacity demand estimated by the proposed model to the observed values for natural period of 0.2 s and damping 5% of critical. The solid blue line corresponds to the result of the proposed GMPE and the red circles are the observed values. Note that the vertical axis has been normalized.
elastic structures and for inelastic structures having ductilities 1.5, 2 and 4. This justifies the assumption that the functional form of Eq. 3 proposed by Ambraseys et al. (1996) for linear elastic systems is equally valid for inelastic structures too.

Selected spectral ordinates representing capacity demand are plotted in Fig. 5 as a function of fault distance. The capacity demand decreases with increasing distance as expected. Furthermore, stiff soil requires a higher capacity demand than rock site and so do stiff structures ($T = 0.2$ s) compared to the more flexible ones ($T = 1.0$ s). The corresponding structural behaviour factors are presented in Fig. 6. It is seen that the ductility value, equal to 1.5, yields a structural behaviour factor almost equal to 1.5, which is in accordance with Eurocode 8. On the other hand, the structural behaviour factor tends to be smaller for stiff structures ($T = 0.2$ s) than anticipated by Eurocode 8, e.g. for a ductility factor equal to 4.0 the structural behaviour factor varies from 2.5 to 3.5 and for a ductility factor equal to 2, the structural behaviour factor varies from 1.75 to 2.25. For a flexible structure ($T = 1.0$ s), and ductility factors of
1.5 and 2, structural behaviour factors are higher than the Eurocode 8 recommendations; however, for a ductility factor of 4, the structural behaviour factor is smaller than 4.

Figure 7 shows the elastic/inelastic spectra and the structural behaviour factors for a moment magnitude 6.5 earthquake at a distance of 1 km from the fault. It is evident from Fig. 7 that the spectra are jagged, reflecting the irregularities observed in the estimates of the model parameters (see Fig. 1). This is an undesirable feature in general and a smoother spectrum is more attractive for practical purposes. To achieve a smoother spectrum, two approaches were tested. In the first approach the model parameters were smoothed using the Savitzky-Golay procedure (Savitzky and Golay 1964) with a span of 19 and a polynomial of second degree. For the smoothed parameters, the standard deviation of the residuals was recalculated. The results are presented in Fig. 8. In the second approach, the inelastic spectrum itself was smoothed by using a similar procedure (see Fig. 9). It is found desirable to have the model parameters smoothed rather than to smooth the spectrum for each magnitude or distance to the fault. In such a way, the GMPE can be directly used to obtain a reasonably...
Fig. 6  Structural behaviour factor as a function of distance to fault. Moment magnitude is kept equal to 6.5. Unsmoothed model parameters presented in Fig. 1 are used in producing these figures. The solid lines represent the results of the present study and the dashed lines are the Eurocode 8 recommendations.

smooth spectrum. On the other hand, it is necessary to verify that the smoothing of model parameters does not disturb the inherent correlation between the model parameters (if any). To explore this we have calculated the correlation matrix of the original unsmoothed parameters obtained by regression and compared with the correlation matrix of the parameters obtained after smoothing. For elastic structures the correlation matrices before and after smoothing are respectively equal to

\[
\begin{bmatrix}
1 & -1 & 0.33 & 0.86 & 0.92 \\
-1 & 1 & -0.34 & -0.84 & -0.9 \\
0.33 & -0.34 & 1 & 0.02 & 0.27 \\
0.86 & -0.84 & 0.02 & 1 & 0.81 \\
0.92 & -0.9 & 0.27 & 0.81 & 1
\end{bmatrix}
\]

and

\[
\begin{bmatrix}
1 & -1 & 0.33 & 0.86 & 0.92 \\
-1 & 1 & -0.34 & -0.84 & -0.9 \\
0.33 & -0.34 & 1 & 0.02 & 0.27 \\
0.86 & -0.84 & 0.02 & 1 & 0.81 \\
0.92 & -0.9 & 0.27 & 0.81 & 1
\end{bmatrix}
\]
It can be observed that the correlation between the model parameters is not greatly altered by the smoothing procedure. Similar behaviour was observed for ductile inelastic structures. It can be seen by comparing the standard deviation of residuals from Figs. 1f and 8f that the smoothing procedure does not result in any significant change in the standard deviation of the residuals. Figure 9 presents the comparison of the spectra computed by using the smoothed parameters of Fig. 8 with the Eurocode 8 recommendations.

$$
\begin{bmatrix}
1 & -1 & 0.34 & 0.89 & 0.92 \\
-1 & 1 & -0.35 & -0.87 & -0.91 \\
0.34 & -0.35 & 1 & 0.03 & 0.29 \\
0.89 & -0.87 & 0.03 & 1 & 0.83 \\
0.92 & -0.91 & 0.29 & 0.83 & 1
\end{bmatrix}
$$

Fig. 7 Inelastic spectrum and structural behaviour factor for a magnitude 6.5 earthquake, at a site where distance to fault is 1 km. Unsmoothed model parameters presented in Fig. 1 are used to produce these figures. a and b correspond to the rock site and c and d correspond to the stiff soil site. The solid lines are results of the proposed GMPEs and the dashed lines are the Eurocode 8 recommendations.
5 Discussion and conclusions

The correlation matrix of the model parameters indicates that model parameters are highly correlated. This reflects a strong multi-collinearity; a condition that implies imprecise estimates of the regression coefficients. However, the fitted model may still be useful. In such cases, interpolations in the original space of the predictors are satisfactory and the standard deviation of the residuals is reasonably small, which is the case as seen in Figs. 1f and 8f. This implies that while the individual parameters may be poorly estimated, the function represented by the model is estimated fairly accurately. However, it should be pointed out that extrapolation of the model outside the data region of independent variables, i.e. magnitude and distance, used in this study could possibly lead to unreliable results.

We have observed that smoothing of the regression parameters in a model as such is found to be desirable. It is noticed that smoothing of the model parameters, despite of not causing any significant changes in the standard deviation of the residuals, resulted in a fairly smooth spectrum.

It is observed by investigating the Appendix Table 2 (see also Fig. 1) that the parameter $b_3$ is in the range $-1.23$ to $-1.06$ for linear elastic structures. Furthermore, for inelastic systems it is observed that the $b_3$ parameter is in a similar range but with a tendency towards decreasing attenuation with increasing ductility (see Appendix Table 3, Table 4 and Table 5). In general it is found that the attenuation revealed by this study is comparable with the values obtained by Ambraseys et al. (2005) for magnitude values in the range 6–6.5, but contradicts the fast attenuation indicated by studies dealing with individual earthquakes in the South Iceland Seismic Zone (Sigbjörnsson and Ólafsson 2004; Sigbjörnsson et al. 2007, 2008). However, a regression procedure similar to that used in this present study when applied to the 17 records generated during the 29 May 2008 earthquake resulted in the values of the parameters $b_3$ and $b_4$ equal to $-1.87$ and 6.4, respectively. These observations
Fig. 9 Comparison of the spectra computed by using the smoothed model parameters of Fig. 8 with the Eurocode 8 recommendations. The solid lines represent the results of the present work and the dashed lines represent the Eurocode 8 recommendations. a and b are for rock site and c and d are for stiff soil sites. The moment magnitude is taken equal to 6.5 and the distance to the fault is 1 km.

are in close agreement to the depth parameter of 6.2 km and attenuation proportional to the inverse of the squared distance as reported by Sighjörnsson et al. (2008). This indicates a faster attenuation of the ground motion in the South Iceland region, however, a definite conclusion based on this study alone is not feasible due to the limited number of records available.

The comparison of the spectra computed by the proposed model and the recommendations of Eurocode 8, which is shown in Fig. 7, is interesting to analyze. It should be noted that the Eurocode 8 spectra for inelastic systems are computed based on the assumption that the structural behaviour factor is equal to the ductility factor. It is observed that for elastic structures on rock sites the Eurocode 8 recommendations are conservative for structures having periods less than about 0.6 seconds. However, for flexible structures on rock sites, Eurocode 8 recommendation is seen to under-estimate the spectral ordinates even for elastic structures. This illustrates the limitations of a fixed spectral shape scaled to the peak ground acceleration. For structures designed for target ductilities of 1.5 and 2, the Eurocode 8 recommendations are in good match with the results of the present work especially for stiff structures. How-
ever, for structures with design ductility of 4, the Eurocode 8 recommendation seems to under-estimate the capacity demand on structures located at rock sites for almost whole spectral range considered in the present work.

For stiff soil sites which are classified as site class B and C in Eurocode 8, the soil factor of 1.2 recommended for site Class B is adopted. It is observed that Eurocode 8 recommendations for stiff sites tend to over-estimate the spectral accelerations for structures with higher lateral strength and stiffness. However, as the design ductility is increased to 4, the Eurocode recommendations are under-estimating the capacity demand especially for structures having natural period less than 0.5 s. Another observation worth noticing is that the assumption of adopting structural behaviour factor recommended in Eurocode 8 provisions is not conservative, especially for stiff structures designed for higher values of ductility factors. We observe from Fig. 6 that the structural behaviour factor does not depend significantly on source to site distance. However, it is observed that Eurocode 8 recommendations seem to yield higher values of structural behaviour factor, especially for stiff structures located close to the fault for relatively strong structures. For stiff structures, and a ductility factor of 4, Eurocode 8 recommendations seem to significantly overestimate structural behaviour factor. This implies that stiff structures close to the fault and structures designed assuming higher values of ductility factor will possess inadequate lateral strength. Based on these observations we conclude that the structural behaviour factors to be used in the study area can deviate significantly from the code provisions.

Examination of the standard deviation of residuals in Fig. 1 clearly reveals that the uncertainty involved in prediction of inelastic response is for all practical purposes similar to that involved in the prediction of linear elastic response. This observation forms a strong basis for the development of GMPEs as those proposed in this study for inelastic response. We believe that incorporation of such equations in the framework of PSHA can result in more accurate prediction of peak response of inelastic structures.

Acknowledgements The authors acknowledge the support from the University of Iceland Research Fund. Furthermore we are thankful to the reviewers for constructive criticism. Especially, the authors thank Dr. John Douglas for many comments that resulted in several improvements.

Appendix

Table 1 The earthquake data set used in the present study (WID represents waveform identification within ISESD databank and station identification for ICEARRAY)

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DATA SOURCE: ISESD website (Ambraseys et al. 2004a)

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Table 3  Model parameters for inelastic system with ductility 1.5

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Table 4 Model parameters for inelastic system with ductility 2

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Table 5 Model parameters for inelastic system with ductility 4

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Paper 2

Ground-Motion Prediction Equations (GMPEs) for inelastic displacement and ductility demands of constant-strength SDOF systems

R. Rupakhety · R. Sigbjörnsson

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Abstract The objective of this paper is to present ground-motion prediction equations for ductility demand and inelastic spectral displacement of constant-strength perfectly elasto-plastic single-degree-of-freedom (SDOF) oscillators. Empirical equations have been developed to compute the ductility demand as a function of two earthquake parameters; moment magnitude, and source-to-site distance; one site parameter, the ground type; and three oscillator parameters, an undamped natural period, critical damping ratio, and the mass-normalized yield strength. In addition, a comparative study of the proposed model with selected previous studies and recommendations of Eurocode 8 is presented. Proposed equations can easily be incorporated in existing probabilistic seismic hazard analysis (PSHA) software packages with the introduction of an additional parameter. This leads to hazard curves for inelastic spectral displacement, which can provide better estimates of target displacement for nonlinear static procedures and an efficient intensity measure for probabilistic seismic demand analysis (PSDA). Proposed equations will be useful in performance evaluation of existing structures.

Keywords Eurocode 8 · Ground-motion prediction · Attenuation relationship · Yield displacement · Ductility demand · Inelastic response · Response spectrum

1 Introduction

Many current seismic provisions permit seismic design of structures on the basis of either static or dynamic analyses of elastic models. Seismic forces applied to such models are often defined in terms of an elastic acceleration spectrum, which represents peak seismic actions on elastic structures at a certain damping level, usually taken at 5% of critical damping. The strong-motion intensity level representative of these design spectra is severe, commonly
characterized by a mean return period of 475 years. Designing structures to remain elastic at these levels can be uneconomical, especially in areas characterized by high seismicity. Therefore, structures are expected to undergo a certain level of inelastic deformations, thereby reducing the seismic forces they should be designed for. To accomplish this, some factors are introduced to reduce the elastic spectra to inelastic/design spectra. In Eurocode 8 (European Committee for Standardization 2003), this quantity is termed structural behaviour factor ($q$).

The concept of inelastic response spectrum in earthquake engineering dates back to the 1960s when Veletsos and Newmark (1960), for the first time, studied the response of elasto-plastic SDOF systems subjected to earthquake ground-motions. Based on these studies, Newmark and co-workers developed a procedure for constructing an inelastic spectrum from the elastic one (see for instance Newmark and Hall 1982). They divided the elastic response spectrum into different regions, each associated with a certain reduction factor, depending on the system properties, and a design ductility factor. Several researchers in the past have studied, in one way or another, the procedure to construct inelastic strength and/or deformation spectrum from the elastic one (e.g., Riddell and Newmark 1979; Iwan 1980; Elghadamsi and Moraz 1987; Nassar and Krawinkler 1991; Miranda 1993; Miranda and Bertero 1994; Vidic et al. 1994; Ordaz and Pérez-Rocha 1998; Borzi and Elnashai 2000; Riddell et al. 2002; Chopra and Chintanapakdee 2004; Tiwari and Gupta 2000; Farrow and Kurama 2004).

Results of previous research, including those mentioned above, show that these reduction factors depend on parameters such as the natural vibration period, ductility factor, local site conditions, earthquake magnitude, distance of site from the source and the type of force–deformation relationship assumed. However, structural behaviour factors recommended in Eurocode 8 are highly simplified and depend on ductility factor and natural period. Such simplification may result in inaccurate estimation of seismic design forces. To overcome this shortcoming, Ground-Motion Prediction Equations (GMPEs) have been proposed to compute inelastic design spectra, without resorting first to elastic response spectra (Rupakheti and Sigbjörnsson 2009). Such equations, when incorporated into the framework of probabilistic seismic hazard assessment (PSHA), are useful in constructing uniform-hazard inelastic (design) spectra.

In force based design, the primary quantity of interest to the designer is the design acceleration spectrum. After a system is designed for adequate strength, it has to be ensured that ductility demand of the structure under the application of design forces does not exceed the ductility factor assumed in design process. Therefore, ductility demand of a structure is an important parameter for design verification. It has been observed that elastic displacement does not correlate well with the inelastic response of structures (see Luco et al. 2005 and references therein). On the other hand, damage experienced by a structure and its elements correlates well with ductility demand, which means that ductility demand is a useful quantity for estimating structural performance and assessing risk.

The peak lateral displacement demands of building structures are often estimated by nonlinear static procedures (FEMA 356 2000; ATC 40 1996). The underlying idea behind these procedures is to convert a structure into an equivalent SDOF system. Such approaches form the basis of the so-called capacity-spectrum method and the displacement coefficient method (Reinhorn 1997; Chopra and Goel 1999; Tena-Colunga 2001). These methods make use of simplified expressions to compute peak displacement demands of SDOF systems, often treated like idealized representation of a real structure. Displacement-based approaches for seismic design of structures are also, to some extent, based on the estimation of peak displacement demands of SDOF oscillators. In fact, peak relative displacement is one of the most commonly used parameter to evaluate performance of structures responding in the inelastic domain (see for e.g., Moehle 1992; Kowalsky et al. 1995; Fajfar 2000; Borzi et al. 2001).
should be noted that, for elasto-plastic SDOF systems, peak relative displacement demand is commonly calculated by multiplying the ductility demand with the yield displacement, which in turn is completely defined by the strength and initial elastic stiffness of the SDOF.

Three different approaches, for computing peak relative displacements of SDOF systems, can be found in the literature. In the first approach, yield displacement is obtained by dividing the peak elastic displacement by the so-called force reduction factor. Then, peak inelastic displacement is obtained multiplying yield displacement by ductility factor. In the second approach, also known as displacement modification procedure, peak displacement of an inelastic system is calculated by multiplying the peak displacement of a linear elastic system by a displacement modification factor. Such displacement modification factors are dependent on the undamped natural period of the system and the level of expected nonlinearity (see for e.g., Newmark and Hall 1982; Ruiz-García and Miranda 2003; Goel and Chopra 2001; Decanini et al. 2003; Tothong and Cornell 2006). In the third approach, also known as equivalent linearization, an inelastic system is represented by an equivalent linear elastic system having lower stiffness and higher damping ratio (see for e.g., Rosenblueth and Herrera 1964; Gulkan and Sozen 1974; Iwan 2002; Kwan and Billington 2003). A comprehensive review of such procedures is beyond the scope of this work and interested readers are referred to Akkar and Miranda (2005).

One common feature in these procedures is: they resort to pre-specified elastic response spectra to compute elastic displacement demand. Alternatively, elastic displacement demand is computed by using empirical relationships such as those proposed by Bommer and Elnashai (1999). Yet another approach that is commonly used is adopting the mean elastic spectral displacement obtained by time history analyses over an ensemble of recorded accelerograms, scaled to a common level of certain ground motion intensity, such as peak ground acceleration (PGA). PGA in such methods is commonly estimated by elastic attenuation models (for a comprehensive review see Douglas 2003). The practice of using empirical models to convert mean elastic displacement to mean inelastic displacement involves combination of two basic uncertainties. The first source of uncertainty lies in the estimation of elastic spectral displacement itself (or the PGA). The second source of uncertainty lies in the model that relates inelastic displacement demand of an ensemble of ground motion records to the elastic counterpart. Jarenprasert et al. (2006) developed empirical equations for computing inelastic spectra independent of the elastic one. Yi et al. (2007) developed methods to compute probabilistic displacement response spectra based on time history analyses of a set of ground motion records. However, in these analyses, ground motion records were scaled to a common level of PGA. Therefore the empirical models developed assumed PGA a priori. Bozorgnia et al. (2006) proposed attenuation models for constant ductility and constant damage inelastic spectra and correlated inelastic response to earthquake magnitude, fault distance, fault mechanism, local soil properties among other properties. Unlike their model, which finds its application in design process, proposed model is intended for evaluation of existing structures.

In the present work, an attempt is made to eliminate the source of uncertainty described as the second source in the preceding paragraph. We present Ground-Motion Prediction Equations (GMPEs) to estimate ductility demand (and inelastic displacement for predefined yield strength) without having to resort to any sort of elastic spectra. Our results show that inelastic displacement demand of an elastoplastic SDOF oscillator can be estimated at the same level of accuracy as the elastic displacement demand. The drawback of many of the two-step procedures described above is that they do not consider the magnitude dependence of inelastic displacement ratio (IDR), which is defined as the ratio of peak inelastic displacement to peak elastic displacement. This means that although such models are useful for
scenario based computations, they can not be directly applied within a probabilistic framework. Tothong and Cornell (2006) have demonstrated that IDR depends on earthquake magnitude. In the proposed model, magnitude dependence is implicit as magnitude is one of the independent parameters of the model. Most of the empirical models mentioned above are based on response of constant-ductility oscillators which is useful for specifying design forces (Rupakhetty and Sígbjörnsson 2009). On the other hand, the goal behind seismic evaluation and/or design verification is not to achieve a specified level of ductility but rather to evaluate the ductility demand of oscillators with given strength, stiffness and energy dissipation properties. It should be noted that using IDR derived from constant-ductility oscillators under-estimates the inelastic displacement demand of systems with given lateral strength due to the fact that, in constant-ductility approach, inelastic displacement is constrained to achieve specified target ductility. Therefore, inelastic displacement demand should be treated as a random variable, as in the proposed model. Moreover, the common practice of using force reduction factors \((R)\) or structural behaviour factors \((q)\) to relate elastic and inelastic displacement demands assume that \(R \) (or \(q \)) is a priori. This contradicts reality because \(R \) (or \(q \)) depends on the elastic spectral displacement \((S_{dd})\), which is a random quantity. Therefore we believe that the mass-normalized yield strength, \(C_y\), provides a better representation of oscillator strength, especially, because it depends solely on the properties of the system and can be estimated, in practical applications, through static pushover analysis. This explains the difference of the proposed constant-strength model for structural performance evaluation, from an earlier constant-ductility model (Rupakhetty and Sígbjörnsson 2009), which was indented for designing structures for a specified target ductility.

Based on these observations, we propose an empirical model for estimating the ductility demand, \(\mu = S_{dd}/u_y\), where \(u_y = (4\pi^2C_y)/(T^2)\) is the yield displacement of the oscillator. The ductility demand proposed here is assumed a function of two earthquake parameters; moment magnitude \((M_w)\), and source-to-site distance \((d)\); one site parameter, the ground type; and three oscillator parameters, an undamped natural period \((T)\), critical damping ratio \((\zeta)\), and the mass-normalized yield strength \((C_y)\). Definition of ground types in this study is based on Eurocode 8 recommendations. Results presented in the following sections provide an efficient intensity measure for probabilistic seismic demand analysis (PSDA), in terms of displacements, and are useful for performance-evaluation and design-verification of existing and design-specified structures respectively.

2 Methodologies

2.1 System configuration

The structural system considered is an SDOF oscillator, completely described by \(T\), \(\zeta\) and \(C_y\), which is defined as:

\[
C_y = \frac{F_y}{m}
\]

Here, \(F_y\) is the yield force of the system and \(m\) is its mass. The force–deformation relation of the system is elastic-perfectly-plastic and damping ratio is taken 5% of critical damping. The governing equation of motion for the system subjected to ground acceleration described by \(\ddot{u}_g(t)\) can be written as:

\[
\dddot{u}(t) + 2\zeta \omega \dot{u}(t) + C_y \ddot{z}(t) = -\ddot{u}_g(t)
\]
Here, $\omega = 2\pi / T$, is the angular frequency of undamped natural vibration; single dot ($\cdot$) and double dots ($\ddot{\cdot}$), on top of a symbol, imply first and second derivatives, respectively, with respect to time. Moreover $\dot{z}(t)$ is given as:

$$
\dot{z}(t) = \left\{ 1 - \left[ 1 - H(-\dot{u}(t)) \right] \left[ H(z(t) - 1) \right] - \left[ 1 - H(\dot{u}(t)) \right] \left[ H(-z(t) - 1) \right] \right\} \frac{\omega^2 \dot{u}(t)}{C_y}
$$

(3)

In Eqs. 2 and 3, $u(t)$ is the relative displacement, $\dot{u}(t)$ is the relative velocity, $\ddot{u}(t)$ is the relative acceleration and $H(\cdot)$ is the Heaviside step function defined herein as:

$$
H(x) = \begin{cases} 
1 & \text{for } x \geq 0 \\
0 & \text{for } x < 0
\end{cases}
$$

(4)

The sets of differential equations, Eqs. 2 and 3, can be solved numerically to evaluate the displacement response history ($u(t)$). Then ductility demand can be obtained as follows:

$$
\mu = \frac{\omega^2 \max_t |u(t)|}{C_y}
$$

(5)

Runge-Kutta method (see Boyce and DiPrima 2000; Zarowski 2004) along with state-space formulation was used to solve these equations numerically.

2.2 Data applied

The study area of this work is Iceland, in particular the South Iceland Seismic Zone. The seismic activity in this zone is characterized by shallow, moderate to strong, strike-slip earthquakes. The fault planes of the largest earthquakes are, in all cases, close to vertical and the rupture propagates to the surface in most cases (Sigbjörnsson et al. 2008; Rupakhety and Sigbjörnsson 2009).

The strong-motion records used in this work are obtained from South Iceland, but are augmented by data from continental Europe and the Middle East. Most of these data are obtained from European Strong Motion Database (Ambraseys et al. 2004a,b). A total of 186 strong motion acceleration time histories (two horizontal components from 93 recording stations) have been used. Of the two horizontal components, the one which produces larger ductility demand is used in the regression analysis. This is not necessarily the component with the highest PGA. A list of ground motion records, the details of their processing and correction procedure, site class identification, building types, etc. can be found in Rupakhety and Sigbjörnsson (2009).

2.3 Regression model

The proposed model is based on the assumption that the functional form of ground motion prediction equation used for spectral acceleration of linear elastic systems (see Ambraseys et al. 1996) can be applied to ductility demand of constant-strength systems as well. This model is the same one as applied by Rupakhety and Sigbjörnsson (2009) in a previous study. The mathematical representation of proposed model is given in the following equation:

$$
\log_{10}(\mu) = b_1 + b_2 M_w + b_3 \log_{10} \sqrt{d^2 + b_4^2} + b_5 S
$$

(6)

where, $\mu$ is the ductility demand; $b_1, b_2, b_3, b_4$ (units of km), and $b_5$ are model parameters obtained by regression analysis; $M_w$ is the moment magnitude; $d$ is the distance from site to
the surface projection of causative fault measured in km; and $S$ is a site factor, which is taken as 0 for rock sites and 1 for stiff soil conditions. Rock sites are equivalent to ground type A, and stiff soil to ground types B and C in Eurocode 8 provisions. For detailed discussion on various aspects of the proposed model, readers are referred to Rupakhety and Sigbjörnsson (2009) and references therein. The regression procedure used here is similar to the one used by Ambraseys et al. (1996), which is briefly explained in Rupakhety and Sigbjörnsson (2009).

3 Results and discussion

3.1 Proposed model

Figure 1 displays the plots of regression model parameters and standard deviation of the residuals as a function of undamped natural period for four values of mass-normalized yield strength. The standard deviation of residuals is in the order of 0.3, which is comparable to what has been reported in literature for elastic response spectral ordinates [for a comprehensive overview see Douglas (2003)]. This indicates that inelastic displacement demands can be estimated at the same level of accuracy as their elastic counterparts. However, for natural periods less than 0.25 s and normalized yield strength of 0.5 m/s$^2$, the standard deviation of residuals are as high as 0.65. This happens due to the fact that, as the natural period approaches 0, system stiffness approaches infinity, implying that the yield displacement approaches 0. Therefore, ductility demand approaches infinity, implying that ductility for extremely stiff structures is not defined. Therefore, such high values of standard deviation at lower periods should not be interpreted as sources of inaccuracy in the proposed model. This is because real structures are designed to have limited ductility and extremely high values of ductility are not of practical interest.

Figure 2 presents the variation of ductility demand, for a given earthquake magnitude, as a function of distance to the surface projection of the causative fault and normalized

\begin{figure}[ht]
\centering
\includegraphics[width=\textwidth]{figure1.png}
\caption{Fig. 1 Regression model parameters and the corresponding standard deviation of residuals. Critical damping ratio is 5% in all cases.}
\end{figure}
yield strength. The information presented in these figures should be interpreted with careful considerations of the physical meaning of ductility. For example, ductility values below 1 are not shown because they imply elastic response. Moreover, low strength structures located very close to the fault are exposed to extreme ductility values, as high as 1,000, in the green curve in Fig. 2c. The high values of ductility do not imply that the structures need to be designed for such high ductility, which would be practically impossible, due to the fact that ductility capacity of a structure is limited by material properties and high ductility is undesirable due to serviceability considerations. Most seismic codes allow a maximum design ductility of 8. Therefore, the high values of ductility should be interpreted as demand that is hypothetically imposed on the structure. On the other hand, the usefulness of these curves lies in deciding whether the ductility capacity of a given structure is higher than the ductility demand predicted for a given magnitude, source-to-site distance, site conditions and the normalized strength of the system. As expected, structures on stiff soil foundations are exposed to higher ductility demands than those on rock foundations. The amplification in ductility demand for stiff soil conditions compared to rock site conditions is found to be more pronounced for stiffer structures than for more flexible ones.
Figure 3 shows the ductility demand spectra for a magnitude 6.5 earthquake at a site very close to the fault (3 km distance). It can be observed that the spectra for systems having lower strength are smoother than those for structures having higher strengths. This is due to the fact that inelastic spectra are smoother than elastic spectra by nature. Only those values of ductility that lie in the range of 1–10 are displayed because ductilities outside this range are of little practical interest. From Fig. 3a, it is observed that, for a moment magnitude 6.5 event originating at 3 km distance from a structure located at rock sites, all structures having undamped natural period larger than 1.5 s and normalized yield strength of 5 m/s², behave as linear elastic systems. However, structures having lower strength exhibit inelastic responses, in the spectral range considered, and are subjected to increasing ductility demands with decreasing strength. These curves also provide a convenient method to select a combination of stiffness and strength of the system so as to limit the ductility demand to a certain level. This is illustrated schematically in Fig. 3a. The intersections of constant-ductility line (6 in this case), with the red, the blue and the green curve, provide, for corresponding strength, the minimum period a structure should possess so that the ductility demand imposed on it is limited to 6. In this case, structures with normalized yield strengths of 2, 1 and 0.5 m/s² will experience a ductility demand less than 6 if their undamped periods of vibration are greater than 0.54, 1.1 and 1.9 s, respectively.

Figures 4 and 5, show the variation of ductility demand as a function of undamped natural period and distance to the fault for different site conditions. It can be observed from Fig. 4 that systems with normalized strength of 0.5 m/s² are exposed to ductility demands as high as 10 when the distance to the site is less than 30 km. As expected, increasing the strength of the system brings the contour lines of Figs. 4 and 5 closer to the origin of the applied coordinate systems, which indicates that the minimum value, of natural period and the distance from the source, for the structure to remain elastic, decreases as the strength of the system is increased. Comparison of Figs. 4 and 5 shows that structures on stiff soils experience higher ductility demands than those on rock sites.

For practical applications, smoother spectra are more desirable. Savitzky and Golay (1964) method was used for smoothing the model parameters [see Rupakhety and Sigbjörnsson...
Fig. 4 Variation of ductility demand as a function of natural period and source-to-site distance for a moment magnitude 6.5 event and rock site conditions.

(2009). Selected values of the smoothed model parameters for a sample of natural periods are presented in Appendix Tables 1, 2, 3 and 4.

To investigate whether or not smoothing disturbs the inherent correlation between model parameters, we computed correlation matrices of original as well as smoothed parameters. It was observed that the differences between the corresponding elements of the correlation matrices of original and smoothed parameters are in the order of 1%. This makes us conclude that the correlation between the model parameters is not significantly altered by smoothing procedure. Figure 6 presents the smoothed model parameters as a function of undamped natural period and normalized strength.

It is also worth noting the difference between the ductility demand spectra computed from smoothed and original parameters. Such a comparison is presented in Fig. 7. It can be observed that smoothing of model parameters does not result in any significant difference in spectral ductility demands. Comparison of Figs. 1f and 6f shows that the statistical property of the residuals is practically unaltered by the smoothing process.
Fig. 5  Variation of ductility demand as a function of natural period and source-to-site distance for a moment magnitude 6.5 event and stiff soil conditions

It is worthwhile to compare the suggested GMPEs for inelastic response of constant strength systems to some of the models found in the literature. In the following three different models are considered.

3.2 Recommendations of Eurocode 8

Eurocode 8 has adopted the inelastic spectra proposed by Vidic et al. (1994). The relationship between structural behaviour factor (which is equivalent to force reduction factor, \( R \)) is expressed by following equations:

\[
\mu = q \text{ for } T \geq T_C \\
\mu = 1 + (q - 1) \frac{T_C}{T} \text{ for } T < T_C
\]
Fig. 6 Model parameters obtained after smoothing. Savitzky and Golay (1964) procedure with a span of 29 and a polynomial of second degree was used for smoothing the parameters. Standard deviation of the residuals was recalculated for smoothed parameters.

Fig. 7 Comparison of ductility spectra calculated using unsmoothed model parameters displayed in Fig. 1 (solid lines) with those calculated using smoothed model parameters of Fig. 6 (dotted lines).

Here $T_C$ is the transition period of elastic response spectrum between its constant spectral pseudo-acceleration and constant spectral pseudo-velocity regions. Two different values of $T_C$ are applied, respectively, corresponding to Type 1 and Type 2 Eurocode 8 spectra. Equation 7 expresses the well-known 'Newmark’s equal displacement rule', i.e. the empirical observation that in the constant pseudo-velocity range, peak displacement response of inelastic and of elastic SDOF systems are approximately the same.
The inelastic response spectrum is derived by dividing the elastic response spectrum by the structural behaviour factor, \( q \), obtained from Eqs. 7 and 8.

### 3.3 Results of Ruiz-García and Miranda (2003)

Ruiz-García and Miranda (2003) proposed a method of approximating the maximum inelastic deformation demand as the product of maximum deformation demand of a linear elastic system and a displacement modification factor as follows:

\[
S_{di} = C_R S_{de} \tag{9}
\]

Here \( C_R \) is the displacement modification factor defined as:

\[
C_R = 1 + \left[ \frac{1}{a \left( \frac{T}{T_s} \right)^b} - \frac{1}{c} \right] (R - 1) \tag{10}
\]

Here \( a, b, c \) and \( T_s \) are site dependent coefficients reported by the authors; \( T \) is the natural period of vibration and \( R \) is the force reduction factor defined as \( R = (S_d(T, \zeta))/(C_y) \); \( S_d(T, \zeta) \) is the elastic spectral pseudo-acceleration. Using Eqs. 9 and 10, ductility demand in this case can be calculated as:

\[
\mu = \frac{C_R S_{de} \omega^2}{C_y} \tag{11}
\]

### 3.4 Results of Tothong and Cornell (2006)

Tothong and Cornell (2006) proposed inelastic displacement ratios (IDR) to be multiplied with an \( S_{de} \) attenuation model to obtain the inelastic spectral displacement \( S_{di} \). The IDR proposed by them is a function of moment magnitude and a predicted median strength-reduction factor \((\hat{R})\). They identify that the strength reduction factor, \( R = S_{de}/u_y \), which is often treated as a deterministic quantity, is not a priori as it implicitly contains the random variable \( S_{de} \), which is estimated as a median estimate or predicted median \((\hat{S}_{de})\) from an elastic GMPE. Hereafter we use the symbols \( S_{de} \) and \( R \) for \( \hat{S}_{de} \) and \( \hat{R} \) respectively. IDR proposed by Tothong and Cornell (2006) is expressed in following equations:

\[
\ln(\text{IDR}) = \begin{cases} 
0 & \text{for } R \leq 0.2 \\
g_1(M_w, R) + g_2(R) \cdot (M_w - 6.5) - g_1(M_w, 0.2) + \epsilon_{\ln(\text{IDR})} & \text{for } 0.2 \leq R \leq 10
\end{cases} \tag{12}
\]

where

\[
g_1(M_w, R) = (\beta_1 + \beta_2 M_w) \cdot R + (\beta_3 + \beta_4 M_w) \cdot R \cdot \ln(R) + \beta_5 R^{2.5} \tag{13}
\]

\[
g_2(R) = \begin{cases} 
0 & \text{for } R \leq 0.3 \\
0.37 \beta_6 (R - 0.3) & \text{for } 0.3 \leq R \leq 3 \\
\beta_6 & \text{for } 3 \leq R \leq 10
\end{cases} \tag{14}
\]

In Eqs. 12–14, \( \beta_1 \) through \( \beta_6 \) and \( \epsilon_{\ln(\text{IDR})} \) are the regression parameters reported by the authors for natural periods in the range of 0.3–5 s. Once IDR is computed, it is multiplied by \( S_{de} \) to obtain \( S_{di} \), which in turn is divided by the yield displacement, \( u_y \), to compute the ductility demand.
3.5 Comparison

It is worthwhile to note here that the displacement modification factor proposed by Ruiz-García and Miranda is dependent on $R$, and the ductility factor proposed in Eurocode 8 is dependent on an equivalent parameter, the structural behaviour factor, $q$. They are both functions of the system properties as well as the strong-motion excitation, being represented by the elastic acceleration spectrum. Strong-motion excitation in this study is taken into consideration by moment magnitude of the event, distance of site to the surface projection of fault and the ground type. To make a comparison between the results of these approaches, we need to define the elastic acceleration spectrum $S_a(T, \zeta)$. Then the elastic displacement spectrum can simply be computed as: $S_{de} = (S_a(T, \zeta))/(\omega^2)$. For a valid comparison, this elastic spectrum has to be compatible with the mean spectrum of ground motion records used in calibrating the proposed model. To accomplish this, we compute the elastic acceleration spectrum using the GMPEs proposed by Rupakhety and Sigbjörnsson (2009) due to the fact that the GMPEs were calibrated for the same set of ground motion records as used this study.

Figure 8 shows the comparison of the proposed GMPEs with previous studies and Eurocode 8 provisions mentioned above. It is observed that the results of current study match very well with those of Ruiz-García and Miranda (2003) for the four different combinations of normalized strength and source-to-site distance considered in Fig. 8. The results based on Eurocode 8 Type 1 spectra are also in close agreement with the results of proposed GMPEs except for systems located very close to the site (3 km here), in which case the Eurocode 8 Type 1 spectra consistently over-estimates the ductility demand below natural period of around 0.4 s. It has been observed that for a moment magnitude of 6.5 and natural periods below 0.5 s, Eurocode 8 Type 2 spectrum under-estimates the ductility demand in all cases considered here. Note that the Eurocode 8 Type 1 and Eurocode 8 Type 2 ductility demand curves follow the structural behaviour factor curve for periods larger than 0.25 and 0.4 s, respectively; which represent the values of $T_C$ at rock sites for these two types of spectra. The results of Tothong and Cornell (2006) are also in good agreement with the results of present study as seen by inspecting Fig. 8. However, it should be noted that they provide the parameters of their empirical model for natural periods above 0.3 s; which explains the reason why ductility demand proposed by them is not shown for shorter periods in Fig. 8. By careful examination of Fig. 8, we arrive at the conclusion that for undamped natural periods above 0.6 s, ductility demand can be safely assumed to be equal to the structural behaviour factor, also known as force reduction factor.

4 Conclusions

A simple and effective empirical model is presented to predict the ductility demand and inelastic displacement of constant-strength elastic-perfectly-plastic SDOF oscillators. Unlike in most previous studies, we do not make use of pre-specified elastic spectra to compute the ductility demands of inelastic systems. Instead the proposed model relates the ductility demands to the earthquake magnitude, distance of site to the surface projection of causative fault, ground type, undamped natural period, critical damping ratio and the mass-normalized yield strength of the oscillator. Our results indicate that the displacement demand of inelastic systems can be estimated at the same level of accuracy as their elastic counterpart. The standard deviations of the residuals associated with the proposed model are comparable to what has been reported in the literature for elastic response spectral ordinates. This eliminates the necessity to use so-called force reduction factors or displacement modification factors.
Fig. 8 Comparison of the proposed model with previous studies and Eurocode 8 provisions displaying ductility spectra for different combinations of source-to-site distance and normalized yield strengths of the system. Note that the combination is selected to limit the values of ductility within the range of practical importance in seismic performance evaluation of existing structures. Ductility demands computed by the proposed GMPEs can be interpreted as structure specific ground motion intensity measures that can easily be incorporated into the framework of PSDA. We have observed that smoothing of model parameters is desirable and does not lead to significant disturbance in their correlation properties. Our results are in fair agreement with the models proposed by Ruiz-Garcia and Miranda (2003) and Tothong and Cornell (2006). In addition, the results of proposed model are in better agreement with the Eurocode 8 Type 1 spectrum than with Type 2 spectrum, except for sites which are located very close to the causative fault, in which case, results based on Eurocode 8 Type 2 spectrum are closer to our results. The advantages of proposed model are that it is simpler than many existing models; it implicitly considers the magnitude dependence of inelastic displacement ratios; it does not require a pre-specified elastic spectra; and it makes use of, as a measure of structural strength, the normalized yield
strength, instead of ground-motion dependent quantities such as \( R \) or \( q \). The proposed model can be incorporated in probabilistic as well as deterministic seismic demand assessment of existing structures.

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**Appendix**

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**Table 2**  Model parameters and standard deviation of residuals for normalized strength of 1 m/s²

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<th>( b_4 )</th>
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<th>( \sigma )</th>
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### Table 3  Model parameters and standard deviation of residuals for normalized strength of 2 m/s²

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Table 4 Model parameters and standard deviation of residuals for normalized strength of 5m/s²

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<th>$b_4$</th>
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Paper 3

Estimating coseismic deformations from near source strong motion records: methods and case studies

R. Rupakhety · B. Halldorsson · R. Sigbjörnsson

Abstract  Digital strong-motion accelerographs have opened up the possibility of extracting ground motion characteristics at much lower frequencies than was offered by analogue instruments. High-quality digital data obtained close to the faults have tempted several efforts to retrieve permanent ground displacements after an earthquake. Such attempts have been partly successful, and somewhat subjective, the main reason being the presence of baseline offsets in the accelerometric data. We review existing methods for such applications, discuss their limitations and propose a more objective and improved scheme to make baseline adjustments and obtain permanent displacements. The proposed technique is applied to 26 digital recordings from the 29 May 2008 Ölfus Earthquake in South Iceland and 9 recordings from the 1999 Chi-Chi, Taiwan, Earthquake, and the permanent displacements obtained are compared with published results and GPS measurements from nearby stations. Our case studies show that the proposed technique, in addition to being simple and objective, is effective in making adjustments for baseline errors in accelerometric data.

Keywords  Baseline correction · Long-period motion · Near-fault ground motions · Permanent displacement

1 Introduction

Areas close to the causative fault of a strong earthquake are likely to experience permanent displacements after the event. This might occur due to either local inelastic effects or elastic deformation of ground caused by coseismic movement. When permanent displacements occur, the velocity record often contains a strong one-sided pulse, unlike when there are no
static displacements and the velocity records are two-sided (Mavroeidis and Papageorgiou 2003). An estimate of static ground displacements is important for seismological as well as earthquake engineering applications. Good estimates of coseismic displacements can be used to infer slip on the fault plane and thereby impose realistic constraints on physical models of earthquake sources. For earthquake resistant design, an estimate of expected permanent displacement is an important input parameter. This information is particularly important for designing infrastructures and lifeline systems that cross an active fault.

In theory, ground velocity and displacement can be accurately computed if 6-component (3 translational and 3 rotational) strong motion acceleration data are available (Bogdanov and Graizer 1976; Graizer 1979, 1989). Unfortunately, most strong motion instruments record only 3 translational components of ground acceleration. With high-quality digital data, it is in some cases possible to extract long-period motion, including permanent displacements from 3-component translational accelerometric data. However, the presence of various long-period noises superimposed on accelerometric signal makes this task challenging, and accurate results are hard to obtain.

In the near-source area, the common assumption that movement of an instrument’s base is purely translational like in the far field is not valid (Graizer 2006). Near-source acceleration data are likely to be contaminated by instrument response to small rotational motion. These effects are commonly referred to as tilts. Integration of tilt-contaminated acceleration produces linear trend in velocity and quadratic trend in displacement. Such trends make it almost impossible to obtain final displacement by simply integrating uncorrected acceleration data twice.

Apart from tilts, various other long-period noises are present in digital accelerometric data. These noises vary in nature and origin (Wang et al. 2003). Shift in the zero level, also known as baseline shift, is one of them. Baseline shifts produce effects similar to those induced by tilts, making the extraction of static displacement a challenging task. Researchers have put forward different schemes to tackle long-period noises and baseline shifts and approximately recover static displacements from 3-component translational acceleration data (Bogdanov and Graizer 1976; Iwan et al. 1985; Boore 2001; Boore et al. 2002; Wu and Wu 2007; Wang et al. 2003). Our goal in this paper is to assess different procedures in the literature, propose an improved technique, and demonstrate its efficiency applying case studies.

2 Tilts

Graizer (2006) proposed a method of identifying instrument tilts from tri-axial translational motion. His method is based on the different sensitivity of horizontal and vertical acceleration sensors to rotational ground motion. Vertical acceleration is less sensitive to small tilts than horizontal acceleration. The presence of tilts causes the Fourier amplitude spectra of horizontal acceleration to approach a finite value at zero frequency, unlike that of the vertical component which increases from zero to a maximum value at a few hertz. If the ratio of Fourier amplitude spectra of horizontal and vertical acceleration is much greater than one at low frequencies, tilt is possibly playing a part. Figure 1 shows the ratio of horizontal to vertical acceleration Fourier amplitude spectra for 33 components obtained from 11 stations of ICEARRAY (Halldorsson et al. 2009; Halldorsson and Sigbjörnsson 2009) recordings during the 29 May 2008 Ólfus Earthquake of South Iceland (Sigbjörnsson et al. 2009). In subsequent sections all stations starting with ‘IS’ indicate different recording stations of the ICEARRAY. The east–west component is considered in (a) and the north–south in (b) of Fig. 1. We observe that there are variations in these ratios at low frequencies for individual
stations (shown by grey lines). Following Graizer (2006), this could possibly be due to tilts. It can be expected that the presence of tilt can cause an order of magnitude difference in the low frequency Fourier amplitude spectra of horizontal and vertical accelerations. In Fig. 1 these ratios at low frequencies reach a maximum value of about 4, which does not indicate an order of magnitude difference. For these data we assume that tilt is not dominant, and that long-period motion is due to permanent ground displacement and other noises. In addition, the effects of tilts are similar to those of baseline offsets discussed in subsequent sections, and baseline adjustments can be used to remove some of the tilt effects. We do not try to retrieve tilts, if any are present, but simply try to eliminate their effects so as to retrieve permanent ground displacements. For the former task, the readers are referred to Graizer (2006).

3 Baseline shifts in digital accelerograph data

High-resolution digital accelerometric recordings can be used to extract long-period strong-motion characteristics. As a direct example, we illustrate the recovery of permanent ground displacement from a near-fault recording. We use the north–south component of IS610 station. The first column of graphs in Fig. 2 shows acceleration, integrated velocity and displacement for this station indicating a static displacement of about 9 cm at the end of the record. Such applications are generally not feasible, and the accuracy of derived information is often in doubt due to the presence of baseline shifts and distortions as well as superimposed long-period noises. Baseline shifts are small variations in acceleration zero-level, often occurring as near-instantaneous shifts (Akkar and Boore 2009). Baseline shifts in acceleration result in nonzero velocities at the end of the record, which are unphysical (Boore 2001; Boore et al. 2002). This effect is clearly evident in the north–south component of another station, IS603, recording the same earthquake. The integrated velocity in this case shows a linear trend resulting in a drift in displacement (see Fig. 2 second column of graphs). In such cases, baseline shifts should be assessed and removed from the acceleration data before it is integrated to obtain peak ground displacement or residual displacement or is used to compute long-period spectral displacements.
4 Adjustments for baseline shift

Baseline corrections can be performed with estimates of offsets in acceleration by fitting a straight line to a portion of integrated velocity (Graizer 1979). Several modifications of this idea have been applied by other researchers in the past (see, for instance, Douglas 2001, 2002). In this section we discuss existing methods of baseline adjustment and propose
a new procedure. We will consistently use the north–south component of IS604 acceleration to illustrate and evaluate each method. In addition, we will also present applications to the recordings of the 1999 Chi-Chi, Taiwan, Earthquake.

4.1 Method of Iwan et al. (1985)

The baseline correction procedure proposed by Iwan et al. (1985) is based on the assumption that variations in acceleration baseline are confined between time points \( t_1 \) and \( t_2 \), where \( t_1 < t_2 \). The idea behind this method is shown schematically in Fig. 3. We take the north–south component of IS610 acceleration and remove pre-event mean from the whole record (see also the top left graph of Fig. 2). Velocity corresponding to this acceleration is shown with the green line in Fig. 3. The baseline shift in acceleration is modelled as shown with the thick blue line. This shift is added to the pre-event-mean reduced acceleration. Then the artificially contaminated acceleration is integrated to obtain velocity, shown with the red line. The black line corresponds to the integral of shift in acceleration baseline. Obviously the linear trend in velocity due to baseline shift is caused by the slope of the black line after \( t_2 \). Iwan et al. (1985) proposed that baseline shifts occur during the strong shaking and can be modelled as two offsets. The first offset \( a_m \) represents an average of several complicated shifts between \( t_1 \) and \( t_2 \). A second offset \( a_f \) is assumed to occur at \( t_2 \). If we assume this model to be correct, then baseline adjustment can be performed in a very straightforward way. A straight line is fitted to the linear trend in velocity, and its slope gives the value of \( a_f \). In practice, the linear trend might not be obvious right after \( t_2 \), in which case linear fitting in velocity is done between time points \( t_{FITb} \) and \( t_{END} \), where \( t_{FITb} \) is chosen well after strong motion has subsided, and \( t_{END} \) is the end of the record. The fitted line is extrapolated to \( t_2 \) where its value is equal to \( v_f(t_2) \). The average baseline shift to be applied between \( t_1 \) and \( t_2 \) can be computed as

\[
a_m = \frac{v_f(t_2)}{t_2 - t_1} \tag{1}
\]

Although this method is simple, selections of \( t_1 \), \( t_2 \) and \( t_{FITb} \) are arbitrary, and researchers have proposed different schemes for selecting these parameters. Iwan et al. (1985) chose \( t_1 \) and \( t_2 \) based on laboratory tests on instruments with which they were concerned. They proposed \( t_1 \) to be the instant where the absolute value of acceleration first exceeds 50 cm/s\(^2\). They proposed \( t_2 \) be selected as: (1) the time after which acceleration never exceeds 50 cm/s\(^2\) or (2) the instant that minimizes final displacement. We use the north–south component of IS604 station to evaluate this model. Different values of \( t_{FITb} \) were selected and the results are shown in Fig. 4. The displacement waveforms are sensitive to \( t_{FITb} \), although the differences are not great. On the other hand, the choice of \( t_{FITb} \) is subjective, and this is the main disadvantage. Looking at Fig. 4, a value of \( t_{FITb} = 22 \) s seems to give reasonably flat displacement towards the end of the record. Note that there are small drifts in the displacement before 14 s. In all subsequent analysis we remove this effect by de-trending this part of recorded acceleration. By ‘raw’ acceleration we mean pre-event mean removed and detrended acceleration. The pre-event time segment for ICEARRAY data is 15 s. More details on the de-trending technique are discussed in Sect. 6 below. Apart from the sensitivity of results on \( t_{FITb} \), this method also suffers from the shortcoming that the threshold values of acceleration corresponding to \( t_1 \) and \( t_2 \) may not be universal. Boore (2001) noted that the threshold of 50 cm/s\(^2\) proposed by Iwan et al. (1985) is not suitable for several recordings of the 1999 Chi-Chi Earthquake.
Fig. 3 Illustration of baseline shift in acceleration and the corresponding linear trend in integrated velocity. The green line corresponds to the velocity obtained by integrating the acceleration record (pre-event mean removed) of the north–south component of IS 610 record. Baseline shifts equal to $a_m$ and $-a_f$ added to the acceleration record are shown with the thick blue line. The black line corresponds to the integral of baseline shift, and the red line corresponds to the velocity obtained by integrating the acceleration record contaminated with the artificial baseline shifts.

Fig. 4 Raw displacement (dotted line) and corrected displacement (solid lines) using the method proposed by Iwan et al. (1985) for the IS604 north–south component data of the ICEARRAY. Solid lines with different colours correspond to different values of $t_{\text{FITb}}$.

4.2 Method of Boore (2001) and improvements by Akkar and Boore (2009)

Boore (2001) generalized the correction procedure of Iwan et al. (1985) by making $t_1$ and $t_2$ as free parameters, not determined by a threshold of shaking, and subject to the constraints $t_1 > 0, t_2 > t_1$ and $t_2 < t_{\text{END}}$, where $t_{\text{END}}$ corresponds to the end of the record. In a further simplification he proposed that $t_2$ be chosen as the time where a linear fit to the later
portion of raw velocity becomes zero. This scheme was named by him as $v_0$ correction. It cannot be applied if the fitted line crosses the zero line before $t_1$ or after the end of the record. In addition, the selection of different correction time parameters, namely $t_1$ and $t_2$, were found to give widely differing results (Boore 2001). Akkar and Boore (2009) proposed additional parameters $t_{BLB}$ (time of baseline beginning), $t_{BLe}$ (time of baseline end), and $t_{FITb}$ (time after which a straight line is fitted to the velocity). These parameters were intended to enforce additional constraints on $t_1$ and $t_2$. They also proposed iterative schemes to estimate these parameters. Detailed definitions of the parameters and the iterative procedure to estimate them will not be mentioned here, and interested readers are referred to Akkar and Boore (2009).

We only demonstrate their process with an example and show the results for the north–south component of the IS604 recording. Figure 5 illustrates the procedure for selecting different parameters. The selected parameters are shown in respective plots of Fig. 5. Time $t_2$ lies between $t_1$ and $t_{BLe}$. We chose a number of values in this range and performed baseline correction for each value. The results are shown in Fig. 6, which shows that the quadratic

Fig. 5 Iterative procedure proposed by Akkar and Boore (2009) for choosing time parameters in their baseline correction method. a Uncorrected velocity b selection of $t_{FITb}$ c selection of $t_{BLe}$ d selection of $t_{BLB}$. The north–south component of IS604 station is used in this example
trend in displacement is hardly removed. We would expect that the results for $t_{FBTb} = 36$ s in Fig. 4 should be similar to those corresponding to at least one value of $t_2$ in Fig. 6. This, however, is not the case. This could be due to the effect of the parameter $t_1$. According to Iwan et al. (1985) this parameter would be 16.215 s. Akkar and Boore (2009) do not specify any distinct value of $t_1$ and only mention that it should be greater than $t_{BLb}$. Figure 6 corresponds to $t_1 = t_{BLb}$. It is obvious that the results could be improved by selecting different values of $t_1$. We did not try this simply because the procedure then becomes too subjective. The fact that the results are very sensitive to $t_1$, which is not objectively defined, is a disadvantage of this method. We have found that this can be improved significantly by selecting a different value of $t_{FBTb}$, in which case $t_1$ will always be taken equal to $t_{BLb}$. With this in mind, we show that the poor performance of this method is due to the high value of $t_{FBTb}$ selected for this record. We propose that this parameter be chosen as close to $t_{BL}c$ as possible, provided that the ratio of slopes around this time as shown in Fig. 5b is not very much different than one. In other words, if there is a range of time parameters where the slope ratio is close to one, we choose the time closest to $t_{BL}c$ as the preferred value for $t_{FBTb}$. In Fig. 5b such a time would be around 25 s. This modification is also supported by the definitions used by Akkar and Boore (2009). The authors mention: “We determine $t_{BL}c$ on the basis of increasing variations of the zoc [zero order corrected] velocity from the line corresponding to $t_{FBTb}$, working towards decreasing time from $t_{FBTb}$. The idea here is to find the time beyond which the zoc velocity has little variation about a linear trend, implying that there is little change in the ground shaking.” Therefore $t_{FBTb}$ is the time after which uncorrected velocity is nearly linear and so is $t_{FBTb}$. This leads to the conclusion that these two parameters should be almost equal. In fact, these two parameters can also be selected as a single instant, which not only simplifies the model but also reduces the chances of selecting a very high value of $t_{FBTb}$. We test this modification by keeping all the parameters the same but changing the value of $t_{FBTb}$ to 25 s instead of 38 s, and the results are shown in Fig. 7. Corrected displacement with the proposed modification is more physical and the sensitivity of the results to $t_2$ is greatly reduced.
This issue is very important, and we further demonstrate its effects with a record from the 1999 Chi-Chi Earthquake, which was originally used by Akkar and Boore (2009). The objective is to make sure that the proposed modification does not lead to poor results in this case, where Akkar and Boore (2009) obtained satisfactory results with their definition of \( t_{FITb} \). The record used is the east–west component of TCU068 station. Figure 8 demonstrates the iterative procedure proposed by Akkar and Boore (2009). Akkar and Boore (2009) preferred to choose 63 s for \( t_{FITb} \), whereas we propose that 45 s be chosen, as shown in Fig. 8b. Next, we examine the differences in corrected displacement waveforms with these two choices. The results are shown in Fig. 9. We observe that the results are slightly different, and which choice is more appropriate is hard to tell. Nevertheless, displacement waveforms are physical and for some values of \( t_2 \), either choice of \( t_{FITb} \) give similar results. In subsequent analysis we use the proposed modification in selecting \( t_{FITb} \) when we refer to Akkar and Boore (2009).

With additional complexity in terms of more parameters, there is still a wide range of values of \( t_2 \) that can be chosen between \( t_1 \) and \( t_{BLE} \). To account for the uncertainty in the selection of different time instants in the correction scheme, Akkar and Boore (2009) used Monte Carlo simulations to obtain average values of ground displacement and spectral displacements. Their method seems effective in incorporating the uncertainties in the correction scheme. The details of such simulations will not be discussed here but can be found in Akkar and Boore (2009). Their approach has been successful in estimating average ground displacements and spectral quantities by considering the uncertainties in the decision of different time segments as well as the noise level. Based on the results of their simulations, they concluded that the simplest correction procedure, without any simulations, often gives the peak ground displacements and spectral displacements that compare well with the values obtained from simulation-based procedures. They also conclude that the mean residual displacements are not sensitive to the noise (or baseline offsets) model used in the simulation procedure. In this study we use the noise model of Iwan et al. (1985), without performing Monte Carlo simulations.
Fig. 8 Iterative scheme of Akkar and Boore (2009) for choosing time parameters in their baseline correction procedure. a) The uncorrected velocity b) selection of time $t_{FITb}$, two selections are considered here, $t_{FITb1} = 63$ s, as proposed by Akkar and Boore (2009) and $t_{FITb2} = 45$ s, as proposed by us. c) Selection of $t_{BLe}$ d) selection of $t_{BLb}$. The east–west component of TCU068 recording is used in this example.

4.3 Method of Wu and Wu (2007)

Wu and Wu (2007) proposed a different scheme to select the parameters $t_1$ and $t_2$. It is based on the observation that in the near-source region, the corrected displacement history should be similar to a ramp function if permanent displacement occurs. The parameter $t_1$ is selected as the time when the ground just starts to move from its initial position. They then define another parameter $t_3$, which they propose as the time when the displacement has reached the final value. Since the corrected displacement is not available a priori, except if there are collocated GPS stations with continuous measurements, the choice of $t_3$ is subjective in the beginning but can be improved by performing iterations. They then select $t_2$ as any time between $t_3$ and the end of the record and perform the correction as proposed by Iwan et al. (1985). It should be noted here that the parameter $t_2$ in this method is the same as in Iwan et al. (1985). For each value of $t_2$, they compute a flatness coefficient, which shows the degree of flatness of the corrected displacement between $t_3$ and the end of the record. They regard the mean value of the corrected displacement history from $t_3$ to the end of the record as the permanent
displacement and determine its standard deviation $\sigma$. They fit a least squares straight line for the displacement waveform from $t_3$ to the end of the record and compute its slope, denoted as $b$. Then they compute $r$, the linear correlation coefficient of the displacement time history from $t_3$ to the end of the record. Using these values the flatness coefficient is computed as:

$$\phi = \frac{|r|}{|b|/\sigma}$$

(2)

The value of $t_2$, which yields the maximum value of the flatness coefficient is chosen as the final $t_2$. Corresponding to this $t_2$, a correction is applied. Then the corrected displacement waveform is studied to refine the values of $t_1$ and $t_3$, if necessary, and the whole procedure is repeated. This scheme was validated by the authors, using data from shake-table tests of a known displacement on 249 accelerographs as well as strong-motion data from two earthquakes in Taiwan, respectively the 1999 Chi-Chi Earthquake and the 2003 Cheng-Kung Earthquake. The disadvantage of this model is that there is no clear prescription when the iterative scheme should be stopped or, in other words, there is ambiguity as to which values of $t_1$ and $t_3$ are appropriate. However, this approach has the benefit that the final results are less sensitive to the choices of these parameters than in the previously mentioned methods.

4.3.1 Sensitivity of corrected displacement to the parameter $t_3$

From our preliminary analysis, we observed that the method of Wu and Wu (2007) gave reasonable displacement waveforms for the ICEARRAY data. However, in this method, the choice of $t_3$ is arbitrary and is not easy to estimate. Let us for example consider the case when we want to apply this method to the north–south component of IS604 acceleration record. It can be seen from Fig. 10 that the ground starts to move from its initial position at around 15 s, so this time can be chosen as parameter $t_1$. However, it is difficult to decide the parameter $t_3$ because the final displacement is unknown before the appropriate correction is
applied. Figure 10 shows the displacement waveform obtained after applying adjustment to the baseline using different values of $t_3$, namely 18, 18.5, 19, and 22 s. It can be seen that the peak displacement is not sensitive to the choice of $t_3$, however, the final displacement is. In addition, the flatness of the final portion of a displacement waveform also depends on the choice of this parameter and is the maximum for $t_3 = 22$ s for this particular accelerogram. Although this method involves fewer parameters than the previous method considered herein, it gives satisfactory results. This observation is valid for almost all of the 33 components of the 11 recording stations of ICEARRAY. Any baseline adjustment procedure should be as objective as possible, and the model of Wu and Wu (2007) has behaved the best of the models dealt with so far for our dataset. Having said this, the selection of $t_3$ in this model is still subjective, and the output is somewhat dependent on this parameter.

4.4 Proposed baseline correction technique

We present an alternative scheme of baseline adjustment in this section. Our proposal is primarily based on the work of Iwan et al. (1985) and Wu and Wu (2007). We use the baseline shift model proposed by Iwan et al. (1985), which is shown in Fig. 3. In this model, we select parameter $t_1$ as the time when the displacement starts to move from its initial position. We make use of the idea, proposed by Wu and Wu (2007), that the corrected displacement resembles a ramp step function, to make a rational estimate of time $t_2$. We further note that since $t_2$ is the time after which the baseline offset in the acceleration is constant, the velocity should follow a linear trend after this time, which means that a straight line can be fitted to it by using least squares regression. We therefore propose that the $t_2$ and $t_{FTB}$ in the model of Iwan et al. (1985) should be the same. This assumption is valid because, by definition, $t_2$ is the time when the last shift in baseline is assumed to have occurred, and it therefore is not necessary to look for a later time $t_{FTB}$ from which to perform a linear fit to the velocity. There might be some cases when a straight line does not fit well from $t_2$ to the end of the record. This happens if there are additional shifts in baseline after $t_2$. Such conditions simply imply that the baseline shift model with adjustments at two instants of time is not accurate, and in such cases more complicated models should be applied. Fitting multiple straight lines to the velocity is one possibility. For the records we have analyzed, we have found that the two-instant baseline adjustment scheme is adequate. Therefore, the only other parameter that
is not known is $t_2$. As mentioned earlier we use the notion that the displacement is ramp-like to estimate this parameter. This model of displacement is the simplest possible time domain description for near-fault stations where permanent displacement at the end of the record is expected. The following section describes the reasoning behind our selection of $t_2$.

4.4.1 Ramp displacement, corresponding velocity and the Fourier amplitude spectrum

We start by assuming that the displacement waveform is a perfect ramp step function:

$$D(t) = \begin{cases} 
0 & t < t_1 \\
H(t - t_1) & t_1 \leq t \leq t_3 \\
HT & t > t_3
\end{cases}$$

Then the corresponding velocity becomes a boxcar given as:

$$V(t) = \begin{cases} 
H & \text{for } t_1 \leq t \leq t_3 \\
0 & \text{elsewhere}
\end{cases}$$

where

$$t_1 = c - \frac{T}{2}$$

$$t_3 = c + \frac{T}{2}$$

Here $T$ denotes the duration between the time points $t_1$ and $t_3$ and $c$ their average, i.e. the boxcar is centred at time $c$, furthermore, it has the height $H$. The displacement and the corresponding velocity are shown in Fig. 11.

4.4.2 Proposed criteria to select the value of $t_2$

Next, we evaluate the Fourier amplitude spectra (FAS) of the boxcar velocity. If $f$ is the frequency then the Fourier transform of velocity is given by

$$F(f) = H e^{-i2\pi fc} \frac{\sin (\pi fT)}{\pi f}$$

then the FAS is given by

![Fig. 11 Displacement modelled as a ramp (top) and the corresponding velocity (bottom), which is a boxcar of width $T$ and centred at $c$. The displacement starts at $t_1$ and reaches a maximum value of $TH$ at time $t_3$ after which it remains constant at this value. The slope of the ramp is equal to the height of the boxcar.](image-url)
The Fourier amplitude of velocity at zero frequency can be computed by taking the limit on Eq. (7) as $f \to 0$, which gives

$$FAS_v(0) = \lim_{f \to 0} H T \left| \sin \left( \frac{\pi f T}{f T} \right) \right| = H T$$  \hspace{1cm} (8)$$

From Eqs. (7) and (8) it is clear that the FAS of velocity for a displacement modelled as a ramp approaches the permanent displacement at low frequencies and decays as $f^{-1}$ above the corner frequency. If additional extremely long-period noise is not present in the signal, and if there are no baseline offsets, the FAS of velocity should approach the permanent ground displacement asymptotically as the frequency approaches 0. In addition, the FAS of the corrected velocity should be flat at low frequencies. We verify this observation with the north–south component of acceleration at IS610, which is shown in the left column of Fig. 2. We chose this record, because it does not contain any significant shifts in baseline as is discussed earlier, even though some minor shifts in baseline might be present. Figure 12 shows FAS computed from raw data for this record with ample descriptions in its caption.

From the above discussion it is clear that if the baseline adjustment method is aimed at modelling the displacement waveform as a ramp step function, then corrected displacement should be very close to the value of the Fourier amplitude spectra of the corrected velocity at zero frequency. This condition can be used as an additional constraint on $\tau_3$ in the correction

---

**Fig. 12** Fourier Amplitude Spectra (FAS) of the velocity obtained by integrating the north–south component of acceleration recorded at IS610 station of the ICEARRAY. The FAS approaches a value of 9.28 cm at low frequencies. The permanent displacement obtained by integrating twice the acceleration is 9.3 cm (see also bottom left plot of Fig. 2)
scheme proposed by Wu and Wu (2007). However, in some cases, selecting the FAS of the corrected velocity at zero frequency might not give the desired results, especially if there is a trend in the FAS to decrease at low frequencies. We expect the FAS at low frequencies to be as flat as possible. We have observed that setting this criterion of maximizing the flatness of low frequency FAS of velocity also satisfies the criterion that the permanent displacement is nearly equal to the mean value of flat portion of the FAS. As a summary our scheme involves the following main steps:

1. Integrate the raw acceleration twice and estimate parameter $t_1$ as the time where the displacement starts to move from its initial position.
2. Select a value of $t_2$ greater than $t_1$.
3. Fit a quadratic function, $At^2 + Bt + C$, to the raw displacement from time $t_2$ to the end of the record.
4. The corresponding fit to the velocity would be described by the line $2At + B$. Using this equation we compute the velocity at time $t_2$, which would be given by $v_f(t_2) = 2At_2 + B$. The slope of this line is simply $a_f = 2A$.
5. Compute the value of $a_m$ by using Eq. (1), i.e., $a_m = v_f(t_2)/(t_2 - t_1)$.
6. From the raw acceleration, subtract $a_m$ between time points $t_1$ and $t_2$ and $a_f$ from time point $t_2$ to the end of the record to obtain the corrected acceleration.
7. Integrate the corrected acceleration to obtain the corrected velocity and compute its Fourier amplitude spectrum.
8. Compute the flatness coefficient of the Fourier amplitude spectrum between frequencies 0 and $f_{\text{min}}$. The frequency $f_{\text{min}}$ is related to the corner frequency; in particular, it must be chosen smaller than the corner frequency. As we know that the corner frequency depends on the magnitude of the earthquake, the frequency $f_{\text{min}}$ is also magnitude dependent. For larger earthquakes, $f_{\text{min}}$ will be smaller. For the May 29 2008, $M_w$ 6.3 earthquake of South Iceland, a value of $f_{\text{min}}$ equal to 0.05 Hz gave good results, whereas for the 1999 Chi-Chi earthquake of $M_w$ 7.6, this frequency was chosen to be equal to 0.03 Hz. More discussion in the selection of this frequency is presented later. The flatness coefficient is as defined in Eq. (2). In this case, it is computed for the FAS of the velocity.
9. Increase the value of $t_2$ and compute a new flatness coefficient.
10. Repeat step 9 until $t_2$ has reached the time when the ground motion has almost died out. The limiting value of $t_2$ should be such that there are sufficient points between $t_2$ and the end of the record to perform step 3.
11. The value of $t_2$ yielding the maximum flatness coefficient is chosen as final, and steps 3 to 6 are performed with this $t_2$.

4.4.3 Results of proposed scheme

In this section we evaluate our proposal for the north–south component of the IS604 record and compare our results with the results of other methods presented above. Our method is straightforward and robust, and users need to decide only a single parameter $t_1$, which, as discussed previously, is relatively simple to assess. The results are shown in Fig. 13, the caption of which provides ample description. It is also worthwhile to compare our results with those of the other methods discussed earlier.

Figure 14 compares the results obtained by applying different correction schemes to the north–south component of the IS604 acceleration record. It can be seen that all methods deliver almost equivalent results. It should however be noted that the results of other methods
Fig. 13 Permanent displacement. (a) Displacement waveform obtained by performing the proposed correction scheme. The red line shows the mean value of the displacement from time \( t_2 \) to the end of the record, which is equal to 17.11 cm. The value of \( t_2 \) in this case is 19 s. (b) Corresponding Fourier amplitude spectra of corrected velocity. The red line shows the mean value of the FAS in the low-frequency region, which is very close to the permanent displacement. The north–south component of IS604 is used in this example.

The selection of \( f_{\text{min}} \) is crucial in the computation of the flatness coefficient of the FAS of velocity. As mentioned earlier, this frequency depends on the earthquake magnitude, being smaller for larger earthquakes. For \( M_w \) 6.3 earthquake in South Iceland a value of 0.05 Hz provided good results, whereas for \( M_w \) 7.6 Chi-Chi earthquake a value of 0.03 Hz was found more satisfactory. Based on these observations, we believe that appropriate values can be chosen for magnitudes in this range. What is more important is that the results are not very sensitive to the actual value of \( f_{\text{min}} \). We illustrate this with the north–south component of IS602 recording in this section. The corrected displacement history obtained by using four different values of \( f_{\text{min}} \), respectively 0.01, 0.03, 0.05 and 1 Hz, are shown in Fig. 15. It is seen that the results are not highly sensitive to the selection of this frequency due to the fact that the FAS of velocity is flat below each of these frequencies. For large earthquakes containing significant motion in the low-frequency range, such a high value of \( f_{\text{min}} \) might not yield reasonable results. For example, we present the correction of the vertical component of the TCU068 recording. We present the results obtained for this recording for four different values...
Fig. 14 Comparison of displacement waveforms obtained from different baseline correction schemes as denoted in the legend of the figure. The black line corresponds to the model proposed by Akkar and Boore (2009), with the modification proposed by us for selection time $t_{FITb}$. The time parameters used for this model are $t_1 = 15$ s, $t_{BLE} = 22$ s, $t_{FITb} = 23$ s, and the value of $t_2$ is chosen to be equal to 20 s, which is estimated by visual inspection. For the Wu and Wu (2007) method, time $t_1$ is set as 15 s and $t_3 = 17.5$ s.

of $f_{\text{min}}$, namely 0.01, 0.03, 0.05 and 0.5 Hz, respectively, in Fig. 16. The red and the black lines in Fig. 16a show that the quadratic trend in displacement is not completely removed. The green and blue lines corresponding to lower values of $f_{\text{min}}$ give reasonable results. Selecting too high a value of $f_{\text{min}}$ can give unreasonable results, as shown by the black line in Fig. 16a. If by lack of experience or by accident we assign too high a value to $f_{\text{min}}$, we are likely to obtain absurd results. The advantage of the proposed method is that such incidents can easily be identified and corrected. If we look at the black line of Fig. 16b, we realize that the FAS of velocity is not flat up to 0.5 Hz, a clear indication that the selected value of $f_{\text{min}}$ was too high. Then we should reduce this frequency well below the frequency where the FAS just starts to decay. Hence the parameter of the proposed model can be checked and adjusted if necessary.

5 Application of the proposed method to some of the recordings from the 1999 Chi-Chi earthquake

In this section we apply the proposed baseline adjustment procedure to some of the recordings from the 1999 Chi-Chi Earthquake. The recordings from this earthquake have been used by researchers in the past to estimate coseismic displacements. We intend to compare our results with those available in the literature. Although the proposed method seems to work very well for the ICEARRAY recordings, it has to be kept in mind that these data are obtained from identical instruments of a small aperture array (Halldorsson et al. 2009). In addition, these records are produced by a single earthquake of $M_w$ 6.3. It would be useful to check the validity of the proposed method against recordings of other earthquakes, preferably those producing large permanent displacements. The purpose of this section is to investigate whether the proposed method is applicable to records from the Chi-Chi Earthquake, especially because large permanent displacements were observed in some of the stations during it. We analyze the TCU068,
Fig. 15  Corrected displacement history for the north–south component of IS602 recording. Different lines correspond to different values of $f_{\text{min}}$. Note that the red, the black and the green lines are overlapping.

Fig. 16  a Corrected displacement waveform for the vertical component of the TCU068 recording. Different lines correspond to different values of $f_{\text{min}}$. b Corresponding FAS of velocity.
Fig. 17 Comparison of displacement waveforms obtained from different baseline correction schemes as denoted in the legend of the figure. The thin black line corresponds to the model proposed by Akkar and Boore (2009), with the modification proposed by us for selection time $t_{fitb}$. The time parameters used for this model are $t_1 = 25$ s, $t_{pre} = 40$ s, $t_{fitb} = 45$ s, and the value of $t_2$ is chosen to be equal to 35 s. This value is chosen to obtain displacement similar to that obtained by Akkar and Boore (2009). For Wu and Wu (2007) method; time $t_1$ is set as 25 s and $t_3 = 38$ s. The east–west component of the TCU052 acceleration record is used in making this figure.

TCU052 and TCU129 station recordings of this earthquake. As an illustration we present displacement waveforms from the east–west component of the TCU052 station in Fig. 17.

Table 1 shows the values of permanent displacements computed by us for the three components of TCU068, TCU052 and TCU129 stations during the 1999 Chi-Chi earthquake. Where available, we also present a comparison with past studies. The values reported for Akkar and Boore (2009) are based on Monte Carlo simulations performed by them. The nearest GPS station data reported by Yu et al. (2001) are shown in the last column of the table. We see that the proposed method gives results comparable to the results of other researchers. However, there is some variability in the results from different researchers, and which of them are the most accurate is hard to tell. The clear advantage of the proposed scheme is that the selection of model parameters is automated, based on robust and rational reasoning thereby eliminating subjectivity in its applications.

6 Application of the proposed method to 29 May 2008 Mw 6.3 Ölfus earthquake

We applied the proposed procedure to perform baseline adjustment on the recordings of 11 stations of the ICEARRAY. Note that there is a small drift in the displacement waveform before about 14 s (see, for example, the displacement waveform in Fig. 4 before 14 s). Such a drift is unphysical and might be due to small baseline variations or other long-period noise. We try to minimize such drifts by de-trending the pre-event mean removed acceleration. This is done by subtracting a polynomial of higher order (generally 3) from the early part of the
Table 1  Permanent displacements at three stations of the 1999 Chi-Chi Earthquake

<table>
<thead>
<tr>
<th>Station</th>
<th>Displacement (cm), East, North up as positive</th>
</tr>
</thead>
<tbody>
<tr>
<td>TCU052</td>
<td>E–W −395.0</td>
</tr>
<tr>
<td></td>
<td>N–S 628.6</td>
</tr>
<tr>
<td></td>
<td>V 348.4</td>
</tr>
<tr>
<td>TCU068</td>
<td>E–W −674.1</td>
</tr>
<tr>
<td></td>
<td>N–S 617.7</td>
</tr>
<tr>
<td></td>
<td>V 350.5</td>
</tr>
<tr>
<td>TCU129</td>
<td>E–W 150.0</td>
</tr>
<tr>
<td></td>
<td>N–S −73.8</td>
</tr>
<tr>
<td></td>
<td>V −8.4</td>
</tr>
</tbody>
</table>

Table 2  Mean horizontal displacements computed at the 11 stations of the ICEARRAY located in Hveragerdi and SCH (Selfoss Town Hall)

<table>
<thead>
<tr>
<th>Station</th>
<th>Displacement (cm)</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N–S (North positive)</td>
<td>E–W (East positive)</td>
</tr>
<tr>
<td>IS601</td>
<td>16.45</td>
<td>−14.27</td>
</tr>
<tr>
<td>IS602</td>
<td>14.1</td>
<td>−14.65</td>
</tr>
<tr>
<td>IS603</td>
<td>14.91</td>
<td>−11.27</td>
</tr>
<tr>
<td>IS604</td>
<td>17.11</td>
<td>−8.71</td>
</tr>
<tr>
<td>IS605</td>
<td>16.01</td>
<td>−11.31</td>
</tr>
<tr>
<td>IS607</td>
<td>17.57</td>
<td>−10.55</td>
</tr>
<tr>
<td>IS608</td>
<td>20.12</td>
<td>−10.18</td>
</tr>
<tr>
<td>IS688</td>
<td>18.46</td>
<td>−10.11</td>
</tr>
<tr>
<td>IS609</td>
<td>12.46</td>
<td>−10.51</td>
</tr>
<tr>
<td>IS610</td>
<td>11.81</td>
<td>−9.47</td>
</tr>
<tr>
<td>IS611</td>
<td>16.52</td>
<td>−12.57</td>
</tr>
<tr>
<td>SCH</td>
<td>−12.07</td>
<td>11.05</td>
</tr>
</tbody>
</table>

acceleration data. This de-trended acceleration is then subjected to the baseline correction scheme proposed above. Note that the correction procedure is automated and does not require any subjective choice of parameters, except for the selection of a range of $t_2$ values, the optimum value being selected as discussed previously. Table 2 shows the mean displacement computed at the 11 stations of the ICEARRAY. By mean displacement, we refer to the mean value of displacement from time $t_2$ to the end of the record, or the flat part of the displacement waveform, which is also represented by the red line in Fig. 13a for the north–south com-
ponent of IS604. There is a GPS station operated by the Icelandic Meteorological Office at Hveragerdi (IMO website http://hraun.vedur.is/ja/gps.html). The location of the GPS station is 64.0169 N and −21.1850 E. The co-seismic displacement recovered from this station is 17 cm NW. The closest ICEARRAY station to this GPS station is IS609 and is at a distance of 1.6 km. The displacement computed by the proposed procedure is 16.3 cm for this station, which is very close to the GPS measurement. Andrew Chanerley has been using discrete stationary waveform transform technique to estimate the permanent ground displacement after earthquakes (Chanerley and Alexander 2009; Chanerley et al. 2009b). His estimate of displacement at IS605 is 14.5 cm N, 12.1 cm W, yielding a resultant value of 18.9 cm NW (Chanerley et al. 2009a). Our result for this station is 19.6 cm, which is fairly close to their results.

We also processed the acceleration records obtained at Selfoss, which lies southeast of the causative fault. It includes the recordings from the town hall of Selfoss. The uncorrected acceleration data for the station was obtained from the recently updated ISESD database (see, for instance, Ambraseys et al. (2004)). Two acceleration series are processed namely the two horizontal components at the town hall (Waveform ID 13010) of Selfoss. The corrected displacement waveforms and the FAS of velocity are shown in Fig. 18 for the north–south component and Fig. 19 for the east–west component, respectively. This station is marked as SCH (Selfoss Town Hall) in Table 2. We have estimated a resultant displacement of 16.36 cm S42°E for this station. A nearby GPS station (63.9289° N and 21.0322° W) operated by the Icelandic Meteorological Office estimated around 19 cm of permanent displacement. The distance between the GPS station and the accelerometric station is about 3 km.

![Fig. 18](image-url) **a** Corrected displacement waveform for the north–south component of the acceleration record at Selfoss Town Hall during the May 29 2008 Ólfus Earthquake in Iceland. Positive displacement is towards the north. **b** Corresponding FAS of velocity
7 Effect of baseline correction on elastic response spectra

Boore (2001) concluded that the effect of baseline correction on the elastic relative spectral displacement was not significant for undamped natural periods less than 20 s for several recordings of the 1999 Chi-Chi Earthquake. In this section we investigate the sensitivity of the 5% damped elastic spectral displacement to the different schemes of baseline adjustments described previously. It was observed that the findings made by Boore (2001) are also valid for the recordings of the ICEARRAY. There are no significant differences in the elastic spectral displacement when different schemes of baseline corrections were used. We present our results for station IS603 in Fig. 20. It is observed that the lines corresponding to different baseline adjustments overlap each other for natural periods less than 20 s, and any differences for longer periods are negligible. Although only the east–west component of IS603 is presented here, similar results were observed for all the components of all 11 stations of the ICEARRAY data used in this study. Akkar and Boore (2009) pointed out the possibility of increased sensitivity of spectral displacement to baseline correction for inelastic response, due to the fact that the effective oscillation period of inelastic system tends to increase with increasing excitation. We investigated such a behaviour for inelastic spectral displacements of several values of ductilities and found that the final results were not sensitive to the baseline adjustments. The spectral displacements at long-periods are controlled by the peak ground displacement (see the Appendix in Akkar and Boore (2009) for a comprehensive discussion). In case of ICEARRAY data, it has been observed that the peak value of ground displacement is controlled by frequencies higher than those corresponding to the permanent displacement.
This implies that different correction schemes, though yielding different values of final displacements, do not make significant differences in the peak ground displacements, as is evident from Fig. 14. For the case shown in Fig. 20, the corrected peak ground displacements corresponding to the proposed scheme, Iwan et al. (1985); Wu and Wu (2007); Akkar and Boore (2009) are $-25.7$, $-25.6$, $-25.9$ and $-25.5$ cm, respectively. Note in Fig. 20 that the peak relative displacements corresponding to each correction scheme approach the corresponding values of peak ground displacements. This is the reason why the elastic as well as inelastic spectral displacements are not sensitive to the baseline correction schemes for the ICEARRAY data. This observation for the ICEARRAY data might not be valid as a general rule, especially when the permanent displacement is the peak ground displacement.

8 Conclusions

Long-period noise contamination in digital strong motion data poses difficulties in retrieving long-period motion and co-seismic deformations from near-fault stations. Although analogue data are more likely to contain frequent shifts in baseline, digital data are not free from them either. The presence of long-period noise should be removed as effectively as possible before any information regarding long-period motion is inferred from such records. There are several methods of baseline corrections proposed in the literature. Most of them are based on the work of Iwan et al. (1985) with slight modifications. There has been a significant effort in the research community to establish objective criteria in the model of Iwan et al. (1985) and make it more general. The result of such research has been partly successful, but each of them has a certain degree of subjective decision making during the correction process. We have illustrated the existing modifications of such methods and pointed out their limitations. In addition, we have proposed a simple criterion to objectively define the parameters of base-
line correction model. Our method gives satisfactory results when compared to the results of previous studies and independent GPS measurements where available. We conclude that the proposed method can be used with confidence. However, the application of this method should be performed while keeping in mind that there is no significant long-period noise in the later part of the record. Such situations would require a more complicated model for the baseline noise. Modelling multiple instant corrections by looking at linear trends in the raw velocity is one possibility. We have applied our method to some records of the 1999 Chi-Chi Earthquake and several recordings of the 2008 Ölfus Earthquake of South Iceland with satisfactory results.

**Acknowledgments** We are grateful to Dr. Yih-Min Wu for providing the raw data from the 1999 Chi-Chi Earthquake used in this study. We are grateful to the Icelandic Meteorological Office for providing the GPS estimates of coseismic displacements at their Hveragerdi and Selfoss stations. The first author is grateful to Professor A S Papageorgiou for his valuable ideas and the stimulus provided by him on this issue during a course in the University of Patras. We acknowledge the support from the University of Iceland Research Fund. The first author is supported by a grant from the University of Iceland for his doctoral study. Comments and suggestions of Professor John Douglas and Dr. Sinan Akkar resulted in important improvements to this article and are greatly acknowledged.

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Icelandic Meteorological Office (IMO) http://hraun.vedur.is/ja/gps.html


A note on the L'Aquila earthquake of 6 April 2009: Permanent ground displacements obtained from strong-motion accelerograms

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ABSTRACT

On 6 April 2009, 01:32 GMT, an Mw 6.3 earthquake hit the Abruzzi region of central Italy causing widespread damage in the City of L'Aquila and its nearby villages. The mainshock of this earthquake was recorded by 57 digital strong-motion instruments, four of which are located on the hanging wall of the Paganica Fault near L'Aquila. These stations are no more than 6 km from the epicentre. We use accelerometric data from these four stations to estimate permanent ground displacements caused by the mainshock. Our numerical results reveal south-east and downwards directed permanent co-seismic displacements which are in fair agreement with the outcomes of GPS and InSAR measurements reported in preliminary Istituto Nazionale di Geofisica e Vulcanologia (INGV) reports.

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1. Introduction

A strong earthquake of Mw 6.3 with a shallow focal depth hit central Italy near the City of L'Aquila on Monday, 6 April 2009 at 1:32 GMT. Most of the damage caused by the earthquake was in L'Aquila City although many surrounding villages, such as Paganica and Onna were severely affected. The earthquake caused 305 casualties and thousands of injuries, displaced more than 25,000 people and induced significant damage to more than 10,000 buildings in the L'Aquila region [1]. Older masonry structures as well as relatively modern reinforced concrete buildings were most severely damaged. This earthquake is seen as the deadliest one in Italy since the 1980 Irpinia earthquake. The L'Aquila earthquake is the mainshock of a seismic sequence that started on 30 March 2009 with a moment magnitude 4.4 event. The largest aftershock, measuring 5.6 on the moment magnitude scale, occurred on 7 April, a day after the mainshock. Focal mechanisms of the L'Aquila earthquake indicate normal faulting which, along with the geomorphology of the area, demonstrate an extensional system with active master, synthetic and antithetic normal faults [2,3].

Two hypotheses are presented regarding the seismo-tectonic structure that generated the mainshock; they apparently disagree with each other [2]. The first theory assumes that the Paganica Fault, a NW–SE trending normal fault dipping SW, generated the 6 April event and displaced far enough to reach the surface. This hypothesis alone, although plausible, may not satisfactorily explain the low surface displacements observed along the proposed causative fault, which is in the order of 10–12 cm, and the region of maximum downward surface displacement, which lies 3–4 km SW of the surface expression of Paganica Fault. The second hypothesis additionally considers the activation of a low-angle fault whose surface projection is represented by the Bazzano Fault. Bazzano Fault is another NW–SE trending normal fault located just NE of L'Aquila and is antithetic with respect to Paganica Fault. Bazzano Fault dips NE, intersecting the plane of Paganica Fault at a depth of about 2–3 km below the surface. The movement along Bazzano Fault might have been caused by passive remobilization [4].

2. Strong-motion records

The mainshock of the L'Aquila earthquake sequence was recorded by 57 digital strong-motion instruments. These instruments are a part of the Italian Accelerometric Network (RAN—Rete Accelerometrica Italiana) and are owned and maintained by Italian Department of Civil Protection (ITDPC). Four of these stations provide high-quality, near-fault accelerometric data. These stations, coded AQV, AQG, AQJ and AQK, lie on the hanging wall of Paganica Fault. The accelerographs at these stations have 24-bit A/D converters with 200 samples per second. High-quality digital acceleration data obtained at these stations make it possible to partly recover the co-seismic ground...
displacements caused by the mainshock, which is the main objective of this article.

3. Estimation of permanent displacements from strong-motion data

Recovering permanent ground displacements from broadband accelerometric data is an area of much interest among engineering seismologists and earthquake engineers. Permanent displacements and low-frequency motion can be accurately obtained from accelerometric data only if six components of ground motion (three translational and three rotational) are recorded [5–7]. It has been common practice to integrate individual translational acceleration time series to obtain corresponding ground velocity time series, which are further integrated to estimate ground displacement time series. With only translational components of acceleration available, velocities and displacements computed by integrating them are often in doubt. Inaccuracies in velocities and displacements so obtained can be attributed to a number of reasons. In the near-fault region, the movement of the instrument’s base is not purely translational and the recorded signal, in many cases, contains instrument response to rotational components of ground motion. These effects are known as tilts. Apart from tilts, digitized as well as digitally recorded accelerograms are contaminated by complicated shifts of zero acceleration level,

![Fig. 1. Velocity waveforms at AQA, AQV, AQK and AQG stations recording the 6 April 2009 L’Aquila Earthquake. The name of the station is shown in each plot: northwards, eastwards and upwards are taken as positive. Dashed and solid lines represent raw and corrected velocities, respectively. (a) north-south component, (b) east–west component and (c) vertical component.](image)
commonly known as the baseline. Baseline shifts are small but instantaneous offsets in acceleration zero-level. Although they are hardly visible in acceleration series, they amplify upon integration resulting in significant drifts in the integrated velocity and even more clearly seen in displacement time series, where they appear as linear and quadratic trends, respectively. An offset in acceleration as small as 0.01 cm/s², when integrated for 100 s, manifests into an error of 1 cm/s in the velocity and 100 cm in the displacement time series. Such effects make it almost impossible to estimate ground displacement from translational accelerometric data alone. A number of attempts have been made in the past to adjust recorded data for baseline variation with the objective of retrieving ground displacements (see for e.g., [8–13]). Adjustments for baseline shifts can almost never be done with certainty simply because the mechanisms resulting in these variations are not well understood at present. However, carefully designed baseline adjustment schemes have been successful in retrieving, at least partly, ground displacement time series, including permanent displacements.

Rupakhety et al. [14] designed a baseline correction scheme based on a model proposed by Iwan et al. [8] and used it to infer permanent ground displacements from some near-fault recordings of the 1999 Chi-Chi, Taiwan earthquake and the 29 May 2008 Ölfus, South Iceland earthquake [15]. This method is applied in

![Displacement waveforms at AQA, AQV, AQK and AQG stations recording the 6 April 2009 L’Aquila earthquake. The name of the station is shown in each plot: northwards, eastwards and upwards are taken as positive. Dashed and solid lines represent raw and corrected displacements, respectively. Note that a different scale is used for the AQK station, (a) north-south component, (b) east-west component and (c) vertical component.](image)
Fig. 3. Resulting permanent horizontal displacements estimated at four stations AQA, AQG, AQV and AQK, respectively. Dots represent locations of recording stations; triangles indicate places near L’Aquila, while the star depicts the epicentre. Approximate locations of Paganica and Bazzano faults are displayed by black dashed lines next to the respective places. The vectors originating at station locations reveal the direction, sense and magnitude of permanent horizontal displacements. The scale is indicated by the vector marked 5 cm.

Fig. 4. Projection of particle motion on the fault-normal and the fault-parallel planes. NW and NE are taken as positive for fault-parallel and fault-normal directions, respectively. The thick black lines represent the motion between 10 and 14 s. Note that a different scale is being used for the AQK station.
the following on 12 accelerometric records (3 components from each of AQV, AQG, AQK and AQA stations) from the L’Aquila earthquake to estimate co-seismic displacements time series.

4. Corrected velocities and displacements

The velocity and displacement waveforms obtained after applying the above-mentioned baseline correction procedure are presented in this section. The raw acceleration data used were obtained from ITDPC network [16]. It is highlighted that no digital filtering is performed on the acceleration time series. This is judged important because high-pass filtering may remove low-frequency components of the signal carrying information pertaining to permanent displacements.

Baseline corrected velocity waveforms are displayed in Fig. 1 for 12 recordings, two horizontal components and one vertical for each of the four stations considered in this study. Solid lines represent corrected velocities and dashed lines correspond to velocities obtained by integrating raw acceleration. By raw acceleration we mean acceleration time series obtained by removing the pre-event mean from the whole series. The pre-event as well as post-event portion of this pre-event-mean-reduced acceleration is extracted by visual inspection identifying the main event for subsequent analyses. For some records no significant baseline variations are visible, which is indicated by overlapping solid and dashed lines (for example recordings at AQV station, see Fig. 1), whereas some records, such as AQG N–S and E–W components, show considerable shifts in baseline manifested in the linear trends in velocity towards the end of the record (visualised by the dashed lines, see Fig. 1). The baseline correction scheme adopted [14] is effective in removing linear trends in integrated velocity time series and the velocities at the end of the records therefore approach zero as would be expected from theoretical considerations.

In Fig. 2, displacement waveforms obtained by integrating the baseline corrected acceleration time series twice are shown with solid lines. Dashed lines represent the displacements obtained by integrating the raw velocity time series defined in the previous paragraph. Note that the raw displacements show drifts in almost all cases, except for the vertical component of station AQK. Even for stations where differences in raw and corrected velocities (dashed and solid lines, respectively, in Fig. 1) are not prominent, the differences in displacements are more pronounced. All displacement waveforms obtained after applying the baseline correction scheme indicate permanent ground displacements towards the end of the record. At the location of the four stations considered in this work, the analysis reveals that the ground has moved permanently towards south-east and downwards as a result of the earthquake mainshock. Station AQK exhibits maximum downward permanent displacement of about 15 cm whereas the other three stations are associated with lower subsidence of about 4 cm.

The relative locations of stations AQA, AQV, AQK and AQG are plotted in Fig. 3 along with the mainshock epicentre. The location of the epicentre is taken from the latest information provided on the INGV website [17]. The numerical results visualise south-east oriented permanent horizontal displacements at these stations, which agree with the preliminary findings obtained from GPS measurements at nearby stations [18].
Of the four stations considered here, maximum subsidence of about 15 cm is observed at AQK. Interferometric analyses using InSAR data [19] indicate maximum ground subsidence of 25 cm in an area located between L'Aquila City and Fossa, a village southeast of L'Aquila. Station AQK is located just south of L'Aquila City. Higher ground subsidence at this station compared to AQA, AQV, and AQG is possibly due to its proximity to the earthquake epicentre as well as the causative fault (see Fig. 3).

The particle motion projected on the horizontal plane is displayed in Fig. 4, i.e., the displacement evolution with time at the four recording stations considered here. The displacements plotted in the graphs of Fig. 4 are projected in the fault-normal and the fault-parallel direction assuming the fault strike to be 126° (fault strike reported in [16]). For the fault-parallel direction, positive displacement implies movement towards NW and NE is taken as positive for the fault-normal direction. On the other hand, Fig. 5 illustrates the particle motion projected on a vertical plane normal to the strike of the fault. The stations AQA, AQV and AQG, which are located north-west of the City of L'Aquila exhibit initial eastward movement followed by a sudden downward movement, reaching a value of about 5 cm, after which the displacements oscillate primarily in the east–west direction. At the end of the event, these stations are displaced towards south-east and vertically downwards. The results reveal that the particle motion at AQK station is very different compared to that of AQA, AQV and AQG stations. In this case the initial motion is predominantly downwards, followed by oscillations in the strike–normal direction. The reasons for this behaviour are not yet clear but might be due to the location of this station being closer to the causative fault(s).

5. Conclusions

We have processed raw accelerometric data obtained at four near-fault stations recording the 6 April 2009 L'Aquila, Italy, earthquake. We applied a baseline correction scheme proposed by Rupakhety et al. [14] to accelerograms obtained at AQA, AQV, AQK and AQG stations of the Italian accelerometric database [16], with the objective of estimating permanent ground displacements. The resulting velocity and displacement waveforms indicate effective removal of baseline shifts in the recorded data; however, it should be kept in mind that the results are approximate, and complete information about low-frequency motion can only be obtained if translational as well as rotational components of ground motion are available. Our results indicate permanent ground displacements of about 5 cm in the south-east direction at the locations of these stations. Ground subsidence of about 4 cm is estimated at the locations of stations AQA, AQV and AQG north-west of L'Aquila City. Station AQK, which is located near the City of L'Aquila, is estimated to have a permanent downward displacement of about 15 cm. Our results are in fair agreement with permanent displacements inferred from GPS and InSAR data, which have been published by INGV in various reports following the earthquake. We have observed that there are significant changes in zero-levels (baselines) of recorded accelerations. In this study we have used only those stations close to the fault because our main objective was to estimate permanent ground displacements, which are not commonly observed in stations far away from the fault. Although stations, far away from the fault, might not be useful in estimating permanent ground displacement, their recordings should be carefully processed to estimate and remove variations in the acceleration baseline before velocity or displacement time series are inferred from them. We judge the baseline correction scheme used in this work efficient in fulfilling these objectives.

Acknowledgements

The raw acceleration data used in this study were obtained from the online database of Italian accelerometric network [16]. Various INGV reports cited in the references were studied and proved very useful in conducting this study. Furthermore, we acknowledge the support from the University of Iceland Research Fund. The comments made by two anonymous reviewers resulted in significant improvements of this article and are greatly acknowledged.

References

Paper 5

Quantification of ground-motion parameters and response spectra in the near-fault region

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Abstract This study focuses on the characteristics of near-fault ground motions in the forward-direction and structural response associated with them. These ground motions are narrow-banded in nature and are characterized by a predominant period at which structures excited by them are severely affected. In this work, predominant period is defined as the undamped natural period of a single-degree-of-freedom (SDOF) oscillator at which its 5% damped linear elastic pseudo-spectral velocity (PSV) contains a clear and dominant peak. It is found that a linear relationship exists between predominant period and seismic moment. An empirical equation describing this relationship is presented by using a large set of accelerograms. Attenuation equations are developed to estimate peak ground velocity (PGV) as a function of earthquake magnitude and source-to-site distance. In addition, a predictive equation for spectral shapes of PSV (i.e., PSV normalized by PGV) is presented as a continuous function of the undamped natural period of SDOF oscillators. The model is independent of PGV, and can be used in conjunction with any available PGV attenuation relation applicable to near-fault ground motion exhibiting forward-directivity effects. Furthermore, viscous damping of the SDOF is included in the model as a continuous parameter, eliminating the use of so-called damping correction factors. Finally, simple equations relating force reduction factors and displacement ductility of elasto-plastic SDOF systems are presented.

Keywords Near-fault ground motion · Forward-directivity · Elastic response spectra · Peak ground velocity · Iso-ductility spectrum · Force reduction factor
1 Introduction

Near-fault ground motions are known to be a potential cause of severe damage to engineering structures. They usually carry a strong long-period pulse in their velocity records. Directivity effects (see Somerville et al. 1997) and permanent displacement effects (see Abrahamson 2000) have been identified as the most common features of near-fault ground motions. This study is focused on forward-directivity effects in the near-fault region and characterization of elastic as well as inelastic response of single-degree-of-freedom (SDOF) systems.

After the devastating effects of the 1994 Northridge, California, earthquake, simulation of near-fault ground motions and their use in studying structural response gained widespread attention. A primary motivation was the observation that dominant pulses in the velocity records of near-fault ground motion resembled simple waveforms which could be represented by analytical expressions. Many engineers and researchers have used simple waveforms to represent typical velocity pulses observed in near-fault regions. Commonly used waveforms range from simple triangular pulses, square waves, and sinusoidal waves to wavelets of different types. Models proposed by Anderson et al. (1999), Heaton et al. (1995), Makris (1997), Alavi and Krawinkler (2000, 2004), Mavroeidis and Papageorgiou (2003), and Baker (2007) are some examples.

The most important parameters of these models are related to the amplitude and frequency of the velocity pulse. The amplitude of the pulse is representative of the peak ground velocity. Different authors have defined the pulse period in different ways. Nevertheless, they have all found that the pulse period is linearly related to the seismic moment. It has also been found that the pulse period is closely related to the SDOF period where $PSV$ attains its maximum value. If the pulse were a simple harmonic with infinite duration, the peak of $PSV$ would occur exactly at the pulse period. However, near-fault velocity pulses are of finite duration, defined by the pulse period and the number of half-cycles. Because of this, the period at which $PSV$ is the maximum is a fraction of the pulse period, where the fraction depends on the number of half-cycles of the pulse. Apart from this, the presence of other components of ground motion not associated with the pulse itself can also cause the $PSV$ to peak at a period different than the pulse period.

The objective of the present paper is to throw light on some of the characteristics of near-fault ground motions and structural response associated with them. We emphasize that the most important characteristics of forward-directivity motion in near-fault areas are peak ground velocity and frequency content. In most models of velocity pulses mentioned above, frequency content is represented by pulse period. However, we quantify the predominant frequency by the period where the $PSV$ of a 5% damped linear-elastic oscillator contains a clear and dominant peak. We present robust empirical equations to estimate $PGV$ and predominant period from earthquake size, source-to-site distance and other relevant parameters. We then discuss salient features of elastic response spectra of SDOF systems subjected to near-fault ground motions. We present analytical equations of $PGV$-normalized $PSV$ (termed here as spectral shapes) as a continuous function of the SDOF period and its level of viscous damping. Finally, characteristics of inelastic response are studied, and equations relating strength reduction factors to displacement ductility of elasto-plastic SDOF systems are presented.

2 Near-fault strong-motion data

Strong-motion data used in this study consist of acceleration records obtained from 29 different earthquakes. In total 93 records are analyzed, the details of which are presented in Table 8 in Appendix 2.
2.1 Data source

The data listed in Table 8 are collected from different sources. Most of the accelerograms are obtained from the NGA database (see Power et al. 2006), along with their associated metadata. Strong-motion records of the 17 and 21 June 2000 earthquakes and the 29 May 2008 Ölfus earthquake in South Iceland were obtained from the ISESD (Internet Site for European Strong-Motion Data) website (Ambraseys et al. 2004). The waveform for WID 90 (EERC basement, see Table 8) was obtained from the database of Earthquake Engineering Research Center, University of Iceland. Metadata related to the records of Icelandic earthquakes were calculated based on several publications, including Sigbjörnsson and Ólafsson (2004), Halldórsson et al. (2007), and Sigbjörnsson et al. (2009). Accelerograms of the 2004 Parkfield earthquake were obtained from CISN (California Integrated Seismic Network) database. In calculating the metadata for records from this earthquake, various publications were considered, including Shakal et al. (2005), Langbein et al. (2005), Liu et al. (2006), Dreger et al. (2005), and Kim and Dreger (2008). Accelerograms at AQK station during the 2009 L’Aquila earthquake were obtained from the online database of the Italian accelerometric network (Luzi et al. 2008). Since we are primarily concerned with forward-directivity effects in this study, permanent displacements (if any) in the ground motion records were removed by subtracting a half-sine pulse from the acceleration records, which was scaled to match permanent displacements computed by a procedure described in Rupakheti et al. (2010).

2.2 Description of metadata

Various parameters of strong-motion records listed in Appendix Table 8 were collected/computed from different sources. Each record is uniquely identified by its waveform identification number (WID). Records generated by the same earthquake share a common earthquake identification number (EID). The location of an earthquake, the date when it occurred, and the station recording the waveform are also listed. Station names reported in Mavroedi and Papageorgiou (2003) are indicated by abbreviation only, whereas other station names are shown in full. Faulting mechanism, earthquake magnitude (only moment magnitude used in this work), and the component of ground motion being considered are reported along with the peak ground velocity. Different distance metrics are reported, including epicentral distance \( r_{epi} \), and Joyner and Boore distance. Hypocentral depth is also reported when available. The length of fault between station and hypocenter \( (s) \) and the width of fault between station and hypocenter \( (d) \) are also presented. In addition, isochrone velocity ratio as defined by Spudich et al. (2004) is listed for records where it could be computed. Average shear wave velocity in the upper 30 m of the crust \( v_{s,30} \) is also provided. Velocity profiles at the stations recording Icelandic earthquakes are not available, and \( v_{s,30} \) for these stations are based on their Eurocode 8 (CEN 2004) classification. Parameters that could not be reliably estimated are indicated as ‘NA’. Figure 1 shows the distribution of data with respect to magnitude, distance, and faulting mechanism. The distance measure is selected as \( r_{JB} \) where available. In case this measure is not available, \( r_{epi} \) is used. Stations within 30 km from the source are considered.

3 Relationship between predominant period and seismic moment

Predominant period \( (T_d) \) in this work is defined as the period where 5% damped linear-elastic \( PSV \) reaches its peak value. If more than one peaks of comparable amplitude exist, then the
longest period is considered. The period of the velocity pulse is related to $T_d$. On average, the pulse period is about 84% of $T_d$ (Bray and Rodriguez-Marek 2004). An advantage of using $T_d$ is that, unlike pulse periods used in many simple pulse models, it can be unambiguously estimated. Since pulse period has been found to scale linearly with seismic moment, similar scaling for $T_d$ is expected. The relationship between $T_d$ and $M_w$ is modelled by

$$\log(T_d) = \alpha M_w + \beta + \epsilon$$

(1)

where $\alpha$, and $\beta$ are model coefficients determined by regression analysis; and $\epsilon$ is a Gaussian-distributed random variable with zero mean and standard deviation $\sigma$. The model of Eq. 1 was calibrated by using least squares regression. Records corresponding to sites having $v_{s,30}$ less than 240 m/s are not considered in regression analysis. The predominant periods related to directivity pulses are generally greater than 0.5s. In order that the soil response does not distort the characteristics of directivity pulses, we require the vibration period of the soil to be less than 0.5s. Using the quarter wavelength principle, and considering the upper 30m of the crust, the corresponding shear wave velocity is 240 m/s. The record from COG station of the 1991 Sierra Madre earthquake is of low quality, and could not be processed with confidence. Therefore, this record is not considered in regression analysis. Furthermore, the record from Parachute test site of the 1981 Westmorland earthquake, and the record from Petrolia station of the 1992 Cape Mendocino earthquake contain multiple peaks in their response spectra, making it difficult to identify $T_d$ with confidence. These records are not considered in regression analysis. The regression parameters were found to be $\alpha = 0.47$, $\beta = -2.87$, and $\sigma = 0.18$. Applying the maximum likelihood regression of Joyner and Boore (1993), we did not observe significant difference in the regression parameters for this dataset.
Fig. 2 Scaling of $T_d$ with $M_w$. The Solid line is the mean value of a least squares line fitted to the data, while the dashed lines correspond to mean $\pm 2\sigma$ levels.

The regression line and data points are presented in Fig. 2. The mean value predicted by regression is shown with the solid line, while dashed lines correspond to mean $\pm 2\sigma$ levels. The distribution of $\varepsilon$ is compared with a standard normal distribution in the small inset in the top-left corner of Fig. 2. Somerville et al. (1999) argue that self-similar scaling relationships constrain fault parameters. Such scaling implies that the period of predominant velocity pulse is two times the rise time of slip on the fault. This requires that the coefficient $\alpha$ be equal to 0.5. The coefficient obtained by us is 0.47, which is fairly close to 0.5, and indicates that if self-similar scaling is invoked, the definition of predominant period used by us can be considered as an efficient and unambiguous measure of the pulse period.

4 Attenuation equation for PGV

Empirical relations describing $PGV$ as a function of earthquake magnitude, source-to-site distance and other parameters, such as faulting mechanism and site conditions, are abundant in the literature (see Bommer and Alarcon 2006). However, few relations have been developed especially for near-fault conditions with forward rupture-directivity effects. One of the first attempts was made by Somerville (1998), called S98 hereafter, who assumed that $PGV$ varies with the square root of the closest distance to fault for stations located at least 3 km away from the fault. Bray and Rodriguez-Marek (2004), called B&R-M04 hereafter, emphasized the need to predict $PGV$ at distances closer to the fault than 3 km as well, and used a simple functional form to perform regression analysis of $PGV$ data against earthquake magnitude and the closest distance to rupture.
4.1 Attenuation model

The basic functional form adopted in this study is similar to the one used by B&R-M04 and is mathematically expressed as

\[
\log (PGV_{ij}) = a + bM_w + c \log (R^2 + d^2) + \eta_i + \varepsilon_{ij} \tag{2}
\]

where \(PGV_{ij}\) is the \(PGV\) of the \(j\)th recording from the \(i\)th event; \(M_w\) is the moment magnitude of event \(i\); \(R\) is the distance (measured in km) of the \(j\)th recording obtained from the \(i\)th event; \(a, b, c,\) and \(d\) are regression parameters; and \(\eta_i\) and \(\varepsilon_{ij}\) represent inter- and intra-event variations. The error terms \(\eta_i\) and \(\varepsilon_{ij}\) are assumed to be independent, normally-distributed random variables with variances \(\sigma_1^2\) and \(\sigma_2^2\), respectively. The total standard deviation associated with estimated \(PGV\) can be computed from the following equation.

\[
\sigma_t = \sqrt{\sigma_1^2 + \sigma_2^2} \tag{3}
\]

For distance measure \(R\), we use \(r_{JB}\) when rupture model is available and \(r_{epi}\) otherwise. Furthermore, we select only those stations within 30 km from the source. The data from COG station of the 1991 Sierra Madre earthquake is excluded in regression due to the reasons explained in Sect. 3 above. In addition, data obtained at sites with \(v_s, 30\) less than 240 m/s are excluded. This implies that the results obtained are applicable to those sites where significant soil effects are not expected. The decision to exclude soft soil data from regression was made because we found that the data was not sufficient to reliably constrain the soil effects by using site coefficients (or \(v_s, 30\)) in the regression model. We use base 10 logarithms throughout this work. The model of Eq. 2 was calibrated by using the maximum likelihood method of Joyner and Boore (1993). Regression constants and associated values of standard deviations are shown in Table 1. It was found that the magnitude coefficient \(b\) is very small, indicating that \(PGV\) is almost magnitude-independent in the near-fault region. Magnitude scaling obtained by us is much weaker than that obtained by S98 and Alavi and Krawinkler (2000), called A&K00 hereafter. B&R-M04 also observed that their magnitude scaling was weaker, and their magnitude scaling parameter, \(b\), in base 10 logarithmic units, is 0.15, which is close to our results.

Figure 3 compares \(PGV\) data with the model of Eq. 2 and model parameters of Table 1. For comparing attenuation of \(PGV\) with distance, we normalize observed \(PGV\) with a magnitude-scaling term (magnitude-normalized). For comparing scaling of \(PGV\) with magnitude, we normalize observed \(PGV\) with distance (distance-corrected). In Fig. 3a, attenuation of magnitude-corrected \(PGV\) is plotted against model prediction. It is seen that the model captures attenuation of \(PGV\) with distance reasonably well. Figure 3b shows scaling of distance-corrected \(PGV\) with magnitude. A quadratic magnitude term was added to the model of Eq. 2. This resulted in decrease of \(PGV\) at magnitudes larger than 7.0. To avoid this effect, we constrained the model in such a way that the magnitude scaling term becomes a constant at a certain magnitude, which was determined iteratively to ensure continuity. The final model is of the following form.

Table 1: Regression coefficients for the model of Eq. 2

<p>| | | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
<td>0.98</td>
<td>0.13</td>
<td>−0.10</td>
<td>0.66</td>
<td>0.099</td>
<td>0.132</td>
<td>0.17</td>
</tr>
</tbody>
</table>
Fig. 3  Comparison of the model of Eq. 2 with the data. (a) Attenuation of magnitude-corrected $PGV$ with distance. (b) Scaling of distance-corrected $PGV$ with magnitude

Table 2  Regression coefficients for the model of Eq. 4

<table>
<thead>
<tr>
<th></th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>e</th>
<th>$M_{sat}$</th>
<th>$\sigma_1$</th>
<th>$\sigma_2$</th>
<th>$\sigma_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$-5.17$</td>
<td>$1.98$</td>
<td>$-0.14$</td>
<td>$-0.10$</td>
<td>$0.75$</td>
<td>$7.0$</td>
<td>$0.081$</td>
<td>$0.135$</td>
<td>$0.16$</td>
</tr>
</tbody>
</table>

\[
\log (PGV_{ij}) = \begin{cases} 
  a + bM_w + cM_w^2 + d \log (R^2 + e^2) + \eta_i + \epsilon_{ij} & \text{if } M_w \leq M_{sat} \\
  a + bM_{sat} + cM_{sat}^2 + d \log (R^2 + e^2) + \eta_i + \epsilon_{ij} & \text{otherwise}
\end{cases}
\]  

(4)

The results of regression analysis are shown in Table 2, and the comparison of data with the model is shown in Fig. 4. In the legend of Fig. 4, $F(M_w)$ is the magnitude term of Eq. 4, which is equal to $a + bM_w + cM_w^2$ for $M_w \leq M_{sat}$ and $a + bM_{sat} + cM_{sat}^2$ otherwise. With the modified model, the standard deviation of residuals is slightly reduced. It should be noted that the use of the model for magnitudes below 5.5 is not recommended.

4.2 Directivity predictor

Another important aspect we investigate in this work is the effect of directivity. A parameter to quantify the amount of directivity at a given station for a given event is required. Somerville (2000) proposed the ratio of the length of fault between site and hypocenter and the site azimuth as a measure of directivity. Spudich et al. (2004) showed that the isochrone velocity ratio (i.e., average isochrone velocity normalized by shear wave velocity) behaves similarly to the directivity parameters of Somerville (2000). Isochrone velocity ratio $\tilde{c}$ was computed for most of the stations used in this study. For those records, where reliable information regarding the rupture surface is not available, $\tilde{c}$ cannot be estimated. In a recent study Spudich and Chiou (2008), called S&C08 hereafter, proposed a directivity predictor, based on isochrone velocity ratio and hypocentral radiation pattern. We attempt to apply their idea in our attenuation model of $PGV$. The directivity predictor we use is designated $DP$ (directivity predictor) and is given by
Fig. 4 Comparison of the model of Eq. 4 with data. (a) Attenuation of magnitude-corrected PGV with distance. (b) Scaling of distance-corrected PGV with magnitude

\[
DP = CS\mathcal{R}
\]

\[
C = \frac{\min (\tilde{c}, 2.45) - 0.8}{2.45 - 0.8}
\]

\[
S = \log [\min (75, \max (s, d))]
\]

where \(s\) and \(d\) are as defined in Sect. 2.2, \(\tilde{c}\) is isochrone velocity ratio as defined above, and \(\mathcal{R}\) is scalar radiation pattern amplitude, as defined in S&C08. We use strike-normal or strike-parallel hypocentral radiation pattern with a water level of 0.2 at the nodes, depending upon which component is being considered. For further details on radiation patterns and the reasoning behind Eqs. 6–8, readers are referred to S&C08.

We found that the effect of \(\mathcal{R}\) in the directivity predictor is not significant. At sites very close to the fault, the radiation is dominated by a small area of the fault closest to the station. This might be the reason why the radiation pattern based on the hypocenter is not correlated strongly to PGV data being considered here, which come predominantly from stations within a few kilometers from the fault. Therefore, we drop the term \(\mathcal{R}\) in Eq. 6.

Figure 5 shows the correlation between the directivity predictor \(DP = CS\mathcal{R}\) and the residuals of the model presented in Eq. 4. It can be seen that the correlation is very weak. The correlation coefficient was found to be 0.05. The correlation was found to increase to a maximum value of 0.1 when the number ‘2.45’ in Eq. 7 was changed to 1.75. Changing the capping distance, used as 75 km in Eq. 8, did not result in an improved correlation between the directivity predictor and the residuals. Given such weak correlation, we judge that adding the directivity term in the model does not result in an improvement of the model.

These results indicate the limitations of the directivity parameter being considered, however, a general conclusion about their usefulness should not be drawn only based on these results, which are obtained from a limited amount of data. It is also to be noted that the directivity parameter considered here is based on average isochrone velocity. While \(\tilde{c}\) could be modified to the actual isochrone velocity closest to station instead of the average isochrone velocity over the rupture area, such an approach is helpful only if the rupture process is well understood. For future predictions, the heterogeneity of slip (amplitude and velocity) over
the rupture area is impossible to predict with the current state of knowledge; thus, using the exact isochrone velocity is of little use.

4.3 Comparison with other attenuation models

In Fig. 6 we compare the model of Eq. 4 and the associated parameters of Table 2 with models proposed by B&R-M04, S98, A&K00, and Halldorsson et al. (2010), hereafter called HM&P10. These authors use the closest distance to rupture as their distance metric. For comparing their model with ours, we assume a vertical strike-slip event. The thick black line in Fig. 6 corresponds to the mean prediction of the proposed model, while upper and lower fractals with 2 standard deviations are shown by black dashed lines. Circles indicate observed values of $PGV$. $PGV$ corresponding to all magnitudes are shown, and the model predictions are computed at magnitude 6.6, which is also the mean magnitude of our data. The mean prediction of B&R-M04 is shown with the solid blue line. The dashed red line, the dashed blue line, and the solid red line represent mean predictions of S98, A&K00, and HM&P10, respectively.

We note that the magnitude scaling parameters of A&K00 and S98 are high, and our data do not support such a strong magnitude dependence of $PGV$. On the other hand, magnitude scaling is zero in HM&P10. The attenuation of $PGV$ above distances greater than 7 km is very fast in the Model of Bray and Rodriguez-Marek. Fast attenuation in HM&P10’s model is related to their functional form. In their model, $PGV$ attenuates exponentially with distance. Such exponential attenuation is not supported by our data, as shown in Fig. 6.

5 Elastic response spectra

Earthquake response spectra, first introduced by Biot (1933), are widely used for designing and assessing structures subjected to strong ground motion. Following the works of Housner
Fig. 6  Comparison of the model of Eq. 4 with observed data (circles) and the models of other authors as indicated in the legend (see text above for legend keys)

(1959) and Newmark et al. (1973), it has become a standard tool to characterize important features of earthquake accelerograms and to evaluate structural response to earthquake-induced ground shaking. Modern design codes for earthquake resistance specify seismic action on structures in terms of design spectra, a statistical representation of response spectra constructed from accelerograms recorded during past earthquakes. Because a majority of accelerograms are recorded far away from causative faults, code-specified design spectra are dominated by response spectra of ground motion in the far-fault region.

Accelerograms from large earthquakes in the recent past have shown that response spectra of near-fault ground motions, mainly of those affected by forward-directivity effects, are different from those of far-fault ones. One of the characteristic differences between the two is the narrow-banded spectral structure of the former. Response spectra of forward-directivity-affected near-fault ground motion exhibit spectral peak values in a narrow band of periods near the predominant period of ground motion. Predominant period increases with increasing earthquake magnitude, as discussed in Sect. 3. Such magnitude scaling has two important implications for elastic response spectra. First, the acceleration response of moderate-to-large earthquakes is stronger than that of large earthquakes in the high-frequency region; the trend being reversed at longer periods. Second, peak spectral accelerations of moderate-to-large earthquakes are larger than those of very large earthquakes (Somerville 2000; Mavroeidis et al. 2004).

The differences in the response of structures to near-fault and far-fault ground motions imply that design spectra, derived from more far-fault accelerograms than near-fault ones, are biased. They are not capable of capturing the impulsive nature of near-fault ground motion and often lead to unreliable estimates of seismic action on engineering structures located near an earthquake fault. Therefore, it is essential to develop design spectra specifically
suitable for ground motion in the near-fault area. One of the first systematic efforts was made by Somerville et al. (1997). They proposed broad-band amplification factors for spectral ordinates of ground motion prediction models. The narrow-banded nature of pulse-like ground-motion, however, suggests that a more accurate model would amplify spectral accelerations only in a narrow spectral band close to the predominant period of ground motion. We also note that most ground-motion prediction models are not exclusively based on non-pulse-like ground motion. Their calibration is usually performed by using pulse-like as well as non-pulse-like ground motion. Amplification factors for existing ground motion prediction models would be more meaningful if they were calibrated only from non-pulse-like ground motions. In this study, we develop response spectral model applicable strictly to near-fault ground motion exhibiting forward-directivity effects. The proposed model is based on recorded accelerograms within 30 km from the fault generated by earthquakes ranging in magnitude between 5.5 and 7.6. This model should not be extrapolated in terms of earthquake magnitude or source to site distance.

5.1 Elastic response spectral shapes

In the following, the term ‘elastic spectra’ is used for under-damped linear elastic pseudo-spectral velocity, $\text{PSV}$, and the term ‘spectral shape’ is used for $\frac{\text{PSV}}{\text{PGV}}$ normalized by the peak ground velocity, $\text{PGV}$. The predominant period ($T_d$) is as defined in Sect. 3, and increases with increasing earthquake magnitude. We present an illustration of this scaling effect on spectral shapes in Fig. 7.

![Fig. 7 Spectral shapes of near-fault ground motion grouped into six magnitude bins. The range of magnitude in each bin is shown above the plots. Grey lines are spectral shapes of individual accelerograms, while thick black lines correspond to mean values. The dashed vertical line indicates the peak of mean spectral shape](image-url)
In Fig. 7, 5% damped spectral shapes are presented. Only those records included in $M_w - T_d$ scaling presented in Sect. 3 are considered. To illustrate the effect of magnitude on spectral shapes, ground motions are grouped into six distinct magnitude bins as indicated in the figure. The bins are listed in Table 3 along with bin magnitude ranges and the number of records falling into each bin. An ideal bin should be as narrow as possible to represent the continuous nature of magnitude but near-fault ground motion records are not abundant and bin classification is therefore dictated by lack of data. Due to this limitation, we were forced to use non-uniform magnitude bins as shown in Table 3. One of the characteristic features of spectral shapes presented in Fig. 7 is the narrow velocity-sensitive region. Unlike far-fault spectral shapes, near-fault ones have relatively wider acceleration-sensitive and displacement-sensitive regions but a narrower velocity-sensitive region. This narrow velocity-sensitive region is where the peak of $PSV$ generally occurs. The location of peak is shifted to the right (longer periods) as earthquake magnitude increases. As is evident from Fig. 7, peak locations vary from about 0.8 s to about 5.5 s for bins 1 and 6, respectively. Careful examination of the spectral shape of bin 5 indicates an apparent contradiction, with its peak occurring at a smaller natural period than that of bin 4. However, this bin shows a wider velocity-sensitive region ranging from about 1 s to about 3 s with almost-constant ordinates. The reason for this wider velocity-sensitive region is the relatively large size of this bin (wider range of magnitudes), which could not be avoided due to the lack of data. It should be noted that although bins 1 and 5 appear to have the same size as defined in Table 3, their effective size are not the same. Bin 1 contains records from earthquakes having magnitudes 5.74 to 6.0, while Bin 5 contains records from earthquakes having magnitudes 6.8 to 7.28.

The magnitude dependence of spectral shapes observed in Fig. 7 is consistent with the notion that larger earthquakes have a richer energy content in the long-period range. This also implies that, at short periods, response spectra of moderate-to-large earthquakes in the near-fault region are higher than those of large earthquakes. Figure 8 clearly demonstrates this effect. Gray lines in the figure represent the $PSV$ of ‘individual’ records. Solid and dashed lines represent mean $PSV$ of the first and the sixth bin, respectively. It is evident that smaller earthquakes have, on average, higher spectral ordinates at short periods than the larger ones, the trend being reversed at longer periods. In addition, peak amplitudes of average spectral shapes of the two bins are comparable. This implies that peak response spectral ordinates of small earthquakes are comparable to those of larger ones but occur at different periods. This is a remarkable feature of the response spectra of near-fault ground motion. These properties of spectral shapes allow them to be modelled by simple continuous functions of $T_n$ and earthquake magnitude. In the following, we develop such a model.

5.2 Equations for elastic response spectra

In most modern applications, ground motion prediction equations (GMPEs) are used to obtain elastic response spectra for design and risk assessment of structures. GMPEs for 5% damped
linear elastic PSA (pseudo spectral acceleration) at discrete values of $T_n$ are abundant in the literature. Some examples of such equations are Abrahamson and Silva (1997), Boore et al. (1997), Sadigh et al. (1997), Ambraseys et al. (2005), Akkar and Bommer (2007) etc., more recent ones being the Next Generation Attenuation (NGA) equations (see Abrahamson et al. 2008). A comprehensive report on ground-motion prediction equation is given in Douglas (2003). Similar equations for inelastic response are given in Rupakheti and Sigbjörnsson (2009a,b) and Bozorgnia et al. (2010). These conventional GMPEs are associated with a matrix of model coefficients at discrete values of $T_n$.

A different approach to predicting spectral accelerations has recently been adopted by Graizer and Kalkan (2009). They proposed ground-motion spectral shapes for 5% damped PSA normalized by peak ground acceleration ($PGA$). Their spectral shapes are continuous functions of $T_n$. Such an approach significantly facilitates implementing the model. Another distinct advantage of this approach lies in the option to use spectral shapes in conjunction with any attenuation model of $PGA$, thus allowing the use of various models of attenuation relations to estimate response spectra.

We use a similar approach in developing spectral shapes for pulse-like ground motion. It is assumed that spectral shapes depend on variables such as earthquake magnitude, source-to-site distance, shear wave velocity at recording sites, basin effects, etc. For a description of different factors controlling spectral shapes, readers are referred to Graizer and Kalkan (2009). We assume that spectral shapes are predominantly influenced by magnitude of an earthquake and do not consider distance dependence in the near-fault zone. Whereas Graizer and Kalkan (2009) used $PGA$-normalized $PSA$ as their spectral shapes, we consider $PSV$ normalized by $PGV$. This does not limit the applicability of the model developed herein because $PSV$ and $PSA$ are related through $T_n$. Once spectral shapes are available, they can be scaled with $PGV$ to obtain $PSV$, which is easily converted to $PSA$. The attenuation relation of $PGV$ for ground motion records considered here has already been presented in Sect. 4.

5.3 Model function for spectral shapes

We use an empirical approach similar to that of Graizer and Kalkan (2009) in finding a suitable function for spectral shape. The suggested model is defined by a sum of two func-
tions $F_1 (T_n, M_w)$ and $F_2 (T_n, M_w)$. Both of these functions are continuous with respect to the oscillator period ($T_n$) for a given magnitude ($M_w$). The first function is a modified ‘log-normal type’ function and is mathematically expressed as

$$F_1 (T_n, M_w) = I_1 (M_w) \exp \left[ -0.5 \left\{ \frac{\ln (T_n) + C (M_w)}{W (M_w)} \right\}^2 \right] T_n \quad (9)$$

where $I_1 (M_w)$ controls the amplitude of this bell-shaped function, while its location and scale are governed by $C (M_w)$ and $W (M_w)$, respectively. The second function, which is a modified form of SDOF transfer function, is expressed by the following mathematical equation

$$F_2 (T_n, M_w) = I_2 (M_w) \left\{ 1 - \left( \frac{T_n}{T_m} \right)^\theta \right\}^2 + 4D_m^2 \left( \frac{T_n}{T_m} \right)^\theta \right\}^{0.5} T_n \quad (10)$$

where $I_2 (M_w)$ controls the overall amplitude of this function. This function is linearly increasing at short periods and exhibits a bump peaking close to $T_m$ - the amount of bump is controlled by $D_m$. The rate of decay of $F_2$ at long periods is controlled by $\theta$. Both $D_m$ and $\theta$ depend on $M_w$. With magnitude dependence implied by binning, we drop $M_w$ from our equations with the understanding that the parameters are calibrated independently for each magnitude bin. By summing $F_1 (T_n)$ and $F_2 (T_n)$, spectral shape for normalized PSV, denoted by $PSV_n$, is given by the following equation.

$$PSV_n = \left\{ I_1 \exp \left[ -0.5 \left( \frac{\ln (T_n) + C}{W} \right)^2 \right] + I_2 \left\{ 1 - \left( \frac{T_n}{T_m} \right)^\theta \right\}^2 + 4D_m^2 \left( \frac{T_n}{T_m} \right)^\theta \right\}^{0.5} T_n \quad (11)$$

5.4 Calibration of spectral shapes

The model spectrum, Eq. 11, is calibrated against the mean spectral shapes of different magnitude bins. Non-linear optimization, and verification by visual inspection, is used in estimating the parameters.

5.4.1 Model parameters

At the first step, the parameter $\theta$ is constrained. This parameter controls the slope of the spectrum at long periods. At long periods, the response spectra computed from ground motion records is not reliable due to the effects of low-pass filtering that is commonly employed in data processing. In this work we constrain the slope of the long period spectra (i.e. $\theta$ in our model) by the asymptotic behaviour of the Fourier near-fault source spectrum (see, for instance, Brune 1970) using the random vibration theory. The procedure is outlined in more detail in Appendix 1. Following this approach, $\theta$ was constrained to a value of 2.0. Note that the reliable period (dictated by the quality of data) up to which the proposed spectral shapes can be used depends on the earthquake magnitude. As a general recommendation, the spectral shapes proposed here can be used for SDOF periods less than about 2 times the predominant period. Nevertheless, we present the results for periods up to 10 seconds for all magnitude
Table 4  Parameters of Eq. 11
for different magnitude bins

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<thead>
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<th>Bin</th>
<th>$I_1$</th>
<th>$T_m$</th>
<th>$D_m$</th>
<th>$I_2$</th>
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<td>1</td>
<td>1.43</td>
<td>0.85</td>
<td>0.50</td>
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</tr>
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<td>2</td>
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<td>1.35</td>
</tr>
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<td>3</td>
<td>0.95</td>
<td>1.55</td>
<td>0.62</td>
<td>1.27</td>
</tr>
<tr>
<td>4</td>
<td>0.88</td>
<td>1.80</td>
<td>0.65</td>
<td>1.10</td>
</tr>
<tr>
<td>5</td>
<td>1.35</td>
<td>2.45</td>
<td>0.67</td>
<td>0.94</td>
</tr>
<tr>
<td>6</td>
<td>0.95</td>
<td>4.95</td>
<td>0.79</td>
<td>0.54</td>
</tr>
</tbody>
</table>

bins, and emphasize that the long period part of the spectrum is approximated by the procedure outlined in Appendix 1. While we recognize the possible errors in estimating response spectral ordinates at such long periods, we believe that these errors are not critical in terms of engineering application of the proposed model. One might argue that the long-period spectral ordinates are critical for very tall and flexible structures as they have long fundamental period of vibration. But, given the very small amplitudes of spectral ordinates at these periods, the total response of such structures is most likely controlled by higher modes of vibration, more so if inter-story drift demands are considered as the response quantity of interest.

The results obtained by calibrating the remaining parameters showed that $W$ and $C$ can be taken as constant quantities equal to 1 and 1.4, respectively. The other parameters are shown in Table 4. The quality of the fit between simulated shapes (i.e., spectral shapes approximated by Eq. 11) and mean spectral shapes of actual ground motion is presented in Fig. 9. Simulated shapes are represented in Fig. 9 by thin dark lines. Mean spectral shapes, on the

![Fig. 9 Comparison of simulated spectral shapes with mean spectral shapes](image-url)
other hand, are plotted as thick gray lines. It is observed that simulated spectral shapes match mean spectral shapes very well.

5.4.2 Magnitude dependence of model parameters

Careful examination of Table 4 shows that some of the parameters listed are strongly correlated with the average magnitude of their corresponding bins. The model being proposed could possibly be simplified by expressing some of these parameters in terms of $M_w$. As an average measure of $M_w$, we select the mean value in a bin and explore its relation with different parameters listed in Table 4.

We found a strong relation between $T_m$ and mean magnitude. The mean magnitudes for the bins selected in this study are 6.0, 6.2, 6.5, 6.7, 6.9, and 7.6, respectively, for bins 1, 2, 3, 4, 5 and 6. Their relation with $T_m$ is shown in Fig. 10. Note that the vertical axis is in logarithmic scale. Grey circles in the figure represent the values of $T_m$ from Table 4 plotted against the mean bin magnitude. The relation between $T_m$ and median magnitude was found to be very similar to the relation between predominant period and magnitude (see Eq. 1). For comparison, this equation is shown with the black line in Fig. 10. This is expected following the definition of the predominant period. The figure clearly shows that $T_m$ can be taken as equal to $T_d$ whose mean value is estimated from Eq. 1. These observations allow us to simplify the spectral shape model with the following equation.

$$PSV_n = \left[I_1 \exp \left\{-0.5 \left( \ln \left( \frac{T_n}{T_d} \right) + 1.4 \right)^2 \right\} + I_2 \left\{1 - \left( \frac{T_n}{T_d} \right)^2 \right\}^2 + 4D_m^2 \left( \frac{T_n}{T_d} \right)^2 \right]^{-0.5} T_n \right]$$

We emphasize that there are only three free parameters in Eq. 12 and all others have been constrained. We recalibrated the remaining parameters and found a strong relationship between mean bin magnitude and $I_2$, which is displayed in Fig. 11. Their relationship can be described by the straight line plotted in Fig. 11. The equation of the fitted line is also presented.
The free parameters of Eq. 13 were recalibrated, and their values are presented in Table 5. Finally, the comparison of approximate spectral shapes computed by using Eq. 13 with its parameters from Table 5 and mean spectral shapes is displayed in Fig. 12. These results indicate that the proposed model can capture the salient features of spectral shapes of forward-directivity-affected near-fault ground motion as a continuous function of $T_n$. The proposed model is simple and involves only two free (independent) parameters. It also considerably facilitates constructing the spectral shapes due to the continuous nature of the function with respect to $T_n$. We recognize that spectral shapes, to some degree, are affected by source-to-site distance, local soil conditions, and basin-generated effects. Whereas these additional parameters might be useful in applying more robust constraints on spectral shapes, the instances of recorded ground motion with forward-rupture directivity in the near-fault region are not sufficient to model these effects.

Table 5 Parameters of Eq. 13 for different magnitude bins

<table>
<thead>
<tr>
<th>Bin</th>
<th>$I_1$</th>
<th>$D_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.43</td>
<td>0.46</td>
</tr>
<tr>
<td>2</td>
<td>1.07</td>
<td>0.52</td>
</tr>
<tr>
<td>3</td>
<td>0.95</td>
<td>0.58</td>
</tr>
<tr>
<td>4</td>
<td>0.88</td>
<td>0.65</td>
</tr>
<tr>
<td>5</td>
<td>1.27</td>
<td>0.79</td>
</tr>
<tr>
<td>6</td>
<td>1.11</td>
<td>0.77</td>
</tr>
</tbody>
</table>

in the figure. With these additional constraints applied to $I_2$, the approximate spectral shape is given by:

$$PSV_n = I_1 \exp \left\{ -0.5 \left( \ln (T_n) + 1.4 \right)^2 \right\} + (4.92 - 0.58M_w) \left( 1 - \left( \frac{T_n}{T_d} \right)^2 \right)^2 + 4D_m^2 \left( \frac{T_n}{T_d} \right)^2 \right\}^{-0.5} T_n$$

(13)
The results presented in Sect. 5 correspond to SDOF systems with a viscous damping ratio equal to 5% of critical damping. The conventional approach for creating response spectra for other values of damping has been to modify 5% damped spectra with the so-called damping correction factors (see, for example, Lin and Chang 2003, and references therein). Although such an approach is popular and easy to apply, we believe they involve certain uncertainties. Because highly-damped spectra are smoother than those corresponding to low damping values, the variation of individual spectra from the mean of an ensemble of ground motion decreases as damping level is increased. The approach of modifying 5% damped spectra, on the other hand, not only propagates the uncertainty involved with 5% damped spectra but also introduces additional uncertainty due to simplified damping reduction factors. We present an alternative approach to create response spectral shapes for different levels of viscous damping by evaluating the parameters of the spectral shape model as a function of viscous damping ratio. To accomplish the task, we computed spectral shapes for different values of critical damping ratio ($\zeta$), namely 2%, 5%, 7%, 8%, 10%, 12%, 14%, 17%, and 20%. For damping ratios above 20%, pseudo-spectral velocity tends to differ from spectral velocity, and we therefore limit our analysis to 20% of critical damping.

Initial results indicated that $W$, $C$, and $T_m$ are not dependent on the level of damping. It was also found that $I_2$ is not strongly affected by damping level and can be computed from...
the equation shown in Fig. 11. The remaining parameter $I_1$ displayed strong relationship with damping level. In Fig. 13, their relationship is displayed for magnitude bin 1. Results for other bins are similar, and are not shown. It is observed that $I_1$ and $\zeta$ are related by an equation of the form $I_1 = a\zeta^{-0.5}$, where $a$ is evaluated separately for each bin by fitting a least squares regression line, as shown in Fig. 13 for the first bin. In the proposed model, $I_1$ contributes to the amplitude of spectral shapes mostly at intermediate frequencies. The power law type of scaling observed for $I_1$ is similar to the $\sqrt{1/\zeta}$ type of relationship commonly used as damping correction factors.

With the value of $I_1$ constrained as discussed above, the remaining parameters $D_m$ and was recalibrated. It was observed that $D_m$ increases as the damping level increases. In the proposed model $D_m$ controls the peak of the spectral shape (lower $D_m$ results in higher peak) around a narrow band close to the period where $PSV_n$ is the maximum. It is well known that the effect of damping is to decrease the amplitude of spectral ordinates near the resonant period. In this sense, the observed relation between $D_m$ and $\zeta$ is expected. It was found that a liner relationship of the form $D_m = c + d\zeta$ can be used to capture the dependence of $D_m$ on $\zeta$. The relation between $D_m$ and $\zeta$, along with least squares lines and their equations, are shown in Fig. 14 for all magnitude bins. These developments make it possible to estimate the spectral shapes for different levels of damping from 2% to 20% and for different magnitudes by using Eq. 13 and its parameters listed in Table 6. The critical damping ratio should be expressed as a fraction in using Table 6 (for example, use $\zeta = 0.02$ if viscous damping ratio is 2% of the critical level).

Finally, we present the comparison between the mean spectral shapes of the different magnitude bins against the simulated shapes using Eq. 13 and its parameters from Table 6 for critical damping ratios of 2% and 20% in Figs. 15 and 16, respectively. The result for 5% of critical damping remains the same as in Fig. 12.
Fig. 14  Relationship between $D_m$ and viscous damping ratio

<table>
<thead>
<tr>
<th>Bin</th>
<th>$I_1$</th>
<th>$D_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$0.320\zeta^{-0.5}$</td>
<td>$1.54\zeta + 0.39$</td>
</tr>
<tr>
<td>2</td>
<td>$0.239\zeta^{-0.5}$</td>
<td>$1.73\zeta + 0.44$</td>
</tr>
<tr>
<td>3</td>
<td>$0.211\zeta^{-0.5}$</td>
<td>$2.41\zeta + 0.47$</td>
</tr>
<tr>
<td>4</td>
<td>$0.204\zeta^{-0.5}$</td>
<td>$2.82\zeta + 0.50$</td>
</tr>
<tr>
<td>5</td>
<td>$0.283\zeta^{-0.5}$</td>
<td>$4.18\zeta + 0.58$</td>
</tr>
<tr>
<td>6</td>
<td>$0.242\zeta^{-0.5}$</td>
<td>$3.38\zeta + 0.59$</td>
</tr>
</tbody>
</table>

7 Handling uncertainties in spectral shapes

For probabilistic applications, it is beneficial to have a measure of uncertainty in estimated ground motion parameters. Most attenuation relations come with an estimate of standard deviation of residuals from regression analysis. To estimate the uncertainty in $PSV$, it is necessary to have an estimate of standard deviation of $PSV_n$ residuals. In this section we provide such a measure. The residuals in this context are the deviations of response spectral shapes of individual records from the value computed by Eq. 13. We use magnitude of individual records in Eq. 13 to compute spectral shapes and their residuals. Note that the two parameters $I_1$ and $D_m$ could not be expressed as a continuous function of $M_w$, and are known for mean bin magnitudes only. In computing the spectral shapes for individual records, we use these parameters corresponding to the mean magnitude of the bin these records lie in. This introduces some approximation which could be minimized if more data are available to construct small magnitude bins. Residuals are computed and specified at base 10 logarithmic scale at different values of $T_n$. The standard deviation of these residuals is termed $\sigma_{log \, PSV_n}$ and is plotted in Fig. 17 as a function of $T_n$ for three different levels of damping. Note that the $T_n$ axis is in logarithmic scale. The figure clearly shows that standard deviation reduces as the damping level increases. This is because as damping ratio increases, spectral shapes become smoother. The decrease in the standard deviation of residuals with increasing damping level
Fig. 15  Comparison of 2% damped mean spectral shapes with the ones simulated by Eq. 13 and Table 6

is the largest in the high-frequency region. We found that uncertainty in spectral shapes predicted by our model is smallest in the range $0.2s < T_n < 4s$. This is, in most cases, the period range of greatest interest for engineering design. We also notice that the variation of $\sigma_{\log PSV_n}$ with $T_n$ can be approximated by a simple curve as represented by the black line in Fig. 18 for 5% damped systems. The equation related to this approximation is the following.

$$\sigma_{\log PSV_n} = \begin{cases} 
0.18 - 0.04 \sin [2.9 (\log T_n - 1.7)] & \text{if } -1.73 < \log T_n < 1.0 \\
0.16 & \text{if } \log T_n \leq -1.73
\end{cases} \quad (14)$$

We suggest this approximation function, which is similar to the computed values of $\sigma_{\log PSV_n}$, to avoid a long list of $\sigma_{\log PSV_n}$ at discrete $T_n$ values. Equation 14 corresponds to 5%-damped systems. For higher levels of damping, the standard deviations are smaller. On average it was found that standard deviation for damping ratios of 0.02, 0.07, 0.08, 0.1, 0.12, 0.14, 0.17, and 0.2 were 1.06, 0.98, 0.97, 0.95, 0.93, 0.92, 0.90, and 0.88 times that for 5% damped system respectively. In order to compute the standard deviation of $PSV$ from $\sigma_{\log PSV_n}$ and $\sigma_{\log PGV}$, the correlation between $PSV_n$ and $PGV$ need to be established. The computed coefficients of these correlation for our data was found to be negative ($-0.2$ to $-0.3$) between SDOF periods of 0.01s and 1s beyond which it increased to 0.3 at a SDOF period of 2s, and then decreased steadily to 0 at about 10s. Because these coefficients are small, $PSV_n$ and $PGV$ can be assumed to be uncorrelated, and the uncertainty in $PSV$ can be approximated from the following equation.

$$\sigma_{\log PSV} = \sqrt{\left(\sigma_{\log PSV_n}\right)^2 + \left(\sigma_{\log PGV}\right)^2} \quad (15)$$
Fig. 16 Comparison of 20% damped mean spectral shapes with the ones simulated by Eq. 13 and Table 6

Fig. 17 Standard deviation of residuals of spectral shapes for three different levels of viscous damping
In Eq. 15, $\sigma_{\log PGV}$ can be computed from any appropriate attenuation equation for $PGV$, such as the one proposed in Sect. 4.2. The value of $\sigma_{\log PGV}$ computed in Sect. 4.2 was 0.16. Considering this, $\sigma_{\log PSV}$ varies from about 0.21 to 0.27.

In Fig. 19, the distribution of $PSV_n$ residuals is compared with standard normal distribution for nine different values of $T_n$, as indicated at top of each plot, and for a 5% damped system. The residuals are found to have zero mean in general although there are some differences between the residual distribution and normal distribution. Such differences are commonly experienced and may possibly be due to lack or non-uniform distribution of data. For practical application, an assumption of normal distribution for $\sigma_{\log PSV}$, i.e., log-normal distribution for $PSV_n$ seems reasonable from the results presented in Fig. 19.

The model that we have developed here is valid only in the near-fault area (within 20–25 km from the fault) and within the range of earthquake magnitudes considered in this study (5.5 to 7.6 in moment magnitude scale). Furthermore, the model is applicable only if forward-directivity effects are expected. If, in hazard or risk analysis, an analyst uses the proposed model along with other models to account for scenarios not applicable to our model, abrupt changes in scaling of ground motion quantities with respect to distance can result. This leads to the common dilemma of finding a rational way of combining different models. In PSHA, this is modelled as epistemic uncertainty, and the so-called logic trees are applied to combine different ground-motion models. While this approach is, to a large extent, ad-hoc procedure in terms of selecting the weights assigned to different models, a more rigorous approach has not yet evolved. In combining our model with other models valid in the far-field area, we recommend using a weighting function (of distance from the fault) $w$ which the analyst can choose to vary from 1 at the fault plane to 0 at a distance of about 25 km from the fault. Then any far-field model to be combined with the proposed model will be given a weight of $1-w$. We recognize that this is an ad-hoc approach, and further research is necessary in finding convincing methods to handle epistemic uncertainties in seismic hazard or risk assessment exercises.
Force-based design of engineering structures for earthquake resistance generally requires inelastic response spectra of SDOF systems to estimate design lateral strengths. Inelastic response spectra for earthquake ground motion are typically constructed by reducing elastic response spectra by the so-called force reduction factor or structural behaviour factor. Hysteretic energy dissipation during inelastic deformation is a major contributor to these reduction factors apart from damping and structural over-strength. The reduction in design forces due to hysteretic energy dissipation \( R_\mu \) is defined as the ratio of elastic strength demand to inelastic strength demand required to maintain a displacement ductility \( \mu \) less than or equal to a pre-determined target ductility ratio when subjected to the same excitation. Miranda and Bertero (1994) provide detailed review on \( R_\mu \) and the factors affecting its magnitude. Other researchers carrying out extensive studies on force reduction factors include Borzi and Elnashai (2000); Jalali and Trifunac (2008), and Watanabe and Kawashima (2002), to name a few. The results of past studies have shown that \( R_\mu \) depends mainly on \( \mu \) and \( T_n \).

In this section we present the relationship between \( R_\mu, \mu, \) and \( T_n \) for forward-directivity affected near-fault ground motions.

To explore the dependence of \( R_\mu \) on SDOF period and target ductility level, yield strength (iso-ductile) spectra were computed for target displacement ductility levels of 1.5, 2, 3, 4, 5, and 6. The hysteretic behaviour of inelastic SDOF was assumed to be elastic-perfectly-plastic,

---

Fig. 19  Distribution of residuals of 5% damped spectral shapes compared with a standard normal distribution
and the level of viscous damping was considered to be 5% for elastic as well as inelastic systems. Figure 20 displays the variation of mean force reduction factors with undamped natural periods of SDOF for six different values of target ductilities. It is observed that as $T_n \to 0$, $R_\mu \to 1$. The equal deformation rule of Veletsos and Newmark (1960) - at long periods, peak elastic deformation is equal to peak inelastic deformation, and therefore $R_\mu = \mu$ - is apparently not strictly valid up to undamped natural periods of 10 s. Although the rule is conservative in the sense that it predicts lower values of $R_\mu$, it is observed that the reduction factor at 10 s is about 10% higher than the assumed target displacement ductility. However, the results at long periods are likely to be influenced by the cut-off frequency of high-pass filters used in processing the accelerograms.

Following the pioneering works of Veletsos and Newmark (1960), several researchers have proposed mathematical equations describing the $R_\mu - \mu - T_n$ relationship (see, for example, Borzi and Elnashai 2000, and references therein). To establish such a relationship we use a mathematical model similar to the one used by Watanabe and Kawashima (2002). Their model was designed specifically to satisfy the equal deformation rule at long periods, which we calibrate to match the force reduction factors displayed in Fig. 20. We idealize mean force reduction factors by the following equations.

$$R_\mu = [\mu - 1] \psi (T_n) + 1$$

$$\psi (T_n) = \frac{T_n - \gamma}{\gamma \exp (\tau T_n)} + 1$$

In Eqs. 16 and 17, the constants $\gamma$, and $\tau$ are functions of $\mu$ and were accordingly calibrated using mean reduction factors presented in Fig. 20. The reduction factors given by these equations satisfy the condition $R_\mu \to 1$ as $T_n \to 0$. As $T_n \to \infty$, the force reduc-
Table 7  Parameters of Eqs. 16 and 17 describing the $R_\mu - \mu - T_n$ relationship

<table>
<thead>
<tr>
<th>$\mu$</th>
<th>$\gamma$</th>
<th>$\tau$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.50</td>
<td>6.00</td>
</tr>
<tr>
<td>2.0</td>
<td>1.00</td>
<td>4.50</td>
</tr>
<tr>
<td>3.0</td>
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<td>3.00</td>
</tr>
<tr>
<td>4.0</td>
<td>2.50</td>
<td>2.00</td>
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<tr>
<td>6.0</td>
<td>3.25</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Fig. 21  Comparison of mean force reduction factors (solid lines) with the idealized ones (dashed lines) given by Eqs. 16 and 17 for displacement ductilities of 1.5, 2, 3, 4, 5, and 6.

It is important to note that force reduction factors of individual ground motion records show a considerable variation around the mean. In this sense it might not be conservative to use the mean force reduction factors. Ground motion prediction equations for inelastic spectral ordinates like the ones proposed by Rupakheti and Sigbjörnsson (2009a), and Bozorgnia et al. (2010) provide a more reliable estimate of design lateral strengths than the approach...
of using force reduction factors. The number of ground-motion records containing forward-directivity pulses is not sufficient to develop such a model.

Another important consideration to keep in mind while using force reduction factors is the level of damping in structures. Watanabe and Kawashima (2002) computed force reduction factors with three assumptions regarding the damping ratio of elastic and inelastic SDOF, namely (i) 2% for both elastic and inelastic systems, (ii) 5% for elastic and 2% for inelastic systems, and (iii) 5% for both elastic and inelastic systems. Their results indicated that the first assumption provides the largest force reduction factors, the second assumption provides the smallest force reduction factors, while the third one estimates force reduction factors between the two cases. We performed a similar study on our data, and the results were in agreement with their observations. Before applying force reduction factors, it is important to consider the assumptions regarding the damping ratio of SDOF used in deriving them. The reduction factors presented here correspond to 5% of critical damping for both elastic and inelastic SDOF systems.

9 Conclusions

A large number of recorded near-fault ground-motion data have been collected and studied, with special emphasis on forward-directivity effects. The period where 5% damped pseudo-spectral velocity contains a clear peak is proposed as a measure of predominant period \( T_d \) of forward-directivity affected near-fault ground motions. The main advantage of this definition is that, unlike pulse period as defined by various authors, this measure is unambiguous and is easily calculated. A robust equation is developed to relate \( T_d \) to earthquake magnitude. The relationship between \( T_d \) and \( M_w \) is similar to scaling relations between pulse period and magnitude proposed by different researchers in the past.

An empirical model is developed to estimate peak ground velocity (PGV) as a function of earthquake magnitude and source-to-site distance. However, we found that the available data are not sufficient to constrain a reliable model for the effects of source mechanism and local site conditions. A weak dependence of PGV on moment magnitude is observed. A potentially useful predictor for directivity, based on the work of Spudich and Chiou (2008), is tested as a potential parameter in PGV attenuation model, although with limited success.

Properties of elastic response spectra of forward-directivity-affected near-fault ground motion are discussed in depth. A simple model is proposed to estimate mean spectral shapes of SDOF response to such ground motions. The proposed analytical model is a continuous function of the undamped natural period of SDOF oscillators, and its parameters are magnitude dependent. The model is calibrated by using recorded ground motions. The dependence of the parameters of the proposed model on earthquake size is investigated, constraining their relationship in a step-by-step manner. It was found that several parameters of the model can be effectively expressed in terms of earthquake size, thereby reducing the number of free variables.

In addition, the effects of viscous damping ratio on spectral shapes were thoroughly examined. By studying spectral shapes for different levels of viscous damping, we were able to express the parameters of the spectral shape model as a continuous function of damping ratio. This avoids the use of so-called damping correction factors commonly used to derive response spectra for various levels of damping from that corresponding to 5% of critical damping.

The proposed model is found to have small uncertainties in the period range of common engineering structures, whereas uncertainties concerned with very high frequencies are
larger. The standard deviation of the residuals of the proposed model was found to be smaller for highly damped systems. The proposed model can be used with any reliable attenuation model of $PGV$, in order to estimate the elastic response spectra for forward-directivity ground motion in the near-fault area.

Finally, constant-ductility spectra of elasto-plastic SDOF systems are studied for ductility ratios ranging from 1.5 to 6. An approximate equation to estimate force reduction factors as a function of displacement ductility and SDOF period is also presented.

Acknowledgments

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Appendix 1

Approximation of the response spectrum at long periods

To obtain an approximation we assume that the Fourier acceleration spectrum is proportional to $\omega$ when $\omega \to 0$, $\omega$ being the frequency. This assumption is in accordance with the Brune near-fault model (Brune 1970), according to which we have

$$|A_N(\omega)| \propto \omega$$  \hspace{1cm} (A.1.1)

Hence, it follows that the velocity spectrum approaches a constant value for $\omega = 0$:

$$|V_N(0)| = \text{constant}$$  \hspace{1cm} (A.1.2)

Using this basic property of the near-fault spectrum and random vibration theory, it is possible to derive the asymptote of the earthquake response spectrum when the natural structural period approaches infinity. The solution can be approximated as follows:

$$PSV \propto \frac{1}{\sqrt{T_n}} \left( \sqrt{2 \log \left( \frac{2\Delta T}{T_n} \right)} + \frac{\gamma}{\sqrt{2 \log \left( \frac{2\Delta T}{T_n} \right)}} \right) \sqrt{1 - \exp \left( -4\pi \zeta \frac{\Delta T}{T_n} \right)} \quad \text{if } T_n \gg T_d$$  \hspace{1cm} (A.1.3)

where, $T_n$ is the undamped natural period of the structure, $\zeta$ is the damping ratio, $\Delta T$ is the duration of strong-motion, $T_d$ is the period of the spectral peak, and $\gamma = 0.5772$ is the Euler’s constant.

We adopt the definition of Trifunac and Brady (1975) taking significant duration of the strong ground motion as the time interval between the 5% and the 95% of the Arias intensity (Arias 1970). Average duration for each magnitude bin listed in Table 3 in the text was computed accordingly. Using these duration values and 5% critical damping, Eq. A.1.3 was used to compute the asymptote of $PSV$ at long periods. It should be noted that Eq. A.1.3 can not be used for $T_n \geq 2\Delta T$. In such cases, the second term in Eq. A.1.3 can be replaced by the peak factors given in Cartwright and Longuet-Higgins (1956).
Since the slope (in log-log scale) of the long period part of the proposed spectral shape model is controlled by $\theta$, its value can be estimated to match the asymptote computed from Eq. A.1.3. The comparison between the proposed model and the asymptotic solution of Eq. A.1.3 for different magnitude bins is shown in Fig. 22 below. The asymptotic solution is hinged to the spectral shape at a period equal to two times the period of the spectral peak. The values of $\theta$ required to match the asymptotic slope was found to lie between 1.9 for the first bin to 2.25 for the sixth bin. In order to simplify the model, we use a constant value of 2 for all magnitude bins. This is also consistent with the commonly used assumption that the PSV spectrum has a slope of -1 in a tripartite representation (see, for example, Newmark et al. 1973).

Fig. 22 Comparison between the asymptotic solution (red line) and the proposed spectral shapes (blue line) for different magnitude bins. The asymptotic solutions are hinged to the spectral shapes at a period two times the period of the spectral peak.

Appendix 2

See Table 8.
Table 8 Near-fault records with distinct velocity pulses

<table>
<thead>
<tr>
<th>EID</th>
<th>WID</th>
<th>Location</th>
<th>Date</th>
<th>Mechanism</th>
<th>( M_a )</th>
<th>Station</th>
<th>Component</th>
<th>PGV (cm/s)</th>
<th>( Z^2 ) (km)</th>
<th>( r_{sp}^4 ) (km)</th>
<th>( r_{bd}^5 ) (km)</th>
<th>( s^6 ) (km)</th>
<th>( d^7 ) (km)</th>
<th>( c^8 )</th>
<th>( V_{s30} ) (m/s)</th>
</tr>
</thead>
<tbody>
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<td>10.00</td>
<td>31.04</td>
<td>6.27</td>
<td>24.94</td>
<td>9.99</td>
<td>3.71</td>
<td>185</td>
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<tr>
<td>2</td>
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<td>San Fernando, CA, USA</td>
<td>09/02/1971</td>
<td>RV</td>
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<td>PCD</td>
<td>SN</td>
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<td>13.00</td>
<td>11.86</td>
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</tr>
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<td>3</td>
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1 SS, strike-slip; RV, reverse; OB, oblique; NM, normal
2 SN, strike-normal; SP, strike-parallel
3 Hypocentral depth
4 Epicentral distance
5 Joyner and Boore distance
6 Length of fault that ruptures towards site
7 Width of fault that ruptures towards site
8 Isochrone velocity normalized by the shear wave velocity (Spudich et al. 2004)
References


