# DESIGN GROUND MOTIONS NEAR ACTIVE FAULTS

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## SUMMARY

Forward-Directivity (FD) in the near-fault region can produce intense, pulse-type motions that differ significantly from ordinary ground motions that occur further from the ruptured fault. Near-fault FD motions typically govern the design of structures built close to active faults so the selection of design ground motions is critical for achieving effective performance without costly over-design. Updated empirical relationships are provided for estimating the peak ground velocity (*PGV*) and period of the velocity pulse ($T_v$) of near-fault FD motions. *PGV* varies significantly with magnitude, distance, and site effects. $T_v$ is a function of magnitude and site conditions with most of the energy being concentrated within a narrow-period band centred on the pulse period. Lower magnitude events, which produce lower pulse periods, might produce more damaging ground motions for the stiff structures more common in urban areas. As the number of near-fault recordings is still limited, fully nonlinear bi-directional shaking simulations are employed to gain additional insight. It is shown that site effects generally cause $T_v$ to increase. Although the amplification of *PGV* at soil sites depends on site properties, amplification is generally observed even for very intense rock motions. At soft soil sites, seismic site response can be limited by the yield strength of the soil, but then seismic instability may be a concern.


## FORWARD-DIRECTIVITY

Near-fault ground motions are significantly influenced by the rupture mechanism and slip direction relative to the site and by the permanent ground displacement at the site resulting from tectonic movement. When the rupture and slip direction relative to a site coincide, and a significant portion of the fault ruptures towards the site, the ground motion can exhibit the effects of forward-directivity (FD) [1]. Most of the energy in FD motions is concentrated in a narrow frequency band and is expressed as one or more high intensity velocity pulses oriented in the fault-normal direction. These intense velocity pulses can lead to severe structural damage.

Ground motions close to the surface rupture may also contain a significant permanent displacement, which is called fling-step, and this may lead to a high intensity velocity pulse in the direction of the fault displacement. Pulses from fling-step have different characteristics than FD pulses. Whereas FD is a dynamic phenomenon that produces no permanent ground displacement and hence two-sided velocity pulses, fling-step is a result of a permanent ground displacement that generates one sided velocity pulses. The development of design ground motions for a project site close to an active fault should account for these special aspects of near-fault ground motions. Fling-step considerations are discussed in [2]. In this paper, near-fault forward-directivity effects are addressed.

The effects of forward-directivity are generated because the velocity of the fault rupture front is only slightly less than the shear wave propagation velocity [1]. As the rupture front propagates from the focus of the event, a shear wave front is formed by the accumulation of the shear waves travelling ahead of the rupture front. When a site is located at one end of the fault and rupture initiates at the other end of the fault and travels towards the site, the arrival of the wave front is seen as a large pulse of motion that occurs near the beginning of the record. Thus, FD motions typically occur at sites near the end of a strike-slip fault when the rupture moves towards the site, and at sites located in the up-dip projection of a ruptured dip-slip fault (i.e., reverse or normal fault). The radiation pattern of the shear dislocation on the fault causes this large pulse of motion to be oriented in a direction perpendicular to the fault plane [1]. These effects are typically long-period in nature and are best observed in the velocity-time history. FD conditions produce ground motions that have large amplitude and short durations. However, if a site is located at one end of the fault and rupture propagates away from the site, the opposite effect is observed (i.e., backward-directivity), and the motion is characterized by longer duration and lower amplitude ground motions.

Some examples of near-fault FD motions are shown in Figure 1. The use of the velocity-trace plot shown at the right is useful, because FD motions typically exhibit a systematic difference between the fault-normal and fault-parallel components of motion. The fault-normal component is systematically more intense than the fault-parallel component of motion. It is important to remember that the average of the two components of motion is systematically more intense at long periods than ordinary motions as well. Hence, near-fault FD fault-normal components of motion are especially severe and potentially destructive.


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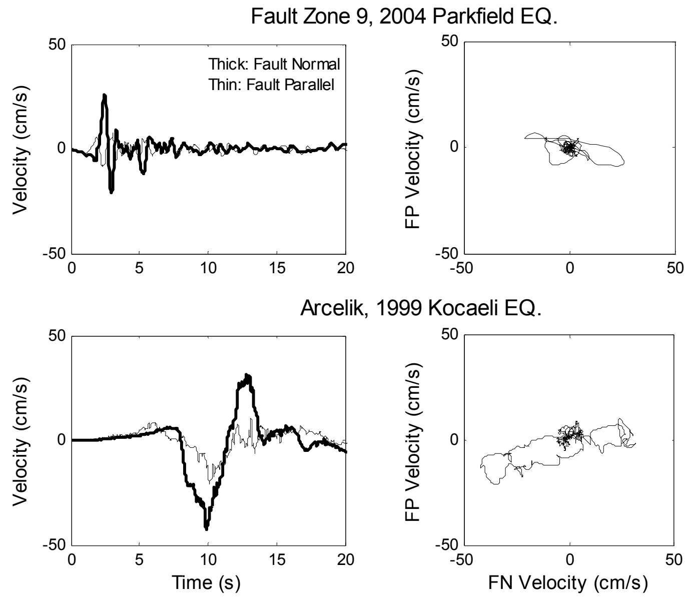



***Figure 1:*** ***Fault normal (FN) and fault parallel (FP) horizontal velocity-time histories and velocity-traces for near-fault records from the 2004 Parkfield ($M_w$ = 6.0) and the Kocaeli ($M_w$ = 7.5) earthquakes.***

Pulse-type motions are critical in the design of structures in the near-fault zone. Two approaches have been used to account for near-fault ground motions in design. The frequency-domain approach uses empirical factors to modify acceleration response spectra for sites that are affected by forward-directivity effects [1, 3]. However, advanced dynamic analyses indicate that the amplitude, period, and number of significant pulses in the velocity-time history primarily control the performance of structures (e.g. [4, 5]). The alternative time-domain approach characterizes the FD motion through its velocity-time history (Figure 2), with its peak ground velocity (*PGV*), predominant pulse period ($T_v$), and number of significant velocity pulses ($N_c$). In this preferred approach, it is crucial that reliable estimates of *PGV* and $T_v$ be obtained for near-fault FD motions.

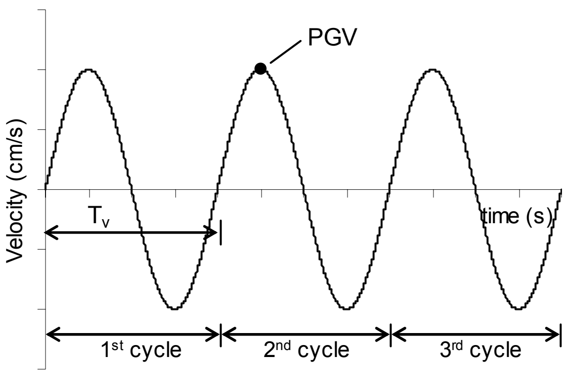



***Figure 2:*** ***Simplified Velocity-Time History Showing Peak Ground Velocity (PGV), Period of the Velocity-Pulse ($T_v$), and number of significant cycles of motion ($N_c$).***

## EMPIRICAL RELATIONSHIPS

Empirical ground motion relationships may be used to develop reasonable estimates of *PGV* and $T_v$ for FD motions. A number of researchers have developed empirically based predictive relationships for these near-fault ground motion parameters (e.g. [6, 4, 7]). Most recently, Bray and Rodriguez-Marek [8] used a comprehensive database of FD ground motions to develop empirical relationships for *PGV* and $T_v$. This database was enhanced with FD records from recent earthquakes, and the relationships of Bray and Rodriguez-Marek [8] are updated in this paper.

Near-fault FD records were selected from the strong motion database of the Pacific Earthquake Engineering Research Center (http://peer.berkeley.edu/). Records with geometric conditions leading to FD were used. Records were selected if the ratio of fault-normal to fault-parallel spectral acceleration at a period of three seconds predicted by [1] was greater than one. Recordings not possessing at least some features of FD characteristics were excluded from the analysis. FD characteristics are positive fault-normal to fault-parallel response spectral ratios for long periods, and a reasonably well-defined velocity pulse in the fault-normal direction. The records are from shallow earthquakes ($M_w \geq 6$) in active tectonic regions at rupture distances (R = closest distance to the fault plane) less than 20 km. Fourteen near-fault records from four earthquakes (i.e., the 1986 Palm Springs, 2002 Denali, 2003 Bam, and 2004 Parkfield earthquakes) were added to the database of Bray and Rodriguez-Marek [8]. Three records from the 1999 Chi-Chi, Taiwan earthquake were processed to remove the fling-step present in the records by fitting a hyperbolic tangent function to the displacement-time history of the record and subsequently subtracting this motion from the time history (Rathje, pers. Comm. 2000). Ground motion sites were classified as either rock/shallow stiff soil (i.e., only 0 – 20 m of soil or weathered rock over competent rock) or soil (i.e., mostly stiff soil with shear wave velocity, $V_s$ > 180 m/s). Soft soil and liquefiable sites were excluded. Additional details about the earthquakes, records, and fault-normal component orientation used are provided in [8] and [9].

The predictive relationships for *PGV* and $T_v$ in this study (as well as the previous study by Bray and Rodriguez-Marek, [8]) include the influence of site conditions (i.e., "rock" or "soil") as well as earthquake moment magnitude ($M_w$) and the closest distance from the site to the ruptured fault ($R$). The empirical evidence clearly points to a systematic difference between near-fault FD motions recorded on rock and on soil sites. Ground motions recorded in soil tend to have longer pulse periods and larger *PGV*s than those recorded at rock sites [8]. An example of this is the set of ground motions recorded in Gilroy during the 1989 Loma Prieta earthquake as shown in Figure 3.

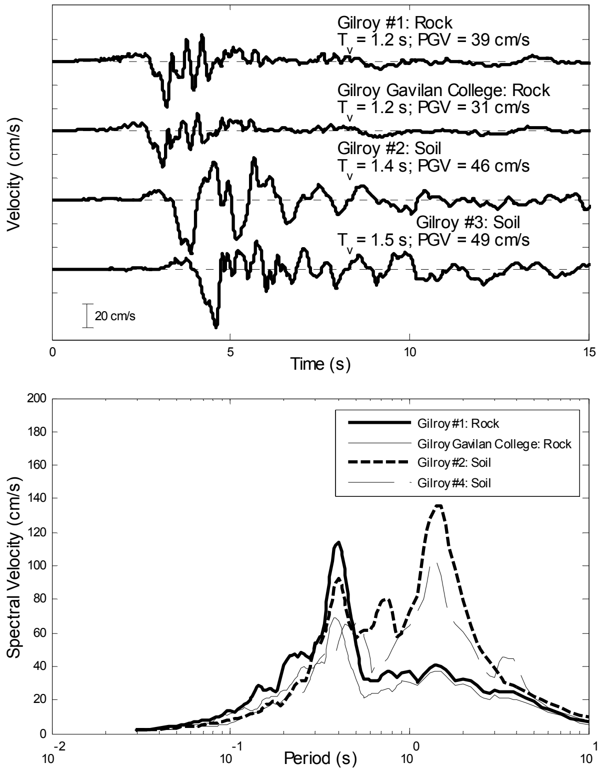



***Figure 3: Recorded fault-normal motions during the 1989 Loma Prieta earthquake. Gilroy #1, Gavilan Coll., Gilroy #2, and Gilroy #3 have rupture distances of 11, 12, 13, and 14 km, resp. (modified from [8]).***

**Peak Ground Velocity**

With the data for near-fault ground motions being restricted to relatively small source-to-site distances, the functional form of the model for estimating *PGV* can be simplified to:

$$\ln(PGV_{ij}) = a + b\, M_w + c \ln (R^2 + d^2) + \eta_i + \varepsilon_{ij} \qquad (1)$$

where $PGV_{ij}$ is the peak ground velocity in units of cm/s of the $j^{th}$ recording from the $i^{th}$ event; $M_w$ is moment magnitude of event *i*; *R* is rupture distance in km; and *a*, *b*, *c,* and *d* are regression parameters; and $\eta_i$ and $\varepsilon_{ij}$ represent the inter- and intra-event variations, respectively, obtained using the random effects model [10]. The inter-event and the intra-event error terms are assumed to be independent normally distributed random variables with variances $\tau^2$ and $\sigma^2$, respectively. The standard error associated with the estimate of *PGV* is then

$$\sigma_{total}^2 = \tau^2 + \sigma^2 \qquad (2)$$

The functional form of Equation (1) for *PGV* as a function of distance results in a nearly zero slope at close distances to the fault, and it decreases linearly with the logarithm of distance at larger distances. The statistical analysis was performed on the entire dataset and then separately on the rock and soil motions. The parameters for Equation (1) are provided in Table 1.

**Table1. Parameters for the PGV relationship (Equation 1).**

| Data Set | *a* | *b* | *c* | *d* | $\sigma$ | $\tau$ | $\sigma_{total}$ |
|---|---|---|---|---|---|---|---|
| All Motions | 2.05 | 0.55 | -0.39 | 5.00 | 0.37 | 0.24 | 0.44 |
| Rock* | 1.86 | 0.55 | -0.39 | 5.00 | - | - | 0.40 |
| Soil | 2.11 | 0.55 | -0.39 | 5.00 | 0.33 | 0.30 | 0.44 |

* Parameters had to be obtained using maximum likelihood.

**Pulse Period**

Somerville [6] provides justification for using self-similar scaling relationships to constrain fault parameters. The use of this scaling relationship indicates that the pulse period is about two times larger than the rise time of slip on a fault, which measures the duration of slip at a single point in the fault. From the mechanics of fault rupture, the rise time can be established as a lower bound for pulse period [6]. Because the logarithm of rise time is a linear function of moment magnitude, the use of a linear relationship between logarithm of rise time and magnitude is justified. Thus, the relationship used by [8] for pulse period is:

$$\ln(T_v)_{ij} = \mathrm{f} + hM_w + \eta_i + \varepsilon_{ij} \qquad (3)$$

where $(T_v)_{ij}$ is the pulse period of the $j^{th}$ recording from the $i^{th}$ event; $M_w$ is the moment magnitude of event *i; f* and *h* are regression parameters determined by the data through the random effects model; $\eta_i$ is the inter-event term; and $\varepsilon_{ij}$ represents the intra-event variations. The updated parameters for Equation (3) based on the regression analysis of the larger data included in this paper are provided in Table 2.

**Table 2. Parameters for the relationship for velocity pulse period (Equation 3).**

| Data Set | *f* | *h* | $\sigma$ | $\tau$ | $\sigma_{total}$ |
|---|---|---|---|---|---|
| All Motions | -4.42 | 0.75 | 0.41 | 0.381 | 0.56 |
| Rock | -6.37 | 1.00 | 0.46 | 0.29 | 0.55 |
| Soil | -3.71 | 0.65 | 0.35 | 0.37 | 0.51 |

**Number of Significant Cycles**

The number of significant cycles of motion is defined as half the number of half-cycle (one-sided) velocity pulses that have an amplitude at least 50% of the peak ground velocity of the ground motion. Due to chaotic nature of fault rupture and the uncertainty in characterizing the details in the rupture process that determine the number of significant cycles, it is not possible at this time to develop a robust relationship for predicting this important parameter. However, an examination of FD records does provide useful insights.

Some earthquake events have a well-defined pulse sequence for nearly all of its near-fault motions. This might be expected for faults that have a relatively uniform slip distribution or earthquakes where slip is concentrated over a single zone. For these events, stations that are close to each other will likely be equidistant to regions of high slip. Moreover, path effects are minimized for stations in the near-fault region. However, for an earthquake with highly non-uniform slip, such as the 1994 Northridge earthquake, the type of pulse sequence observed depends on the instrument's distance relative to the asperities. Somerville [6] suggests that the number of half-sine pulses in the velocity time-history might be associated with the number of asperities in a fault rupture. Thus, details in the rupture process, such as the number of asperities of the fault and the slip distribution on the causative fault, determine the number of significant pulses in a FD motion.

Examination of the near-fault FD velocity-time histories included in this study does indicate that it is unlikely that a near-fault FD record will have more than two significant full cycles of motion. More than half of the FD records contained only one significant full cycle of motion (i.e., one two-sided velocity pulse). Hence, in developing design ground motions for use in projects, it is reasonable to select records that have only one or two significant cycles of motion. It would be unnecessarily conservative to use simulated or modified ground motions records that have a large number of significant cycles of motion in their velocity-time history.

Although near-fault FD motions are more intense than ordinary records, they are shorter in duration. The seismic energy is compact, which leads to the high intensities, but also requires that the duration of significant shaking be short. Hence, it would also be unnecessarily conservative to use high intensity motions with long durations of strong shaking. It is more likely for near-fault FD motions to be at or below the median minus one standard deviation for significant duration.

### Application into Probabilistic Seismic Hazard Analyses and Performance Based Design

Estimates of seismic hazard are usually made using Probabilistic Seismic Hazard Analyses (PSHA). PSHA predicts the mean annual rate of exceedance of a ground motion parameter (e.g., an *Intensity Measure*). These intensity measures, in turn, can be used to predict structural response in what is termed Performance Based Design (PBD). Equations (1) and (3) can be used in PSHA or in PBD for near-fault sites provided that the probability of occurrence of the pulse is known, and the cross correlation of $PGV$ and $T_v$ is also known. Thotong et al. [11] present a preliminary model for the probability of occurrence of a pulse. The correlation coefficient between $\ln(T_v)$ and $\ln(PGV)$ for the dataset used in this study is 0.24. The positive correlation coefficient implies that $T_v$ increases as $PGV$ increases, which is an expected outcome. However, the residuals of $T_v$ and $PGV$ (i.e., the difference between measured values their estimated values using Equations 1 or 3) are uncorrelated.

## SEISMIC SITE RESPONSE FOR FD MOTIONS

### Analytical Framework

Site conditions were found to be potentially of great importance in discerning the characteristics of near-fault FD motions. It should not be surprising that site effects have the potential for significantly modifying the ground motion at a deep or soft soil site compared to that which occurs at depth in the bedrock. The importance of local site conditions has been highlighted in a large number of empirical and analytical studies and is reflected in most building codes (e.g., the 2006 *International Building Code* [12]).

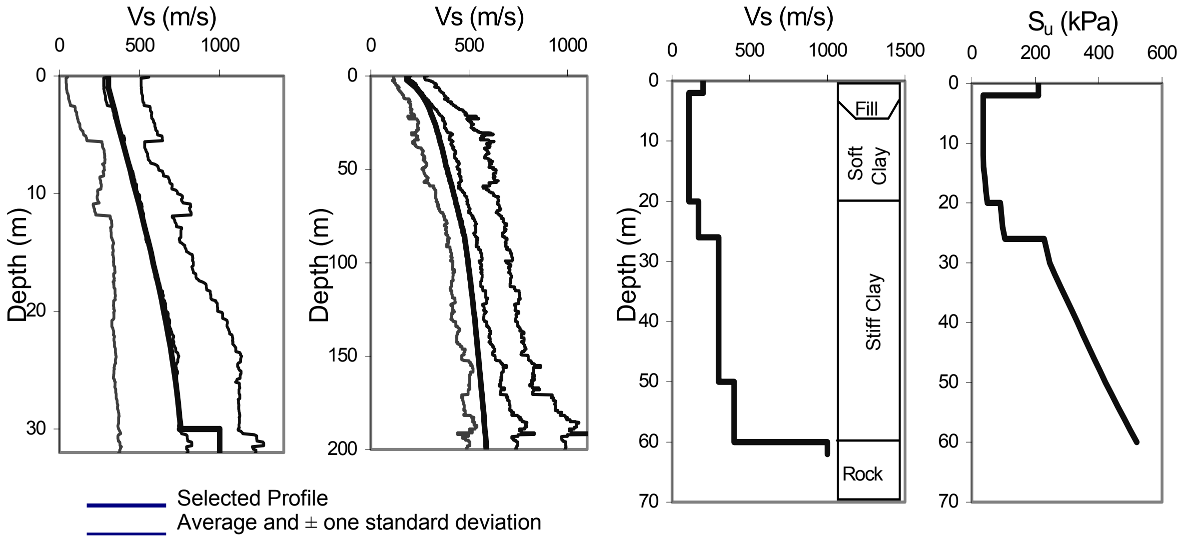
There are not a sufficient number of rock and soil recordings in close proximity to each other that contain near-fault FD characteristics to allow a detailed empirical study of site effects. Instead, numerical simulations are utilized. In a study by Rodriguez-Marek and Bray [13], seismic site response is modelled by means of a time-domain finite element analysis using the fully nonlinear multi-axial total stress soil constitutive model of Borja and Amies [14]. Bi-directional shaking is imposed to explore the combined effects of the more intense fault-normal component and the less intense, but still important, fault-parallel component of motion. One-dimensional propagation of horizontal shear waves is modelled by a column of 8-node tri-linear brick elements, where each node is allowed to move in two horizontal directions. Stress-free boundary conditions are imposed at the top of the column, and viscous dashpots are placed at the base of the soil column to model the energy absorption of the elastic half-space [15]. The implementation of the constitutive model in the finite element code GeoFeap [16] is discussed in [17]. This seismic site response analysis procedure has been validated using field downhole array recordings and laboratory shaking table measurements [17].

Eight parameters are required to define the soil model. Elastic soil response is determined by the shear wave velocity ($V_s$) and Poisson's ratio ($v$), which is assumed to be 0.49 to approach a fully undrained behavior. The exponential interpolation function of Borja and Amies' model is defined by two model parameters and the kinematic hardening parameter of the bounding surface. Soil strength is defined by the radius of the bounding surface, $R$, which is given by 1.6 $S_u$, where $S_u$ is the soil's undrained strength in triaxial compression. When the soil will not reach shear failure, the parameter $R$ can be used as a curve fitting parameter. Energy dissipation is naturally produced by the constitutive model through hysteretic damping. At small strain levels, damping is incorporated through Rayleigh damping, which is fully defined by the equivalent damping ratio at small strains, $\xi$, and a frequency band where $\xi$ is matched.

Nonlinear site response is performed for generalized soil profiles subject to simplified pulse-type input motions so that insights can be made regarding the effects of site conditions in the near-fault region. Studies by structural engineers have shown that these simplified representations are capable of capturing the salient response features of structures subjected to near-fault ground motions (e.g., [4, 5]). Bray and Rodriguez-Marek [8] developed a simplified representation of FD velocity-time histories using sine pulses in both the fault-normal and fault-parallel directions. Ground motions are fully defined by the period of each cycle, their corresponding amplitude, and the number of significant pulses (see Figure 2). The $PGV$ is the largest amplitude of all cycles and the pulse period of the record, $T_v$, is defined as the period of the cycle with the largest amplitude. A series of input velocity-time histories were created by parsing together sequences of sine pulses. The amplitude and period of these ground motions were varied systematically. Pulse periods were varied from 0.6 s to 4.0 s, and pulse amplitudes from 75 to 300 cm/s.

### Generalized Site Profiles

Three generalized site profiles were created to represent common site classes used in building codes (Very Stiff Soil, Stiff Soil, and Soft Soil; corresponding to 2006 IBC Site classes C, D, and E, respectively). The selected shear wave velocity profiles are obtained from an extensive database of shear wave velocity logs of sites located largely within California (Silva, pers. comm. 2000). The shear wave velocities for all three generalized site profiles are shown in Figure 4. Shear wave velocities at depth (where $V_s$ data is scarce) for the Site D profile were obtained assuming that shear wave velocities are proportional to $(\sigma'_m(z))^n$, where $\sigma'_m$ is the mean effective stress at depth $z$ and $n$ = 0.25 [18]. The soft clay profile (Site E) represents typical Bay Mud sites from the San Francisco Bay region. The density of the stiff soils was about 1.9 Mg/m$^3$; whereas the density of Holocene clay was 1.6 Mg/m$^3$. The profiles were placed on 3 m of weathered rock that in turn overlies a rock elastic half-space with a shear wave velocity of 1200 m/s and a density of 2.4 Mg/m$^3$. For Site D, the depth of the profile was varied from 30 to 200 m to study the effects of variations of depth to bedrock on site response.




(a) Site C (b) Site D (c) Site E

***Figure 4:*** ***Three generalized site profiles used in the dynamic analyses and representative values from a set of recorded $V_s$ profiles in California (Silva, pers. Comm.) (from [13]).***

The strength profile of the clayey soils was developed using $S_u/\sigma_v' = 0.8$ for the stiff soils and $S_u/\sigma_v' = 0.3$ for the soft clay in the Site E profile. A lower bound of $S_u = 150$ kPa and $S_u = 25$ kPa were used for the stiff and soft clays, respectively. The static shear strength was multiplied by a factor of 1.4 to account for rate effects during the one primary cycle of rapid earthquake loading [19]. The nonlinear properties of the soil were obtained by matching the Borja and Amies [14] model to widely used strain-dependent shear modulus reduction and material damping relationships. The PI = 15 and PI = 30 curves of Vucetic and Dobry [20] were used for the stiff clay soils, and the curves of Isenhower and Stokoe [21] were used for the soft clays. Additional details are provided in [17].

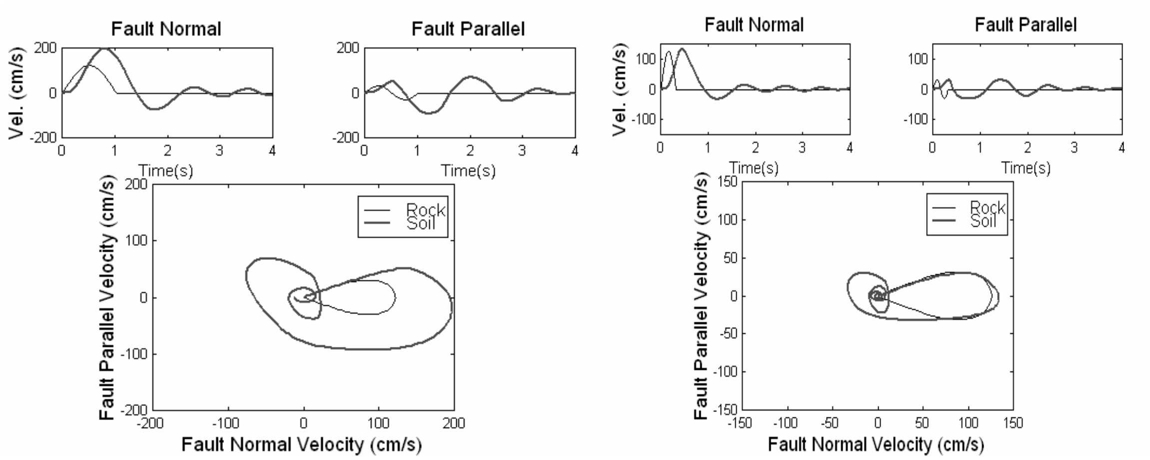
## Discussion of Results

The concept of an equivalent-linear "degraded site period" is still useful for interpreting the results of these nonlinear response analyses to intense near-fault FD motions. The degraded site period ($T_s'$) is calculated as: $T_s' = 4H/V_s'$, where $H$ is the soil depth and $V_s'$ is the average effective shear wave velocity of the soil deposit using a shear modulus that is consistent with the effective shear strain induced in each layer of soil (i.e., $\gamma_{eff} = (n)\gamma_{max}$, where $n$ is nearly 1.0 for pulse-type motions). The "degraded site period" increases with increasing soil depth, decreasing soil stiffness, and increasing soil nonlinearity resulting from more intense rock motions.

The largest amplification of $PGV$ through a soil site occurs near its "degraded site period." The pulse period of the soil motion tends to approach the "degraded site period" when it is initially lower than the "degraded site period." Hence, input pulses that have lower periods undergo more elongation than those with higher periods. When the input velocity pulse period is higher than the "degraded site period," the soil deposit has a pseudo-rigid body response and pulse period is not affected. Although the stronger fault-normal component of near-fault FD motion is more critical, the fault-parallel component can also affect the response of a site. Larger fault-parallel component velocities lead to larger earthquake-induced shear strains, a softer response, and hence, a larger "degraded site period." Different shapes of the input pulse period also affect shear strain levels in the soil. Depending on the coincidence of fault-normal and fault-parallel peaks in velocity, motions can induce larger strains and result in higher degraded site periods [17].

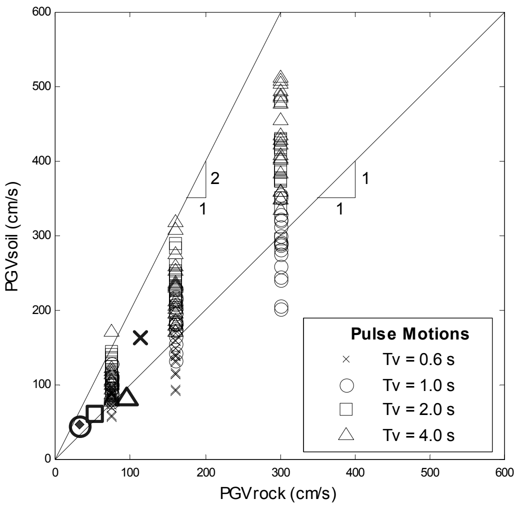
Representative results are shown in Figure 5 for a set of bi-directional seismic site response analyses performed for a deep stiff soil site undergoing near-fault FD simplified half-cycle motions. For the relatively low pulse period input motion (Figure 5b), there is significant elongation of the pulse period due to the soil, but there is not amplification of the $PGV$ as the input pulse period is not near the degraded period of the soil deposit. However, for the 2 s pulse period input motion (Figure 5a), there is significant amplification of the $PGV$, because it more closely coincides with the "degraded site period" of the deep, stiff soil deposit.

The relationship between output ($PGV_{soil}$) and input ($PGV_{rock}$) intensities for all site response analyses for the Stiff Soil profile (IBC Site D with soil depths ranging from 30 to 200 m) are shown in Figure 6. The ratio of $PGV_{soil}$ to $PGV_{rock}$ is generally between one and two. As a comparison to these analytical results, the results from an empirical study by Silva (pers. comm. 1998) exhibit a trend that is consistent with the results from these analyses. The computed amplification of $PGV$ also agrees fairly well with the mid-period amplification factor for Site D in the 2006 IBC, suggesting that spectral amplification factors in the mid-period range ($T \sim 1$ s) are consistent with these $PGV$ amplification factors. As indicated by the results shown in Figure 5, significant amplification still occurs for intense rock motions. Hence, the amount of nonlinearity in the amplification of $PGV$ (or mid-period spectral acceleration amplification) is fairly minor for stiff soil profiles. Closer examination of the results in Figure 6 indicate that generally more amplification of $PGV$ occurs for rock input motions with higher pulse periods and less amplification occurs for input motions with lower pulse periods.




a) b)

***Figure 5:*** ***Responses of a deep, stiff soil site to a half-sine-pulse input rock motion with PGV = 120 cm/s: (a) results for an input pulse period of 2.0 s, and (b) results for an input pulse period of 0.6 s.***




× Pacoima Dam, San Fernando EQ.

○ Gilroy Gavilan College, Loma Prieta EQ

□ Pacoima Dam Downstream, Northridge EQ

Δ KJMA, Kobe EQ

♦ Average for near-fault motions:
$M = 6 – 7$, $R < 10$ km (Silva, pers. comm. 1998) comm.)

***Figure 6:*** ***Calculated PGV amplification at Stiff Clay profiles (see [17] for more details).***

Seismic site response analyses of the Soft Soil site (Site E) indicate that earthquake-induced shear stresses fully mobilized the dynamic strength of the soft clay for even relatively moderate near-fault motions. Thus, the soft soil deposit's low strength limits the site's peak seismic response. Significant variations in the shape of the soil's strain-dependent shear modulus reduction and material damping curves at intermediate strain levels do not affect the calculated *PGV* or $T_v$ of the surface motion. Soil yielding leads to a significantly higher "degraded site period" with a significant amount of energy dissipation through plastic yielding. This leads to relatively more velocity period elongation at soft soil sites and greater attenuation of input rock PGV for intense rock motions than at stiff soil sites (Figure 7). This attenuation of pulse amplitude is not reflected in mid-period amplification factors in the 2006 IBC, because these factors are largely obtained from extrapolation of empirical site amplification factors for less intense motions and from equivalent-linear analyses that do not capture soil failure. Although seismic site response can be limited by the yield strength of soft soil, seismic instability effects must now be evaluated.

Seismic site response also affects the pulse period of the near-fault FD motion. Deep soil deposits with long degraded site periods tend to lengthen the pulse period of the input rock motion for all cases except when the rock pulse period is much greater than the degraded site period. Representative results for IBC 2006 Site D profiles are shown in Figure 8. The ratio of soil to rock pulse period may be as high as 2 for short pulse periods ($T_v$ < 1 s), and this ratio approaches one as the input rock pulse period exceeds a few seconds. A greater amount of pulse period elongation occurs for very intense FD motions because of the greater nonlinearity in the seismic response of the soil at these intense levels of shaking. Hence, site effects are an important consideration in characterizing near-fault FD ground motions for use in design.

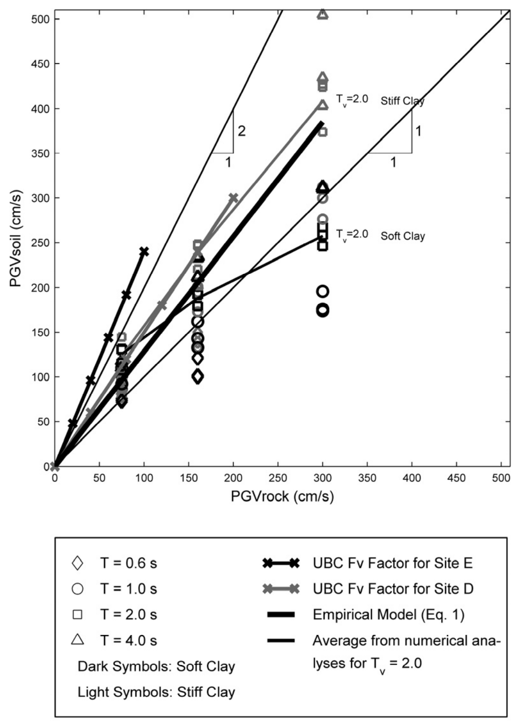



***Figure 7:*** ***Results of seismic analyses of the Soft Clay and Stiff Clay profiles with soil depths of 60 m. Results are shown for various pulse shapes (see [17] for more details). Also shown for comparison are the mid-period amplification factors ($F_v$) from the 2006 IBC, and the predictions using the empirical model in Equation (1).***

**Findings from Analytical Study**

The characteristics of the near-fault FD motions at the surface of a soil deposit are primarily influenced by the characteristics of the input rock motion (i.e., its intensity, pulse period, and number of significant cycles) and the characteristics of the soil profile (i.e., soil type, stiffness, and depth to bedrock). Importantly, the *PGV* of the motion computed at the top of the soil is generally larger than the *PGV* of the rock input motion, with the exception of input motions with large intensities and short pulse periods. The $T_v$ of the soil motion also systematically increases with increasing soil depth or increasing rock input motion intensity. Soil stiffness also affects the amplitude and period of input pulses. The largest amplification of *PGV* for the Very Stiff Soil profile (Site C) occurs for input motions with short pulse periods, whereas the largest amplification of *PGV* for the Stiff Soil profile (Site D) occurs at intermediate periods.

Site conditions play an important role in shaping the characteristics of near-fault FD motions at soil sites. Thus, the important influence of local soil conditions on FD motions should be considered when designing structures in the near-fault region. Site conditions affect the amplitude of the surface motion (i.e., its *PGV*) and its frequency content (i.e., its $T_v$). Fully nonlinear site-specific response analysis is required to capture the nonlinear response of soil deposits under the intense levels of shaking of FD motions. As guidance for this site-specific analysis, the likely range of site and intensity dependent amplification factors for *PGV* can be estimated using Figures 6 and 7, and the amount of pulse period elongation can be estimated using Figure 8.

**CONCLUSIONS**

Near-fault forward-directivity motions typically govern the design of structures built close to active faults. Hence, ground motions for use in evaluating designs in the near-fault region should be selected carefully to represent satisfactorily the unique nature of FD motions. Forward-directivity motions are often intense, pulse-type motions, which are significantly different from ordinary ground motions. These motions are best described by their velocity-time history, which requires estimation of its peak ground velocity (*PGV*), predominant pulse period ($T_v$), and number of significant velocity pulses ($N_c$).

Using recent FD motions, empirical relationships have been updated for estimating the *PGV* and $T_v$ of near-fault FD motions. *PGV* varies significantly with magnitude, distance, and site effects. $T_v$ is a function of magnitude and site conditions with most of the energy being concentrated within a narrow-period band centred on the pulse period. As lower magnitude events produce lower pulse periods, which better matches the low natural period of common buildings in urban areas, FD ground motions from these events have the potential to produce more damage than higher magnitude earthquakes in the near-fault region. Empirical relationships cannot be used at this time to predict $N_c$, because it depends on details of the rupture mechanism that cannot be known a priori. However, it is most likely that near-fault FD motions have only one or two significant cycles of motion. The compact FD wave form produces intense motions that are of short duration. Design near-fault FD velocity-time histories should not have a large number of significant cycles of motion.

Fully nonlinear bi-directional shaking simulations confirm indications from empirical evidence that site effects are important to consider in the near-fault region. It is shown that site effects generally cause $T_v$ to increase, and that amplification of *PGV* depends on site properties, but amplification is generally observed even for very intense rock motions. At soft soil sites, seismic site response can be limited by the yield strength of the soil. In these cases, the seismic stability of the site and the building's foundation elements should be evaluated in terms of seismically induced permanent deformations.

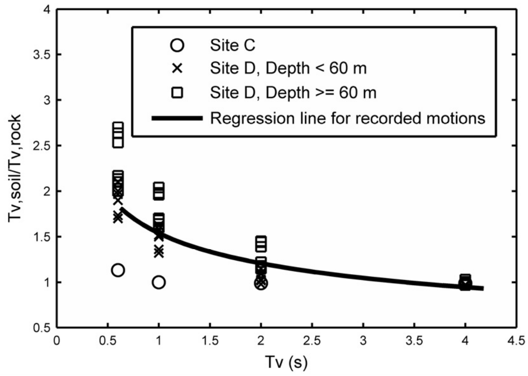



***Figure 8:*** ***Pulse period of soil motion normalized by pulse period of rock motion vs. rock motion pulse period. Results are from seismic site response analyses for IBC 2006 Site C and D profiles. The heavy line represents the results from the Bray and Rodriguez-Marek (2004) regression of empirical records (modified from [13]).***

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