**Eleventh U.S. National Conference on Earthquake Engineering**
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# ACCOUNTING FOR THE INFLUENCE OF GROUND MOTION RESPONSE SPECTRAL SHAPE AND DURATION IN THE EQUIVALENT LATERAL FORCE DESIGN PROCEDURE

Reagan Chandramohan[1], Jack W. Baker[2], and Gregory G. Deierlein[3]

## ABSTRACT

A framework is proposed to explicitly account for the influence of ground motion response spectral shape and duration in the ASCE 7-16 equivalent lateral force design procedure, which currently considers only ground motion intensity, as quantified by $S_a(T_1)$. The scalar, dimensionless parameter $S_a Ratio$ is used to characterise response spectral shape, while significant duration, $Ds$, is used to quantify duration. Design base shear adjustment factors are computed based on ($i$) the *extended* seismic hazard at a site, expressed in terms of the $S_a Ratio$ and $Ds$ values of the anticipated ground motions; and ($ii$) the sensitivity of the structure to the effects of response spectral shape and duration. Since these factors account for the influence of additional ground motion characteristics on structural collapse risk, their use in structural design should help achieve a more uniform distribution of collapse risk over different geographical regions and structural systems, in line with the objective of using risk-targeted seismic design maps. Sample calculations using the *extended* seismic hazard in Los Angeles as a benchmark indicate, for example, that a reinforced concrete moment frame building in Eugene with fundamental elastic modal period 1.0 s would need to be designed to a base shear 67 % higher than the current standard, while a similar structure in San Francisco would need to be designed to a base shear 43 % higher.

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# Accounting for the influence of ground motion response spectral shape and duration in the equivalent lateral force design procedure

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## Introduction

The equivalent lateral force (ELF) procedure is the most widely used among the design procedures described in the ASCE 7-16 [1] structural design standard. It entails the static analysis of a linear structural model under equivalent lateral loads computed from the ordinate of the risk-targeted maximum considered earthquake ($\mathrm{MCE_R}$) spectrum at the fundamental elastic modal period of the structure, $S_a(T_1)$. Hence, the ELF procedure explicitly accounts for only the intensity of the ground

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motions anticipated at a site, as quantified by $S_a(T_1)$, while effectively ignoring their response spectral shapes (spectral ordinates at periods above and below $T_1$) and durations, both of which have been demonstrated by recent studies to influence structural collapse risk [2–5]. This paper proposes a framework to explicitly account for the influence of ground motion response spectral shape and duration in the ELF procedure. Factors are proposed to adjust the design base shear of a structure based on (*i*) the response spectral shapes and durations of the ground motions anticipated at the site it is located; and (*ii*) its sensitivity to their individual effects. Since these factors account for the influence of secondary ground motion characteristics, in addition to intensity, on structural collapse risk, they are expected to help achieve a more uniform distribution of collapse risk over different geographical regions and structural systems, in line with the objective of introducing risk-targeted seismic design maps [6]. Base shear adjustment factors are computed for reinforced concrete moment frame buildings in cities such as San Francisco and Seattle. The framework described in this paper builds on previous recommendations by Liel et al. [7] to account for the differences in the characteristics of ground motions produced by crustal and subduction earthquakes, in structural design.

## Characterisation of ground motion response spectral shape and duration

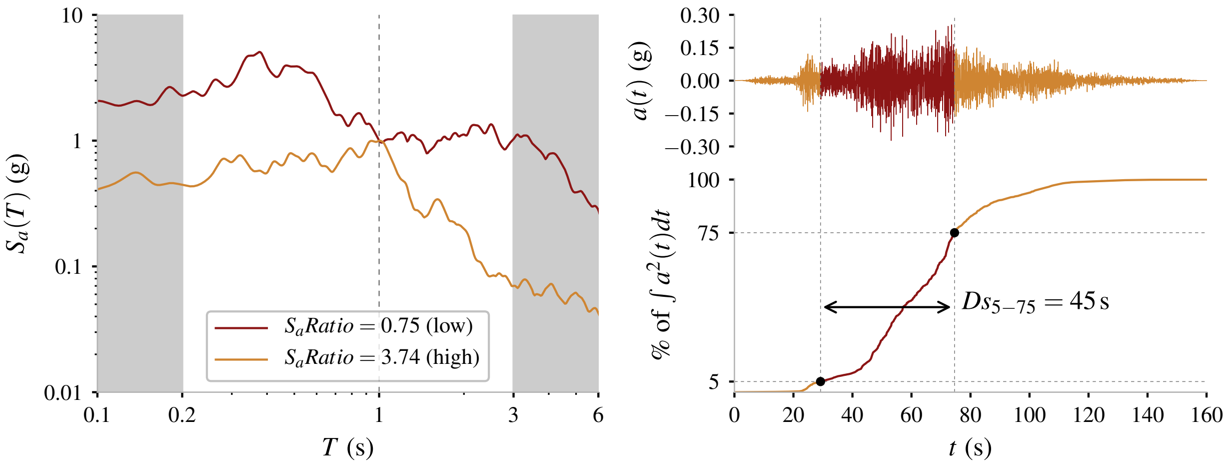
The response spectral shape of a ground motion is characterised in this study, by the scalar, dimensionless parameter $S_aRatio$ proposed by Eads et al. [8]. $S_aRatio$ is computed according to Equation (1a), as the ratio of the pseudo spectral acceleration at a specific period, $S_a(T)$, and the geometric mean of the portion of the response spectrum that lies between the periods $T_{start}$ (usually $< T$) and $T_{end}$ (usually $> T$), denoted by $S_{a,avg}(T_{start}, T_{end})$. $S_{a,avg}(T_{start}, T_{end})$ is, in turn, computed according to Equation (1b), as the geometric mean of response spectral ordinates, discretely sampled at $n$ linearly spaced periods from $T_{start}$ to $T_{end}$: $\tau_1$, $\tau_2$, …, $\tau_n$, such that $\tau_1 = T_{start}$ and $\tau_n = T_{end}$ [2; 9].

$$S_aRatio(T, T_{start}, T_{end}) = \frac{S_a(T)}{S_{a,avg}(T_{start}, T_{end})} \tag{1a}$$

$$S_{a,avg}(T_{start}, T_{end}) = \left( \prod_{j=1}^{n} S_a(\tau_j) \right)^{1/n} \tag{1b}$$

The response spectra of two ground motions with low and high $S_aRatio(1.0\,\mathrm{s}, 0.2\,\mathrm{s}, 3.0\,\mathrm{s})$ values, normalised to have $S_a(1.0\,\mathrm{s}) = 1\,\mathrm{g}$, are plotted in Figure 1a. The ground motion with a low $S_aRatio$ value has relatively high spectral ordinates at periods between 0.2 s and 3.0 s compared to the spectral ordinate at 1.0 s, while the ground motion with a high $S_aRatio$ value exhibits the opposite trend. The period range $0.2T$ to $3.0T$ is used to compute $S_aRatio$ since it was found by Eads et al. [9] to be the most efficient in predicting the collapse capacity of a structure with fundamental elastic modal period $T$.

Ground motion duration is characterised using significant duration [11], $Ds$, which is computed as the time interval over which a specific percentage range of the integral $\int_0^{t_{max}} a^2(t)dt$ is accumulated. $a(t)$ in the integrand represents the ground acceleration at time $t$, and $t_{max}$ represents the length of the accelerogram. 5–75 % significant duration, $Ds_{5-75}$, is used in this paper, and its computation is illustrated in Figure 1b.



(a)

(b)

**Figure 1:** (a) Response spectra of two ground motions with low and high $S_aRatio(1.0\,\mathrm{s}, 0.2\,\mathrm{s}, 3.0\,\mathrm{s})$ values, normalised to have $S_a(1.0\,\mathrm{s}) = 1$ g. The vertical line at 1.0 s corresponds to the period at which $S_a$ in the numerator of Equation (1a) is computed, and the unshaded period range from 0.2 s to 3.0 s corresponds to the domain over which $S_{a,avg}$ in the denominator of Equation (1a) is computed. (b) Computation of the 5–75 % significant duration of an accelerogram illustrated using a plot of the normalised, cumulative integral of $a^2(t)$, known as a Husid plot [10].

## Extended seismic hazard assessment in terms of response spectral shape and duration

The first component required to compute the design base shear adjustment factors is the site-specific *extended* seismic hazard, which provides a description of the median response spectral shape and duration of the anticipated ground motions, in addition to their intensity. Since ground motions of different intensities observed at a site are expected to possess inherently different response spectral shapes and durations, it is conventional practice to compute median anticipated $S_aRatio$ and $Ds$ values conditional on the exceedance of a specific spectral acceleration value at the fundamental elastic modal period, $S_a(T_1)$. The conditional median anticipated $S_aRatio$ can be directly computed as the $S_aRatio$ value of the conditional mean spectrum [12], using Equations (1a) and (1b). The conditional median anticipated $Ds$, on the other hand, can be computed using the generalised conditional intensity measure (GCIM) framework [13], following the procedure outlined in Chandramohan et al. [5]. Critical ingredients in these computations are prediction models for both response spectra [e.g., 14; 15] and duration [e.g., 16; 17], and models for the correlation between their total prediction residuals ($\varepsilon$-values) [e.g., 18; 19].

## Structural sensitivity to the effects of response spectral shape and duration

The second component in the computation of design base shear adjustment factors is the sensitivity of the collapse capacity of the structure to the effects of ground motion response spectral shape and duration. To estimate the sensitivity of a given structure, it is first analysed by conducting incremental dynamic analysis (IDA) [20] using a generic set of ground motions that cover a wide range of $S_aRatio$ and $Ds$ values. The multiple linear regression model described by Equation (2) is then fit to the estimated ground motion collapse intensities, using the $S_aRatio$ and $Ds$ values of the ground motions as predictors. The computed regression coefficients $c_{ss}$ and $c_{dur}$ quantify the sensitivity of the structure to the effects of response spectral shape and duration respectively.

$$\ln S_a(T_1) \text{ at collapse} = c_0 + c_{ss} \ln S_aRatio + c_{dur} \ln Ds + \epsilon \tag{2}$$

Conducting IDA to estimate the $c_{ss}$ and $c_{dur}$ coefficients for each newly designed structure would, however, be cumbersome and impractical. Hence, simplified equations are required to predict these coefficients as functions of structural characteristics. It is recognised that the sensitivities of structures of different materials and lateral force resisting systems to the effects of response spectral shape and duration, are likely to be different. Different equations would, therefore, need to be developed to predict $c_{ss}$ and $c_{dur}$ for each class of structures. 51 ductile reinforced concrete moment frames designed by Raghunandan et al. [21], and Haselton and Deierlein [22]—ranging in height from 1 to 20 stories—were analysed as part of this study to develop such a predictive model for reinforced concrete moment frames. Two-dimensional nonlinear lumped plasticity models of the structures were created in OpenSees [23]. The hysteretic behaviour of the plastic hinges located at either end of all beams and columns was modelled using the Ibarra-Medina-Krawinkler peak-oriented model [24], which captures the in-cycle and cyclic deterioration in strength and stiffness. The destabilising effect of the adjacent gravity frames was modelled using pin-connected leaning columns. Previous studies by the authors [e.g., 25; 26] have shown both these model attributes to be necessary to capture the effect of duration on structural collapse capacity. The models were analysed by conducting IDA using a set of 88 ground motions, consisting of 44 short duration records from the FEMA P65 far field set [27] and 44 long duration records. The computed values of $c_{ss}$ and $-c_{dur}$ are plotted in Figure 2, which indicates a decreasing trend in the magnitudes of both coefficients with fundamental elastic modal period, $T_1$. These trend lines can now be used to estimate the coefficients as a function of $T_1$. Trends with respect to other structural characteristics were investigated, but found to be insignificant.

## Computation of design base shear adjustment factors

ASCE 7-16's implicit performance objective, as stated in Section C1.3.1.3 of [1], is to achieve a 10 % probability of collapse under the $\text{MCE}_\text{R}$ level ground motion, for Risk Category I and II structures. The collapse potential of a ground motion is, however, seen to be influenced not only by its intensity, but also its response spectral shape and duration. This indicates that the statement of the performance objective is incomplete and requires the additional definition of the $S_aRatio$ and $Ds$ values of the ground motion under which it holds true. These reference $S_aRatio^{ref}$ and $Ds^{ref}$ values are assumed to be equal to the median $S_aRatio$ and $Ds$ values of the ground motions anticipated at Los Angeles, conditional on the exceedance of the intensity level corresponding to $2.2 \times \text{MCE}_\text{R}$. The $2.2 \times \text{MCE}_\text{R}$ intensity level approximately corresponds to the median collapse capacity of a newly designed structure, assuming its collapse fragility curve is (*i*) anchored to a 10 % collapse probability at the $\text{MCE}_\text{R}$ intensity level; and (*ii*) characterised by a lognormal standard deviation of 0.6 [1]. The choice of Los Angeles as the reference site was motivated by the historical emphasis on sites in coastal California when calibrating seismic design codes, which suggests that the stated performance objective is likely to be valid here. Although San Francisco could alternatively be considered as the reference site by the same arguments, Los Angeles is used in this study since it represents the more conservative of the two options. Additional benchmarking studies are, however, required to validate the choice of reference $S_aRatio^{ref}$ and $Ds^{ref}$ values.

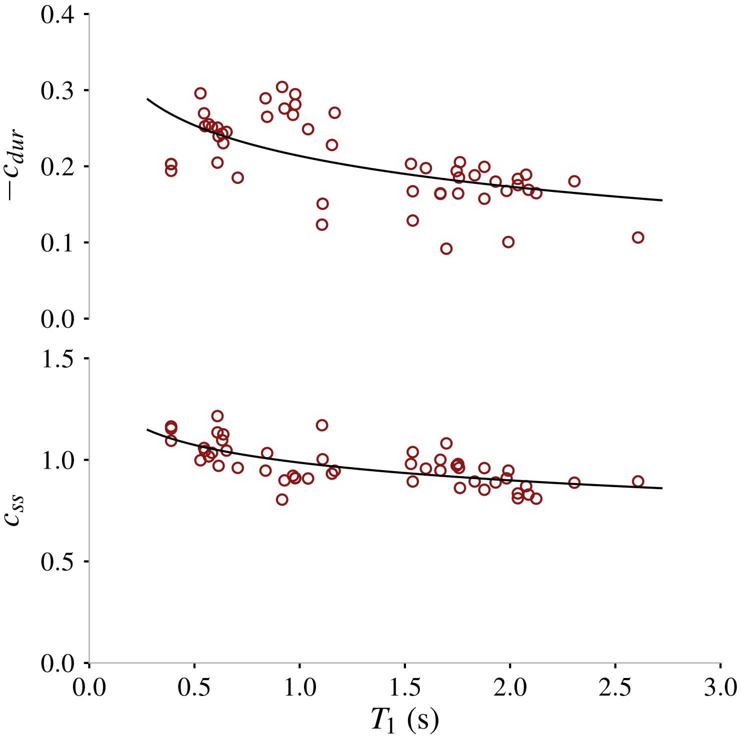


**Figure 2:** $-c_{dur}$ and $c_{ss}$ values for all analysed reinforced concrete moment frames plotted against their fundamental elastic modal periods, $T_1$. $-c_{dur}$ is plotted instead of $c_{dur}$ since the coefficient is generally negative. The least-squares regression curves are computed by regressing the coefficients against $\ln T_1$.

As per the current ELF procedure, the design base shear of a structure, $V$, is computed using Equation (3), where $C_s$ denotes the seismic response coefficient, and $W$ the effective seismic weight of the structure.

$$V = C_s W \tag{3}$$

Since $C_s$ is directly proportional to the ordinate of the $\mathrm{MCE_R}$ spectrum at the fundamental elastic modal period of the structure, so is the design base shear, $V$. The ASCE 7-16 performance objective, therefore, indicates an implicit relationship between the design base shear and the collapse capacity of a structure. This suggests that the influence of response spectral shape and duration could be accounted for in the ELF design procedure, by modifying the design base shear based on the effect they are expected to have on the collapse capacity of the structure. Although this line of reasoning appears to be reasonable, additional benchmarking studies are required to validate the underlying assumptions, particularly in cases where conditions such as the minimum base shear requirement govern the design of the structure.

The linear relation between the logarithms of a ground motion's collapse intensity and its $S_aRatio$ and $Ds$ values was previously established in Equation (2). Hence, if the structure is to be designed at a site with median anticipated $S_aRatio$ and $Ds$ values denoted by $S_aRatio^{site}$ and $Ds^{site}$ respectively, in order for it to possess a 10 % collapse probability at the $\mathrm{MCE_R}$ level, it must be designed to a modified base shear given by Equation (4a). The base shear adjustment factors $k'_{ss}$ and $k'_{dur}$ are computed using Equations (4b) and (4c) respectively, and are observed to depend on both the site-specific *extended* seismic hazard ($S_aRatio^{site}$ and $Ds^{site}$), as well as structural sensitivities ($c_{ss}$ and $c_{dur}$). It is worth noting that these equations remain valid even if new structures possess an average collapse probability of some $x$ % under the reference ground motion, that is different from 10 %.

$$V = k'_{ss}k'_{dur}C_sW \tag{4a}$$

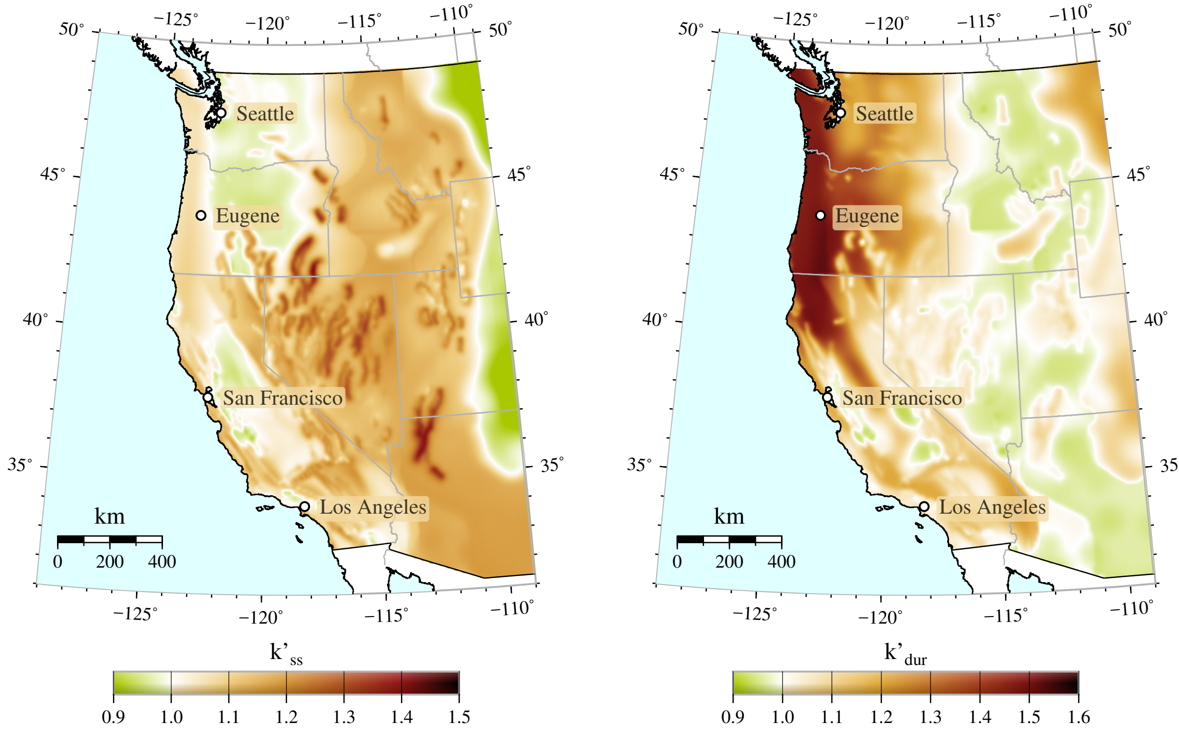
$$k'_{ss} = \left(\frac{S_aRatio^{ref}}{S_aRatio^{site}}\right)^{c_{ss}} \tag{4b}$$

$$k'_{dur} = \left(\frac{Ds^{ref}}{Ds^{site}}\right)^{c_{dur}} \tag{4c}$$

Maps of base shear adjustment factors $k'_{ss}$ and $k'_{dur}$ for reinforced concrete moment frame buildings with $T_1 = 1.0$ s in Western USA are plotted in Figure 3. $k'_{ss}$ and $k'_{dur}$ factors for reinforced concrete moment frame buildings of different periods, located at a few representative sites in Western USA, are listed in Table 1. $k'_{dur}$ values are observed to be large in the Pacific Northwest region, since it is susceptible to long duration ground motions from large magnitude subduction earthquakes ($M_W \sim 9.0$), while the reference site, Los Angeles, experiences mostly short duration ground motions from moderate magnitude earthquakes ($M_W \sim 7.0$) on adjacent crustal faults. It is worth noting that San Francisco has $k'_{dur}$ values comparable to Seattle since it is susceptible to relatively large magnitude crustal earthquakes ($M_W \sim 8.0$) on the San Andreas fault, while Seattle is prone to both moderate magnitude crustal and in-slab earthquakes ($M_W \sim 7.0$), as well as large magnitude subduction earthquakes ($M_W \sim 9.0$). $k'_{ss}$ values, on the other hand, are seen to be high at sites located along active crustal faults. These factors may be interpreted as follows. A 1.0 s reinforced concrete moment frame building in Eugene needs to be designed to a base shear 67 % higher than the value computed using Equation (3), in order to satisfy the stated performance objective. A similar building in San Francisco needs to be designed to a base shear 43 % higher. Choosing San Francisco as the reference site instead of Los Angeles would, however, entail no modification to the design base shear of structures designed in San Francisco, illustrating the impact of the choice of reference $S_aRatio^{ref}$ and $Ds^{ref}$ values. $k'_{ss}$ and $k'_{dur}$ values computed for a few other cities and conditioning periods using both Los Angeles and San Francisco as reference sites can be found in Chandramohan [28, Chapter 6].

## Conclusion

A framework was developed to explicitly account for the influence of ground motion response spectral shape and duration in the ELF design procedure, which currently considers only ground motion intensity. This framework is based on the premise that structures designed using the ELF procedure possess an approximately constant collapse probability under the MCE$_\text{R}$ level ground motion with a prescribed reference response spectral shape and duration, which directly follows from ASCE 7-16's stated performance objective. Response spectral shape is characterised using the scalar, dimensionless parameter $S_aRatio$, while duration is quantified using significant duration, $Ds$. Design base shear adjustment factors $k'_{ss}$ and $k'_{dur}$ are computed based on ($i$) the *extended*



(a)

(b)

**Figure 3:** Base shear adjustment factors (a) $k'_{ss}$ and (b) $k'_{dur}$ for 1.0 s reinforced concrete moment frame buildings based on Los Angeles as the reference site, computed using anticipated $S_a Ratio$ and $Ds$ values conditional on the 0.5 % in 50 year exceedance probability of $S_a(1.0\,\mathrm{s})$, assuming $Vs_{30} = 760\,\mathrm{m/s}$.

seismic hazard at the site, expressed in terms of the $S_a Ratio$ and $Ds$ values of the anticipated ground motions; and (*ii*) the sensitivity of a structure to the effects of response spectral shape and duration. Since these factors account for the influence of secondary ground motion characteristics on structural collapse risk, their use in structural design is likely to help ensure a more uniform distribution of collapse risk over different geographical regions and structural systems. $k'_{dur}$ is observed to be large at sites susceptible to long duration ground motions from large magnitude earthquakes, e.g., sites in the Pacific Northwest. $k'_{ss}$, on the other hand, is observed to be large at sites located adjacent to active crustal faults. Sample calculations indicate that a reinforced concrete moment frame building in Eugene with $T_1 = 1.0$ s needs to be designed to a base shear 67 % higher than the current standard, while a similar structure in San Francisco needs to be designed to a base shear 43 % higher, in order to satisfy the stated performance objective. The $S_a Ratio$ and $Ds$ values of the ground motions anticipated in Los Angeles were used as a reference to compute the $k'_{ss}$ and $k'_{dur}$ factors in this paper. This choice of reference $S_a Ratio$ and $Ds$ values has a significant impact on the computed factors, and additional benchmarking studies are necessary to determine appropriate reference values. Although the framework developed here is applicable to the ELF design procedure, it could potentially be extended to other design procedures such as the modal response spectrum and response history analysis procedures, which are also assumed to satisfy the same performance objectives, with some additional considerations.

**Table 1:** Median anticipated $S_a Ratio$ and $Ds$ values conditional on the exceedance of the 2.2×MCE$_R$ ground motion intensity level at different periods, assuming $Vs_{30} = 760\,\mathrm{m/s}$; and corresponding $k'_{ss}$ and $k'_{dur}$ base shear adjustment factors for reinforced concrete moment frame buildings computed at different sites in Western USA using Los Angeles as the reference site.

| **City** | **Conditioning period** (s) | **Median anticipated** $S_a Ratio$ | $k'_{ss}$ | **Median anticipated** $Ds_{5-75}$ (s) | $k'_{dur}$ | $k'_{ss}k'_{dur}$ |
|---|---|---|---|---|---|---|
| Eugene | 0.5 | 1.97 | 1.04 | 23.4 | 1.65 | 1.72 |
| | 1.0 | 2.16 | 1.11 | 29.4 | 1.50 | 1.67 |
| | 2.0 | 2.12 | 1.10 | 34.7 | 1.37 | 1.50 |
| Seattle | 0.5 | 1.89 | 1.09 | 4.6 | 1.10 | 1.20 |
| | 1.0 | 2.20 | 1.09 | 9.6 | 1.18 | 1.29 |
| | 2.0 | 2.25 | 1.04 | 19.5 | 1.24 | 1.29 |
| San Francisco | 0.5 | 2.05 | 1.00 | 6.7 | 1.20 | 1.21 |
| | 1.0 | 2.02 | 1.19 | 10.2 | 1.20 | 1.43 |
| | 2.0 | 1.91 | 1.20 | 13.0 | 1.16 | 1.40 |
| Los Angeles | 0.5 | 2.06 | 1.00 | 3.2 | 1.00 | 1.00 |
| | 1.0 | 2.40 | 1.00 | 4.3 | 1.00 | 1.00 |
| | 2.0 | 2.34 | 1.00 | 5.4 | 1.00 | 1.00 |

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