GROUND MOTION INTENSITY MEASURES FOR COLLAPSE CAPACITY PREDICTION: CHOICE OF OPTIMAL SPECTRAL PERIOD AND EFFECT OF SPECTRAL SHAPE

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ABSTRACT

Previous research has shown that for extreme ground motions in California, the spectral shape is much different than that of a uniform hazard spectrum or a code design spectrum; and that differences in spectral shape significantly impact results of nonlinear dynamic analyses. This paper builds on this previous research and looks more closely at the question of selecting and scaling ground motions for nonlinear dynamic *collapse* analyses.

Based on dynamic collapse simulation of single-degree-of-freedom models subjected to 70 ground motions, this work confirms the significant effects that spectral shape has on collapse capacity estimates. However, for collapse analyses, we find that the use of a more appropriate ground motion intensity measure can reduce how sensitive the collapse capacity estimates are to the spectral shape of the ground motions.

A common ground motion intensity measure is the spectral acceleration at the building first mode period. However, when first-mode dominated ductile structures behave nonlinearly and are near collapse, the "effective period" of the structure may be much larger than the fundamental period. For these types of structures, this paper shows that the spectral acceleration at a period larger than the fundamental period is a more appropriate intensity measure for use in collapse simulation. This paper proposes an equation that predicts this optimal period as a function of the building properties: fundamental period, ductility, and the negative stiffness after reaching the ductility capacity. For ductile structures, this optimal extended period is *approximately twice* the fundamental period of the building.

Introduction and Goals

One of the many challenges of using analytical models to predict structural collapse capacity is the choice of ground motions to use in simulation. A key characteristic of ground motions, which is often not well quantified, is the *spectral shape*. Baker (Baker 2005, chapter 6) has shown that for a 2% in 50 year ground motion in California, the spectral shape is much different than the shape of the Code design spectrum or a uniform hazard spectrum.

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To illustrate Baker's finding regarding the spectral shape, Fig. 1 show the acceleration spectrum of a Loma Prieta ground motion[3] (PEER 2005) that has a 2% in 50 year intensity at a period of 1.0 second [which is $Sa(T_1 = 1.0 \text{ sec}) = 0.9g$ for this example site]. This figure also shows the ground motion predicted by Boore et al. (1997), consistent with this event and site.

Fig. 1 clearly shows that this extreme 2% in 50 year motion has an unusual spectral shape with a "peak" from 0.6 to 1.8 seconds; much different than the shape of a uniform hazard spectrum. Notice that this peak corresponds with the period for which this motion is considered to have a 2% in 50 year intensity. At 1.0 second, the spectral value is 1.9 standard deviations above the predicted median spectral value, so this record is said to have "$\varepsilon = 1.9$ at 1.0 second." ε (epsilon) is defined as the number of standard deviations between the observed spectral value and the median prediction from an attenuation function (so ε depends on the attenuation function used). Similarly, this record has $\varepsilon = 1.1$ at 1.8 seconds, showing that ε depends also on period.

Baker has shown that observations made from Fig. 1 are general to all coastal California sites, so we can expect approximately $\varepsilon = +2$ for the 2% in 50 year ground motion level at such sites. Therefore, if $\varepsilon = +2$ is expected for 2% in 50 year motions in coastal California, when at such sites, we should select records with consistent ε (Baker 2005, chapter 6). It should be noted that this expected ε is both hazard level and site dependent. For example, for the 50% in 5 year ground motions in coastal California, $\varepsilon = 0$ to $\varepsilon = -2$ are expected (Haselton at al. 2005). In addition, in the Eastern United States, $\varepsilon = 0$ is expected for 2% in 50 year motions.
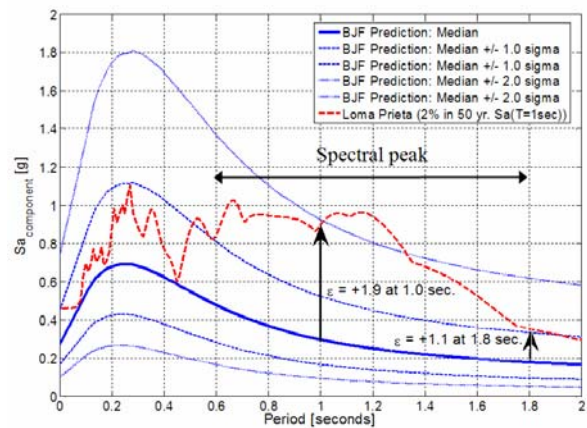

Figure 1. Comparison of an observed spectrum with spectra predicted by Boore, Joyner, and Fumal (1997).

Research also shows that this peaked spectral shape significantly increases the collapse capacity when the peak of the spectrum is near the fundamental period of the building ($T_{1,struct}$) and we scale the ground motions based on $T_{1,struct}$ (Goulet et al. 2006; Baker 2005a, chapter 6) [e.g. for an example structure with $T_{1,struct} = 1.0$ sec., we would select records to have $\varepsilon = +2$ at 1.0 sec. and then scale all records to a target $Sa(T_{IM} = 1\text{sec})$[4]]. What if we select and scale records at a different period that may be more appropriate for collapse analyses?

Subjected to large ground motion, ductile structures will behave nonlinearly and soften causing the "effective period" of the structure to increase. This suggests that if we want to better explain how the ground motions effect the nonlinear building response, it may be appropriate to use $Sa(T_{IM})$, where T_{IM} is not $T_{1,struct}$, but is a lengthened period. In this case, we would select the ground motions to have the proper ε at the extended period ($\varepsilon = +2$ for examples in this paper) and scale the records to a target $Sa(T_{IM})$ at the same extended period. The questions

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[3] This spectrum was scaled by a factor of 1.4 to be consistent with the point being illustrated.
[4] T_{IM} is the period used in the ground motion intensity measure. This is the period at which ground motions are both a) selected to have the proper ε, and b) scaled to the target spectral value.

becomes this: If we select and scale ground motions at an extended period, thus having the peaks of the spectra at this extended period instead of the fundamental period, how will the difference in location of spectral peaks change collapse capacity predictions?

This research has two goals: 1) to find the optimal IM period, $T_{col,opt}$, which can be used to predict collapse capacity with the least error, and 2) to learn if ε affects the collapse capacity differently when the ground motions are selected and scaled for $Sa(T_{IM} = T_{col,opt})$ instead of $Sa(T_{IM} = T_{1,struct})$ (i.e. when the peaks of the spectra are at $T_{col,opt}$ rather than $T_{1,struct}$). To answer these questions, we selected sets of ground motions both without considering ε and with $\varepsilon = +2$ at various periods.[5] We then used these motions to predict the collapse capacities of single-degree-of-freedom oscillators of various periods, ductilities, and other structural parameters.

Ground Motion Sets

We selected eight ground motion sets (PEER 2005) which are roughly consistent with motions that may cause collapse of new buildings, excluding near-fault motions: M > 6.5, R > 10km, and PGA > 0.2g (Kircher 2005). Table 1 outlines the sets: Set A was selected without considering ε and the other seven sets were selected to have $\varepsilon = +2$ at a given period[5] (termed "Set $\varepsilon_\#$", where # is the period at which the ground motions were selected to have $\varepsilon = +2$). The ε-selected sets are not mutually exclusive (because the $+\varepsilon$ peak extends over a range of periods, as Fig. 1 shows), so the ε-selected sets consist of 65 records in total. Fig. 2 compares the mean spectra of Sets A, $\varepsilon_{1.0}$, $\varepsilon_{2.0}$, and the Code spectrum (only two of seven ε-selected sets are shown for clarity). Fig. 2 shows the clear effect that ε has on the mean spectral shape.

For the purpose of more clearly illustrating the points in this paper (i.e. the effects of ε), we scaled Set A to have a target Sa value at a 1.0 second period, then used the median spectrum of Set A as the 2% in 50 year uniform hazard spectrum (used for later comparisons).

Table 1. Brief summary of eight ground motion sets used in this study.

Ground Motion Set	Period Used for Record Selection, T_{IM} (sec.) ($\varepsilon = +2$)	Number of Records
Set A	n/a*	26
Set $\varepsilon_{0.5}$	0.5	20
Set $\varepsilon_{1.0}$	1.0	20
Set $\varepsilon_{1.5}$	1.5	20
Set $\varepsilon_{2.0}$	2.0	20
Set $\varepsilon_{2.4}$	2.4	20
Set $\varepsilon_{3.0}$	3.0	20
Set $\varepsilon_{4.0}$	4.0	20

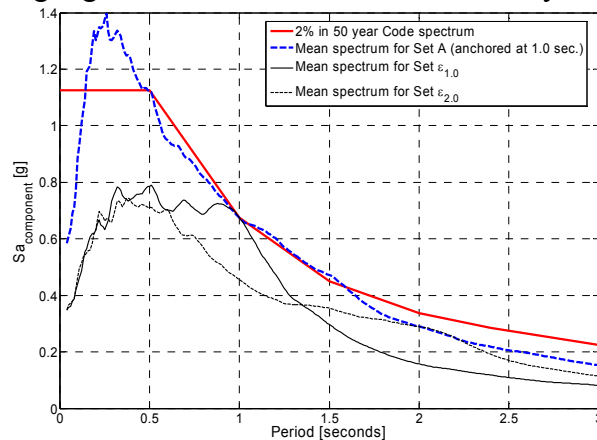* set selected without considering ε

Figure 2. Comparisons of mean spectra for three ground motion sets and Code spectrum (ASCE 7-02 2002).

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[5] The criteria was relaxed to $\varepsilon > +1.5$ or $+1$ for some periods, in order to find at least 20 ground motion records for each set.

Single-Degree-of-Freedom Models

We used an array of single-degree-of-freedom (SDOF) models to meet the objectives of this study. The material model used was developed by Ibarra and Krawinkler (Ibarra 2003, Ibarra et al. 2005). This model has a trilinear backbone and is similar to the Clough model (i.e. peak-oriented) but also accounts for strength deterioration (both monotonic and cyclic). Fig. 3 shows the backbone of the material model. μ is the displacement ductility to the onset of the negative stiffness; and K_c is the post-capping negative stiffness ($K_c = \alpha_c K_e$). This study uses a cyclic deterioration rate consistent with conforming reinforced concrete elements (Haselton et al. 2005, chapter 5).
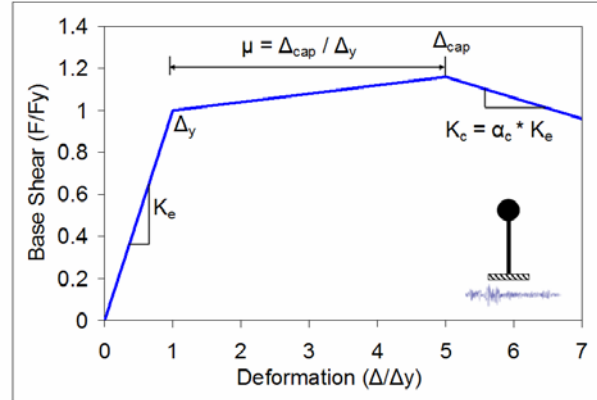

Figure 3. Backbone of material model used for SDOF (Ibarra et al. 2005)

Each SDOF was designed for 2/3 of the 2% in 50 ground motion level (ASCE7 2002) with Sa_{design} / Sa_{yield} = 2.4 (to account for an R-factor and overstrength). Table 2 lists the SDOF models used in this study, which represent a range of structural parameters ($T_{1,struct}$, μ, and α_c) to ensure that the conclusions of this study are not limited to a specific structural configuration. P-Δ is treated according to guidelines proposed by Ibarra (2003, chapter 4); this is based on a stability coefficient and the value used results in a relatively small amount of P-Δ.

For the collapse simulation of each SDOF model, we used the incremental dynamic analysis (IDA) approach and systematically scaled up the ground motion intensity until reaching the collapse capacity (Vamvatsikos 2002). The collapse capacity associated with a given ground motion is theoretically defined as the Sa(T) that first causes dynamic instability where drifts increase without bounds. For simpler implementation, collapse capacity is strictly defined as the Sa(T) that causes the ductility demand to be $5.0(\Delta_{cap} / \Delta_y)$[6].

Table 2. Summary of parameters of SDOF models use in this study.

$T_{1,struct}$ (sec)	μ	α_c (no P-Δ)	α_c (with P-Δ)
1.0	2	-0.10	-0.12
1.0	2	-0.30	-0.33
1.0	4	-0.05	-0.07
1.0	4	-0.10	-0.12
1.0	4	-0.20	-0.22
1.0	4	-0.30	-0.33
1.0	4	-0.40	-0.44
1.0	4	-0.50	-0.53
1.0	6	-0.10	-0.12
1.0	6	-0.30	-0.33
2.0	2	-0.10	-0.15
2.0	2	-0.30	-0.35
2.0	4	-0.10	-0.15
2.0	4	-0.30	-0.35
2.0	6	-0.10	-0.15
2.0	6	-0.30	-0.35

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[6] The 5.0 factor is appropriate for most cases, but 10.0 would be more appropriate for systems with low ductility and shallow post-capping stiffness.

Effects of Record Selection and ε on Predicted Collapse Capacities

This section shows an example of how collapse capacity prediction is affected by considering ε in ground motion selection. This example shows that using an extended period for the ground motion IM causes the collapse capacity prediction to be less sensitive to spectral shape (i.e. the ε of the ground motion spectra). Results in this section are based on an SDOF with $T_{1,struct}$ = 1.0 sec., μ = 4, and $α_c$ = -0.10. Even though the presented results are based on a single SDOF model, trends are similar for all SDOF models considered in this study.

Fig. 4 compares the collapse capacity distributions for ground motion Sets A, $ε_{1.0}$, and $ε_{2.0}$. Note that for the ε-selected ground motion sets, the period used for ground motion selection and scaling are consistent (i.e. T_{IM}). Fig. 4a shows this comparison for T_{IM} = 1.0 second and shows that the median collapse capacity predicted using Set A is *45% smaller* than the capacity predicted using Set $ε_{1.0}$. This difference is consistent with the difference between the median spectra of the two ground motion sets (seen in Fig. 2) at a period of 2.0 seconds (which Table 3 will later show to be near the optimal period for use in the IM). This difference in collapse capacities is consistent with differences observed in similar work (Baker 2005, chapter 8). Fig. 4b shows the same comparison as 4a, but using an IM period of T_{IM} = 2.0 seconds (i.e. selecting records to have ε = +2 at 2.0 seconds, and scaling the records at 2.0 seconds). This shows that the median collapse capacity is *only 25% smaller* when using Set A as compared to Set $ε_{2.0}$.
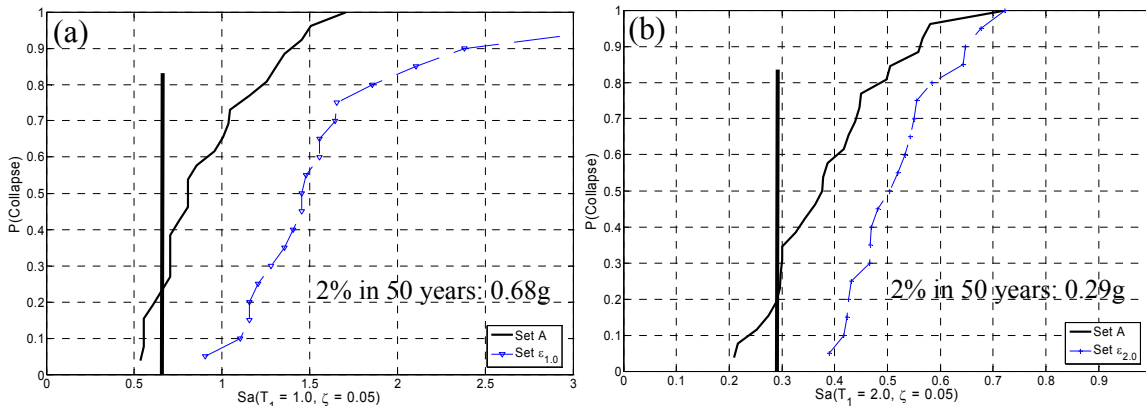

Figure 4. Collapse capacity distributions for SDOF with ($T_{1,struct}$ = 1.0 sec., μ = 4, and $α_c$ = -0.10), with and without considering ε, for ground motion selection at a) T_{IM} = 1.0 sec, and b) T_{IM} = 2.0 sec. 2% in 50 year Sa levels shown for reference.

Figs. 4a-b show that using an extended period in the ground motion IM will make collapse capacity predictions *less sensitive* to ground motion selection (i.e. less sensitive to ε). However, even when the extended period is used, the collapse capacity with ground motion Set A is *still 25% smaller* than with Set $ε_{2.0}$, which is still a *significant difference*. We consider using ε-selected sets to be the correct method, so selecting ground motions not considering ε can lead to *extremely conservative collapse predictions* (25-45% depending on period used for IM).

Fig. 4 compared the median collapse capacities for only T_{IM} = 1.0 and 2.0 sec. Fig. 5a extends this comparison to all seven ε-selected ground motion sets. This figure shows the ratio between the median collapse capacity and the 2% in 50 year demand, termed "collapse capacity

margin." This shows that collapse capacity margins for Set A and the ε-selected sets are most similar between 1.5 and 2.5 seconds, meaning that in this period range the collapse capacity predictions are least sensitive to record selection (and the effects of ε).

Fig. 5b shows the dispersions of collapse capacity for Set A and the ε-selected sets. This shows that the dispersion values are similar for both ground motion selection methods, and they tend to be lowest when T_{IM} is between 1.5 and 2.5 seconds.
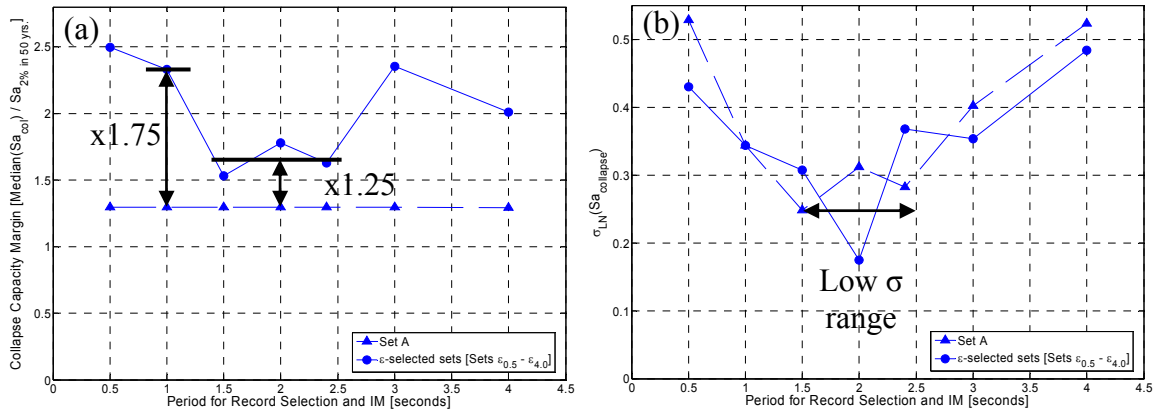

Figure 5.   a) Collapse capacity margin and b) dispersion of collapse capacity [$\sigma_{LN(Sacollapse)}$], with and without considering ε in ground motion selection.

Figs. 4 and 5 illustrated the effects of ground motion selection method on the predicted distribution of collapse capacity. To complete this simplified probabilistic collapse assessment, one can integrate the conditional probabilities of collapse (given the ground motion intensity) with the ground motion hazard curve to obtain the mean annual frequency of collapse ($\lambda_{collapse}$) (Baker et al. 2003, Eq. 3.66). Fig. 6a illustrates the results of this process for Set A (at various IM periods) and the ε-selected sets (at periods consistent with the ground motion selection).

Several interesting observations can be made from the results of Fig. 6a; these results are consistent with those obtained for similar analyses of the other SDOF structures considered.
- The $\lambda_{collapse}$ predictions using Set A are sensitive to the period used for the IM, while the values obtained using the various ε-selected sets (based on different periods) are reasonably consistent (i.e. for the ε-selected sets the computed $\lambda_{collapse}$ does not seem to be significantly affected by the period used for the IM).
- Even though there is no observed trend between $\lambda_{collapse}$ and T_{IM} (for the ε-selected sets), the use of different ground motion records causes significant scatter in the prediction of $\lambda_{collapse}$ (*a factor of two* difference for T = 1.5-2.5 for the ε-selected sets).
- If this $\lambda_{collapse}$ assessment was completed only at $T_{1,struct}$ (as is common practice today), Set A would result in a $\lambda_{collapse}$ that is *a factor of six larger* than if Set $\varepsilon_{1.0}$ were used. For all SDOFs used in this study, this factor ranges from *three to six*.
- At periods where Sets A and the ε-selected sets give most similar results (near $T_{col,opt}$), the $\lambda_{collapse}$ predicted with Set A is *only a factor of two larger* than with the ε-selected sets. For all SDOFs used in this study, this factor ranges from *one to three*.
- When using an extended period for the IM (near $T_{col,opt}$) rather than $T_{1,struct}$, the $\lambda_{collapse}$ prediction is less sensitive to ground motion selection and ε (i.e. spectral shape).
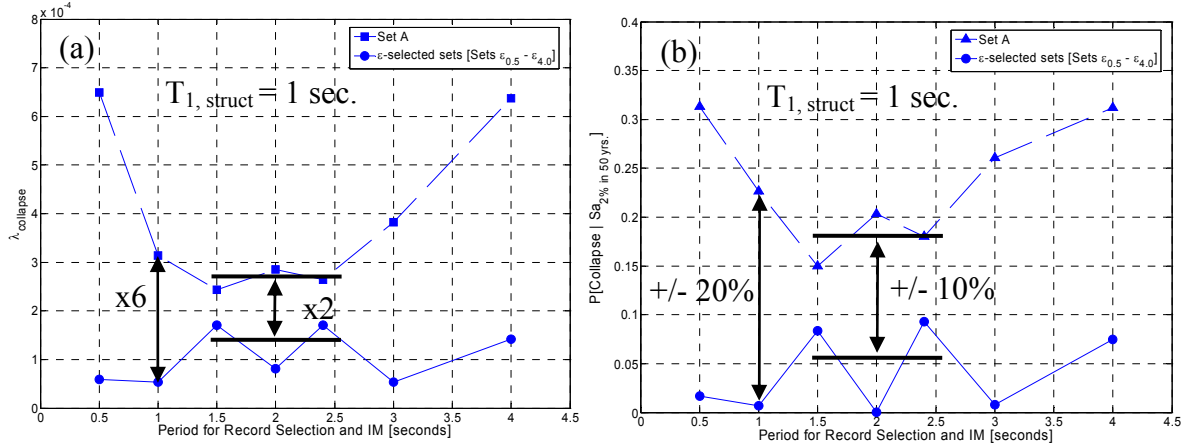
Figure 6. a) Mean annual frequency of collapse and b) P[Collapse | Sa$_{2\% \text{ in } 50 \text{ yrs.}}$] with and without considering ε in ground motion selection.

Fig. 6b is similar to 6a but shows the P[Collapse | Sa$_{2\% \text{ in } 50 \text{ yrs.}}$] rather than λ$_{\text{collapse}}$. This shows trends similar to Fig. 6a, but results are more variable because they are sensitive to the estimated tails of the distributions (see Fig. 4). At T$_{1,\text{struct}}$ = 1 sec., the P[Collapse | Sa$_{2\% \text{ in } 50 \text{ yrs.}}$] is 20% larger for Set A; this difference reduces to 10% when using an extended period.

This section has shown that the collapse capacity and collapse rate predictions are less sensitive to ground motion selection method when an extended period is used in the IM. This is simply because the IM at the extended period can predict the collapse capacity with less uncertainty. The next section determines the optimal extended period to use in the IM.

Optimal Period for Ground Motion Intensity for Collapse Analysis

As a structure is damaged and softens, the effective fundamental period of the structure increases. When the structure is near collapse the fundamental period may be significantly longer than the initial undamaged period; and the amount of elongation is a function of the properties of the structural system (μ, α$_c$, etc.). This section shows how to predict the optimal extended period to use in the IM for collapse analyses. Using this extended period (as opposed to T$_{1,\text{struct}}$) as the period for the ground motion IM will provide two benefits:

- Collapse capacity estimates will be *less sensitive* to effects of ε.
- The IM using the extended period is more *efficient*, so fewer simulations are required to obtain equally precise predictions (Shome et al. 1998).
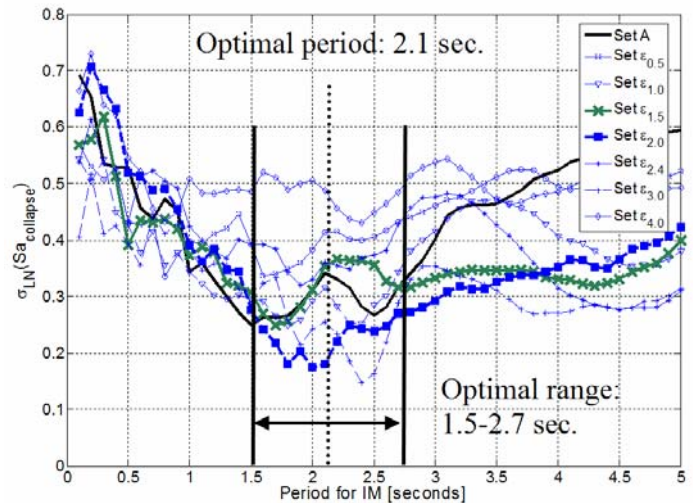

Figure 7. Dispersion of collapse capacity [σ$_{LN(\text{Sacollapse})}$] as a function of IM period, for all eight record sets.

Fig. 7 shows the dispersion [$\sigma_{LN(Sacollapse)}$] of collapse capacity for all eight record sets, using many periods for the IM (T_{IM} = 0 to 5 seconds). This example is based on the same SDOF used in previous examples: $T_{1,struct}$ = 1.0 sec., μ = 4, and α_c = -0.10. The results for ground motion Set A, and Sets $\varepsilon_{1.5}$ and $\varepsilon_{2.0}$ (which are for periods closest to the optimal extended period) are shown with heavier lines for emphasis. This figure shows that the optimal collapse period range (i.e. the periods associated with the lowest dispersion in collapse capacity) is 1.5-2.7 seconds[7] and the optimal period is approximately 2.1 seconds.[8]

The same procedure of finding the optimal period was completed for all of the SDOF models listed in Table 2. Table 3 presents the results for all SDOFs and shows that the optimal period clearly depends on μ and α_c. Specifically, for the SDOFs with $T_{1,struct}$ = 1.0 second and μ = 4, α_c only affects the optimal period if α_c > -0.2. This is reasonably consistent with findings of Ibarra regarding the effect of α_c on collapse capacity (Ibarra 2003, Figure 4.12). The trends are the same for μ = 4 and 6, but the trends seem to be slightly different for μ = 2. Using the results presented in Table 3, we created a simple equation to predict the optimal period as a function of T_1, μ, and α_c.

Table 3. Summary of optimal ground motion IM periods ($T_{col,opt}$) for SDOFs considered in this study.

$T_{1,struct}$ (sec.)	μ	α_c (with P-Δ)	Optimal period range for all sets (sec.)		Optimal Period (sec.)
1.0	2	-0.12	1.2	2.7	1.7
1.0	2	-0.33	1.0	2.5	1.6
1.0	4	-0.07	1.7	2.8	2.3
1.0	4	-0.12	1.5	2.7	2.1
1.0	4	-0.22	1.2	2.6	1.7
1.0	4	-0.33	1.2	2.6	1.8
1.0	4	-0.44	1.2	2.5	1.7
1.0	4	-0.53	1.2	2.7	1.7
1.0	6	-0.12	1.5	2.9	2.4
1.0	6	-0.33	1.4	2.7	2.1
2.0	2	-0.15	2.2	4.5	3.4
2.0	2	-0.35	1.9	4.5	3.2
2.0	4	-0.15	3.7	> 5.0 **	--
2.0	6	-0.15	--*	> 5.0 **	--
2.0	4	-0.35	3.7	> 5.0 **	--
2.0	6	-0.35	--*	> 5.0 **	--

* $\sigma_{Ln(SaCollapse)}$ not < 0.3 for three or more ground motion sets
** the optimal period range extends beyond 5.0 seconds, which goes beyond the periods considered (0 < T < 5)

$$T_{col,opt} = 0.85 T_{1,struct} \sqrt{\mu} \left[1.56 + 2.8 \alpha_c \right] \geq 0.85 T_{1,struct} \sqrt{\mu} \tag{1}$$

Eq. 1 is based only on the data for μ = 4 and 6, and does not appear to be as effective for explaining the μ = 2 results. Consequently, this equation will under-predict the optimal period for μ = 2. Refining Eq. 1 to better capture the trends for lower ductility systems (e.g. μ = 2) is a topic of continued research. Eq. 1 uses the inequality to cause α_c to only increase the optimal period if α_c > -0.20 (based on observations from Table 3).

For first mode dominated structures, Eq. 1 can be used to approximate the optimal IM period with the following procedure: 1) complete a static pushover analysis of the structure, 2) estimate μ and α_c from the static pushover results, 3) estimate $T_{1,struct}$ using an eigenvalue analysis or other appropriate method, then 4) use these values in Eq. 1. Further work to verify this procedure and refine the prediction of $T_{col,opt}$ is in progress.

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[7] Optimal period range is defined as range for which $\sigma_{LN(Sacollapse)}$ < 0.30 for three or more record sets.

[8] Often the period with lowest $\sigma_{LN(Sacollapse)}$ is not central to the optimal period range (e.g. 1.5 sec. for Set A, see Fig. 7), so the reported optimal period is judgmentally adjusted to be more centered in the optimal period range. In decision of optimal period, more weight was given to Set A and the ε-selected sets near the optimal period.

Summary and Conclusions

This paper discussed ground motion selection for collapse simulation and looked specifically at questions regarding using an extended period in the ground motion intensity measure. Previous work (Baker 2005; Goulet et al. 2006) has shown that considering ε when selecting and scaling ground motions at $T_{1,struct}$ leads to significant changes in collapse capacity prediction. This paper follows that work by selecting ground motions considering ε at periods other than $T_{1,struct}$ (specifically periods near $2T_{1,struct}$, which this paper shows to be appropriate for collapse analyses of ductile structures) to see how this affects collapse capacity predictions.

Summary and conclusions of this paper are as follows:

a) The optimal period ($T_{col,opt}$) for use in the ground motion IM for collapse can be predicted using Eq. 1; and is based on the $T_{1,struct}$, μ, and α_c of the structure. For typical conforming structures ($\mu = 4$), $T_{col,opt} = 2T_{1,struct}$ is generally appropriate.

b) When comparing collapse capacity predictions for ground motion sets selected with and without considering ε ($\varepsilon = +2$), the median predicted collapse capacity (for an example SDOF with $T_{1,struct} = 1.0$ sec., $\mu = 4$, and $\alpha_c = -0.1$)[9] is:
   - 45% higher when using the ε-selected set at $T_{1,struct}$.
   - 25% higher when using the ε-selected set at $2T_{1,struct}$ (which is close to $T_{col,opt}$).

c) Conclusion (b) shows two things:
   - The collapse capacity prediction is less sensitive to ground motion selection (i.e. the ε values) when T_{IM} is near the optimal period, $T_{col,opt}$.
   - Even when $T_{col,opt}$ is used as T_{IM}, the ground motions with $\varepsilon = +2$ still increase the median collapse capacity by 25% (which is significant).

d) When comparing $\lambda_{collapse}$ predictions for ground motion sets selected with and without considering ε ($\varepsilon = +2$),
   - $\lambda_{collapse}$ is a factor of 3-6 lower when using the ε-selected set at $T_{1,struct}$.
   - $\lambda_{collapse}$ is a factor of 1-3 lower when using the ε-selected set at $T_{col,opt}$.

e) The results obtained using the epsilon-selected ground motion sets are believed to be more accurate (Baker 2005, Chapter 6), and so by minimizing the difference in estimated $\lambda_{collapse}$, one is minimizing the error induced by not selecting records appropriate for collapse analyses.

f) $\lambda_{collapse}$ predictions using the various ε-selected sets are reasonably consistent (this is an important finding that shows the stability of $\lambda_{collapse}$ predictions when using ground motions selected based on ε).

g) To follow (f), even though there is no observed trend between $\lambda_{collapse}$ and T_{IM} (for the ε-selected sets), the use of different ground motion sets causes $\lambda_{collapse}$ to vary by a factor of two. This suggests that even when ε-based record selection is used, the results should only be assumed accurate within a factor of +/- 2. Sensitivities of similar magnitude have also been documented in similar work (Haselton et al. 2005, chapter 6).

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[9] Note that this comparison depends on the ductility of the system (differences are more drastic for a more ductile system). For example if we instead had a system with $\mu = 2$ and $\alpha_c = -0.10$, these numbers would change to 40% and 30%, respectively.

Limitations of these conclusions are as follows:

    a) SDOF models were employed, so these results are limited to first-mode dominated structures for which the SDOF approximation is appropriate.

    b) The SDOF models covered a relatively small range of structural parameters. Therefore, conclusions are limited to the range of SDOF models considered. Incorporating a wider range of SDOF models is a subject of current research.

Acknowledgments

References

ASCE, 2002. *ASCE 7-02: Minimum Design Loads for Buildings and Other Structures*, Reston, VA.

Baker, J., 2005a. *Vector-Valued Ground Motion Intensity Measures for Probabilistic Seismic Demand Analysis*, Ph.D. Dissertation, Department of Civil and Envir. Engineering, Stanford University.

Baker J.W. and C.A. Cornell C.A., 2005b. A vector-valued ground motion intensity measure consisting of spectral accel. and epsilon, *Earthquake Engr. & Structural Dynamics*, **34** (10), 1193-1217.

Baker, J.W. and C.A. Cornell, 2003. *Uncertainty Specification and Propagation for Loss Estimation Using FOSM Methods*, PEER Report 2003/07, Pacific Engineering Research Center, University of California, Berkeley, California, 89 pp.

Boore, D.M., Joyner W.B., and Fumal T.E., 1997. Equations for estimating horizontal response spectra and peak acceleration from western north american earthquakes: A summary of recent work, *Seismological Research Letters*, **68** (1), 128-153.

Goulet, C., C.B. Haselton, J. Mitrani-Reiser, G.G. Deierlein, J.P. Stewart, and E. Taciroglu (2006), "Evaluation of the Seismic Performance of a Code-Conforming Reinforced-Concrete Frame Building - Part I: Ground Motion Selection and Structural Collapse Simulation", *8th National Conference on Earthquake Engineering*, San Francisco, California, April 18-22, 2006.

Haselton, C. B., C. Goulet, J. Mitrani-Reiser, J. Beck, G.G. Deierlein, K.A. Porter, J. Stewart, and E. Taciroglu , 2005. *An Assessment to Benchmark the Seismic Performance of a Code-Conforming Reinforced-Concrete Moment-Frame Building*, PEER Report 2005/xx, University of California, Berkeley, California, (in preparation).

Ibarra, L., Medina, R., and H. Krawinkler, 2005. "Hysteretic models that incorporate strength and stiffness deterioration," *Earthquake Engr. and Structural Dynamics*, Vol. 34, pp. 1489-1511..

Ibarra, L.F., 2003. *Global Collapse of Frame Structures Under Seismic Excitations*, Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University.

Kircher & Associates Consulting Engineers, 2005. *Strong-Motion Earthquake Record Set for Soil Sites not Near Source*, personal communication, August 6, 2005.

PEER, 2005. *Pacific Earthquake Engineering Research Center:* PEER *Strong Motion Database*, University of California, Berkeley, http://peer.berkeley.edu/smcat/ (September 15, 2005).

Shome, N., C.A. Cornell, P. Bazzuro, and J.F. Carballo, 1998. "Earthquakes, Records, and Nonlinear Responses," Earthquake Spectra, **14**(3), 469-500.

Vamvatsikos, D. and C.A. Cornell, 2002. "Incremental Dynamic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 31, Issue 3, pp. 491-514.