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HOW TO CHOOSE EARTHQUAKE RECORDINGS FOR NON-LINEAR SEISMIC ANALYSIS OF STRUCTURES

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SUMMARY

The research project presented here dealt with an important aspect of seismic structural response that is still not well understood. Its objective was to develop criteria to choose suitable earthquake recordings to be used in non-linear dynamic analyses for seismic design, evaluation and upgrade of ductile structures. The methodology adopted consists of systematic investigations of the non-linear response of single-degree-of-freedom (SDOF) -systems subjected to different types of earthquake recordings. The structural behavior is described by six different recognised hysteretic models. A database of 164 recorded time-histories was built based on the European Strong Motion Database. Different earthquake characteristics are investigated (effective peak ground acceleration, spectral acceleration, slope of response spectra, spectral intensity, etc.). The ductility demand of the SDOF systems is then correlated with the different earthquake characteristics. According to the conclusions of others, spectral intensity defined by Nau and Hall gives a good prediction of the seismic impact on structures. A modified spectral intensity is proposed here in order to improve its prediction. This modification takes into account the structural natural frequency and the design ductility. Corresponding recommendations are formulated for structural engineers that are believed to increase the reliability of non-linear seismic analysis. In addition, the impact of fling on structural behavior is addressed, too. 20 recordings from the Chi-Chi Taiwan earthquake containing fling were applied to the SDOF systems. The related ductility demand is compared with the one of the same recordings after extraction of the fling. The preliminary results, in general, show relatively small differences. Surprisingly, if differences occur, larger ductility demands result for the recordings without fling.

INTRODUCTION

The research project presented here [1] dealt with an important aspect of seismic structural response that is still not fully understood. Its objective was to develop criteria to choose suitable earthquake recordings to be used in non-linear dynamic analyses for seismic design, control and upgrade of ductile structures such as reinforced concrete (RC) structures.

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Standard simplified methods of seismic calculation generally contain an unknown margin of conservatism. This is usually a minor problem for new structures, the cost of an unnecessary conservative margin being most of the time insignificant. The situation, however, is different for existing structures: A conservative recalculation may lead to the result that the structure is unsafe, whereas a more realistic calculation would show that it is safe enough. Money may be spent unnecessarily for upgrading. Since one "unit of earthquake safety" is more expensive to be obtained for an existing building than for a new one, it is important to predict the seismic behavior of existing structures as realistically as possible. Non-linear dynamic time history analyses are usually thought to be the most realistic approach.

Up to now, if non-linear dynamic analyses were performed, they were usually done with artificial acceleration time histories. The reason is that until recently, there were not enough recordings of acceleration time histories available. Furthermore, most engineers believe that artificial time histories covering the whole frequency range of a design response spectrum represent a conservative input for the analysis. However, this is not necessarily true for non-linear systems. Since more and more recorded time histories have become available in recent times, it would now often be possible to work with recorded time histories. However, no aid is given to the engineer how to choose time histories that are appropriate to a given design or upgrade situation.

If several acceleration time histories have the same spectral acceleration at the natural frequency of a single-degree-of-freedom-system (SDOF system), the same displacement or ductility demand would result for this SDOF system from standard response spectra analyses. However, non-linear dynamic analyses would lead to different displacement or ductility demands. The key question is what characteristics of the acceleration time history influence the system's displacement or ductility demand. The structural engineer will only be in a position to choose the earthquake recordings for non-linear dynamic analyses in a rational manner once this question is answered.

The research project presented here aimed at answering that question. As a conclusion, recommendations for structural engineers are formulated that are believed to increase the reliability of non-linear seismic analysis.

A tentative extension of the research project was devoted to a recent open question in engineering seismology and earthquake engineering: the impact of "fling" on structural behavior. Fling is a strong velocity pulse that results in permanent ground displacement. It has been observed in recent earthquake recordings stemming from stations situated close to the earthquake source. The displacement and ductility demands were calculated for time histories containing fling before and after extracting the fling, and the results were compared with each other. In order to guide their future research, the seismologists would like to know from the engineers whether fling significantly influences structural behavior or not.

METHODOLOGY

Structures

In the current study, it was chosen to model ductile structures by means of non-linear SDOF systems and use their displacement and ductility demands as the decisive characteristics of the non-linear response. Each SDOF system is defined by its:

- "initial" natural frequency (f_0 = 0.25; 0.5; 0.75; 1.0; 1.5; 2.0; 3.0 and 4.0 Hz)
- yield displacement (d_y)
- hysteretic model (six recognized hysteretic models)

The initial natural frequency corresponds to the mean stiffness between the position at rest and the position at first yielding of the structure.

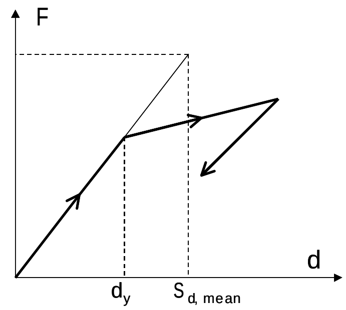


Figure 1: Definition of the structure.

For a given natural frequency, the yield displacement is chosen as being (Fig. 1):

$$d_y = \frac{S_{d,mean}}{R}$$

where $S_{d,mean}$ represents the mean value of the spectral displacements of the recorded time histories that were used. R is the strength reduction factor that takes the following values: 2, 3, 4 and 5. This means that for a given initial natural frequency and a given strength reduction factor, an identical SDOF system is subjected to the different acceleration time histories. Therefore, for the consideration of the correlation coefficients, displacement and ductility demand are interchangeable. They are related by the yield displacement d_y, being the same for all time histories. Therefore, the results and conclusions are valid for displacement as well as for ductility demand.

The six hysteretic models that were considered are the elastoplastic (bi-linear) model, the γ-model [2], the Takeda-model [3] and the Q-model [4]. Both Takeda and Q-model were used twice, with and without considering stiffness degradation. The displacement and ductility demands were computed for each of the six hysteretic models. As they all yielded very similar results, it was decided to only focus on the Takeda model with consideration of stiffness degradation for the interpretation of the results.

Earthquake recordings

Database

A database of 164 recorded time histories was extracted from the European Strong Motion Database [5]. Only time histories with a peak horizontal acceleration (PHA) of at least 0.6 m/s^2 were used, 0.6 m/s^2 being the peak ground acceleration of zone 1 in the Swiss building code. Furthermore, only recordings for magnitude values of at least M = 5.0 were maintained in the data base (no distinction was made for different definitions of magnitude), since lower magnitude events rarely cause structural damage. No condition was imposed on epicentral distance while selecting the time histories. Figure 2 shows the distribution of the magnitude with respect to the epicentral distance.

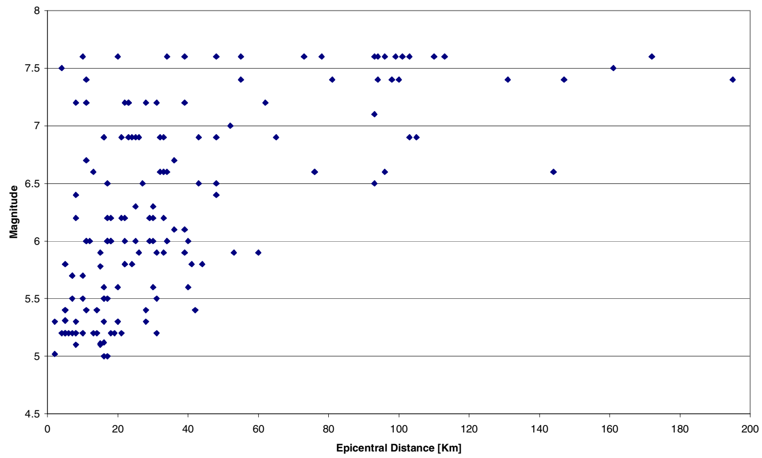
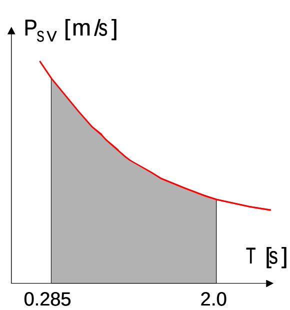


Figure 2: Magnitude versus epicentral distance plot for the 164 events.

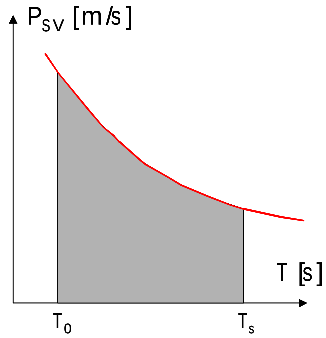
Earthquake characteristics
A literature review led to the following selection of earthquake characteristics:

- The **peak horizontal acceleration**, also called peak ground acceleration (PGA) [6], which is known to correlate badly with structural response, was chosen here for reference.
- PGA is sometimes a bad indicator as an acceleration time history might have a sudden peak in an otherwise low energy ground motion. Hence, a further parameter called **effective peak ground acceleration** (EPGA) is used according to Musson [7]. EPGA is defined as the mean of the spectral acceleration between 0.1 s and 0.5 s at an interval of 0.02 s, divided by a standard spectral amplification of 2.5.
- The **magnitude** is also examined. The values used are those given in the European Strong Motion Database [5]. They may correspond either to the moment magnitude M_W, the surface wave magnitude M_S or the local M_L magnitude.
- The **Arias intensity duration** is the interval during which a certain proportion of the total Arias intensity gets accumulated. The definition used by Elenas [6] is chosen here.
- The **uniform duration**, originally proposed by Bolt using narrow band filtered accelerograms, is the sum of the intervals during which the absolute acceleration level exceeds a particular threshold [8], here 0.5 m/s^2.
- The **spectral acceleration (S_a)** is the best-known parameter to structural engineers. It is the peak elastic acceleration of a SDOF system for the entire range of frequencies or periods for a given damping ratio, here being 5%.
- The **spectral intensity** is based on the pseudo-velocity response spectrum. The definition given by Nau and Hall [9] is called **SI a** in the current study:



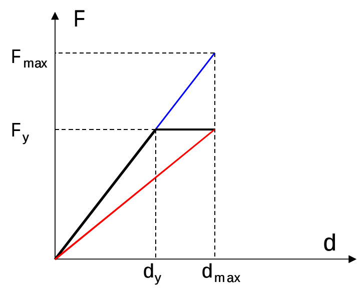
$$SI\ a(\zeta) = \frac{1}{1.715}\int_{0.285}^{2.0} P_{SV}(\zeta)\cdot dT\ , \qquad \zeta = 5\%$$

- A new definition of **spectral intensity**, called **SI b** in the current study, is introduced. The interval used for the computation is closely related to the expected structural seismic response.



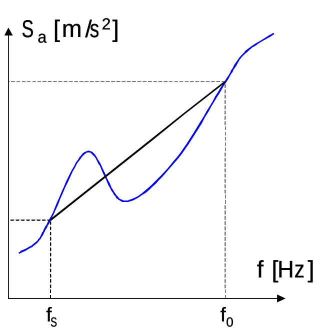
$$SI\,b\left(\zeta, f_0, R\right) = \frac{1}{\Delta T}\int_{T_0}^{T_s} P_{SV}\left(\zeta\right)\cdot dT \,, \quad \zeta = 5\% \,,$$

$$T_s = T_0 \cdot \sqrt{R} = \frac{\sqrt{R}}{f_0}$$



The period T_s corresponds to the secant stiffness that can easily be linked to the initial fundamental period T_0 if the equal displacement rule is assumed. The slope of the steep line in the figure beside corresponds to the initial fundamental period T_0. The slope of the less inclined line corresponds to the secant period T_s.

- Since the initial fundamental vibration characteristics of RC elements change due to the occurrence of damage, it makes sense to analyse the **slope m** of the acceleration response spectrum (S_a) between the initial (f_0) and the degraded (f_S) natural frequency.



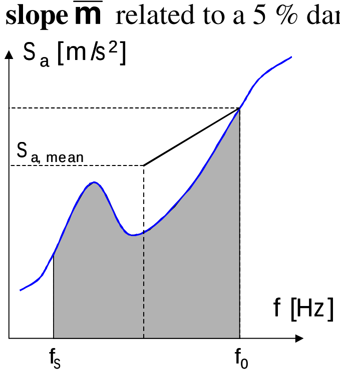
$$m\left(\zeta, f_0, R\right) = \frac{S_a\left(f_0,\zeta\right) - S_a\left(f_s,\zeta\right)}{f_0 - f_s} \,,$$

with

$$\zeta = 10\% \text{ - } 20\% \text{ - } 30\% \,,$$

$$f_s = \frac{f_0}{\sqrt{R}}$$

- The major drawback of the slope characteristic is that it has to be performed in a highly damped response spectrum. It was therefore decided to smooth the jagged spectrum by using the **average slope $\overline{m}$** related to a 5 % damped response spectrum as usual.



$$\overline{m}\left(\zeta, f_0, R\right) = \frac{S_a\left(f_0,\zeta\right) - S_{a,mean}\left(f_0,\zeta\right)}{\frac{f_0 - f_s}{2}} \,, \quad \zeta = 5\% \,,$$

$$S_{a,mean}\left(f_0,\zeta\right) = \frac{\int_{f_s}^{f_0} s_a\left(\zeta\right)\cdot df}{f_0 - f_s}$$

Computation of correlation coefficients

For the various SDOF models, defined by f_0 and R, the correlation coefficients between the displacement or ductility demand and the examined earthquake characteristics were computed. Correlation coefficients superior to 0.6 were considered as significant. An example of a poor and a good correlation is given in Figures 4 and 6, respectively.

RESULTING CORRELATION COEFFICIENTS

The direct use of the original acceleration time histories for the computation of the correlation coefficients would lead to some difficulties in interpreting the results because of the large scatter of the recorded spectral accelerations. The original acceleration time histories were therefore scaled in such a way that they all had the same spectral acceleration at the SDOF system's initial natural frequency, namely the mean value $S_{d,mean}$. As a direct consequence, the peak linear displacement demands became the same for all time histories.

The scaling described above corresponds to the question brought up in the introduction of the present paper: what characteristics of the acceleration time histories influence the SDOF system's displacement or ductility demand, given the same spectral acceleration at the system's natural frequency. Furthermore, the scaling made sure that all ground motions pushed the SDOF system into the non-linear range.

The spectral intensities **SI a** and **SI b** gave the highest correlation coefficients. The discussion that follows focuses therefore on spectral intensities and slope characteristics – as well as on magnitude for comparison. The correlation coefficients obtained for the other characteristics are significantly lower than those found for spectral intensities and slope characteristics [10].

Magnitude

Figure 3 shows the correlation between ductility demand and magnitude for R = 3 as a function of the initial natural frequency. The results are poor since the correlation coefficients remain below 0.3. The fact that the coefficients are positive is logical though. Figure 4 highlights the scatter of the ductility demand as a function of magnitude for the 164 time histories.

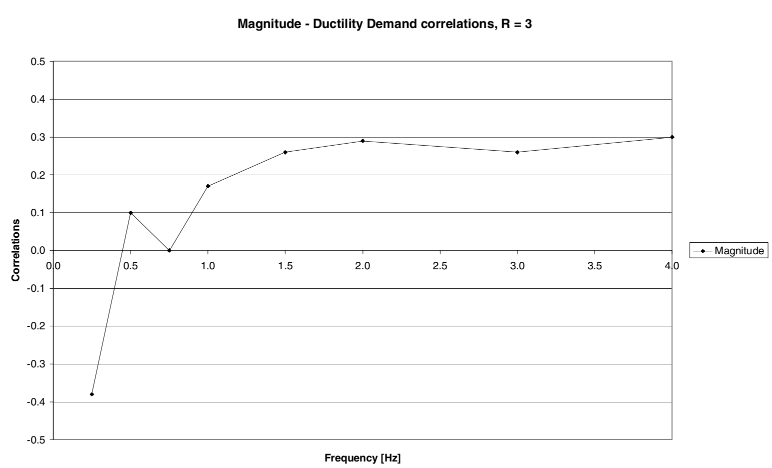


Figure 3: Correlation coefficients between ductility demand and magnitude.

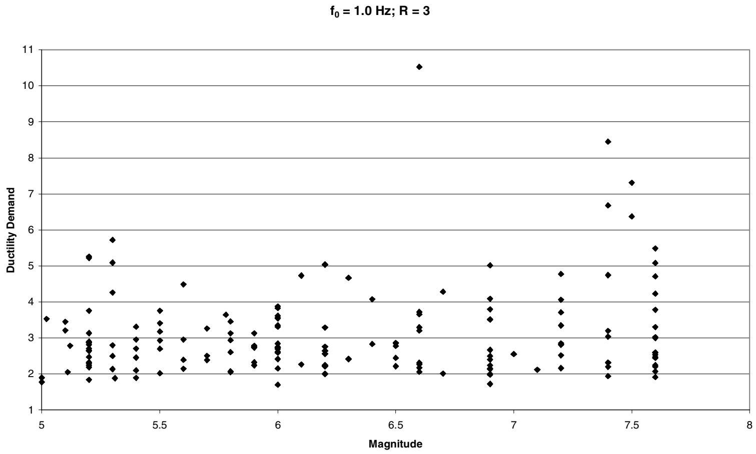


Figure 4: Scatter of the ductility demand as a function of magnitude for f_0 = 1.0 Hz and R = 3. Correlation coefficient r = 0.17.

Spectral intensities

Figure 5 shows the correlation between ductility demand and spectral intensities for R = 3 as a function of the initial natural frequency. With correlation coefficients constantly between 0.6 and 0.9, the newly defined spectral intensity SI b can be considered as being very satisfactory. Nau & Hall's [9] definition, SI a, yields higher values for frequencies above 1.5 Hz, but shows poor performance below 1.0 Hz. Figure 6 is the counterpart of Figure 4 for a much better characteristic, namely SI b.

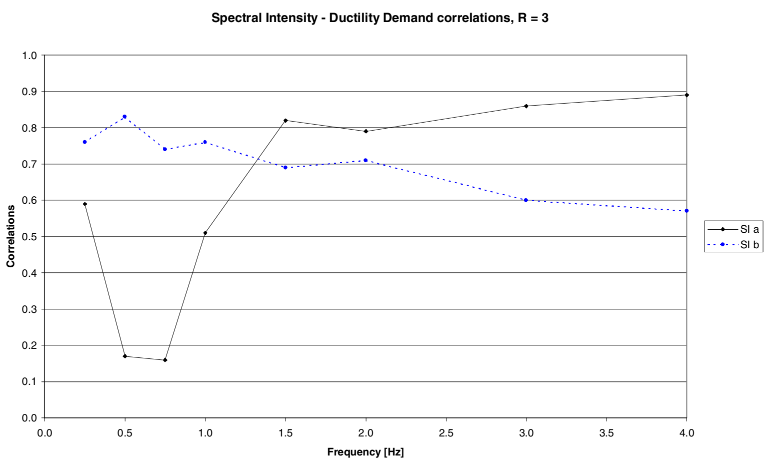


Figure 5: Correlation coefficients between ductility demand and spectral intensities according to Nau & Hall [9] (**SI a**, straight line) and according to the new definition (**SI b**, dotted line), respectively.

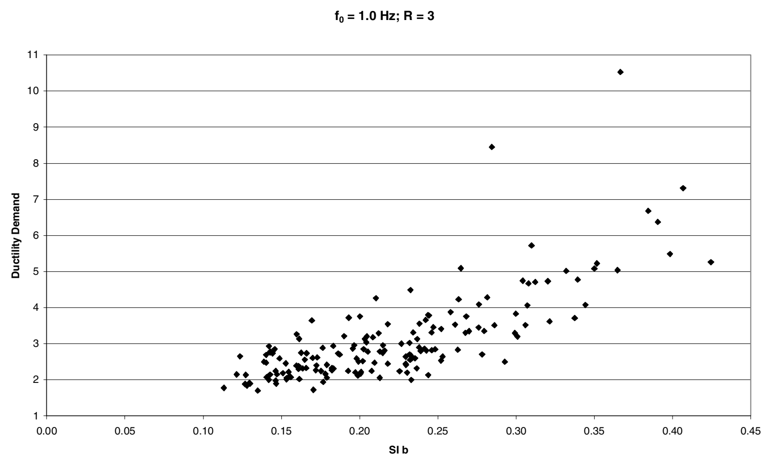


Figure 6: Scatter of the ductility demand as a function of spectral intensity SI b for f_0 = 1.0 Hz and R =3. Correlation coefficient r = 0.76.

Slope

Figure 7 shows the correlation between ductility demand and slope for R = 3 as a function of the initial natural frequency. Here, the correlation coefficients are negative, which is plausible. A negative correlation means that the flatter the response spectrum below the initial natural frequency, the higher the expected displacement or ductility demand.

The computations for three different damping ratios show that the best correlation is obtained for a reasonable damping ratio that does neither totally flatten the spectrum nor leaves too many isolated "aleatory" peaks.

The positive correlation that appears for 0.25 Hz is an artefact. The frequency content of most recorded ground motions is unreliable below 0.2 Hz or 0.3 Hz owing to standard high pass filtering that is often applied for base line corrections.

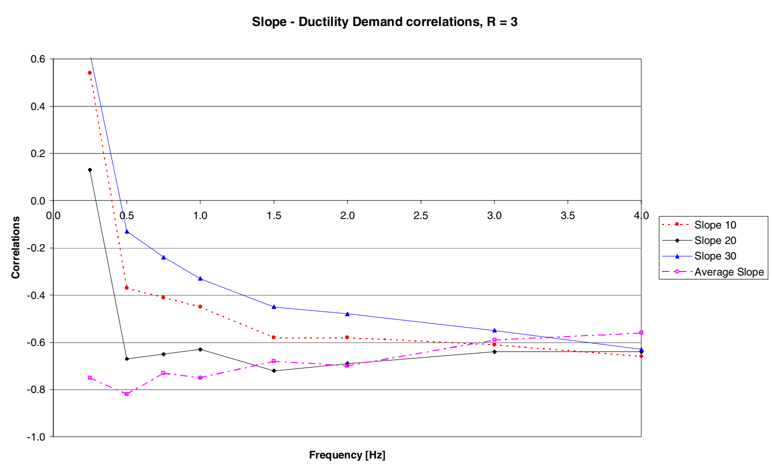


Figure 7: Correlation coefficients between ductility demand and slope for 10 % (dotted line), 20 % (circles) and 30 % (triangles) damping. The results for the average slope (dashed line) are plotted as well.

Average slope
The correlation between ductility demand and average slope for R = 3 as a function of the initial natural frequency is already shown in Figure 7. It turns out that the average slope gives similar results as the slope for frequencies above 1.0 Hz. However, for lower frequencies, average slope performs much better.

Figure 8 focuses on the differences between the correlation for average slope and the newly defined spectral intensity **SI b**. It was of course necessary to take the absolute values of the average slope correlation coefficients. The results are nearly identical. Nevertheless, the spectral intensity seems to give a slightly better correlation.

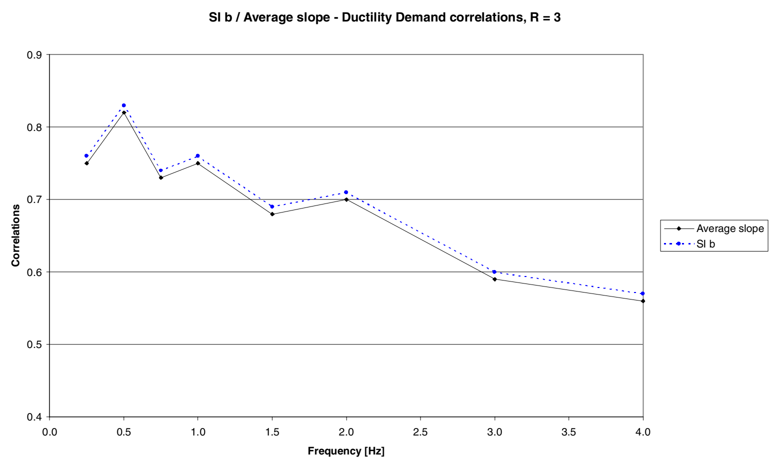


Figure 8: Correlation coefficients between the average slope (continuous line) and the newly defined spectral intensity **SI b** (dotted line).

Conclusions
The main findings obtained so far may be summarised as follows:

- Based on the results presented before, displacement or ductility demand shows the best correlation with the newly defined **spectral intensity SI b**. This result remains also valid if acceleration time histories are used that have different spectral accelerations at the SDOF system's initial natural frequency.
- The correlation with the **average slope** $\overline{m}$ characteristic is practically as good as with SI b. However, average slope is only a valid characteristic if time histories with identical spectral acceleration at the SDOF system's initial natural frequency are compared.
- For unscaled time histories featuring quite different spectral accelerations, the classical **spectral acceleration** $\mathbf{S_a}$ still gives a rational indication of the "severeness" of the earthquake ground motion.

FLING

A tentative extension of the research project was devoted to a recent open question in engineering seismology and earthquake engineering: the impact of "fling" on structural behaviour. Fling is a strong velocity pulse that results from the permanent tectonic ground displacement that corresponds to the slip on the causative fault. Fling was recently observed in the near fault ground motions of the 1999 Kocaeli (Turkey) and Chi-Chi (Taiwan) earthquakes.

Figure 9 shows the TCO 052 E recording of the Chi-Chi earthquake in its original version, i.e. with fling, and after extraction of the fling, i.e. without fling. A strong velocity pulse corresponding to a step in displacement can clearly be observed in the original recording. In acceleration, the fling corresponds to one period of more or less a sinus function, but little can be seen of this in the acceleration time history. Note that the algorithm used for the computation of the SDOF system's ductility demand did not converge for this time history; this was probably due to the strong fling present in the TCO 052 E recording.

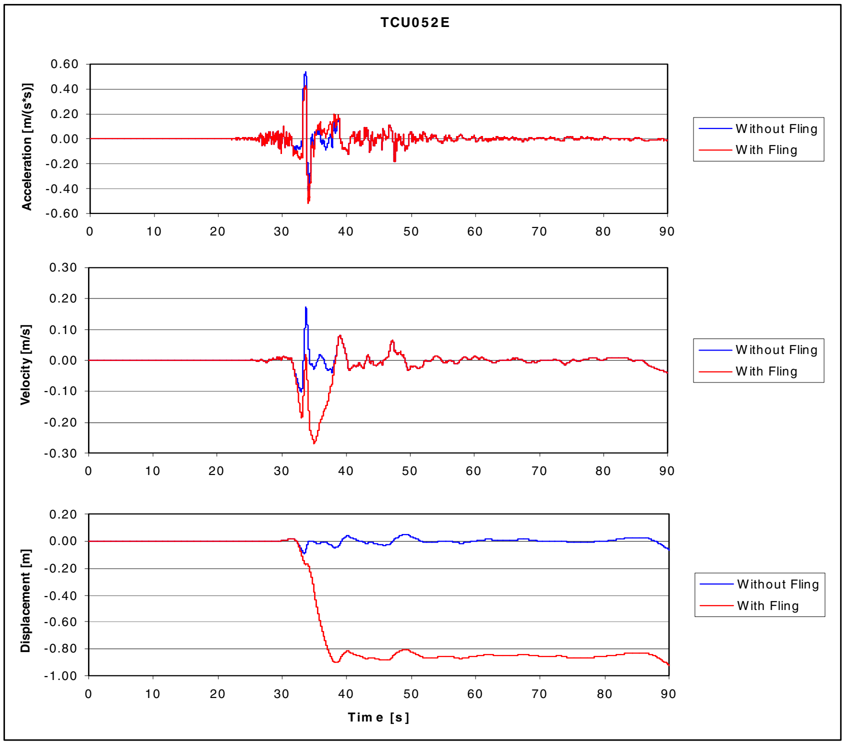


Figure 9: TCO 052 E recording of the Chi-Chi (1999) earthquake with fling and after extraction of the fling: a strong velocity pulse as well as a kind of step function for the displacement can be seen.

Up to now, research on fling seems to be limited to the seismological community; the impact of fling on structural behaviour has not yet been studied. In order to guide their future research, the seismologists would like to know from the engineers whether fling significantly influences structural behaviour or not.

Methodology

As a very preliminary study, 18 recordings (acceleration time histories) of the Chi-Chi earthquake containing fling were applied to the SDOF systems. Then, the same recordings, after "manual" extraction of the fling, were applied again to the same SDOF systems. The displacement or ductility demands were calculated for the time histories with and without fling, and the results were compared with each other. The set of 18 recordings was not homogenous in the sense that the amplitude of the fling strongly varied from one case to the other.

The seismic structural responses were computed with the Takeda model and for four strength reduction factors: R = 2, 3, 4 and 5. Since fling has a relatively low frequency content, only initial natural frequencies ranging from f_0 = 0.25 Hz to f_0 = 2 Hz were considered. In order to focus on fling only, the yield displacement of the SDOF systems was kept constant for each recording with and without fling and was equal to the mean value of the related spectral displacements, divided by the strength reduction factor R.

Results and conclusions

As an example, Figure 10 presents the computed ductility demand, (μ_Δ, without fling "*"compared to the one with fling "°") for different initial frequencies (f_0 = 0.25 to 1 Hz) and strength reduction factors (R = 2 to 4) for the recording TCU 051, the east component above and the north component below. In many cases such as figure 10 below, rather small differences in the displacement or ductility demand resulted from the time histories with and without fling. In fact, many of the earthquake recordings that were used only contained a relatively weak fling.

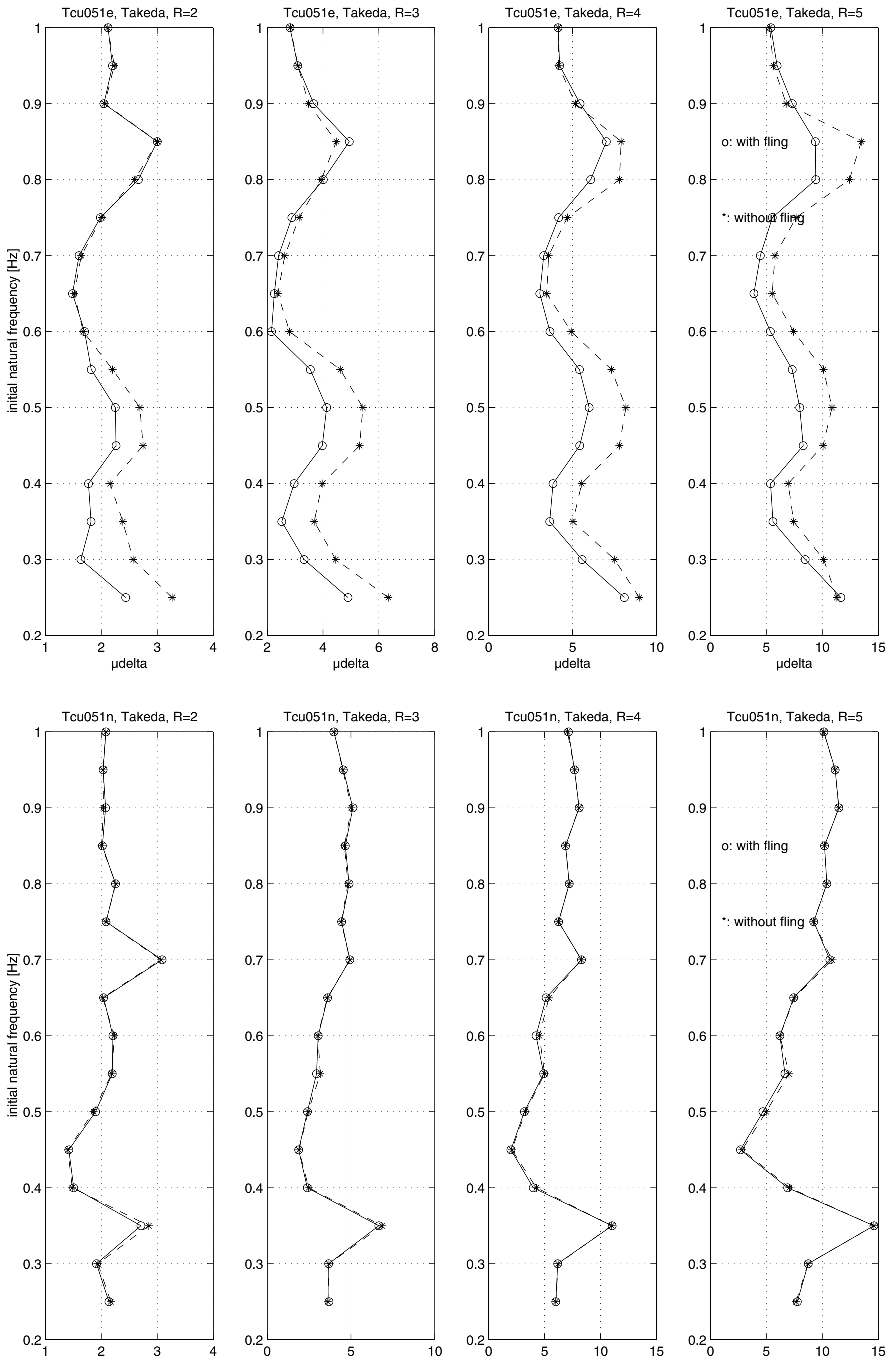


Figure 10: Ductility demand without fling "*" and with fling "°" (μ_Δ), for different initial frequencies (f_0) and strength reduction factors (R) for TCU 051 (east component above and north component below).

Where significant differences occurred, they were limited to the lowest frequency domain (< 1 Hz), as can be expected from the low frequency character of flings. The differences tended to increase with the level of the strength reduction factor R (see figure 10 above). Surprisingly, where differences occurred, larger displacement or ductility demands resulted for the recordings *without* fling, except for one single case. The reasons for this peculiar result are not understood. The calculations were carefully checked in order to make sure that there was no confusion between the cases with and without fling.

Figure 11 presents the differences in ductility demand, $\Delta(\mu_\Delta)$ (ductility demand without fling minus ductility demand with fling), as a function of the fling amplitude in acceleration for the case of R = 3. The $\Delta(\mu_\Delta)$ given in Figure 11 corresponds to the maximum difference, irrespective of frequency, that could be found. A relatively poor correlation can be observed, with nevertheless a rough tendency of larger $\Delta(\mu_\Delta)$ for stronger flings.

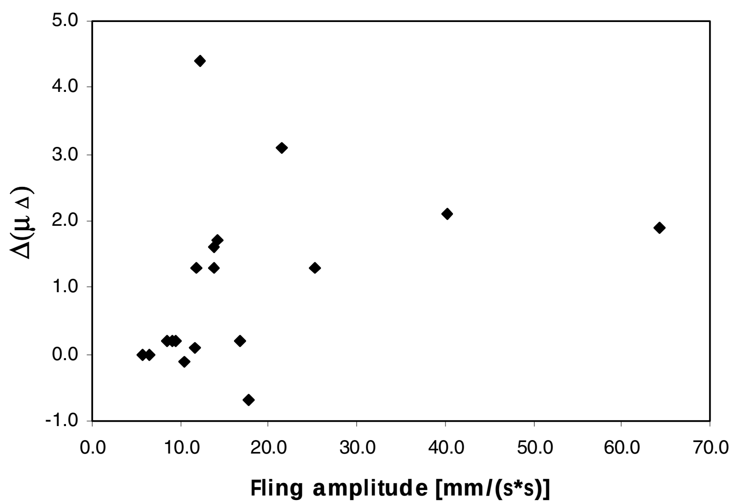


Figure 11: Ductility demand without fling minus ductility demand with fling, $\Delta(\mu_\Delta)$, as a function of the fling amplitude in acceleration for R = 3

The limited number of recordings used in this investigation does not yet allow to draw general conclusions, particularly in view of the counterintuitive results. Much more research work is necessary to understand the impact of fling on ductile structures.

RECOMMENDATIONS AND CONCLUSIONS

The following recommendations are based on the present study of ductile SDOF systems. Therefore, they can be expected to be valid for structures that can reasonably be modelled as SDOF systems. The behaviour of more or less regular ductile structures is usually well approximated by SDOF models. More research work, however, is necessary in order to further develop and test these recommendations for strongly irregular structures that behave more like multi-degree of freedom systems.

The following recommendations can be given for the choice of acceleration time histories to be used for non-linear seismic analyses:

- The spectral acceleration S_a of the acceleration time history should be equal or close to the spectral acceleration of the given design spectrum at the initial fundamental period T_0 of the structure under study and, as far as possible, within the range between T_0 and the period T_s, T_s

being the fundamental period that corresponds to the secant stiffness for either the expected ductility demand or the design ductility.

- The "severeness" of several time history candidates that fulfil the above conditions can be ranked with the aid of the newly defined spectral intensity SI b: the larger SI b, the higher the displacement and ductility demand. It is up to the structural engineer to decide whether to use the most "severe" time histories for worst case studies or to use time histories that have neither particularly high nor particularly low values of SI b.

For very important structures, as well as for checking simplified methods of seismic evaluation, it is expected that more and more non-linear seismic analyses will be performed in the future. The results of the present research project allow to choose the acceleration time histories needed for such calculations in a more rational manner than before. As a by-product, the "severeness" of earthquake recordings with respect to a given ductile structure can be evaluated with the aid of the newly defined spectral intensity SI b.

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REFERENCES

1. Schwab P., Lestuzzi P., Koller M, Lacave C.: *Choice of Recorded Acceleration Time Histories for Non-linear Seismic Analysis of Reinforced Concrete Structures*. Research Project Report. EPFL-ENAC-IS-IMAC, Applied Computing and Mechanics Laboratory. Publication Nr. 2. Lausanne, August, 2003.
2. Lestuzzi P., Badoux M. (2003). *The γ-Model : a simple hysteretic model for reinforced concrete walls.* Proceedings of the fib-Symposium; Concrete Structures in Seismic Regions, Athens 2003.
3. Takeda T., Sozen M.A., Nielsen N.N. (1970). Reinforced concrete response to simulated earthquakes. Journal of the Structural Division. Proceedings of the American Society of Civil Engineers (ASCE). Vol. 96, No. ST12. December 1970.
4. Saiidi M., Sozen M.A. (19981). *Simple nonlinear seismic analysis of R/C structures*. Journal of the Structural Division. Proceedings of the American Society of Civil Engineers (ASCE). Vol. 107, No. ST5. May 1981.
5. Smit P. et al. (2002). *European Strong Motion Database*. European Council, Environment and Climate Research Program, 2000.
6. Elenas A. (2002). *Seismic Damage Potential Described by Spectral Intensity Parameters*. 12th European Conference on Earthquake Engineering. Paper 267. London 2002.
7. Musson R.M.W. (2002): *Effective Peak Acceleration as a Parameter for Seismic Hazard Studies.* 12th European Conference on Earthquake Engineering. Paper 32. London 2002.
8. Bommer J. J., Martinez-Pereire A. (1999). *The Effective Duration of Earthquake Strong Motion*. The Journal of Earthquake Engineering 1999; Vol. 3 No. 2
9. Nau M., Hall W.J. (1984). *Scaling Methods for Earthquake Response Spectra.* Journal of Structural Engineering 110 (7) pp 1533-1548, 1984.
10. Sharma S.: Non-linear Dynamic Response of Structures Subjected to Recorded Earthquakes. Internship project IIT-EPFL, December 2002.