SEISMIC DESIGN OF STEEL STRUCTURES
BY SPECTRUM ANALYSIS AND
EQUIVALENT DAMPING

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Seismic inelastic structural response can be determined by nonlinear (geometrically and materially) dynamic analysis in the time domain. However, this approach is not frequently used in design due to its complexity.

Seismic codes determine the maximum inelastic seismic response through a linear elastic analysis involving modal superposition, use of design spectrum and the behavior factor (strength reduction factor).

In this work, the maximum inelastic seismic structural response is obtained through a linear elastic analysis involving modal superposition and a design spectrum but without using the crude reduction factor. Instead, use is made of rationally defined and computed equivalent modal damping ratios.

The validity of the proposed method is tested against the usual method of seismic codes and nonlinear dynamic analyses.
EQUIVALENT MODAL DAMPING RATIOS

• According to the equivalent damping concept, a non-linear structure can be substituted by an equivalent linear elastic structure with the same mass and initial stiffness and appropriately defined modal damping ratios so as to create a work of dissipation equivalent to that due to all nonlinearities (material and geometric).

• The modal damping ratios are obtained through an iterative formation of a frequency response transfer function until this function satisfies certain criteria that ensure its smoothness, and are given as a function of deformation-damage.
TRANSFER FUNCTION FOR A LINEAR PLANE STEEL FRAMED STRUCTURE

\[
R_r(\omega) = \frac{\ddot{y}_r(\omega)}{\ddot{u}_g(\omega)} = 1 + \sum_{k=1}^{N} \left[ \frac{\omega^2 \cdot \phi_{rk} \cdot \Gamma_{rk}}{(\omega_k^2 - \omega^2) + i(2 \cdot \xi_k \cdot \omega_k \cdot \omega)} \right]
\]

\[
|R_r(\omega)|^2 = 1 + 2 \cdot \sum_{k=1}^{N} \frac{\phi_{rk} \cdot \Gamma_{rk} \cdot \omega^2 \cdot (\omega_k^2 - \omega^2)}{(\omega_k^2 - \omega^2)^2 + (2 \cdot \xi_k \cdot \omega_k \cdot \omega)^2} + \sum_{k=1}^{N} \left[ \frac{\phi_{rk}^2 \cdot \Gamma_{rk}^2 \cdot \omega^4 \cdot (\omega_k^2 - \omega^2)^2 + 4 \cdot \xi_k^2 \cdot \omega_k^2 \cdot \omega^2}{(\omega_k^2 - \omega^2)^2 + (2 \cdot \xi_k \cdot \omega_k \cdot \omega)^2} \right] +
+ 2 \cdot \sum_{k \neq j \neq k}^{N} \frac{\phi_{rk} \cdot \Gamma_{rk} \cdot \phi_{rj} \cdot \Gamma_{rj} \cdot \omega^4 \cdot \left[ (\omega_k^2 - \omega^2) \cdot (\omega_j^2 - \omega^2) + 4 \cdot \xi_k \cdot \xi_j \cdot \omega_k \cdot \omega_j \cdot \omega^2 \right]}{\left[ (\omega_k^2 - \omega^2)^2 + (2 \cdot \xi_k \cdot \omega_k \cdot \omega)^2 \right] \cdot \left[ (\omega_j^2 - \omega^2)^2 + (2 \cdot \xi_j \cdot \omega_j \cdot \omega)^2 \right]}
\]

The above set of non-linear eqs. with known \[ |R_r(\omega)|^2 , \phi_{rk} , \Gamma_{rk} , \omega_k \]
can be solved for the damping ratios \[ \xi_k \].
TRANSFER FUNCTION FOR A LINEAR STRUCTURE
TRANSFER FUNCTION FOR A NON-LINEAR STRUCTURE

![Graph showing frequency response with modulus and frequency (Hz) axes.]

- Modulus
- Frequency (Hz)
WORK BALANCE AND SMOOTHNESS CRITERIA

maximum values

point of change of monotonicity
violation of interstorey drift limit
acceleration (m/sec$^2$)
time (sec)
earthquake
roof response
NUMERICAL DATA

- 20 Steel MR frames (EC3, Greek Seismic Code)

- 36 Seismic motions (near fault, long duration)
HIGHLY DAMPED SPECTRA

Mean plus one deviation acceleration spectra for near fault seismic motions having magnitude of seismic moment $\leq 6.8$. 
### DESIGN EQUATIONS FOR EQUIVALENT DAMPING AS A FUNCTION OF DEFORMATION - DAMAGE

<table>
<thead>
<tr>
<th>Seismic motions</th>
<th>Mode</th>
<th>IDR = 0.6%</th>
<th>IDR = 1.5%</th>
<th>IDR = 2.0%</th>
<th>IDR = 2.5%</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\zeta = 0.015$ for $0.5 \leq T \leq 2.5$ sec</td>
<td>$\zeta = 0.08$ for $0.5 \leq T \leq 2.5$ sec</td>
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<td></td>
<td>1</td>
<td>$\zeta = 0.006$ for $0.15 \leq T \leq 0.35$ sec</td>
<td>$\zeta = 0.045$ for $0.15 \leq T \leq 0.35$ sec</td>
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<tr>
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<td>2</td>
<td>$\zeta = 0.006$ for $0.11 \leq T \leq 0.48$ sec</td>
<td>$\zeta = 0.275 , (T - 0.3) + 0.025$ for $0.3 \leq T \leq 0.5$ sec</td>
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<tr>
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<td>$\zeta = 0.01$ for $0.5 \leq T \leq 2.5$ sec</td>
<td>$\zeta = 0.08$ for $0.5 \leq T \leq 2.5$ sec</td>
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<tr>
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<td>$\zeta = 0.004$ for $0.15 \leq T \leq 0.35$ sec</td>
<td>$\zeta = 0.06$ for $0.15 \leq T \leq 0.35$ sec</td>
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<tr>
<td></td>
<td>1</td>
<td>$\zeta = 0.004$ for $0.11 \leq T \leq 0.48$ sec</td>
<td>$\zeta = 0.05$ for $0.3 \leq T \leq 0.35$ sec &amp; $\zeta = 0.615 , (T - 0.3) + 0.05$ for $0.35 \leq T \leq 0.48$ sec</td>
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<tr>
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<td>3</td>
<td>$\zeta = 0.006$ for $0.11 \leq T \leq 0.32$ sec</td>
<td>$\zeta = 0.035$ for $0.11 \leq T \leq 0.48$ sec</td>
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<td>-</td>
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<tr>
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<td>4</td>
<td>$\zeta = 0.007$ for $0.10 \leq T \leq 0.24$ sec</td>
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<tr>
<td></td>
<td>5</td>
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<td>-</td>
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<tr>
<td>Long duration</td>
<td>1</td>
<td>$\zeta = 0.015$ for $0.5 \leq T \leq 2.5$ sec</td>
<td>$\zeta = 0.025 , (T - 0.5) + 0.10$ for $0.5 \leq T \leq 2.5$ sec</td>
<td>$\zeta = 0.47$ for $0.5 \leq T \leq 2.5$ sec</td>
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<tr>
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<td>$\zeta = 0.005$ for $0.15 \leq T \leq 0.35$ sec</td>
<td>$\zeta = 0.055$ for $0.15 \leq T \leq 0.35$ sec</td>
<td>$\zeta = 0.11$ for $0.15 \leq T \leq 0.35$ sec</td>
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<tr>
<td></td>
<td>3</td>
<td>$\zeta = 0.004$ for $0.11 \leq T \leq 0.48$ sec</td>
<td>$\zeta = 0.035$ for $0.11 \leq T \leq 0.48$ sec</td>
<td>$\zeta = 0.10$ for $0.32 \leq T \leq 0.47$ sec</td>
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<tr>
<td></td>
<td>4</td>
<td>$\zeta = 0.004$ for $0.11 \leq T \leq 0.32$ sec</td>
<td>$\zeta = 0.035$ for $0.11 \leq T \leq 0.27$ sec &amp; $\zeta = 0.8 , (T - 0.27) + 0.035$ for $0.27 \leq T \leq 0.32$ sec</td>
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<td>-</td>
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<tr>
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<td>5</td>
<td>$\zeta = 0.004$ for $0.10 \leq T \leq 0.24$ sec</td>
<td>$\zeta = 0.929 , (T - 0.17) + 0.035$ for $0.17 \leq T \leq 0.24$ sec</td>
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</table>
A steel moment resisting frame having 12 stories and 4 bays is considered. Each bay of the steel frame has 4.0m span and each storey 3.0m height. The dead plus live load on beams is equal to 27.5KN/m and the expected ground motion is defined by using the mean plus one standard deviation spectrum for near fault ground motions having magnitude of seismic moment ≤ 6.8.

The frame is designed according to EC3 structural code and for the case of 1.5% IDR and damage \( \theta_p = \theta_y \). HEB profiles are used for columns and IPE for beams. The modal damping ratios needed are found from the design equations given previously.
• The equivalent modal damping ratios read as:

\[ \xi_1 = 0.08, \xi_2 = 0.045, \xi_3 = 0.052, \xi_4 = 1, \xi_5 = 1. \]

One finally has the following sections: 400/450/450/450/400-400 (1-5) & 360/400/400/400/360-360 (6-9) & 340/360/360/360/340-330 (10-12).

• Non – linear dynamic analyses are executed using the accelerograms used for the construction of the highly damped spectra. The results regarding median values for IDR and plastic hinge rotation have as follows:

\[ IDR_{med} = 1.52\%, \quad IDR_{max} = 1.58\% \]

\[ \theta_{p,med} = 1.05\theta_y, \quad \theta_{p,max} = 1.12\theta_y \]
The moment resisting frame of the previous example is designed with the aid of SAP 2000 according to EC3 for IDR = 1.5% considering the elastic spectrum of EC8 for max PGA 0.24g and soil class B.

Equivalent modal damping read as:

\[ \xi_1 = 0.133, \xi_2 = 0.055, \xi_3 = 0.035, \xi_4 = 0.035, \xi_5 = 0.05 \]


Non-linear dynamic analyses are executed using eight accelerograms compatible to the aforementioned design spectrum:

\[ \text{IDR}_{\text{med}} = 1.45\%, \text{IDR}_{\text{max}} = 1.49\% \]
• The same frame is designed according to the EC8 using the aforementioned design spectrum and q=6. One finds the following sections: 240/260/260/260/240-300 (1-5) & 240/260/260/260/240-270 (6-9) & 240/240/240/240/240-270 (10-12).


• Non–linear dynamic analyses are executed using eight accelerograms compatible to the aforementioned design spectrum. The IDR values found are: IDR_{med} = 1.74%, IDR_{max} = 1.82%.
CONCLUSIONS

• An equivalent linear MDOF structure is constructed by retaining the mass and initial stiffness of the original non-linear MDOF structure and expressing material and geometrical non-linearities in the form of time-invariant modal damping values. These values can be viewed as playing the role of the strength reduction factor in code-based seismic design.

• Expressions providing deformation and damage dependent equivalent damping ratios for the first few modes as well as design acceleration spectra for high damping values are constructed. Thus, the design base shear can be easily determined by spectrum analysis and modal synthesis.

• Application of the proposed method to plane steel moment resisting frames leads to more accurate and rationally obtained results than those coming out of the usual code-based method.