DISPLACEMENT BASED LOSS ASSESSMENT

METHOD PROPOSED BY CALVI

•A displacement approach for the evaluation of the vulnerability of classes of buildings is presented.

• The method is derived from concepts developed for the detailed analysis of existing buildings, based on the estimation of their displacement and energy dissipation capacity.

• The results are presented in terms of probability of occurrence of each specific damage limit state for a given earthquake motion, represented through an appropriate displacement response spectrum for each building.

This approach requires the following steps:

- An examination of all possible local mechanisms of damage and collapse, with a calculation of the relative strength of all critical elements, to identify a probable sequence of events in case of seismic input.

- The consequent selection of a probable nonlinear response mechanism, evaluating its equivalent stiffness, energy dissipation and deformation capacity.

- The definition of a single or multi-degrees-of freedom model equivalent to the expected real response of the structure.

- A comparison of the displacement capacity of the model with the displacement demand, resulting from a displacement spectrum, appropriately scaled to take into account the effective energy dissipation capacity of the structure.

- As a first classification, ‘well designed’ buildings may be separated from ‘poorly designed’ buildings, assuming for the first a ductile response with a uniform distribution of energy dissipation capacity (i.e. ‘strong column weak beam’ design approach), and for the second a limited ductility capacity, with a high potential for damage concentration (i.e. a soft story type of response) or for brittle modes (e.g. shear failure modes).

- The expected response will be defined by two limit values defining the probable main period of vibration, two limit values defining the probable displacement capacity, and by a displacement correction factor depending on the expected energy dissipation, for each limit state of interest.

- A series of rectangles will result in the displacement spectrum plane, and each of them may be intersected by the spectral curve defining the input motion.

The procedure can be summarized in the following steps:

- Define the elastic displacement response spectra for the area of interest.
- Define a set of limit states to discretise the expected building response.
- Define simplified structural models for different building types.
- For each model and for each limit state, compute minimum and maximum expected displacement capacity, compute minimum and maximum expected period of vibration from secant stiffness, evaluate a displacement demand reduction factor as a function of the expected energy dissipation level.
- Plot the rectangles defined by the displacement and period values.
- Compare displacement demand and capacity, i.e. whenever a spectral line intersects a rectangle compute the probability associated with demand greater or less than capacity.

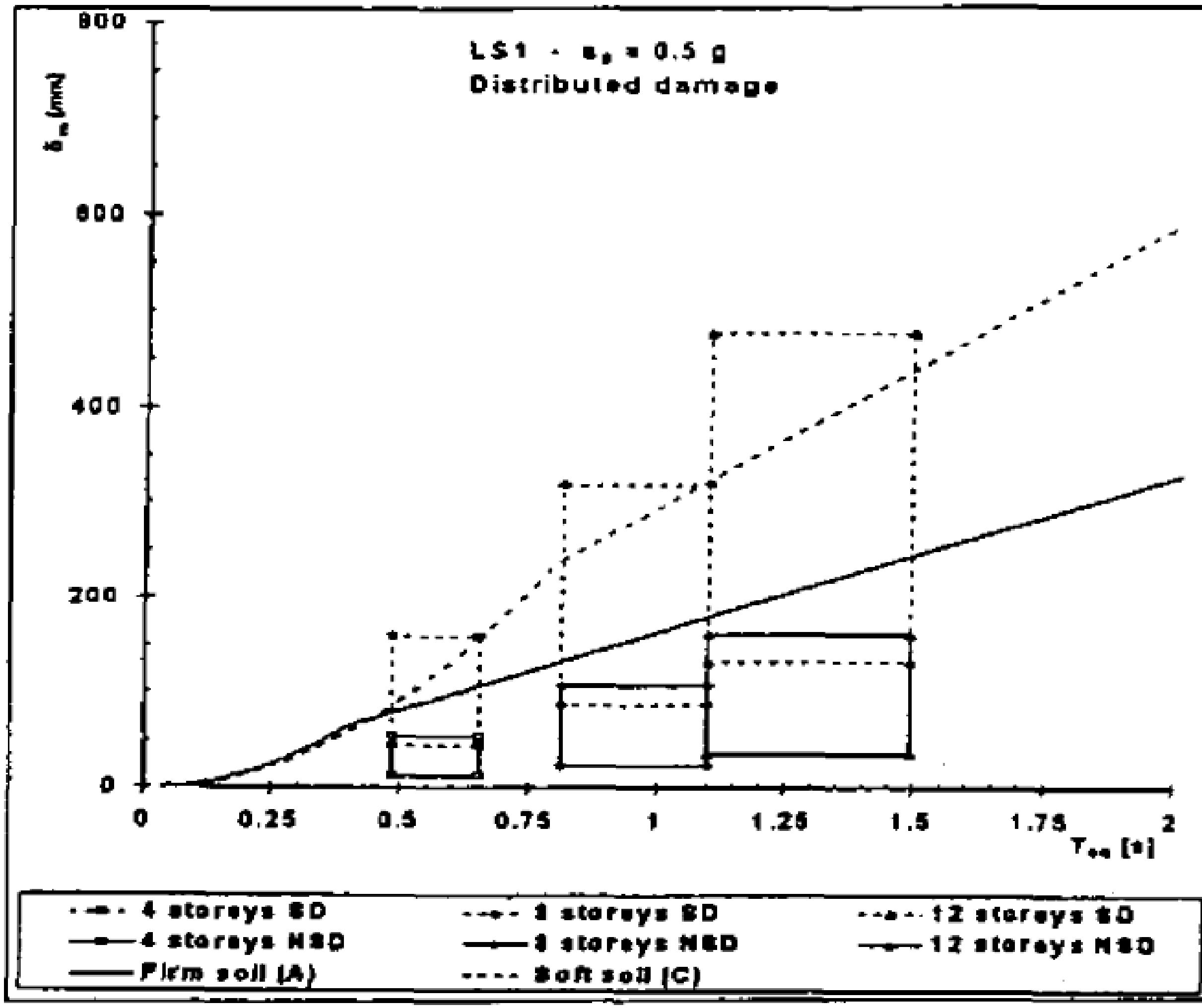

Fig. 2. Intersection of capacity areas with demand curves [Calvi, 1999].

Four limit states are here considered.

- LS1 No damage either structural or nonstructural. The expected response is essentially linear elastic, yielding is not attained in any critical section. Nonstructural damage is considered to initiate at drift ratio between 0.1% and 0.3%.

- LS2 Minor structural damage and/or moderate nonstructural damage. The building can be utilized after the earthquake without any need for significant strengthening and repair to structural elements. For assessment of reinforced concrete buildings the ultimate limit state values of ε_c = 0.0035-0.004 and ε_s = 0.01-0.015 are suggested. For slight nonstructural damage an interstory drift of 0.3 to 0.5% is considered.

- LS3 Significant structural damage and extensive nonstructural damage. It includes spalling of cover concrete, formation of wide flexural cracks requiring injection grouting. The essential aspect of response to this limit state is that the required repair is superficial with no fracture of tranverse reinforcement or buckling of longitudinal reinforcement. Local deformations in the critical sections in the order of ε_c = 0.006-0.01 and ε_s = 0.03-0.04. For nonstructural damage the average drift is about 0.5 to 1.5%.

- LS4 Collapse. This limit state corresponds to the inability of the structure to sustain the gravity loads.

Undamaged buildings	demand does not exceed LS1
Slightly damaged, usable buildings	demand exceeds LS1 but does not exceed LS2
Buildings extensively damaged, but still repairable	demand exceeds LS2 but does not exceed LS3
Buildings not collapsed, but so severely damaged that they have to be demolished	demolished exceeds LS3 but does not exceed LS4
Collapsed buildings	demand exceeds LS4

Definition of displacement spectra.

- Displacement spectra are considered as essentially linearly increasing with period.

- A relation suggested in EC8 for damping values different than 5% was adopted to calculate the correction factor η as a function of equivalent viscous damping:

$$\eta = (10/(5+\xi))^{1/2}$$

- The relation between equivalent damping (ξ_e) and ductility (μ) is:

$$\xi_e = \alpha(1-1/\mu^{\beta})+\xi_{\upsilon}$$

where α is a factor of 20-30 and β is between 0.5 and 1.

Equivalent structural models.

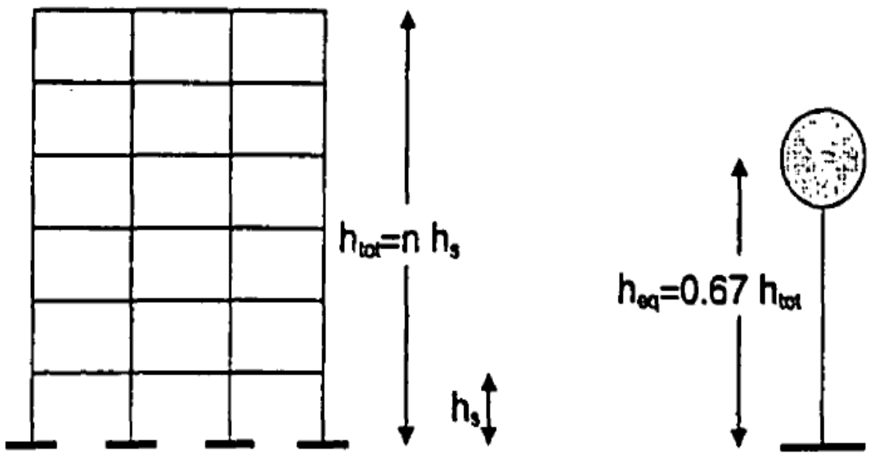

Fig. 2. A representation of a single-degree-of-freedom model equivalent to the real structure. A linear response of the reinforced concrete structure is here assumed (LS1).

Reinforced concrete buildings – limit state LS1

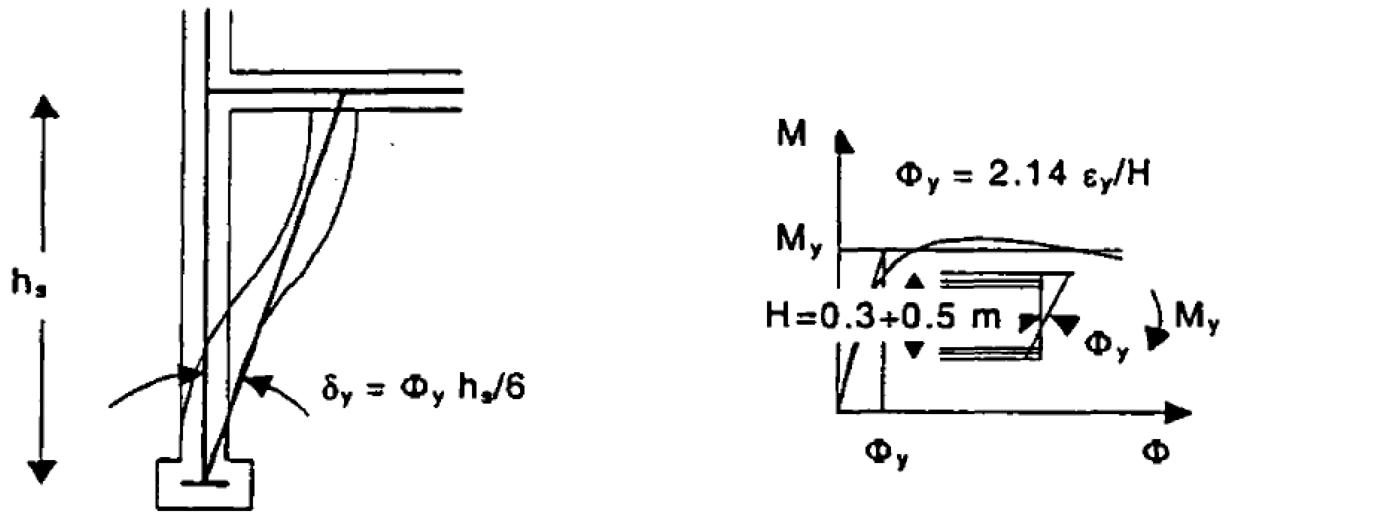

Fig. 3. Idealised structural response of reinforced concrete sections and columns at yield.

Reinforced concrete buildings – limit state LS1

- The displaced shape is assumed linear with the vertical axis.
- The constant drift is assumed to correspond to the theoretical yielding point of an idealized bilinear structural response.
- The resulting SDOF equivalent model has its center of gravity at 2/3 the height of the building.
- The SDOF structural displacement is given by:

$$\Delta 1 = 0.67\ n\ h_s\ \delta_y$$

where n number of stories, h_s story height, δ_y story drift

$$\delta_y = \Phi_y\ h_s / 6\ ,\quad \Phi_y = 2.14\ \varepsilon_y / H$$

where Φ_y is the yield curvature , H the depth of the column section assumed between 0.3 and 0.5 and ε_y the steel deformation at yield.

The story height is assumed between 3 and 4.5 m.

For older structures ε_y is assumed equal to 0.0018.

Reinforced concrete buildings – limit state LS1

Accordingly,

- $\Phi_{1A,Max} = 0.013$, $\Phi_{1A,Min} = 0.008$
- $\delta_{1A,Max} = 0.010$, $\delta_{1A,Min} = 0.004$
- $\Delta_{1A,Max} = 0.029\ n$, $\Delta_{1A,Min} = 0.008\ n$ (m)

According to EC8:

- $T_{1A,Max} = 0.075\ (4.5\ n)^{3/4}$, $T_{1A,Min} = 0.075\ (3\ n)^{3/4}$ (s)

For recent structures ε_y is assumed equal to 0.0025.
Accordingly,

- $\Phi_{1B,Max} = 0.018$, $\Phi_{1B,Min} = 0.011$
- $\delta_{1B,Max} = 0.013$, $\delta_{1A,Min} = 0.005$
- $\Delta_{1B,Max} = 0.040\ n$, $\Delta_{1B,Min} = 0.011\ n$ (m)

Limit values for non structural damage is:

- $\Delta_{2AB,NSD,Min} = 0.001\ n\ h_s$, $\Delta_{2AB,NSD,Max} = 0.003\ n\ h_s$ (m)

Reinforced concrete buildings – limit state LS2

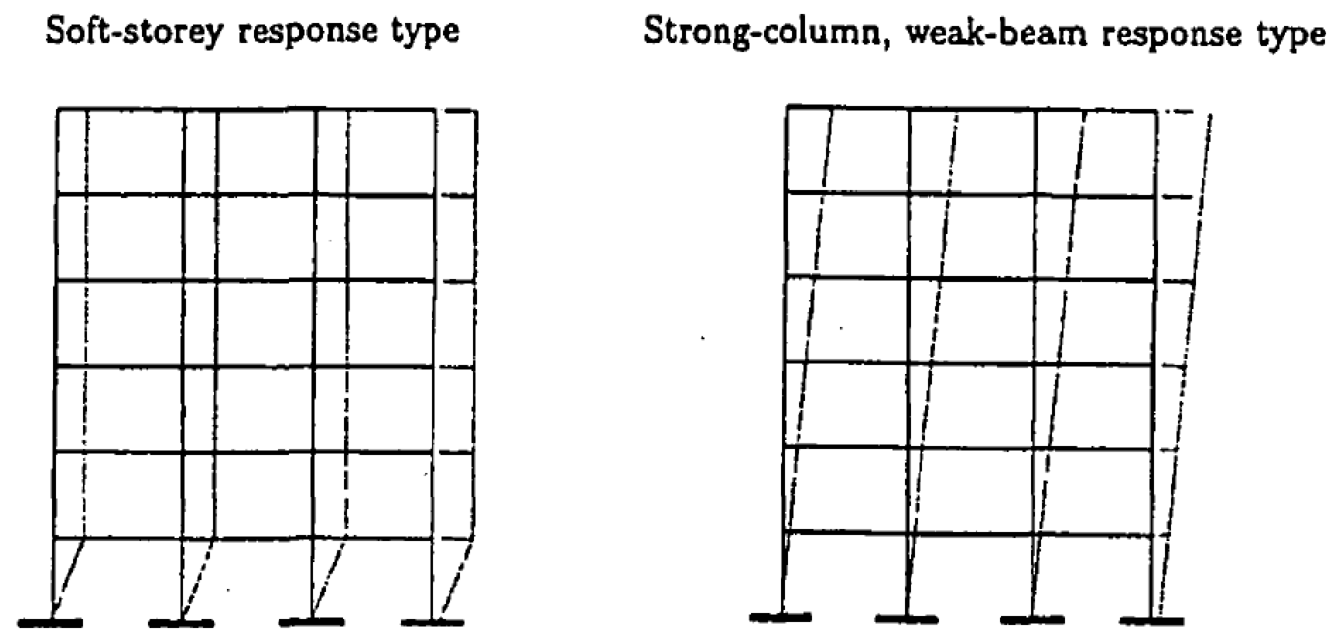

Fig. 4. Post-elastic response assumed for buildings designed with or without considering capacity design principles.

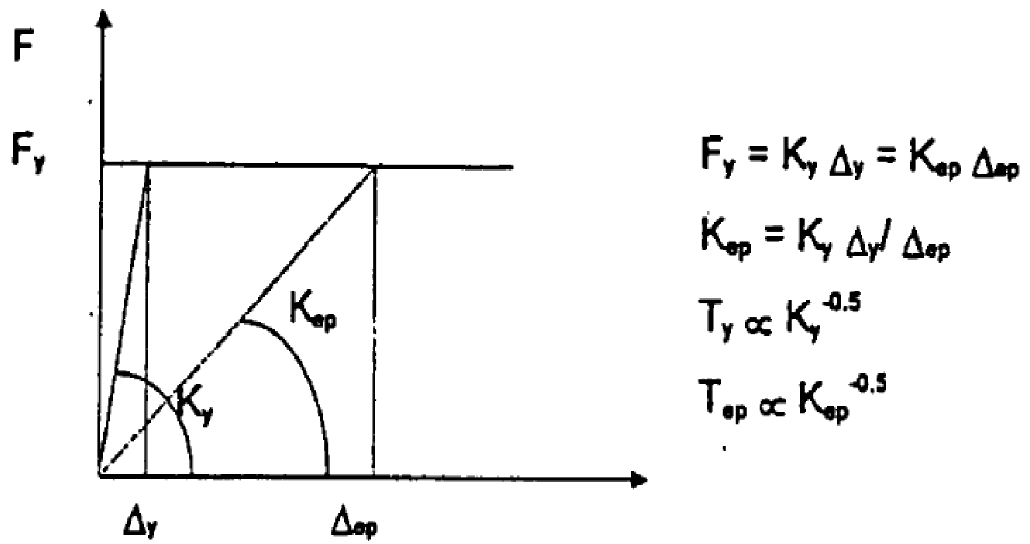

Fig. 5. Elastic-perfectly-plastic response. The period of vibration is inversely proportional to the square root of the secant stiffness, which is, in turn, inversely proportional to the displacement.

Reinforced concrete buildings – limit state LS2

- It is necessary to define the nonlinear response mechanism. For older buildings a soft story mechanism is considered. For recent buildings beam hinging is considered with a uniform distribution of the deformation along the height of the structure.

  Soft story type response.

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- $\Phi_{2A} = (\varepsilon_c+\varepsilon_s)/H$, $\varepsilon_c = 0.0035$, $\varepsilon_s = 0.01$

  Two extreme situation are considered:

- the limit values will be simultaneously reached in concrete and steel
- when the limit value is reached in concrete, only half of the assumed deformation capacity is exploited in steel.

Reinforced concrete buildings – limit state LS2

- $\Phi_{2A,Max} = 0.045$, $\Phi_{2A,Min} = 0.017$
- $\theta_{2A,Max}=0.45(0.045-0.013) = 0.014$,$\theta_{2A,Min} = 0.3(0.017-0.008) = 0.003$
- $\Delta^{P}_{2A,Max} = 4.5\ 0.014=0.065$, $\Delta^{P}_{2A,Min} = 3\ 0.003=0.008$ (m)
- $\Delta_{2A,Max} = 0.065 + 0.029n$, $\Delta_{2A,Min} = 0.008 + 0.008n$ (m)

- $T_{2A,Max} = 0.075(4.5n)^{3/4}(1+2.243/n)^{1/2}$ (s)
- $T_{2A,Min} = 0.075(3n)^{3/4}(1+1.081/n)^{1/2}$ (s)

- $\mu_{\Delta 2A,Max}=1+2.24/n$, $\mu_{\Delta 2A,Min}=1+1.08/n$

- $\xi_{2A.Max}(\%)=25(1-1/(1+2.24/n)^{0.5}) + 2$,
- $\xi_{2A.Min}(\%)=25(1-1/(1+1.08/n)^{0.5}) + 2$

- $\eta_{2A}=1-0.4/n$

Reinforced concrete buildings – limit state LS2

Distributed damage response type

- $\Delta_{2B,Max} = h_{eq}(\theta_{p,Max}+\delta_{y,Max})= 0.67\ 4.5n\ (0.013+0.010) =0.081n$
- $\Delta_{2B,Min} = h_{eq}(\theta_{p,Min}+\delta_{y,Min})= 0.67\ 3n\ (0.005+0.004) =0.018n$ (m)

- $T_{2B,Max} = 0.075(4.5n)^{3/4}1.38$ (s)
- $T_{2B,Min} = 0.075(3n)^{3/4}1.16$ (s)
- $\mu_{\Delta 2B,Max}=1.91$, $\mu_{\Delta 2B,Min}=1.35$, $\eta_{2B}=0.7$

Limit state LS3

<u>Soft story type response</u>

- $\Delta^{P}{}_{3A} = \ 2\ \delta y\ h_s$
- $\Delta_{3A,Max} = 0.087 + 0.029n$, $\Delta_{3A,Min} = 0.023 + 0.008n$ (m)
- $T_{3A,Max} = 0.075(4.5n)^{3/4}(1+3/n)^{1/2}$ (s)
- $T_{3A,Min} = 0.075(3n)^{3/4}(1+3/n)^{1/2}$ (s)
- $\mu_{\Delta 3A,Max} = 1+2.99/n = \mu_{\Delta 2A,Min}$
- $\eta_{3A} = 1-0.5/n$

<u>Distributed damage response type</u>

- $\Delta_{3B} = 4.\ 0.67\ n\ h_s\ \delta_y$
- $\Delta_{3B,Max} = 0.160\ n$, $\Delta_{3B,Min} = 0.044\ n$
- $T_{3B,Max} = 0.075(4.5n)^{3/4}2$ (s)
- $T_{3B,Min} = 0.075(3n)^{3/4}2$ (s)
- $\eta = 0.5$

METHOD PROPOSED BY GLAISTER AND PINHO

- Fundamental in concept to this simplified procedure is the relationship between the different qualitative damage states usually defined in loss estimation studies and interstory or global drifts in buildings.

- Using analytical relationships between displacement capacity and height, and empirical relationships between height and elastic period, it is possible to plot capacity curves for different limit states in terms of period and displacement

- In the derivation of the capacity curves, it is important to recognize that demand is represented by a displacement spectrum providing the expected displacement induced by an earthquake on a SDOF oscillator.

- The displacement capacity provided must also be that of a SDOF substitute structure than that of the structure itself.

- The capacity curves must be a function of an effective height coefficient ef_h defined as the ratio between the height to center of mass of a SDOF substitute structure H_{SDOF} that has the same displacement capacity as the original structure at its center of seismic force and the total height of the structure H_T.

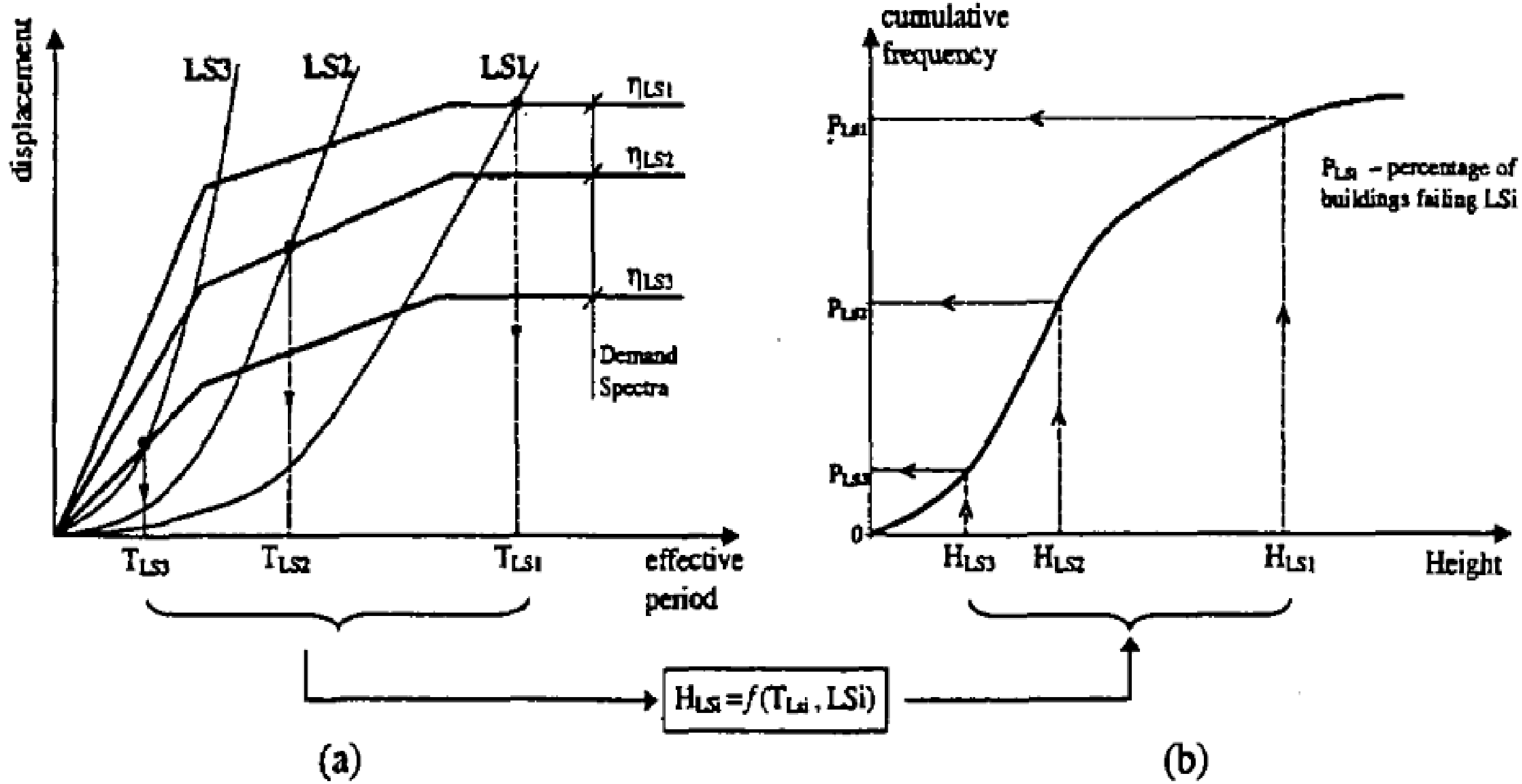


Fig. 3. Proposed methodology: (a) displacement capacity/demand curves (b) CDF of building stock.

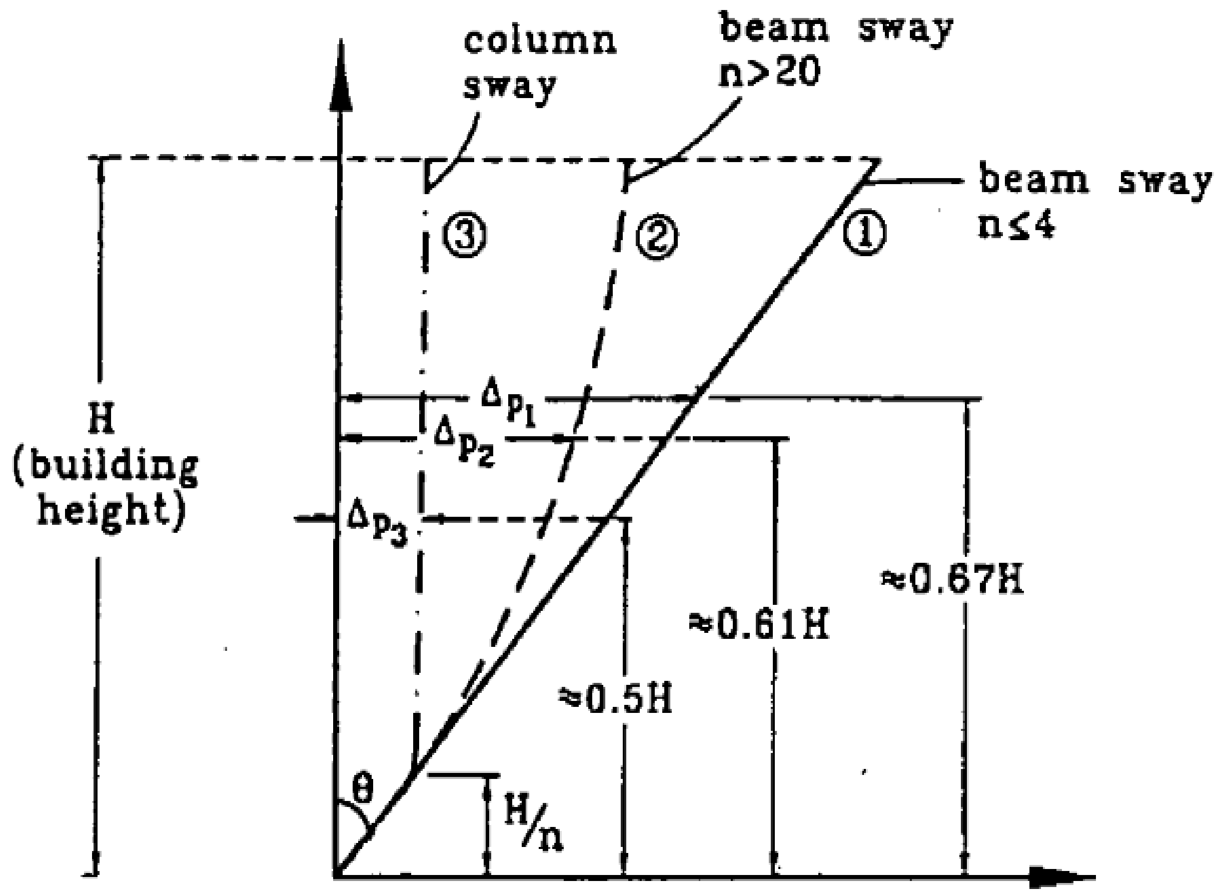


Fig. 4. Deformation profiles for beam-sway and column-sway frames [Priestley, 1997].

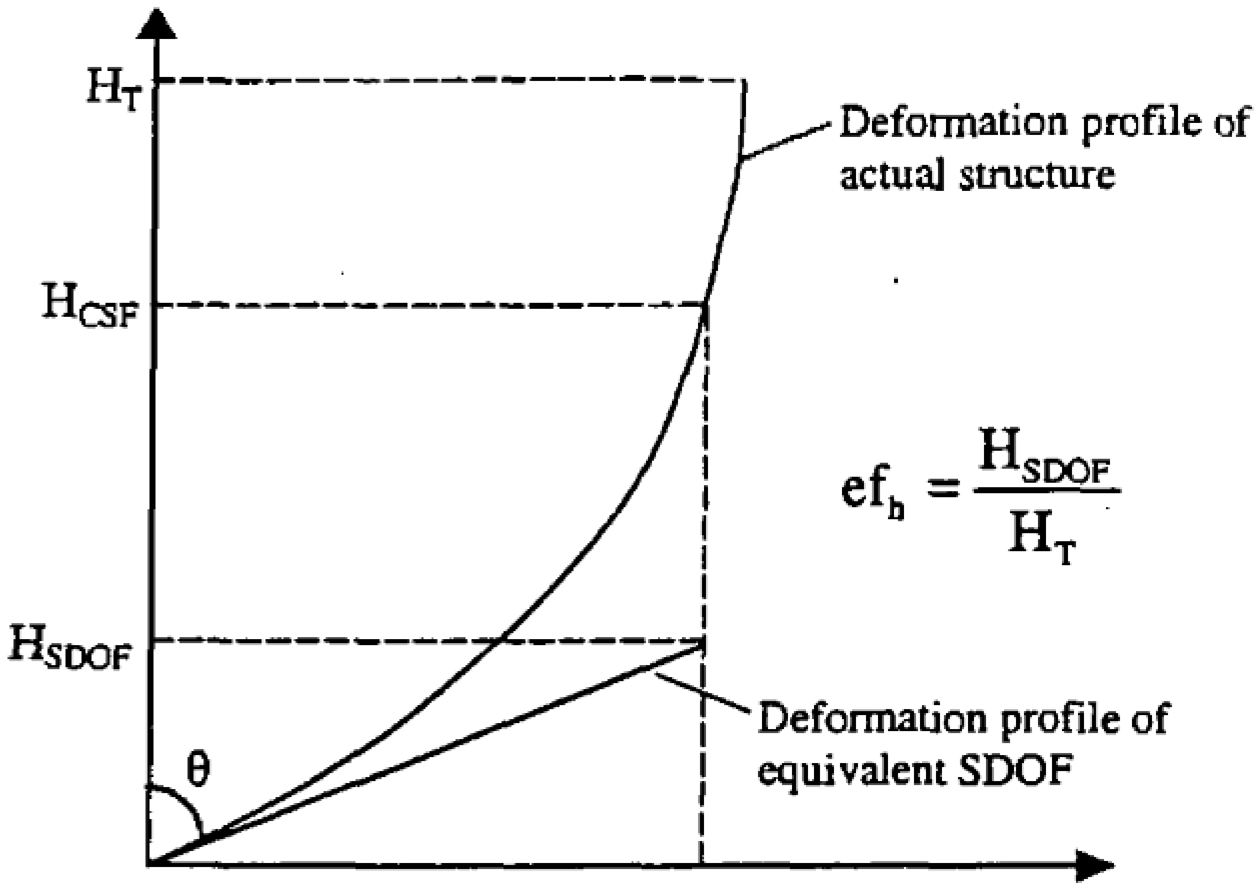


Fig. 5. Definition of effective height coefficient.

EFFECTIVE HEIGHT RELATIONSHIPS

$$ef_h = 0.64 \qquad n \leq 4$$

$$ef_h = 0.64 - 0.0125(n - 4) \qquad 4 < n < 20$$

$$ef_h = 0.44 \qquad n \geq 20\,.$$

Capacity – height relationships

YIELD LIMIT STATE

$$\Phi_y = \mathbf{1.7}\frac{\varepsilon_y}{h_b}$$

$$\theta_{by} = \frac{\mathbf{1}}{\mathbf{6}}\Phi_y l_b = \mathbf{0.283}\varepsilon_y \frac{l_b}{h_b}$$

$$\theta_y = \mathbf{1.75}\times\theta_{by} = \mathbf{0.5}\varepsilon_y \frac{l_b}{h_b}$$

$$\Delta_{LSy} = ef_h H_T \theta_y = 0.5 ef_h H_T \varepsilon_y \frac{l_b}{h_b}$$

Capacity – height relationships

POST YIELD LIMIT STATE

$$\Phi_{LSi} = (\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)})\frac{\mathbf{1}}{h_b}$$

$$\Phi_{pi} = \Phi_{LSi} - \Phi_y$$

$$L_{ph} = ef_p h_b = \mathbf{0.5} h_b$$

$$\theta_{pi} = L_{ph}(\Phi_{LSi} - \Phi_y) = \mathbf{0.5} h_b(\Phi_{LSi} - \Phi_y)$$

$$\Delta_{pi} = \theta_{pi} ef_h H_T = \mathbf{0.5} h_b(\Phi_{LSi} - \Phi_y) ef_h H_T$$

$$\Delta_{LSi} = \mathbf{0.5} ef_h H_T\left[\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - \left(\mathbf{1.7} - \frac{l_b}{h_b}\right)\varepsilon_y\right]$$

$$\mu_{LSi} = \frac{\Delta_{LSi}}{\Delta_{LSy}} = \mathbf{1} + \frac{\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - \mathbf{1.7}\varepsilon_y}{\varepsilon_y}\frac{h_b}{l_b}$$

Period – height relationships

YIELD LIMIT STATE

$$T_{LSy} = \mathbf{0.1} H_T^{3/4}$$

$$H_T(T_{LSy}) = \left(\mathbf{10} \cdot T_{LSy}\right)^{4/3}$$

Period – height relationships

POST YIELD LIMIT STATE

$$T_{LSi} = T_{LSy}\sqrt{\mu_{LSi}}$$

$$H_T(T_{LSi}) = C_3 T_{LSi}^{4/3}$$

$$C_3 = 10^{4/3}\left(1 + \frac{\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - 1.7\varepsilon_y}{e_y}\frac{h_b}{l_b}\right)^{-2/3}$$

Capacity curves

$$\Delta_{LSy}(T_{LSy}) = \frac{\mathbf{0.5} e f_h l_b \varepsilon h_y}{h_b} (\mathbf{10} \cdot T_{LSy})^{\mathbf{4}/\mathbf{3}}$$

$$\Delta_{LSi}(T_{LSi}) = \mathbf{0.5} e f_h C_{\mathbf{3}} T_{LSi}^{\mathbf{4}/\mathbf{3}} \left[\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - \left(\mathbf{1.7} - \frac{l_b}{h_b} \right) \varepsilon_y \right]$$

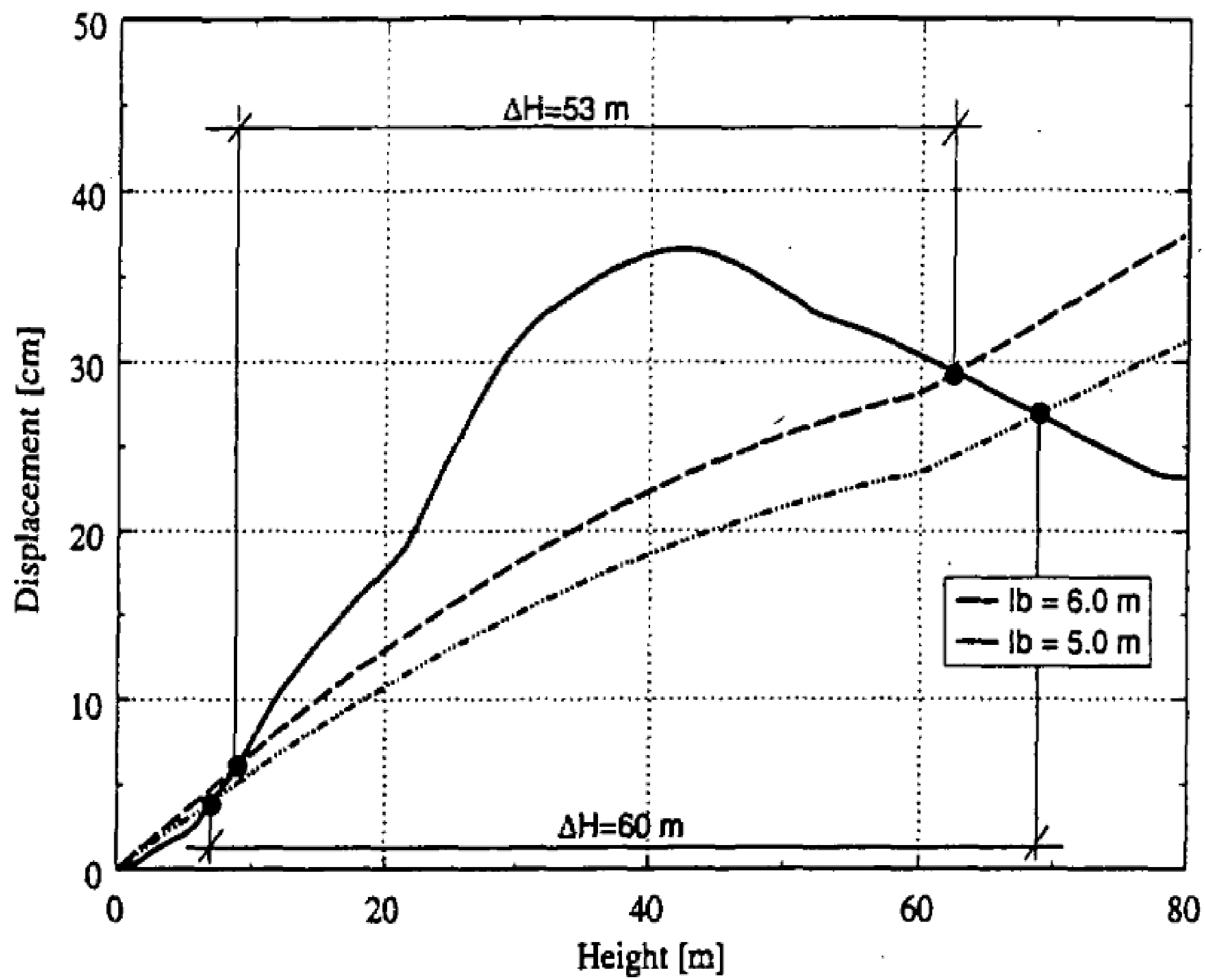
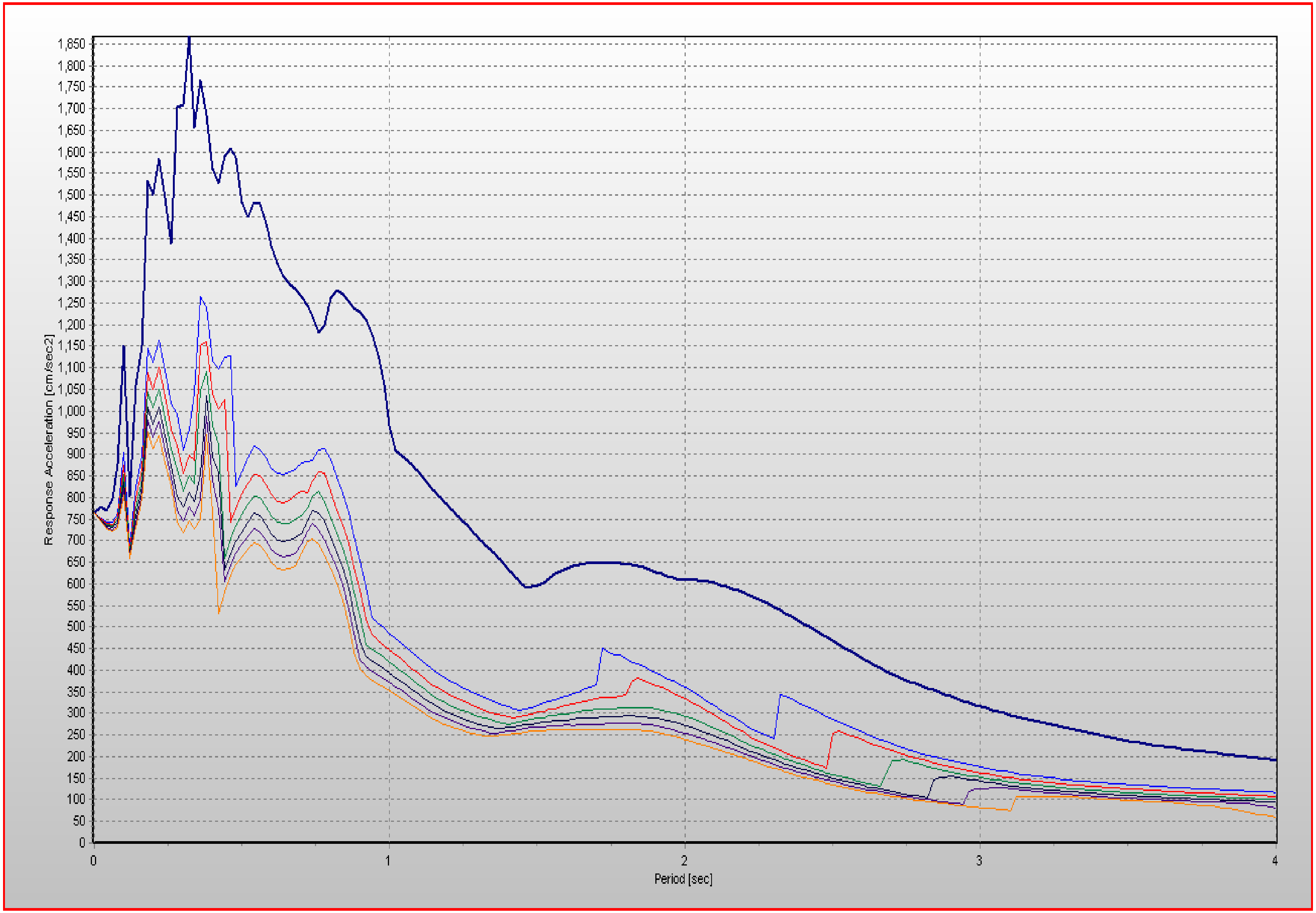


Fig. 16. Beam-sway yield capacity/demand curves for different beam lengths (record scaled down by 50%).

HOMEWORK 6



T_1 = 2.24 sec , SA_y= 200 cm/sec^2 , SD_y= 25.4 cm

T_2= 1.94 sec , SA_y= 320 cm/sec^2 , SD_y= 30.5 cm

μ_1=2.4 , Sd_{u1} = 60 cm

μ_2= 1.9, Sd_{u2} = 58 cm

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\overline{S}_{d,ds}}\right)\right]$$

$\overline{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state, ds, and

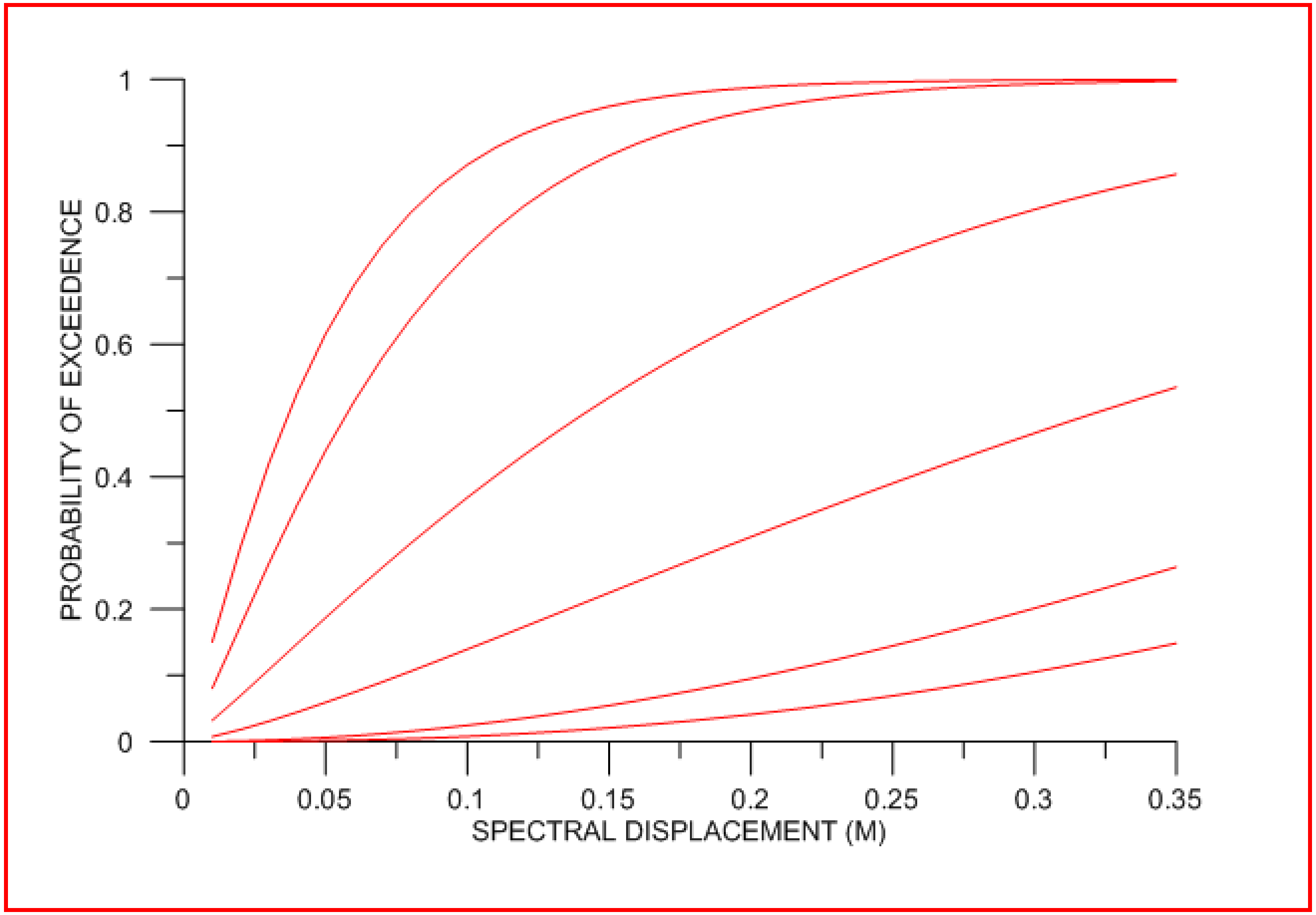
Φ is the standard normal cumulative distribution function.

Section 2.2. Normal Distribution

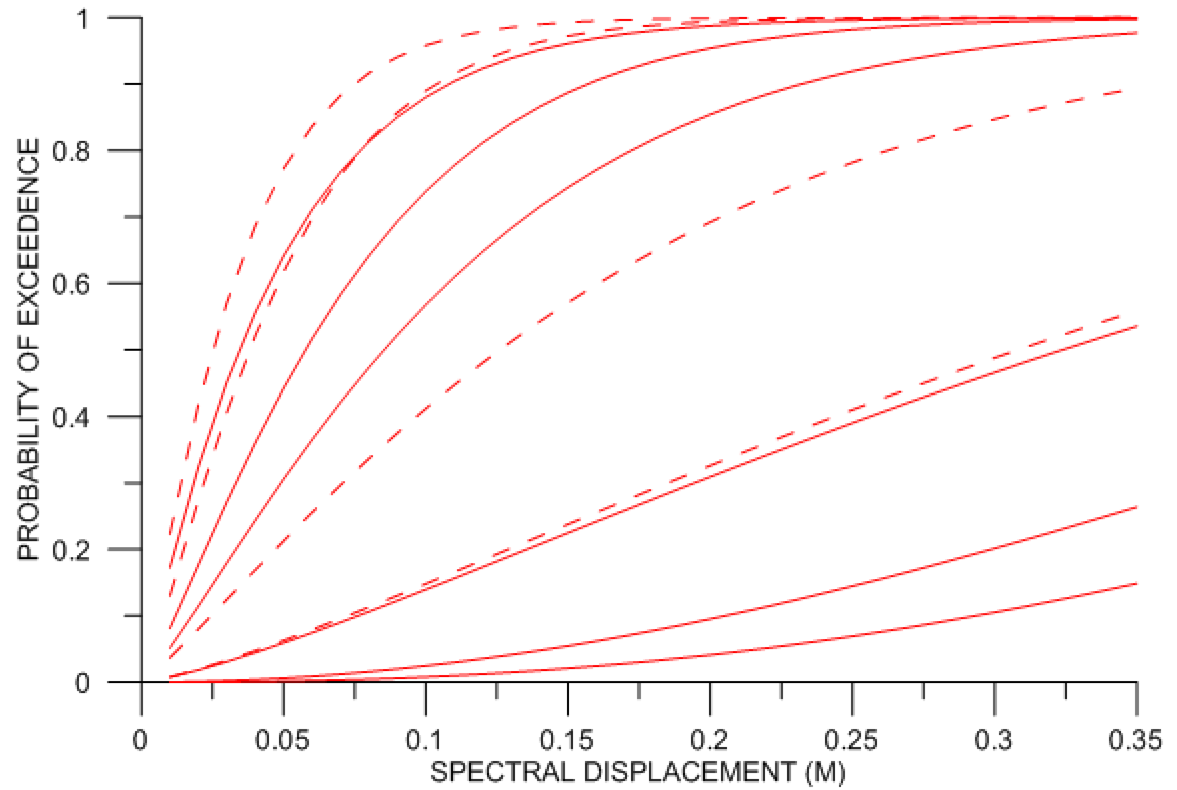
TABLE C-1 Values of the CDF of the standard normal distribution, $F_Z(z) = 1 - F_Z(-z)$

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
−3.4	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0003	0.0002
−3.3	0.0005	0.0005	0.0005	0.0004	0.0004	0.0004	0.0004	0.0004	0.0004	0.0003
−3.2	0.0007	0.0007	0.0006	0.0006	0.0006	0.0006	0.0005	0.0005	0.0005	0.0005
−3.1	0.0010	0.0009	0.0009	0.0009	0.0008	0.0008	0.0008	0.0008	0.0007	0.0007
−3.0	0.0013	0.0013	0.0013	0.0012	0.0012	0.0011	0.0011	0.0011	0.0010	0.0010
−2.9	0.0019	0.0018	0.0017	0.0017	0.0016	0.0016	0.0015	0.0015	0.0014	0.0014
−2.8	0.0026	0.0025	0.0024	0.0023	0.0023	0.0022	0.0021	0.0021	0.0020	0.0019
−2.7	0.0035	0.0034	0.0033	0.0032	0.0031	0.0030	0.0029	0.0028	0.0027	0.0026
−2.6	0.0047	0.0045	0.0044	0.0043	0.0041	0.0040	0.0039	0.0038	0.0037	0.0036
−2.5	0.0062	0.0060	0.0059	0.0057	0.0055	0.0054	0.0052	0.0051	0.0049	0.0048
−2.4	0.0082	0.0080	0.0078	0.0075	0.0073	0.0071	0.0069	0.0068	0.0066	0.0064
−2.3	0.0107	0.0104	0.0102	0.0099	0.0096	0.0094	0.0091	0.0089	0.0087	0.0084
−2.2	0.0139	0.0136	0.0132	0.0129	0.0125	0.0122	0.0119	0.0116	0.0113	0.0110
−2.1	0.0179	0.0174	0.0170	0.0166	0.0162	0.0158	0.0154	0.0150	0.0146	0.0143
−2.0	0.0228	0.0222	0.0217	0.0212	0.0207	0.0202	0.0197	0.0192	0.0188	0.0183
−1.9	0.0287	0.0281	0.0274	0.0268	0.0262	0.0256	0.0250	0.0244	0.0239	0.0233
−1.8	0.0359	0.0352	0.0344	0.0336	0.0329	0.0322	0.0314	0.0304	0.0301	0.0294
−1.7	0.0446	0.0436	0.0427	0.0418	0.0409	0.0401	0.0392	0.0384	0.0375	0.0367
−1.6	0.0548	0.0537	0.0526	0.0516	0.0505	0.0495	0.0485	0.0475	0.0465	0.0455
−1.5	0.0668	0.0655	0.0643	0.0630	0.0618	0.0606	0.0594	0.0582	0.0571	0.0559
−1.4	0.0808	0.0793	0.0778	0.0764	0.0749	0.0735	0.0722	0.0708	0.0694	0.0681
−1.3	0.0968	0.0951	0.0934	0.0918	0.0901	0.0885	0.0859	0.0853	0.0838	0.0823
−1.2	0.1151	0.1131	0.1112	0.1093	0.1075	0.1056	0.1038	0.1020	0.1003	0.0985
−1.1	0.1357	0.1335	0.1314	0.1292	0.1271	0.1251	0.1230	0.1210	0.1190	0.1170
−1.0	0.1587	0.1562	0.1539	0.1515	0.1492	0.1469	0.1446	0.1423	0.1401	0.1379
−0.9	0.1841	0.1814	0.1788	0.1762	0.1736	0.1711	0.1685	0.1660	0.1635	0.1611
−0.8	0.2119	0.2090	0.2061	0.2033	0.2005	0.1977	0.1949	0.1922	0.1894	0.1867
−0.7	0.2420	0.2389	0.2358	0.2327	0.2296	0.2266	0.2236	0.2206	0.2177	0.2148
−0.6	0.2743	0.2709	0.2676	0.2643	0.2611	0.2578	0.2546	0.2514	0.2483	0.2451
−0.5	0.3085	0.3050	0.3015	0.2981	0.2946	0.2912	0.2877	0.2843	0.2810	0.2776
−0.4	0.3446	0.3409	0.3372	0.3336	0.3300	0.3264	0.3228	0.3192	0.3156	0.3121
−0.3	0.3821	0.3783	0.3745	0.3707	0.3669	0.3632	0.3594	0.3557	0.3520	0.3483
−0.2	0.4207	0.4168	0.4129	0.4090	0.4052	0.4013	0.3974	0.3936	0.3897	0.3859
−0.1	0.4602	0.4562	0.4522	0.4483	0.4443	0.4404	0.4365	0.4325	0.4286	0.4247
−0.0	0.5000	0.4960	0.4920	0.4880	0.4840	0.4801	0.4761	0.4721	0.4681	0.4641

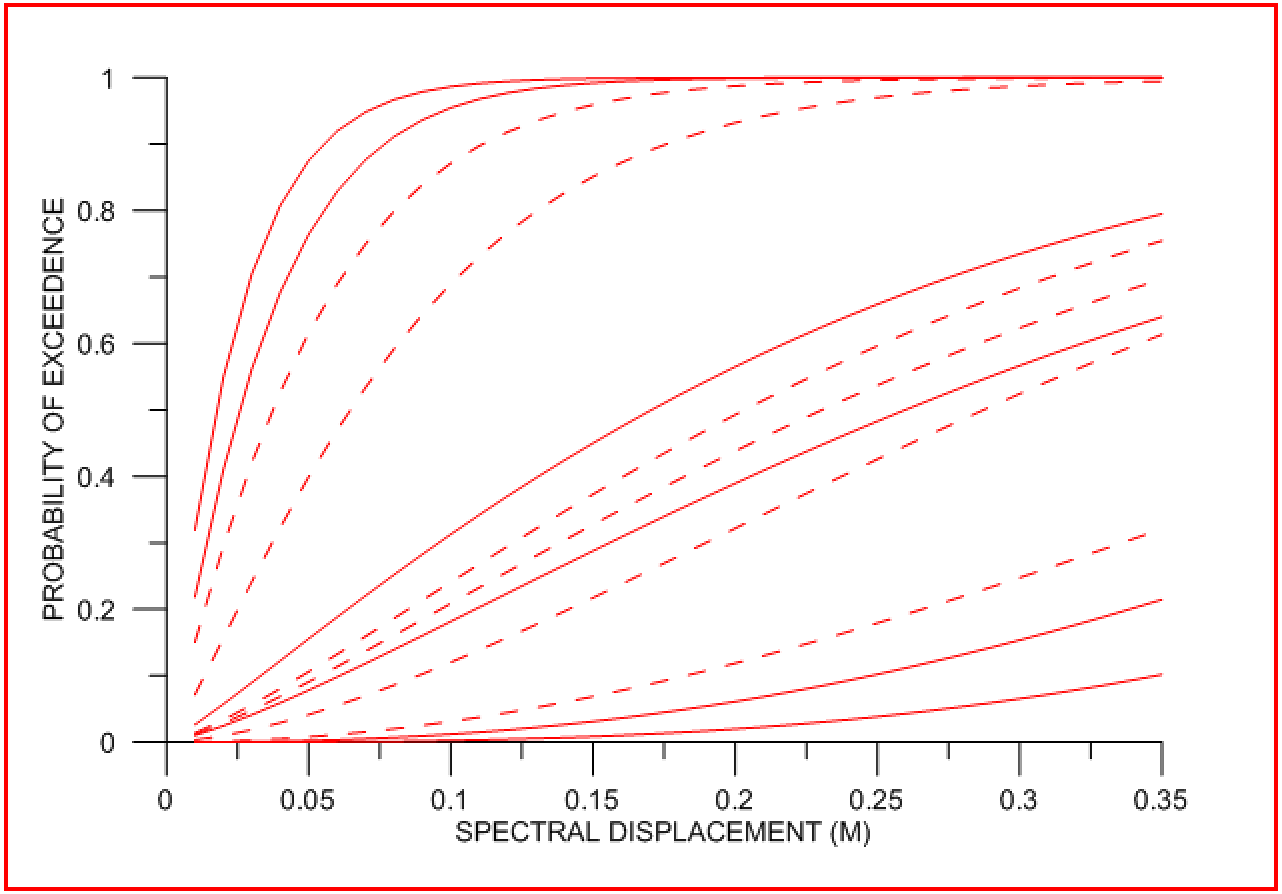
Damage	Sd,ds1	Prob1	Sd,ds2	Prob2
Slight	20	0.997	28	0.9656
Moderate	25.4	0.9236	38	0.758
Extensive	38	0.7157	76	0.3669
Collapse	60	0.50	107	0.2709



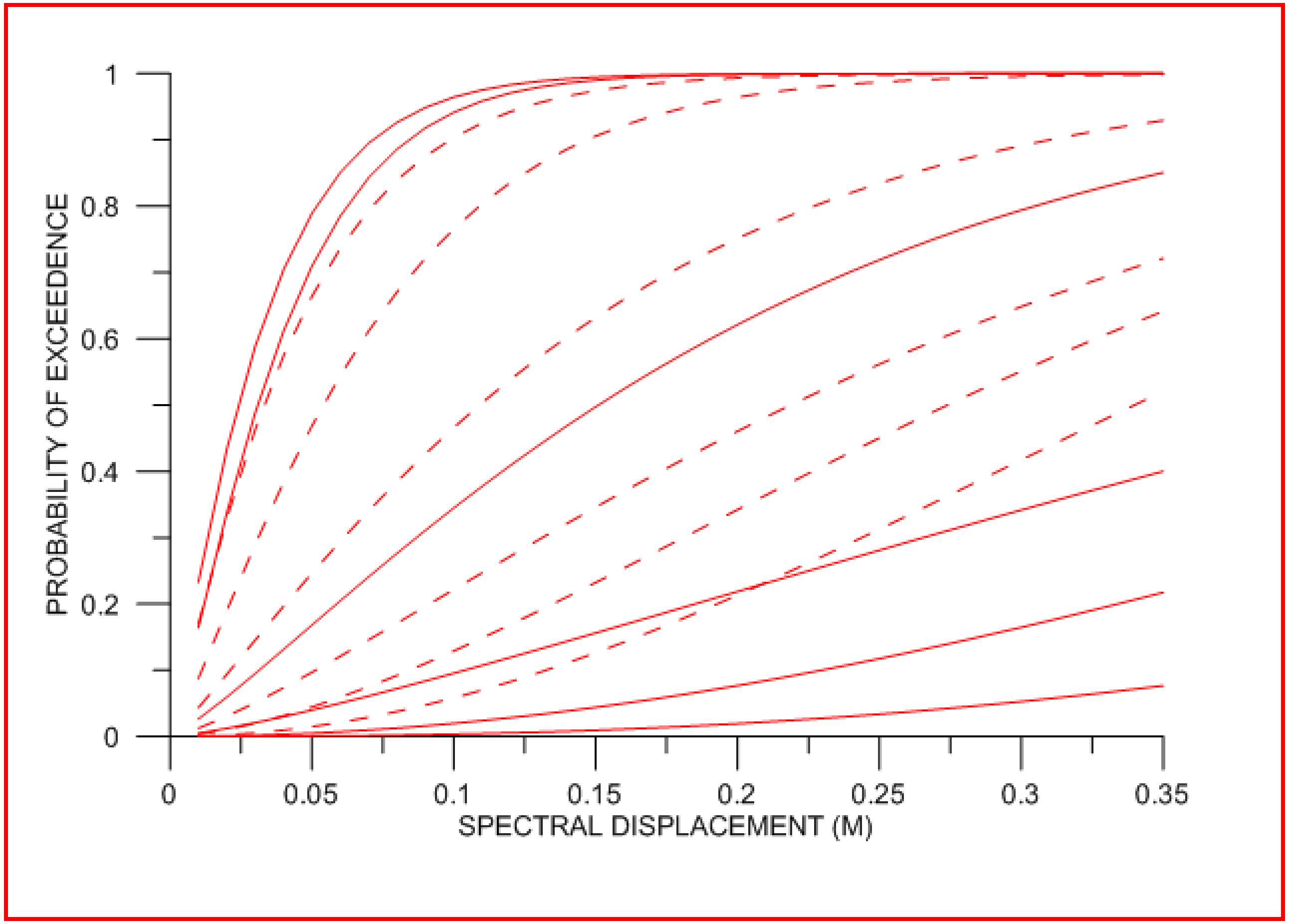
PROBABILITY OF EXCEEDENCE
1
0.8
0.6
0.4
0.2
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
SPECTRAL DISPLACEMENT (M)



BARE vs INFILLED FRAMES



LOW vs MID-RISE FRAMES



OLD vs PRE-CODE FRAMES