0155

AN EVALUATION OF THE DISPLACEMENT CONTROLLED DESIGN PROCEDURES

Vladimir SIGMUND[1], Ivica GULJAS[2] And Kresimir HERMAN[3]

SUMMARY

When yielding occurs in a structure during extreme earthquakes formation of a desired earthquake resistant mechanism does not in itself guarantee that repair cost would be tolerable. By controlling inter-story drift we can reduce the expected damage and economic risk. It has been proved in many research works that adequate strength does not have a decisive influence on expected structural drift. Therefore, maximum displacements, rather than maximum stresses, represent the proper design criteria. This differs from current force-based design philosophy that is based on acceleration spectra, code performance factors that correlate poorly with damage potential, and displacement checks to ensure that non-structural drift limitations are not exceeded.

The quality of drift estimates for the r/c frames by three different methods of drift evaluation is evaluated on the model structures. The structures were subjected to inelastic time history analysis of three different earthquakes using LARZWD (4) and the results are compared.

The first procedure (Priestley, 6) is based on design displacement spectra and characterisation of the structure by an equivalent secant stiffness to maximum response, with hysteretic energy dissipation represented by equivalent viscous damping.

The second procedure is N2 (Fajfar et al., 5) uses two separate mathematical models (MDOF and SDOF) and combines response spectrum approach and non-linear static (push-over) analysis method.

The third procedure (Sozen et al., 8) is based on the linear elastic analysis and limit state design of the sections. Design process defined by this method assigns a minor role to lateral strength in earthquake-resistant design.

INTRODUCTION

The building codes use strength as the main design parameter and place computation of forces at the centrepiece of earthquake resistant design, relegating drift calculations to the background in design process. In the case of extreme earthquakes, when yielding occurs in a structure, formation of a desired earthquake resistant mechanism does not in itself guarantee that repair cost would be tolerable. Limit state can best be represented by deformation rather than by strength as damage can be directly correlated with drift. Therefore, maximum displacements, rather than maximum stresses, represent the proper design criteria.

At the initial stage of structural design, a structural engineer should consider the necessary repair costs under various earthquake intensities. The prediction of important characteristics of input ground motion and the basic structural parameters are uncertain. Therefore, there is no sense in carrying out calculation to an excessive degree of accuracy. It is reasonable to use, in a design process, relatively simple mathematical models which yield adequate accuracy at acceptable cost.

[1] University J.J.Strossmayer, Faculty of Civil Engineering, Drinska 16a, 31000 Osijek, Croatia
[2] University J.J.Strossmayer, Faculty of Civil Engineering, Drinska 16a, 31000 Osijek, Croatia
[3] University of Zagreb, Faculty of Civil Engineering, Kaciceva 26, 10000 Zagreb, Croatia

Expected drifts during earthquakes of the model structures, calculated by the three methods are correlated with inelastic time history analysis (MDOF) using time three different time history records.

PROCEDURES

Three different methods for the evaluation of expected maximum displacements and interstory drifts during earthquakes are evaluated and compared.

(1) The first method for estimation of the expected lateral drifts is defined in (Priestley, 1998) and is only a part of the Displacement based approach to limit state design of the new structures (subsequently referred to as DBD method).

For r/c frames, based on the analysis of extensive data base of beam/column subassemblage testing, he suggested an expression for the story yield drift Θy in the form

1. $$\Theta y = 0{,}5 * \varepsilon y * \left[\frac{lb}{hb} \right]$$

where εy =yield strain of longitudinal reinforcement (=0,002), lb is the beam bay length and hb the beam depth.

The peak design drift, for the frame structures, can be determined as:

2. $$\Theta d = \mu * \Theta y \leq \Theta c$$

where the design drift Θd is comprised of elastic components Θy and design ductility limit μ.

If the frame members are not unusually deep, and low strength reinforcement is used, then the code drift limit of 0,025 for the estimate of μ governs. Design procedure requires determination of design displacement and effective mass and damping for the equivalent SDOF system. The yield displacement at the height of the resultant lateral seismic force for the r/c frames can be estimated as

3. $$\Delta y = 0{,}5 * \varepsilon y * (lb / hb) * (0{,}6 * hn)$$

where the height of resultant force is, for the regular structures, estimated at the 0,6*hn.

and design displacement as

4. $$\Delta d = \Sigma(mi\Delta i^2) / \Sigma(mi\Delta i)$$

where mi are story masses.

The ductility can now be determined as

5. $$\mu_s = \Delta d / \Delta y$$

Design displacement at the ith level is calculated by using the ductility value μ_s, and design displacements (extreme displacements) from the characteristic displacement profiles at maximum response based on elastic time history analysis as

6. $$\Delta i = \Theta d * hi * (1 - 0{,}5 * hi / hn)$$

where hi and hn are the heights of the ith and nth level and n is the number of stories.

In this work only a part of the suggested displacement design procedure is used in order to evaluate the quality of the drift estimates (an upper bound of) while we did not correlate it directly with the earthquake.

Required data for the estimate of displacements are: (a) basic structural data (framing and member sizes); (b) code drift limit; (c) ground characteristics;

(2) The N2-method (Fajfar et al., 4) uses two separate mathematical models (MDOF and SDOF) and combines response spectrum approach and non-linear static (push-over) analysis. It is commonly used as a second level procedure to estimate displacements and strain levels as damage indicators in the structure during design earthquake motion.

The first step is to perform non-linear static (push-over) analysis of MDOF system. By assuming displacement shape we determine vertical distribution of lateral loading and obtain base-shear roof displacement relationship. An equivalent SDOF system is then formed by transforming the MDOF quantities to bilinear force-displacement relationship of SDOF. The non-linear displacements of SDOF are calculated by non-linear dynamic analysis. Seismic drift of MDOF model is obtained by transforming the calculated SDOF displacement to the roof displacement (Dr) of MDOF model. Local quantities (story drifts corresponding to Dt) are calculated by repeating push-over analysis of MDOF up to the roof displacement Dr.

Required data are:(a) basic structural data; (b) moment rotation (M-φ) relationship for the members; (c) ground characteristics, (d) elastic (pseudo)acceleration spectrum, Ae.

(3) Linear elastic analysis (LINEAR) is performed according to the methodology outlined in (Lepage & Sozen, 1997), and is explained in short here.

Lepage and Sozen have concluded that linear spectral analysis could be used for the evaluation of expected non-linear drifts during earthquakes and to distinguish among various structural systems in design phase. The expected non-linear drifts are lower or equal to the drifts calculated by linear spectral analysis for 2% damping. The following equation can be used as a bound for the expected non-linear drift calculations:

7. $$DR=\begin{cases} 1/TR & \text{for } TR<1 \\ 1 & \text{for } TR\geq 1 \end{cases}$$

where: TR=period ratio= (To*$\sqrt{2}$)/Tg (earthquake period); To=initial structural period calculated with gross sections; DR=drift ratio= (non-linear drift)/(calculated linear drift for 2% damping using idealised spectral response).

For a MDOF system, with a reasonably uniform distribution of story mass and stiffness, the maximum displacement at any level i (Dmax,i) may be estimated using:

8. $$D_{max,i}=\gamma*\varphi i*\frac{Fa*\alpha*g*Tg}{(2*\pi)^2}*Teff$$

where: γ =participation factor for a given mode shape; φ i=ordinate defining the assumed mode shape at level i; Fa=acceleration amplification factor (usually 3.75); α =peak ground acceleration expressed as a coefficient of the acceleration of gravity; g=acceleration of gravity; Tg= characteristic period of the ground motion; Teff=To*$\sqrt{2}$ is effective structural vibration period for the first mode.

Base shear strength plays a minor role by drift evaluation, but it should be above a minimum value defined by an equation:

9. $$Cy=\alpha*(1-TR)\geq\alpha/6$$

Drift estimate is only a part of a Method for drift control in earthquake resistant design of r/c building structures (8).

Required data are: (a) basic structural data (framing and member sizes); (b) building period of vibration based on the gross plain sections, To; (c) design earthquake defined in terms of peak ground acceleration coefficient , α , and characteristic period for ground motion, Tg.

MODEL STRUCTURES

Three different r/c frame buildings were chosen as model structures. They cover broad period range from 0.4 to 1.2 sec. The first has 4 stories and 2 bays (4s_6m_6m), the second 7 stories and 3 bays (7s_6m_5m_6m) and the third 10 stories and 2 bays (10s_4m_4m). Framing, member sizes and required reinforcement were chosen according to the present (HRN) codes and with traditional limits on sizes and proportions. Basic information about the structures are presented in Table 1 and shown on Fig. 1.

Table 1. Basic structural data

Structure	1.Story (m)	2.-Roof (m)	Roof (m)	Bay (m)	Columns (cm)	Beams (cm)	Story W (kN)	To (sec)
4s_6m	3.75	3	12.75	6,6	50/50	30/60	720	0.431
7s_6m	3.75	3	21.75	6,5,6	50/50	30/50	1096	0.667
10s_4m	3.75	3	30.75	4,4	45/45	30/40	320	1.197

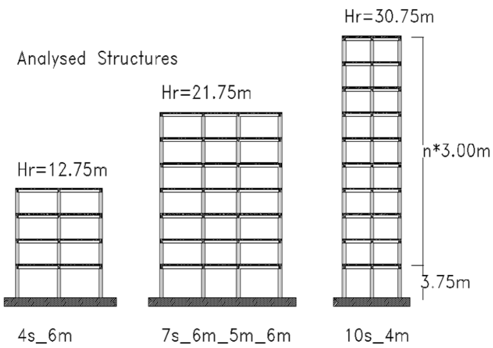

Figure 1. Analysed structures

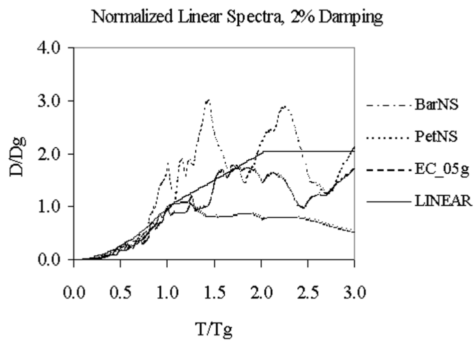

Figure 2. Normalised displacement response spectra for the accelerograms and 2% damping

Table 2. Accelerogram characteristics

Acc.	tD (sec)	ag (g)	v (cm/sec)	d (cm)	SI_20% (cm)	Tg (sec)	Dg (cm)
Bar NS	47.84	0.364	41.2	9.8	148.9	0.978	32.400
Petrovac NS	48.26	0.436	41.3	8.2	128.6	0.458	8.500
ElCentroNS	42.4	0.509	47.3	15.89	135.7	0.550	14.300

where: ag=peak ground acceleration in terms of g; v=peak ground velocity; d= peak ground displacement; SI=Housner spectrum intensity for 20% damping; Tg=characteristic period of the ground motion (period at which the assumed constant acceleration region ends); Dg=characteristic displacement for TG with 2% damping.

Three different input accelerogram time histories were chosen in order to include the ground motions of different characteristics (regarding ground, frequency content and duration). Accelerograms Bar N-S and Petrovac N-S were recorded during earthquake of 15.04.1979 in Monte Negro, El Centro S00E, 1940 is scaled to 0.5g). Calculated and idealised linear response displacement spectra for 2% damping of these accelerograms according to (8) are shown on Figure 2.

ANALYSIS RESULTS

For each of the model structures calculated are global (Mean Drift Ratio (MDR)=roof displacement / building height) and local (Maximum Story and Interstory Drift Ratios) response values according to each of the three outlined methods for all three accelerograms.

1. Inelastic time history analysis was performed by LARZWD (4) using time history records of ground motions and the results could be regarded as “exact” values of the expected drifts.

2. Maximum drifts and drift distribution along the structure's height are calculated as explained under the DBD method by using only the frame geometry and code drift limit of 0,025 for the "extreme" earthquakes. No special correlation with the ground motion intensity was made and therefore estimated drifts represent an upper bound for all three ground motions.

3. For the N2 method “push-over” analysis was performed by using a static version of the computer program LARZWS (4) and triangular lateral load distribution. Global seismic response values were determined based on the inelastic seismic analysis of an equivalent SDOF system (7).

4. Response values for LINEAR method were calculated by knowing the earthquake characteristics (Tg), initial structural period (To) and linear elastic mode shapes.

The calculated global response values for each structure and each earthquake are presented in Table 3. and Table 4. which contains also an average quality of the drift estimates along the height (method/inelastic*100).

Table 3. Calculated Roof Displacements

4s_6m	To (sec)	Teff (sec)	MDOF(m)	N2 (m)	DBD (m)	LINEAR-	DR	TR
Bar-NS	0.431	0.608	0.246	0.246	0.199	0.243	1.012	0.621
Pet-NS	0.431	0.608	0.139	0.149	0.199	0.136	1.022	1.327
El Centro	0.431	0.608	0.158	0.140	0.199	0.191	0.827	1.105
7s_656	To	Teff	MDOF	N2	DBD	LINEAR	DR	TR
BarNS	0.666	0.939	0.251	0.393	0.295	0.400	0.627	0.960
PetNS	0.666	0.939	0.147	0.204	0.295	0.214	0.686	2.050
El Centro	0.666	0.939	0.308	0.282	0.295	0.307	1.003	1.707
10s_4m	To	Teff	MDOF	N2	DBD	LINEAR	DR	TR
BarNS	1.197	1.688	0.240	0.336	0.376	0.733	0.327	1.726
PetNS	1.197	1.688	0.152	0.209	0.376	0.223	0.681	3.685
El Centro	1.197	1.688	0.290	0.221	0.376	0.376	0.771	3.069

Table 4. Calculated Mean Drift Ratio, % (Roof displacement/Building height)

Structure	EQ	MDOF(%)	N2 (%)	Average%	DBD (%)	Average%	LINEAR%	Average%
4s_6m	Bar NS	1,93	1,55	80	1,56	84	1,91	100
	Pet NS	1,09	1,17	113	1,56	158	1,07	106
	EC_0,5	1,24	1,1	85	1,56	126	1,5	118
7s_6m	Bar NS	1,15	1,39	143	1,36	133	1,84	177
	Pet NS	0,68	0,71	117	1,36	214	0,98	151
	EC_0,5	1,42	1,19	100	1,36	109	1,41	110
10s_4m	Bar NS	0,78	1,09	155	1,22	194	2,38	308
	Pet NS	0,49	0,68	133	1,22	261	0,78	129
	EC_0,5	0,95	0,72	78	1,22	147	1,22	121

where: Average% is average value of drift ratio (estimated drift/inelastic drift)

Calculated deflected shapes (maximum story displacements normalised with respect to building height -Mean Drift Ratio) and Interstory Drift Ratios (maximum relative story displacement normalised with respect to story height), for 4s_6m, 7s_6m and 10s_4m structures and for the accelerogram El Centro N-S scaled to 0,5g are presented in Figures 3-8.

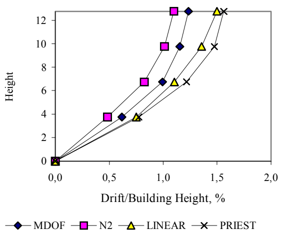

Figure 3. Maximum Drift Ratio, %

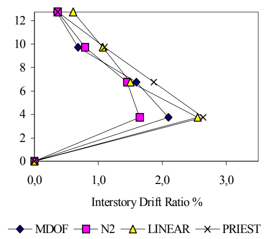

Figure 4. Interstory Drift Ratio for 4s_6m, %

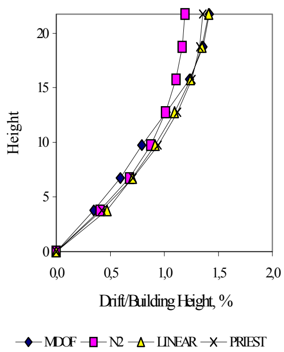

Figure 5 Maximum Drift Ratio, %

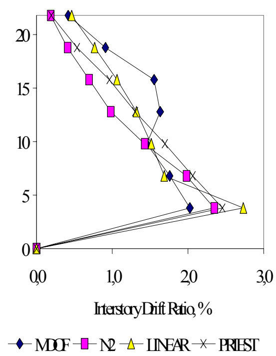

Figure 6 Interstory Drift Ratio for 7s_6m, %

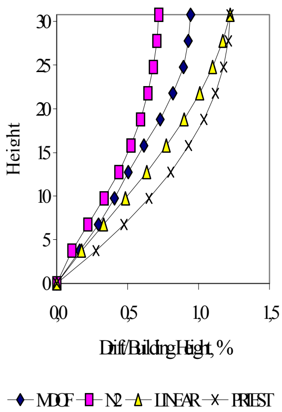

Figure 7. Maximum Drift Ratio, %

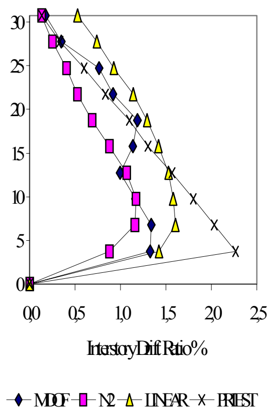

Figure 8. Interstory Drift Ratio for 10s_4m, %

CONCLUSION

For inelastic dynamic (MDOF) analysis and N2 method all structural data, framing, member sizes and material characteristics including M-φ relationship for the members, are required. DBD and LINEAR method require only the information which are normally obtained by design based on gravity loading. Necessary time to prepare input data is obviously on the side of DBD and LINEAR method. That is especially so, if we still have to make a distinction among various dispositions, framing sizes and structural systems (pre-design phase).

All observed input accelerations could be considered as extreme events and inelastic behaviour was noted in all structures. Calculated maximum Mean Drift Ratios (roof displacement/ building height = MDR) have amounted to 1.93% for 4s_6m, 1.42% for 7s_6m and 0.95% for 10s_4m structure (Table 4). Interstory Drift Ratios were concentrated in the lower half of the structure (except for a 10 storey structure which had also pronounced contribution of the higher modes) and had the maximum values of 2.92%, 2.02% and 1.34% for 4s_6m, 7s_6m and 10s_4m structures respectively. Values of calculated MDR for MDOF and N2 method were in a very close agreement. Average quality of the drift estimates (calculated/inelastic drift along the height) is 111,5% (N2), 142,8%(LINEAR) and 147,0%(DBD). Values of calculated MDR for DBD and LINEAR method were always an upper bound of the expected displacements during earthquakes.

Maximum displacements (Figures 3, 5 and 7) were by all methods better represented than drift distribution along the height (Figures 4, 6 and 8). Drift distribution along the height shows that N2, DBD and LINEAR model represent the structures responding predominantly in the first mode, which is one of their basic assumptions. The maximum story drifts calculated by LINEAR and DBD methods represented the upper bound of the expected ("exact") drifts. The N2 model represents better the exact values. While DBD and LINEAR method was always on the safe side, N2 method has shown greater sensitivity and oscillates on both sides.

REFERENCES

1. Sigmund, V., Comparison of linear and non-linear methods for evaluation of non-linear seismic drifts in R/C frames, IX ECEE, Moscow, 1989.

2. Sigmund, V., Comparison of two methods for modelling non-linear dynamic behaviour of r/c frame members, Research paper, University of Illinois, Urbana, 1989

3. Sigmund, V.et all, Methods for evaluation of seismic drift in R/C frames, Proceedings of the XI ECEE, Paris, 1998.

4. Lopez, R. and Sozen, M.A., A Guide to Data Preparation for LARZWD and LARZWS Computer Programs for Non-linear Analysis of Planar Reinforced Concrete Structures Incorporating Frame and Walls, University of Illinois, Urbana, 1988 and 1992.

5. The N2 method for the seismic damage analysis of RC buildings, Earthquake Engineering and Structural Dynamics, Vol. 25, pp. 31-46, 1996

6. Priestley, M.J.N., Displacement based approaches to rational limit states design of new structures, Proceedings of the XI ECEE-Invited lectures, Paris, 1998.

7. Charney, F.A, Non-linear Dynamic Time History Analysis of SDOF Systems, FEMA, 1996

8. Lepage, A. and Sozen, M.A., A Method for Drift-control in Earthquake-Resistant Design of RC Building Structures, University of Illinois, Urbana, 1997.