DISPLACEMENT-BASED SEISMIC DESIGN FOR BUILDINGS

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ABSTRACT

This paper presents a seismic design method based on displacements rather than force to include inelastic behavior of buildings. By using the substitute structure approach, a rational linear iteration method is proposed where a target displacement is specified and the required design force, member strength and stiffness are obtained. Examples that illustrate a few typical designs and results of parametric studies are also presented. The procedure shown here has been developed for multiple degree-of-freedom (MDOF) systems.

I. INTRODUCTION

According to the current force-based design procedure, the practice in seismic design is as follows. Given a design elastic acceleration response spectrum and an estimate of the structural period, the elastic response acceleration is determined. This is then reduced by a force reduction factor to obtain a modified design acceleration. Using the reduced design acceleration, the design force is then determined based on Newton's second law. A displacement check is usually made after the structural members satisfy the force requirement. However, in the displacement-based design (DBD) procedure, the engineer carries out seismic design by specifying a target displacement. Strength and stiffness are not the design variables in the prodedure, they are the end results. This is different from force-based design procedures. (1) It has no need to use a force reduction factor. (2) It directly addresses the inelastic nature of a structure during an earthquake. (3) It is common knowledge that both structural and nonstructural damage experienced during an earthquake are due primarily to lateral displacements. Therefore, displacement-based design procedure can provide a reliable indication of damage potential.

In recent years, two well-known displacement-based seismic retrofit methods have been used: the coefficient method proposed by FEMA273 (1997) and the capacity spectrum method proposed by ATC-40 (1996). For both methods, monotonically increasing lateral forces are applied to a nonlinear mathematical model of the building until the displacement of the control node at the roof of the building exceeds a target value (termed the target displacement). The lateral forces should be applied to the building using distributions or profiles that bound, albeit approximately, the likely distribution of inertial forces in the design earthquake. The target displacement is a mean estimate of the likely displacement of the yielding building in the design earthquake. Although coefficient method and capacity-spectrum method

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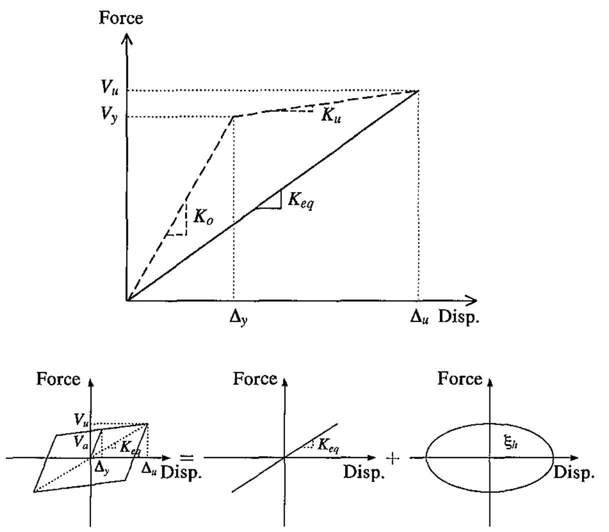


Fig. 1 Substitute structure approach

don't use force reduction factors and can directly address the inelastic nature of a structure during an earthquake, they are applied to seismic evaluation and rehabilitation of existing buildings. Also, the two methods need a nonlinear mathematical model to perform static nonlinear analyses (i.e. pushover analysis). Furthermore, the target displacement of the coefficient method is obtained from empirical formulas. It may result in less accuracy, especially for buildings with short periods.

In 1994, Kowalsky et al. (1994) developed a so-called direct displacement-based design procedure to design a single degree-of-freedom (SDOF) bridge pier. This method is applied to design new structures and only the static linear analysis is needed when formulating the design. Furthermore, although only the static linear analysis is needed when formulating the design, this method can show the ductility, ultimate displacement, yield displacement and yield moment of designed buildings. Therefore, according to the advantaged process (direct displacement-based design method), this paper proposed a seismic design procedure for multiple degree-of-freedom (MDOF) conventional buildings based on specified displacements.

II. THE SUBSTITUTE STRUCTURE APPROACH

The displacement-based seismic design procedure presented in this paper adopts the Substitute Structure Approach(Gulkan and Sozen,1974; Shibata and Sozen, 1976). The substitute structure approach is a procedure where an inelastic system is modeled as an equivalent elastic system (Fig. 1) that has properties of equivalent stiffness, K_{eq}; and equivalent damping, ξ_{eq}, to the inelastic system. Fig. 1 illustrates a bilinear approximation to the structural force-displacement response of a system. The substitute structure has the same ultimate force (V_u) and ultimate displacement (Δ_u) characteristics as the inelastic structure.

Since the equivalent properties of the substitute structure are elastic, a set of elastic displacement response spectra can be used for design. Therefore, the substitute structure approach allows an inelastic system to be designed and analysed by using elastic displacement response spectra.

III. DISPLACEMENT-BASED DESIGN PROCEDURE

The displacement-based design described here is an iterative process. The substitute structure approach is used to characterize the response by the equivalent stiffness to maximum (ultimate or target) roof displacement response (Δ_u). An equivalent damping (ξ_{eq}) is used based on the expected hysteretic characteristics and ductility level. The step by step procedure of the proposed displacement-based design procedure is as follows (Fig. 2).

1. Determine the target roof displacement (Δ_u) and assume a yield roof displacement (Δ_y) for the designed building. The initial ductility then can be calculated as $\mu=\Delta_u/\Delta_y$.

   The value of target roof displacement (i.e., maximum or ultimate displacement, Δ_u) depends on the design limit state. For example, buildings with a drift ratio of 1.5% would be reasonable. For a serviceability limit state, a lower drift ratio would be appropriate. At the first iteration cycle, the yield displacement may be assumed arbitrarily. Note that in this study, the value of Δ_y corresponds to the formed point of the first significant plastic hinge of the structure.

2. Determine the equivalent damping ($\xi_{eq}=\xi_I+\xi_h$).

   ξ_I=inherent damping ratio (it is assumed to be 2% for steel buildings and 5% for reinforced concrete, RC, buildings in this study), ξ_h=hysteretic damping ratio. The hysteretic damping ratio may be derived based on the energy dissipated during inelastic deformations (Jennings, 1968; Iwan and Gates, 1979). The relation used in this study is expressed in Eq. (1) and Fig. 3 which is based on Takeda hysteretic model with a bilinear stiffness ratio α (Kowalsky *et al.*, 1994).

$$\xi_h=\frac{1}{\pi}[1-(\frac{1-\alpha}{\mu}+\alpha)] \quad (1)$$

3. Convert the target roof displacement and mass of the MDOF building to the equivalent target

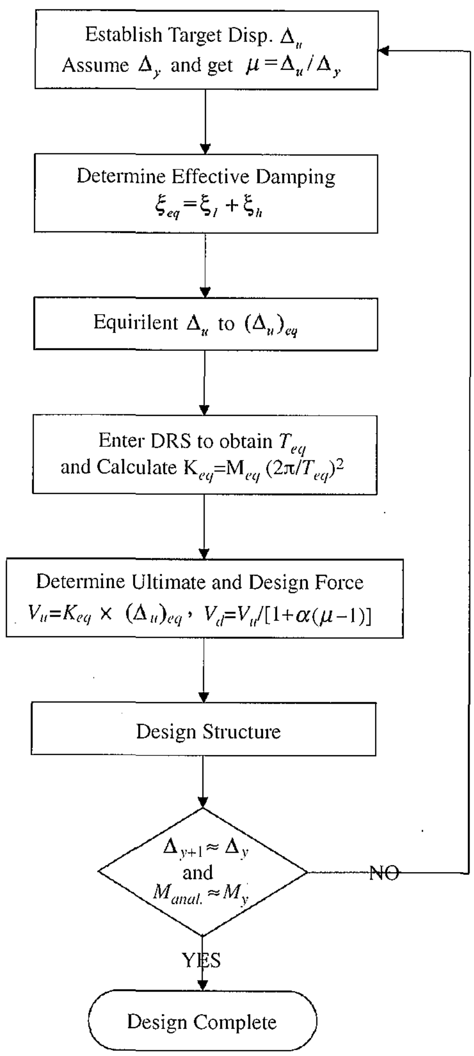


Fig. 2 Displacement-based design flowchart

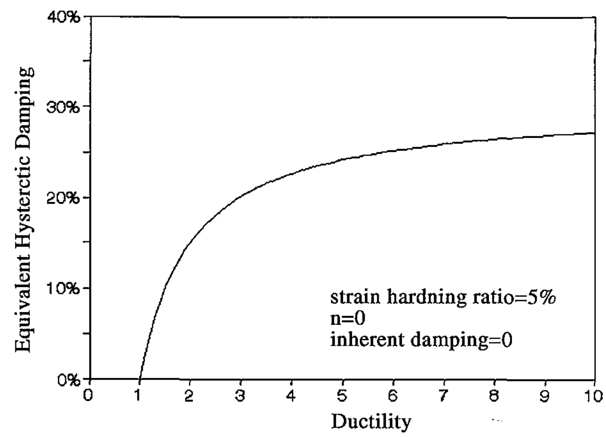


Fig. 3 Takeda Hyster. Damping model

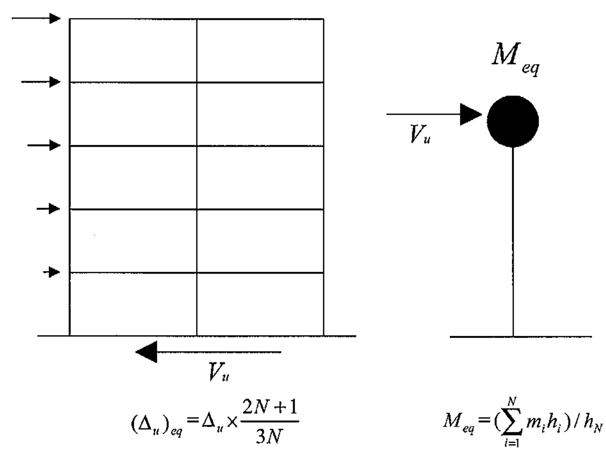


Fig. 4 Simulation of MDOF building by SDOF system

displacement $(\Delta_u)_{eq}$ and equivalent mass M_{eq} of the SDOF substitute structure.

In order to use the displacement response spectrum, the target roof displacement and mass of the MDOF building must be equivalent to a SDOF system (Fig. 4). Considering only the fundamental mode and assuming a uniform story height with uniform mass distribution and a triangular (i.e., linear) displacement shape, the equivalent target displacement for the equivalent SDOF system may be expressed as (Miranda, 1999)

$$(\Delta_u)_{eq}=\Delta_u\times\frac{2N+1}{3N} \tag{2}$$

where N is the number of stories in the building. In a similar manner, by consideration of mass participation in the fundamental mode, the equivalent mass for the equivalent SDOF system is (Tsai and Chang, 1999)

$$M_{eq}=(\sum_{i=1}^{N}m_ih_i)/h_N \tag{3}$$

where m_i is the mass of the i-th story and h_i is the height above the base to the i-th story.

4. Determine the equivalent period (T_{eq}) and equivalent stiffness (K_{eq}) of the SDOF substitute structure.

   With the value of $(\Delta_u)_{eq}$ and ξ_{eq}, the equivalent period of the SDOF system can be determined from the displacement response spectrum, as shown in Fig. 5. The equivalent stiffness at maximum response displacement can then be obtained by

$$K_{eq}=M_{eq}(\frac{2\pi}{T_{eq}})^2 \tag{4}$$

5. Obtain the ultimate force capacity (V_u) and the design force(V_d)

   Since the SDOF substitute structure is elastic, V_u can be calculated according to Fig. 1 as

$$V_u=K_{eq}\times(\Delta_u)_{eq} \tag{5}$$

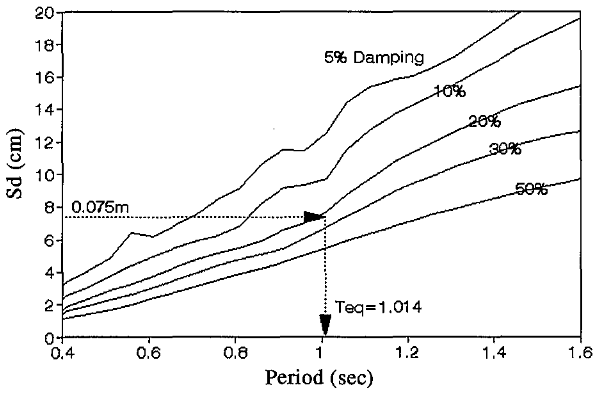


Fig. 5 Elastic Disp. response spectrum (TWA Soil Type II, PGA=0.33g)

Based on a bilinear force-displacement model (Fig. 1), the design force (i.e., yield force, V_y) can also be obtained as

$$V_d = V_y = \frac{V_u}{1+\alpha(\mu-1)} \tag{6}$$

6. Design structure based on V_d and Δ_y.

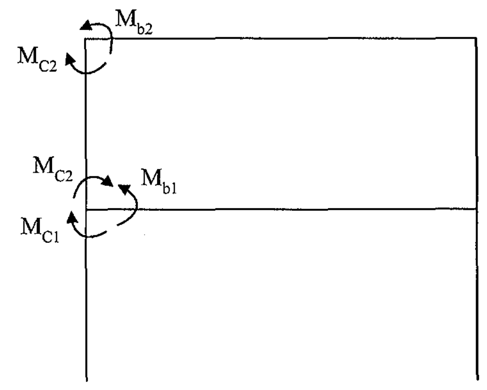
Because the design force (V_d or V_y) is still in the elastic range, the building can be designed and analysed in a way identical to the conventional approach. For example, the design force shall be distributed over the height of the building with Eq. (7). where w_x is the mass of the x-th story. Then, the structural members can be determined based on the design manuals (ACI, 1995; UBC, 1997) such that the building produces a roof displacement the same as Δ_y assumed in Step 1 under the distributively lateral force.

$$F_x = V_d \frac{w_x h_x}{\sum_{i=1}^{N} w_i h_i} \tag{7}$$

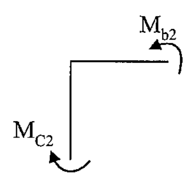
In this paper, the rough strong-column-and-weak-beam design criteria are adopted to decide the structural members (Fig. 6) assuming the cross-section area of beams first and then, according to the strong-column-and-weak-beam criteria shown in Fig. 6, the cross-section area of columns can be determined.

7. Check the end moment of each member and its yield moment

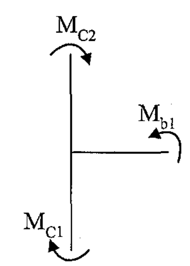
After Step 6, although the designed building will deflect Δ_y at the roof, as assumed in Step 1, under the distributively lateral force, it may not be a real yield point of the structure (Fig. 7). In order to make sure that the roof displacement obtained from Step 6 is really a yield point, the end moment of each member



$\sum M_C \geq 1.2 \sum M_b$ → Take $\sum M_C = \gamma \sum M_b$, $\gamma \geq 1.2$



$$\sum M_C = \gamma \sum M_b$$
$$M_{C2} = \gamma M_{b2}$$
$$S_{C2} \times F_y = \gamma S_{b2} \times F_y$$
$$S_{C2} = \gamma S_{b2} \tag{1}$$



$$M_{C1} + M_{C2} = \gamma M_{b1}$$
$$S_{C1} \times F_y + S_{C2} \times F_y = \gamma S_{b1} \times F_y$$
$$S_{C1} + S_{C2} = \gamma S_{b1}$$
$$S_{C1} = \gamma S_{b1} - S_{C2}$$
$$S_{C1} = \gamma (S_{b1} - S_{b2}) \tag{2}$$

Fig. 6 Determining of structural members

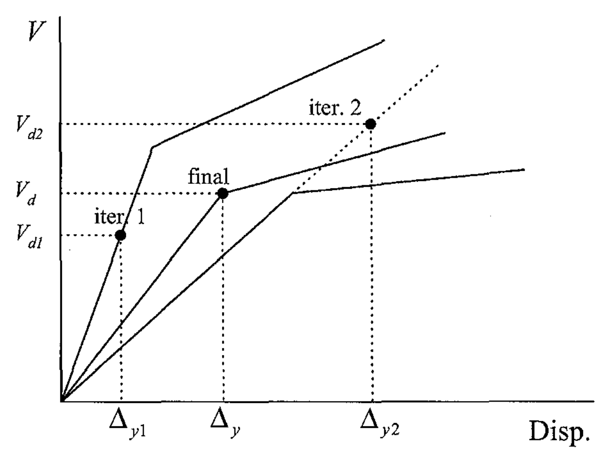


Fig. 7 Is it a real yield point?

must be checked with its yield moment capacity. Iteration will terminate, only if a member reaches yield moment in the building (i.e. the end moment of the member is equal or approximately equal to its

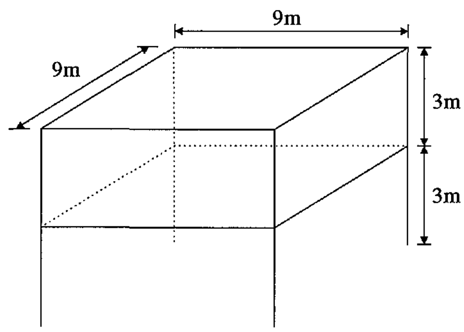


Fig. 8 Single-bay two story steel building

yield moment). Otherwise modify the yield displacement based on Eq. (8) and go to Step 1 until there is convergence, where M and M_y are the end moment and yield moment of the member that may yield first in the building.

$$\Delta_{y+1} = \Delta_y \frac{M_y}{M} \tag{8}$$

IV. EXAMPLE

1. Single-Bay Two Story Steel Building

The design parameters of a single-bay two story steel building (Fig. 8) are as follows. Structural dimensions: 9m×9m in plane and 3m in height for each story; Dead load and live load: 7.8 KN/m^2 (0.8 tonf/m^2) and 2.9 KN/m^2 (0.3 tonf/m^2) for each floor; Design spectrum: as shown in Fig. 5 which is the displacement response spectrum derived from an artificial earthquake consistent with the Taiwan design spectrum for Soil Type II with the peak ground acceleration (PGA) of 0.33g; Maximum story drift ratio: 1.5% under the design spectrum; Yield stress of steel material: F_y=247000 KN/m^2 (25200 tonf/m^2); Modulus of elasticity of steel: E=2.0E8 KN/m^2 (2.04E7 tonf/m^2).

1. In this example, a drift ratio of 1.5% is chosen, then Δ_u=1.5%×3m×2F=0.09m. Assume a yield displacement (Δ_y) of 0.03m, thus initial ductility $\mu = \Delta_u/\Delta_y$=0.09/0.03=3.0
2. For steel structures, an inherent damping of 0.02 and a strain hardening ratio of 0.05 are assumed. According to Eq. (1), the hysteretic damping ratio is ξ_h=0.2016. Therefore, the equivalent damping $\xi_{eq}=\xi_I+\xi_h$=0.02+0.2016=0.2216

$$\xi_h = \frac{1}{\pi}[1-(\frac{1-\alpha}{\mu}+\alpha)] = \frac{1}{\pi}[1-(\frac{1-0.05}{3}+0.05)] = 0.2016 \tag{1A}$$

3. Based on Eq. (2) and Eq. (3), the equivalent target displacement, $(\Delta_u)_{eq}$ and equivalent mass, M_{eq} of the SDOF substitute structure can be obtained as

$$(\Delta_u)_{eq} = \Delta_u \times \frac{2N+1}{3M} = 0.09m \times \frac{2\times 2+1}{3\times 2} = 0.075m \tag{2A}$$

$$M_{eq} = (\sum_{i=1}^{N} m_i h_i)/h_N = \frac{635.7}{9.81}(\frac{3+6}{6}) = 97.2 \tag{3A}$$

4. Enter the displacement response spectrum (Fig. 5) with the value for $(\Delta_u)_{eq}$=0.075m and ξ_{eq}=0.2216, then read down to find the equivalent period T_{eq}= 1.014sec.
5. $K_{eq} = M_{eq}(\frac{2\pi}{T_{eq}})^2 = 97.2 \times (\frac{2\pi}{1.014})^2 = 3732 KN/m$ (4A)

$$V_u = K_{eq} \times (\Delta_u)_{eq} = 380.4 \times 0.075 = 280\ KN \tag{5A}$$

$$V_d = V_y = \frac{V_u}{1+\alpha(\mu-1)} = \frac{280}{1+0.05(3-1)} = 254\ KN$$

$$\approx 0.20W \tag{6A}$$

6. Using Eq. (7), the lateral forces (F_x) are 84.8 KN at the 1st story and 169.5 KN at the 2nd story, respectively. Using a linear static analysis program, assuming $\Sigma M_c = 1.2 \Sigma M_b$ (Fig. 6) to account for the strong-column-and-weak-beam design and try the cross-section of beam until the building produces a roof displacement 0.03m under the lateral force (F_x). Results are H294×147 ×22×22 for beam of each floor and □ 233×233×22 ×22 for box column of each floor. All units are "mm".
7. In order to make sure that the roof displacement obtained from the above step is a really yield point, the end moment of each member must be checked with its yield moment capacity. After the checks for each member, a beam located on the first floor has a minimum capacity/demand ratio of 1.99 (M_y/M=200.1/100.5=1.99). Because the ratio is not close to 1.0, return to Step 1 of the procedure and reiterate by replacing Δ_y with $\Delta_{y+1}=\Delta_y(M_y/M)$= 0.03*(200.1/100.5)=0.06m until there is convergence.
8. The final results are: Δ_y=0.054m, μ=1.67, ξ_h= 0.121, ξ_{eq}=0.141, T_{eq}=0.890 sec, K_{eq}=4844 KN/m, V_u=363.4 KN, $V_d=V_y$=351.6 KN (≈0.277W), Beam: H268×134×22×22, Box column: □ 212×212×22 ×22, All units are "mm"., Fundamental period of the designed building T_o=0.766 sec, The first hinge of the building will be at the fixed-end of column located on the first story.

2. Three-Bay Five Story and Three-Bay Twenty Story Steel Buildings

The design parameters of both three-bay five story and three-bay twenty story steel building (Figs. 9a,b) are the same as the above single-bay two story example but the height of each story is 4m. Following exactly the same design procedure, Table 1 summarizes the designed results.

V. PARAMETRIC STUDY

In this section, parametric studies on a few designs using the proposed displacement-based approach are presented to illustrate the effects of design parameters on the design outcome.

1. Drift Ratio

Keep all the design parameters the same as before, except for the maximum design drift ratio, which are 0.5%, 1.0%, 1.5% and 2.0%, respectively. Tables 2a,b listed the selected design results which can be summarized as follows. (1) Because the maximum roof displacement increases with increasing drift ratio, the yield displacement increases in a non-linear fashion which cause the ductility to increase in an approximately linear fashion, and the equivalent damping ratio of the substitute structure also increases. (2) The fundamental period of a building designed by the proposed method increases as the design drift ratio increases, i.e. the building has smaller member size as the adopted drift ratio increases. It can be seen from Tables 2a,b that the results of the proposed displacement-based design procedure are strongly dependent on the chosen maximum drift ratios.

2. Effects of Story Height

The design parameters are the same as the previous three structures except that the height of each story is changed to 2m, 3m, 4m, 5m and 6m, respectively. Table 3 list the selected design results. (i) Although the maximum roof displacement increases with story height, the yield displacement also increases accordingly, thus that results in an inapparent variation of the ductility and the equivalent damping. (ii) In spite of longer fundamental period as the story height increases, the member size also becomes bigger as the story height increases.

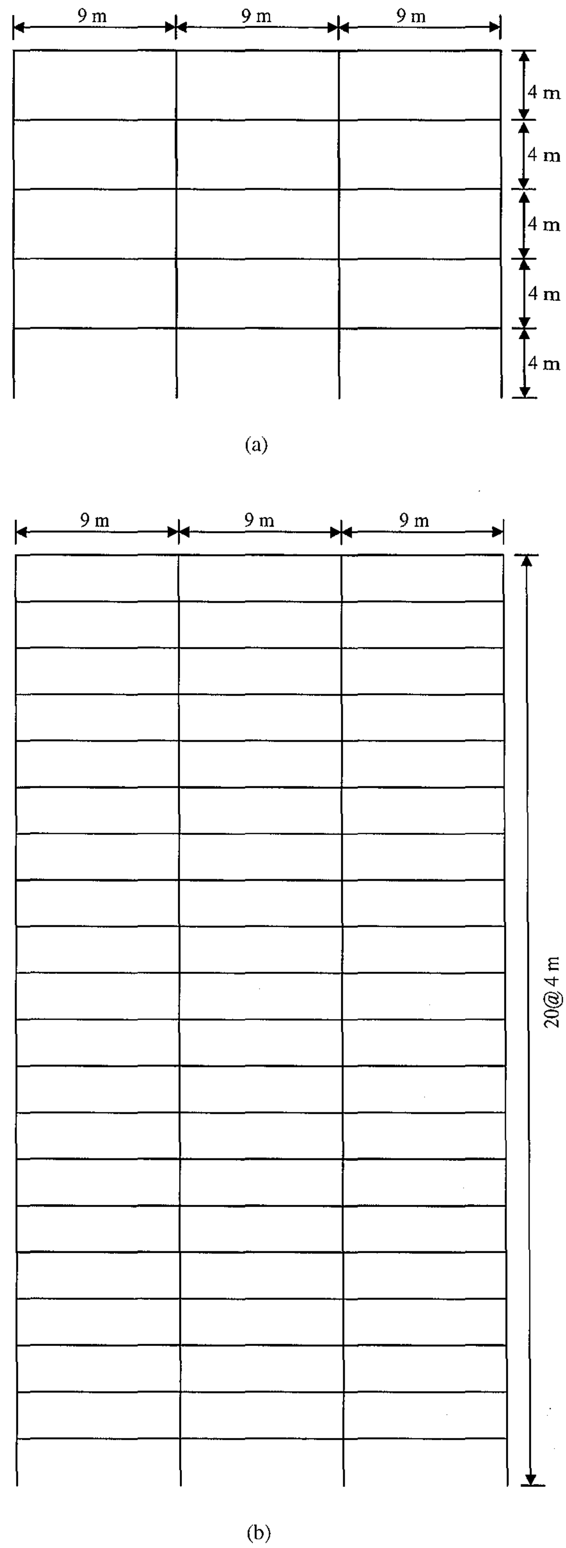


Fig. 9 (a) Three-bay five story building, (b) Three-bay twenty srory building

3. Effects of the Intensity of Design Spectrum

The design parameters are the same as the previous three structures except that the intensity of design displacement response spectrum is altered by PGA=0.2g, 0.33g, 0.5g, and 0.6g, respectively. Table 4 summarizes the selected design results. (i) It can be observed that the yield displacement clearly

Table 1 Results for three-bay five story and three-bay twenty story steel building

	Δ_u (m)	Δ_y (m)	μ	ξ_h	ξ_{eq}	T_{eq} (sec)	K_{eq} (KN/m)	V_u (KN)	V_d (KN)	T_o (sec)
5 Story	.20	.09	2.22	.1663	.1863	1.459	10820	1586	1495	1.168
20 Story	.50	.25	2.00	.1512	.1712	2.551	12380	4231	4029	2.238

Member size for three-bay 5 story building (mm)

Beam	1F-2F	H580×290×20×20	Column	1F-5F	□ 700×700×20×20
	3F-5F	H500×250×20×20			

Member size for three-bay 20 story building (mm)

Beam	1F-6F	H1000×500×25×25	13F-18F	H700×350×25×25
	7F-12F	H900×450×25×25	19F-20F	H600×300×25×25
Column	1F-6F	□ 1220×1220×40×40	13F-16F	□ 850×850×40×40
	7F-12F	□ 1100×1100×40×40	17F-20	□ 740×740×40×40

Note: The drift ratios adopted for 5 story and 20 story building are 1.0% and 0.625%, respectively.

Table 2a Effects of drift ratio for 1-Bay 2 story building

Drift Ratio	Δ_u (m)	Δ_y (m)	μ	ξ_{eq}	T_{eq} (sec)	K_{eq} (KN/m)	V_u (KN)	V_d (KN)	T_o (sec)
0.5%	0.03	0.03	1.00	0.02	0.276	50374	1260	1260	0.299
1.0%	0.06	0.046	1.30	0.091	0.603	10556	528	520	0.576
1.5%	0.09	0.054	1.67	0.141	0.890	4844	363	352	0.766
2.0%	0.12	0.059	2.03	0.174	1.095	3200	320	304	0.858
2.4%	0.145	0.064	2.27	0.189	1.249	2459	297	280	0.917

Table 2b Effects of drift ratio for 3-bay 20 story building

Drift Ratio	Δ_u (m)	Δ_y (m)	μ	ξ_{eq}	T_{eq} (sec)	K_{eq} (KN/m)	V_u (KN)	V_d (KN)	T_o (sec)
0.5%	0.4	0.25	1.60	0.1334	2.238	16088	4398	4269	2.144
0.63%	0.5	0.25	2.00	0.1712	2.551	12380	4231	4029	2.238
1.0%	0.8	0.29	2.76	0.2128	3.631	6113	3341	3072	2.748
1.5%	1.2	0.30	4.00	0.2468	4.409	4146	3399	2956	2.856
2.0%	1.6	0.32	5.00	0.2619	5.633	2540	2776	2314	3.266

Table 3 Effects of Story Height for 3-Bay 5 Story Building

Drift Ratio	Δ_u (m)	Δ_y (m)	μ	ξ_{eq}	T_{eq} (sec)	K_{eq} (KN/m)	V_u (KN)	V_d (KN)	T_o (sec)
3 m	0.15	0.072	2.08	0.1772	1.154	17285	1902	1804	0.943
4 m	0.20	0.090	2.22	0.1863	1.459	10820	1586	1495	1.168
5 m	0.25	0.114	2.19	0.1845	1.864	6627	1214	1147	1.520
6 m	0.30	0.136	2.21	0.1853	2.109	5177	1139	1074	1.699

Table 4 Effects of Design Spectrum Intensity for 1-Bay 2 Story Building

PGA	Δ_u (m)	Δ_y (m)	μ	ξ_{eq}	T_{eq} (sec)	K_{eq} (KN/m)	V_u (KN)	V_d (KN)	T_o (sec)
0.2g	0.09	0.067	1.34	0.097	1.089	3235	243	239	1.027
0.33g	0.09	0.054	1.67	0.141	0.890	4844	363	352	0.766
0.5g	0.09	0.044	2.05	0.175	0.711	7591	569	541	0.548
0.6g	0.09	0.040	2.25	0.188	0.640	9369	703	661	0.473

Table 5 Verification of DBD Using Dynamic Inelastic Analysis

		Design Values	Dynamic Nonlinear Analysis (ξ_l=2%)	
1-Bay 2 Story	Δ_u (m)	0.09	0.092	0.33g
Building	Δ_y (m)	0.054	0.055	0.11g
3-Bay 5 Story	Δ_u (m)	0.20	0.232	0.33g
Building	Δ_y (m)	0.09	0.088	0.143g
3-Bay 20 Story	Δ_u (m)	0.50	0.47	0.33g
Building	Δ_y (m)	0.25	0.24	0.113g

Table 6a Result for Force-Based Design

		Roof Disp. (m)	Drift Ratio 1F	Drift Ratio 2F	Base Shear (KN)
Static Anal.		0.018	0.26%	0.34%	.097W=124
Dynamic Nonlinear	0.33g	0.066	1.22%	1.13%	398
Anal.	0.172g yield	0.043	0.61%	0.81%	294

Table 6b Comparison DBD with Force-Based Design

	Force-Based	Displacement-Based Design			
Drift Ratio	0.34%	0.5%	1.0%	1.5%	2.0%
V	.097W	.991W	.409W	.277W	.239W
Period T_o (sec)	.520	.299	.576	.766	.858

decreases as the intensity of design spectrum increases, that causes the ductility and the equivalent damping to obviously increase. (ii) A stiffer structure will be obtained if the intensity of design spectrum increases.

VI. VERIFICATION OF DBD USING DYNAMIC INELASTIC ANALYSIS

In order to assess the performance of displacement-based design (DBD), dynamic inelastic time history analysis was carried out. For the previous design examples of single-bay two story, three-bay five story and three-bay twenty story buildings, a summation of comparison of the maximum displacement and yield displacement are made in Table 5. It is clear that the maximum displacement and yield displacement can be reliably predicted by the proposed method. Because the strong-column-and-weak-beam design criteria are adopted to decide the structural members in this paper, the failure mechanism of the example building is beam sway.

VII. COMPARISON WITH FORCE-BASED DESIGN

Following the regular equivalent static design

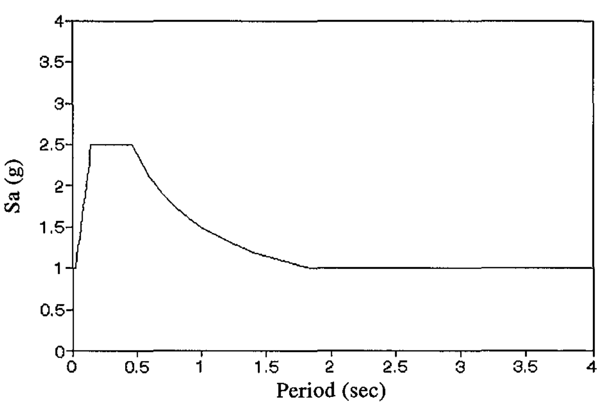


Fig. 10 Elastic Acc. response spectrum (TWA Soil Type II)

procedure of 1997 Uniform Building Code Code (UBC 1997) and using the Taiwan design spectrum of Soil Type II with the peak ground acceleration (PGA) of 0.33g (Fig. 10), structural members of the single-bay two story steel building are H340mm ×170mm×22mm×22mm for beam and □270mm ×270mm×22mm×22mm for column. The design base shear (V) is 0.097W and the fundamental period (T_o) is 0.520 sec. Table 6a lists the roof displacement, drift ratio and base shear of the building using static linear analysis and dynamic nonlinear analysis.

Comparing Table 6a with Table 5, especially for the case of nonlinear analysis under peak acceleration of 0.33g, it can be seen that the maximum roof displacement and maximum drift ratio obtained from force-based design procedure (0.066m, 1.22%) are smaller than that obtained from displacement-based design procedure (0.092m, 1.50%). The difference is mainly due to the fact that the UBC Code has a strict upper limit of drift ratio (0.42% for this building).

For the single-bay two story steel building, the design base shear based on displacement-based design (0.277W, Table 6b) is quite high in comparison with that based on force-based design (0.097W), but the fundamental period of the building designed with displacement-based design (0.766 sec) is longer than that designed with force-based design (0.520 sec). The difference is mostly due to the following: (1) UBC Code prescribes design base shear at the allowable stress level but the method of displacement-based design prescribes design base shear at the yield stress level. (2) Again, the UBC Code has a strict upper limit of drift ratio. For the single-bay two story steel building, if a drift ratio of 1.0% is chosen, the fundamental period of the building designed with displacement-based design (0.576 sec, Table 2a) will be close to that designed with force-based design (0.520 sec), if a drift ratio of 0.5% is chosen, the fundamental period of the building designed with displacement-based design (0.299 sec) will be shorter than that designed with force-based design (0.520 sec).

VIII. SUMMARY AND CONCLUSION

A procedure for displacement-based design was proposed. This procedure addresses the following problems with the force-based approach: (1) eliminates the need for the use of a force reduction factor and an estimate of the structural period; (2) addresses service and ultimate limits states using the same design procedure; and (3) provides a rational seismic design procedure that is compatible with the philosophy that structures are designed to undergo plastic deformation in a large earthquake while satisfying service criteria in small earthquakes.

It is concluded that by using the substitute structure approach, the ultimate displacement of buildings can be well estimated by the displacement-based design procedure. The only initial design parameters of displacement-based design are the target displacement and the story height. Strength and stiffness are a result of the design procedure and are dependent on the target displacement chosen. The most important parameter of displacement-based design is drift ratio. It influences obviously the yield displacement, ductility, equivalent damping and the fundamental period of the designed building.

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REFERENCE

1. ACI, 1995, *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*, ACI Committee 318, American Concrete Institute, Farmington Hills, Michigen.
2. AIJ, 1995, *Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake*, Architectural Institute of Japan, April 1995.
3. ATC-40, 1996, *Seismic Evaluation and Retrofit of Concrete Building*, Applied Technology Council, Redwood City, California.
4. Darragh, R., Cao, T, Cramer, C., Huang, M., and Shakal, A., 1994, "Processed CSMIP Strong-Motion Records from the Northridge, California Earthquake of January 17, 1994: Release No.1," Report No. OSMS 94-06B, California Strong Motion Instrumental Program, California Department of Conservation Division of Mines and Geology, Office of Strong Motion Studies, 801 K Street, Ms 13-35, Sacramento, California Feb. 1994.
5. FEMA 273. 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington, D.C.
6. Gulkan, P., and Sozen M., 1974, "Inelastic Response of Reinforced Concrete Structures to Earthquake Motions," *ACI Journal* 71, pp. 604-610.
7. Iwan, W. D., and Gates, N. C., 1979, "Estimating Earthquake Response of Simple Hysteretic Structures," *Journal of the Engineering Mechanics Division*, ASCE, Vol. 105, No. EM3, June, pp. 391-405.
8. Jennings, P.C., 1968, "Equivalent Viscous Damping for Yielding Structures," *Journal of the Engineering Mechanics Division*, ASCE, Vol. 94, No.EM1, February, pp. 103-116.
9. Kowalsky, M. J., Priestley, M. J. N., and MacRae, G. A., 1994, "Displacement-Based Design, A Methodology for Seismic Design Applied to SDOF Reinforced Concrete Structures", Structural System Research Project, University of California, San Diego La Jolla, California, SSRP-94/16.
10. Lee, G., Bruneau, M., Buckle, I. G., Chang, S., Flores, P., Goltz, J., O'Rourke, T., Shinozuka, M., Soong, T. T., and Taddeo, L., 1999, "The 921

Taiwan Earthquake", Mutidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, MCEER-99-SP003.
11. Miranda, E., 1999, "Approximate Seismic Lateral Deformation Demands in Multistory Buildings," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 4, Apri, pp. 417-425.
12. Shibata, A., and Sozen, M., 1976, "Substitute Structure Method for Seismic Design in Reinforced Concrete," *Journal of the Structural Division*, ASCE, Vol. 102, No. ST1, January, pp. 1-18.
13. Tsai, M. H., and Chang, K. C., 1999, "A Preliminary Study on Displacement-Based Design of RC Structures with Viscoelastic Dampers," *Proceedings of the Eighth KU-KAIST-NTU Tri-Lateral Seminar/ Workshop on Civil Engineeringtg*, August pp. 20-22, 1999, Taejon, Korea.
14. UBC 1997, *Uniform Building Code.*, 1997, International Conference of Building officials, Whittier, California.

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建築物耐震位移設計

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摘　要

本文提出一以位移為主之耐震設計方法（位移設計法），用以考慮建築物在地震下之非線性行為。其藉由使用「替代結構」，將原本複雜之結構非線性問題轉化為簡單易懂且較為設計者接受的線性迭代問題。經由非線性歷時分析結果顯示，該法可準確預測結構於地震下之非線性行為。本文內容包包括設計流程、設計例及參數研究等。

關鍵詞：位移設計，替代結構，目標位移，等效阻尼。