A Non-iterative Procedure of Direct Displacement–Based Design Method for Portal Steel Bridges

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ABSTRACT

This paper presents a non-iterative procedure of direct displacement-based seismic design method for designing a portal steel bridge based on the substitute structure approach. In general, when the direct displacement-based design method is used, the cross-sections of structural members can be determined based on the pre-selected target displacement and ductility ratio. However, this usually requires a repeatedly iterative procedure and inevitably results in inefficiency when number of iterative cycles is large. For avoiding such shortcoming, this paper presents a new procedure by combining the yielding property with the stiffness property of the designed members. The proposed procedure directly obtains cross-sections of members via the pre-chosen design parameters without the need of iterations. In addition to the illustrative examples, the static nonlinear analyses (pushover analyses) and dynamic nonlinear time-history analysis are also adopted to verify the proposed procedure. The results of the nonlinear analyses show very good agreement with these of the proposed procedure.

Keywords: non-iterative direct displacement-based seismic design, portal steel bridges, substitute structure approach.

非迭代性直接位移設計法應用於門架式鋼橋之設計

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摘 要

本文提出一個非迭代性的替代結構（substitute structure approach）直接位移設計法並應用於一個門架式的鋼橋的設計。傳統上，直接位移設計法應用在非線性結構時，結構桿件之橫斷面(cross-sections)可以由預先選擇的目標位移(target displacement) 與延展比(ductility ratio)推導而得。然而，上述過程經常必須進行反覆的迭代過程，因而經常造成收斂過程中，相當沒有效率的反覆迭代過程。為了避免這項缺點，本文提出一個非迭代性的替代結構直接位移設計法，並應用於鋼橋的設計中。上述方法可以直接由預先選擇的設計參數計算橫斷面(cross-sections)，且不用任何疊代過程即可完成。本文並採用非線性推覆分析(pushover analyses)與非線性歷時分析來檢驗本文之方法，其結果顯示兩者結果相當吻合。

關鍵詞：非迭代直接位移設計法、門架式的鋼橋、替代結構法

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1. INTRODUCTION

Many seismic events during the past decade, such as those occurred in California(1994), Kobe, Japan(1995),Turkey(1999), Taiwan(1999), and Central-Western India (2001) have demonstrated the destructive power of earthquakes. This has drawn the attention of so many investigators, it is not surprising that the literature devoted to this subject is very extensive. Nowadays, there are two well-known displacement-based design methods. One is the displacement coefficient method which was incorporated in the FEMA-273 [1], and the other is the capacity spectrum method adopted in the ATC-40 [2]. Both methods belong to the nonlinear static analysis procedures and are applied to evaluation and rehabilitation of existing buildings. Recently, a so-called "direct" displacement-based seismic design procedure was developed which is different from the two methods. It is applied to the design of new constructions and only the static linear analysis is needed in such procedure.

The source of direct displacement-based seismic design methodology can be traced back three decades. In the 1970s, Gulkan and Sozen [3] proposed an approach of substitute structures by using the equivalent linear systems associated with equivalent stiffness and equivalent viscous damping to predict the responses of nonlinear structures. In 1994, Kowalsky et al. [4] proposed a direct displacement-based design procedure for designing the single degree-of-freedom (SDOF) bridge piers by using the approach of substitute structures (i.e., equivalent linear systems) [3]. This procedure is carried out by initiating the design from a specified target displacement. The strength and stiffness become end-products of the design. For simple SDOF structures, the process has been successfully applied to bridge piers over a possible range of design. The concept was later adopted by Calvi and Kingsley [5], Priestley et al. [6] and Lin et al. [7] for designing SDOF or multi-degree of freedom (MDOF) bridges and buildings. Moreover, Wallace [8]; Sasani and Anderson [9]; Bachman and Dazio [10]; Kowalsky [11] further extended the concept to buildings with wall systems.

In order to design the members of a nonlinear structure, the direct displacement-based seismic design method in general requires an iterative procedure no matter whether the substitute linear structure or inelastic design spectra is used. This would sometimes result in inefficiency if too many iterative cycles are required. In order to keep away from this disadvantage, this paper proposed a non-iterative direct displacement-based design procedure for an MDOF portal steel bridge based on the substitute structure approach. By combining the yielding property with the stiffness property of the designed structures, a non-iterative direct displacement-based procedure is presented in this paper.

2. FUNDAMENTALS OF THE NON-ITERATIVE PROCEDURE

For portal structures (Fig.1) considering the stiffness of columns and beams, the end moments of each member can be determined based on the method of slope deflection [12] as

$$M_{ab} = \frac{2EI_c}{l_c}(\theta_b - 3\frac{\Delta}{l_c}) \tag{1}$$

$$M_{ba} = \frac{2EI_c}{l_c}(2\theta_b - 3\frac{\Delta}{l_c}) \tag{2}$$

$$M_{bc} = \frac{2EI_b}{l_b}(3\theta_b) \tag{3}$$

where a, b, c, and d denote nodes. M and θ are the end moment and the end rotation of members, respectively. I_c, l_c and Δ represent the moment of inertia, the height and the lateral displacement of columns while I_b and l_b represent the moment of inertia and the length of beam, respectively. E is elastic modulus of material. According to force equilibrium, the following equations can be established.

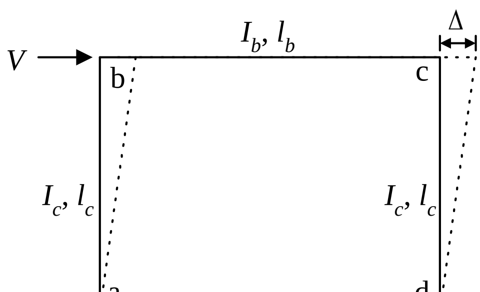


Fig.1 One Story Portal Structure

$$\sum M_b = 0 \quad M_{ba} + M_{bc} = 0 \tag{4}$$

$$\sum F_x = 0 \quad V + 2\frac{M_{ab} + M_{ba}}{l_c} = 0 \tag{5}$$

Solving Eqs.(1) ~ (5) comes out

$$\Delta = \frac{2l_b I_c + 3l_c I_b}{l_b I_c + 6l_c I_b} \frac{Vl_c^3}{12EI_c} \tag{6}$$

$$\theta_b = \frac{3l_b I_c}{l_b I_c + 6l_c I_b} \frac{Vl_c^2}{12EI_c} \tag{7}$$

$$M_{ab} = -\frac{l_b I_c + 3l_c I_b}{2(l_b I_c + 6l_c I_b)} Vl_c \tag{8}$$

$$M_{ba} = -\frac{3l_c I_b}{2(l_b I_c + 6l_c I_b)} Vl_c \tag{9}$$

$$M_{bc} = \frac{3l_c I_b}{2(l_b I_c + 6l_c I_b)} Vl_c \tag{10}$$

For such frame as shown in Fig.1, the design moment of the beam (M_{beam}) is $|M_{bc}|$ whereas that of both columns (M_{col}) are the maximum value of $|M_{ab}|$ and $|M_{ba}|$. Because node a is a fixed end, the design moment of the column is M_{ab}. That is

$$M_{col} = |M_{ab}| = \frac{l_b I_c + 3l_c I_b}{2(l_b I_c + 6l_c I_b)} Vl_c \tag{11}$$

$$M_{beam} = |M_{bc}| = \frac{3l_c I_b}{2(l_b I_c + 6l_c I_b)} Vl_c \tag{12}$$

Rearranging Eq.(6), the moment of inertial of the beam can be acquired in terms of I_c as follows.

$$I_b = \frac{2l_b I_c Vl_c^3 - 12E\Delta l_b I_c^2}{72EI_c\Delta - 3l_c^3 V} \frac{1}{l_c} \tag{13}$$

Furthermore, the yield moments for the columns ($M_{y,c}$) and beam ($M_{y,b}$) are shown as below.

$$M_{y,c} = S_c F_y = \frac{I_c}{d_c/2} F_y \tag{14}$$

$$M_{y,b} = S_b F_y = \frac{I_b}{d_b/2} F_y \tag{15}$$

where S_c and S_b are the section moduli of the columns and beam, respectively. d_c and d_b represent the depth of corresponding cross-sections. F_y is the yield stress of steel material. If the first yielding is occurred in the column, the moment of inertia of this column (I_c) can be obtained by substituting Eqs.(13 and (14) into Eq.(11) as.

$$I_c = \frac{V_y l_c^3 d_c}{12(l_c^2 F_y - E\Delta_y d_c)} \tag{16}$$

where V_y and Δ_y are the yield lateral force and yield lateral displacement of this frame. Otherwise, if the first plastic hinge is developed in the beam, the moment of inertia of the column (I_c) can be obtained by combining Eqs.(13), (15) and (12) as.

$$I_c = \frac{V_y l_c^3 d_b}{4(l_b l_c F_y - 6E\Delta_y d_b)} \tag{17}$$

For a specified d_c with known V_y and Δ_y, it can be shown that I_c can be obtained from Eq.(16) or Eq.(17) thus I_b can be consequently computed from Eq.(13). For instance, if a circular hollowed section is adopted for the column (Fig.2a), its moment of inertia is I_c = $\frac{\pi}{64}[d_c^4 - (d_c - 2t_c)^4]$ and the thickness (tc) of this circular section can be as

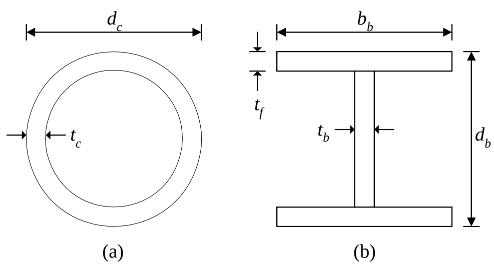


Fig.2 Cross-sections of Circular Hollowed Column (a) and I-Beam (b)

$$t_c = \frac{1}{2}[d_c - \sqrt[4]{d_c^4 - \frac{64I_c}{\pi}}] \tag{18}$$

For the beam, the dimensions of its flange and web can be obtained from the following equations providing the "I" cross-section is chosen.

$$I_b = \frac{1}{12}[b_b d_b^3 - (b_b - t_w)(d_b - 2t_f)^3] \tag{19}$$

where b_b and t_w are the width and thickness of flange, respectively. t_f is the thickness of web. It can be seen from Eqs.(13) and (16) that the section properties of both column and beam depend on V_y,

Δ_y, d_c, E, F_y, l_b and l_c. Especially, they are cubic functions of the column length (l_c).

3. EQUIVALENT LINEAR SYSTEMS

The force-displacement relationship of an idealized bilinear portal structure can be illustrated in Fig.3. where K, α, Vy, Vu, Δ_y, Δ_u and μ are the elastic stiffness, post-yield stiffness ratio, yield force, maximum force, yield displacement, target (maximum) displacement and ductility ratio ($\mu = \Delta_u / \Delta_y$), respectively. The basic concept of the approach [3]; is to model an inelastic system by using an equivalent linear system. This implies that the maximum force and maximum displacement response of a nonlinear system can be approximately estimated by an equivalent linear system associated with equivalent secant stiffness K_{eq}, natural period Teq and equivalent viscous damping ξ_{eq} [13-15].

$$T_{eq} = T_n \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}} \tag{20}$$

$$\xi_{eq} = \xi_0 + \xi_h \quad ; \quad \xi_h = \frac{1}{\pi}[1 - (\frac{1-\alpha}{\mu} + \alpha)] \tag{21}$$

where T_n = elastic natural vibration period of the bilinear system; ξ_0 = inherent damping of the bilinear system vibrating within its linearly elastic range; ξ_h = equivalent hysteretic damping. Notice that the equivalent viscous damping can be derived by considering the effect of ductility on damping and its value is related to the hysteretic energy absorbed. The expression of Eq.(21) used in this paper is based on the Takeda hysteretic model [4,16].

Since the properties of the substitute structure are elastic and linear, the elastic displacement response spectra with various damping can then be used for design. Therefore, an inelastic system can be designed by using static linear analysis and elastic displacement response spectra.

4. DESIGN PROCEDURE OF THE NON-ITERATIVE METHOD

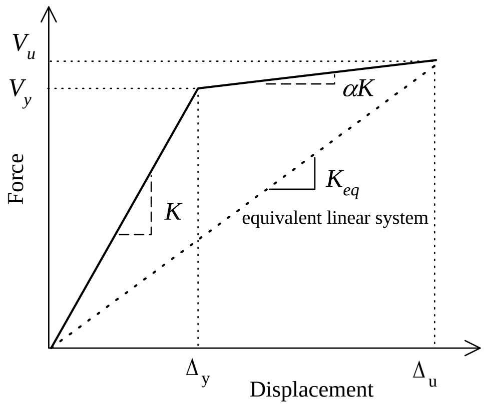


Fig.3 Force-Displacement Relationship of Idealized Bilinear Systems and Equivalent Linear System

By using the equivalent linear systems, a non-iterative direct displacement design procedure for portal steel structures is shown in the following steps (Fig.4).

1. Choose a target displacement (Δ_u) and a ductility ratio (μ) for designed structures under design earthquakes.
2. Then, the yield displacement (Δ_y) can be calculated as $\Delta_y = \Delta_u / \mu$.
3. Estimate the equivalent viscous damping (ξ_{eq}) based on the design ductility from Eq.(23), i.e. $\xi_{eq} = \xi_0 + \xi_h$.

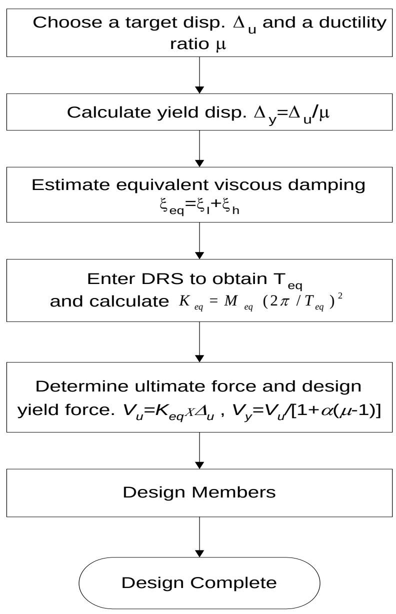


Fig.4 Flowchart of the Non-Iterative Direct Displacement-Based Design Procedure Using Equivalent Linear Systems

4. Enter the elastic displacement design spectrum with the known values of Δ_u and ξ_{eq} to read Teq as shown in Fig.5. Then, the equivalent stiffness (Keq) of the substitute structure can be determined according to the relationship between mass and stiffness.

$$K_{eq} = M(\frac{2\pi}{T_{eq}})^2 \quad (22)$$

where M is the mass of the system.

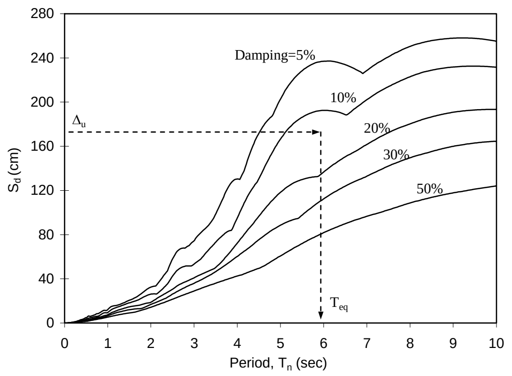


Fig.5 Elastic Disp. Response Spectra for Soil Type II of TWA

5. Obtain the ultimate force (Vu), design yield force (Vy). Since the substitute structure is elastic, Vu can be calculated referring to Fig.3, as shown as

$$V_u = K_{eq} \times \Delta_u \quad (23)$$

Based on the bilinear force-displacement model of Fig.3, the design yield force (Vy) of the nonlinear structure can also be obtained as

$$V_y = \frac{V_u}{1+\alpha(\mu-1)} \quad (24)$$

6. Design the structure.

For a chosen depth of the columns (d_c), its moment of inertia (I_c) can be obtained based on Eqs.(16) or (17). Then, if a circular hollowed section is used, the thickness (t) of this column can consequently be determined by using Eq.(18). As for the beam, its design moment of inertia and dimensions can, respectively, be obtained from Eq.(13) and (19) for an "I" section. It needs to be emphasized that Eq.(16) is applied to the case that a plastic hinge would foremost develop at the ends of columns. If the beam is expected to yield first, Eq.(17) should be employed instead of Eq.(16).

According to the procedure mentioned above, the cross-section dimensions of the designed portal structures can be easily determined without any iteration when the target displacement (Δ_u) and the ductility ratio (μ) of Step 1 are chosen.

5. DESIGN EXAMPLES

The proposed non-iterative procedure for direct displacement-based design of steel portal structures using equivalent linear systems is further illustrated with the following two examples. The elastic fundamental period of the first example falls in the velocity-sensitive region of the design spectrum, and that of the second example falls in the acceleration-sensitive region.

Example 1

The geometrical property of this example is part of a steel bridge (Fig.6). The superstructure is supported by uniform bents with a vertical length of 9m and uniform spacing of 40m. Each bent consists of a portal frame. The span of the beam is 10m. Besides, the types of cross-sections for these two columns and beam are circular hollowed and I-sections, respectively. For the transverse ground motion, the viaduct can be idealized as a simply frame system with lumped mass of M= 767 ton. The design spectrum is shown in Fig.5 which is the displacement response spectrum derived from an artificial earthquake (Fig.7a) is accordance with the Taiwan design spectrum for Soil Type II (Fig.7b) with the peak ground acceleration (PGA) of 0.33g. The yield stress of steel material (Fy) is 250000 KN/m^2 and the modulus of elasticity of steel (E) is 2.0×10^8 KN/m^2. The design procedure is demonstrated as follows.

1. For a drift ratio of 3% and a ductility ratio of 4, Δ_u = 3%*9m=0.27m.
2. The yield displacement can therefore be calculated as $\Delta_y = \Delta_u / \mu$ =0.27/4= 0.0675 m.
3. For α =5% and μ =4, Eq. (21) yields ξ_h =22.68%. The equivalent viscous damping for the substitute structure is $\xi_{eq} = \xi_0 + \xi_h$ =2%+22.68%=24.68%.
4. Applying the displacement design spectrum (Fig.8) of elastic systems

with Δ_u =0.27 m and ξ_{eq} =24.68%, gives Teq=2.462 sec. Then, the equivalent stiffness (Keq) of the substitute structure is $K_{eq} = M(\frac{2\pi}{T_{eq}})^2 = 767(\frac{2\pi}{2.462})^2 = 4991\ {}^{KN}/_{m}$.

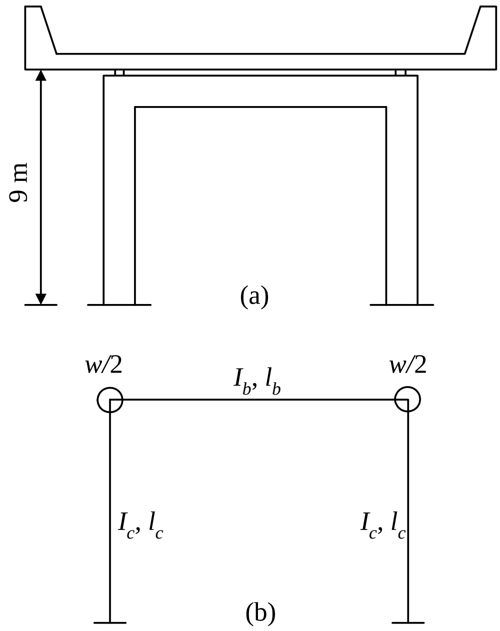


Fig.6 (a) Bridge Examples and (b) Idealized Systems.

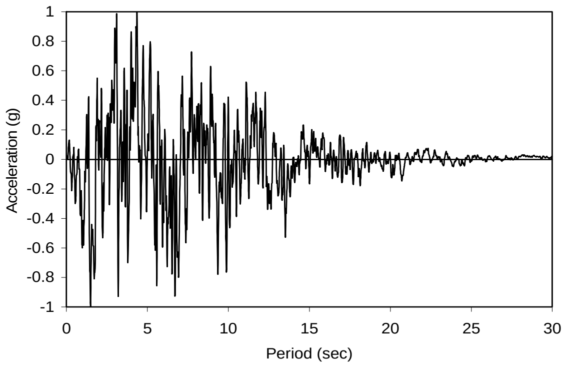


Fig.7a Artificial Earthquake for Soil Type II of TWA Building Code

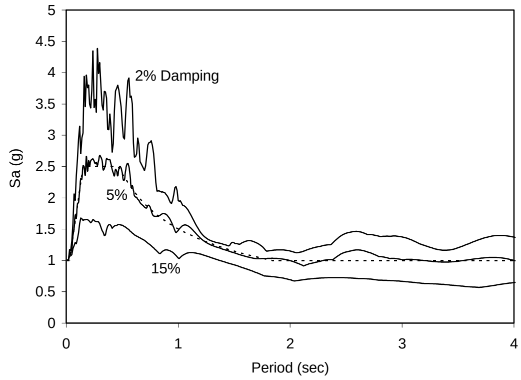


Fig.7b Elastic Acc. Response Spectra for Soil Type II of TWA

5. The ultimate force and design yield force are calculated as.

$$V_u = K_{eq} \times \Delta_u = 4991 \times 0.27 = 1348 \text{ KN} \quad (23a)$$

$$V_y = \frac{V_u}{1+\alpha(\mu-1)} = \frac{1350}{1+0.05(4-1)} = 1172 \text{ KN} \quad (24a)$$

6. It is shown in Fig.9 the relationship between values of I and dc for this example. The two I curves are constructed with Eqs (13) and (16), respectively. As indicated in this figure, a section depth of column of 0.7m is chosen, and the design moment of inertial of the columns is calculated based on Eq.(16) as

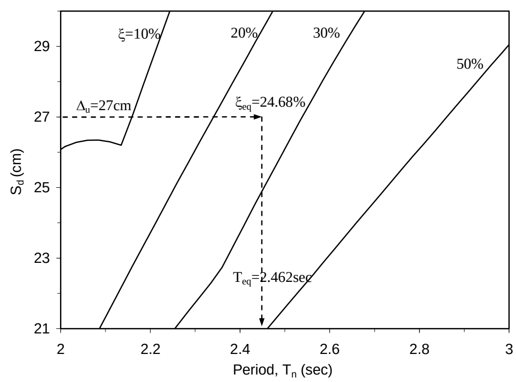


Fig.8 Elastic Disp. Response Spectrum for Soil Type II of TWA Building Code. PGA=0.33g

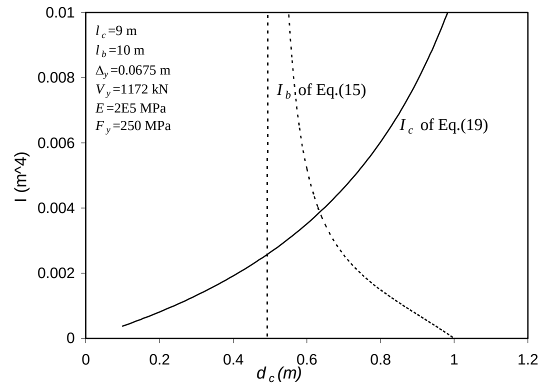


Fig.9 Relationship between (lc, Ib) and dc for example 1.

$$I_c = \frac{V_y l_c^3 d_c}{12(l_c^2 F_y - E\Delta_y d_c)} =$$

$$\frac{1172\times 9^3 \times 0.7}{12(9^2 \times 250000 - 2\times 10^8 \times 0.0675\times 0.7)} = 0.00461 \qquad (16a)$$

As a result, the thickness of the two circular columns can be estimated from Eq.(18).

$$t_c = \frac{1}{2}[d_c - \sqrt[4]{d_c^{\,4} - \frac{64I_c}{\pi}}] =$$

$$\frac{1}{2}[0.7 - \sqrt[4]{0.7^4 - \frac{64\times 0.00461}{\pi}}] = 41\,\text{mm} \qquad (18a)$$

Similarly, on the basis of Eq.(13), the design moment of inertial of the beam is

$$I_b = \frac{2l_b I_c V l_c^3 - 12E\Delta l_b I_c^2}{72EI_c\Delta - 3l_c^3 V}\frac{1}{l_c}$$

$$= \frac{2\times10\times0.00461\times1172\times9^3 - 12\times2\times10^8\times0.0675\times10\times0.00461^2}{72\times2\times10^8\times0.00461\times0.0675 - 3\times9^3\times1172}\frac{1}{9}$$

$=0.00256\, m^4$ (13a)

Next, the dimension of this I-beam is determined from Eq.(19) as 600mm×400mm× 33mm×33mm.

$$I_b = \frac{1}{12}[b_b d_b^3 - (b_b - t_w)(d_b - 2t_f)^3]0.00256$$

$$= \frac{1}{12}[0.40\times 0.6^3 - (0.40 - 0.033)(0.6 - 2\times 0.033)^3] \qquad (19a)$$

The design outcomes can be summarized in the following: Δ_u =0.27m, μ =4, Δ_y =0.0675m, Teq=2.426sec, Tn=1.301sec, ξ_{eq} =24.68%, V_u =1348 KN, V_y =1172 KN section for the columns: ◎700mm×41mm and that for the beam: W600mm×400mm×33mm×33mm. From the above illustration, it is now clear that the proposed direct displacement-based design procedure does not need call for iteration schemes. Once the target displacement and the ductility ratio have been chosen, the cross-section dimensions of the designed example can be easily determined.

Example 2

The structural properties of the second example are the same as those of Example 1 except that the height of the bents (l_c) is 4 m and the span of beam (l_b) is 8m. The elastic fundamental period of this system lies in the acceleration-sensitive region of the design spectrum. A drift ratio of 2.5% and a ductility ratio of 6 are chosen for this system. Following the proposed design procedure, it is obtained that Δ_u =0.1m, Δ_y =0.0167m, Teq=1.223 sec, Tn=0.558 sec, ξ_{eq} =27.2%, K_{eq} =20225 $^{KN}\!/_m$, V_u =2023 KN, V_y =1618 KN, d_c =0.65m, I_c =0.00306 m^4 , t_c =33mm, I_b =0.00122 m^4 , b_d =0.6m, b_b =0.3m and $t_b = t_f$ =19mm. Again, the proposed method does not need any iteration scheme.

6. VERIFICATIONS BY NONLINEAR ANALYSIS

To assess the accuracy of the proposed non-iterative procedure for direct displacement-based design, the static nonlinear (pushover) analysis and the dynamic inelastic time-history analysis are carried out in this section by using the Drain-2D+ program, a static and dynamic analysis program for inelastic 2D structures [17].

Static nonlinear analysis

Figs.(10a) and (10b) show the pushover curve (top displacement vs. lateral force) for both Example 1 and Example 2. The input data of Drain-2D+ for static nonlinear analyses of Example 1 are given in Appendix. Some important modeling parameters are illustrated as follows: (a). The post-yield stiffness ratios for beams and columns are 5%; (b). The designated cross-sectional yielding criterion for beams is “beam type without P-M (axial force-moment) interaction” and that for columns is “steel I-beam type with P-M interaction” [17]. It can be seen from Figs.(10a) and (10b) that the top displacements of designed structures under the static nonlinear analyses are very close to the designed values obtained from the proposed displacement-based design procedure.

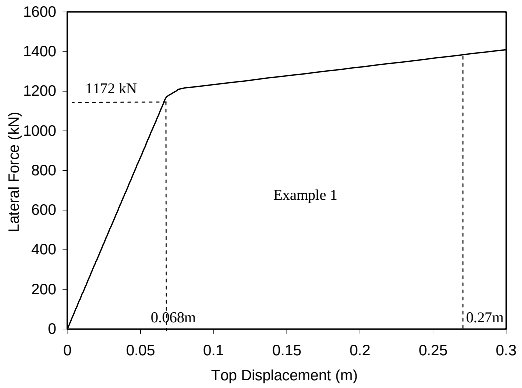


Fig.10a Verification using Static Nonlinear, Pushover, Analysis for Example 1

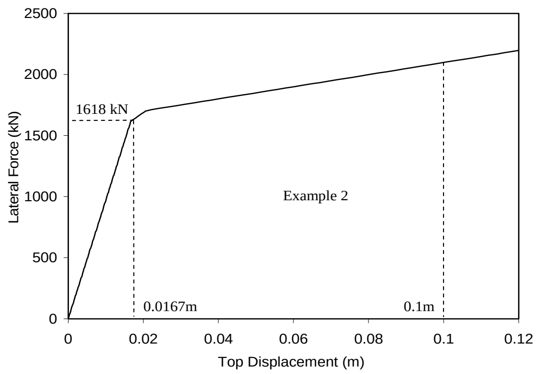


Fig.10b Verification using Static Nonlinear, Pushover, Analysis for Example 2

Dynamic nonlinear time-history analysis

Drain 2D+ is adopted to carry out the dynamic inelastic time-history analyses. An inherent damping ratio of 2% is used in the analysis. For the two design examples, Table 1 concludes a list of comparisons of the target displacements and yield displacements under three artificial earthquakes (Fig.11) which are generated from the Taiwan design spectrum for Soil Type II (Fig.5). Selected history responses of top displacements for the two examples are shown in Fig.12. It can be seen that the target displacements and the yield displacements of the nonlinear structures are reasonably captured by the proposed non-iterative displacement-based procedure.

7. CONCLUSIONS

The direct displacement-based design is one of the available methods to achieve the design objectives. The required initial design parameters of the method are the target displacement and the ductility ratio of the designed structures. Strength and stiffness are outputs of the design procedure and are dependent on the chosen design parameters. In order to avoid the use of iterative schemes, this work proposes a non-iterative direct displacement-based design procedure, which merges the properties of yielding and elastic stiffness of the designed columns to directly obtain their cross-sections and yield displacements from the chosen target displacement and ductility ratio of the steel portal structures. This method is simple, efficient and straightforward. From the dynamic nonlinear time-history analyses, it is concluded that the objectives of efficiently designing steel portal structures can be easily achieved through the use of the presented non-iterative method.

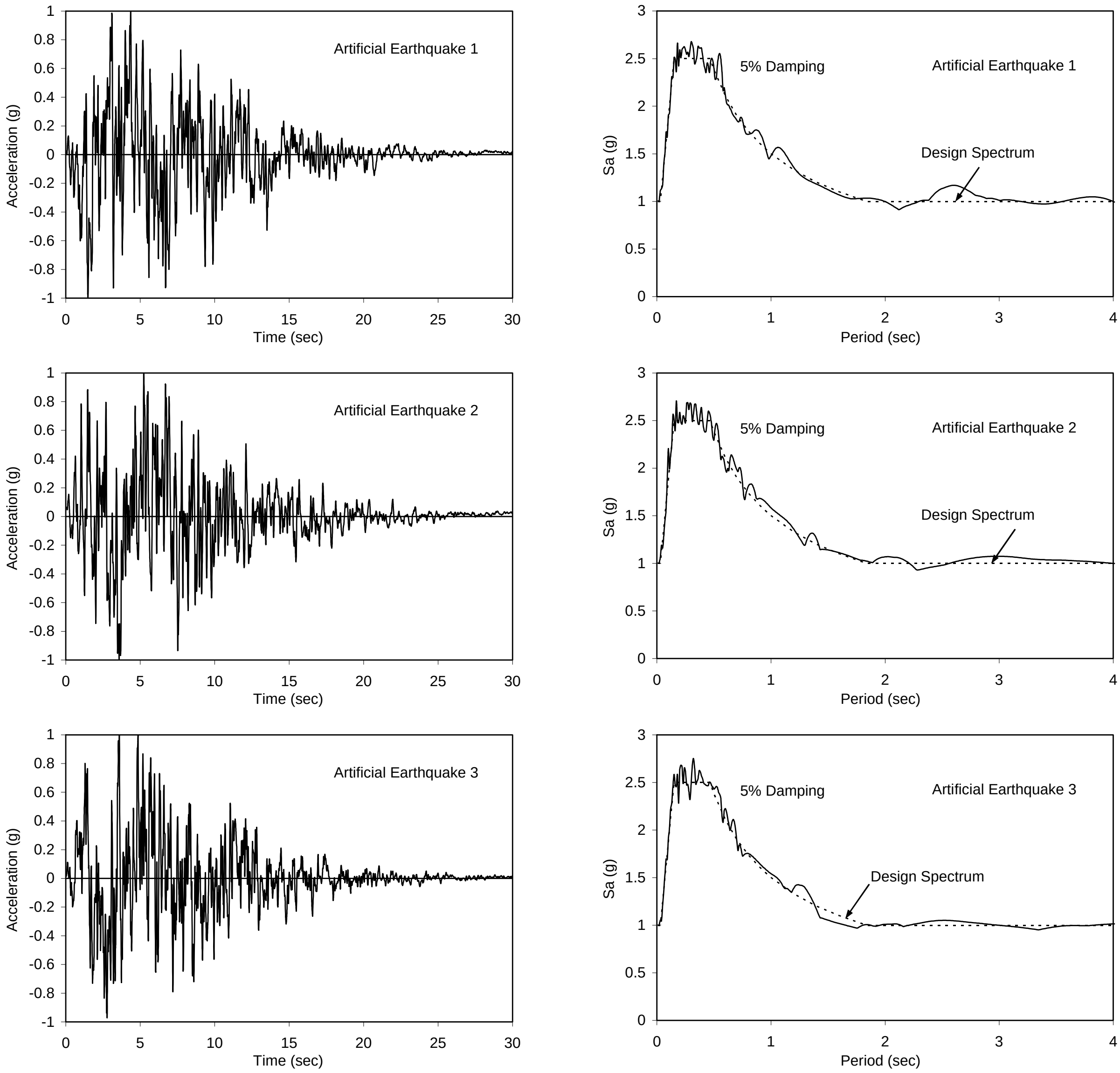
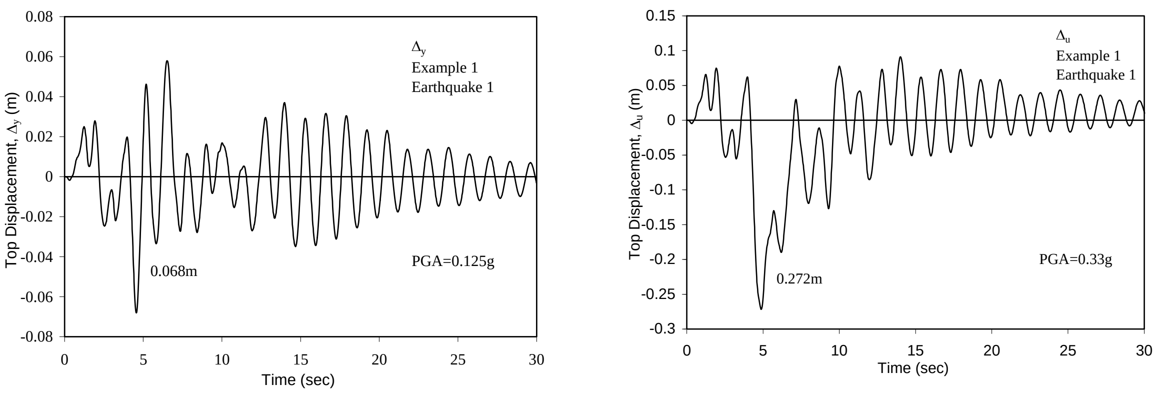


Fig.11 Artificial Earthquakes Generated from Soil Type II of TWA Building Code



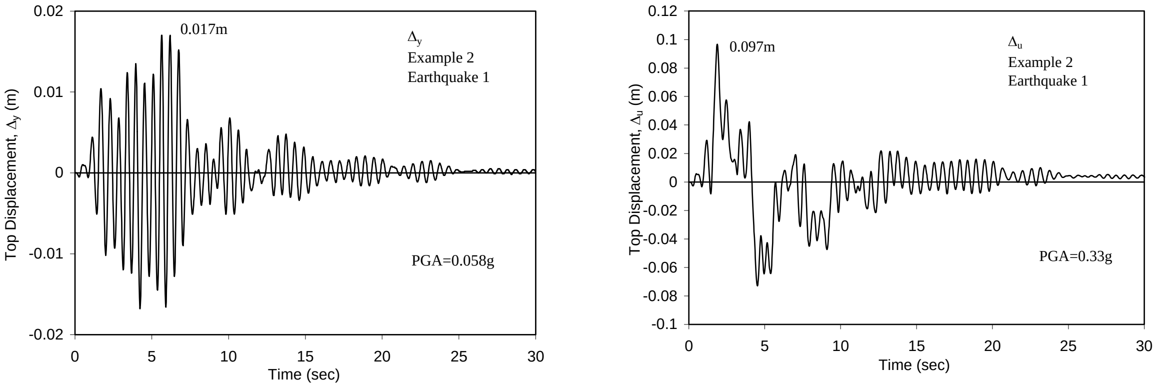


Fig.12 Top Displacements for Earthquakes at yielding level (Δ_y) and at PGA=0.33g (Δ_u)

Table.1 Verifications of non-iterative direct displacement-based design using dynamic nonlinear time-history analysis.

			Design Value	Dynamic Nonlinear Analysis (ξ_I =2%)	
Example 1	Δ_u (m)	Earthquake 1	0.27	0.272	0.33g
		Earthquake 2		0.258	0.33g
		Earthquake 3		0.287	0.33g
	Δ_y (m)	Earthquake 1	0.0675	0.068	0.125g
		Earthquake 2		0.067	0.104g
		Earthquake 3		0.067	0.112g
Example 2	Δ_u (m)	Earthquake 1	0.10	0.093	0.33g
		Earthquake 2		0.110	0.33g
		Earthquake 3		0.90	0.33g
	Δ_y (m)	Earthquake 1	0.0167	0.017	0.058g
		Earthquake 2		0.018	0.050g
		Earthquake 3		0.017	0.055g

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APPENDIX

```
START   PORTAL STEEL BRIDGES, UNIT : t-m-sec
$ CONTROL INFORMATION (9I5,I10,I5)
$ NJT CONJ  CDJ ZERO SAME MASS ELGR  CHK CORE      MEM  IFK IANA
    4    4    0    1    1    1    1    0
$ CONTROL NODE COORDINATE
$   J        X        Y  (I5,2F10.0)
    1      0.0      0.0
    2     10.0      0.0
    3      0.0      9.0
    4     10.0      9.0
$ COMMAND FOR NODES WITH ZERO DISPLACEMENT
$  IJ    X    Y    R   JJ KDIF (6I5)
    1    1    1    1    2    1
$ COMMAND FOR NODES WITH IDENTICAL DISPLACEMENT
$  IDIR NJ
    1    2    3    4
$ COMMAND FOR LUMPED MASS AT NODES (I5,3F10.0,2I5,F10.0)
$  IJ    X-MASS    Y-MASS    R-MASS   JJ KDIF    FACTOR
    3   3758.50      0.0      0.0    4    1      9.81
$
$ LOAD INFORMATION  (3I5,5F10.0)
$STAT CDLD STEP        DT    FACAXH    FACTMH    FACAXV    FACTMV    DISMAX
    2    0 1501      0.02     3.237       1.0
$ NONLINEAR STATIC LOAD CONTROL DATA
```

```
$NPAT CASE
    1    1
$ LOADING PATTERN 1
$ CMD TITLE
    1 apply nonlinear load to roof
$  II   X-FORCE   Y-FORCE  R-MOMENT   IK  DIF
    3    1447.5      0.0      0.0
$ NONLINEAR LOADING COMBINATION
$  ID ISEG   FCASE-1
    1  200       1
$ ACCELERATION RECORD
$NPTH NPTV OUT1 OUT2 TITLE  (4I5,10A6)
 1501               ARTIFICIAL EARTHQUAKE (G)
$ GROUND ACCELERATION IN X DIR. (12F6.0)
D:\old\drain\soil_ii1.acc
$  DAMPING INFORMATION  T=1.321sec   2%=.00841
$    APHLA     BETA     BETA0     DELTA      CM1(5F10.0)
            .008410
$ TIME HISTORY OUTPUT SPECIFICATION (13I5)
$ IPJ  IPE IENV NHOT NVOT NROT  NHR  NVR THPJ THPR THPL  ISJ  ISE IENG ISEC
    1            1             1        2    2                  1    1
$ LIST OF NODES FOR X DISPL. OUTPUT [IPJ] (10I5)
    3
$ LIST OF NODES FOR RELATIVE X DISPL. OUTPUT [IPJ] (10I5)
    3    1
$ E2. BEAM-COLUMN ELEMENT CONTROL INFORMATION
$  ID   NL NSTF NECT NYIE NFIX NINT (7I5)
    2    3    2         2    0
$ BEAM STIFFNESS TYPE (I5,5F10.0,I5)
$  ID         E        ED       AREA         I  Kii  Kjj  Kij     SHEAR
  POISSON
    1    200E6     0.05    .04402   .002563  4.0  4.0  2.0
    2    200E6     0.05    .08463   .004614  4.0  4.0  2.0
$ YIELDING SURFACE (2I5,4F10.0,4F5.0)
$ IYD  SHP      My+      My-      Pyc        Pyt   MA   PA   MB   PB
    1    1      2136     -2136
    2    2      3296     -3296   -21157.    21157.  1.0 0.15  1.0 0.15
$ ELEMENT GENERATION (12I5,2F5.0,I5,F5.0)
$ IEL    I    J KDIF STIF ECCE  YDi  YDj  GEO  THP   DD   LD  FDD  FLD  INF
FINF IENG ISEC
    1    3    4    1    1    1    1
    2    1    3    2    2    2    1     1    1
    3    2    4    2    2    2    1     1    1
STOP
```