Nov 6th, 12:00 AM - 12:00 AM

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Twenty-Second International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, USA, November 5 & 6, 2014

Simplified Seismic Design for Mid-Rise Buildings with Vertical Combination of Cold-Formed Steel and Concrete Framing

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Abstract: Presented in this paper is a practical approach for the seismic design of mid-rise buildings with vertical combination of cold-formed steel and concrete framing. In current design practice the presence of vertical irregularities on both mass and stiffness inherited in such building structures creates a challenge for the seismic design. Currently, a two stage lateral force procedure prescribed in ASCE 7 is prescribed for evaluating the seismic load if the lateral stiffness of the lower structure of the building is considerably more rigid than the upper one. In the proposed approach the requirement associated with the two stage analysis procedure on the lateral stiffness ratio between the lower and upper structures prescribed in ASCE 7 is abandoned. The seismic design can be obtained based on the required stiffness ratio determined by the proposed approach. Two examples are presented to demonstrate the efficiency of the proposed approach. The results obtained from proposed approach are justified by the verification of the dynamic analysis. Also found in this study is that in some cases over increasing the rigidity of lower structure so that the two stage analysis procedure can be applied may lead to a design that is not only uneconomical but also unsafe.

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1. Introduction

Mid-rise buildings with vertical combination of cold-formed steel (CFS) and concrete framing is a structural system in which the upper structure uses lightweight CFS framing while the lower one is a conventional reinforced concrete (RC) framed structure. The structural system is typically adopted in new residential or mixed residential-commercial buildings where the lower structure requires accommodating open spaces with heavier loads such as retail stores or parking garages. Such building structures are also in good fit with adding additional stories on existing RC buildings. In design practice the presence of vertical irregularities on both mass and stiffness creates a challenge for the seismic design of such building structures since the traditional equivalent lateral force (ELF) procedure which can be normally applied for "regular" structures is no longer generally applicable. For the building structures if the lower structure is significantly more rigid than the upper one, then an ELF-based two stage lateral force procedure may be applicable as prescribed in ASCE 7 (ASCE 2006, 2010). However, it is questionable whether the design obtained from the two stage analysis procedure is economical because of the stringent requirement on the lateral stiffness for the lower structure. In the case that the two stage analysis procedure is not applicable, a computer-based dynamic analysis has to be conducted for analyzing and designing such structures. However, appropriate modeling a mid-rise CFS framing system in a computer program can be somewhat complicated due to the presence of large number of members.

Previous researches on the seismic design of building structures with vertical combination of framing systems primarily focused on whether dynamic analysis is required to design such structure (Xiong et.al, 2008) and how to handle the damping irregularity (Papageorgiou et.al, 2011). The available seismic design methods were all based on dynamic analysis (Liu et.al, 2008; Chen et.al 2013). The development of a simplified seismic design approach for engineering practice has not been well addressed. Recently, a simplified method for the evaluation of seismic load for such building structure was proposed by Xu and Yuan (2014), in which the stringent stiffness requirements imposed on the lower and upper structures so that the two stage analysis could be applied were abandoned. The study presented herein extends the authors' previous work with aiming to develop a simplified approach for seismic design of mid-rise buildings with vertical combination of CFS and concrete framing. First, the analytical model of such structure and applicable design equations are introduced. Then, the proposed seismic design procedure is presented followed by two demonstrated design examples. Finally, results obtained from the proposed procedure and modal response spectrum analyses are compared and several design issues are discussed.

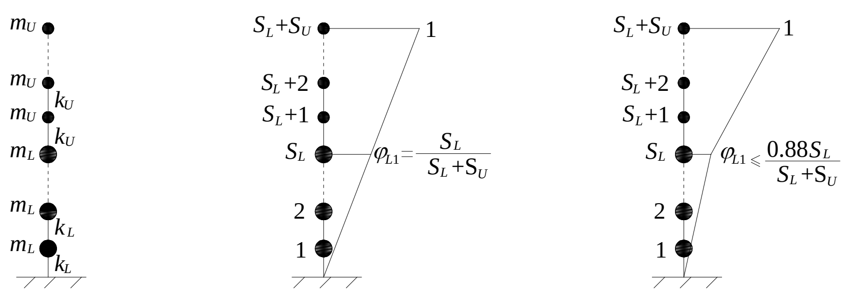
2. Formulation of Design Equation

2.1. Model Assumption

For a mid-rise building with an S_L-storey lower RC structure and an S_U-storey upper CFS structure, the idealized analytical model is shown in Figure 1 with the following assumptions: (1) the storey-mass and lateral storey-stiffness associated with the lower and upper structures, designated respectively by (m_L and k_L) and (m_U and k_U), are uniformly distributed; (2) the damping ratio for each vibration mode is 5% and ASCE 7 (ASCE, 2010) design response spectrum is adopted; (3) the lateral design is governed by earthquake load; and (4) the lower structure is laterally stiffer than the upper one. For the analytical model of a mid-rise building with vertical combination of framing systems as shown in Figure 1, Xu and Yuan (2014) found that if the first mode shape satisfied the relationship $\varphi_{L1} \leq 0.88 S_L/(S_L+S_U)$ as shown in Figure 1(c), then the lower structure is considered being lateral stiffer than the upper one.

2.2. Design Criteria

The seismic design of the mid-rise building with vertical combination of CFS and concrete framing is often governed by the code specified storey-drift limit. The storey-drift limits specified in ASCE 7 (ASCE, 2010) for the lower and upper structures are identical. Meanwhile, as discussed by Xu and Yuan (2014), the largest storey-drift-ratio of the mid-rise building with vertical combination of CFS and concrete framing likely occurs at the first storey of the upper structure. Therefore, if the storey-drift of the first storey of the upper CFS structure satisfies the code requirement, then the storey-drifts associated with other stories should be within the code specified limit.



(a) analytical model (b) linear first mode shape (c) stiffer lower structure

Figure 1: Analytical model of the mid-rise building with vertical combination of framing systems

The storey-drift of the first storey of the upper CFS structure is related to both lower and upper storey-stiffnesses k_L and k_U since the magnitudes of seismic forces in such structures are associated with both stiffnesses. Practically, the lower storey-stiffness k_L associated with the RC structure can be evaluated once the preliminary design is completed based on the gravity loads. In contrast, a tedious trial-and-error procedure has to be carried out to determine the required upper storey-stiffness k_U for evaluating both seismic loads and storey-drifts. Once the required upper storey-stiffness k_U is obtained the seismic loads can then be calculated and corresponding storey drift can be subsequently determined. The approach proposed in the following aims to evaluate the required lateral storey-stiffness of the upper CFS structure directly based on the specified limit on storey-drift-ratio without the trial-and-error process. Presented in section 2.3 is the derivation of the required lateral storey-stiffness, k_U, for the upper CFS structure.

2.3. Stiffness Evaluation of CFS Structures

For the mid-rise building with vertical combination of CFS and concrete framing, the shear force for the first storey of the upper portion V_U can be calculated as (Xu et.al, 2014)

$$V_U = \alpha_U m_U S_U S_a \left(T_U\right) / R \tag{1}$$

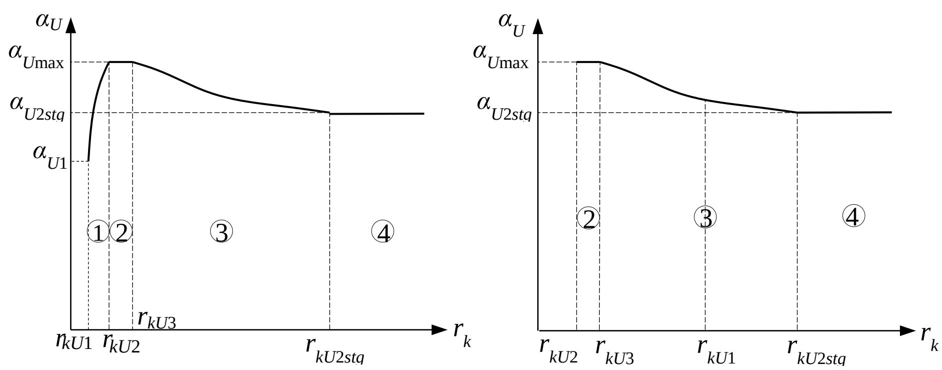
where R is the response modification factor. Factor α_U is the shear-force-amplification factor of the upper structure, which represents an amplification effect on the shear force contributed by the lower structure to the upper one. S_a (T_U) is the acceleration response spectrum at period T_U which can be calculated as

$$T_U = \frac{2\pi}{\bar{\omega}_{1U}} \sqrt{m_U / k_U} = \frac{2\pi}{\bar{\omega}_{1U}} \sqrt{m_U r_k / k_L} \tag{2}$$

where $\bar{\omega}_{1U}$ is the normalized natural frequency of the upper structure, which is only associated with the number of stories of the upper structure as shown in Table 1. Let $r_k=k_L/k_U$ be the storey-stiffness ratio between the lower and upper structures. Based on Xu and Yuan (2014), for the mid-rise building structure with given numbers of the lower and upper stories, designated by S_L and S_U, respectively; and as well as knowing the storey-mass ratio r_m ($r_m=m_L/m_U$) and the period of the upper structure T_U, the relationship between the amplification factor α_U and the storey-stiffness ratio r_k is shown in Figure 2. The factor α_U can be calculated as

Table 1 Normalized first mode natural frequency of uniform structures

number of storey	1	2	3	4	5	6	7	8	9
$\bar{\omega}_{1U}$	1	0.618	0.445	0.347	0.285	0.241	0.209	0.185	0.165



(a) $r_{kU1}<r_{kU2}$ (b) $r_{kU1}\geq r_{kU2}$

Figure 2: Shear force amplification factor of upper structure (Xu et.al, 2014)

$$\alpha_U = \begin{cases} \alpha_{U1}\left(r_k / r_{kU1}\right)^{x_1} & r_{kU1} \leq r_k < r_{kU2} \\ \alpha_{U\max} & r_{kU2} \leq r_k \leq r_{kU3} \\ \alpha_{U2stg}\left(r_k / r_{kU2stg}\right)^{x_2} & r_{kU3} < r_k < r_{kU2stg} \\ \alpha_{U2stg} & r_k \geq r_{kU2stg} \end{cases} \tag{3}$$

where exponents x_1 and x_2 can be calculated as

$$x_1 = \ln\left(\alpha_{U\max} / \alpha_{U1}\right) / \ln\left(r_{kU2} / r_{kU1}\right) \tag{4}$$

$$x_2 = \ln\left(\alpha_{U\max} / \alpha_{U2stg}\right) / \ln\left(r_{kU3} / r_{kU2stg}\right) \tag{5}$$

In which, factors $\alpha_{U\max}$ and α_{U1} can be obtained from Eqs. (6) and (7).

$$\alpha_{U\max} = \begin{cases} \alpha_{U\max1} & T_U / T_s \geq 1 \\ \alpha_{U\max2} & T_U / T_s \leq 0.769\left(r_m S_L / S_U\right)^{0.059} \\ \alpha_{U\max1}\left(T_U / T_s\right)^{x_3} & 0.769\left(r_m S_L / S_U\right)^{0.059} < T_U / T_s < 1 \end{cases} \tag{6}$$

$$\alpha_{U1} = \begin{cases} \alpha_{U11} & T_U / T_s \geq 1 \\ \alpha_{U12} & T_U / T_s \leq \sqrt{\left(S_U + 0.12 S_L\right) / \left(S_L + S_U\right)} \\ \alpha_{U11}\left(T_U / T_s\right)^{x_4} & \sqrt{\left(S_U + 0.12 S_L\right) / \left(S_L + S_U\right)} < T_U / T_s < 1 \end{cases} \tag{7}$$

where exponent x_3 and x_4 can be evaluated as

$$x_3 = \ln\left(\alpha_{U12} / \alpha_{U11}\right) / \ln\left(\sqrt{\left(S_U + 0.12 S_L\right) / \left(S_L + S_U\right)}\right) \quad (8)$$

$$x_4 = \ln\left(\alpha_{U\max 2} / \alpha_{U\max 1}\right) / \ln\left[0.769\left(r_m S_L / S_U\right)^{0.059}\right] \quad (9)$$

Critical storey-stiffness ratios r_{kU1}, r_{kU2}, r_{kU3} and $r_{kU2\mathrm{stg}}$, and factors $\alpha_{U\max1}$, $\alpha_{U\max2}$, α_{U11}, α_{U12} and $\alpha_{U2\mathrm{stg}}$ are functions of S_L, S_U, and r_m and can be obtained based on previous study (Xu et.al, 2014).

Once the shear force of the upper structure V_U is obtained from Eq. (1), the corresponding elastic storey-drift can be computed as

$$\delta_U = V_U / k_U = V_U r_k / k_L \quad (10)$$

Substituting Eqs.(2) to (9) into Eq.(1) and then subsequently substituting Eq.(1) into Eq.(10), the story-stiffness-ratio r_k corresponding to the elastic storey-drift δ_U can be obtained as:

(1) If r_k is located in region 1 of Figure 2 (a) ($r_{kU1} \le r_k < r_{kU2}$)

$$r_k = \begin{cases} \left[\dfrac{2\pi R}{\alpha_{U11} S_U \bar{\omega}_{1U}} \left(r_{kU1}\right)^{x_1} \sqrt{\dfrac{m_U}{k_L}} \dfrac{k_L \delta_U}{m_U S_{D1}}\right]^{\frac{1}{x_1+0.5}} & r_k \ge r_{ks1} \quad (11a) \\ e^y r_{kU2} & r_{ks3} < r_k < r_{ks1} \quad (11b) \\ \left[\dfrac{R}{\alpha_{U12} S_U} \left(r_{kU1}\right)^{x_1} \dfrac{k_L \delta_U}{m_U S_{DS}}\right]^{\frac{1}{1+x_1}} & r_k \le r_{ks3} \quad (11c) \end{cases}$$

(2) If r_k is located in region 2 of Figure 2 ($r_{kU2} \le r_k \le r_{kU3}$)

$$r_k = \begin{cases} \left(\dfrac{2\pi R}{\alpha_{U\max1} S_U \bar{\omega}_{1U}}\right)^2 \dfrac{m_U}{k_L} \dfrac{\left(k_L \delta_U\right)^2}{\left(m_U S_{D1}\right)^2} & r_k \ge r_{ks1} \quad (12a) \\ \dfrac{R}{\alpha_{U\max2} S_U} \dfrac{k_L \delta_U}{m_U S_{DS}} & r_k \le r_{ks2} \quad (12b) \\ \left[\dfrac{R}{\alpha_{U\max1}} \left(\dfrac{T_s \bar{\omega}_{1U}}{2\pi} \sqrt{\dfrac{m_U}{k_L}}\right)^{x_4} \dfrac{k_L \delta_U}{m_U S_{DS}}\right]^{\frac{1}{1+0.5 x_4}} & r_{ks2} < r_k < r_{ks1} \quad (12c) \end{cases}$$

(3) If r_k is located in region 3 of Figure 2 ($r_{kU3} < r_k < r_{kU2\mathrm{stg}}$)

$$r_k = \begin{cases} \left[\dfrac{2\pi R}{\alpha_{U\max 1} S_U \bar{\omega}_{1U}} \left(r_{kU3}\right)^{x_2} \sqrt{\dfrac{m_U}{k_L}} \dfrac{k_L \delta_U}{m_U S_{D1}} \right]^{\frac{1}{x_2+0.5}} & r_k \geq r_{ks1} \quad (13a) \\ e^y r_{kU2stg} & r_{ks2} < r_k < r_{ks1} \quad (13b) \\ \left[\dfrac{R}{\alpha_{U\max 2} S_U} \left(r_{kU3}\right)^{x_2} \dfrac{k_L \delta_U}{m_U S_{D1}} \right]^{\frac{1}{1+x_2}} & r_k \leq r_{ks2} \quad (13c) \end{cases}$$

(4) If r_k is located in region 4 of Figure 2 ($r_k \geq r_{kU2\mathrm{stg}}$)

$$r_k = \begin{cases} \left(\dfrac{2\pi R}{\alpha_{U2stg} S_U \bar{\omega}_{1U}} \right)^2 \dfrac{m_U}{k_L} \dfrac{\left(k_L \delta_U\right)^2}{\left(m_U S_{D1}\right)^2} & r_k > r_{ks1} \quad (14a) \\ \dfrac{R}{\alpha_{U2stg} S_U} \dfrac{k_L \delta_U}{m_U S_{DS}} & r_k \leq r_{ks1} \quad (14b) \end{cases}$$

In Eqs. (11) ~ (14), conditions $r_k \geq r_{ks1}$, $r_k \geq r_{ks2}$ and $r_k \geq r_{ks3}$ are equivalent to conditions $(T_U/T_S) \geq 1$, $(T_U/T_S) \geq 0.769(r_m S_L/S_U)^{0.059}$ and $(T_U/T_S) \geq [(S_U+0.12S_L)/(S_L+S_U)]^{0.5}$, respectively, and vice versa. The critical storey-stiffness ratios r_{ks1} , r_{ks2} and r_{ks3} can be calculated as: $r_{ks1}=(T_s\bar{\omega}_{1U})^2 k_L/m_U$, $r_{ks2}=0.591(r_m S_L/S_U)^{0.118} r_{ks1}$ and $r_{ks3}=[(S_U+0.12S_L)/(S_L+S_U)] r_{ks1}$. The coefficient y in Eqs.(11b) and (13b) are the roots of the following quadratic equation

$$ay^2 + by + c = 0 \quad (15)$$

where coefficient a, b and c are defined as

$$a = \begin{cases} x_3 / \ln\left(r_{kU1} / r_{kU2}\right) & r_{kU1} \leq r_{ks3} < r_k < r_{ks2} < r_{kU2} \quad (16a) \\ \left(x_3 - x_4\right) / \ln\left(r_{kU2} / r_{kU1}\right) & r_{kU1} \leq r_{ks2} < r_k < r_{ks1} < r_{kU2} \quad (16b) \\ x_4 / \ln\left(r_{kU3} / r_{kU2stg}\right) & r_{kU3} \leq r_{ks2} < r_k < r_{ks1} \leq r_{kU2stg} \quad (16c) \end{cases}$$

$$b=\begin{cases} 2A+\dfrac{x_3\ln\left(Br_{kU2}\right)}{\ln\left(r_{kU1}/r_{kU2}\right)}+2 & r_{kU1}\le r_{ks3}<r_k<r_{ks2}<\ r_{kU2} \qquad (17a)\\ x_4+2F+\dfrac{\left(x_4-x_3\right)\ln\left(Br_{kU2}\right)}{\ln\left(r_{kU1}/r_{kU2}\right)}+2 & r_{kU1}\le r_{ks2}<r_k<r_{ks1}<\ r_{kU2} \qquad (17b)\\ 2D+\dfrac{x_4\ln\left(Br_{kU2stg}\right)}{\ln\left(r_{kU3}/r_{kU2stg}\right)}+2 & r_{kU3}\le r_{ks2}<r_k<r_{ks1}\le\ r_{kU2stg} \qquad (17c)\end{cases}$$

$$c=\begin{cases} -2\ln\left(E/r_{kU2}/\alpha_{U\max2}\right) & r_{kU1}\le r_{ks3}<r_k<r_{ks2}<r_{kU2} \qquad (18a)\\ -2\ln\left[\dfrac{E\left(\dfrac{\bar{\omega}_{1U}T_s}{2\pi}\right)^{x_4}}{r_{kU2}\alpha_{U\max1}}\left(\dfrac{k_L}{m_U r_{kU2}}\right)^{\frac{x_4}{2}}\right] & r_{kU3}\le r_{ks2}<r_k<r_{ks1}<r_{kU2stg} \qquad (18b)\\ -2\ln\left(E/r_{kU2stg}/\alpha_{U2stg}\right) & r_{kU2}\le r_{ks2}<r_k<r_{ks1}\le\ r_{kU3} \qquad (18c)\end{cases}$$

where $A=\ln(\alpha_{U11}/\alpha_{U\max2})/\ln(r_{kU1}/r_{kU2})$, $B=[2\pi/(T_s\bar{\omega}_{1U})]^2\left(m_U/k_L\right)$, $D=\ln(\alpha_{U\max1}/\alpha_{U2stg})/\ln(r_{kU3}/r_{kU2stg})$, $E=(Rk_L\delta_U)/(m_U S_U S_{DS})$ and $F=\ln(\alpha_{U11}/\alpha_{U\max1})/\ln(r_{kU1}/r_{kU2})$.

From Eqs. (11) to (14), it can be seen the storey-stiffness-ratio r_k monotonically increases as the increase of the storey-drift δ_U. Therefore, the required storey-stiffness-ratio r_{kreq} can be calculated by setting $\delta_U=\delta_{U\lim}$ in Eqs.(11) to (14). An upper structure of which the storey-stiffness ratio is less than r_{kreq} satisfies the specified storey-drift limit but may not be economic as the corresponding lateral stiffness of the upper structure is more than necessary. Obviously, if the stiffness ratio is greater than r_{kreq}, then the storey drift limit is violated. Based on the obtained required storey-stiffness ratio r_{kreq}, the storey-stiffness of the upper structure can be determined as $k_U=k_L/r_{kreq}$.

3. Design Procedure

Before providing the specified design procedure, it is worthy to discuss how to determine the design response spectrum accelerations S_{DS} and S_{D1} in Eqs. (11) to (14). According to FEMA P695 (FEMA, 2009), the average value of collapse probability for buildings designed based on ASCE 7 (ASCE, 2010) is 10% under the maximum considered earthquake (MCE). This indicates the non-existence (NE) probability of the storey-drift greater than the storey-drift limit $\Delta_{U\lim}$ is 90%. However, the design spectrum specified in ASCE 7 represents the median demand (50%) for the specified hazard level. In order to design for a target NE probability of storey-drift greater than the median, which is 90%, the design spectrum value must be scaled up to reflect an increase of NE probability. The design spectral acceleration adjusted for NE probability $\overline{S}_a$ is

$$\overline{S}_a = C_{NE} S_a \tag{19}$$

where S_a (median) is the code-specified acceleration value and the scale factor C_{NE} is assumed to be log-normal distributed with a median value of 1.0 and a logarithmic standard deviation, β_R, which accounts for the uncertainty of the ground motions as well as the uncertainty associated with the design procedure. According to the investigation of Pang et.al (2011), it is reasonable to let β_R be 0.75. Therefore, to satisfy the 90% NE probability, the corresponding scale factor is $C_{NE} = \exp\left[\Phi^{-1}\left(NE\right)\beta_R + \ln(1)\right] = 2.61$.

With the adjustment on design response spectrum acceleration, the seismic design of the mid-rise building with vertical combination of CFS and concrete framing can be carried out in accordance with the following procedure:

Step 1: **Evaluate the effective seismic weight distribution**

The effective seismic weight should be calculated in accordance with provisions 12.7.2 of ASCE 7 (ASCE, 2010). Then, based on the effective seismic weight distribution, calculate the storey-mass ratio r_m between the lower and upper structures ($r_m = m_L / m_U$).

Step 2: **Compute the storey-stiffness of the lower structure k_L**

The elastic storey-stiffness of the lower structure k_L can be simply evaluated as

$$k_L = \sum_{i=1}^{N_c} 12\left(EI\right)_i / h^3 \tag{20}$$

where N_C is the number of columns which contribute to the lateral stiffness of the lower structure in each storey; $(EI)_i$ is the flexural stiffness of the i-th column; and h is the storey height of the lower structure.

Step 3: Calculate elastic storey-drift limit $\delta_{U\lim}$ for the upper CFS structure

The elastic storey-drift limit of the upper CFS structure $\delta_{U\lim}$ can be evaluated as

$$\delta_{U\lim} = \Delta_{U\lim} I_e / C_d \tag{21}$$

In which the inelastic seismic storey-drift limit of the upper CFS structure $\Delta_{U\lim}$, the importance factor of the building I_e, and the deflection amplification factor C_d can be determined according to sections 12.12.1, 11.5.1 and 12.2.3.1 of ASCE 7 (ASCE, 2010), respectively.

Step 4: Determine the required lateral stiffness of the upper structure k_U

By setting $\delta_U=\delta_{U\lim}$, the optimal storey-stiffness-ratio r_{kreq} can be calculated based on Eqs. (11) to (14) with use of the adjusted design response spectrum accelerations $\overline{S}_a$ determined by Eq. (19). The corresponding required upper-storey stiffness can then obtained as $k_U = k_L / r_{kreq}$.

Once the required upper storey-stiffness is obtained, the corresponding seismic loads for both upper and lower structures can now be evaluated. The layout of the selected lateral load resisting system of the upper structure can be determined based on the required upper storey-stiffness and architectural design. Individual lateral load resisting elements can then be designed based on the required upper storey-stiffness and the magnitude of the applied seismic load.

4. Design Examples

Two hypothetical mid-rise buildings with vertical combination of CFS and concrete framing, assuming located in Los Angeles, California, are presented in the following to illustrate the proposed approach.

4.1. Example 1

Shown in Figure 3 is the floor plan of the lower structure of an eight-storey building structure. The two-storey lower structure is constructed with the special RC moment frame while the six-storey upper structure is to be built with CFS framing. The storey-height of the lower and upper structure is 10.8ft (3.3m) and 10ft (3.06m), respectively. The specified dead loads associated with the upper and lower structures are taken as 0.416psi (2.87kPa) and 0.701psi (4.83kPa), respectively. The elastic stiffness of the concrete is 4.351×10^6psi (3×10^7 kPa).

The soil condition for the building is assumed as Class B, with the building risk category being II. Seismic design parameters can be determined according to ASCE 7 (ASCE, 2010). The response modification coefficient R =6.5, the deflection amplification factors C_d =4 and the importance factor I_e=1 for the

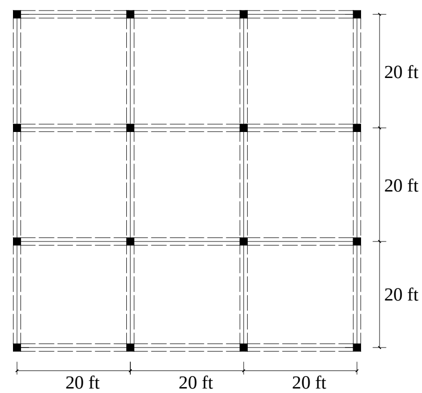


Figure 3: Floor plan of lower RC structure

designed building. The site spectrum S_S and S_1 are 2.447g and 0.858g, respectively. The long transition period T_L is 8s, which results in the design spectrum being S_{DS}=1.632g and S_{D1}=0.572g, and as well as the factored design response spectrum being $\overline{S}_{DS} = 2.61\times1.632 = 4.26\text{g}$ and $\overline{S}_{D1} = 2.61\times0.572 = 1.49\text{g}$.

The preliminary gravity design of the lower structure yields the size of the RC column is 19.69 in×19.69 in (500mm×500mm). For the upper structures, CFS shear wall will be the lateral load resisting system. The effective seismic weights of each storey for the upper and lower structures are m_U=0.416×20^2×9×12^2/10=2.16×10^{4}lb (96,113kg) and m_L=0.701×20^2×9×12^2/10=3.63×10^{4}lb (161,820kg), respectively. The storey-mass ratio r_m =3.63/2.16 =1.7. The lateral-storey-stiffness of the lower structure k_L can be calculated as per FEMA 356 (FEMA, 2000), which specifies the flexural stiffness $(EI)_{\mathrm{stf}}$ should be 0.5 times of the actual component flexural stiffness if the axial load ratio is not greater than 0.3. Therefore, the storey-stiffness of the lower structures is k_L=2.88×10^{4}kip/ft (4.174×10^{5}kN/m). The permissible storey-drift of the CFS shear wall system and concrete frame are 0.02h. Therefore, the storey-drift limit of the upper structure is $\delta_{U\mathrm{lim}}$=0.02×10/4=0.05ft (15.3mm).

Based on Xu and Yuan (2014), the corresponding critical storey-stiffness ratios are r_{kU1}=1.82, r_{kU2}=0.71, r_{kU3}=1.97, $r_{kU2\mathrm{stg}}$=2.38, r_{ks1}=0.79, r_{ks2}=0.43, and shear-force-amplification factors $\alpha_{U\mathrm{max}1}$=1.02, $\alpha_{U\mathrm{max}2}$=1.16, α_{U2stg}=1.1. As it requires that the stiffness ratio $r_k > r_{kU1}$ and because $r_{kU1} > r_{kU2}$, required r_k can only be located in regions 2, 3 or 4 as shown in Figure 2 (*b*). Meanwhile, as r_{kU1} is greater than both r_{ks1} and r_{ks2}, it is concluded that for $r_k > r_{kU1}$, r_k is also greater than both r_{ks1} and r_{ks2}. Then, based on section 2.3, the required stiffness ratio r_{kreq} can be calculated as:

(1) If $r_{kU2} \leq r_{kreq} \leq r_{kU3}$, the corresponding value of r_{kreq} obtained from Eq.(12*a*) is 3.66 which is greater than r_{kU3} = 1.97; thus, it is not a correct solution.

(2) If $r_{kU3}<r_{kreq}<r_{kU2\text{stg}}$, with the exponent x_2 = 0.41, from Eq.(13*a*), it yields r_{kreq}=2.77>$r_{kU2\text{stg}}$ =2.38. It is not a correct solution, either.

(3) If $r_{kreq} \geq r_{kU2\text{stg}}$, the correct solution of r_{kreq} =3.13> $r_{kU2\text{stg}}$ =2.38 is obtained from Eq.(14*a*). The corresponding storey-stiffness of the upper structure is k_U=2.88×10^4/3.13=9.21×10^{3}kip/ft (1.34×10^{5}kN/m).

Based on experimental investigation conducted by Branston (2004), the initial stiffness for CFS shear wall with 7/16 in (11mm) single-sided OSB sheathing and screw spacing 4/12in (100/300mm) is 40.058 kip/ft per feet (1918kN/m per meter). Assume the OSB sheathing is applied on both sides of the shear wall and the wall studs are adequately designed. Therefore, the initial stiffness of the double-sided shear wall is 80.117 kip/ft per feet (3836.1kN/m per meter), and the required length of shear wall is L = 9.138×10^3/80.117 = 115.02ft (34.76m).

4.2. Example 2

The lower structure and its floor layout of the building in this example are the same as that of Example 1, except that it is a nine-storey building with CFS and RC framing. The lower six-storey structure is a special RC moment frame while the upper three-storey is CFS framing. The reinforced concrete column is 23.6in×23.6in (600mm×600mm). The specified dead load for the lower RC structure is 0.950psi (6.550kPa) and the corresponding storey weight is m_L=0.932×20×20×9×12×12/10=4.92×10^{4}lb (219,352kg).

Similar to that of the previous example, the storey weight of the upper CFS structure is taken m_U=0.416×20^2×9×12^2=2.16×10^{5}lb (96,113kg). Thus, the storey-mass ratio is r_m=4.83/2.16=2.3. The storey-stiffness of the lower structure k_L=5.952×10^{4}kip/ft (8.655×10^{5}kN/m). By following the procedure stated in section 2.3, it is found the required stiffness ratio r_{kreq}=5.49, which is located in region 1 shown in Figure 2 (a). The corresponding required storey-stiffness of the upper structure is k_U=k_L/r_{kreq}=1.084×10^4 kip/ft (1.576×10^{5}kN/m) and the corresponding required CFS shear wall length is L=1.084×10^4/80.117 =135.3ft (41.1m).

5 Design Verification and Discussion

Modal spectrum analysis with complete quadratic combination (CQC) rule to combine the peak modal responses (Chopra, 2007) was carried out for the foregoing two buildings as eight- and nine-storey structures based on the analytical model shown in Figure 1 (a) with the effective storey weights and storey stiffness evaluated in the examples. The storey-drift-ratios of the first-storey of CFS shear wall for Examples 1 and 2 obtained from the modal spectrum analysis are both 1.7%, which are less than the specified limit of 2%. Therefore, the required storey-stiffness of the upper structure obtained from the proposed approach is conservative.

Because of the stringent requirement on the storey-stiffness ratio, application of the two-stage analysis procedure on the mid-rise building with vertical combination of CFS and concrete framing is limited. For instances in the foregoing two examples, the required storey-stiffness ratio r_{kreq} of the first example is located in region 4 of Fig. 2, where the two-stage analysis procedure is applicable. However, the required storey-stiffness ratio r_{kreq} of the second example is located in region 1, where the two-stage analysis procedure cannot be used to evaluate the seismic load and the corresponding storey-drift.

To use the two-stage analysis procedure prescribed in ASCE 7 (ASCE, 2006) for designing mid-rise buildings with vertical combination of CFS and concrete framing, it is assumed that there is no stiffness interaction between upper and lower structures while evaluating the seismic loads and the shear force amplification factor α_U=1. By substituting α_U=1 into Eq.(1) and then substituting Eq.(1) into Eq.(10), the corresponding storey-stiffness of the upper CFS structure k_U can be calculated by setting $\delta_U=\delta_{U\lim}$. The resulting required storey-stiffness of the upper structure is k_U=6434.7kip/ft (93,908kN/m) for the building structure shown in Example 2. As the two-stage analysis procedure requires the stiffness ratio to satisfy $r_k \geq 17$, the corresponding required storey-stiffness for the lower structures is $k_L=17\times6434.7=1.09\times10^5$kip/ft ($1.596\times10^6$kN/m). Consequently, it requires the size of the concrete column to be increased to 27.6in×27.6in (700mm×700mm) from 23.6in×23.6in (600mm×600mm). Comparing the results obtained from the proposed approach and the two-stage analysis, the two-stage analysis procedure yields a lower value of the required storey-stiffness for the upper structure but a larger value for the lower structure. Considering the upper structure has only three stories and the lower one has six stories, it would be more economical to adopt the design generated by the proposed approach. In fact, for the result obtained by the two-stage analysis, the modal spectrum analysis shown the corresponding storey-drift associated with the first-storey of the upper structure is 2.7% which violates the specified limit of 2%. Therefore, based on the foregoing discussion, over increase the lateral rigidity of the lower structure so that the two stage analysis procedure is applicable may lead to a design that is not only uneconomical but also unsafe in this case.

4. Conclusion

A practical approach for the seismic design of mid-rise buildings with vertical combination of CFS and concrete framing is presented. Unlike the two-stage analysis procedure prescribed in ASCE 7, the story-stiffness ratio requirement is no longer a requirement. The effects of the stiffness interaction on seismic loads between the upper and lower structures are accounted for with no need of using computer-based dynamic analysis. In the proposed approach the required storey-stiffness associated with the upper structure can be obtained directly based on specified storey-drift-limit without involving any lengthy trial-and-error design routines. Two examples are presented to demonstrate the efficiency of the proposed approach. The results obtained from proposed approach are justified by the verification of modal spectrum analysis. It is also found in this study that in some cases over increase the lateral rigidity of the lower structure so that the two stage analysis procedure is applicable may lead to a design that is not only uneconomical but also unsafe.

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Appendix A. –Notation

m_L	storey-mass of the lower structure
m_U	storey-mass of the upper structure
r_m	storey-mass ratio between the lower and upper structures
k_L	lateral storey-stiffness of the lower structure
k_U	lateral storey-stiffness of the upper structure
r_k	storey-stiffness ratio between the lower and upper structures
S_L	number of the storey of the lower structure
S_U	number of the storey of the upper structure
$\overline{\omega}_1$	normalized first mode natural frequency of the uniform structure
T_U	first mode period of the upper structure
T_L	long transition period
T_s	period at which the horizontal and descending curves of the response spectrum in ASCE 7 intersects
V_U	shear force for the first storey of the upper structure
α_U	shear-force-amplification factor of the upper structure
δ_U	elastic storey-drift
$\delta_{U\lim}$	elastic storey-drift limit
$\Delta_{U\lim}$	inelastic storey-drift limit
R	response modification factor
C_d	deflection amplification factor
S_a	response spectrum acceleration
C_{NE}	adjustment factor for the response spectrum acceleration
$\bar{S}_a$	adjusted response spectrum acceleration