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SEISMIC DESIGN OF SINGLE-STORY BUILDINGS WITH A NONLINEAR FLEXIBLE ROOF DIAPHRAGM

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Abstract

Single-story buildings with large footprints form a significant portion of the inventory of industrial buildings in North America. Such buildings are designed to serve a variety of functions, including recreational, commercial, educational and industrial. Steel decks or wooden deck panels, which exhibit low in-plane stiffness, often form the roofing system for these buildings. Previous studies have shown that the flexibility of the diaphragm significantly affects the seismic response of single-story buildings.

A roofing system with low in-plane stiffness lengthens the period of the structure, increases the ductility demand on the seismic force resisting system, and alters the distribution of the earthquake-induced inertia forces along the length of the diaphragm. Change in the distribution of inertia forces leads to an increase in the internal forces in the diaphragm. Specifically, the shear force at quarter-span and bending moment at mid-span both undergo significant magnifications.

The common approach to the design of single-story buildings with flexible diaphragms is to ensure that the roofing system remains elastic, while the seismic force resisting system is allowed to deform into the nonlinear range to dissipate the seismic energy imposed on the system. Although this approach is widely practiced as the design strategy for such systems, it often leads to uneconomical designs. Recent experimental studies have shown that steel deck panels have fair capacity for energy dissipation through inelastic cyclic deformations. A design in which the diaphragm is allowed to become inelastic would therefore be acceptable, provided the ductility demand is limited. In fact, several building codes now allow a limited amount of inelastic deformation in the diaphragm. However, related information on the design is lacking. For instance, the relationship between the ductility related force modification factor and the target ductility has not been established.

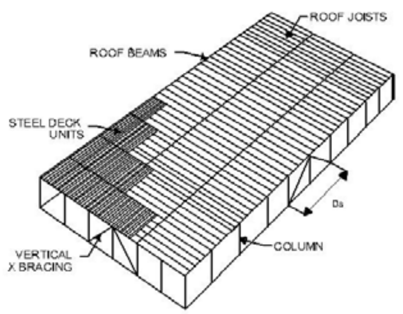
This paper reviews the findings of previous studies on the seismic behavior of single-story buildings with flexible diaphragms and presents the results of an analytical study to examine the structural implications associated with a design that relies on the roofing system to act as the energy dissipation system. A set of buildings is selected and designed using several alternative energy dissipation mechanisms. The selected buildings are subjected to a number of spectrum compatible records and the response results are used to develop empirical expressions for determining the key design parameters. Procedures are then developed for the design of single-story buildings including buildings in which the flexible steel deck diaphragm is expected to undergo inelastic deformations.

Keywords: Single-Story Buildings; Flexible Diaphragm; Energy Dissipation; Seismic Design; Steel Structures

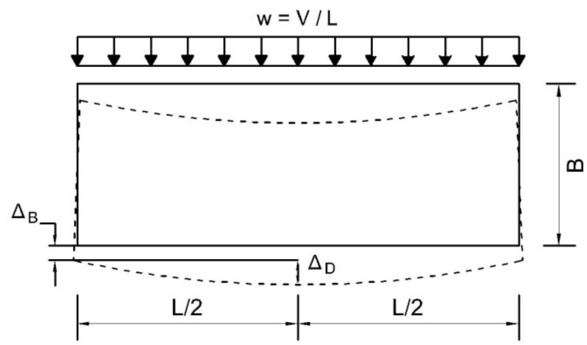
1. Introduction

Large number of single-story buildings are built throughout the world to serve a variety of functions. Figure 1 (a) is a schematic illustration of a single-story building. The gravity loads acting on the roofing system of such buildings are relatively small. This combined with the often large foot print of such buildings makes the use of concrete reinforced slabs or steel decks with concrete topping both unnecessary and uneconomical. As a result, the roofing system commonly used in single-story buildings consists of steel decks or wooden panels. The in-plane stiffness of these systems is quite low compared to that of reinforced concrete slabs, composite slabs, and steel decks with concrete toppings.

An important role of the diaphragm is the distribution of earthquake-induced inertia forces among the elements of the lateral load resisting system (LLRS), such as braced bays, concrete shear walls, masonry shear walls, etc. Earthquake excitations give rise to inertia forces in the plane of the diaphragm, hence, the diaphragm in-plane stiffness can greatly affect the seismic response of the building. Considering this, different codes and design guidelines have classified the diaphragm systems into three different categories, based on their in-plane stiffness relative to that of the LLRS: (1) rigid, (2) flexible, and (3) stiff. Stiff diaphragms are neither rigid nor flexible and their behavior lies somewhere in between. The classification methods are both calculation-based and prescriptive. Humar and Popovski [1] reviewed the classification methods and concluded that the guidelines recommended by the Federal Emergency Management Agency of United States (FEMA) [2] were quite appropriate. The FEMA classification is related to the drift ratio, which is a measure of the flexibility of the diaphragm and is defined as the ratio of the maximum horizontal deformation of the diaphragm under a uniformly distributed static lateral load (Δ_D) to the average inter-story drift of the story immediately below the diaphragm (Δ_B) under the same load, as illustrated in Fig. 1 (b). FEMA 356 [2] classifies the diaphragm as flexible if the drift ratio (Δ_D/Δ_B) is equal to or greater than 2.0, as rigid if the drift ratio is equal to or less than 0.5, and as stiff when the drift ratio lies within the range 0.5 and 2.0.



(a)



(b)

Fig.1 – (a) Schematic illustration of single-story buildings with flexible diaphragms, (b) Displacements under a uniformly distributed static lateral load

The flexibility of the diaphragm significantly affects the dynamic characteristics of the system and alters the response of single-story buildings to seismic excitations. These changes include an increase in the fundamental period of the system, significant increase in the ductility demand on the LLRS, which in turn would limit the force modification factor to be used in the seismic design of the LLRS, and a change in the distribution of inertia forces acting along the length of the diaphragm, which leads to a magnification of the in-plane shear force at quarter span and the bending moment at mid span. Previous studies have investigated the effects of the diaphragm flexibility on the seismic response of single-story buildings. Some of the research studies on this subject are outlined in the following paragraphs.

Jain and Jennings [3] and Kim and White [4] focused on the development of methods to account for the flexibility of the diaphragm on the seismic response of the system. In other studies, the research was specific to the types of the LLRS. For instance, Button et al [5] and Lee et al [6] studied the effects of the flexibility of the diaphragm on the seismic response of reinforced concrete buildings. Tena-Colunga and Abrams [7] and Kim and White, 2004b [8] studied the response of buildings with wood panels in which the LLRS consisted of masonry shear walls.

Other analytical studies on the effect of diaphragm flexibility on the seismic response of single-story buildings include those by Mortazavi and Humar [9], Humar and Popovski [10], Trudel-Languedoc et al [11], Adebar et al [12] and Tremblay and Stiemer [13]. These studies confirm the increase in the ductility demand on the LLRS and the magnification of bending moment and shear forces along the length of the diaphragm.

Paquette and Bruneau [14] and, Tremblay et al [15] have carried out experimental studies on the seismic response of single-story buildings with flexible diaphragms. Such studies have shown that the diaphragm flexibility increases the deformations and the shear forces along the length of the diaphragm.

In the classical design approach to the design of single-story buildings, the elements of the LLRS such as braced frames, shear walls, etc. are relied upon to dissipate the earthquake energy through their inelastic action while the diaphragm is designed to remain elastic. Most of the afore-mentioned studies were also based on the assumption that the nonlinearity was confined to the LLRS while the diaphragm remained elastic. For example, Humar and Popovski [10] and Mortazavi and Humar [9] examined the response of single-story buildings in which the LLRS exhibited an elastoplastic hysteretic response.

Recent studies, such as the one by Tremblay et al [16], have investigated the possibility of adopting an alternative design approach for the design of single-story buildings in which the roof diaphragm panels dissipate energy as they are strained into the nonlinear range. Tremblay et al [16] have noted that if designed and detailed properly, the diaphragm can exhibit satisfactory ductility and energy dissipation.

Studies on the nonlinear behavior of the steel deck diaphragm panels include those by Rogers and Tremblay [17, 18] and Essa et al. [19]. Studies by Essa et al [19], and Massarelli et al [20] showed that the nonlinear behavior of steel deck panels, which is governed by their shear capacity, is dominated by the behavior of their fasteners. Essa et al [19] showed that steel deck panels exhibit severe pinching and strength degradation in their hysteretic response.

The previous studies by Humar and Popovski [1] and [10] and Mortazavi and Humar [9], in which the steel deck diaphragm was designed to remain elastic, are extended in the present study. The study also investigates the feasibility of allowing the roof diaphragm to deform into the nonlinear range and to dissipate the seismic energy. Prescriptive methods are presented for such a design approach. For this purpose, buildings from the same database as used in the previous studies are redesigned so that the diaphragm would act as the energy dissipating system. The buildings are then subjected to the same ground motions as used in the previous studies to relate the variations in the results to the nonlinearity in the diaphragm. The advantages of designing the diaphragm to exhibit nonlinearity, if any, are discussed. In the end, provisions for the seismic design of single-story buildings having different energy dissipation mechanisms are proposed.

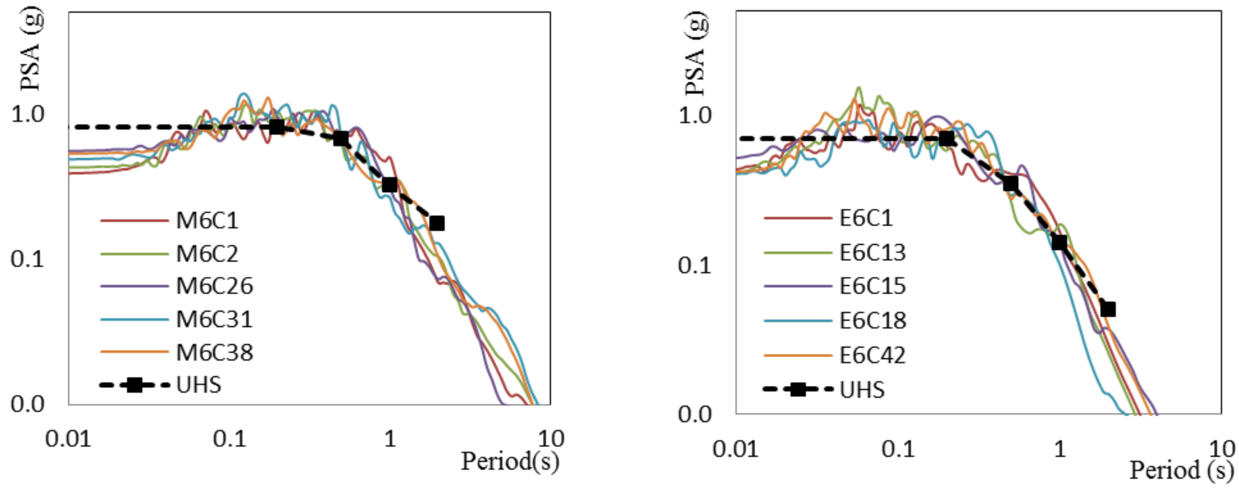
2. Building Database

For their 1996 study, Tremblay and Stiemer [13] designed a set of 36 single-story buildings with flexible diaphragms. The roofing system consisted of steel deck panels and the LLRS was comprised of concentric steel braces located along the perimeter of the buildings. The buildings were designed according to the requirements of the 1995 National Building Code of Canada [21] for 6 different site classes as defined in that code. The buildings were of 3 different sizes: small (15×30×5.4m), medium (30×60×6.6m), and large (60×120×9.0m). They had two different types of roofing system: a heavy roofing system and a light roofing system. Further information on the buildings can be found in Tremblay and Stiemer [13] and Humar and Popovski [10]. The drift ratio values for the buildings varied over a wide range, from 1.04 to 10.3, while the fundamental period of the buildings varied from 0.254s to 1.38s. The buildings analyzed in the current study are selected from the set designed by Tremblay and Stiemer [13].

3. Spectrum Compatible Ground Motions

Atkinson [22] has generated a comprehensive database of simulated records using a stochastic finite fault approach. The records were generated for a range of tectonic conditions, earthquake magnitudes, durations, distances, and site conditions, contributing to the 2% in 50 years uniform hazard spectrum (UHS) defined in the NBCC 2010 [23]. When properly scaled, synthetic ground motions can match the target design spectrum for any specific site. The records used in the present study represent the seismological characteristics of eastern (Montreal) and western (Vancouver) Canada motions.

In the current study records M6C1, M6C2, M6C26, M6C31 and M6C38, with scaling factors of 0.78, 0.87, 1.19, 0.99 and 1.43, respectively, are selected as ground motion time histories compatible with the Vancouver UHS while records E6C1, E6C13, E6C15, E6C18 and E6C42 with scaling factors of 0.55, 0.74, 0.56, 0.61 and 1.01 are selected to match the Montreal UHS. The uniform hazard spectra for Vancouver and Montreal as well as the pseudo-acceleration response spectra of the scaled records are shown in Fig. 2 (a) and Fig. 2 (b), respectively.



(a) (b)

Fig. 2 – The uniform hazard spectra and the spectrum compatible ground motions for; (a) western Canada (Vancouver), and (b) eastern Canada (Montreal)

4. Analytical Models

4.1 General model

In the current study, single-story buildings with flexible diaphragms are modelled as deep beams supported by springs. The deep beam represents the diaphragm and the springs represent the LLRS. The earthquake induced inertia forces are generated in the diaphragm (beam) and transferred to the braces (massless springs). The shear force in the diaphragm is resisted by the web of the beam, which represents the diaphragm and its connections, while the chord members along the perimeter of the diaphragm resist the entire bending moment acting along the length of the diaphragm. The properties of the beam/spring models are given in the paper by Tremblay and Stiemer [13] and Humar and Popovski [10]. These properties include the moment of inertia of the beam, I, the effective shear stiffness, G', the mass per unit length of the diaphragm, and the stiffness of the braces, K_B.

Figure 3 is a schematic illustration of the analytical model, where the beam is divided into 6 interconnected elements. For the present study, the beam (diaphragm) is, in fact, divided into 20, rather than 6 interconnected elements. Both the shear and the flexural deformations are taken into account. The mass of the diaphragm is lumped at the nodes at intersections of the elements. The beam/spring model is modeled in OpenSEES software. In all of the dynamic analyses, Rayleigh damping of 5% is assumed for the first and the third modes and direct time step integration of the equations of motion is carried out using a time-step of 0.001s.

Different material behaviors are assigned to the structural members in different portions of the study.

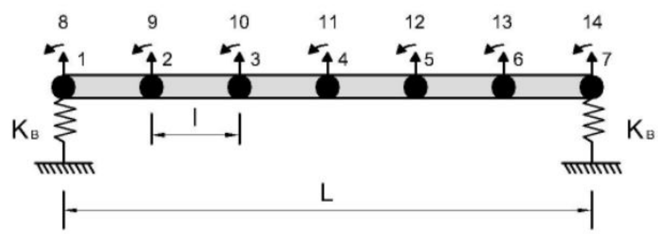


Fig. 3 – Schematic representation of the beam/spring model

4.2 Braces as the energy dissipating system

In order to study the response of single-story buildings with flexible diaphragm in which the LLRS acts as the energy dissipating element, Humar and Popovski [10] and Mortazavi and Humar [9] assigned a linear elastic force-deformation relationship to the diaphragm and a nonlinear elastoplastic hysteretic response with post-yield stiffness ratios of α = 0.00, 0.02, 0.05 and 0.10 to the LLRS.

4.3 Diaphragm as the energy dissipating system

Essa et al [19] showed that steel deck panels exhibit severe pinching and strength degradation in their hysteretic response. They further noted that the behavior was dominated by the fasteners and concluded that diaphragm specimens with screwed sidelap fasteners and weld deck to frame connections showed the most satisfying energy dissipation in their response. The hysteretic force-displacement relationship used in the current study is calibrated against the experimental results obtained by Essa et al. [19]. Fig. 4 compares the calibrated pinched hysteretic response with strength degradation and the hysteretic response obtained in the experimental study by Essa et al [19].

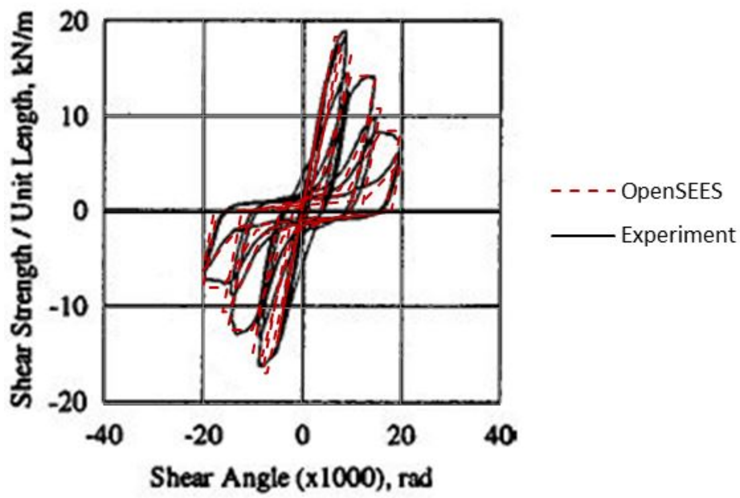


Fig. 4 – Comparison of the calibrated hysteresis relationship for the diaphragm and the experimental results obtained by Essa et al [19] (with permission from ASCE)

In order to investigate the seismic response of single-story buildings in which the flexible diaphragm is relied upon to dissipate the seismic energy, a linear elastic model is assigned to the braces and a pinched hysteretic response with strength degradation, as described above, is assigned to the steel deck diaphragm.

5. Design Expressions for Buildings Relying on Braces for Energy Dissipation

5.1 Increase in the elastic period

In buildings with flexible diaphragms, the fundamental period of the building is increased as a result of the diaphragm flexibility. NBCC 2015 [24] proposes an empirical expression for determining the fundamental period of such buildings. Previous studies, such as Humar and Popovski [1], also provide equations for obtaining the fundamental period of the system.

5.2 Ductility demand on the LLRS

As stated earlier, one of the effects of diaphragm flexibility on the seismic response of buildings with an elastic diaphragm is a significant increase in the ductility demand on the LLRS. This suggests that the force modification factor must be reduced so that the ductility demand on the LLRS does not exceed the target values. Using the analytical model, described in Section 4.2, Humar and Popovski [10] studied the nonlinear seismic response of 33 buildings, selected from the building set described earlier, when subjected to synthetic records that were compatible with the Vancouver spectrum. Response analyses were carried out for each of the two orthogonal directions. Expressions were proposed for determining the appropriate force modification factor associated with each target ductility demand. The analysis has been extended in the current study, by subjecting the same building set to ground motions that are compatible with the Montreal UHS. The results of the analyses provide the value of an adjustment factor κ to be applied to the ductility capacity $\mu_{b,}$, such that, when the brace yield strength is set as the elastic response value divided by $R_y = \kappa\mu_b$, the ductility demand on the bracing system is equal to the target ductility $\mu_{b..}$ Figure 5 plots the value of κ against the drift ratio. Equations (1) and (2) show empirical expressions that relate the force modification factor for the seismic design of the LLRS to the building's drift ratio, for several different values of the target ductility demand. The empirical expressions, which are in fact slightly revised versions of those proposed by Humar and Popovski [10], are seen to provide good representations of the analytical values.

$$R_y = \kappa\mu_b \tag{1}$$

$$\begin{aligned} &\kappa = -0.5r + 1.5 \\ &1 \geq \kappa \geq 0.65 \quad \text{for} \quad \mu_b = 2 \\ &1 \geq \kappa \geq 0.51 \quad \text{for} \quad \mu_b = 3 \\ &1 \geq \kappa \geq 0.44 \quad \text{for} \quad \mu_b = 4 \end{aligned} \tag{2}$$

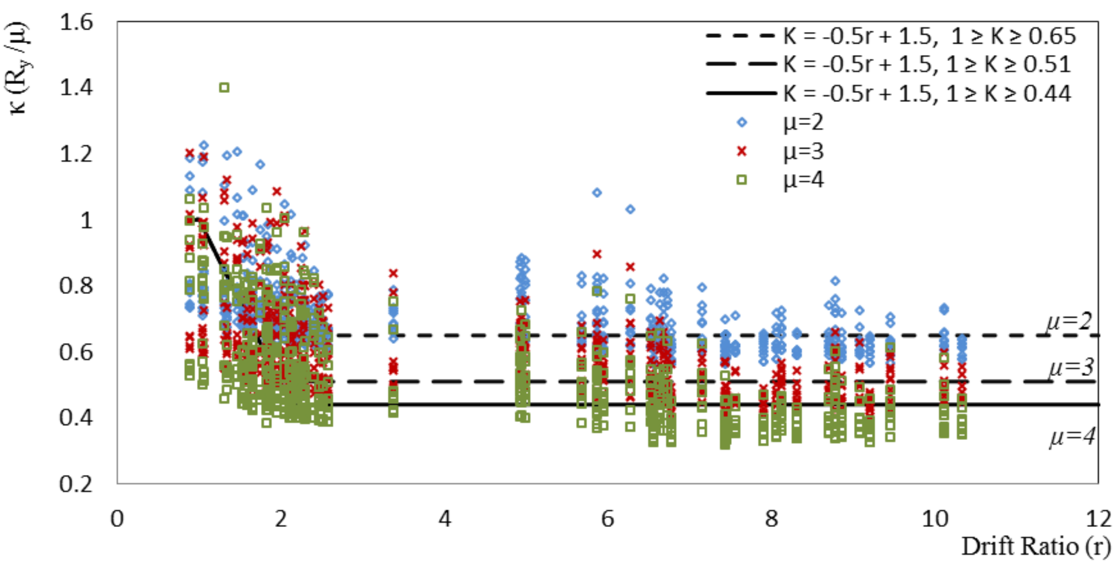


Fig. 5 - Comparison of the proposed equations for prediction of the force reduction factor for the seismic design of the LLRS with the data obtained from the analyses for UHS compatible ground motions for both Montreal and Vancouver

5.3 Magnification of internal force along the diaphragm

In addition to increasing the ductility demand on the LLRS, the diaphragm flexibility increases the internal forces acting along the length of the diaphragm. Specifically, the shear force at quarter span and bending moment at mid span are substantially magnified. Humar and Popovski [10] noted that when the total seismic load is applied using the parabolic distribution given by FEMA 356 [2], reasonable estimate is obtained for of the bending moment at mid span provided the LLRS remains elastic. Eq. (3) shows the parabolic distribution suggested by FEMA 356 [2].

$$f_d = \frac{1.5F_d}{L_d}\left[1 - \left(\frac{2x}{L_d}\right)^2\right] \tag{3}$$

where f_d is the intensity of the inertia force at distance x from the diaphragm center, L_d is the unsupported diaphragm span and F_d is the integrated inertia force acting along the length of the diaphragm.

Mortazavi and Humar [9] analyzed the response of the selected 33 buildings, along both principal axes, to the ten ground motions referred to in Section 3. They determined the distribution of internal forces along the length of the diaphragm for 4 target ductility demands and for 4 post-yield stiffness values of α = 0.00, 0.02, 0.05 and 0.10 assigned to the LLRS. The authors observed that the distribution was affected by the increased contribution of higher modes, which resulted from the flexibility of the diaphragm. The changes in the distribution led to a magnification of the internal forces, and such magnification was amplified with an increase in the nonlinearity in the LLRS. Expressions for predicting the magnification in the internal forces were developed in terms of the following response quantities.

$\boldsymbol{M_f}$ The ratio of the actual bending moment at mid span, obtained from the nonlinear time-history analyses, to the corresponding value obtained by applying the total seismic load to the diaphragm using the parabolic distribution suggested by FEMA 356 [2].

$\boldsymbol{V_f}$ The ratio of the actual shear force at quarter span, obtained from the nonlinear time-history analyses, to the corresponding value obtained by applying the total seismic load to the diaphragm using the parabolic distribution suggested by FEMA 356 [2].

For the sake of completeness the expressions obtained by Mortazavi and Humar [9] are presented here in Eq. (4) through (7). Figure 6 shows a plot of the derived expressions and the corresponding analytical results.

$$\begin{aligned} M_f &= 0.83 + 0.17\mu - 0.68\alpha(\mu - 1) \text{ for } \mu \le 2 \\ M_f &= a\mu + b \text{ for } \mu > 2 \end{aligned} \tag{4}$$

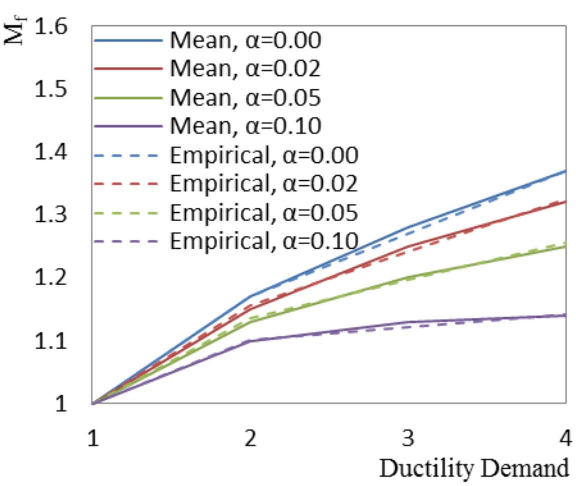
where

$$\begin{aligned} a &= -0.80\alpha + 0.10 \\ b &= 0.92\alpha + 0.97 \end{aligned} \tag{5}$$

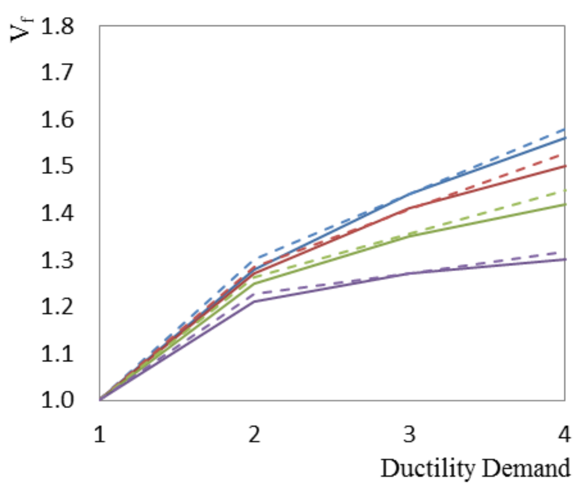
$$\begin{aligned} V_f &= 0.72 + 0.28\mu - 0.70\alpha(\mu - 1) \text{ for } \mu \le 2 \\ V_f &= c\mu + d \text{ for } \mu > 2 \end{aligned} \tag{6}$$

where

$$\begin{aligned} c &= -0.94\alpha + 0.14 \\ d &= 1.14\alpha + 1.02 \end{aligned} \tag{7}$$



(a)



(b)

Fig. 6 - (a) Linear regression fit for M_f equation, (b) Linear regression fit for V_f equation

6. Design Expressions for Buildings Relying on the Steel Deck for Energy Dissipation

In order to derive design expressions for the seismic design of a single-story building with flexible diaphragm in which the steel deck is relied upon to sustain the earthquake energy, buildings SL1, SH1, ML1, MH1, LL1 and LH1 are subjected to the previously mentioned simulated records along both principal axes. Additionally, buildings SH4, ML2, MH6 and LH5 are analyzed for the same set of ground motions acting in the direction parallel to the short side of the building. This is done to ensure inclusion of buildings of all sizes, weights, and drift ratios. The LLRS of the buildings are capacity protected to remain elastic during the seismic events.

6.1 Increase in the elastic period

Evidently, nonlinearity of the diaphragm will not affect the elastic period and, therefore, the same methods as previously mentioned can be used to obtain the elastic period.

6.2 Ductility demand on the steel deck panels

In the first set of dynamic analyses, the steel decks were designed using a ductility related force modification factor of R_d = 1.5. The results indicated that the ductility demands on the steel decks were much greater than 1.5, at times exceeding 7. Response analyses, in which R_d is adjusted iteratively so as to obtain the desired ductility demand in the deck, are carried out. Three target ductility demands of μ_d = 1.5, 2.0 and 3.0 are selected and the corresponding R_d values are determined. The results show that for a given ductility demand correlation exists between the force modification factor and the drift ratio. The following empirical equations are obtained from the analyses.

$$R_d = 0.016\,\mathrm{r} + 1.06 \ \text{ for } \ \mu_d = 1.5 \tag{8}$$

$$R_d = 0.028\,\mathrm{r} + 1.13 \ \text{ for } \ \mu_d = 2.0 \tag{9}$$

$$R_d = 0.051\,\mathrm{r} + 1.32 \ \text{ for } \ \mu_d = 3.0 \tag{10}$$

The design expressions are compared with the analytical results in Fig. 7. As an alternative to the expressions given in Eq. (8), (9), and (10), a single expression shown in Eq. (11) can be used.

$$R_d = 0.58 + 0.024\,\mathrm{r} + 0.29\,\mu_\mathrm{d} \tag{11}$$

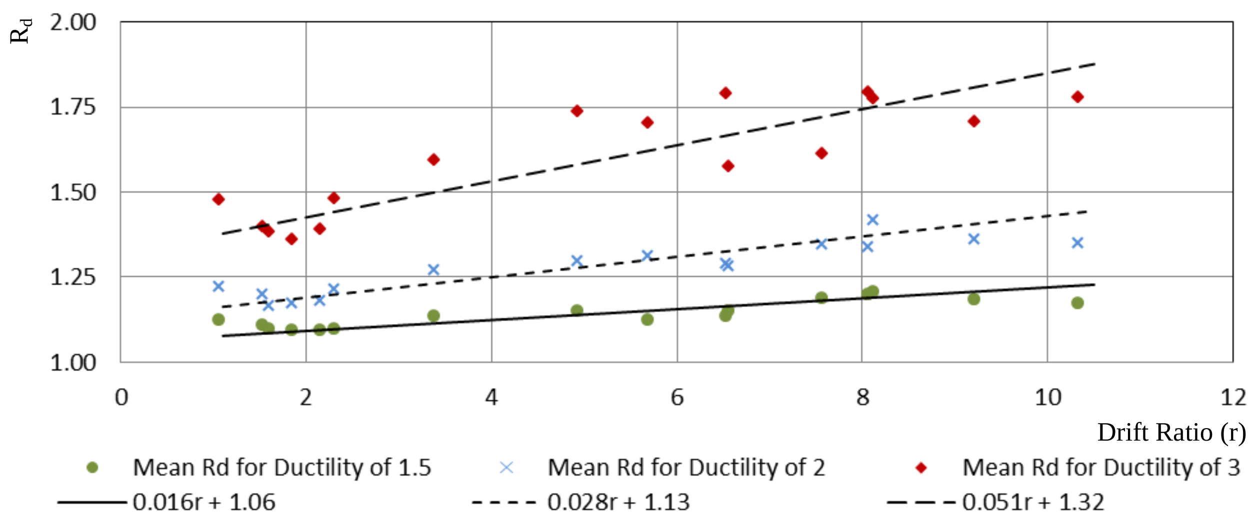


Fig. 7 - Force reduction factors for different target ductility demands: μ = 1.5, μ = 2.0, and μ = 3.0

6.3 Magnification of internal forces along the diaphragm

As pointed out by Humar and Popovski [10], when the braces are designed to remain elastic, the parabolic distribution proposed by FEMA 356 [2] provides good estimates of the internal forces. The results of the analyses indicate that the diaphragm nonlinearity does not increase the magnification of internal forces and, hence, the FEMA parabolic distribution can be used as the means to determine the internal forces acting along the length of the diaphragm.

7. Advantages of Designing the Diaphragm as the Energy Dissipating Element

7.1 Residual deformations

Residual deformations are permanent deformations observed in the buildings at the end of their response to seismic events and are caused by the nonlinear action of the energy dissipating elements. Current codes of practice do not provide methods to predict the residual deformations in buildings designed according to the code provisions. Although life-safety is ensured, the buildings might exhibit significant residual deformations under the design earthquake. In single-story buildings with steel deck panels, limiting the nonlinear behavior to the roofing system will ensure that residual deformations, if any, are confined to the roofing system and the non-structural walls and partitions stay intact during the seismic response. The results of the analyses confirm this assessment. Figure 8 presents examples of the displacement histories of LLRS, both when the inelasticity is confined to the LLRS, and when the diaphragm is the only element strained into the nonlinear range.

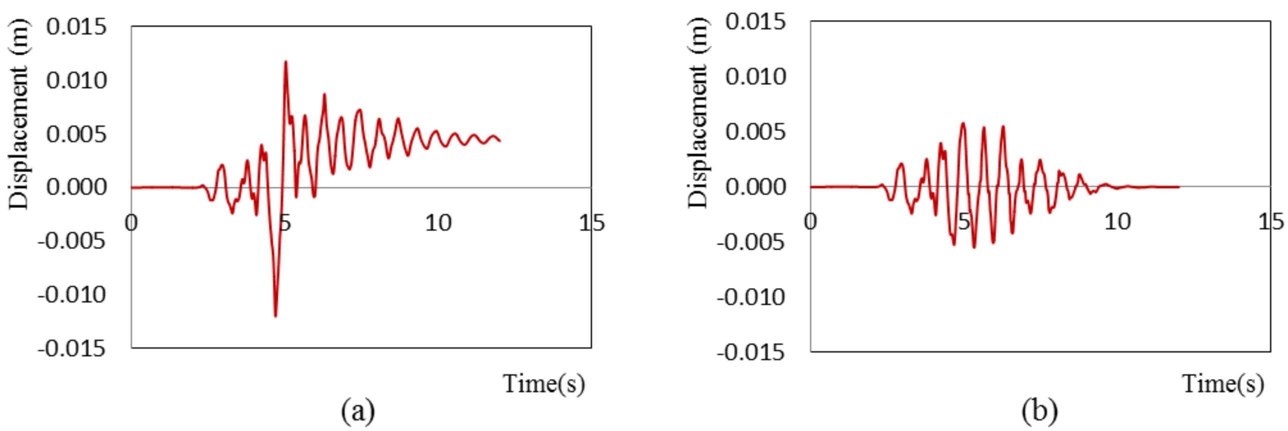


Fig. 8 – Typical displacement time-histories for LLRS; (a) LLRS as the energy dissipating system, (b) diaphragm as the energy dissipating member.

7.2 Cost efficiency

Single-story buildings with steel deck panels are relatively light systems, consequently the seismic induced forces on these structures are comparatively small. When the LLRS, such as steel braces, is designed to dissipate the seismic energy, the slenderness limits provided by the codes of practice such as CSA S16-09 [25], rather than the design forces, will govern the design. This leads to over designed braces [16]. The adoption of a capacity design approach to the design of the diaphragm, as well as the magnification of diaphragm internal forces caused by nonlinearity in the LLRS, means that the design forces for the diaphragm are also larger. All of this leads to an uneconomical design. Allowing the diaphragm to respond nonlinearly and adopting a capacity design approach for the brace, ensures that the slenderness limits on the braces as provided in the codes are automatically satisfied. This gives an overall reduction in the cost.

8. Design of Single-Story Buildings with Flexible Diaphragms

It would be beneficial to develop design guidelines for the design of single-story buildings with flexible diaphragms in which either the diaphragm or the LLRS is expected to perform nonlinearly during seismic events. The following simple procedure can be used for the design of such systems.

1. Determine the fundamental period of the system, taking into account the flexibility of the diaphragm. This can be done either by using the prescriptive empirical expressions given in the code, or by analyzing a model that takes the flexibility of the diaphragm into account. Alternatively, the system can be modeled using the beam/spring model. Humar and Popovski [1] provide simplified expressions for calculating the fundamental period of the system using the beam/spring model.
2. Determine the elastic base shear using the system fundamental period and the uniform hazard spectrum for the site.

$$V_e = S(T)M \tag{12}$$

   where $S(T)$ is the spectral acceleration for the period, T, and M is the total mass assigned to the level of the diaphragm. Equation (12) assumes that the entire mass of the diaphragm participates in the first mode. This is a conservative, yet reasonable, assumption [1].
3. Determine the drift ratio of the system (Δ_D/Δ_B) using an appropriate analytical model. Alternatively, the simple equations provided by Humar and Popovski [1] may be used.
4. Select the desired design approach from the following: (1) LLRS as the energy dissipating element, and (2) diaphragm as the energy dissipating element.

   For buildings with nonlinear LLRS and elastic diaphragm:

   a) Select the target ductility and using Eqs. (1) and (2) determine the ductility related force modification factor.
   b) Divide the elastic base shear obtained in Step 2 by the ductility related force modification factor, R_y, determined in Step (a) and an appropriate over-strength related force modification factor, R_o, to determine the design base shear. Design the elements of the LLRS for the design base shear.

$$V_d = \frac{V_e}{R_y R_o} \tag{13}$$

   c) Using Eq. (14) scale up the design base shear to account for the post-yield strength reached in the brace.

$$V_u = V_d\,(1+\alpha(\mu_b - 1)) \tag{14}$$

   where α is the post-yield stiffness ratio, V_u is the ultimate base shear, and μ_b is the ductility demand on the LLRS.

   d) Distribute the total inertia force along the length of the diaphragm using Eq. (3).
   e) Proceed to Step 5.

   For buildings with nonlinear diaphragm and elastic LLRS:

   a) After selecting the desired target ductility for the diaphragm, determine the appropriate ductility related force modification factor, R_d, for the seismic design of the diaphragm using Eqs. (8), (9), (10) or Eq. (11),
   b) Divide the elastic base shear obtained in Step 2 by the ductility related force modification factor, R_d, determined in Step (a) and an appropriate over-strength related force modification factor, R_o, to determine the design base shear.

$$V_d = \frac{V_e}{R_d R_o} \tag{15}$$

c) Design the elements of the LLRS to remain elastic under the obtained base shear.
d) Use the parabolic distribution of Eq. (3) to distribute the base shear along the diaphragm span.
e) Proceed to Step 5.

5. Use Eq. (4), (5), (6) and (7) to determine the magnification factors to be applied to the mid span bending moment and the quarter span shear force.
6. By applying magnification factors to bending moment and shear force determined in Step 5, determine the critical bending moment at mid-span and the critical shear force at quarter-span and design the diaphragm for these actions. The design shear force for the diaphragm is the larger of the end shear and the magnified shear at quarter span.

9. Conclusion and Recommendations

An analytical study is carried out to investigate the effects of diaphragm flexibility on the seismic response of single-story buildings. The study expands and improves on the previous studies and provides design expressions for the seismic design of single-story buildings with flexible diaphragms for two different cases: (1) when LLRS is the energy dissipating source, and (2) when the flexible diaphragm serves as the source of energy dissipation. The main focus of the study is on investigating the possibility of allowing the steel deck diaphragm to dissipate the seismic energy through inelastic action and to provide guidelines for the seismic design of such systems. The study leads to the following conclusions.

1. When the LLRS is designed to dissipate the seismic energy, diaphragm flexibility lengthens the period of the system, increases the ductility demand on the LLRS and causes an increase in the mid span bending moment and quarter span shear force. The magnification of the internal forces is caused by an increased contribution of higher modes when the diaphragm is flexible. Expressions are provided to account for such behaviors in the design.
2. When appropriately detailed, steel deck panels can be relied upon to dissipate the earthquake energy through inelastic action in the deck. Previous studies have shown that steel deck panels show a pinched hysteretic response with strength degradation when subjected to cyclic loads, and that their response is governed by the behavior of the diaphragm and its connections in shear.
3. The alternative design strategy, in which the diaphragm steel decks are relied upon to sustain the earthquake energy, leads to a more economical design.
4. When the diaphragm is the energy dissipating element, any inelastic deformation is limited to the roof, hence the LLRS does not experience a residual displacement.
5. Expressions are proposed for the seismic design of single-story buildings in which the diaphragm is the energy dissipating system. The current study is based on the cyclic response of the diaphragm having screwed sidelap fasteners and weld deck to frame connections. It will be useful to confirm the findings of the current study for different connection types.
6. Additional experimental studies on the response of steel deck panels to cyclic loadings will be beneficial.
7. Further studies on buildings with different lay-outs, different LLRS (i.e. shear walls), located in different seismic regions and with different roofing systems (i.e. wood deck panels) are recommended to confirm the validity of the results obtained.

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