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Consideration of diaphragm flexibility in the seismic design of one-story buildings

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ABSTRACT

The response of single-story buildings with a flexible roof diaphragm to earthquake excitations is strongly influenced by the flexibility of the diaphragm. Diaphragm flexibility increases the period of the building, magnifies the ductility demand on the lateral load resisting system, and changes the manner in which the inertia forces are distributed along the length of the diaphragm, which leads to a magnification of the internal forces in the diaphragm. An analytical study is carried out to examine the influences of diaphragm flexibility on the seismic response of one-story buildings. The magnification in bending moment and shear force is investigated. An alternative approach to the seismic design of one-story buildings with flexible diaphragms is studied. In such approach, the diaphragm is designed to act as the energy dissipating system while the braces are designed to remain elastic. The effects of the nonlinearity of the diaphragm on the seismic response of the system are examined and methods are proposed to account for the structural implications associated with designing the diaphragm to act as the energy dissipating system.

1. Introduction

The roofing system, also referred to as a diaphragm, is an important component of the structure of a one-story building. The diaphragm resists the gravity loads imposed on the roof through it's out of plane stiffness. Another important role of the diaphragm is the distribution of lateral loads imposed by wind, earthquake, and blast among the elements of the lateral load resisting system (LLRS) of the building. For the distribution of such loads, the roofing system relies on its in-plane stiffness. The in-plane stiffness of the roof diaphragm relative to the stiffness of the LLRS greatly influences the response of the structure to lateral loads. Based on the ratio of the in-plane stiffness of the roofing system to that of the LLRS, codes and design guidelines have classified the diaphragms into three categories, namely: (1) flexible, (2) rigid, and (3) stiff. FEMA [1] classifies a diaphragm as flexible if the ratio of the maximum horizontal deformation of the diaphragm along its length under a uniformly distributed lateral load (Δ_D) to the average lateral displacement of the LLRS of the story immediately below the diaphragm (Δ_B), known as the drift ratio r, is equal or >2.0. The diaphragm is classified as rigid if the drift ratio is equal or less than 0.5. For drift ratio values that lie within the range of 0.5 and 2.0, the diaphragm is classified as stiff.

In one-story buildings in which the gravity loads are comparatively small, roofing system usually consists of un-topped steel deck panels or wood structural panels. Such structural systems exhibit low in-plane stiffness compared to reinforced concrete slabs or steel panels with concrete topping. Consequently, in such cases the diaphragm could be categorized as a flexible diaphragm.

A large number of studies have been carried out on the seismic response of buildings with flexible diaphragms, particularly one-story buildings. Humar and Popovski [2] provide brief descriptions of several such studies.

The common approach to the design of one-story buildings with flexible diaphragms is to design the diaphragm to remain elastic and have the members of the LLRS such as braces, and shear walls dissipate the seismic energy imparted to the system through inelastic deformation. The diaphragm, collectors, chords, struts and their connections are then designed not to yield under the ultimate load reached in the LLRS. Humar and Popovski [2] and Mortazavi and Humar [3] have examined the response of single-story buildings with flexible diaphragms in which the LLRS consisting of concentric

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steel braces exhibits an elasto-plastic force displacement relationship.

In a recent study, Tremblay et al. [4] have noted that, when properly designed, the diaphragm can exhibit satisfactory ductility and have investigated the possibility of allowing the roof diaphragm panels and the connections to act as the energy dissipating members. If this approach is found acceptable, the flexible roof diaphragm would be relied upon to sustain the lateral load as it is strained into the inelastic range. Even when the LLRS is designed to dissipate the seismic energy, so that the ductility of the LLRS can be used to reduce the design seismic shear, the slenderness limit specified by the steel design code, such as, CSA S16-09 [5] and AISC 2010 [6], often results in overdesigned brace members. This in turn leads to overdesign of the diaphragm, which has to be capacity protected. Thus, allowing the diaphragm to be strained into the inelastic range could lead to a more cost-efficient design procedure [4]. It also offers the advantage that the residual displacements, if any, are confined to the roofing system and the building stays intact for post-hazard occupancy. In fact, several building codes and design guidelines provide provisions for the seismic design of one-story buildings with diaphragm nonlinearity. For example, NBCC 2015 [7] allows the diaphragm to be designed as the energy dissipating system under certain conditions.

Based on a parallel study, FEMA has issued a guideline, designated as FEMA P-1026 [8], which provides recommendations on the seismic design of single-story rigid wall-flexible diaphragm (RWFD) buildings. The guidelines recognize that in such buildings the diaphragm is usually the pre-dominant energy dissipating system and is expected to yield during the design earthquake. An alternative procedure is suggested for the seismic design of such structures. It is a two-stage procedure in which the diaphragm and vertical structure are designed for forces corresponding to two different periods, one for the diaphragm and the other for the vertical system, and two different ductility and overstrength related force modification factors (R). The procedure has also been incorporated in 2015 NEHRP Recommended Seismic Provisions for New Buildings [9], and it is intended for future codes and standards to adopt it. At present, the procedure is applicable only to diaphragms of wood panels and vertical system of concrete or masonry shear walls.

The current study takes a different approach to the seismic design of RWFD buildings. The seismic force is computed for the period of the composite structure taking the stiffness of both the vertical system and diaphragm into account [2]. This force is then modified based on the ductility and overstrength in the system. Two different cases are considered, one in which the inelasticity is confined to the vertical system and the other in which it is confined to the diaphragm. In each case, it is expected that the nonyielding component will be designed as a capacity protected system. A study of the case in which both the vertical system and the diaphragm are strained into the nonlinear range is in progress. The application of the procedure suggested here is illustrated for a vertical system consisting of steel braces and a steel deck diaphragm, but with appropriate choice for the R factors, it is equally applicable to systems with shear walls of concrete or masonry and deck of wood panels. Experimental studies on the nonlinear behavior of the steel deck diaphragm panels include those by Rogers and Tremblay [10] and [11], Essa et al. [12], Davies and Bryan [13], and Massarelli et al. [14]. Some of these studies have shown that the nonlinear behavior of the diaphragm, which is governed by its shear capacity, is dominated by the behavior of the fasteners. Essa et al. [12] showed that steel deck panels exhibit severe pinching and strength degradation in their hysteretic response. The result of these studies can form the basis for modeling the hysteretic behavior of steel deck panels.

The study by Humar and Popovski [2] provides the background for the work reported here. In that study the authors carried out time history analyses on a set of 33 representative single-story buildings having a flexible diaphragm for their response to El-Centro 1940 earthquake and five synthetic ground motions compatible with the uniform hazard spectrum (UHS) for Vancouver specified in the 2010 National Building Code of Canada [7]. The authors examined the linear and nonlinear seismic response of the buildings in which the nonlinearity was confined to the LLRS and proposed a method for predicting the appropriate force reduction factor for the seismic design of the LLRS.

The research reported here includes: (1) extension of the study by Humar and Popovski [2] for the UHS for Montreal and refinement of the equations that predict the force reduction factor for the seismic design of the LLRS, (2) determination of the magnification of internal forces acting along the length of a flexible diaphragm, and (3) a study on the response of buildings in which the diaphragm has been designed to be the sole source of energy dissipation.

For the analytical studies reported here the set of 33 buildings, referred to earlier, is subjected to ten synthetic ground motions, five of which are compatible with the UHS for Vancouver and the other five compatible with the UHS for Montreal as specified in the 2010 National Building Code of Canada [7]. For each building, two separate sets of analyses are carried out: for the earthquake motions parallel to the short side of the building, and for earthquake motions parallel to the long side of the building.

To examine the effect of nonlinearity in the diaphragm, appropriate nonlinear shear behavior is assigned to the diaphragm in the finite element model. The design approach in which the diaphragm is strained into the inelastic range while the LLRS remains elastic is investigated.

2. Analytical models

Fig. 1 shows a one-story building in which the roof diaphragm consists of steel deck panels while concentric braces form the LLRS. The figure also shows the displacements Δ_D and Δ_B, defined earlier, that are produced when the diaphragm is subjected to a uniformly distributed static lateral load.

The diaphragm is modeled as a deep flexural beam supported by springs which represent the LLRS. The beam is divided into 20 interconnected beam elements in which the shear deformations as well as the flexural deformation are taken into account. The mass of the diaphragm is lumped at the nodes located at the intersections of the beam elements. The shear force in the diaphragm is resisted by the web consisting of the steel deck and its connections. The chord members located at the boundaries of the diaphragm are responsible for providing almost the entire bending moment resistance. Braced frames, concrete shear walls, masonry shear walls or wood panel walls can form the LLRS. However, in the current study, the emphasis is on concentric braces. Fig. 1 also illustrates a schematic representation of the analytical model in which the beam is divided onto 6 elements rather than 20. The beam/spring system is modeled in OpenSEES [15] platform.

Throughout 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 0.001 s.

3. Buildings selected for the study

The 33 buildings analyzed in this study are selected from a set of buildings designed by Tremblay and Stiemer [16] according to the provisions of the 1995 National Building Code of Canada [7] for