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3-D nonlinear dynamic behavior of steel joist girder structures

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ABSTRACT

As the trend towards developing performance-based design specifications for the seismic design of structures gains momentum, it is clear that very little is known about the performance of light industrial structures under large lateral loads. Among the main outstanding issues related to the seismic design of these structures are (1) the determination of appropriate response modification factors (R, C_d and Ω_0), (2) the establishment of drift limits to avoid damage of structural and nonstructural components, and (3) clarification of the role that the roof diaphragm plays on the seismic behavior of light-weight roof structures. This study attempts to elucidate some of those issues for a particular class of light-weight industrial structures, those composed of one-story, weak column–strong beam joist girder frames. Two types of analysis models were developed for the nonlinear dynamic analyses of these structures. The first is a simplified 2-D analysis model, using SAP2000 and the second is a complex 3-D analysis model, using ABAQUS. Nonlinear time history analyses were performed for sites in Los Angeles (CA), Boston (MA), and Memphis (TN). The accuracy of the simplified 2-D model was verified by comparison with the results from the 3-D model. The results indicate that the behavior of these structures is almost always in the elastic range, and that substantial roof bracing should be installed for this type of structure, to prevent excessive drifts in the weak direction. When two horizontal components of excitations were applied concurrently to check the effect of torsion of the frame, it was found that torsional effects were negligible for structures regular in plan, and that a 2-D model can provide reasonable analysis results. Column base fixity effects on the dynamic behavior were also investigated and it was determined that column base fixity should be considered, to obtain more accurate dynamic behavior of the steel joist girder structures.

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1. Introduction

Joist girder frame structures consist of repetitive, open, tall, one-story frames with or without additional bracing along the perimeter (Fig. 1). These structures are inherently very flexible, and in the past their design has controlled by drift criteria for wind. Their performance during past earthquakes has been satisfactory, with damage limited to brittle façade elements and poorly detailed column bases [1]. This is in spite of the fact that no specific seismic design guidelines exist for these structures, and that these structures are generally weak column–strong beam systems. With the advent of seismic performance-based design (PBD), there is a need to evaluate the performance of these structures under a wide range of seismic loads, in order to provide rational design guidelines. Among the main issues to be addressed are the determination of response modification coefficient (R) and deflection amplification factor (C_d) for design, and the determination of the drifts associated with different levels of seismic excitations. For these flexible structures, two important analysis parameters are the degree of column base fixity and the amount of diaphragm action on the roof. Both of these have a large effect on both the displacements and forces attracted to the joist girders, as almost all of the seismic mass is concentrated on the first sidesway mode [2]. To study the seismic behavior of joist girder systems, a combined analytical and experimental program was carried out under the auspices of the Steel Joist Institute (SJI) [3]. As far as the authors know, this is the only work available on the seismic performance of moment frames with joist girders, and one of the few to address industrial structures of this type [4].

There are typically four levels of structural analyses conducted for seismic design: linear static, linear dynamic, nonlinear static and nonlinear dynamic. Because of the regularity of the structures and the insignificant influence of higher modes, linear static and nonlinear static (nonlinear pushover) analyses should be sufficient to design a typical steel joist girder structure [5]. Nevertheless, as part of these studies, the following tasks were performed to simulate the actual behavior of the steel joist girder structures under large seismic excitations:

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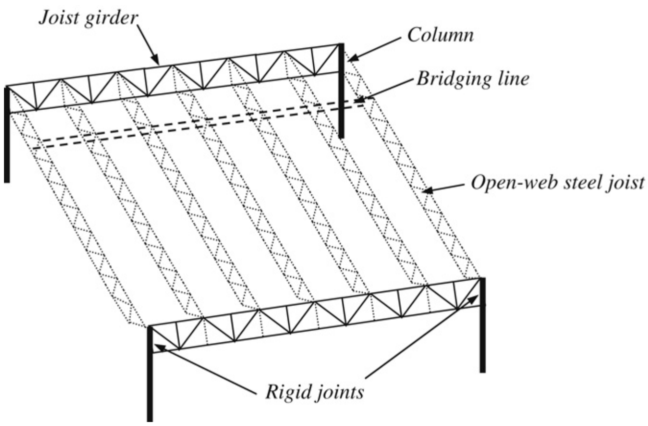

Fig. 1. Typical joist girder structure.

- Develop a simplified, two-dimensional (2-D) analysis model and verify its accuracy and robustness, through a comparison between this 2-D analysis for a Los Angeles (CA) site on firm soil (S_D) and a 3-D nonlinear FE analysis results.
- Perform nonlinear time history analyses for Los Angels for pinned and partially restrained (PR) column base conditions, to assess the effect of base fixity.
- Perform linear and nonlinear dynamic analyses for other two locations (Boston, MA and Memphis, TN) using the simplified 2-D analysis model to determine the nonlinear effects on this type of structures.

2. Input data for time history analyses

Several measured and simulated ground motions for three cities (Los Angeles, Boston and Memphis) were used to perform a time history analyses. For Los Angeles and Boston, the simulated ground motions developed by one of the SAC joint project teams were selected for time history analyses [6,7]. Ten pairs of ground motions with a probability of exceedance of 10% in 50 years were used in this study as shown in Tables 1 and 2. Each pair of ground motions is comprised of two horizontal components, a fault-normal and a fault-parallel component. For Memphis, the simulated ground motions used (Table 3) were developed by the Mid-America Earthquake Center [8], also for a probability of exceedance of 10% in 50 years for a representative soil site. Because these structures are assumed to be designed for a comparatively short life, ground motions with a probability of 10% in 50 years rather than 2% in 50 years were used. From these tables, the average maximum PGA values are 0.59g for Los Angeles, 0.20g for Boston and 0.08g for Memphis, respectively. The average of pseudo-acceleration values at the natural period of the prototype frame in this study ($T = 1.5$ s) are 0.49g for Los Angeles, 0.059g for Boston and 0.021g for Memphis, respectively.

To perform time history analyses, the use of distributed masses were deemed necessary for an accurate 2-D and 3-D dynamic analysis, given the relatively small mass of the structure. The solid circles on the top chord of the joist girder (Fig. 2) indicate the location of the lumped masses. The following parameters were assumed for the mass calculations:

(1) Lumped masses are comprised of 1.0 times dead loads plus 0.2 times snow loads.
(2) A portion of column weight (37%) was incorporated [9].
(3) Dead loads are comprised of built-up roof gravel surface (287.3 N/m^2), roof deck (81.4 N/m^2 for 22 gage deck), insulation (47.9 N/m^2) and mechanical systems (239.4 N/m^2). The total dead loads are 656.0 N/m^2.
(4) Based on the above assumptions, the lumped masses for all three cities are summarized in Table 4.

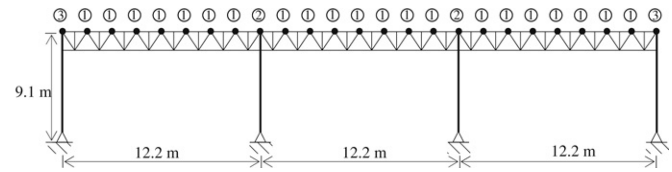

Fig. 2. Elevation of 3-bay frame.

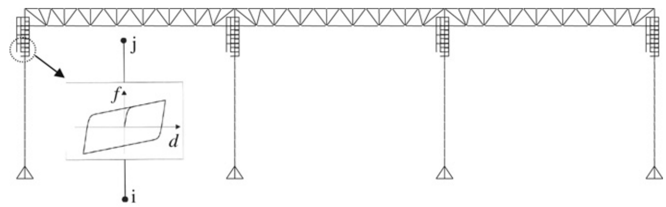

Fig. 3. Two-dimensional model for nonlinear dynamic analysis.

3. Development of the analysis model

Two kinds of analysis models were developed for the nonlinear dynamic analyses. The first was a simplified 2-D analysis model, using SAP2000 [10]. It was used as a compromise between a SDOF system and a complex 3D FE model to reduce the analysis time cost, but yet achieve reasonable results. It was deemed to be as complete as a designer may do as he/she attempts to gain confidence in the performance of these structures. The second is a complex 3-D analysis model using ABAQUS [11] to determine 3-D global behavior and local performance (local buckling, hinge rotations, etc.). It was intended to provide as accurate analytical results as possible. The design of the structures and a comparison between the experimental results and the 2-D model has been presented elsewhere [2]. Table 5 shows the main member sizes for the structures analyzed herein. The structures had periods of 1.56 (Los Angeles), 1.83 (Boston) and 1.64 (Memphis) seconds, respectively. Flexible metal siding with little stiffness and mass was assumed in the design.

The basic assumption of the 2-D model (Fig. 3) is that the joist girder members remain in the elastic range throughout the full range of the ground excitation, and that the plastic zones are concentrated on the column-to-joist connection. These assumptions are based on both observations during the full-scale test and the results of previous analyses [3]. They are considered reasonable, except for extremely severe ground excitation cases. The 2-D model was developed using SAP2000 with NLLINK elements used for the upper region of each column as shown in Fig. 3. One of the NLLINK element's restrictions is that this element can be used for a limited region of plastification only. Thus, in case of globally spreading plastification, the accuracy of the nonlinear analysis results cannot be guaranteed. Again, observations from the full-scale test indicate that plastification will localize over a small distance, probably less than twice the column depth [3]. Thus the assumptions appear reasonable. To verify the stability of frames, a careful consideration of global and local imperfections should be required and recommendation values of imperfections are given in Eurocode 3 [12]. Exceptionally, sway imperfection may be disregarded where horizontal loads are greater than 15% of vertical loads. In this study, imperfections are not incorporated in the analysis according to this special condition, however, imperfections should be considered under routine design procedures to insure that instability issues do not occur [13–15].

To consider global plastification and 3-D effects, a larger model was developed, using ABAQUS (Fig. 4). The span in the transverse direction is 40 ft. The column members were modeled using B32OS elements, a 3-D beam element that uses quadratic interpolation and accounts for warping effects. Each column was comprised