Engineering Structures 141 (2017) 624–633

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Seismic assessment of existing tall buildings: A case study of a 35-story steel building with pre-Northridge connection

Shanshan Wang [a,*], Jiun-Wei Lai [c], Matthew J. Schoettler [b], Stephen A. Mahin [a,b]

[a] Department of Civil and Environmental Engineering, University of California, Berkeley, CA 94720, USA
[b] Pacific Earthquake Engineering Research Center, University of California, Berkeley, 1301 S. 46th street, Richmond, CA 94804, USA
[c] Degenkolb Engineers, 1300 Clay Street, Suite 900, Oakland, CA 94612, USA

ARTICLE INFO

Article history:
Received 3 September 2016
Revised 1 March 2017
Accepted 22 March 2017

Keywords:
Damage and loss analysis
Existing structures
Performance Based Earthquake Engineering
Seismic performance evaluation
Steel moment resisting frames
Tall buildings

ABSTRACT

The Tall Buildings Initiative (TBI) program of Pacific Earthquake Engineering Research Center has been expanded to consider the seismic performance of existing tall buildings. This paper selects a 35-story steel moment resisting frame (SMRF), designed in 1968 with construction details representative of that period, for detailed seismic evaluation in the framework of Performance Based Earthquake Engineering (PBEE). A three-dimensional numerical model capturing the mechanical properties of the most critical structural elements was generated using the program: Open System for Earthquake Engineering Simulation (OpenSees). Systematic nonlinear response history analysis (NRHA) under two basic safety earthquake (BSE) hazard levels for existing buildings were performed following ASCE 41-13 guideline. Probabilistic checks on the confidence levels of the building to achieve collapse prevention (CP) and immediate occupancy (IO) at different hazard levels were conducted based on FEMA 351. In addition, damage and loss analysis was carried out using FEMA P-58 PBEE methodology. Analysis results following different procedures all predicted that the case-study building failed to meet the recommended performance objectives and had a variety of seismic vulnerabilities, and possible retrofits were needed.

1. Introduction

With a resurgence of tall building construction at the beginning of the 21st century, research on seismic performance of high-rise buildings has been an important topic; one of these is the Tall Building Initiative (TBI) program of Pacific Earthquake Engineering Research Center (PEER). The phase I of the TBI has developed background documents and design guidelines for new tall buildings to achieve target performance goals [1,2]. Similarly, more recent works have been taken by researchers worldwide to investigate methods for numerical modeling and seismic analysis of new high-rise structures [3–5].

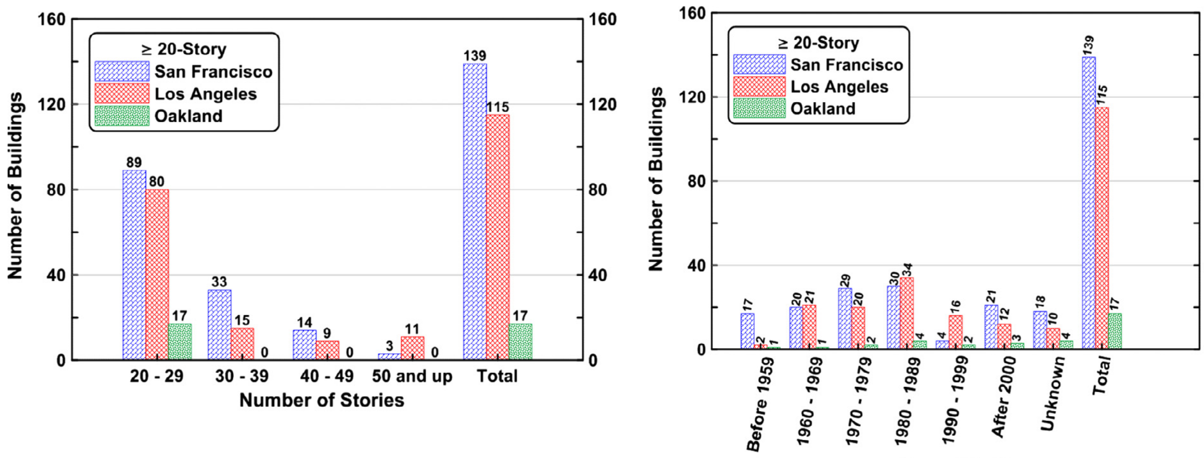
However, many buildings taller than 20 stories were constructed in the west coast of U.S. between 1960s and 1990s (Fig. 1), when understanding of earthquake hazards and structural behaviors were not as advanced as now. Many of these buildings used welded steel moment resisting frames (SMRF) as lateral resistance system. Brittle fractures of beam-to-column connections were observed in nearly two hundred SMRF structures following the 1994 Northridge earthquake. Previous studies have examined these and other SMRFs [6–8], but few of these considered buildings taller than 20 stories. Given the number and importance of these tall buildings, their seismic performance and the feasibility of seismic upgrade, if needed, became the focus of Phase 2 of the TBI. In this paper, a case-study 35-story SMRF that built in the late 1960s was selected for seismic assessment. Several evaluation procedures were used to assess its seismic performance, including ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 41-13) [9], FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* (FEMA 351) [10] and FEMA P-58, *Seismic Performance Assessment of Buildings* (FEMA P-58) [11].

After establishing performance criteria, defining seismic hazards at the site, and constructing a numerical model, evaluation results are presented for two basic safety earthquake (BSE) hazard levels: BSE-1E (225-year mean return period) and BSE-2E (975-year mean return period). These analysis results are interpreted to identify potential structural deficiencies, the likely cost of structural damages, and retrofit strategies.

* Corresponding author at: Department of Civil and Environmental Engineering, University of California, Berkeley, CA 94720, USA.

E-mail addresses: shanonwang@berkeley.edu (S. Wang), jlai@degenkolb.com (J.-W. Lai), mschoettler@berkeley.edu (M.J. Schoettler), mahin@berkeley.edu (S.A. Mahin).

http://dx.doi.org/10.1016/j.engstruct.2017.03.047
0141-0296/



(a) Grouped by number of stories (b) Grouped by year constructed

Fig. 1. Inventory of tall buildings located at three major cities in California (San Francisco, Los Angeles, and Oakland).

2. The case-study building

2.1. Building description

The existing building considered is a 35-story tall steel office building located in San Francisco, California; construction began in 1968. The tower is about 56 m × 41 m in plane, and 150 m in height. Typical beam span is 9.15 m or 9.35 m, and typical floor height is 3.96 m. The building is constructed over three basement levels, and uses moment resisting frames as lateral force resisting system in two horizontal directions. Fig. 2 shows the isometric view, elevation of one frame and plane view of a typical floor of the building model. Dimensions of typical floor height and bay widths are demonstrated. Note that columns are not exhaustively placed at girder intersections, as indicated by the box shape or W-shape in Fig. 2(c).

The steel frame members consist of built-up or wide flange sections. Typical member sizes for one exterior frame are listed in Table 1, with section sketches and notations illustrated in Fig. 3. It should be noted that many of the built-up structural elements used in the case-study building are unable to fully yield in flexure because of the undersized welds used, indicating that the members may yield or fail in shear prior to forming plastic hinges at the both ends. This mode of behavior is not well understood, and the potential shear failure by this mechanism poses a potential vulnerability in existing tall buildings containing built-up sections.

As was typical, a 15.2-cm-thick normal weight concrete slab on a metal deck is provided on top of a metal deck at each floor. Framing irregularities occur at the ground level due to an extra-tall story that includes a mezzanine. The building's foundation consists of a 2.1-m-thick mat located 12.2 m below grade.

Beam-to-column moment connections used were those that were typical for buildings built pre-Northridge. Wide-flange beam-column connection details are shown in Fig. 4. These types of connections are considered very brittle based on field observations and laboratory tests conducted after Northridge earthquake [7,12].

Column splices were erected using Partial Joint Penetration (PJP) welds, located 1.5 m above the lower floor level. A representative detail for a wide flange column splice is shown in Fig. 5. Such PJP welds range from a quarter to a half of the thickness of the members being joined and are not expected to be able to develop the nominal capacity of the net welded section due to considerations of fracture mechanics [13].

Heavy concrete cladding is provided around the perimeter of the building. These add weight and mass to the building, but not stiffness or strength.

Member sizes, structural detail drawings, foundation details and other information were collected from the building owner and the City of San Francisco's Department of Building Inspection.

2.2. Performance objectives

ASCE 41-13 provides guidance on selecting the basic performance objectives of existing building (BPOE). These depend on the Risk Category of the building and the evaluation procedure used. A Risk Category of III is selected for this office building, considering its functional importance and the number of occupants. According to ASCE 41, the first two tiers of its three-tier evaluation approach are not needed, and a Tier 3 evaluation based on dynamic analyses is required. The BPOE criteria considered in the Tier 3 procedure are damage control (DC) at the BSE-1E hazard level and limited safety (LS) at the BSE-2E hazard level.

2.3. Ground motion selection and scaling

Ground motion selection and scaling were completed with the assistance of Prof. Jack Baker from Stanford University. Several sets of ground motions at a site very near the case-study building were developed based on various code requirements and probability of exceedance levels; each consisting of 20 three-component records. Ground motions were extracted from the PEER NGA West2 database, and no more than five ground motions were taken from any single record. Ground motions were selected such that the distance to rupture was less than 50 km, the magnitude of event was 6.5 or larger, and the amplitude scale factor needed was less than 9. A selection and scaling algorithm was proposed that allows the user to select a set of ground motions whose response spectra match a target mean and variance [14]. All three components of each ground motion were scaled by a same scale factor. Fig. 6 shows the horizontal response spectra of the selected ground motions and target spectra at BSE-1E and BSE-2E hazard levels.

2.4. Mathematical modeling

A 3-dimensional building model was constructed using the program: Open System for Earthquake Engineering Simulation (OpenSees) [15] to investigate the nonlinear dynamic behavior of the