Engineering Structures 32 (2010) 1123–1132

Contents lists available at ScienceDirect

Engineering Structures

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

An innovative passive control technique for industrial precast frames

Paolo Martinelli *, Maria Gabriella Mulas

Department of Structural Engineering, Politecnico di Milano, 20133 Milan, Italy

ARTICLE INFO

Article history:
Received 10 August 2009
Received in revised form
2 December 2009
Accepted 22 December 2009
Available online 21 January 2010

Keywords:
Passive control
Friction device
Seismic analysis
Industrial buildings
Precast RC structures

ABSTRACT

A feasibility study is presented on the extension to reinforced concrete (RC) precast industrial buildings of a passive control technique aiming to provide additional damping to the structure and previously proposed for RC and steel framed structures. This technique, based on the insertion of friction devices in the region of beam-to-column connections, is particularly advantageous due to the reduced dimension and low cost of the devices adopted. In the case of precast buildings, it offers the additional advantage of providing ductility to the hinged connections. In this work a general criterion for the device calibration is proposed; the modeling strategy for the device within a well-known simulation platform is presented. A prototype building is designed for a medium-to-high seismic risk and the device is fine-tuned for this particular building. The device efficiency in modifying favorably the structural behavior is analyzed by comparing the seismic response of the bare and redesigned frame to twelve accelerograms having a PGA equal to the design value. A non-critical shear increase in the zones where the device is inserted is found, largely counteracted by the reduction in extreme values for top displacement, bending moment at the column base and amount of energy dissipated in the hysteresis of materials.

1. Introduction

Precast reinforced concrete (RC) structures are largely adopted for industrial buildings. Several advantages have favored this construction technique, such as the possibility to cover large areas due to the adoption of prestressed beams, the ease and reduced time in erection, the quality of materials. However, when compared with traditional RC buildings, it must be noted that for precast structures the design and construction of connections is still a crucial point. In fact, the connections of cast-in-situ structures present a higher degree of continuity between adjacent elements, also due to passing reinforcing steel; in case of seismic events, connections represent a source of energy dissipation and allow for a redistribution of internal forces. These advantages are absent in many precast buildings, especially in those with industrial applications, where the typical one-story frame is composed of a beam simply supported by two columns. A higher degree of constraint can be given in situ, but this would require a costly supplement of work, thus reducing the advantages of precasting.

Passive control techniques aiming to energy dissipation can offer an innovative solution to this problem. Different from active control techniques, these are based on the adoption of relatively simple devices and do not require external energy input, drastically reducing maintenance costs. As stated by [1], passive control techniques “are characterized by their capability to enhance energy dissipation in the structural systems in which they are installed”; the energy dissipation can be due either to transformation of kinetic energy in heat or to the transfer of energy among normal modes. The first approach is pursued with energy dissipating devices (EDDs), based on friction, yielding of metals, deformation of viscoelastic solids and fluids [2,3,1,4]. The second approach includes supplemental oscillators (tuned mass dampers) that act as dynamic absorbers [2,3,1]. Previous works have shown that the application of friction or yielding EDDs in moment resisting frames, close to the zones where high ductility requirements are expected during a seismic event, is an effective redesign approach. Studies have been conducted first with reference to RC frames [5–7] and then extended to steel frames [8,9].

In this work an innovative passive control technique for precast industrial frames is proposed, with the aim of providing ductility and a moderate degree of continuity for the connections. A modified version of the friction EDD adopted in [5–9] is proposed to be inserted around the beam–column connection. At present only a limited number of studies have analyzed the effectiveness of EDDs at the beam–column connection of precast industrial buildings [10,11]. The extension to these buildings of the positive findings described in [5–9] is not straightforward, due to the deeply different aim of the original redesign technique, developed for RC and steel moment resisting frames. The validity of this proposal is numerically analyzed on a case-study of a 1-story prototype building.

* Corresponding author. Tel.: +39 0223998785; fax: +39 0223998771.
E-mail address: pmartinelli@stru.polimi.it (P. Martinelli).

0141-0296/$ – see front matter © 2009 Elsevier Ltd. All rights reserved.
doi:10.1016/j.engstruct.2009.12.038

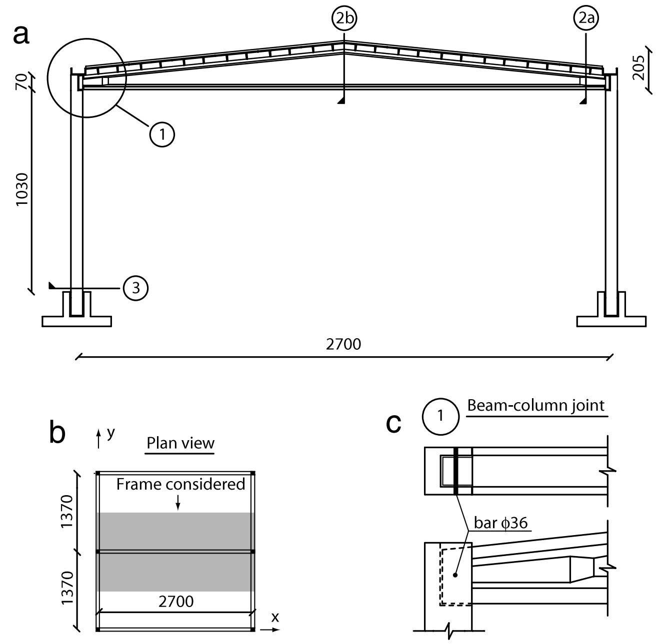

Fig. 1. The structure at study: (a)–(b) front and plan view; and (c) detail of the beam–column joint (units cm, ϕ in mm).

In the following, the building geometry, loads, materials and design criteria will be described. The EDD adopted will be presented, together with the general criteria needed for its calibration and the modeling strategy. The numerical results of the dynamic analyses under a set of twelve selected ground motions will show the satisfactory performance of the redesigned frame incorporating the devices, with respect to the bare frame without devices.

2. Prototype structure: Geometry, materials and loads

The 1-story single-bay frame considered in this study (Fig. 1(a)), is the central frame of a typical precast industrial building having 3 single-bay frames in x direction and 2 double-bay frames in y direction, spaced at 13.7 m (Fig. 1(b)). The behavior of the building in y direction is not considered here, assuming that the structure is sufficiently braced in this direction by RC panels. The central frame in x direction is composed of a prestressed double slope beam lying on columns; the roof is composed of precast double-T units simply supported by the beams. The bay length is 27 m while the story height is 10.3 m. The square columns have dimension 0.6 m × 0.6 m. The beam height varies from 0.7 m at the support to 2.05 m at mid-span.

To avoid the unsettlement of the beam from its supports during seismic excitation, the main cause of collapse of precast buildings [12], a dowel connection is inserted at the beam–column joint, as shown in Fig. 1(c). A pocket foundation solution is adopted for the precast columns. Fig. 2(a) and (b) show the reinforcement details for the beam and the columns, respectively. The stirrups spacing varies in columns from 5 cm close to the beam connection, to 10 cm at the base and 20 cm elsewhere, and in the beam from 5 cm close to the supports to 30 cm at mid-span.

The frame has been designed for gravity loads and for seismic actions following prescriptions of Eurocode 2 (EC2) and Eurocode 8 (EC8) [13,14], respectively. Design has been performed on the basis of the following assumptions: (a) concrete having cylindrical compressive strength of $f'_c = 45$ MPa and Young's modulus $E_c = 33$ GPa; (b) reinforcing steel having yielding stress $f_y = 430$ MPa and Young's modulus $E_s = 205$ GPa and prestressing steel with $f_y = 1860$ MPa and $E_s = 195$ GPa; (c) structure of medium ductility class (DCM) corresponding to a behavior factor $q_p = 3.3$; and (d) design spectrum type 1 corresponding to a ground soil type C and a peak ground acceleration (PGA) equal to $0.25g$, associated to a damping ratio of 5%, typical for concrete structures. The value of PGA assumed in design is associated to the zone 2 of the Italian seismic risk map (medium-to-high seismic risk).

In determining the part of gravity loads directly acting on the central frame, a tributary area equal to 27 m × 14 m (gray area in Fig. 1(b)) is assumed, resulting in a dead load on the beam equal to 56.7 kN/m, given by the sum of floor dead load of 45.2 kN/m and of beam average own weight of 11.5 kN/m. When computing masses for the seismic design, the in-plane rigid roof assumption was considered, so the tributary area is equally subdivided on the three frames. The design of the frame was governed by the requirements of capacity design philosophy. Further details on the frame design can be found in [15].

2.1. Ground-motion selection

In the case of nonlinear dynamic analysis, EC8 requires a minimum of 3 accelerograms if the most unfavorable seismic response is to be considered, or a least 7 accelerograms, if the seismic average response is to be considered. However, no guidelines are given to the engineer to determine how to choose the accelerograms that are appropriate to a given design situation. In this work, aiming to study an innovative design solution, a set of 12 natural accelerograms was selected for the nonlinear time history analyses. Considering a larger number of records does not produce significant variations in the average values of the response parameters at study. The accelerograms used in this study have been selected from the PEER NGA database [16]. They come in couples from six well known and destructive earthquakes from all the world (Friuli 1976, Irpinia 1980, Chalfant 1986, Loma Prieta 1989, Northridge 1994, Kobe 1995). The main characteristics of the accelerograms are listed in Table 1. The accelerograms were then modified to be compatible with the elastic spectrum of EC8 [17] and normalized to the PGA adopted in design. The acceleration response spectra of the scaled accelerograms and the target design spectra for 5% damping are given in Fig. 3. The signal number and the associated signal name used in this study are listed in the first two columns of Table 1.

3. Energy dissipating device

The criterion to select an EDD for the prototype building at study, among the many proposed in the literature [3,1,4,18–20], was based on two requirements of (a) adopting a device of small dimensions and (b) enhancing significantly the structural damping, with a limited increase in the structural stiffness [4]. The first requirement has the practical advantages of an easy installation with a low impact on both the electrical system and the interruption of an overhead traveling crane. The second requirement leads to reduced increments in base shear and thus in building weight. Both requirements should produce negligible alterations to building layout and avoid a significant amount of construction work. These considerations led to exclude viscous devices of great dimensions, more apt to bracing structures. On the other hand, several metallic or friction devices satisfy the above mentioned requirements and, in particular, rotational friction devices, concentrating in a single point the formation of a "plastic hinge".

The EDD proposed in this work is a rotational friction device incorporated in the region of the beam–column connection as shown in Fig. 4(a). The L-shaped EDD adopts the same geometry of a device previously proposed for RC and steel framed structures [5–9] and is composed of two UPN200 channel sections