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Seismic performance of wide flange steel beams partly covered by side plates

Cheng-Cheng Chen, Erwin *, Charles Julius Salim, Rudy Tiara

Department of Civil and Construction Engineering, National Taiwan University of Science and Technology, No. 43, Sec. 4, Keelung Rd., Taipei, Taiwan

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ABSTRACT

In this study, the seismic performance of wide flange steel beams partly covered by side plates was investigated experimentally and analytically. The beams investigated were laterally supported at top flange but not at bottom flange. Multiple cases were tested including beams with and without side plates, and the results showed that the beams with side plates showed acceptable performance in terms of strength, ductility, and hysteresis behavior. The lateral torsional buckling was suppressed in beams with side plates. A finite-element model was established and validated using the test results. A parametric study was conducted, which revealed that the lengths of the side plates played an important role in determining the performance of the beams. The side plate length required to achieve the desired performance depended on the strength of the side plate used, beam slenderness ratio, and beam section dimensions such as beam depth-to-width ratio and width-to-thickness ratio of the flanges and web. A design guideline for beams partly covered with side plates was proposed based on the results of the parametric study. The proposed equation to calculate the required side plate length provides a result that is precise enough compared to the analytical result with slight conservatism.

1. Introduction

In a building with relatively long beam span, the moment capacity of a wide flange steel beam subjected to earthquake loading is mostly controlled by lateral torsional buckling (LTB) when it is not properly braced laterally. Fig. 1 shows a properly braced beam in the field. The top flange bracing is provided by floor slab while bottom flange bracing is provided by angle section connecting the bottom flange and floor slab. This kind of beam system has become very common and its LTB behavior had also been extensively discussed in several books [1–3] and articles [4–28]. Most of the previous studies [6–21] focused on finding reliable procedures for determining the elastic or inelastic LTB strengths of beams with and without lateral braces. Some studies [22–26] have revealed that both the stiffness and the strength of the lateral brace should be considered in the design of a braced beam. All these studies were based on experimental or analytical results regarding to the beams subjected to monotonic loading. However, Nakashima et al. [27] confirmed that the lateral instability effects of beams subjected to monotonic and cyclic loading differ significantly. The out-of-plane deformation is generally more severe under reversal loading. The LTB behavior of a wide flange steel beam under earthquake-type loading had already been investigated by Okazaki et al. [28]. In all of these researches, the LTB problem was solved by using lateral braces spaced at a certain limited distance which was determined based on the desired ductility demand.

A problem arises when the presence of lateral brace to the bottom flange is not desired considering the space usage in a building or aesthetic issue. Some engineers were trying to solve the LTB problem by adding side plates to the wide flange beam as shown in Fig. 2. The use of side plates can significantly increase the torsional rigidity of the beam section because it changed from open to closed section. The new breakthrough made in this paper was the idea to only add the side plates at region near beam end where the moment demand was higher and had higher potential to undergo LTB when subjected to earthquake loading. The middle span of the beam was left uncovered by side plate. The LTB behavior of this kind of beam system had never been studied at all in the past. In future, this kind of beam system will be called as beam partly covered with side plates.

Here, the LTB behavior of beams partly covered with side plates under cyclic loading is discussed; this discussion assumes that other possible brittle failure modes are mitigated. The two main purposes of this paper are (1) to experimentally confirm the effectiveness of using side plates to prevent LTB and (2) to establish a design guideline for beam partly covered by side plates. Therefore, both experiment and analysis are described. A finite-element (FE) model was established and verified by comparisons of computer solutions with existing experimental results; the FE model was used in an extensive study of parameters affecting the required length of the side plates. The design equation was derived based on the results of the parametric study.

* Corresponding author.
E-mail address: d10205805@mail.ntust.edu.tw (Erwin).

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0143-974X/

Fig. 1. Beam with lateral bracing.

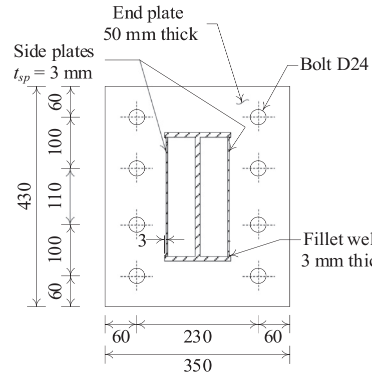

Fig. 3. Cross section of beam with side plates (unit: mm).

2. Experimental program

2.1. Specimens

Four beam specimens—designated as HSP-0, HSP-30, HSP-40, and HSP-50, with length L of 4400 mm—were tested under earthquake-type loading. The beam section used was H250 × 125 × 6 × 9. Both ends of each beam were welded to 50-mm-thick end plates, as shown in Fig. 3, using full penetration weld. HSP-0 was the beam without side plate, as shown in Fig. 4, whereas HSP-30, HSP-40, and HSP-50 were beams with side plates, as shown in Fig. 5. Four edges of the side plate were each fillet welded to the tip of the beam top and bottom flange, end plate, and transverse stiffener. The thickness of the fillet weld was 3 mm. The thicknesses of the side plates and transverse stiffener used were 3 and 6 mm, respectively. The number following each specimen name represents the percentage of L covered by the side plates. For example, HSP-30 indicates that the beam has side plates with length of 0.15L at each beam end. All steel materials used were ASTM A36, and the actual mechanical properties of the steel obtained from the tension coupon test are listed in Table 1. To represent the floor slab casted above the beam top flange, lateral braces, with a nominal axial stiffness of 126 kN/mm, were provided at the top flange of the specimens by connecting them to the plates, denoted as "CP" in Fig. 4-6. It is noted that a preliminary FE analysis had been conducted prior to the experiment and the results showed that the hysteretic behavior was similar when the lateral brace stiffness was larger than 15 kN/mm. On the other hand, typical reinforced concrete slab can provide lateral stiffness much larger than this value.

2.2. Experimental setup and instrumentation

Fig. 7 presents a 3D view of the entire test setup. Both ends of each beam specimen were connected to rigid columns through end plate connections. The flexural rigidity of the rigid columns was more than 20 times greater than that of each specimen. The columns were considered to be rigid in the data analysis. The bottom end of each rigid column was connected through a pin connection to a floor beam, which was secured to the floor. To reduce the axial loads induced on the test specimens, a load transfer beam was used. The lateral load was introduced to the load transfer beam through a servo-controlled actuator with a capacity of 500 kN. The actuator was equipped with a built-in load cell to measure the applied lateral load P. The test setup results for the moment distribution of a specimen are illustrated in Fig. 8. The moment at beam end M was computed using Eq. (1). The stability columns, reaction beam, transverse beam, transverse bracing, and horizontal bracing were arranged such that the entire frame was kept stable and only the rigid columns could transfer the horizontal load to the base beam.

$$M = P \times H \times (L/L_{cc}) \tag{1}$$

A linear variable differential transformer, designated as L1, was used to measure the lateral displacement Δ of the frame at the mid-height of the specimen. The rotation angle at the beam end θ was assumed to be equal to the drift ratio, which can be calculated using Eq. (2). The specimen was loaded under displacement control using the loading history presented in Fig. 9. The loading amplitude was started from elastic cycles, which comprised six cycles each for loading amplitudes of 0.00375, 0.005, and 0.0075 rad. The loading was then continued for four cycles with an amplitude of 0.01 rad, and then followed by two cycles each with increasing amplitude values from 0.02 to 0.08 rad in 0.01 rad increments. The test was stopped when this loading procedure was successfully finished or when the beam strength decreased to 80% of the corresponding peak strength.

$$\theta = \Delta / H \tag{2}$$

A 3D coordinate optical measurement instrument was used to measure the out-of-plane displacement at top flange u_t and bottom flange u_b of each specimen at the positions indicated in Fig. 4 and Fig. 5, respectively. Due to the measurement range limitations of the device, only part of each beam was monitored. The twist angle ϕ of the specimen, as illustrated in Fig. 10, was computed using Eq. (3).

$$\phi = \frac{u_b - u_t}{h} \tag{3}$$

Fig. 2. Beam with side plates.

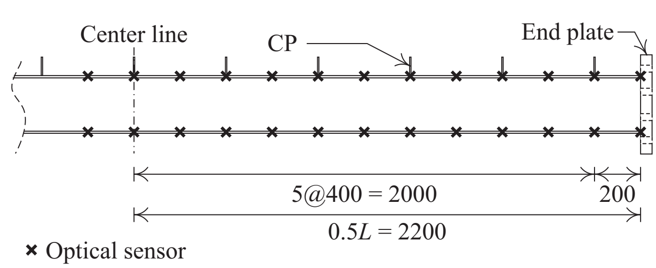

Fig. 4. Specimen HSP-0 (unit: mm).