Built-up battened columns under lateral cyclic loading

Dipti R. Sahoo, Durgesh C. Rai*

Department of Civil Engineering, Indian Institute of Technology, Kanpur, 208016 UP, India

Received 20 July 2006; accepted 21 February 2007
Available online 31 May 2007

Abstract

Built-up members with battens designed for typical 2–2.5% of axial load may not behave satisfactorily in the presence of lateral seismic loads. Analytical evaluation of double-channel battened cantilever members designed as per the current practice, and subjected to constant axial compressive load and gradually increasing lateral load showed that the members failed to reach their expected flexural capacity due to lateral instability. The design of members was modified by changing the configuration of battens in the expected plastic-hinge region, i.e., reducing the spacing of battens in end panel by half, and designing battens for a shear demand due to moment capacity of section. The members with battens designed for moment capacity could able to reach the expected flexural strength. Five half-scale test specimens of battened members designed as per the current practice and improved design method were subjected to axial load and gradually increasing cyclic load. The specimens designed as per improved design method showed excellent performance in terms of lateral strength, lateral stiffness, moment rotation characteristics and energy dissipation capacity.

Keywords: Battened; Column; Seismic; Buckling

1. Introduction

A built-up battened column is a kind of compression member consisting of two identical longitudinal elements slightly separated and connected to each other at only a few places along their length by means of battens. These members are frequently used as light compression members, such as struts in truss moment frames and as columns in light steel structures. Double-channel sections are often used as built-up battened columns.

Built-up columns are significantly weaker against shear deflections as compared to solid columns. The reduction in axial buckling strength of these columns due to the shear deflection depends on the configurations and the structural connections of the connectors. Many researchers [1–3] have developed analytical criterion considering the effects of axial load, shear deformation and connection rigidity. In addition to the shearing effect, the axial buckling strength may also be reduced due to compound buckling, where the localized buckling of components between the battens interacts with the global buckling mode [4].

End plates in battened members distribute the applied forces or moments to the component elements and may contribute significantly to the axial buckling strength. An earlier study of a special battened column, in which the battens are attached to the longitudinal elements by hinged connections, revealed that the axial buckling strength of the member without end plates is no greater than the sum of the critical loads of individual elements [5]. In this case, the battens do not transmit shear between the component elements. However, the buckling load with end plates is the lower bound buckling strength of battened member.

During seismic overloads, the built-up columns are subjected to excessive compressive strains due to lateral loads in addition to axial compressive load. Under large lateral deformations, plastic hinged connections are formed at critical sections of the member, where the maximum moment is expected. Ductility and energy dissipation capacity of members greatly depend on the performance of these plastic hinges. The smaller width-to-thickness ratio of members could reduce the severity of local buckling,
leading to an increase in ductility and energy dissipation capacity [6,7].

The battens of built-up columns are designed for a shear demand of 2.0–2.5% of total axial load on the column as per the current practice [8,9]. These columns may not behave satisfactorily when subjected to lateral loads due to earthquake events in addition to axial compressive load. In the present study, a nonlinear finite element (FE) analysis, using ABAQUS [10] was carried out to study the effect of configurations of battens on behavior of battened members subjected to constant axial compressive load and gradually increasing lateral load. In addition, the behavior of half-scale double-channel built-up battened specimens subjected to constant axial load and gradually increasing cyclic lateral load was investigated experimentally. The objective was to develop a design of ductile built-up battened beam column, which can reach its expected flexural strength without any instability.

2. Analytical evaluation

Battens of a built-up member are generally subjected to shear force and bending moment due to both axial and lateral load. For design purposes, it is assumed that the points of contra-flexure of the main components and battens occur at their mid-points. The configuration of battens depends on their center-to-center spacing, which is governed by the criteria that the slenderness ratio of the main component over the distance between the battens shall not be greater than 50 or 0.7 times slenderness ratio of the battened member as a whole about the axis parallel to the battens [8]. A flow chart for the design of battens of a built-up beam column is given in Fig. 1.

2.1. FE study

Built-up battened cantilever members subjected to constant axial compressive load and gradually increased lateral load were studied analytically using a nonlinear FE package, ABAQUS [10]. The members consisted of two hot-rolled Indian standard channel sections (ISMC 200) as the main components and mild steel plates of 10 mm thickness as battens. The battened member with loadings and section properties of main components are shown in Fig. 2.

Four different designs, namely, current practice, design option-I, -II and -III were considered in the analysis of built-up beam columns of equal slenderness ratio. In the current design practice, battens were designed for a shear demand of 2.0% of axial compressive load and spacing of

![Fig. 1. Flow-chart for design of battens of built-up members.](image-url)
the battens was uniform for all panels of the member. The spacing of battens in the panel near fixed end was reduced by half and an additional batten of the same configuration was placed at the mid-way of the panel in design option-I. In case of design option-II, all the battens were designed for a shear demand due to the reduced plastic moment capacity \((M_{pc})\) of main components and other battens were designed as per the current practice in design option-III. The properties of double-channel built-up battened members are given in Table 1 and the details of configurations of battens of members designed as per the current practice and design option-II are given in Table 2.

Four-noded doubly curved shell (S4R) elements with 6-degrees of freedom at each node were used to model main components and battens. The verification of mesh was carried out by the element failure criteria, i.e., the face corner angle less than 30°. The material property was considered as elasto-plastic (Young’s Modulus = 200 GPa, Poisson ratio = 0.25, and yield stress = 250 MPa) and geometric non-linearity (i.e., large deformation) was included in the analysis. The FE models with three different configurations of battens analyzed in this study are shown in Fig. 3.

2.2. FE results

Nonlinear static collapse analysis was carried out for the built-up battened FE models preloaded with axial compressive load. Four levels of axial compressive loads, namely, 15%, 30%, 45% and 60% of axial yield load, were considered. Table 3 compares the peak lateral loads carried by the battened FE models at different levels of axial load. The models designed as per the current practice carried lesser lateral loads at each axial load level as compared to models designed as per the three improved design methods. The lateral load carrying capacity of the model designed as per design option-I was found to be slightly higher than the models designed as per the current practice. The increase in lateral strength of models designed as per design option-II and -III was about 30–50% of that designed as per the current practice. However, the models designed as per design option-II and option-III carried nearly same peak lateral loads for all levels of axial load.

2.3. Comparison

The plastic moment capacity \((M_p)\) of a member is reduced in the presence of axial compressive load \((P)\). The

![Fig. 2. Built-up battenected beam column considered in this study.](image-url)
reduced moment capacity ($M_{pc}$) for the value of axial load ($P$) greater than or equal to 15% of axial yield load ($P_y$) can be given by the following expression [8,9]:

$$M_{pc} = 1.18M_p \left( 1 - \frac{P}{P_y} \right).$$ \hspace{1cm} (1)

Eq. (1) is primarily intended for determining the reduced moment capacity of beam column of I-section bending about its major axis. However, this relation can be applicable to the double-channel built-up beam column. The axial load-bending moment interaction for the built-up battened beam column is derived analytically assuming that the components can reach their plastic moment capacity without any instability. The analytical result matched very well with the interaction curve given by Eq. (1) as shown in Fig. 4.

The flexural strength of the FE models at different levels of axial load was compared with the design moment capacity given by Eq. (1) as shown in Fig. 5. FE models with the battens designed as per current practice failed to
reach their expected flexural capacity of components. However, the models designed as per design option-II and option-III showed higher plastic moment-carrying capacity and reached the design moment capacity of beam column for axial load of 15% and 30% of yield load. As the axial load in the models increased beyond 40% of yield load, the moment capacity was found to be less than the design moment capacity. The reason may be due to the fact that as the axial load level in the member increased, the moment capacity of the member decreased. Hence, the shear demand \( V_b \) on the battens was reduced requiring smaller depth of battens. Since, the designers generally restrict the axial load in the member to the 40% of yield load, designing the battens for the axial load beyond this limit is of lesser importance. The model designed as per design option-I could not reach its expected flexural strength; however, it carried the higher lateral load as compared with the model designed as per the current practice.

3. Experimental evaluation

Half-scale test specimens of double-channel built-up battened cantilever members were investigated experimentally to study the behavior under constant axial compressive load of 32% of axial yield load and gradually increased cyclic lateral load. The main parameters of interests were the spacing and the configuration of battens in the expected plastic hinge region of built-up battened members.

3.1. Test specimens

Five test specimens with three different configurations of battens consisted of ISMC 200 were used as main components and mild steel plates as battens. The minimum specified yield stress of channel sections and mild steel plates was 250 MPa. The overall length of all specimens was 1.32 m and the cyclic lateral loading was applied at a distance of 1.2 m from the fixed end. All specimens were of equal slenderness ratio.

The specimens were designated as DC1C, DC1MR, DC2MR, DC1MB and DC2MB. DC represents double-channel battened built-up specimens and numeric 1 or 2 stands for number of specimens tested experimentally. C, MR, and MB represent the design of battens as per the current practice, modified with reduced spacing and modified with ‘box’ configuration, respectively (Fig. 6). The battens of specimen DC1C were placed at uniform spacing of 490 mm center-to-center; whereas an additional batten of the same configuration was placed at mid-way of the panel near fixed end of specimen DC1MR. Specimen DC1MB had a very wider batten near the fixed end so that a ‘box’ configuration was formed in the expected plastic hinge region. The details of configuration and spacing of battens of specimens are given in Table 4.

3.2. Test setup

Test setup shown in Fig. 7 consisted of a reaction frame, two double-acting servo-hydraulic actuators, and a reaction block. The axial compressive load was applied by means an actuator (force rating = \( \pm 500 \text{kN} \) and stroke length = \( \pm 125 \text{mm} \)), whereas the cyclic lateral load was applied by means of another actuator (force rating = \( \pm 250 \text{kN} \) and stroke length = \( \pm 125 \text{mm} \)). Both the actuators were connected at the free end of the specimen, whereas the fixed end of specimen was connected to a reaction block. Four linear variable differential
transducers were used to measure the lateral displacement of specimens. The state of strain at the mid-points of webs and flanges of channel sections near fixed end and mid-points of end battens were measured by means of strain gauges.

3.3. Displacement history

A simple, multiple-step displacement history (Fig. 8) consisting of symmetric cycles of increasing amplitude was used for the purpose of assessment of seismic performance of battened specimens [11]. The displacement (or drift ratio) levels in the displacement history were \( \pm 3 \text{ mm} \) (0.25%), \( \pm 6 \text{ mm} \) (0.5%), \( \pm 9 \text{ mm} \) (0.75%), \( \pm 12 \text{ mm} \) (1%), \( \pm 15 \text{ mm} \) (1.25%), \( \pm 18 \text{ mm} \) (1.5%), \( \pm 21 \text{ mm} \) (1.75%), \( \pm 30 \text{ mm} \) (2.5%), \( \pm 40 \text{ mm} \) (3.3%) and \( \pm 60 \text{ mm} \) (5%). The displacement cycle at each excursion level was repeated for three times to get the repetitive behavior of the specimens at a particular level.

4. Experimental results

The cantilever test specimens were subjected to both axial compressive load of 450 kN (32% of axial yield load) and cyclic lateral load at free end. Experimental results presented in the following sections illustrate how lateral load–lateral displacement response, lateral stiffness, moment-curvature response and energy dissipation capacity of built-up battened cantilever specimens of equal slenderness ratio was influenced by different configurations of battens near the fixed end.

4.1. Lateral load–lateral displacement response

Fig. 9 shows the lateral load–lateral displacement response of the test specimens. The elastic behavior of
Fig. 9. Lateral load–lateral displacement response of specimens.
the specimens was observed up to a drift ratio of 1.25%, which is approximately linear in the hysteretic curve. Yielding ($Y$) may be defined as an event when significant departure from linear behavior of specimen was observed in the load–displacement curve. Yielding of the specimens was observed in the form of cracks in the white wash at a drift ratio of 1.5%, which corresponds to a lateral load of 48.5 kN. Local buckling of components of the specimen DC1C was observed in the panel near fixed end at a drift ratio of 1.8% and the global lateral buckling of the specimen was observed at a drift ratio of 3.1% with closing of the gap between the main components in the panel near fixed end. The maximum lateral load carried by the specimen DC1C was 68.5 kN, which was about 70% of its expected flexural strength. The maximum lateral loads carried by specimen DC1MR and DC2MR were 87.6 kN and 71.9 kN at drift ratio of 3.2% and 2.5%, respectively. Local buckling of components of specimens DC1MB and DC2MB was observed in the region just after the 'box' configuration at a drift ratio of 3.3%. The maximum lateral loads carried by the specimens were 99.0 kN and 94.4 kN, respectively, at drift ratio of 3.4% and 3.2%, respectively. The details peak lateral loads and peak lateral displacements of test specimens are given in Table 5.

The hysteretic curves showed the higher lateral load carried by the specimens in negative lateral displacement excursion level. The reason may be due to additional work done by the specimens against the force of gravity, when the specimen was displaced in upward direction. Therefore, the recorded lateral load value was higher in one direction than the other direction. Hence, the lateral strengths of the test specimens were obtained considering the average of lateral load in both directions so that the effect of gravity load was nullified.

4.2. Lateral stiffness

The lateral stiffness of specimen was calculated as the ratio of lateral load required to produce unit lateral displacement. The initial stiffness of specimen DC1C was found to be 4.5 kN/mm, whereas that of specimen DC2MB was found to be 6.5 kN/mm. The increase in initial stiffness of DC1MR as compared with specimen DC1C was 20%. Similarly, the increase in initial stiffness of specimen DC2MB was 32%. As the spacing of battens in the expected plastic hinge region decreased, the lateral stiffness of battened built-up beam columns increased and specimens with ‘box’ configuration in the expected plastic hinge region showed higher lateral stiffness. The lateral stiffness of specimens was decreased with increase of cyclic excursion level and the stiffness at the failure was about 40% of the corresponding initial lateral stiffness of specimens.

4.3. Moment–curvature response

The curvature at a section of specimen was calculated from the strains recorded by the strain gauges. The strains corresponding to the peak lateral displacement of an excursion level were used to determine the curvature of the specimen. The ideal moment–curvature response of double-channel sections was obtained assuming the material to be elasto-plastic with a yield stress of 250 MPa. Fig. 10 shows the actual moment–curvature response of specimens along with the ideal moment–curvature response of double-channel section. Specimen DC1C showed small curvature of $1.5 \times 10^{-5}$ with small moment capacity. The maximum curvature-attained specimens DC1MR and DC2MR was about $3.3 \times 10^{-5}$, which was about twice of specimen DC1C. However, the specimens failed to reach its expected plastic moment-carrying capacity. In contrast, specimens DC1MB and DC2MB reached the curvature of $5.3 \times 10^{-5}$, which was about 3.5 times of specimen DC1C. In addition, the specimens DC1MB and DC2MB reached expected plastic moment capacity of the sections. The slope of the ideal moment–curvature curve was higher than the slope of moment–curvature response of test specimens. This was due to the effect of shear flexibility of battens, which was not considered for ideal moment–curvature curve.

4.4. Energy dissipation capacity

Fig. 11 shows the cumulative energy dissipated by the specimens at different cyclic excursion levels. The energy...
dissipated at an excursion level was normalized to the maximum energy dissipated due to a maximum lateral displacement of 40 mm and flexural strength of 100 kN. The energy dissipated in an excursion level was calculated by taking average of energy dissipated in three cycles of the displacement level. Specimen DC1C showed the least energy dissipation capacity in each cyclic excursion level as compared to other specimens and could reach about 65% of the ideal energy dissipation. The energy dissipated by the specimens DC1MR and DC2MR was more than specimen DC1C and about 80% of the ideal energy dissipation. However, specimens DC1MB and DC2MB showed higher energy dissipation capacity as compared with other specimens.

5. Comparison of experimental and FE results

The experimental results are compared with the FE analysis results. Three FE models, namely, FEM1, FEM2 and FEM3 similar to the specimens DCC, DCMR and DCMB, respectively, were analyzed. Four-noded shell elements were used to model main components as well as battens of built-up members. Axial compressive load of 450 kN (same value as applied in experiment) was applied at the free end, distributed equally at eight points symmetric to the cross-section that would eliminate the effect of eccentric loading on the member. Static analysis was carried out for the axial compressive load and the state of the models at the end of analysis was considered as initial conditions for the gradually increased lateral displacement. Nonlinear static collapse analysis was carried out in the present study to predict maximum load-carrying capacity as well as the post-buckling behavior of batten members.

5.1. Load–displacement response

Fig. 12(a) compares the experimental load–displacement response of specimens designed as per current practice with the FE study. The maximum lateral load carried by the battened member was about 83.0 kN in the FE analysis, which was about 20% higher than the experimental value. Both experimental and FE studies showed exactly same
lateral load and lateral stiffness for small lateral displacements. For the higher lateral displacements, the FE analysis predicted the higher lateral loads than the experiment. This may be due to the presence of residual stresses in the components developed by welding, absence of absolute fixity of the fixed end and unintentional eccentricity of loading in the test specimen, which was not considered in the FE analysis.

The built-up battened members with reduced batten spacing in the panel near fixed end carried a lateral load of 91.5 kN in FE analysis, which was about 12% more than the both experimental value. Fig. 12(b) shows the comparison of experimental and FE load–displacement response of the battened members having reduced batten spacing. The load–displacement behaviour of both the test specimens was comparable with the analytical study.

Fig. 12(c) shows the comparison of experimental load–displacement response of battened members with the box configuration in the panel near fixed with the FE study. The average maximum load carried by the specimens was about 96.7 kN as compared with 104 kN in the FE study. Both the experimental and FE load–displacement results matched very well. The difference in the lateral loads carried by the specimens in the experimental study as well as the FE study was about 7%.

5.2. Mode of failure

The mode of failure of battened member as observed in experimental and FE studies is shown in Fig. 13. The local buckling in form of depression in the webs and bulging of flanges of battened member was observed in the panel near the fixed end. Thus, the initial gap between the main components was reduced as shown in Fig. 13(i). The local buckling in form of outward bulging of webs of main components of battened members with reduced spacing of battens was observed in the panel between the batten near fixed end and the additional batten of the specimen. However, the initial gap between the main components of the member was maintained after the failure. The plastic hinge of battened member with ‘box’ configuration shifted away from the fixed end. The local instability of components was not observed within the ‘box’ region. The mode of failure of battened members observed in the experimental study matched very well with the FE study.

Fig. 13. Comparison of mode of failure of battened members.
6. Conclusion

The following conclusions can be drawn from the present study:

(i) The battened double-channel beam columns designed as per current code provisions could not reach their expected flexural strength due to local buckling of components. The uniform spacing of battens of battened members resulted in low plastic rotation, energy dissipation, and ductility. The built-up members with reduced spacing of battens in the expected plastic hinge region performed better in terms of flexural strength, lateral stiffness, plastic rotation and energy dissipation. However, the members with ‘box’ configuration in the expected plastic hinge region, using very wide battens, was able to reach their expected flexural strength and exhibited excellent performance in terms of plastic rotation, and energy dissipation capacity.

(ii) The design of battens should be based on the moment-shear interaction of the member. However, designing all the battens for the shear demand due to moment capacity requires higher depth of battens and the battens remain ineffective as the distance from the fixed end increases. Hence, designing only the battens near the fixed for a shear demand due to moment capacity of member and other battens for a shear demand of 2% of axial load as per current practice is sufficient to reach the expected plastic moment-carrying capacity of the member.

References