Shear Strength Analysis of Reduced Beam Section (RBS) on Castellated Beam

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Abstract. Shear strength analysis of the reduced beam section (RBS) on the castellated beam with exterior connection aimed to analyze the structural elements to determine the shear force acting due to cyclic loading. The study was developed from the theory of moments due to shear forces based on the American Institute of Steel's Prequalified Connection Construction for Special and Intermediate Moment Frames for Seismic Applications (AISC 358-10). This study used an experiment of RBS on a castellated beam with an exterior connection model subjected to cyclic loading. The result shows that the shear strength occurred after the bending failure of the beam.

1. Introduction
Seismic design for steel moment frame is aimed to make structure elastic during small to medium seismic events. In large earthquakes, the structure must be strong and secure from collapse. Investigation of the 1994 Northridge earthquake and the earthquake in Kobe show that the damage occurred to connections to the frame structure during the earthquake. [1] reported that the crack model on the Northridge Earthquake began at the point of intersection between the weld access hole and the beam bottom flange. In Kobe Earthquake, [2] described the fractures occurred on metal welding, heat-affected zones, base metals, and diaphragm plates.

Many experiments and analysis were done to improve the performance of the connection. One popular method is the reduced beam section (RBS). In connection with the RBS method, the beam flange is trimmed to ensure the flexural mechanism strategy in the desired RBS on the beam. Meanwhile, [3] explains that beam with RBS experienced failure caused by bending on the beam. In the study, the top of the beam had a maximum tensile while the bottom of them had compressive stress. Moreover, the shear stress on the beam web was larger than on the beam flange.

[4] examined the performance of frame beam connection with RBS against fires by numerical methods. The results show that the strength of axial forces and deflections was generated in beam connections with the RBS method. The first yield in the RBS region was well exposed to fires locally and global, which shows the bending failure on the beam.

In a study conducted by [5], the connection shows a reliable ductility to maintain the desired inelastic deformation with no brittle fracture. In the test, the three specimens showed damage to the...
beam flanges while the RBS area did not have faults on the column-beam welded connection. Thus, the behavior of the castellated beam without RBS with an opening distance of 15 cm from the column face is strong enough as it can deform and withstand a load of 12.5 kN. Meanwhile, with a load of 10.9 kN the power decreased by 12.8%. With ductility of 4.1, the castellated beam has the ability to withstand cyclic loading [6].

This paper used experiments of the reduced beam section. The resulting shear analysis of the RBS on the castellated showed sufficient shear capability, where the shear force occurred after the bending force.

2. Reduced beam section (RBS)

Reduced beam section (RBS) is a modification of the beam cross-section by trimming the flange at a specified distance from the column face. The reduction was made in a way that all melting and plastic joint occurred in the RBS. Moreover, the trimmed area also could reduce the moment on the column and control the inelastic deformation on the column. According to AISC 358-10 subchapter 5.8 [7], the procedure of designing RBS is as follows.

\[
R = \text{Radius of Cut} = \frac{4c^2 + h^2}{8c}
\]

Figure 1. RBS design on beam flange.

Determination of RBS geometry (Figure 1) is limited by:

\[
0.5 bf \leq a \leq 0.75 bf \\
0.65 d \leq b \leq 0.85 d \\
0.1 bf \leq c \leq 0.25 bf
\]

with:

- \(bf\) = width of beam flange
- \(d\) = depth of the beam

3. Calculation of shear force in the center of RBS (\(V_{RBS}\))

The shear force was determined by calculating the effect of yield moment on the center of RBS (\(M_{RBS}\)) and the shear effect of a combination of gravity (\(w\)) and earthquake force. The loading combination used was 1.2D + \(f_1L + 0.2S\).

Figure 2. The free body used to calculate \(V_{RBS}\).
The maximum moment that may occur on the column face can be calculated from the free body diagram of the moment in the RBS center as can be seen in Figure 2. Based on the diagram, the moments on the column face are as follows:

\[ M_f = M_{pr} + V_{RBS}S_h \]  

With:
- \( M_f \) = the maximum moment that may occur on the column face (N-mm)
- \( M_{pr} \) = the plastic moment that may occur (N-mm)
- \( V_{RBS} \) = the maximum shear force of two shear forces in the center of RBS at each end of the beam (N)
- \( S_h \) = \( a + b/2 \) (mm)

The plastic moment on the beam based on the expected yield strength was calculated as follows.

\[ M_{pe} = Z_b R_y F_y \]  

with:
- \( M_{pe} \) = plastic moment based on the expected yield strength (N-mm)
- \( Z_b \) = modulus of the plastic cross-section (mm³)
- \( R_y \) = ratio of the expected yield strength to the minimum yield strength
- \( F_y \) = minimum yield strength (MPa)

The \( M_f \) must be lower than \( \Phi_d M_{pe} \). If not, then the value of \( c \) was increased of the values of \( a \) and \( b \) were decreased.

\[ M_f \leq \Phi_d M_p \]  

with:
- \( M_f \) = maximum moment that may occur on the column face (N-mm)
- \( \Phi_d \) = reduction factor of the maximum ductility
- \( M_{pe} \) = plastic moment based on the expected yield strength (N-mm)

Meanwhile, the ultimate shear force of the beam was calculated using the following equation.

\[ V_u = \frac{2 M_{pr}}{L'} + V_{grav} \]  

With:
- \( V_u \) = the ultimate shear force of the beam
- \( L' \) = distance between the center points of RBS (mm)
- \( V_{grav} \) = the beam shear force from \( 1.2D + 1.6L + 0.2S \)

The thickness limit of column flange was calculated using the following equation.

\[ t \geq \frac{(d_z + w_z)}{2} \geq \]  

with:
- \( t \) = thickness of column flange (mm)
- \( d_z \) = thickness of the panel (mm)
- \( w_z \) = width of the panel (mm)

The ratio of column moment to the beam was calculated using the following equation.

\[ \Sigma M_{pb} \approx \Sigma (M_{pr} + M_v) \]  

with:
M_{pr} = \text{plastic moment that may occur (N-mm)}

M_{V} = V_{RBS} (a + b/2 + d_c /2)

V_{RBS} = \text{the maximum shear force of two shear forces in the center of RBS at each end of the beam (N)}

d_c = \text{column height (mm)}

4. Experimental design

The welding procedure was done with a system that was considered semi-rigid on the plate and connection with bolts and endplates as shown in Figure 3. The exterior joint was subjected to the additional cyclic loading as specified. It was set at the end of the beam with the displacement of the roller joint. The test was adopted to support the lateral loads near the actuator, which aimed to prevent lateral deformation of the beam and damage to the actuator. Global deformation of the specimen was measured during the test to determine the component of the beam end deformation that was contributed to the column.

5. Results

The identified parameters were the plastic moment, local buckling, and yield stress. These parameters were used to analyze the strain and stress distribution resulting in a plastic limit on the beam with RBS. Based on the loading and straining data, the first yield point of the steel material \((f_y)\) was obtained at 350 MPa, divided by 200,000 steel elasticity to obtain the value of 0.0017. The first yield point occurred at the strain of 0.0017 or as shown in Figure 4, the first yield point occurred at 1700 µ. The yield point was connected with perpendicular lines drawn from the P load.

Figure 3. Setup of the experiment.
As can be seen in Figure 4, the yield point occurred at the strain of $1700 \times 10^{-6}$ at the load of 9,015 kN, deflection of 32.86 mm at the push-positive direction and the load of -9.015 kN, deflection of 32.82 mm at the pull-negative direction. This shows that the first yield point occurred in the upper flange with the strain value reaching 1692 $\mu$, then moved to the lower flange at the load of -9.015 kN. The results show that the damage occurred in the column face due to the pure bending of the beam, and did not indicate failure at the connection of the beam and column [6].

As can be seen in Figure 5, the yield point on the web occurred on the shear area, with the strain yield point occurred on the load of 12.04 kN on the push-positive direction. This caused an increased load that led to shear failure on the beam. Figure 6 shows the peak load at 12.045 kN reached deflection of 97.5 mm that caused shear failure and buckling on the beam web.
The results show that the initial failure was caused by the bending on the beam, then it spread to the beam web due to the shear force as displayed in Figure 7. The failure in the RBS area was caused by buckling due to the shear force. This is similar with the study conducted by [3], where beam with RBS experienced failure caused by buckling on the beam with the shear stress on the beam web was larger than on the beam flange.

6. Conclusion
The use of reduced beam section on the beam flange could suppress the effect of cyclic loading; thus, the damage did not occur on the connection of beam and column flange. The damage only occurred on the column face. The experiment results show that shear failure occurred on the column face of RBS area. This means that the castellated beam had a buckling on beam web due to the cyclic loading, which caused a damage to the RBS area.

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