Experimental studies on cyclic behaviour of steel base plate connections considering anchor bolts post tensioning

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ABSTRACT: This paper presents the experimental tests on cyclic behaviour of the base plate connections that are connected to the foundation with and without fully post tensioned anchor rods. The main aim is to evaluate these connections that are designed with available design procedures from the low damage aspect. Also, the effect of post tensioning on the seismic performance of this type of connection is presented. To characterize the base plate connection damageability, each column base was designed for a particular major inelastic deformation mode such as anchor rod yielding, yielding of the column, or column and base plate yielding. It is shown that considered joints are not able to be categorized as "a low damage". Also, post tensioning of the base plate increases the rotational stiffness of the base, and results in more ductility of the column with low axial force.

1 INTRODUCTION

The poor performance of steel frame base plate connections was revealed after Northridge (1994) and Hyogo-ken Nanbu (1995) earthquakes. Midorikawa (1997) carried out the statistical analysis of the structural damage for Hyogo- ke Nanbu (1995) earthquake. In this earthquake, column base plate connection damage occurred more commonly than other structural elements. Many studies have been conducted to improve the seismic performance of the base plate connection (Astaneh et al. 1992, Fahmy et al. 1999, Gomez et al. 2010). However, since these elements are not replaceable, any damage or permanent deformation here can result in building demolition. So, there is a need to develop the base connections that sustain almost no damage during a major earthquake.

Most of the work to develop a low damage steel structure is focused on the moment resisting beam to column joint (Clifton 2005, MacRae et al. 2010) and the brace (Chanchí Golondrino et al. 2012). However, even with these features, a building with damaged column bases may not be a low damage building. So, a study to assess and develop low damage base connections is being conducted at the University of Canterbury. In the first part of this project, the effect of the base flexibility (Borzouie et al. 2013) on the demand of the structure was studied. The second part of this project is the experimental tests on cyclic behaviour of the base plate connections that are connected to the foundation with and without fully post tensioned anchor rods. The main scope of these tests is to evaluate the performance of the base plate connection. Here, the base plate connections are designed in such a way that each of them covers a specific yielding pattern such as yielding of the anchor rods, column yielding and combination of the base plate and the column yielding in order to present a comprehensive view of the available base plate connections.

Past studies (Kanvinde et al. 2012, Gomez 2010) show that allowing anchor rod elongation can increase the base rotational stiffness. Also, column base rotational stiffness can increase frame displacement (Borzouie et al. 2013). Moreover, anchor rod elongation can be reduced by the post tensioning of the anchor rod for low axial force column bases, and it increases the initial rotational stiffness. So, post tensioning of the base plate connection to the foundation can significantly affect the behaviour. This paper aims to answer the following questions:

1. Can available base plate connection be categorized as a low damage?
2. What is the effect of anchor rods post tensioning on the seismic performance of the base plate connection?

2 EXPERIMENTAL PROGRAM

2.1 Test specimens

Three base plate connections were tested under cyclic loading in the strong axis of the column with 310UB 46.2 section. Since the moment resisting frames are typically drift governed, UB or HB sections are more common for moment resisting frame. No axial force was applied on the column to consider the performance of these connections and column under light axial force. The base plates are designed for 60% of yielding moment of the column according to AISC method (Fisher and Kloiber 2006). Also, the base plate and the foundation are connected by threaded rods M24 and length of 650 mm (full length is threaded) that were anchored at the bottom of the foundation block. These specimens did not have a shear key and friction between base plate and foundation only resisted the shear force.

The main characteristics of the specimens are summarized in Table 1. The first base plate connection, BC1, was designed such that the base plate and the threaded rods which are used as anchor bolts yielded before the plastic hinge formed in the column. The dimension of the second base plate, BC2, is similar to BC1. However, the Grade 8.8 threaded rods were replaced with Grade 10.9 in order to change the yielding pattern from yielding in the anchor rod to a combination of base plate yielding and column yielding. Finally, the third joint, BC3, was fully post tensioned to the foundation block with 6M24 Grade 10.9 rods. The number of anchor rods and post tensioning values were calculated such that no uplift occurred before column yielding. This post tension value was about 60% of the anchor rod's proof load.

The column to the base plate connection consisted of partial joint penetration (PJP) welding together with fillet welding. For the partial joint penetration welding, the bevel groove weld was carried out from the outside flange face up to 85% of the flange thickness (10mm). Moreover, 12 mm fillet welding was carried out in the inside face of the flanges and the web. This combination of PJP and fillet welding provides a total throat length 80% larger than the flange's thickness. Welding terminology is not consistent with New Zealand practice, and this detail was selected based on the design practice survey for steel moment resisting frame in US (Gomez et al. 2010). Also, Myers et al. (2009) stated from experimental tests with different types of welding between column and baseplate, that this detail is effective for high seismic performance.

Table 1. Specimens' Characteristics

Specimen	Anchor rod	Base Plate	Rod Tightening
BC 1	4M24 Grade 8.8	Base plate No1. 32X300X470	Snug-tightening
BC2	4M24 Grade 10.9	Base plate No1. 32X300X470	Snug-tightening
BC 3	6M24 Grade 10.9	Base plate No2. 32X300X470	Torque wrench (post tensioned)

2.2 Test Setup

One reaction frame causing bending about the strong axis of the column, and the other reaction frame provided lateral restrained for the column with two actuators, which allowed the column to move in-plane only. The test setup and specimens' dimensions are shown in Figure 1.

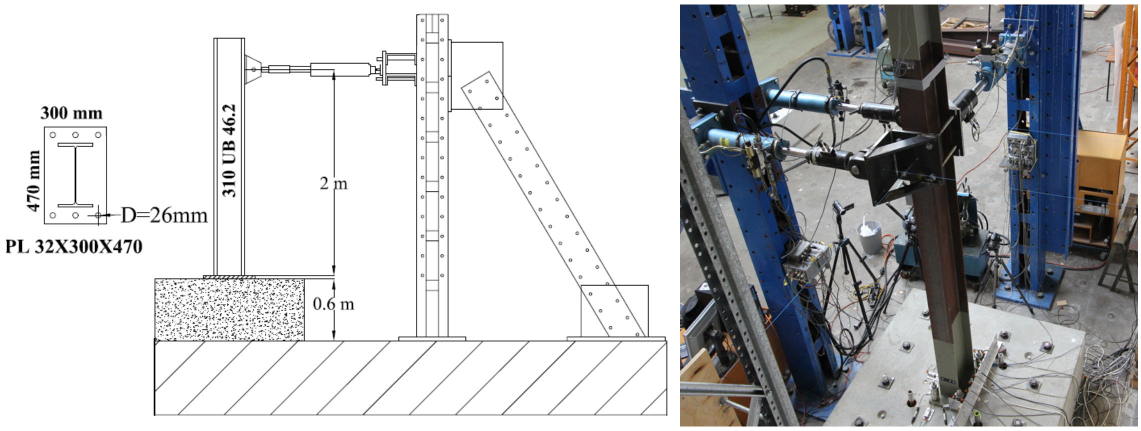


Figure 1. Dimensions and arrangement of the test setup

2.3 Loading regime

Cyclic loading regime according to ACI report T1.1-01 was applied to each specimen. The initial loading started from 0.2% drift (4 mm) and finished at 4% drift (80 mm). The increase in the new drift is between 1.25 to 1.5 times that of the previous step. For each level of drift, three full cycles and one cycle up to half of the drift is applied as shown in Figure 2.

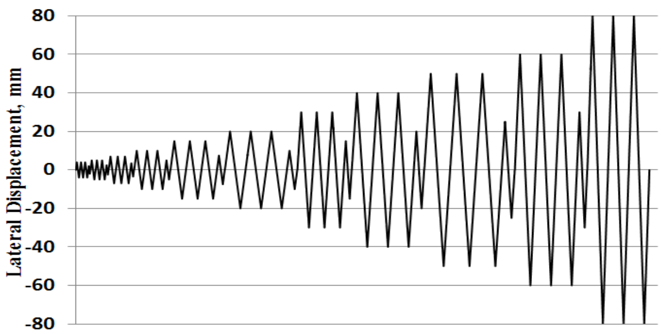


Figure 2. Test Protocol

2.4 Instrumentation

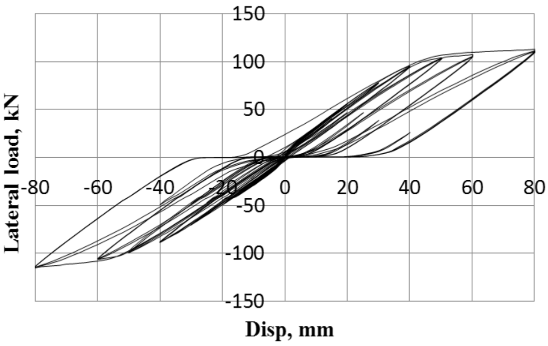
The tension force of the anchor rod was measured by the load-cell placed beneath the bolt head. Nine displacement gauges were placed on the base plate to measure the uplift profile along the length of the base plate. Also, another series of the displacement gauges was placed around the base plate and concrete block to monitor the sliding of the base plate and also the sliding and uplift of the foundation. Six strain gauges on the column and the base plate recorded the strains. Finally, the force- displacement of top of the column was recorded by the load-cells and rotary pots on the top.

3 BEHAVIOUR

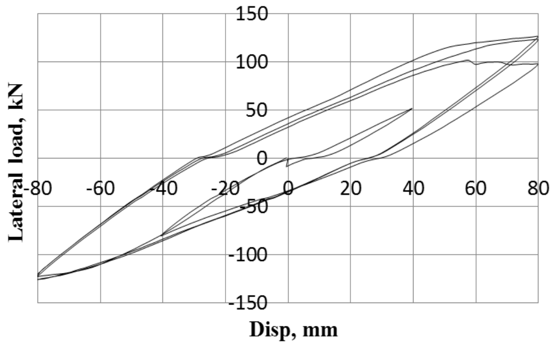
3.1 Force Displacement Response

The force-displacement diagrams of the columns with bases BC1 to BC3 are shown in Figure 3. The main failure modes for BC1 were anchor bolts yielding, fracture and there was almost no column yielding. In BC2, fracture of the welding occurred after the base plate and the column yielded. Finally for BC3 the main failure modes were buckling of the flange and the fracture of the welding at 4% drift. Because unit BC2 was the same as BC1 and only anchor bolts were changed in BC2 compared to BC1, only 4% drift cycles were applied to this base. Moreover, the cyclic performance of this connection is similar to BC1 for drifts lower than 4%. Figure 3 shows that post tensioning changed the shape of the force-displacement graph. Also, the maximum peak strength of BC3 is 10% higher than the same base plate connection without post tensioning (BC2), and the hysteresis curve for BC3 was fuller than for BC1. .

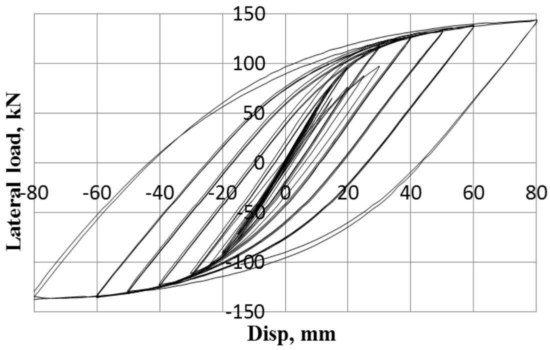
Shape of the force-displacement graph for BC1 is affected from yielding of the anchor bolt. The yielding of the anchor rods in BC1 caused rocking of this base and the column started rocking after the first yielding cycle. In BC2 and BC3, the bilinear shape was observed due to yielding of the column. Also, fracture of welding in the third cycle of BC2 reduced the peak strength about 20%.



a. BC 1



b. BC 2



c. BC 3

Figure 3. Force-Displacement diagrams

The vertical displacement of the base plate along its length is shown in Figure 4. The main source of the deformation for BC1 is the elongation of the anchor rod that was eliminated by anchor rod post tensioning in BC3. Also, the deformation of the base plate in BC2 and BC3 are higher than BC1 due to higher tension force in the anchor bolts.

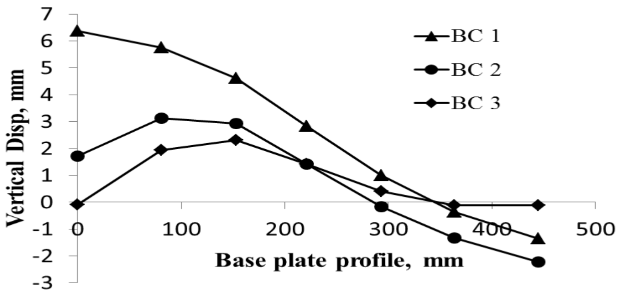


Figure 4. Uplift of the base plate along its length for drift 4%

3.2 **Damage Scenario**

Damage scenario of these base plate connections and idealized nonlinear model of them are shown in Figure 5. The snug tightening method cannot provide so much post tensioning (about 40 kN) in the anchor bolts. So, the initial stiffness of the column for BC1 and BC2 specimens drops immediately in small drifts. In all of these cases, the yielding in the compression and tension sides were appeared as a first yielding point. However, these yielding did not considerably reduce the lateral stiffness of the column.

The main change in the performance of first base plate connection started by yielding of the anchor bolts. The rotational stiffness of BC1 considerably dropped after the yielding. Moreover, the plastic deformation in the anchor bolts caused the rocking behaviour as shown in Figure 3.a. Generally, rocking behaviour is one of the low damage mechanisms. However, this behaviour resulted to brittle fracture of the anchor bolt in 4% drift for this base. So, the performance of the building is highly decreased by this poor performance of the base connection. Japanese design procedures proposed some criteria to avoid this type of failure by forcing to use ductile anchor bolts, unless the base plate connection should be designed for two times of actual demands on the base. The yield ratio is limited to 0.75 for ductile anchor bolt. Furthermore, the yield ratio is the ratio of the yield strength over the tensile strength of the bolt. This provides higher ductility on the anchor bolt.

In the second and third test, the brittle failure was observed due to fracture of the welding. The main reason for this type of the fracture can be excessive deformation of the base plate as can be seen in Figure 4 and also poor performance of the welding in the bending. The deformation of the base plate in BC2 and BC3 are 2.2 to 3.1 times of BC1 respectively. Moreover, the maximum applied moment in BC2 is 9% higher than BC1, and no welding crack was observed at the end of the first test. So, it doesn't seem that 9% increase in the applied moment can produce this type of fracture, and the bending of the base plate caused the fracture of the welding.

3.3 **Stiffness**

Force and displacement at the top of the column can be converted to the moment and rotation by using simple analytical equations such as Equations 1, 2. Where M is the moment at the base, H is the height of the column, V is the lateral force at the top, θ is base rotation and Δ_{top} is top displacement of the column.

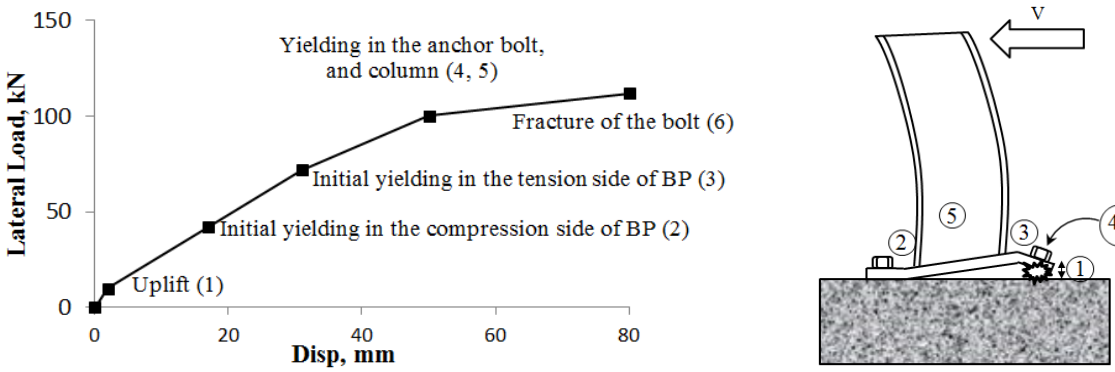
So, rotational stiffness of these connections can be calculated as presented in Table 2. The post tensioning of the anchor bolts increased the rotational stiffness of the joint about 6 times. Moreover, the lateral stiffness of the column is increased about 2.5 times as a result of rotational stiffness increment. Although both of these bases act as a semi rigid, the performance of the post tensioned base plate connection is closer to the rigid joint. According to NZS 3404 minimum rotational stiffness for a fixed base column base is *1.67 EI/H*. So, the performance of a base with post tensioned anchor bolts is closer to fixed base rather than un-post tensioned base..

$$M = V \times H \tag{1}$$

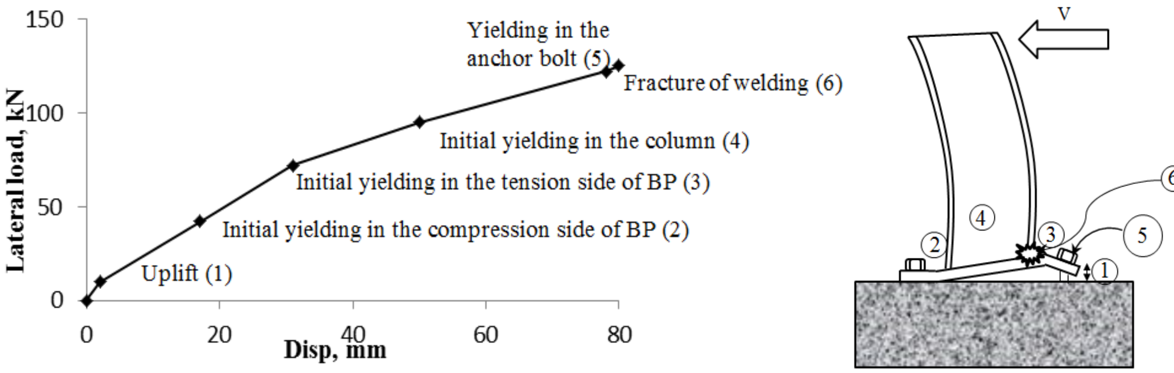
$$\theta = (\Delta_{top} - \frac{V \times H^3}{3 \times E \times I}) \times \frac{1}{H} \tag{2}$$

Table 2. Rotational stiffness of the base and lateral stiffness of the column

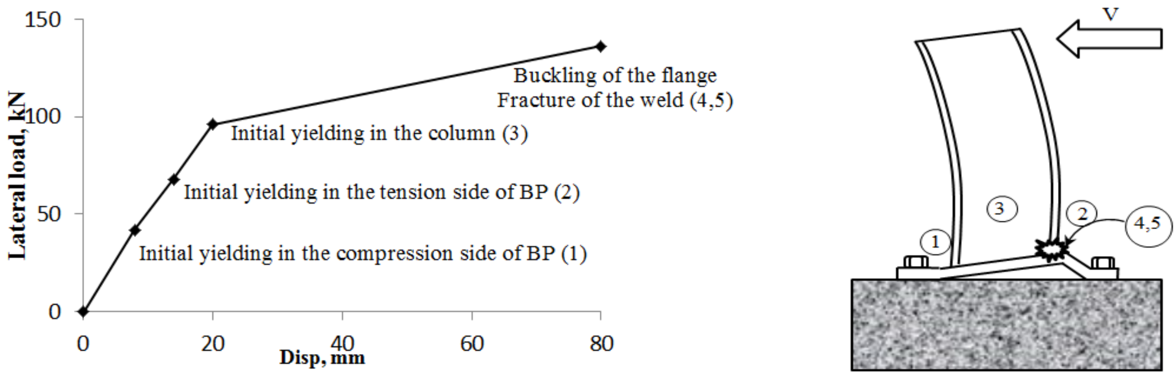
Specimen	K_θ	K
BC 1, 2	1.2 EI/L_{col}	0.85EI/L_{col}^3
BC 3	7.0 EI/L_{col}	2.1EI/Lc_{ol}^3



a. BC1



b. BC 2



c. BC 3

Figure 5: Idealized force- displacement hysteretic curve and hierarchy of failures

3.4 **Damageability**

In order to compare the damageability of different base plate connections some responses such as ductility of the column (μ), lateral force of the column at zero displacement (P_0), maximum nonlinear rotation of the column ($\theta_{NL\text{-}Col}$), permanent rotation of the base plate at the end of the test ($\theta_{P\text{-}BP}$), average plastic deformation of the anchor bolts ($\Delta_{NL\text{-}Bolt}$) and permanent deformation of the column at the end of the test ($\Delta_{P\text{-}col}$), reduction of lateral ($K_{End}/K_{Initial}$) and rotational stiffness ($K_{\theta\text{-}End}/K_{\theta\ initial}$) at the end of the tests compare to the initial value, are compared in Table 3. Comparing of $\theta_{NL\text{-}Col}$, $\theta_{P\text{-}BP}$, $\Delta_{NL\text{-}Bolt}$ shows that the main plastic hinges for the first joint (BC1) formed in the anchor bolts, for the second connection (BC2) the plastic hinges were at the base plate and the column and for the third base plate connection (BC3) happened in the column. Total permanent nonlinear rotation at the base for BC2 and BC3 are $18.2 \times 10^{-3}$and 21.5×10^{-3}rad respectively, and it can cause high permanent displacement at the top of the building. The reduction of base rotational stiffness due to plastic deformation of the anchor bolts in BC1 is such that it will act as a fully pinned base for any further loading. In contrast, the column with BC3 base lost about 40% of the initial lateral stiffness. The ultimate drift of the column with BC3 joint over the drift that column yielded is 2.5 times of the columns with BC1 and BC2 bases. This performance produces a reliable boundary of safety for BC3 base.

Table 3. Damageability of the base plate connections

Response	BC 1	BC 2	BC 3
$K_{End}/K_{Initial}$	0	0.28	0.57
$K_{\theta\ End}/K_{\theta\ initial}$	0	0.11	0.23
μ	1.6	1.6	4
P_0	0 kN	35 kN	95 kN
$\theta_{NL\text{-}Col}$	3.7×10^{-3} rad	10.7×10^{-3}rad	17.7×10^{-3}rad
$\theta_{P\text{-}BP}$	1.1×10^{-3} rad	7.5×10^{-3}rad	3.8×10^{-3}rad
$\Delta_{NL\text{-}Bolt}$	3.8 mm	0.3 mm	0 mm
$\Delta_{Pl\text{-}col}$	9.6 mm	30 mm	43 mm

4 CONCLUSIONS

A series of the experimental tests on the base joints were conducted to study the damageability of the current base plate connections under low axial load. Three bases were designed such that followings possible yielding patterns observed: Anchor bolts yielding, yielding in the column and combination of the base plate and yielding in the column. It was shown that:

1. The anchor bolt Grade 8.8 is not a reliable member to be designed weaker than any other base elements due to the brittle failure of high strength bolt. In order to prevent the fracture of the anchor bolts, they should be designed as strongest link of the base plate connections. If not, ductile anchor bolts should be used to prevent any brittle failure of them.
2. Post tensioning of the anchor bolts can highly change the seismic performance of the column base plate connection by considerable increase of rotational stiffness and ductility.
3. The available base plate connections cannot be categorized as a low damage because of the welding and anchor bolts brittle failures. Moreover, the rotational stiffness of the base considerable dropped at the end of loading. So, they show higher flexibility that can increase demands of the structures in any further earthquake. Also, at the end of the tests nonlinear rotation at the base caused for permanent deformation at the top of the column that is so hard to be straightened. Finally, one or some base elements yielded in each test specimen that are required to be replaced after the earthquake in the real building. However, the elements of the base are not replaceable. So, the building demolition is required for a building that all of the elements remain elastic but the base connections are yielded.

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