Determining the realistic rotational stiffness of column base connections in steel seismic resisting systems

J.C.W. He & G.C. Clifton

Department of Civil and Environmental Engineering, University of Auckland, Auckland.

S. Ramhormozian

Department of Built Environment Engineering, Auckland University of Technology, Auckland.

ABSTRACT

The New Zealand Steel Structures Standard, NZS 3404, provides an upper and lower rotational stiffness boundary for fixed and pinned column base connections for structural analysis, respectively. These boundaries aim to represent a realistic rotational stiffness of column bases. For a fixed base steel seismic resisting frame, it is expected that some degree of inelastic behaviour will occur at the column base under a severe earthquake, when the superstructure yields. However, no column base yielding was observed in steel frame structures after the 2010/2011 Christchurch earthquake series, even in structures where the desired inelastic mechanism in the superstructure was fully developed. One possible explanation is that the column base connections performed in a more flexible manner than anticipated, hence generated a smaller bending moment under a given rotation.

Motivated by this, a new PhD research project at the University of Auckland (UoA) and Auckland University of Technology (AUT) aims to determine the realistic column base rotational stiffness considering all possible sources of flexibility for several common column base systems. A non-linear time history analysis was undertaken as a preliminary study to determine the rotational stiffness at which no column base yields when the structure was subjected to the Feb 2011 Christchurch earthquake. The paper starts with a research background and a brief description of the research plan. It is followed by a discussion of the preliminary study results. Results showed that the actual column base stiffness could be half of what the standard suggested if non-structural elements were not considered.

1 BACKGROUND

The column base connection is a critical component in seismic resisting systems, due to its ability to affect the deformation of the whole structure as well as the inelastic demands of individual structural components. Failure of the column base connection could lead to rapid collapse of the structure in a severe earthquake and even partial failure will change the response of the structure significantly. Based on the rigidity of the connection, column base connections can be simply classified as "Pinned" or "Fixed". Pinned column base connections typically consist of two or four anchor bolts placed within the column flanges and are normally used for gravity columns in multi-storey building where the primary demand is gravity load (as shown in Figure 1). Fixed column base connections have more variations in their configurations and are used in seismic resisting systems to provide the desired rotational stiffness (as shown in Figure 2).

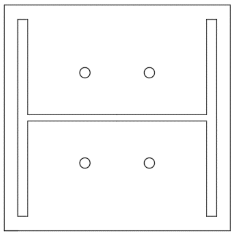

Figure 1: Base plate pinned (BPP) connection

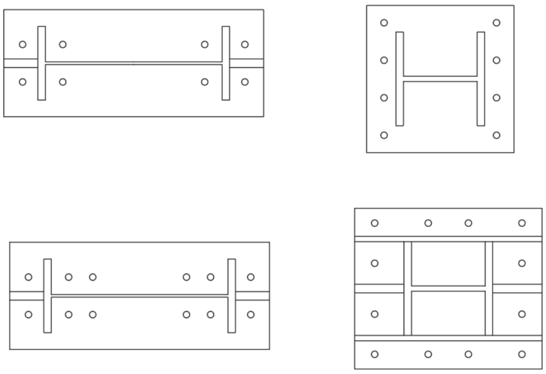

Figure 2: Variations of fixed column base connections

1.1 Column base rotational stiffness in NZS 3404

The traditional design approach assumes true pinned or true fixed connections to promote a fast and simple structural response analysis. However, past research has shown that the structural response is sensitive to column base flexibility. Mis-representing the actual column base rotational stiffness could significantly affect the structural performance (Aviram et al., 2010; Borzouie et al., 2016; Cui, Wang, & Yamada, 2019; Falborski et al., 2020; Maan et al., 2002; Stamatopoulos, 2012a, 2014b; Rodas et al., 2018; Zareian & Kanvinde, 2013). Specifically, Zareian and Kanvinde showed that the fixed base assumption for a semi-rigid column base could lead to soft-storey formation as column base flexibility lowers the point of inflection within the column resulting in an increase in flexural demand at the column top. Pinned base assumption, on the other hand, may result in a conservative design as the stiffness is neglected, resulting in a larger column size required for the bottom storey and larger beam size at the first-floor level. However, the inherent fixity of the connection will attract some degree of flexural demand at the base of a nominally pinned-base column and could potentially lead to column base hinging if such demand gets larger. These concerns and findings urged the investigation of realistic column base rotational stiffness.

The New Zealand Steel Structures Standard (1997), referred to as NZS 3404 from here on, recognizes the issues with simple fixed/pinned base assumptions and recommends boundaries for the rotational stiffness for fixed and pinned column bases to be used in design. According to Clause 4.8.3.4.1, the stiffness ratio at a joint in a rectangular frame can be expressed as:

$$\gamma = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{\beta_e I_b}{L_b}} \quad (1)$$

where β_e = modifying factor that accounts for the conditions at the far ends of the beam; I_c, I_b = second moment of inertia with respect to the axis of rotation of the column and beam connecting to the joint under consideration, respectively; and L_c, L_b = length of column and beam, respectively.

When applying Equation (1) to a column base, the denominator represents the stiffness of the column base connection rather than the beams. Theoretically, γ is equal 0 for a true fixed base and infinite for a true pinned base. Both conditions, however, are practically impossible to achieve. To represent a more realistic situation, it is recommended by NZS 3404 that a minimum γ-value of 10 and 0.6 is used for pinned and fixed column base connections, respectively. With some re-arrangements, Equation (1) can be re-written as the following:

$$k_{fixed} = \frac{1.67 E_c I_c}{L_c} \quad (2)$$

$$k_{pinned} = \frac{0.1 E_c I_c}{L_c} \quad (3)$$

where k_{fixed} , k_{pinned} = rotational stiffness for fixed and pinned column bases, respectively; and E_c = Young's Modulus of the steel column.

The 0.1 in Equation (3) provides an approximation of the inherent fixity of a pinned connection where the 1.67 in Equation (2) indicates the maximum rigidity that can be achieved in practice. Equation (2) and (3) aim to represent a more realistic situation by taking all possible sources of flexibility into account including the steel connection, reinforced concrete footing and the soil. However, in the following sections, earthquake observations and research findings showed that these boundaries are still questionable, and some uncertainties remain.

1.2 Observations from the 2010/2011 Christchurch earthquake series

Although NZS 3404 provides recommendations to the rotational stiffness of column base connection, concerns have been raised over the accuracy of these stiffnesses. This is because these equations (Equations (2) and (3)) are a simplistic representation of a multitude of effects. Based on reconnaissance of the 2010/2011 Christchurch earthquake (Clifton et al., 2011; Clifton & MacRae, 2013; Clifton, 2013; Clifton et al., 2012; MacRae et al., 2015), no column base yielding in steel structures was observed. This observation was interesting because the magnitude of the excitation was 1.5 to over 2 times the Ultimate Limit State (ULS) 500-year return period design level specified in the New Zealand seismic loading standard NZS 1170.5 (2004). Based on the design philosophy of strong-column-weak-beam design (SCWB), it was expected that the strong earthquake motion would cause the column base to hinge (MacRae, 1989). Ainsworth et al. (2015) modelled a 4-storey moment-resisting frame assigned with rotational stiffness $1.67EI/L_c$ and 1.5 EI/L_c, and subjected the frame to the Feb 2011 Christchurch earthquake motion (M_w=6.3). It was found that both models exhibited some degree of column base hinging. Therefore, it was unexpected that no column base hinging was observed after the Christchurch earthquake.

Study on the stability of wide-flange steel columns (Elkady & Lignos, 2018; Inamasu et al., 2019) showed that an increase in column base flexibility delayed the formation of column plastic hinge at the base and less column shortening was observed for a flexible column base connection. Based on these observations, it seems that the column base connections performed in a more flexible manner during the Christchurch earthquakes than what they were initially designed for.

1.3 Past research findings

Past research on pinned column bases has shown that they possess significant rotational stiffness (Hon & Melchers, 1988; Jaspart & Vandegans, 1998; Kavoura et al., 2017; Liu, 2001; Picard & Beaulieu, 1985; Robertson, 1991). By converting the connection stiffness to a ratio of the column flexural stiffness, the rotational stiffness of the specimens was found to be 2 times or more greater than the specified pinned base boundary ($0.1EI_c/L_c$) specified in NZS 3404. Recall that underestimating the rotational stiffness of the column base could lead to unexpected column base hinging. A gravity column is loaded with high and constant axial load. High constant axial load in conjunction with column base hinging and reversed cyclic loading will lead to axial shortening of the column (Inamasu et al., 2017, 2018) and subsequently differential settlement of the structure, increasing the difficulty and cost for post-earthquake repair. Furthermore, the axial load on the shortened column will be reduced and redistributed to adjacent columns (MacRae, 1989) reducing the moment capacity of the adjacent columns, due to the increase in axial load.

Unlike pinned base connections, fixed base connections are more flexible in terms of their configurations. Experimental and numerical studies on heavy steel fixed connections showed that the rotational stiffness is governed by many factors (e.g., base plate thickness, anchor rod configuration, axial load, and embedment depth if embedded) resulting in a wide range of rotational stiffness values from $1.19EI_c/L_c$ to $7.35EI_c/L_c$ (Barnwell, 2015; Cui et al., 2009; Gomez et al., 2010; Grilli et al., 2017; Hanks, 2016; Latour et al., 2014; Stamatopoulos & Ermopoulos, 2011; Trautner et al., 2016, 2017). Different combinations of parameters could result in great differences. It is therefore inappropriate to use a singular rotational stiffness to represent all column base connections.

Past research has shown that foundation rotation contributed significantly to the total structural displacement, reducing the deformation demand on the superstructure (Algie, 2011; Millin, 2012; Sa'don, 2012; Storie, 2017). In fact, the concrete footings in the previously mentioned experimental research were all post-tensioned to the strong floor and hence would perform in a much stiffer manner compared to practical condition. Few researchers have provided recommendations and attempted to address foundation rotation when investigating the rotational stiffness of column base connections (Borzouie et al., 2016; Eröz et al., 2008; Krystosik, 2018; Stamatopoulos, 2012; Zareian & Kanvinde, 2013). In their work, the elastic foundation stiffness was evaluated using existing theoretical equations and was subsequently combined with the column base rotational stiffness in series. It was found that, through rough estimation, soil flexibility reduced the column base rotational stiffness by 30% or more depended on the soil type and foundation system.

Although attempts have been made to account for foundation rotation, the equations used are for elastic soil behaviour (i.e., small strain). Past earthquake observations and experiments have found that the foundation system is more likely to behave in a nonlinear manner (Bartett, 1976; Drosos et al., 2012; Gazetas et al., 2013, 2003; Gazetas & Apostolou, 2004; Kutter et al., 2012; Salimath, 2018). This nonlinear behaviour is mainly contributed by geometry nonlinearity (i.e., gapping between foundation and surrounding soil) and soil nonlinearity (i.e., soil yielding at large strain). Pender et al. (2011) found that the foundation rotational stiffness was extremely sensitive to foundation rotation. This implies that the foundation stiffness is not a constant singular value and can vary significantly depending on the nonlinear effects of the foundation system. Therefore, it is important to incorporate the nonlinear behaviour of the foundation system and determine a realistic rotational stiffness for column base connections to be used in design.

2 RESEARCH OBJECTIVES AND PLANS

Earthquake observations and past research findings reviewed problems associated to the rotational stiffness values outlined in NZS 3404 and the potential consequences of mis-representing the actual rotational stiffness. Motivated by this, the main goal of this research is to determine the realistic column base rotational stiffness. In order to determine the realistic column base rotational stiffness, every source of flexibility should be taken into account. These sources will include the structural aspects of the connection and soil.

Three main objectives are derived from the research goal, when combined, illustrate the full scope of the research. These objectives are:

- Determining the rotational stiffness of the most common types of column base connections, especially focussing on those columns where column base plasticity will be most detrimental to post-earthquake restoration of the building function
- Incorporating soil flexibility and investigating the rotational stiffness of the connection system
- Developing a method to evaluate the rotational stiffness of column base

Each of these objectives and associated steps to achieve the objectives will be briefly described in the following sections.

2.1 Determining the rotational stiffness of the most common and critical column base connections

Gravity columns and inner columns of moment resisting frame are the primary focus in this research. They are subjected to high constant axial load and therefore susceptible to axial shortening when yield. This research is primarily based on numerical modelling. The rotational behaviour of column base connection will be determined through parametric finite element analysis. A wide range of connection configurations including Base Plate Pinned (BPP) and Moment End Plate (MEP) that are commonly used in New Zealand practice will be studied. This is to investigate the effects of various connection configurations and parameters (e.g., base plate thickness, axial load, anchor bolt size etc.) on the column base rotational stiffness.

2.2 Incorporating soil flexibility and investigate the rotational stiffness of the combined system

As mentioned previously, foundation rotation contributed significantly to the total structural displacement, reducing the deformation demand on the superstructure. Therefore, it is important to consider soil flexibility when determining the rotational stiffness of column base connection. In this step, different soil conditions as well as their non-linear behaviour will be incorporated into the connection model. The research aims to cover both shallow footing and deep pile foundation as they exhibit different rotational characteristics. The challenge of this step includes accurately modelling the foundation system while maintaining considerable computational efficiency.

2.3 Developing a method to evaluate the rotational stiffness of column base

This objective aims to convert results from this research to practical implications. Based on previous sections, a range of rotational stiffnesses will be recommended. These stiffnesses can be used in preliminary design of the structure directly to promote an efficient first phase design. In addition, the authors, ambitiously, aim to develop a general and simple-to-use method to evaluate the rotational stiffness of column base connections which can then be utilized in the final design for more realistic structural responses and more importantly, to ensure ground columns remain elastic during earthquakes in order to prevent axial shortening.

3 TIME HISTORY ANALYSIS

As a preliminary study of the research, a programme of non-linear time history analysis has been undertaken in order to get an indication of the column base rotational stiffness at which the columns do not yield when subjected to strong ground motions from the Feb 2011 Christchurch earthquake.

3.1 Building information

A plan view of the structure is shown in Figure 3. The structure is a 60 m long and 36 m wide moment resisting steel building. It is a 5-storey structure with a total height of 21 m (4.2 m storey height). Each side of the structure consisted of a 6-bay moment resisting frame throughout the entire height of the building. The structure has a 50-year life span and was designed under subsoil class D and E in accordance with NZS 1170.5. The reason for using two subsoil classes is to match the soil conditions at which the applied ground motions were recorded. A singular base rotational stiffness ($1.67EI_c/L_c$) was used for the design. Member sizes of the moment resisting frame are shown in Table 1.

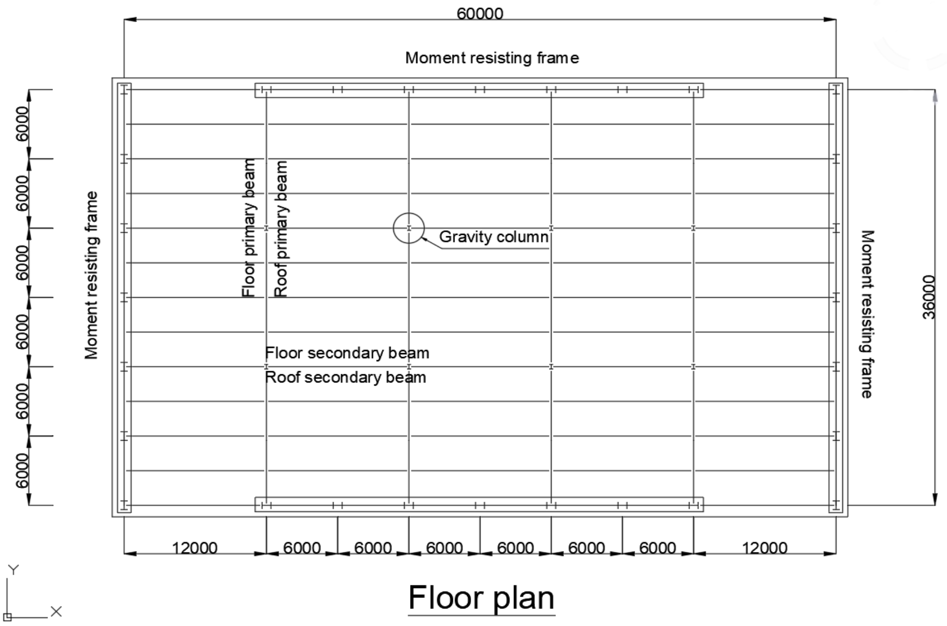

Figure 3: Floor plan of the structure (dimensions in mm)

Table 1: Member sizes of the moment resisting frame

		Subsoil Class D	Subsoil Class E
Beam	Level 1-2	700WB 130	900WB 218
	Level 2-3	610UB 101	700WB 130
	Level 5	460UB 67.1	460UB 67.1
Inner Columns	Level 1-3	900WB 257	1200WB 342
	Level 3-5	800WB 192	900WB 257
End Columns	Level 1-3	900WB 257	1200WB 342
	Level 3-5	700WB 150	800WB 192

3.2 Selected ground motions

The Feb 2011 Christchurch earthquake is considered in this analysis, as it is in that event that column base hinging was unexpectedly not observed. Four earthquake motions recorded at the Christchurch CBD were used: CCCC (Christchurch Cathedral College), CBGS (Christchurch Botanical Gardens Station), CHHC (Christchurch Hospital) and REHS (Christchurch Resthaven). The earthquake component with the higher peak acceleration was selected. According to the Tonkin & Taylor geological interpretative report (2011), site investigations near the REHS revealed plastic silts with peat layers near the ground surface. Due to the weak peat layers, the soil condition of REHS was classified as subsoil class E in this study. The rest of the stations possess subsoil class D. Due to the different soil conditions of the stations, the structure designed with soil class D was subjected to the CCCC, CBGS and CHHC ground motion where structure designed with soil class E was subjected to the REHS ground motion. Figure 4 shows the response spectra of the ground motions as well as the NZS 1170.5 soil D and E target spectrum for comparison. These applied ground motions were not scaled for the reason given above.

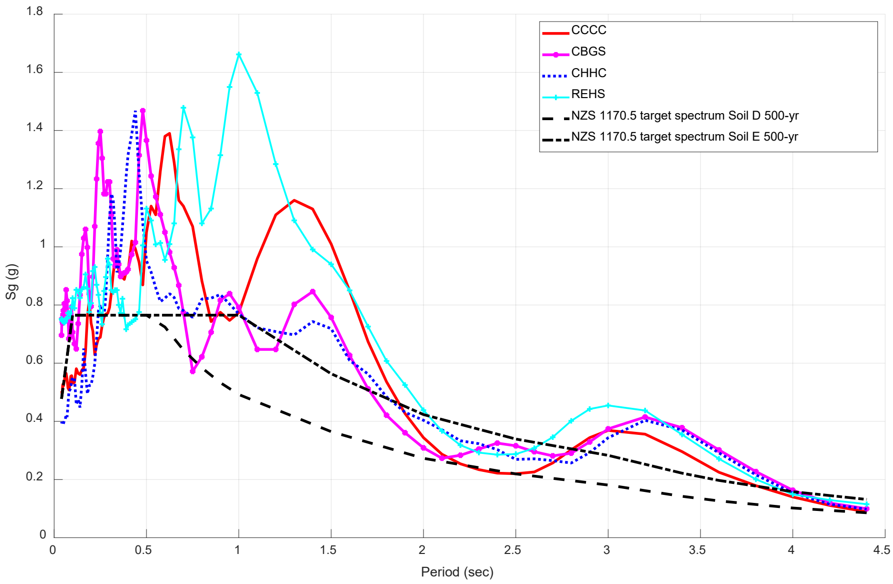

Figure 4: NZS 1170.5 Target spectra and recorded ground motions in from Christchurch CBD

3.3 Model information

A 2-dimensional frame was modelled in SAP2000 (CSI, 2020) for the time history analysis. The moment resisting frame (MRF) in the Y-axis (see Figure 3) was considered in this study. Grade 300 steel was used, with a minimum compressive yield (f_{y_min}) strength of 300 MPa and an expected compressive yield strength (f_{y_exp}) of 330 MPa (i.e., $1.1f_{y\_min}$). The default stress-strain relationship of steel in SAP2000 was assumed. Fixed connections were modelled at the beam-column intersections. Continuous columns were used throughout the entire height of the structure as they can assist in structural self-centring and are recommended in practice. The major-axis second moment of inertia (I_x) of the moment resisting beams were multiplied by a factor of 1.2 to represent the composite actions from the concrete floor slab, in accordance with NZS 3404 Clause N1.1.2 (a)(i). A rigid zone factor of 0.5 was used at the beam ends to model panel zone elastic stiffness and rigid diaphragms were assumed at each storey. Non-structural elements were not modelled. It is recognized that non-structural elements could increase the stiffness of the structure and potentially reduce the likelihood of column base yielding. However, in this analysis, the aim is to determine the column base rotational stiffness at which the column base does not yield. Without the non-structural elements, the structure is at its most flexible state and the column base stiffness is less likely to be overestimated. Although an exact rotational stiffness was not attained, a rough threshold was established which would be useful for future studies. Different column base rotational stiffness values ($1.67EI/L$, $1.5EI/L$, $1.2EI/L$, $1.0EI/L$, $0.8EI/L$, $0.6EI/L$, $0.5EI/L$) were assigned at the base of MRF columns. Recall that the structure was designed using a singular rotational stiffness of $1.67EI/L$. This is to represent the situation where engineers assumed a rotational stiffness in design while the actual rotational stiffness varies in practice.

In addition to the 2D MRF, two "dummy" columns were modelled on the sides of the frame. These "dummy" columns have compound geometric properties of the gravity and MRF columns (in the X-axis frames) of half of the structure. Seismic weight of half of the structure (excluding the portion that is supported by the Y-axis MRF) was distributed to the two "dummy" columns on each floor. "Dummy" columns were pinned at the base to reduce their contribution to lateral stiffness of the frame. Rigid beam elements were assigned with high Young's Modulus of Elasticity (E) and were used to connect the "dummy" columns and MRF columns. These rigid beam elements were again pinned at each end to eliminate their stiffness contribution.

Non-linear hinges were assigned to the ends of the MRF members according to ASCE 41-17 (2017). Non-linear direct integration time history analysis was performed in SAP2000. Damping $[C]$ was formulated using the common Rayleigh damping formulation, Equation (4). $[K]$ and $[M]$ is the stiffness and mass matrix respectively, where the mass and stiffness proportional coefficients (α_m and α_k respectively) were calculated by defining a constant damping ratio of 5% at the first and fifth mode period. The P-delta effects were modelled with reduction in column elastic stiffness due to initial compression loading and large displacement analysis.

$$[C] = \alpha_m[M] + \alpha_k[K] \tag{4}$$

3.4 Results and discussions

To identify the column base stiffness at which the column base does not yield, the hinge formation envelop for each base stiffness was investigated and shown in Figure 5. Dummy columns were omitted for clarity. The darker circles indicate greater extend of plastic deformation. Figure 5 shows varying levels of column base yielding from 1.67 to $1.0EI/L$ until $0.8EI/L$ was used. Recall that no column base yielding was observed after the Christchurch earthquake. This result shows that the actual rotational stiffness at the base of the structure could be half (or more than half if non-structural elements were considered) of what the standard suggests. Structures under the CBGS ground motion did not exhibit column yielding. Under the CHHC ground motion, only the end column yielded, and the plastic demand quickly disappeared as the rotational stiffness decreased to $1.0\ EI/L$. Under the REHS ground motion, $0.5EI/L$ was the stiffness where no column base yielded (rather than $0.8EI/L$ as for CCCC) possibly due to the much greater peak acceleration of the REHS ground motion. It was also observed that end columns were more likely to yield compare to inner columns. It is because of the greater compressive loading on the end columns when the building sways to one side, reducing the rotation required to yield the column base. This observation, however, does not contradict the decision to focus on inner columns in this PhD study as constant gravity load is the primary cause of axial shortening of the column. End columns although experience much greater compressive load, they are also loaded in tension, neutralizing/reducing the plastic shortening at the column base.

The effects of rotational stiffness on the structural behaviour were also investigated. Figure 6 shows the drift envelopes of the structure under CCCC ground motion for different rotational stiffnesses. The drift at the top storey for all cases were below the 2.5% drift threshold with a maximum of 2.27% drift for $1.67\ EI/L$. It is interesting to note that the top storey drift in the positive direction (rebound direction) is greatly influenced by the column base rotational stiffness where the drift in the negative direction is not. Specifically, the top drift in the positive direction decreases as the rotational stiffness decreases. At lower storeys (e.g., 1st and 2nd storey), drift increased as rotational stiffness decreased due to the greater demand imposed on the lower storey elements. Under the CBGS ground motion where no column yielded, the top storey drift increased with rotational stiffness (opposite to the CCCC case), indicating the influence of column base hinging on the overall performance of the structure. Considering all ground motions, the difference between top storey drift at $1.67EI/L$ and the stiffness at which columns remained elastic ($0.5EI/L$ for REHS and $0.8EI/L$ for the others) ranged from 10-20% equivalent to 48-79 mm.

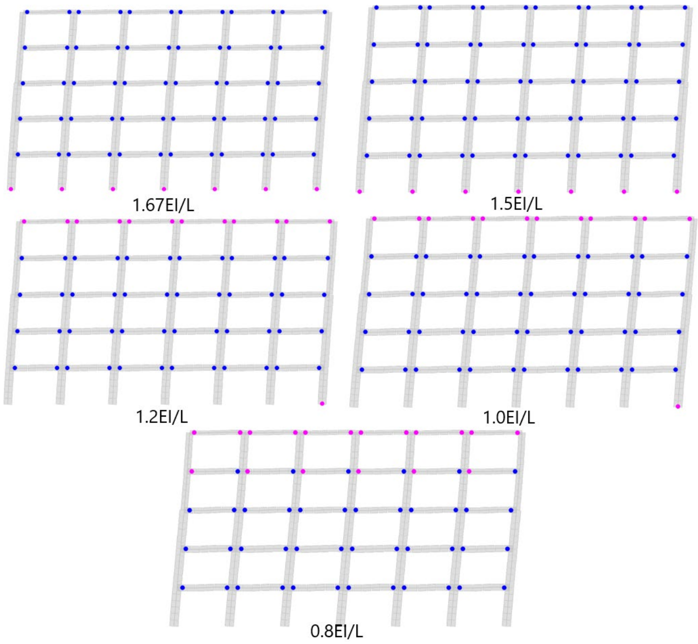

Figure 5: Hinge formation envelopes for structure under CCCC ground motion

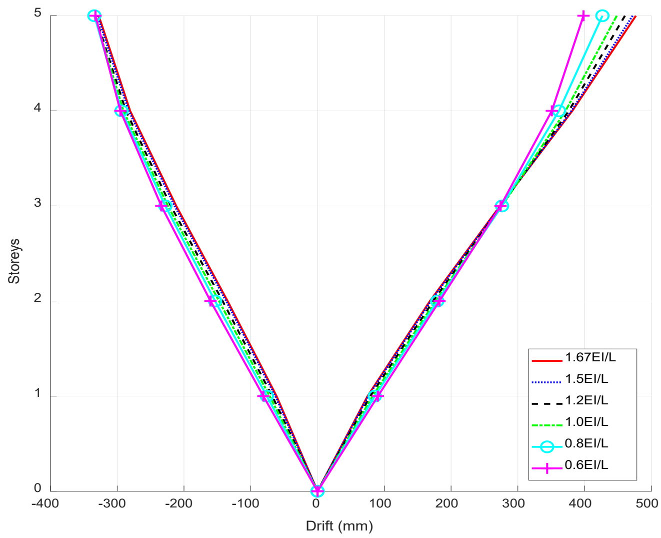

Figure 6: Drift envelops for structure under CCCC ground motion

The maximum plastic rotation and building displacement at first yield for ground motion CCCC are summarized in Table 2. As mentioned previously, end columns experienced much greater plastic rotation compared to inner columns due to the greater compressive force. As the stiffness at the column base decreased, a greater displacement was required to yield the end column, increasing from 315 mm (for $1.67EI/L$) to 393 mm (for $1.0EI/L$). After the column base yielded, it practically acts as a pinned connection. The column base could yield earlier with a greater base stiffness meaning the duration of pinned base condition became longer, which could lead to greater structural displacement. This explains why greater top storey drift was observed for greater base stiffness.

Recall that one of the advantages to keep ground columns elastic is the self-centring capability after an earthquake. Table 3 shows the residual drifts of the structure under different ground motions. A general trend of decreasing residual drift was observed as rotational stiffness decreased. Under the CBGS ground motion, residual drift of the structure remained low for the range of stiffnesses considered because all columns remained elastic. For case CHHC, as the rotational stiffness reduced to $1.0EI/L$ the residual drift decreased significantly as no column yielded under this stiffness. Case CCCC was an exception where the residual drift remained high even though no column yielded (102 mm at $0.8EI/L$). Further investigation showed that the plastic rotation of the beams at the top storey for case CCCC was significantly higher compare to the other cases, which could be the reason for the high residual drift. A residual drift limit of approximately 0.14% was suggested due to the observation from the HSBS tower after the Christchurch earthquake. Most of the residual drifts in this analysis are greater than this value primarily because non-structural elements were not modelled in this analysis.

Table 2: Summary of roof displacement at first yield and maximum plastic rotation under CCCC ground motion

Columns	Parameters				
	1.67EI/L	1.5EI/L	1.2EI/L	1.0EI/L	0.8EI/L
	Roof displacement at first yield (mm)				
Inner column*	337 (1.6%)**	385 (1.83%)	N/A***	N/A	N/A
End column	315 (1.5%)	313 (1.49%)	365 (1.74%)	393 (1.87%)	N/A
	Maximum plastic rotation (milliradians)				
Inner column	0.187	0.113	N/A	N/A	N/A
End column	3	2.5	1.4	0.32	N/A

*The centre column of the MRF frame.

**Value in the bracket shows the associated percentage drift of the structure.

***The associated column remains elastic.

Table 3: Residual drifts

Ground motions	Residual drift (mm)					
	1.67EI/L	1.5EI/L	1.2EI/L	1.0EI/L	0.8EI/L	0.6EI/L
CCCC	114 (0.54%)*	112 (0.53%)	109 (0.52%)	107 (0.51%)	102 (0.49%)	86 (0.41%)
CBGS	7.1 (0.03%)	2.3 (0.01%)	10 (0.05%)	16 (0.08%)	15 (0.07%)	2 (0.01%)
CHHC	129 (0.61%)	121 (0.58%)	101 (0.48%)	1.3 (0.01%)	46 (0.22%)	1.7 (0.01%)
REHS	90 (0.43%)	84 (0.4%)	70 (0.33%)	58 (0.28%)	46 (0.22%)	36 (0.17%)

*Value in the bracket shows the associated percentage drift of the structure.

4 FUTURE WORK

At this stage, the column base configurations to be studied are determined. The next step is to model the connections using finite-element modelling program. Once the models are validated, a parametric study will be performed to determine the effects of the considered parameters (e.g., base plate thickness, axial load and anchor bolt size) on the rotational stiffness for each connection configuration.

5 CONCLUSIONS

The column base connection is a critical component of the structural system in terms of its influence on the structural response. Traditional fixed or pinned designs misrepresent the actual rotational stiffness of the connection and can lead to incorrect results. The New Zealand Steel Standard, NZS 3404, recognizes such issue and provides a boundary for both fixed and pinned base conditions. Steel seismic resisting systems designed with the fixed base rotational stiffness outlined in NZS 3404, subjected to severe earthquakes, are expected to exhibit column base yielding when the desired mechanism is developed in the structure. However, observations from the 2010/2011 Christchurch earthquake series revealed no column base yielding occurred in steel structures, even where the earthquake was strong enough to push steel superstructures into the inelastic range. A possible explanation could be that the actual rotational stiffness at the column base was less than what the standard suggested due to soil flexibility. On the other hand, research on pinned column base connection revealed significantly greater connection stiffness than that outlined in NZS 3404. Determining the realistic rotational stiffness could facilitate accurate estimation of the structural responses. In addition, if the columns can be designed to remain elastic during an earthquake using the realistic rotational stiffness, the risk of column axial shortening can be eliminated, and the residual drift of the structure could be significantly reduced. Motivated by this, a research program was initiated to determine the realistic rotational stiffness of the column base connection.

The research program can be divided into several main stages: (1) Determine the rotational stiffness of the most common and critical column base connections, (2) incorporate soil flexibility and investigate the rotational stiffness of the combined system and (3) implement results which would include a suggested range of realistic stiffnesses to be used in design and potentially a method to evaluate the rotational stiffness for practical purposes.

A non-linear time history analysis was conducted to determine the rotational stiffness at which no column base yields when the structure was subjected to the strong ground motions from the Feb 2011 Christchurch earthquake. Under the CCCC ground motion, it was found that the column base rotational stiffness could be as low as half of what the standard suggested, if the non-structural element's contribution in the building's seismic response is neglected. In terms of building drift, a 10-20% difference was observed between the standard stiffness and stiffness at which column remained elastic, considering all ground motions. Furthermore, remaining column elasticity after an earthquake could significantly reduce the residual displacement. These results indicate the necessity to determine the accurate column base rotational stiffness.

ACKNOWLEDGEMENT

The authors would like to give special thanks to the University of Auckland for providing the doctoral scholarship for this research.

REFERENCES

Ainsworth, J. B., Conlan, C. J., Clifton, G. C., & MacRae, G. A. (2015). Influence of column base rotational stiffness and Non- structural elements on moment resisting steel frame building response to severe earthquakes. *2015 NZSEE Conference*, New Zealand, O-19.

Algie, T. B. (2011). *Nonlinear Rotational Behaviour of Shallow Foundations on Cohesive Soil*. PhD Thesis Dept. of Civil and Environmental Engineering, University of Auckland, Auckland.

American Society of Civil Engineers. (2017). *Seismic Evaluation and Retrofit of Existing Buildings* (41st ed.). American Society of Civil Engineers. https://doi.org/10.1061/9780784414859

Aviram, A., Stojadinovic, B., & Kiureghian, A. D. (2010). *Performance and reliability of exposed column base plate connections for steel moment-resisting frames*. Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Barnwell, N. V. (2015). *Experimental testing of shallow embedded connections between steel columns and concrete footings*. MSc Thesis Dept. of Civil Engineering, Brigham Young University, Provo.

Bartett, P. E. (1976). *Foundation rocking on a clay soil*. M.E., Dept. of Civil Engineering, University of Auckland.

Borzouie, J., Chase, J. G., MacRae, G. A., Rodgers, G. W., & Clifton, G. C. (2016). Spectral assessment of the effects of base flexibility on seismic demands of a structure. *Advances in Civil Engineering*, *2016*, 1–8. https://doi.org/10.1155/2016/3984149

CSI. (2020). *SAP2000: V22 Integrated Finite Element Analysis and Design of Structures* (Version V22) [Computer software]. Computers and Structures Inc.

Cui, Y., Nagae, T., & Nakashima, M. (2009). Hysteretic behavior and strength capacity of shallowly embedded steel column bases. *Journal of Structural Engineering*, *135*(10), 1231–1238. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000056

Cui, Y., Wang, F., & Yamada, S. (2019). Effect of column base behavior on seismic performance of multi-story steel moment resisting frames. *International Journal of Structural Stability and Dynamics*, *19*(01), 1940007. https://doi.org/10.1142/S0219455419400078

Drosos, V., Georgarakos, T., Loli, M., Anastasopoulos, I., Zarzouras, O., & Gazetas, G. (2012). Soil-foundation-structure interaction with mobilization of bearing capacity: Experimental study on sand. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, *138*(11), 1369–1386.

Eröz, M., White, D. W., & DesRoches, R. (2008). Direct analysis and design of steel frames accounting for partially restrained column base conditions. *Journal of Structural Engineering*, *134*(9), 1508–1517. https://doi.org/10.1061/(ASCE)0733-9445(2008)134:9(1508)

Falborski, T., Hassan, A. S., & Kanvinde, A. M. (2020). Column base fixity in steel moment frames: Observations from instrumented buildings. *Journal of Constructional Steel Research*, *168*, 105993. https://doi.org/10.1016/j.jcsr.2020.105993

Gazetas, G., Anastasopoulos, I., Adamidis, O., & Kontoroupi, Th. (2013). Nonlinear rocking stiffness of foundations. *Soil Dynamics and Earthquake Engineering*, *47*, 83–91. https://doi.org/10.1016/j.soildyn.2012.12.011

Gazetas, G., & Apostolou, M. (2004). Nonlinear soil-structure interaction: Foundation uplifting and soil yielding. *Proc. Third UJNR Workshop on Soil-Structure Interaction*, Menlo Park, California.

Gazetas, G., Apostolou, M., & Anastasopoulos, I. (2003). Seismic uplifting of foundations on soft soil, with examples from Adapazari. *Foundations: Innovations, Obervations, Design, and Practice, British Geotechnical Association*, Thoas Telford, 37-50.

Gomez, I., Kanvinde, A. M., & Deierlein, G. (2010). *Exposed column base connections subjected to aixal compression and flexure*. Final Report Presented to the American Institute of Steel Construction, Chicago.

Grilli, D., Jones, R., & Kanvinde, A. (2017). Seismic performance of embedded column base connections subjected to axial and lateral loads. *Journal of Structural Engineering*, *143*(5), 04017010. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001741

Hanks, K. N. (2016). *Rotational strength and stiffness of shallowly embedded base connections in steel moment frames*. MSc Thesis Dept. of Civil Engineering, Brigham Young University, Provo.

Hon, K. K., & Melchers, R. E. (1988). Experimental behaviour of steel column bases. *Journal of Constructional Steel Research*, *9*(1), 35–50. https://doi.org/10.1016/0143-974X(88)90055-7

Inamasu, H., Kanvinde, A., & Lignos, D. (2017). The seismic stability and ductility of steel columns interacting with concrete footings. *Proc. 8th International Conference on on Composite Construction in Steel and Concrete*, American Institute of Steel Construction.

Inamasu, H., Lignos, D. G., & Kanvinde, A. M. (2018). Effect of column base flexibility on earthquake-induced residual deformations of steel columns. *Key Engineering Materials*, *763*, 149–156.

https://doi.org/10.4028/www.scientific.net/KEM.763.149

Jaspart, J. P., & Vandegans, D. (1998). Application of the component method to column bases. *Journal of Constructional Steel Research*, *48*, 89–106.

Kavoura, F., Gencturk, B., & Dawood, M. (2017). Reversed cyclic behaviour of column-to-foundation connections in low-rise metal buildings. *Journal of Structural Engineering*, *143*(9), 04017095. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001821

Krystosik, P. (2018). Influence of supporting joints flexibility on statics and stability of steel frames. *International Journal of Steel Structures*, *18*(2), 433–442. https://doi.org/10.1007/s13296-018-0008-z

Kutter, B. L., Mason, H. B., Trombetta, N. W., Chen, Z., Bray, J. D., & Hutchinson, T. C. (2012). Seismic soil–foundation–structure interaction observed in geotechnical centrifuge experiments. *Soil Dynamics and Earthquake Engineering*, *48*, 162–174.

Latour, M., Piluso, V., & Rizzano, G. (2014). Rotational behaviour of column base plate connections: Experimental analysis and modelling. *Engineering Structures*, *68*, 14–23. http://dx.doi.org/10.1016/j.engstruct.2014.02.037

Liu, T. C.-H. (2001). Investigation of rotational characteristics of column 'PINNED' bases of steel portal frames. *Steel and Composite Structures*, *1*(2), 187–200. https://doi.org/10.12989/SCS.2001.1.2.187

Maan, O., Osman, A., & Consultant, A. D. A.-H. (2002). The influence of column bases flexibility on the seismic response of steel framed structures. *Proc. 4th Structural Specialty Conference of the Canadian Society for Civil Engineering*, Montreal, Quebec, Canadian.

MacRae, G. A. (1989). *The seismic response of steel frames*. PhD. Thesis, Civil Engineering Department, University of Canterbury.

Millin, R. J. M. (2012). *Characterising the snap-back response of single piles in stiff clays*. Master Thesis Dept. Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand.

NZS. (1997). *Steel structures standard*. NZS 3404: Part 1 & 2: 1997, New Zealand.

Pender, M. J., Algie, T. B., Orense, R. P., Wotherspoon, L. M., & Sa'Don, N. M. (2011). Snap-back testing for estimation of nonlinear behaviour of shallow and pile foundations. *Proc. 9th Pacific Conference on Earthquake Engineering*, Auckland, New Zealand, paper No. 200.

Picard, A., & Beaulieu, D. (1985). Behaviour of a simple column base connection. *Canadian Journal of Civil Engineering*, *12*(1), 126–136. https://doi.org/10.1139/l85-013

Robertson, A. P. (1991). A study of base fixity effects on portal frame behaviour. *The Structural Engineer*, *69*(2).

Sa'don, N. M. (2012). *Full scale static and dynamic lateral loading of a single pile*. PhD Thesis Dept. of Civil and Environmental Engineering, University of Auckland, Auckland.

Salimath, R. (2018). *Experimental and finite element analysis of nonlinear moment-rotation behaviour of rocking shallow foundations on stiff clay*. PhD Thesis Dept. Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand.

Stamatopoulos, George N. (2014). Seismic response of steel frames considering the hysteretic behaviour of the semi-rigid supports. *International Journal of Steel Structures*, *14*(3), 609–618. https://doi.org/10.1007/s13296-014-3019-4

Stamatopoulos, G.N. (2012). Contribution of the flexible supports to the distribution of the internal forces in steel frames. *International Journal of Steel Structures*, *12*(3), 419–426. https://doi.org/10.1007/s13296-012-3010-x

Stamatopoulos, G.N., & Ermopoulos, J. Ch. (2011). Experimental and analytical investigation of steel column bases. *Journal of Constructional Steel Research*, *67*(9), 1341–1357. https://doi.org/10.1016/j.jcsr.2011.03.007

Standards New Zealand. (2004). *NZS 1170.5: 2004, Earthquake actions—New Zealand*. Standards New Zealand.

Storie, L. B. (2017). *Soil-foundation-structure interaction in the earthquake performance of multi-storey buildings on shallow foundations*. PhD Thesis Dept. Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand.

Tonkin & Taylor. (2011). *Christchurch Central City Geological Interpretative Report* (REP-CCC-INT). Report Prepared for the Christchurch City Council.

Torres-Rodas, P., Flores, F., & Zareian, F. (2018). Seismic response of steel moment frame considering gravity system and column base flexibility. *Proc. 11th National Conference on Earthquake Engineering*, Los Angeles, p 7383–7393.

Trautner, C. A., Hutchinson, T., Grosser, P. R., & Silva, J. F. (2016). Effects of detailing on the cyclic behavior of steel baseplate connections designed to promote anchor yielding. *Journal of Structural Engineering*, *142*(2), 04015117.

https://doi.org/10.1061/(ASCE)ST.1943-541X.0001361

Trautner, C. A., Hutchinson, T., Grosser, P. R., & Silva, J. F. (2017). Investigation of steel column–baseplate connection details incorporating ductile anchors. *Journal of Structural Engineering*, *143*(8), 04017074. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001759

Zareian, F., & Kanvinde, A. (2013). Effect of column-base flexibility on the seismic response and safety of steel moment-resisting frames. *Earthquake Spectra*, *29*(4), 1537–1559. https://doi.org/10.1193/030512EQS062M