Accounting for the flexibility of beam-column joints within New Zealand steel moment-resisting frame structures

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Abstract. In New Zealand there currently appears to be no simplified, effective method of analysing the rotational stiffness of beam-column joints in steel moment resisting frame structures. Many practicing engineers use simplified design tables to detail beam-column joints for strength requirements, without accounting for the flexibility of joints. This tends to underestimate the flexibility of structures and hence the drifts they undergo in wind and earthquake events. To permit improved consideration of beam-column joint stiffness in a simplified manner, this work adapts the European component method to develop a series of tables that practitioners could look up to quickly identify beam-column joint stiffness values. The potential use for such stiffness values is highlighted by examining the impact of joint flexibility on the drifts expected in a 4-storey steel MRF subject to 1 in 500 year return period earthquake loading.

Introduction

Steel moment resisting frame (MRF) structures are commonly used in mid- and high-rise buildings in seismic regions, such as Christchurch, New Zealand. MRFs are used to resist lateral seismic actions by using stiff, strong connections between beams and columns, that leads to the development of large end-moments in the members and hence the name “moment-resisting” frames. With careful detailing and capacity design, MRFs are also able to dissipate large amounts of energy in intense earthquakes through the yielding of the steel sections in plastic hinge zones (beam ends and column bases). It is commonly assumed in New Zealand that the connections in MRFs are fully rigid but, in reality, a beam-column joint will experience some rotation and therefore is not fully rigid. In this paper a practice-oriented means of accounting for beam-column joint rotational stiffness is proposed. The potential impact of this flexibility is then highlighted by looking at the likely impact on a typical steel moment resisting frame structure.

Current Practice in New Zealand. Design of moment resisting frame steel structures in New Zealand is done in accordance with the New Zealand loadings standard NZS1170.5 [1] and structural steel standard, NZS3404 [2]. Many practicing engineers size members and connections using tables from the HERA design guide [3], which provides the strength of steel members and connections. This guide, which is intended to comply with NZS3404, does not, however, provide any information about the stiffness of the beam-column joints and it would appear that most practicing engineers make the assumption that beam-column joints are perfectly rigid.

International building codes and standards prescribe a limit on the allowable storey drift of a structure, in order to limit damage to drift-sensitive components. In the New Zealand loadings standard NZS1170.5 the drift limit is 2.5% of the storey height (Clause 7.5.1, [1]) for an earthquake with return period of 500 years. By neglecting the rotational flexibility of beam-column joints in MRFs, the fundamental period of vibration of the structure may be underestimated. Considering the typical shape of earthquake response spectra, an underestimation of the period of vibration will tend to imply underestimation of the lateral displacement and drift, such that more damage could be expected in earthquakes or wind storms. This could include damage to structural and/or non-structural elements. A counter argument to this might be that in seismic design, an underestimated period of vibration will tend to attract higher spectral acceleration demands, leading to potentially stronger systems than would otherwise be required. However, this argument is not applicable to typical MRF structures for which the strength requirements of the codes are not critical and instead stiffness/flexibility requirements dictate. As such, it is evident that the current practice in New Zealand of neglecting the rotational flexibility of beam-column joints would appear to be non-conservative. That said, other conservative assumptions in the design process might be expected to counter this non-conservatism and hence the overall risk posed by code-compliant steel structures is uncertain.

Means of accounting for beam-column joint flexibility

For those engineers wishing to properly consider the rotational flexibility of beam-column joints in steel MRFs the two most widely used options internationally would appear to be (i) the use of finite element analyses or (ii) the component method. These two methods are briefly reviewed in the sections below.

Finite element analyses. The finite element (FE) approach involves developing a refined model of the beam-column joint zone, such as that shown in Figure 1, that can adequately account for a range of rather complex phenomena, particularly when cyclic loading and inelastic behavior needs to be predicted. There are many publications that explain the use of FE models to predict the behaviour of different joint types, with the work of Krishnamurthy and Graddy [4] one of the earliest works to use the FE method to predict the behaviour of end-plate connections. Augusto et al. [5] provide an excellent review of different contributions over the past few decades that are relevant to different types of steel beam-column joints.

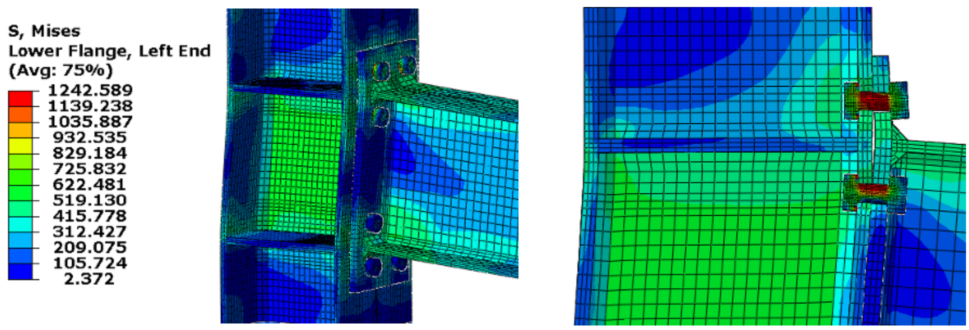

Figure 1. Example of a finite-element model of a beam column joint ([5]).

Despite the FE approach being the most accurate method currently available for the prediction of beam-column joint characteristics, it is not particularly practical, owing to the large computing power required to run analyses and the expertise required to develop an accurate model.

The component method. The component method is a common procedure used in Europe ([6], [7]) to estimate the strength and stiffness of beam-column (and other types of) joints. The component method takes into account various stiffness components of a joint and combines them to find an equivalent joint stiffness. Figure 2 shows the different components that are considered.

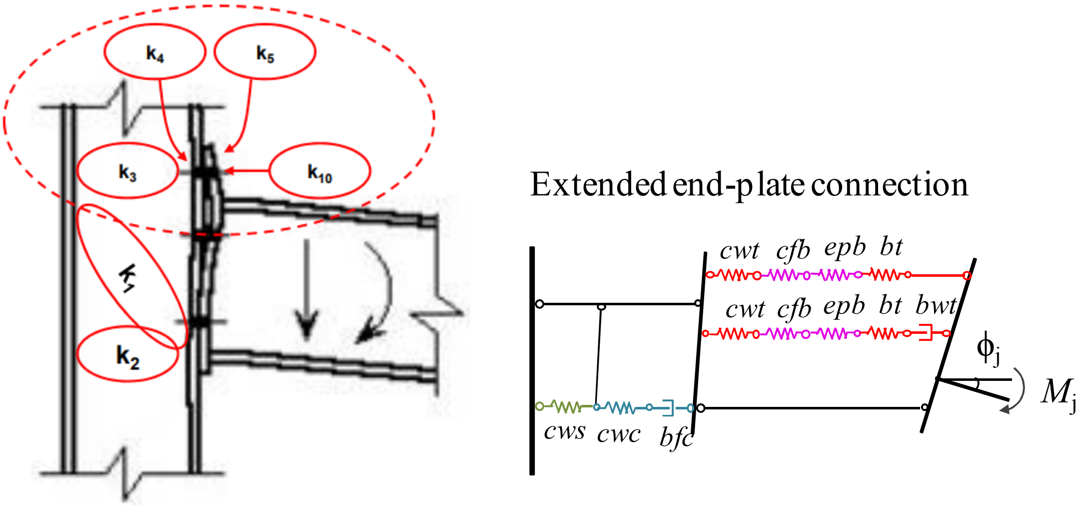

Figure 2. Components contributing to the stiffness of an extended end-plate beam-column joint and an equivalent mechanical model (adapted from [7]).

The stiffness components (k factors in in Figure 2) represent components of deformation provided from the different areas of a beam-column joint when a moment is applied. The stiffness components illustrated in Fig. 1 are as follows:

- k1 refers to the panel shear in the web of the column.
- k2 is the web of the column in compression.
- k3 refers to the column web in tension
- k4 is the bending of the column flange
- k5 is the end plate in bending
- k10 is the bolts in tension

These k factors can be combined to produce an equivalent joint stiffness using mechanics (see, for example, the mechanical model illustrated on the right side of Fig.2).

The component method is not considered as accurate as the FE method but it does permit the various components of joint flexibility to be quantified. Furthermore, Della Corte et al. [7] have shown that it can provide reasonably accurate results for a number of different beam-column joint typologies. Nevertheless, even this method is not considered particularly practical in New Zealand as engineers are used to using tables to size joints and neglect rotational flexibility.

Beam-column joint stiffness values for standard connections in New Zealand

To provide practitioners in New Zealand with a simple means of accounting for the rotational stiffness of beam-column joints in steel MRFs, in this work it is proposed that the component method of Eurocode 3 [6] be used to establish the rotational stiffness of all standard beam-column joints used in New Zealand. This can be done relatively easily once a few assumptions are made about the connection details and design decisions related to relative strengths.

In this work, three different versions of a bolted extended end-plate beam-column joint are evaluated using the component method considering unstiffened joints, joints and stiffened joints with doubler plates, as shown in Figure 3.

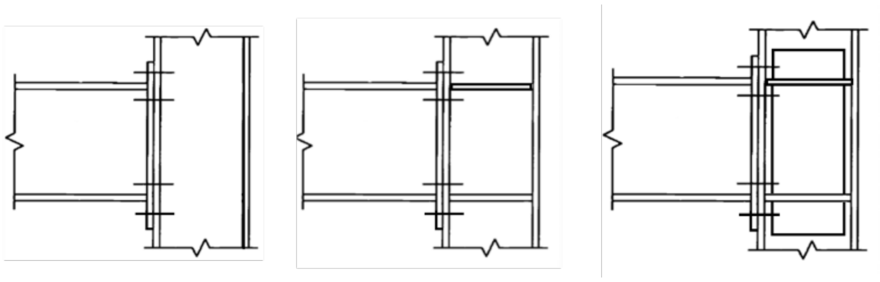

Figure 3. (a) Unstiffened Joint (b) Stiffened Joint (c) Stiffened Joint with Doubler Plates

Tables 1 and 2 illustrate the results of applying the component method to standard New Zealand beam-column sections.

To model the difference in stiffness when stiffeners were used in the joint, it was assumed that there would be no deformation of the column web in shear. Therefore the k3 factor could be considered to be infinite and did not need to be considered. This meant that the overall stiffness of the joint increased. When a doubler plate was used, it was assumed that the shear area would approximately double. This meant that the panel shear in the web of the column doubled. This resulted in additional joint stiffness. Note that no allowance has been made to check that beams are columns are stronger than beams (in line with capacity design principles) and in reality the engineer would need to do this consider the relative section strengths and axial load acting on the columns. Once a suitable beam and column section have been identified by the engineer, it is apparent that the rotational stiffness of the joints could be read off the table relatively quickly.

Table 1. Rotational Stiffness of Unstiffened Single-Sided Joints from Figure 3.a. (kNm/rad): UB Beams – UC Columns

Beams	610UB 125	610UB 113	610UB 101	530UB 92.4	530UB 82	460UB 82.1	460UB 74.6	460UB 67.1	410UB 59.7	410UB 53.7	360UB 56.7	360UB 50.7	360UB 44.7	310UB 46.2
Columns														
310UC158	163631	160029	158816	132439	127692	103085	102440	99534	83581	81997	68554	61156	60490	46425
310UC137	143133	139681	138589	115405	111735	90145	89566	86769	72800	71388	59659	54464	53777	41229
310UC118	120904	117767	116821	97162	94458	76166	75665	73110	61297	60098	50195	47073	46399	35543
310UC96.8	98143	95372	94574	78410	76521	61530	61110	58859	49223	48253	40196	39215	38571	29453
250UC89.5	97835	95691	94990	79667	77783	63311	62927	61093	51627	51124	42646	42388	41802	32575
250UC72.9	77103	75298	74725	62460	61191	49632	49321	47789	40261	39964	33145	34428	33892	26339
200UC59.5	71109	69808	69329	58363	57265	46792	46524	45387	38494	38218	31925	34933	34461	27301

Table 2. Rotational Stiffness of stiffened Single-Sided Joints from Figure 3.b. (kNm/rad): UB Beams – UC Columns

Beams	610UB 125	610UB 113	610UB 101	530UB 92.4	530UB 82	460UB 82.1	460UB 74.6	460UB 67.1	410UB 59.7	410UB 53.7	360UB 56.7	360UB 50.7	360UB 44.7	310UB 46.2
Columns														
310UC158	219018	217896	216809	186498	178355	148656	148017	146832	126859	124709	107447	107054	107127	87022
310UC137	193458	192409	191447	164620	158346	132031	131464	130394	112693	110815	95486	98048	98010	80043
310UC118	163981	163050	162230	139385	134873	112429	111945	111041	95947	94421	81277	86515	86376	70934
310UC96.8	133652	132853	132168	113166	110133	91467	91063	90344	77820	76665	65679	74140	73925	60940
250UC89.5	122173	121424	120819	103920	101267	84630	84271	83570	72366	71778	61461	67103	66841	55413
250UC72.9	96308	95702	95210	81570	79858	66425	66135	65614	56593	56389	47840	55474	55200	45878
200UC59.5	82737	82197	81775	70052	68722	57182	56933	56469	48720	48493	41198	48550	48268	40265

A case-study example

In order to get a clear idea on the implications of assuming a joint is rigid, a case study building was modelled. The storey drift of the building was calculated assuming rigid joints and compared with results when the joint stiffness was taken into account.

Description of the building. A four storey moment resisting frame office structure was designed in line with typical New Zealand design practice. In order to be able to determine the loads acting on the structure, various assumptions had to be made which included;

- The generic site was located in Christchurch city with a soil type D (deep or soft soil), and near field earthquake events were ignored.
- The design ductility factor for the building was three.
- The slab stiffness contribution was considered negligible as it has been assumed to be separated from the columns.
- The column base flexibility was assumed to be and modelled as $1.67\frac{EIc}{Lc}$.
- The mass of the floors were applied as lumped masses on each node and the self-weight of the beams and columns were considered negligible.

Figure 4 shows the 3D structural components of the building. For the purpose of this work, only the moment resisting frames in the transverse direction of the building were designed as shown in Figure 5.

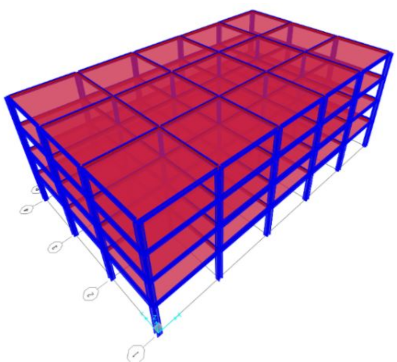

Figure 4. Adopted 3-D structural model of case study building in SAP2000

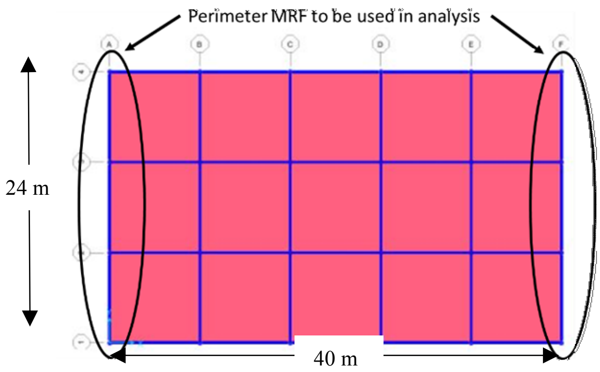

Figure 5. Plan view of perimeter MRFs and flooring in SAP2000

The full-strength beam-column joints were designed using a bolted end plate connection as shown in Figure 6. This is because it is a commonly used MRF joint type in the New Zealand industry and the design is specified in the HERA design guide.

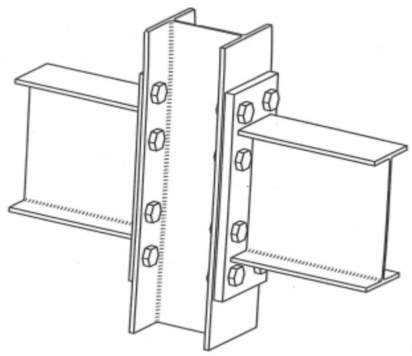

Figure 6. Bolted moment end plate connection [3].

Analysis and design process. The modal response spectrum analysis was used to design the frame, in line with the recommendations of the New Zealand loadings standard NZS1170.5. To account for torsion, the centre of stiffness of the 3D structure was assumed to be a distance of 10 % of the width and length of the building away from the centre of mass. The force on each frame due to torsion was half of that produced on the building. This lead to an increase in drift of approximately 16 %. To account for the additional stiffness provided from the interior columns on the moment resisting frame a dummy column was added to the SeismoStruct [8] model. The equivalent stiffness was input into the model and applied to this dummy column which had a value of the combination of the four interior columns in the member's weak direction. These were constrained to the frame in the in the horizontal direction at each floor.

After the initial drift of the structure had been calculated, P-delta effects on the drift were added. To take into account the effects of P-delta in the model a second dummy column was added, which was again constrained to the frame in the horizontal direction at each of the floors. It was modelled with a pinned base connection and was specified to provide no lateral stiffness to the frame. The horizontal P-delta forces for each floor were calculated following NZS1170.5 section 6.5 [1], and applied to the dummy column at each floor level.

The design process required some iteration to arrive at a set of section sizes that satisfied the various code requirements. Figure 7 shows the final section sizes identified.

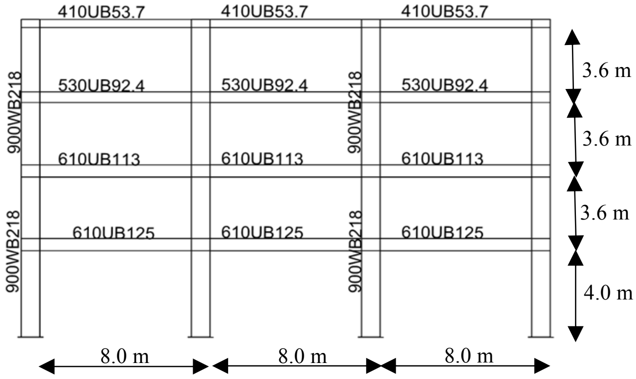

Figure 7. Final seciton sizes for the 4-storey case study MRF building.

To investigate the impact of rotational flexibility on the likely response, response spectrum analyses were repeated with the rotational stiffness of the beam column joints specified. To model the stiffness of a joint on, two nodes were modelled at the centre of the join and a spring connecting the two was added. The rotational stiffness calculated from the component method was applied to this spring in the direction which caused drift of the frame in the horizontal direction. It was assumed that the rotational stiffness in all other directions and all axial stiffness's were infinite. The frame with joint rotational stiffness applied was then reanalysed and the storey drifts were found.

Case-study analysis results. When the model was re-run considering the rotational stiffness of joints, each of these cases yielded drift results of over the code limit of 2.5%. Table 3 compares the maximum drift per floor for each different case. The results show that the assumptions that many designers make by assuming rigid joints offer no flexibility is not appropriate as the maximum drift of the floors is significantly more than assumed.

Table 1. Maximum storey drift (%) for each floor for the different types of joints modelled

	Type of Joint Model (refer Fig.3)			
Floor	Rigid Joints	Unstiffened Joints	Stiffened joints	Stiffened Joints with Doubler Plates
1	1.97	**2.78**	2.35	2.24
2	2.17	**3.72**	**2.92**	**2.70**
3	2.16	**3.83**	**2.90**	**2.67**
4	2.16	**3.79**	**2.84**	**2.62**

Limitations of this case-study. An important limitation of this analysis for the case study building was that the stiffness of the interior pinned joints was not considered. As there are no perfectly rigid joints, there are also no perfectly pinned joints. The interior joints of the building would therefore have added an overall stiffness to the structure and the drift would have reduced. However, this is believed to be less significant on the drift of the moment resisting frame and it would not be very conservative to assume this counters the effect of flexibility of the rigid joints. If this idea were to be investigated further, a 3D model of the structure could have been run where the stiffness of all joints was calculated and the drift was calculated.

The stiffness of the concrete slab was also not considered as it was assumed to be completely separate from the columns. In reality, this would offer a small amount of stiffness to the frame but would not significantly affect the results. This could be an area of future research of this project.

The values in Table 3 should also be seen as upper limits as the beam-column joint nodes were modelled at a point in the middle of the column without also providing rigid links to the beam and column elements. This means that essentially the beams were modelled with slightly more length than they should have and hence would have demonstrated more rotation than they would experience in reality. Despite this simplification, the conclusion that joint stiffness affects storey drifts significantly is not expected to change.

Conclusions

This study considered means of account for beam-column joint flexibility on the response of a steel moment resisting frame structures in New Zealand. In New Zealand design practise, beam-column joints tend to be treated as rigid. This has negative implications for the structural response and leads to the underestimation of the fundamental period of vibration of the structure, implying greater drift and most likely damage in rare wind and earthquake events. To permit improved consideratoin of beam-column joint stiffness in a simplified manner, this work has adapted the European component method to develop a series of tables that practitioners could look up to quickly identify beam-column joint stiffness values.

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