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Calculation of Strength Hierarchy at Reinforced Concrete Beam-Column Joints of Buildings

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ABSTRACT: Until the introduction of capacity design principles, reinforced concrete structures (RC) were built using details that are now considered having severe vulnerabilities. Lack of confinement at joint panel zones has been the most important of these vulnerabilities since they were never designed for the shear forces induced by the beams and columns. When capacity design was introduced in the 1970s, the hierarchy of strength concept was introduced into the seismic design philosophy of RC structures. Using this principle, the RC structures can only be as strong as their weakest link. Accordingly, the seismic damage to columns and joints must be prevented. Instead, the damage should be confined only to the beams where a global beam sway mechanism can form. This modern design philosophy can also be used in an analysis method to identify the local failure mechanisms of any RC beam column joints. Such methods were extensively used for RC beam column joint subassembly tests carried out at the University of Canterbury in the last 10 years. This paper reports the adaptation of the hierarchy of strength evaluation method to multi-storey RC structures with the intent of identifying the weakest elements of a structure under a given global shear demand.

1 INTRODUCTION

In 1970s it was well understood that the potential energy imposed on elastic structural systems by earthquakes is high, which transforms into kinetic energy during an earthquake. However, if ductile mechanisms are allowed in a structural system, the potential energy imposed by the earthquakes can be significantly dissipated by plastic hinging mechanisms, which reduces the amount of kinetic energy experienced by the structural system during an earthquake, as shown in Figure 1 (Park and Paulay 1975).

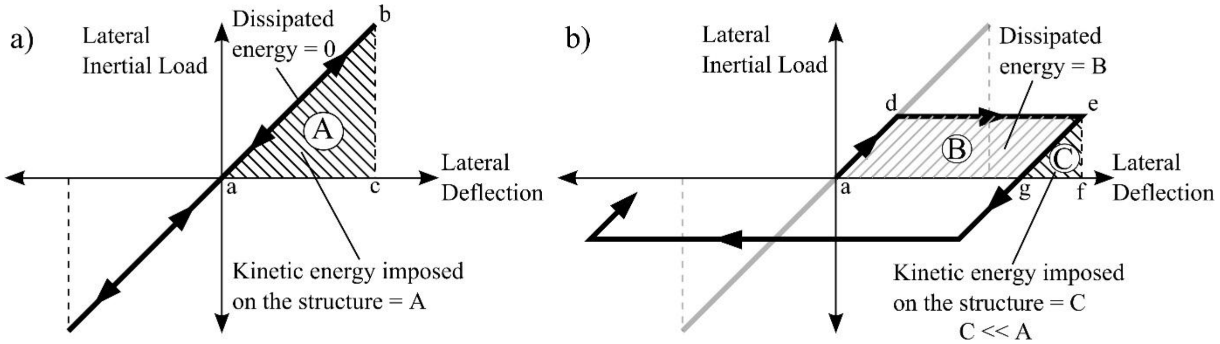


Figure 1 Imposed potential and kinetic energy levels in structural systems (Park and Paulay 1975): a) Elastic structure; b) Ductile structure

Since then, ductility has been an important factor utilized in modern seismic design of RC structures. According to capacity design principles (Park and Paulay 1975, Paulay and Priestley 1992), structural ductility can be provided in a structural design by assuming a beam sway mechanism (weak link), which allows plastic hinging at RC beam ends while protecting RC columns and RC beam column joints from damage. These principles are also valid for modern structural strengthening philosophy. Therefore, analysis of RC structures to identify the strength hierarchy at RC beam column joints is crucial for capacity assessment and strengthening purposes. Considering structural weaknesses such as the lack of confinement/shear reinforcement at RC beam column joints, strength hierarchy assessment becomes more important since such vulnerabilities can severely affect the global performance of buildings.

An earlier version of the strength hierarchy assessment method was used for RC beam column joint subassemblies in some research carried out at University of Canterbury in the last 10 years (Hertanto 2005, Chen 2006, Kam 2010, Akguzel 2011). In this paper, improvements and adaptations to this method have been made in order to make it easily applicable to real life structural engineering problems.

2 SEISMIC DEMAND ON REGULAR STRUCTURAL SYSTEMS: PORTAL FRAME METHOD

In strength hierarchy assessment, the seismic demand and capacities of RC columns, beams and beam column joints are compared via axial load, *N*, and bending moment, *M*, diagram of columns. For this comparison, seismic demand can be approximated by utilizing portal frame analysis method (Smith and Wilson 1915).

In this method, the inflection points of the deflected structure, or points of contra flexure, are assumed to be at the mid-span of the structural members. Using free body diagrams of every floor, total floor shear can be distributed among the columns in proportion to the distances between the nearest inflection points on the beams. Also, using the overturning moment about the column inflections points at a floor level, column axial loads resulting from the total floor shear can be calculated. The outline of this method is shown in Figure 2. For the strength hierarchy assessment method, the shear force, axial force and column base moments (Shear×Storey Height/2) can be calculated for increasing levels of total base shear. This results in axial load and bending moment variation, which can be directly plotted on the related column's *N*-*M* interaction diagram (Figure 2).

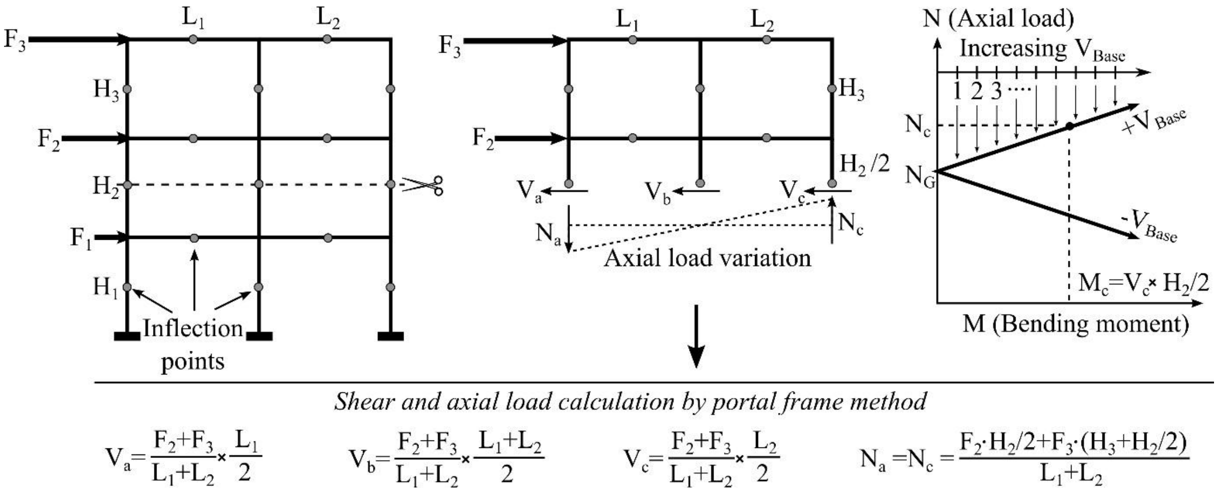


Figure 2 Portal frame method for shear and axial forces at inflection points of a deflected structure

3 CAPACITY OF STRUCTURAL ELEMENTS: RC COLUMNS AND RC BEAMS

The demand expressed in terms of column internal forces (*N*, *M*) can be directly plotted on an N-M interaction diagram of a column, which represents the capacity of the column. Using the equilibrium at RC beam column joints, the beam capacity can be approximately represented on the same column interaction diagram (Figure 3a). If the shear capacity of the beam results in a beam end bending moment value smaller than the flexural capacity, then this moment value should be considered as the beam moment capacity. For other cases, the beam moment capacity is the flexural capacity of the beam. The resulting hierarchy of strength comparison should be similar to the one shown in Figure 3b. It should be noted that the challenging part of this comparison is the calculation of the joint shear capacity and its representation on the interaction diagram of the inspected column, which will be explained in the following section.

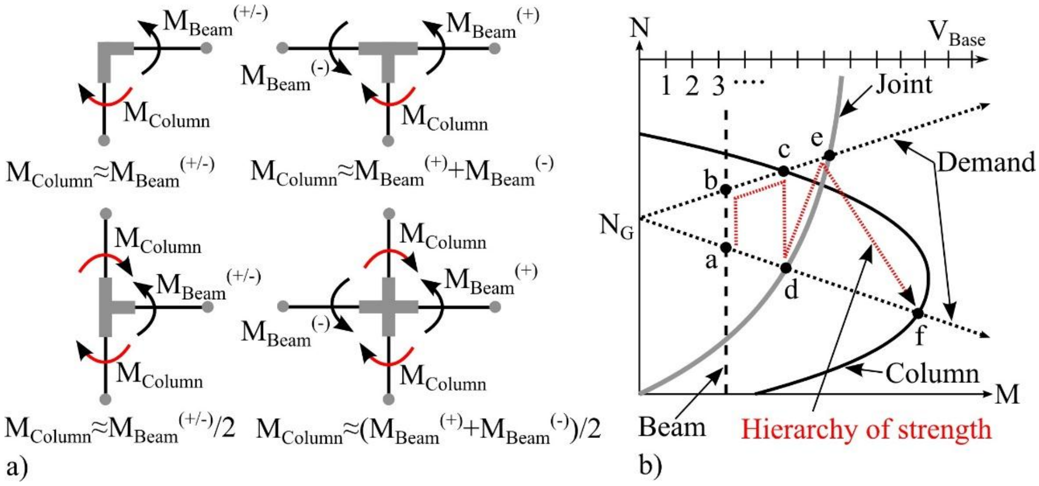


Figure 3 a) Beam moment capacity represented on column for various beam-column joint configurations; b) Hierarchy of strength comparison (a-b: beam hinging, c: column hinging, d-e: joint shear damage, f: column hinging)

4 BEAM COLUMN JOINT SHEAR CAPACITY AND REPRESENTATION ON COLUMN N-M INTERACTION DIAGRAM

4.1 *Joint Shear Capacity Summary: Vulnerable RC Beam Column Joints*

Until the introduction of the first seismic standards in 1970s there was no provision for RC beam column joint panel zone reinforcement requirements, which may still be the case in developing countries due to poor workmanship and lack of dependable quality assurance mechanisms. In various studies, this has been reported as the most common vulnerability observed in RC buildings (Priestley 1997, Genesio 2012). Typically, it is assumed that the buildings designed and built before the 1970s have no transverse reinforcement present in joint panel zones. In such structures, the diagonal tension at the joint is resisted only by the concrete. Therefore, in such structures, the joints are prone to brittle failures which can be determined using equation 1. The derivation of the formula below is based on Mohr's circle theory.

$$\nu_{jt} = \sqrt{P_t^2 + P_t \cdot \frac{N_j}{A_c}} \tag{1}$$

where P_t = Principal tensile strength of concrete in MPa (Value should be > 0); N_j = Axial force on the joint imposed by the column ($N_j > 0$ in compression, $N_j < 0$ in tension); A_c = Gross column area (mm^2); ν_{jt} = Joint shear strength (MPa).

Depending on the beam reinforcement anchorage details in the joint panel zone, the principal tensile strength of concrete to be used in equation 1 varies. Typical principal tensile strength values for different vulnerable RC joint types are shown in Figure 4 (Priestley 1997, Pampanin et al. 2003)

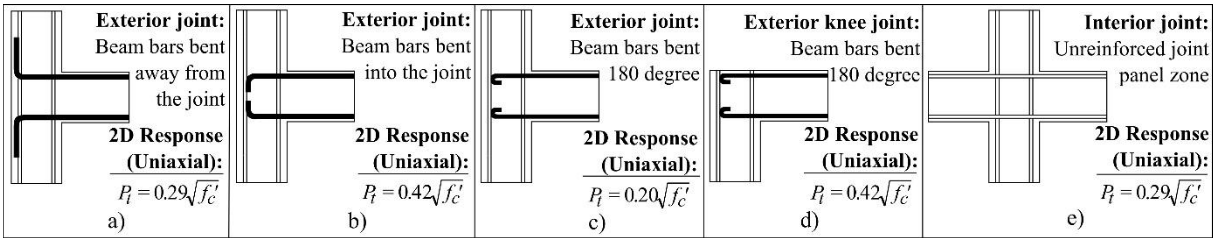


Figure 4 Typical principal tensile strength values for vulnerable RC beam column joints: a-b, d-e) Priestley 1997 and c) Pampanin et al. 2003

4.2 *Joint Shear Capacity Summary: Modern RC Beam Column Joints*

Modern RC structures are required to have transverse reinforcement (or confinement) at RC beam column joint panel zones in order to carry the resulting principal tensile stresses. However, research has shown that these adequately designed RC beam column joints are still susceptible to damage under high principal compression stresses, P_c (Beckingsale 1980, Pessiki et al. 1990). Under these circumstances, the joint shear strength is governed by the principal compression strength in the RC beam column joint panel zone. The joint shear capacity for such structures can be calculated approximately using equation 2, which is also derived using Mohr's circle for state of stress at a joint.

$$\nu_{jt} = \sqrt{P_c^2 - P_c \cdot \frac{N_j}{A_c}} \quad (2)$$

where P_c = Principal compressive strength of concrete in MPa (Value should be > 0); N_j = Axial force on the joint imposed by the column ($N_j > 0$ in compression, $N_j < 0$ in tension); A_c = Gross column area (mm^2); ν_{jt} = Joint shear strength (MPa).

Although more conservative values can be assumed, typical principal compressive strength, P_c, values for modern structures can be approximated as follows (Priestley 1997):

- $P_c = 0.5 f_c'$ for joints under uniaxial response in 2D
- $P_c = 0.45 f_c'$ for joints under biaxial response in 3D

4.3 *Joint Shear Capacity Representation on N-M Interaction Diagram*

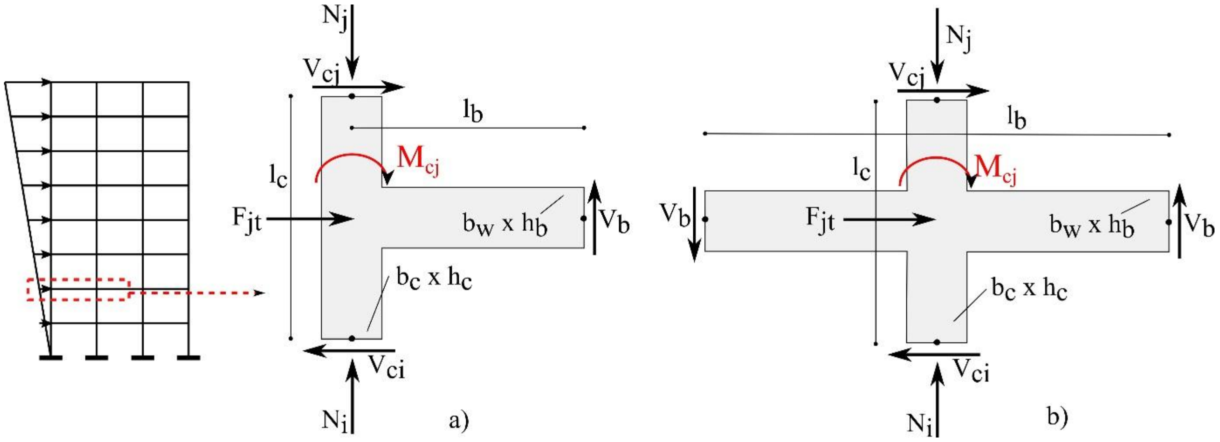


Figure 5 Free body diagram of RC beam column joints in a building: a) External beam column joint; b) Internal beam column joint

Considering equivalent static earthquake actions on a regular RC frame building and the resulting equilibrium equations at beam column joints (Figure 5), the joint shear capacity can be expressed as a moment value, M_{cj}, given in equation 3:

$$M_{cj} = \frac{\nu_{jt}}{\phi_1} + F_{jt} \cdot \phi_2, \quad Units = \frac{kPa}{1/m^3} + kNm \quad (3)$$

where ν_{jt} = Joint shear strength in kPa; F_{jt} = Floor shear force component at the joint in kN ($F_{jt} = F_{Floor} \times l_b / L_{Total}$); ϕ_1, ϕ_2 = Geometric coefficients defined as shown below (ϕ_1 in 1/m^3, ϕ_2 in m).

- For external joints:

$$\phi_1 = \frac{2l_b l_c - l_c h_c - 2l_b jd}{l_b jd b_c h_c (l_c - h_b)}, \quad \phi_2 = \frac{l_c - h_b}{2}\left(\frac{l_c h_c - 2l_b l_c + 4l_b jd}{4l_b l_c - 2l_c h_c - 4l_b jd}\right) \quad (4, 5)$$

- For internal joints:

$$\phi_1 = \frac{l_b l_c - l_c h_c - 2l_b jd}{l_b jd b_c h_c (l_c - h_b)}, \quad \phi_2 = \frac{l_c - h_b}{2}\left(\frac{l_c h_c - l_b l_c + 4l_b jd}{2l_b l_c - 2l_c h_c - 4l_b jd}\right) \quad (6, 7)$$

where jd = Moment arm of the tensile reinforcement in the beam

5 VALIDATION EXAMPLE: VULNERABLE RC TEST FRAME

The summarized method has been applied using a previous experimental study, the results of which were reported in numerous publications (Pampanin et al. 2004, Pampanin et al. 2007). The considered test specimen is a 2D RC frame without transverse reinforcement at beam column joint panel zones. The type of detailing used in this specimen simulated a vulnerable beam column joint with beam bars bent 180 degree into the joint panel zone, typical of pre 1970s. The testing of this specimen confirmed the damage vulnerability of beam column joints without any transverse reinforcement. The external beam column joints at the 1st and 2nd floor levels showed joint shear failure. On the other hand, internal joints did not suffer any shear failure except for column plastic hinging at the 1st and 2nd floor levels. The summary of damage in this test is shown in Figure 6.

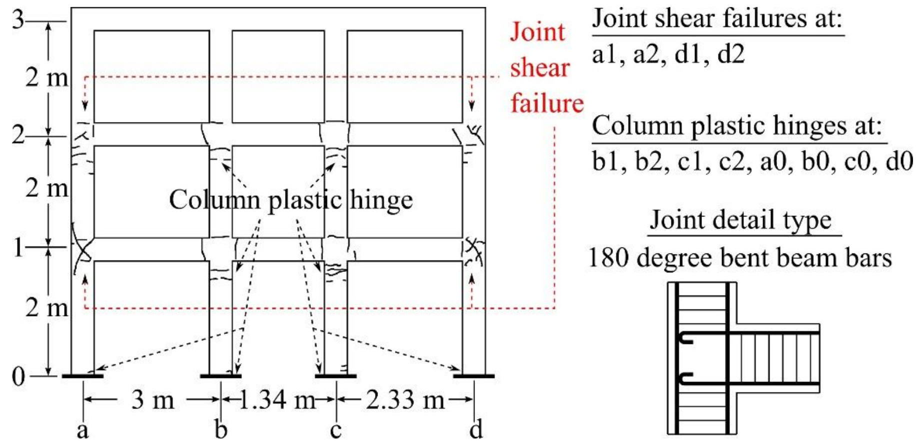


Figure 6 The damage summary of the test specimen (Redrawn from the source: Pampanin et al. 2004 and Pampanin et. al. 2007)

Using the details of this test specimen, hierarchy of strength assessment has been carried out for all the beam column joints considering a triangular loading pattern. In the loading pattern, the floor seismic forces have been denoted as F, 2F and 3F at the 1st, 2nd and 3rd floors respectively. Selecting increasing values of F and carrying out portal frame analysis results in the demand imposed on each beam column joint. The summary of portal frame analysis for the joints at the 1st and 2nd storeys are given in Table 1.

Table 1 The results of the portal frame analysis for the beam column joints on 1st and 2nd storeys: Internal forces expressed in terms of F under the imposed total base shear force 6F

Storey	*Column*	*N_G (kN)*	*V*	*M*	*N*	**ϕ1(1/m³)**	**ϕ2(m)**	*F_{jt} (kN)*
1 (Ground)	a	55.24	1.35F	1.35F	±3.3F	179.693	-0.3479	1.87
	b	76.93	1.95F	1.95F	±0.332F	72.013	-0.2439	2.7
	c	70.79	1.65F	1.65F	±0.995F	70.123	-0.2392	2.28
	d	49.10	1.05F	1.05F	±3.3F	175.387	-0.3462	1.45
2	a	35.26	1.12F	1.12F	±1.65F	179.693	-0.3479	3.76
	b	48.94	1.63F	1.63F	±0.166F	72.013	-0.2439	5.43
	c	45.63	1.38F	1.38F	±0.497F	70.123	-0.2392	4.59
	d	31.95	0.87F	0.87F	±1.65F	175.387	-0.3462	2.92

Note: The total base shear imposed on the specimen is equal to 6F

After calculation of the demand, the capacity of each element has been calculated and plotted on the related column axial load and moment interaction diagram. These graphs are shown in: Figure 7 for joints at the 1st floor level and Figure 8 for joints at the 2nd floor level. According to these results it can be seen that the first elements to fail are the external beam column joints at the 1st and 2nd floor levels. The failure of joints a1 and a2 is estimated to occur at base shear levels of 23 kN-31 kN whilst the joints d1 and d2 are expected to fail at 30 kN-39 kN base shear levels. These failures are estimated to be followed by column hinging at internal column ends at the 1st and 2nd storeys. According to the hierarchy of strength assessment, the columns b1 and c1 are estimated to form plastic hinges at 46 kN and 51-56 kN base shear levels. The column hinging at joints b2 and c2 are expected to occur at 50 kN and 57 kN base shear levels. It should be noted that the difference in capacity values of these joint couples is due to different ϕ1, ϕ2 coefficients and different equivalent static forces acting on the joints, which were calculated using the asymmetric beam span lengths given in Figure 6 and in Table 1.

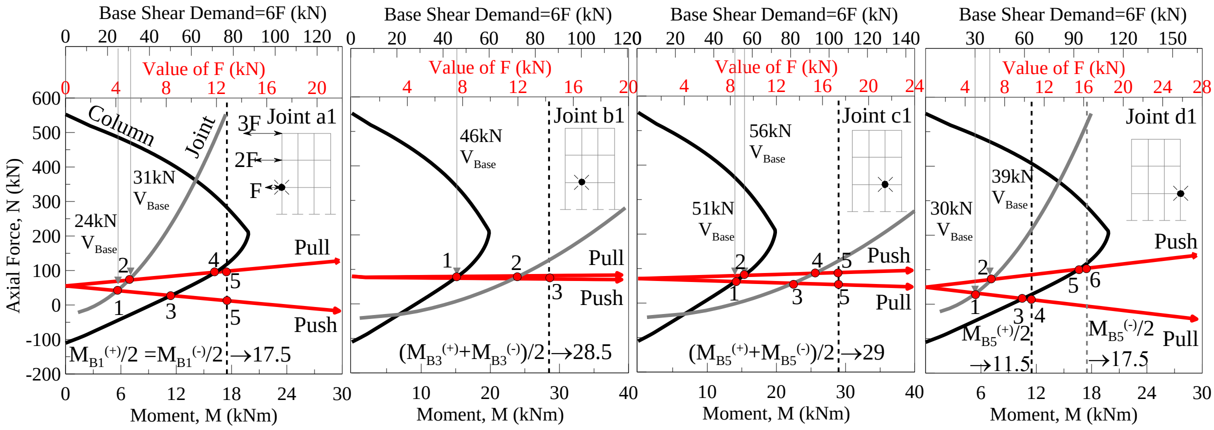


Figure 7 Hierarchy of strength comparison for the beam column joints at 1st floor level (a1, b1, c1, d1)

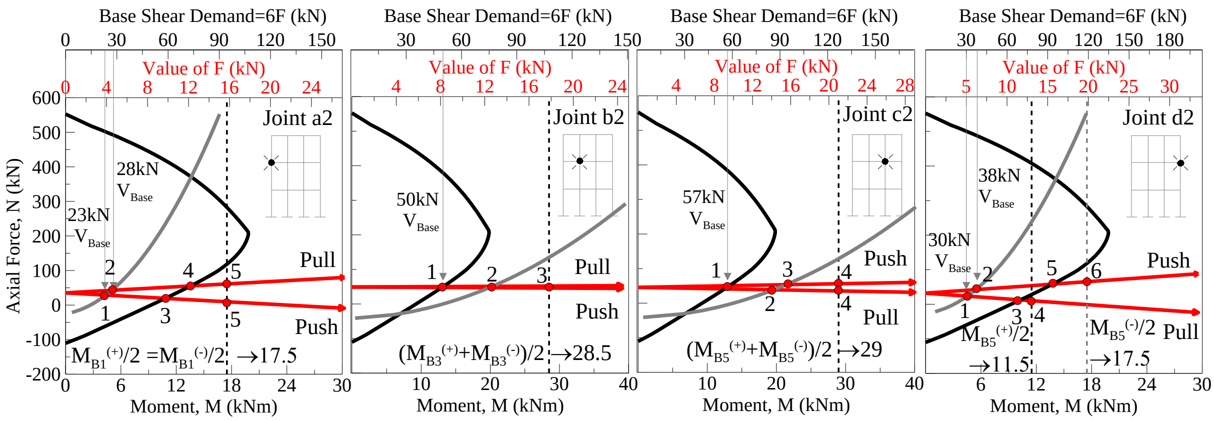
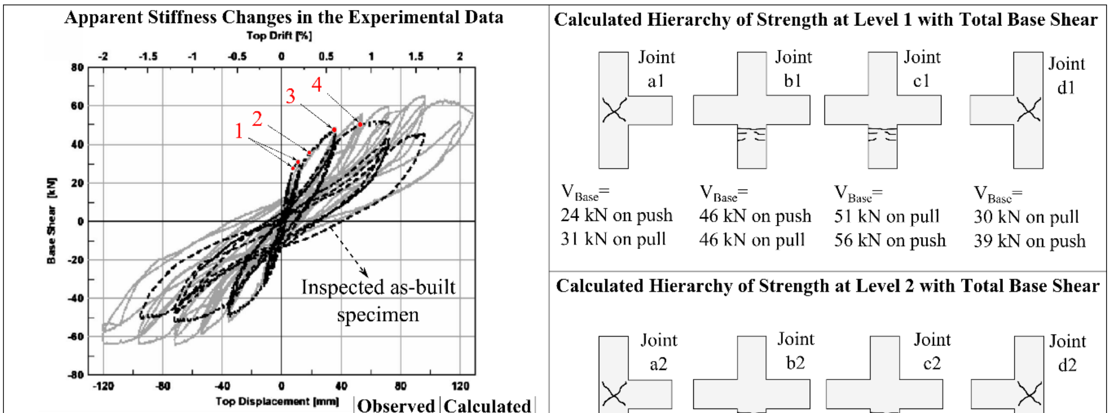


Figure 8 Hierarchy of strength comparison for the beam column joints at 2nd floor level (a2, b2, c2, d2)

When the experimental results of this test specimen are inspected, the points for the apparent stiffness changes can be inferred as the failure values. According to the experimental results, 4 events can be identified for 4 different levels of imposed base shear, which corresponds to 27-31 kN, 37 kN, 48 kN and 50kN, as shown in Figure 9. In the same figure, the comparison of the observed and calculated base shear values are also summarized. It can be seen that the strength hierarchy assessment method reported in this paper gave very close approximations to the experimental results with correct modes of failure.



	Observed	Calculated
Event 1: Joint shear failure at a1 and a2	27, 31 kN	23-31 kN
Event 2: Joint shear failure at d1 and d2	37 kN	30-39 kN
Event 3: Column end hinging at b1 and b2	48 kN	46-50 kN
Event 4: Column end hinging at c1 and c2	50 kN	51-57 kN

Figure 9 Comparison of the test results of the specimen and the calculated hierarchy of strength values

6 APPLICATION TO A MODERN BUILDING: EFFECT OF VERTICAL ACCELERATION ON STRENGTH HIERARCHY

In order to validate the applicability of the method for modern buildings, a ten storey building has been used. The building is a typical building designed according to NZS 1170.5 (NZS1170.5 2004). More details about the building can be found in its detailed design report by Bull and Brunsdon (Bull and Brunsdon 2008). For the sake of brevity, the details about this building are not given in this paper.

The hierarchy of strength assessment confirmed the beam sway mechanism, which is the assumed mode of failure in modern capacity design. It has been identified that the beams connected to the external beam column joints at the 1st floor level are expected to reach their ultimate moment carrying capacity at 1150-1200 kN total base shear. It should be noted that the design base shear for this structure is approximately 1102 kN, which is 4-9% lower than the provided capacity. Therefore, this analysis confirmed the validity of the capacity design with an accurately calculated base shear capacity value. The strength hierarchy comparison of the 1st floor external beam column joint is shown in Figure 10.

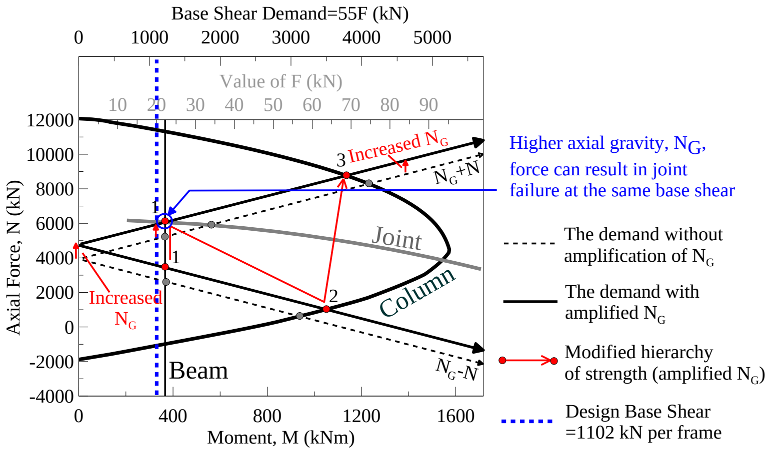


Figure 10 Strength hierarchy comparison for external beam column joint at the 1st floor level of the ten storey modern building example (Redbook building by Bull and Brunsdon 2008)

6.1 *Effect of Vertical Acceleration on Hierarchy of Strength*

The analysis also provided some insight into the effect of vertical acceleration on building response. As it can be seen in Figure 10, the initial axial load on the external beam column joint is about 4000 kN, which is the beginning of the imposed demand shown by the thin dashed lines. In this state, it is apparent that the first element to fail is the beam at 1150-1200 kN and this is followed by the compression failure in the joint at about 2000 kN base shear level, which is twice the design base shear and unlikely to be reached if the imposed vertical acceleration is negligible. However, assuming a vertical acceleration of 0.2g acts on the building at any stage, the axial forces imposed on the compressed columns can be amplified. For example, 4000 kN that develops under 1g can be amplified due to the new 1.2g vertical gravity, which results in 4800 kN axial load. As shown in Figure 10, this may shift the intended hierarchy of strength during the capacity design and may cause premature failure of the external beam column joints under high principal compression stresses. This may be an important concern and may require additional design checks for the effect of vertical accelerations on the response of RC structural systems. Further research in this area is required for the development of future design practices.

7 CONCLUSIONS

The importance of ductility and capacity design in seismic design of RC structures has been reviewed. A simple method to evaluate the hierarchy of strength, or hierarchy of member capacities, at beam column joints of RC multi-storey structures has been reported. The method has been validated using two example structures. One of these examples was a tested RC frame that simulated vulnerable pre 1970s construction practice. The other example was a 10 storey RC frame system designed according to modern design codes and capacity design principles. The results of these assessments showed that the reported method realistically estimates the failure mode of the structural systems. In addition, the global base shear corresponding to the identified failures can be easily computed as part of the developed method. It was also shown that even modern structures can experience joint shear failure under high principal compression stresses, which can occur due to vertical acceleration imposed on a structural system.

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