IMPROVING THE RESISTANCE TO PROGRESSIVE COLLAPSE OF STEEL AND COMPOSITE FRAMES

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Doctor of Philosophy

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Abstract

Several well publicised examples of progressive collapse have heightened concerns about the need to address robustness as a design requirement. Although research around the subject has been aimed at understanding the mechanics of progressive collapse, little work has been done on translating findings into better guidance on how to ensure adequate resistance without relying on the current prescriptive rules. Based on the Imperial College London method, which provides a soundly based analysis framework for calculating and comparing the performance of different designs, the work presented herein introduces a methodology for making realistic and effective design interventions, in order to allow designers to effectively enhance the robustness of their structure. This strategy is illustrated for both steel and composite frames and covers structures designed for both seismic and non seismic locations. Using the proposed step-by-step methodology, it is possible to redesign a simply designed composite frame in a way that it will be sufficiently robust to cope with any sudden column removal scenario. Comparison with simply increasing tying capacity reveals that the latter does not have a direct and proportional effect on the frame's resistance and should be used within a more informed context. With the aim of performing a complementary study for moment resisting steel frames, three types of popular welded connections are modelled under progressive collapse loading conditions using the Component Method. Also, an analytical solution for the prediction of the response of irregular beam systems under sudden column loss is presented. Despite the excellent performance of most floor systems, moment frames are found vulnerable to certain column loss scenarios. Thus, these scenarios are further examined with the express purpose of identifying how the frame might best be configured so as to provide the necessary resistance. The findings show how design for seismic resistance and design to resist progressive collapse do not necessarily align and highlight which structural properties are the most important to consider in each frame type, therefore encouraging the use of the proposed redesigning methodology, which is capable of effectively remediating robustness by efficiently addressing localised weaknesses.

Declaration of Originality

This submission is my own work. Any quotation from, or description of, the work of others is acknowledged herein by reference to the sources, whether published or unpublished.

Carolos Antonios Vidalis

Copyright Declaration

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A	Beam cross-sectional area
A_r	Coefficient for the effective axial stiffness of the beam and the axial support
B_{eq}	Equivalent flange width for the reduced beam section connection (equation 3.7)
d'	Connection equivalent lever-arm *
d	Vertical distance between the compressive centres of the support and mid-span connections of a double-span beam system
C_j	Corner column removal scenario (figure 6-2)
D	Beam depth
e	Connection axial extension due to component deformation
e_{col}	Connection axial extension due to column bending
E	Modulus of elasticity
EA	Beam axial rigidity *
EI	Beam flexural rigidity *
$E_{i,j}$	Exterior column removal scenario in the i direction (figure 6-2)
F_i	Force of connection component *i* *
$F_{i,RD}$	Capacity of connection component *i* *
F^C	Connection compressive force *
$F^C{}_{RD}$	Connection compressive capacity *
$F^C{}_{max}$	Connection maximum compressive force within the compressive arching stage provided the connection compressive capacity is not exhausted
F^T	Connection tensile force *
$F^T{}_{RD}$	Connection tensile capacity *
g_k	Unfactored dead load per unit area
$I_{i,j}$	Interior column removal scenario in the i direction (figure 5-18)
h_i	Lever-arm of connection component *i* *
$k^e{}_{eq}$	Connection initial tensile stiffness
$k^p{}_{eq}$	Connection post-limit tensile stiffness
K_{br}	Effective axial stiffness due to bracing provided to the support column

K_i	Stiffness of connection component *i* *
$K^e{}_i$	Initial stiffness of connection component *i* *
$K^p{}_i$	Post-limit stiffness of connection component *i* *
K^a	Effective axial stiffness of the beam and the axial support
K^{aC}	Effective axial stiffness consisting of K_a and K_C
K^{aEI}	Effective axial stiffness consisting of K_a and K_{EI}
K^C	Connection compressive stiffness
K^{EI}	Equivalent axial stiffness associated with the beam flexural rigidity (*EI*)
$K^a{}_{eff}$	Effective axial stiffness of beam system
$K^e{}_{eq}$	Effective tensile stiffness of the beam system associated with the connection initial tensile stiffness
$K^p{}_{eq}$	Effective tensile stiffness of the beam system associated with the connection post-limit tensile stiffness
$K^C{}_R$	Stiffness of the compressive rigid link of the connection mechanical spring model (Figure 3-2)
$K^T{}_R$	Stiffness of the tensile rigid link of the connection mechanical spring model (Figure 3-2)
$K^S{}_R$	Stiffness of the shear rigid link of the connection mechanical spring model (Figure 3-2)
K_s	Support axial stiffness
L	Beam span
M	Connection bending moment (the contribution of the beam axial load is included) *
$M_{b,Ed}$	Beam design bending moment
$M_{b,Rd}$	Beam sagging moment capacity
M_j	Connection bending moment (the contribution of the beam axial load is excluded) *
$M_{j,Rd}$	Connection moment capacity *
n	Number of the intermediate beams in a single structure al bay
N	Beam axial load
$N^C{}_{max}$	Beam maximum compressive axial load developed within the compressive arching phase
P	Beam or grillage total gravity load
$P^0{}_{d,C}$	Beam or grillage maximum pseudostatic compressive arching capacity associated with elastic connection compressive behaviour
$P^{max}{}_{d,C}$	Beam or grillage maximum pseudostatic compressive arching capacity associated

	with elastic or inelastic connection compressive behaviour
P^{y}_{d}	Yield capacity of the beam or grillage system
$P_{d,Ed}$	Grillage total dynamic design load
q	Beam or grillage distributed gravity load
$q_{b,Ed}$	Beam static design load per unit length
$q^{0}_{b,Ed}$	Beam maximum static compressive arching capacity associated with elastic connection compressive behaviour
q^{0}_{C}	Beam or grillage maximum pseudostatic compressive arching capacity associated with elastic connection compressive behaviour
$q^{0}_{d,C}$	Beam or grillage maximum pseudostatic compressive arching capacity associated with elastic or inelastic connection compressive behaviour
q^{y}_{d}	Yield capacity of beam or grillage system
$q_{d,Ed}$	Grillage dynamic design load per unit area
$q_{d,Rd}$	Grillage resistance
q_{k}	Unfactored imposed load per unit area
q_{Ed}	Grillage static design load per unit area
r_{m}	Ratio between the connection hogging moment capacity and the beam design bending moment
r_{q}	Ratio between the grillage dynamic and static design loads
R	Radius of the reduced beam section cut region (figure 3-4)
R^{C}	Compressive force transferred via the compressive rigid link of the connection mechanical spring model *
R^{T}	Tensile force transferred via the tensile rigid link of the connection mechanical spring model *
s_{t}	Beam transverse spacing
S_{j}	Connection rotational stiffness *
S^{e}_{j}	Connection elastic rotational stiffness *
S^{p}_{j}	Connection post-limit rotational stiffness *
T	Connection tying capacity *
U	Connection axial deformation *
$V_{j,}$	Connection shear loading
$V_{j,Rd}$	Connection shear capacity
w_{d}	Beam dynamic deflection
W	Beam static deflection

$W_{i,Rd}$ Beam section moment capacity

$x_{c,i}$ Ratio between the maximum pseudostatic compressive arching capacity and the yield capacity of beam system *i*

$X_{m,j}$ Ratio between the connection sagging and hogging moment capacities of beam system *i*

z Lever-arm between the connection compressive centre and the beam axial load *

$z_{m,j}$ Coefficient of the design bending moment of beam *i*

Z Vertical distance between the neutral axes of the beam cross-sectional areas located within the regions of hogging and sagging bending moments

α Grillage weighting factor associated with the loading distribution

α_i Weighting factor of beam *i* associated with the loading distribution

β_i Deformation compatibility factor of beam *i*

δ_{max} Beam maximum mid-span flexural deflection

Δ_i Deformation of connection component *i* *

Δ^{α} Axial deformation of the beam system

$\Delta^{\alpha,C}{}_{max}$ Maximum compressive axial deformation of the beam system

Δ_b Beam equivalent axial deformation due to bending

Δ^T Connection tensile deformation *

Δ^C Connection compressive deformation *

$\Delta^C{}_{max}$ Connection maximum compressive deformation *

Δ_s Deformation of axial support

e_u Steel flange ultimate tensile strain *

θ_1 Connection rotation associated with deformation of the tensile components *

$\theta^f{}_1$ Connection rotation capacity associated with deformation of the tensile components*

θ_2 Connection rotation associated with deformation of the compressive components *

μ Strain hardening coefficient

ρ Composite slab reinforcement ratio *

Φ Connection total rotation *

* Where discrimination between quantities associated with sagging and hogging bending moments is required, the parameter associated with hogging bending moment is denoted by the corresponding notation with the addition of a note.

Chapter 1
Introduction

1.1 Background to the research

1.1.1 Structural robustness

Robustness is a necessary structural property in order to ensure that public confidence in infrastructure, i.e. the "built environment", is retained, as new and unforeseen incidents are inevitable in the future.

From a designer's point of view, the concept of Robustness is similar to - although not directly comparable with - the more pervasive Limit States thinking and the Performance Based concepts. The need to address an even greater number of potentially critical situations has gradually and incrementally been recognised during the previous and present century, hence the ability of a structure to survive an unforeseen event or contain the consequences of a localised incident to the original incident, has become an area of intensive research.

In building structures, robustness is generally associated with structural redundancy, which allows the development of alternative load paths and redistributing the forces originally carried by the affected region to the undamaged member(s), thus permitting the structural system to maintain its integrity. This requires that the "links" of the system provide sufficient strength and ductility; otherwise structural continuity can be quickly lost leading to undesirable brittle modes of failure.

1.1.2 Progressive and disproportionate collapse

Progressive or disproportionate collapse is described as "collapse to an extent disproportionate to the cause" and is usually triggered by unforeseen extreme events (ODPM, 2004b). Its effects range from human losses and great financial damage to public psychological shock due to the dramatic extent of the catastrophe. Examples of the potential abnormal loads that can trigger progressive collapse include: aircraft impact, design/construction error, fire, gas explosions, accidental overload, hazardous materials, vehicular collision, bomb explosions etc. The robustness of a building is defined by its ability to resist damage disproportionate to the original cause, rather than prevention of total failure, due to fact that the triggering event assumes structural damage has already taken place.

The difference between the two terms is subtle: progressive collapse occurs when the cause leads to the collapse of additional structural elements apart from those initially damaged; it is not immediate, like, for example, damage from a huge blast. Another definition of progressive collapse (GSA, 2003) is:

"Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause."

Thus, while both terms describe the same thing, it is possible to claim that disproportionate collapse focuses on the damage assessment of the building while progressive collapse focuses more on the structural mechanism involved.

Although the issue did not initially receive extensive attention from structural engineers, a number of high profile disasters brought it into consideration. Nevertheless, designing buildings to resist progressive collapse requires a very different approach compared to designing for other loading cases such as earthquake or wind (Nethercot et al., 2007). In fact, the complex nature of the phenomenon, which includes gross deformations, large strains, inelastic material behaviour, change of geometry effects, dynamic effects and the varying propagating actions (separation of structural members, impact of failed components etc.) requires not only a comprehensive understanding of the main physical features but also a well-thought analysis methodology for evaluating and comparing the performance of different building designs.

Gradually, requirements for avoiding such scenarios have been incorporated in building regulations throughout the world and an effort to put these into practice was carried out by the introduction of provisions in the respective material-specific design codes.

1.1.3 High profile incidents

Due to the unpredictable circumstances of an extreme event, incidents have significantly varied in terms of the triggering event (accident, blast, fire, debris damage, vehicle, train or aircraft impact, hazards due to human errors during the design, construction or operation of a structure, lack of proper maintenance, unauthorised/inadequately planned structural modifications, environmental hazards, malicious acts and attacks), the type of structure involved (bridges, tunnels, towers, etc.) and the extent of the damage both in structural (partial or total collapse) and human loss terms. Although consequences for public opinion and the political and structural world have been very different depending on the combination of the above, the common denominator has been highlighting the need for advanced "pre-emptive" everyday design requirements for certain types of buildings in order to make them safer.

1.1.3.1 Damaged buildings during the World War II

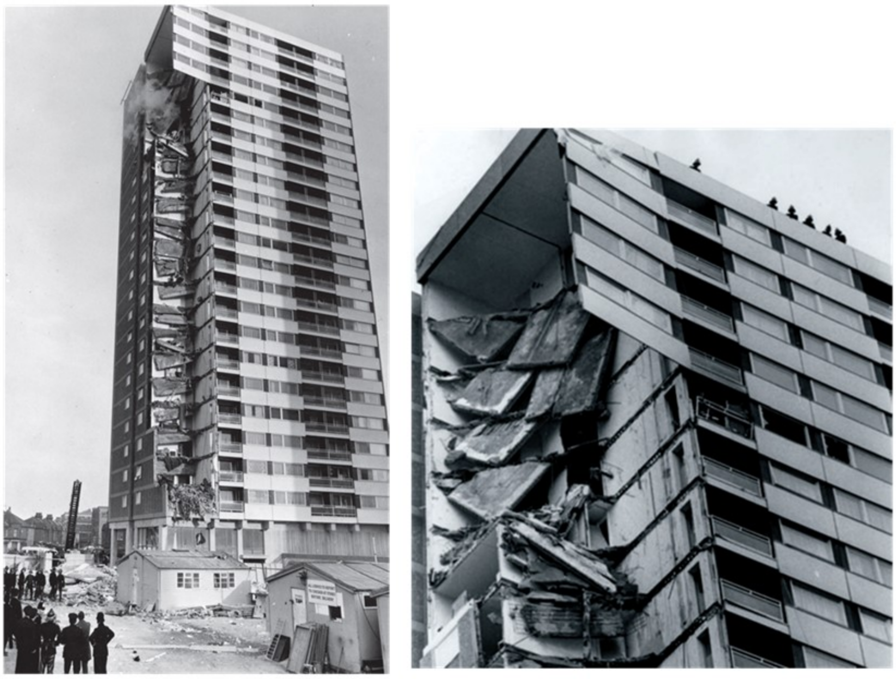
The behaviour of structures following bomb damage during the Second World War is a valuable source of material for identifying certain major points concerning the topic. Progressive collapse failures during that era were principally associated with weak connections in the structural system (Byfield, 2006). Figure 1-1 shows an example of catenary action in a damaged building (Smith et al., 2010), where sufficient anchoring of the members via tying forces at the connections has allowed the structure to attain a new equilibrium position without suffering separation of members, despite the damage sustained and the substantial deformations and deflections comparable to the depth of the beam.

Figure 1-1: Example of a London building damaged from the Blitz in World War II (Smith et al., 2010)

1.1.3.2 Ronan Point Tower

The Ronan Point building collapse in 1968 was a critical event that changed the way UK structural engineers considered robustness and revealed the need for introducing pertinent provisions in the design codes.

The corner area of the 22-storey precast concrete building in Newham (East London) collapsed over its entire height following a piped gas explosion in an 18th floor flat (Figure 1-2). Damage from the blast displaced the load bearing wall elements and collision with the lower floors led to a vertical progressive collapse. Investigations, at that time (Griffiths et al., 1968) and during its demolition (Pearson and Delatte, 2005), led to the conclusion that the primary causes of the initial damage were the limited resistance of the load-bearing walls to lateral loading and – most importantly – workmanship flaws at critical structural connections; the direct damage from the explosion was insignificant. Additionally, failure propagation was not arrested due to the lack of continuity and structural redundancy in the upper floors.



a. Collapsed area **b. Location of the initial damage**

Figure 1-2: Section collapse of the Ronan Point Tower in Newham (source: http://www.newhamstory.com/)

A series of updates in the technical evaluation criteria and the associated guidance for performing a structural assessment of large panel system (LPS) dwelling blocks in particular (Matthews and Reeves, 2012) were considered necessary after the incident. Although no occupied UK LPS dwelling block has experienced any similar disproportionate collapse since 1968, recent demolition of several LPS blocks has resulted in their partial progressive collapse, which raised questions about the potential vulnerability of this form of construction (Confidential Reporting on Structural Safety, 2010).

The collapse triggered intense work on the subject, which led to the UK becoming one of the first countries to have introduced provisions against progressive collapse (Bussel and Jones, 2010, Pearson and Delatte, 2005, Taylor, 1975, Ellingwood and Leyendecker, 1978). Although the annual probability for a similar significant accidental event is very low (10^{-6}), the historical aspects of the partial collapse of the Ronan Point, plus the wider social and emotive considerations can validly introduce a different perspective to the implications of such a phenomenon (Matthews and Reeves, 2012).

1.1.3.3 Murrah Building (Oklahoma City bombing)

In April 1995, the Alfred P. Murrah Federal Office Building in Oklahoma City suffered a large explosion at one of the middle ground floor columns, which resulted in the partial collapse of almost half of the building (Figure 1-3a), claiming 168 lives.

The nine-storey building was made with reinforced concrete and although its perimeter was designed based on an ordinary moment frame arrangement, in order to allow double spacing between the principal columns at the first two levels, a continuous girder transfer arrangement supported every second exterior column (Figure 1-3b). Studies later pointed out that loss of these columns resulted in losing the third-floor transfer girder, which led to collapse propagation well beyond the zone of the immediate blast damage (Mlakar et al., 1998, Sozen et al., 1998). The official report by FEMA (Corley et al., 1996) concluded that the structure was unable to prevent progressive collapse mainly due to loss of structural integrity (despite complying with standing code requirements), as the connections were not designed to provide the increased strength and ductility required to effectively redistribute the new loads following the abrupt shear failure of certain columns.

a. Original structure b. Extent of the collapsed area

Figure 1-3: Murrah Building partial collapse (source: http://www.oklahomacitybombing.com/)

Certain studies (Hayes J.R. et al., 2005, Corley et al., 1998) concluded that the consideration of seismic design provisions would have resulted in a more robust frame, whereas use of a seismic resisting structural system could have contained the collapsed area to 15-50% of its original extent by limiting both the extent of initial damage and the potential for progressive collapse.

Almost immediately following the incident, the U.S Government established the Interagency Security Committee (ISC) charged with the responsibility of ensuring the security of Federal buildings. This resulted in the development of certain very comprehensive guidelines and design methods in the U.S., which are discussed in Section 2.2.3.

1.1.3.4 World Trade Center (WTC Twin Tower and WTC 7)

1.1.3.4.1 Global impact

In the same manner that the Ronan Point collapse changed the way engineers in the UK perceived structural safety, the dramatic events of the 9/11 attacks and the subsequent collapse of the WTC complex had a significant and worldwide impact on both the general interest in progressive collapse and on the public concern about its consequences. For example, Moazami Kamran, one of the structural engineers involved in the design of the Shard Tower (the tallest building in the EU, built in London in 2013), says (Moazami and Agrawal, 2013):

"More attention has been given to the robustness of buildings. With every new building, we examine the possibilities of failures due to terrorist attacks. I remember how, not long after 9/11, we were working on the Barclays Bank Headquarters in London and people were so sceptical about tall buildings that nobody wanted to move into it, so we had to make it special. We made it very robust.

This contrasts greatly with the statement made by Bruce Ellingwood at the 1998 Structural Engineers World Congress:

"There is currently a virtual absence of research activity or interest in the U.S. in the topic".

1.1.3.4.2 WTC Twin Towers collapse

The 110 storey WTC Twin Towers' design consisted of a network of closely-spaced perimeter columns and deep beam spandrels forming together a robust steel frame-tube system as well as a secondary system of more widely-spaced columns in the core. A "hat truss" (type of steel truss system) located at the top four floors connected the perimeter and core columns. The floors were supported by steel truss beams and constructed with lightweight concrete over steel decking.

On the 9th of September 2001, the towers suffered extensive (though not critical) damage from the impact of large commercial aircraft (Figure 1-4a). The damage, combined with the ensuing strong and uncontrolled fires, which significantly weakened the structural steel, resulted in the collapse of both of the towers. The South Tower collapsed in less than an hour after the aircraft impacted and the North Tower collapsed half an hour after that. As a result of the attacks to the towers, 2,752 people died, including all 157 passengers (including the hijackers) and the crew aboard the two airplanes. Two main investigations were carried out after the incident:

i. The Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE) preliminary building performance (Corley, 2002) suggested that fires in conjunction with damage resulting from the aircraft impacts were the key to the collapse of the towers.

ii. The National Institute of Standards and Technology (NIST) began a comprehensive investigation (Shyam-Sunder, 2005), which included laboratory tests and exceptionally detailed computer simulations. The NIST investigation focused on identifying "the sequence of events" that triggered the collapse, rather than on providing a detailed analysis of the collapse mechanism itself (after the point at which events made the collapse inevitable). The study started in 2002, was completed in 2005 and its estimated cost was around $16M. However, in 2007, NIST initiated a long-term project towards understanding and enhancing structural robustness.

The conclusions of the studies were that:

i. The impact heavily damaged key structural components (perimeter and core columns, floor slabs) and destroyed most of the thermal insulation of the remaining members within the affected area.

ii. The structure, however, was designed to sustain aircraft impact and managed to redistribute the forces in the perimeter frame-tube system (thanks to structural redundancy) and in the core to the perimeter (thanks to the top hat truss).

iii. Nevertheless, the intensity of the ensuing fires, spread over several floors, significantly degraded the structural properties of critical load-bearing components within and close to the affected area. This led to loss of the vertical load carrying capability and collapse of the upper part of each tower (Torero, 2011).

iv. Collision of the floors during collapse generated extremely large impact forces, causing a "pancake collapse" (immediate and progressive series of vertical floor failures) that led to the total collapse of each tower (Figure 1-4b).

a. Moment of aircraft impact on the North Tower

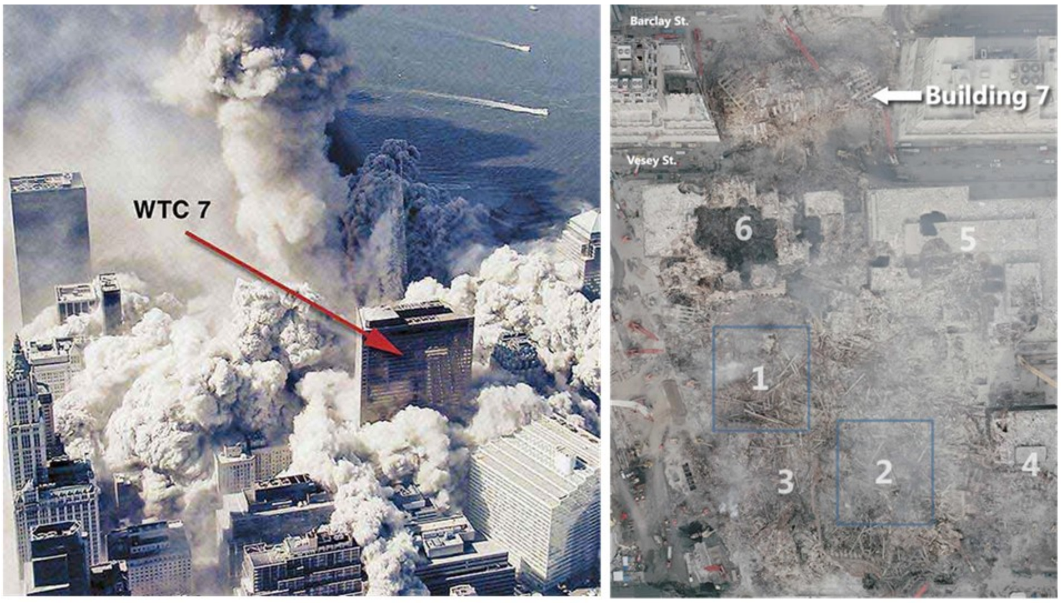
b. Progressive collapse of the North Tower

Figure 1-4: Collapse of the WTC Twin Towers

1.1.3.4.3 WTC 7 Building collapse

Seven hours after the two towers fell, the 47-story WTC 7 collapsed (Figure 1-5). The final report on the collapse of the WTC 7 (Gann, 2008) concludes that it was a fire-induced progressive collapse. The impact of debris from the collapse of the WTC 1 ignited the fires and caused structural damage to the exterior of the frame (Corley, 2002). Collapse initiation was very similar to the scenarios considered in theoretical studies (including those within the present Thesis): a critical interior column lost its load bearing capacity and buckled after it became unsupported over approximately nine stories. This led to a vertical progression of floor failures up to the roof (witnesses mentioned a visible effect on the west roof penthouse). As adjacent interior columns also became unsupported, they started to buckle and as the core started to collapse, the buckling progressed to the exterior columns, leading to the global collapse of the structure.

Among the main contributing factors were thermal expansion (quite pronounced due to the long-span floors) and the inability of the structural system to prevent fire-induced collapse (for example, the gravity-resisting connections at the interior were not designed to cope against thermally induced lateral loads), although assigning the exact proportion of contribution to each factor requires more research. Another possible contributing factor was the unusual design, necessary due to the presence of power transformers in the ground floor, in which ground floor exterior columns supported exceptionally large loads corresponding to approximately 185 m^2 per floor. The preliminary reports' analysis showed that taking out one column of the lower floor could potentially trigger progressive collapse of that section of the building.



a. WTC 7 stands amid the rubble of the recently collapsed Twin Towers

b. Collapsed WTC 7 view from above

Figure 1-5: World Trade Center buildings' total collapse (source: FEMA 304 presentation)

1.2 Robustness design approaches

Designing for robustness can be approached from two different viewpoints (Starossek, 2009, IStructE, 2010, Ellingwood and Dusenberry, 2005):

i. Preventing local failure (risk management, event control and risk reduction approaches)

ii. Assuming local failure (response-based assessment)

The first includes preventing blast, fire, impact or other loading combinations from occurring and requires input from other engineering fields as well. Once local failure is assumed, the response of the structure is naturally a structural engineering concern and is termed as the resistance of the structure against progressive collapse.

In order to prevent local failure, probabilistic methods are used for conducting a risk management study. The basic principles, system representation and risk criteria for integral risk based decision making in engineering have been documented by various bodies (CIB, 2001, Joint Commitee on Structural Safety, 2001, ISO, 1998, COST Action TU0601, 2011b). In the UK, the ICE has published a related document with regards to the ALARP (As Low As Reasonably Practical) and the SFARP (So Far As Reasonably Practicable) risk reduction concept (Institution of Civil Engineers Health and Safety Panel, January 2010). Even though the approach is independent of the latest developments, such as new analytical or numerical methods and special techniques for specific technical investigations coming to the fore, it remains generally methodological and largely philosophical; the main steps, outlined below, appear to depend on subjective criteria:

i. Identify all hazards and define the corresponding hazard scenarios. A variety of techniques is available to assist the engineers including fault tree, event tree, decision tree, causal networks, ALARP, PHA, HAZOR, FORM or others.

ii. Estimate, for every possible hazard scenario, the possible consequences and probability.

iii. Compare with the established risk acceptance criteria, which depend on professional, social, economic and political conditions.

One of the main means of prevention is the implementation of measures to restrict access close to critical structural components (GSA, 2005), such as:

- Zones of protection, also referred to as “standoff” or “buffer zones” (Figure 1-6)
- Securing sites adjacent to the building
- Access control infrastructure & protocols (surveillance, intrusion detection and screening)
- Vehicular control (traffic restrictions, perimeter protection zone)
- Non-structural details of interior and exterior design that can enhance security

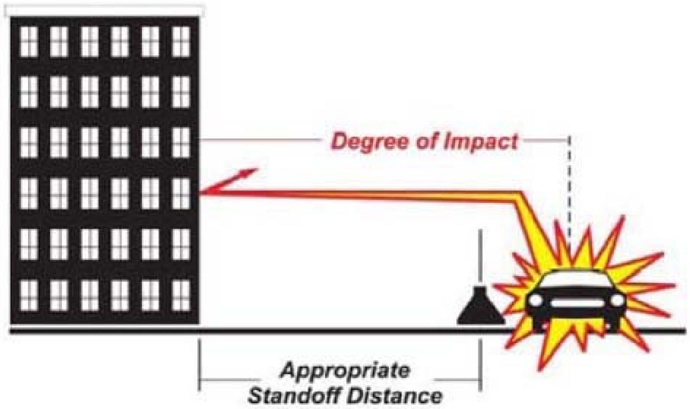


Figure 1-6: Example of a standoff distance measure (source http://transit-safety.volpe.dot.gov)

Although these measures are more readily available (they do not require significant research background), they do not increase the inherent resistance of the structure to progressive collapse in the case that local failure cannot be prevented. Moreover, they are mainly effective in reducing - without totally eliminating - the risk against the effects of a malicious act, while structural robustness in modern buildings is necessary for a broader range of reasons which can be inherently unpredictable and not always related to an attack.

1.3 Progressive collapse resistance design approaches

Following each of the aforementioned high profile incidents, the design approaches that were developed and expanded to safeguard structures from a similar scenario mainly fall within one of the following categories:

i. Prescriptive methods (also referred to as “indirect design methods”)

ii. Performance based methods (also referred to as “deterministic” or “direct design”)

Tying capacity, the main indirect design method, is also the most common amongst provisions employed by present regulations and building design offices in order to evaluate the resistance to progressive collapse. The main idea can be summarized as designing the horizontal elements to alternatively act as a catenary upon the loss of a column and ensuring that the edge connections have the necessary tensile capacity to carry the weight of the floor (Figure 1-7). In this sense, the "tying" provisions could be considered as a special case of the alternate load path approach.

More specifically, for buildings required by regulations to be specially designed to mitigate the effect of accidental removal of supports, the requirements introduced are general tying, tying of edge columns, continuity of columns, resistance to horizontal forces and provision for heavy floor units. Horizontal ties can be steel members, steel bar reinforcement or steel mesh reinforcement and should be provided at each principal floor level and at the roof. All horizontal ties and members should be capable of resisting a factored tensile load which should not be considered as additive to other loads and should exhibit robustness equivalent to the other parts of the structure.

Although tying provisions are of a prescriptive nature, their main advantages are their simplicity to be appreciated and the simple calculations required for their application. They do not require sophisticated design practices except for non-continuous columns, long spans and other special factors, which can lead to considerable tying forces requirements.

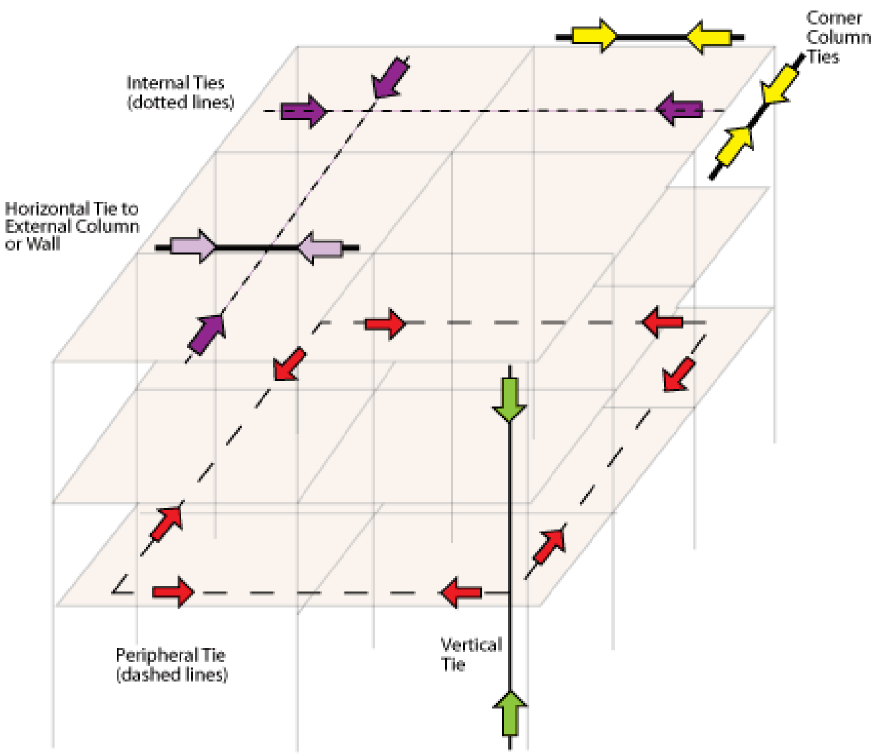


Figure 1-7: Structural tying of framed building, source: UFC 4-023-03 (DoD, 2009)

Usually, if the tying requirements are not met, then “notional removal” provisions are used to avoid progressive collapse as a result of column damage, where performance of the remaining structure to some reduced level of applied load is checked using the **alternate load path approach**. These provisions are performance-based and follow the most deterministic approach. In addition, they have the advantage of being “threat-independent”, i.e. independent of the triggering event. Their level of accuracy depends on the type of analysis used: while conventional design checks may ignore beneficial nonlinear phenomena, if paired with sophisticated nonlinear dynamic numerical or analytical approaches, the alternate load path method can offer not only meaningful insights into structural behaviour during progressive collapse but also more reliable design solutions compared to the other methods presented herein.

Another type of direct design is the **key elements** approach, in which certain principal structural members (for example: transfer girders and their supporting columns) are designed for higher loads or with additional protective measures. This approach focuses on preventing the collapse triggering event and is recommended in the case where the alternative load path analysis concludes that the structure cannot overcome suffering local failure(s). Although preventative measures cannot provide an absolute guarantee of safety as outlined in the previous section, this approach can be used in conjunction with other methods in order to reach the most cost-effective design solution. A detailed critical appraisal of these design approaches is presented in Section 2.2.4.

1.4 Outline of the current study

1.4.1 Motivation for the present research

Even though research activity on progressive collapse has experienced a boom during the past two decades, certain fundamental challenges have yet to be addressed:

i. Obtaining a comprehensive understanding of all the complex phenomena influencing the behaviour of a building during a progressive collapse scenario.

ii. Identifying how and where the structural engineer should intervene in order to efficiently enhance the robustness of the building.

iii. Developing competent guidelines capable of providing efficient and safe design provisions for routine design purposes.

Furthermore, there is a lack of experimental data due to the inherent difficulty of reproducing and monitoring a progressive collapse scenario. Thus, until recently, the biggest challenge had been developing a method able to provide researchers with a simplified framework for progressive collapse assessment. In fact, for new requirements to be introduced, grounds for a widely accepted quantitative method for estimating the robustness of the building must exist. Such a tool should be able to efficiently assess performance and should be readily available to be applied by professional structural engineers. However, most of the analytical approaches that have been developed involve the use of complex and demanding numerical analysis, which renders them unsuitable for use in routine design.

The Imperial College London Method, initially applicable for multi-storey buildings under sudden column loss, has managed to overcome this barrier by capturing all the important physical features. In addition, it involves only manageable calculations and provides the possibility of quantitative evaluation. Since an appropriate tool now exists, researchers can focus on how to enhance robustness of a design and on updating current guidelines.

Moreover, current provisions lack a widely accepted, straightforward and ready to use design framework that entails comparing performance against prescribed limits. The process, typical of conventional structural design, can be summarised with the following set of tasks:

i. Assessment and suitable representation of the loading conditions

ii. Representation of the structure in a way that allows conducting an analysis

iii. Comparison of the results against given provisions

Thus, for example, designing a building subject to wind loading requires constructing an idealised pattern of lateral loading based on observation of past requirements, combined with some form of frame analysis and comparison of the resulting displacements and critical loading resistance against safe values outlined by the relevant design guidelines. Obviously, this requires the skill of making certain assumptions in order to reduce the complexity of the problem but also of being able to interpret the results in the context of the real arrangement.

Evidently, this contrasts with the approaches that are currently available for designing against progressive collapse or enhancing robustness, the most popular of them being tying capacity provisions. Their main advantage is the ease with which they may be applied using simple calculations. Notwithstanding, they remain prescriptive and efficient only in a limited number of cases while being irrelevant in others; it is impossible for the structural engineer to evaluate the actual performance of the structure in an extreme event let alone the safety margin offered.

All these challenges have been considered by researchers at Imperial College London over the past 10 years. The main contribution of the present Thesis, which forms part of this effort, is the introduction of a redesigning methodology that can be applied to any framed structure, allowing to explicitly identify the most effective and efficient interventions for enhancing the robustness of the structure.

1.4.2 Research objectives

The core objectives of the present research are directly linked to the following three issues:

i. If design provisions are to advance from tying capacity, more evidence is needed on the contribution of different mechanisms towards the resistance to progressive collapse for composite and bare steel frames. Although tying capacity is a favourite among designers because of its simplicity, it has been demonstrated to be adequate for some cases while for others not (Byfield, 2006, Nethercot et al. 2010, Stylianidis 2011, Vlassis et al. 2008a). Deeper understanding will help define its shortcomings and strong points and thus identify in which cases further provisions need to be adopted.

ii. Towards that direction, additional case studies will help evaluate the contribution of alternative mechanisms that might also be necessary to take into account. In fact, it appears that there is more than a single solution for improving resistance to progressive collapse, though most are limited by their cost and their compatibility with common construction practices. Thus, it is vital to concentrate on determining the most efficient way to enhance robustness of a building for certain given design configurations.

iii. Buildings designed against special loading cases have different design configurations and may or may not perform better in the case of a progressive collapse scenario. A very common example is structures with seismic reinforcement and sway frames designed for seismic regions, which form an important fraction of the world's buildings. However, it is still unclear whether seismic provisions are an effective and efficient way of enhancing resistance against progressive collapse. Studying the differences in the behaviour of the two types of construction is needed to help the designer identify which priorities need to be considered to make the structure more robust, depending on basic properties such as connection strength, stiffness and ductility, as well as frame arrangement.

1.4.3 Layout of the study

Addressing these questions requires developing the necessary tools for studying continuous structural systems, studying the behaviour of earthquake resistant frames in progressive collapse, developing a methodology for enhancing the robustness of either a simply designed or a moment resisting frame and comparing its efficacy and efficiency with current progressive collapse and seismic provisions.

Chapter 3 and Chapter 4 both focus on the first step by introducing the required advancements to previous connection and beam system models for applying the Imperial College London Method to non-continuous and continuous frames.

Chapter 3 studies the behaviour of three types of fully welded moment resisting connections under the loading conditions experienced in progressive collapse. Based on a previously developed suitable analytical connection model for partially restrained endplate bolted connections (Stylianidis, 2011), an explicit solution linking the connection deformations with the combined bending moments and axial forces (in the presence of axial restraint or bracing in beams) is derived and validated against both experimental, in-house numerical and third-party numerical results. By conducting a series of parametric tests, it is possible to identify which parameters have the most important influence on the performance of fully welded connections under progressive collapse loading conditions.

These models are incorporated into an extended slope-deflection approach within an analytical method for predicting the nonlinear static and pseudostatic response of axially restrained and unrestrained steel beams under sudden column loss. Since the existing model (Stylianidis, 2011) only considers fully symmetrical beam systems satisfying the double span condition, it is necessary, for the study of moment frames with more irregular beam systems (for example, with a simple connection to a support column or with different beam lengths on each side of the removed column), to develop a new analytical solution in Chapter 4, able to consider the behaviour of more complex arrangements.

Chapter 5 presents one of the key outcomes of the Thesis, which is a method to use the findings in a form that allows designers to address progressive collapse in a broadly similar way to that used when considering ultimate static strength or serviceability deflections. In doing this, it builds on the sufficient understanding of the mechanics of progressive collapse that has emerged from several previous projects at Imperial College London.

The initial application of the redesigning methodology aims at proposing a solution to improve the resistance in progressive collapse of the simplified version of the Cardington test frame model and of a bare steel equivalent.

The first part of Chapter 5 examines the beam and grillage systems' pseudostatic responses, assesses the performance of the two frames and highlights the differences in behaviour between composite and bare steel construction. Both arrangements fail to provide the necessary resistance, with the bare steel being inherently less robust.

The second part of Chapter 5 uses the results of parametric studies and of bibliography in order to suggest and justify explicit changes in the connection design that will improve the system response. It introduces the step-by-step method to determine the most efficient and practically applicable changes, making it possible to redesign the composite frame in a way that it will be sufficiently robust to cope with any sudden column removal scenario. The comparison of the methodology with simply increasing tying capacity reveals that the latter does not have a direct and proportional effect on the frame's resistance. This leads on to consideration of how tying capacity might be used within a more informed context.

By employing the tools of Chapter 3 and Chapter 4 in Chapter 6, it is possible to expand the study into the performance of steel moment resisting frames in terms of their ability to withstand progressive collapse following loss of a ground level column. The quantitative performance assessment of five representative exemplar frames, selected from the NIST Robustness Project and the SAC Joint Venture, identifies the main influencing parameters, how the interaction between the continuous and non-continuous systems affects the floor response and the most common vulnerabilities of moment frames. The comparison with the findings of the previous chapter highlights the differences in the main factors that affect behaviour between the two types of frames.

By using the results of the assessment exercise of Chapter 6 and the redesigning methodology of Chapter 5, it is possible to identify solutions that will allow moment frames to withstand removal of any perimeter column. Compared to simply designed frames, more intrusive design interventions are required to enhance robustness, as care is required to avoid a conflict with seismic requirements. By taking into account the findings of all previous chapters, it is possible to clarify on the relationship between seismic provisions and robustness and determine whether the latter are an effective means of improving resistance against progressive collapse.

Finally, the most important conclusions of the study are summarized in Chapter 8 and several suggestions for future research are provided. Among others, it is suggested that the new developments supplied to the Imperial College London Method in Chapters 3, 4 and 5 and the outcomes of Chapters 6 and 7 will facilitate future research studies on the development of more effective, economical and quantitative-based provisions against progressive collapse.

It has been possible to both publish and present aspects of this research prior to final submission of the Thesis and some work can be found in the following:

- Journal publications: (Vidalis and Nethercot, 2013b).
- Conference proceedings: (Nethercot and Vidalis, 2012, Vidalis C A, 2012, Vidalis and Nethercot, 2013a, Nethercot and Vidalis, 2013, Vidalis, 2013, Vidalis C A, 2014).
- Oral presentations: IStructE Young Researchers Conference 2012 and 2013[a] (London, UK), PSSC 2013 (Singapore) and IStructE North Thames Regional Group meeting June 2013 (London, UK).

[a] The paper received the 1st prize in the oral section in the IStructE 2013 Young Researchers Conference.

Chapter 2
Literature review

2.1 Introduction

Structural design, over the past century, has evolved in many ways, which include moving from strength design to performance design and from coping with everyday operational scenarios to ensuring adequate performance in critical scenarios. With the study of probabilistic models, researchers were able to identify, among others:

i. The likelihood of abnormal loads occurring during the life cycle of a structure.

ii. Which safety factors should be considered in the design process in order to ensure an appropriate balance between safety and economy.

iii. The target reliability of design provisions and material minimum qualifications.

A recent comprehensive review, undertaken on behalf of the UK Government (Cormie, 2011), shows that robustness is still under the spotlight and that the need to be able to design more robust buildings is relevant now more than ever before. This is reflected by the important number of regulatory and academic publications, which are presented in the next pages. Section 2.2 presents the current state in formal provisions for robustness, the informal guidance that accompanies them and the critical appraisal (Section 2.2.4) from the scientific community, including the author. Section 2.3 explains which methods can be used for analysing the robustness of a structure, while Section 2.4 documents a novel framework for assessing resistance in progressive collapse called the Imperial College London Method. The current state of the art in research on progressive collapse is presented in Section 2.5. Section 2.6 summarises the main conclusions of this review and identifies which aspects of the problem still remain unexplored.

2.2 Provisions for progressive collapse design

2.2.1 Guidance and formal provisions

Among the first formal guidance in which the words “disproportionate” and “robust” appeared, was the CP 110 code of practice for concrete (BSI, 1972) published in 1972, which stated that:

"...there should be a reasonable probability that (the building) will not collapse catastrophically under the effect of misuse or accident..." and that *"...it should not be damaged to an extent disproportionate to the original cause"*.

Before that but following the Ronan Point collapse, the Institution of Structural Engineers published a paper (IStructE, 1971), which recommended that a continuous framed structure should be able to resist unpredictable loads and effects, as long as it was designed to accord to the codes of practice, and to provide adequate (to the specifications of that time) tying where appropriate, while slabs should be effectively anchored to the supports. It is reasonable to assume that the views presented in this publication, together with other reports (IStructE, 1968, IStructE, 1970), strongly influenced the drafting of future provisions, most of which are still in place today.

In 2010, almost 40 years after the Ronan Point collapse, the IStructE published a comprehensive guide (IStructE, 2010) that addresses most issues concerning progressive collapse and robustness, mainly from the designer's point of view. The document discusses the various concepts of robustness (structural form, element design, failure modes, response to events that can trigger progressive collapse, etc.) and how the legal and other obligations of the professional engineer are reflected in current regulations and codes of practice. The largest part of the guide focuses on how to apply the main methods used by designers for concrete (in situ and precast), steel, timber and masonry structures in order to fulfil the aforementioned requirements.

Due to the fact that, in the aftermath of the high profile incidents of the previous chapter, provisions had to be drafted often without the existence of others before them, most of them were – and still are – of a largely prescriptive nature; their main principle is that by adhering to them a better result will be achieved in terms of a more robust structure better able to resist progressive collapse than would have been the case otherwise. This approach differs from the typical structural design practice of being able to quantitatively assess the merits of a range of alternative arrangements as well as to provide quantitative measures of the different margins of safety associated with the different alternatives.

In some circumstances, formal provisions leave issues which are not specific or which are open to interpretation, such as the "systematic risk assessment" for Class 3 buildings in Approved Document A (ODPM, 2013a), in which case, the designer can refer to published guides (IStructE, 2010, Starossek, 2009, Marchand and Alfawakhiri, 2004, Way, 2004, NIST, 2007). These documents are often significantly more detailed and comprehensive compared to the formal provisions. In fact, practice has shown that guidance documents like these usually precede formal provisions and significantly influence future codes.

Due to the heightened public and professional concern that high profile incidents incite, it is among the responsibilities of the field leaders and policy makers to ensure that, while new structural provisions will offer reasonable safety, they will not impose severe supplementary design requirements, which can lead to certain construction practices to become unnecessary unpopular. The example of the set of steel box girder collapses in the early 1970s, which led to imposing a series of restrictions and significantly strict provisions in the UK (Firth, 2010), shows that extreme conservatism can be just as damaging as failing to restore public and professional confidence. A similar example is the approach that arose post the Ronan Point collapse, which is some cases resulted for uncalled (from today's point of view) strengthening works on large panel system blocks, or even their unnecessary demolition (Building Research Establishment, 1987).

Thus, provisions also need to be efficient for all the stakeholders involved. Just using bigger, larger structural elements is not always the answer. For example, designing robustness into the Shard Tower in London called for the combination of different approaches, which were well above the - usually prescriptive - formal requirements in the UK. Quoting some of the structural engineers involved (Moazami and Agrawal, 2013):

"We also carry out many risk assessments for scenarios such as planes colliding with the building and bombs going off. First of all we carry out security analysis on the building, in order to find out how to avoid potential hazards in the first place. For example, with bollards around the base of the building, trucks containing bombs can't get near enough. Next we look at the dynamic analysis of the building and see how it performs in these scenarios. Here we focus on collapse prevention and look at how we can strengthen specific parts of the building to prevent progressive collapse. However, adding more bones to the body doesn't necessarily make it stronger, so we have to make sure that we are working to optimise the structure as much as possible at the same time, not just adding material for the sake of it.

2.2.2 European and British design codes

2.2.2.1 Eurocodes

The Eurocodes are a comprehensive set of Standards intended to cover all aspects of structural design using conventional construction materials. The choice of certain factors or design methods may be different depending on the country in which the structure will be constructed; these parameters are published in a National Annex. Most of the clauses referring to robustness can be found in Part 1-7 of the BS EN 1991 (BSI, 2006), which also makes references to BS EN 1993 (BSI, 2010) and BS EN 1990 (BSI, 2002b). In fact, the BS 5950-2:2001 was replaced by the BS EN 1090 (BSI, 2008a) which is the UK implementation of Eurocode 3 EN 1090 for structural steelwork. It is accompanied by the actual BS EN 1990 (BSI, 2002b), which lays the basis of structural design. The BS EN 1090 states that:

"A structure shall be designed and executed in such a way that it will not be damaged by events such as explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause".

Amongst the provisions for avoiding and limiting potential damage is the selection of a robust structural form and design as well as the tying of structural members together, found in Section 3 of BS EN 1991-1-7. The approach of these provisions does not seem to have evolved since BS 5950 towards a more quantitative approach and even less practical information is given as to how to apply these requirements to the structural design process apart from what can be found in some designers' guides, which nonetheless usually follows the tendencies of the BS 5950. It is possible that since the Eurocode was established on the basis of a strictly quantifiable approach, the absence of significant research activity at the time led to a more conservative stance.

Annex A of the BS EN 1991-1-7 (BSI, 2006) provides more detailed guidance - even though it is "informative" rather than "normative" - very similar to that of the Approved Document A, which is presented in the next section. The points it covers include:

i. Consequences classes for buildings
ii. Recommended strategies
iii. Effective horizontal ties
iv. Effective vertical ties
v. Key elements

The Steel Construction Institute (SCI) has published a useful guide (Way, 2011) for the design using hot-rolled steel members for structural robustness in accordance with the Eurocodes.

2.2.2.2 Approved document A (AD-A) of the Building Regulations

The requirement for avoiding disproportionate collapse, as set forward under the Building Regulations (ODPM, 2004b), is accompanied by an official guidance document which explains how compliance with the regulatory requirements can be achieved. In England, this guidance document is termed Approved Document A (ODPM, 2013a) and mainly provides guidance on applying the robustness requirement A3 of the Building Regulations, which states that:

"The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause."

Requirements differ based on the Class of a building, as summarised in Table 2-1. A building is categorised in a Class, depending on the type of occupancy and the number of storeys.

Table 2-1: Summary of requirements for each Building Class

Class	Class description	Requirement
1	Small structures	No additional measures likely to be necessary
2A	Medium sized & low-rise	Effective horizontal ties for framed construction
2B	Medium to large sized & medium-rise	Effective horizontal ties with effective vertical ties in all support columns
		Performance check following notional removal of a supporting column or a beam supporting at least one column
		In the case of inadequate performance in the above, then corresponding elements should be designed as protected
3	Large high rise or special	Systematic risk assessment

The minimum levels of tying forces (applicable for Class 1 buildings) are defined in the corresponding material codes (BSI, 2005a, BSI, 2002a, BSI, 2001a, BSI, 1997). For structural steelwork, the minimum limit is 75kN, defined in Clause 2.4.5.2 of BS 5950-1 (BSI, 2001b) and can be provided not only by the network of steel beams and connections but also by reinforcement bars in a composite arrangement.

Effective ties, required for Class 2 buildings, are defined in Clause 2.4.5.3 of BS 5950-1 (BSI, 2001b). The requirements mainly affect vertical tying, as horizontal design tying forces are specified based on the maximum between the nominal value of 75kN and half the vertical load of the member (typically equal to the connection design shear forces, divided by two). Hence, internal and edge (peripheral) horizontal tying forces should be able to satisfy the following factored tensile forces respectively:

$$T_i = \max\left[0.50(1.4g_k + 1.6q_k s_t)\, L \cdot n \,,\, 75kN\right] \tag{2.1}$$

$$T_e = \max\left[0.25(1.4g_k + 1.6q_k s_t)\, L \cdot n \,,\, 75kN\right] \tag{2.2}$$

Where: g_k and q_k are the characteristic dead and imposed loads per unit area
S_t is the beam mean transverse spacing
L is the beam span
n is a factor that accounts for a reduction in the design tying force for less than five storeys

On the other hand, the aim of effective vertical tying is to ensure the continuity of the columns throughout the total height of the building. In order for this to achieved, all column splices should be able to provide tensile resistance equal to the maximum total factored vertical load that each column section supports at any floor level between that splice and the splice of the column at the underneath floor level.

For Class 2B buildings, the performance check requires that the floor area at any storey at risk of collapse should not exceed either 15% of the floor area of that storey or the area of 70 m^2, whichever is smaller (maximum limit: 100 m^2), and that it should not extend further than the immediate adjacent storeys. If certain element loss scenarios are found to be critical, then the corresponding elements have to be designed as protected for a specific value of uniform pressure acting over its surface, applied in each direction (horizontal and vertical) at a time, equal to a minimum of 34 kN/m^2 (equivalent pressure from a notional gas explosion, similar to that of Ronan Point), as specified in BS 6399-1 (BSI, 1996).

For Class 3 buildings, systematic risk assessment is required for all normal and abnormal hazards during the life of the building. This should identify which critical scenarios should be considered in the design process.

It is hard to say whether AD-A is more or less conservative than the Annex A of EN 1990. The Eurocode Annex contains clauses which vary in the following points:

i. The effects of combined actions are accounted for via the use of a coefficient "ψ", which is defined in Annex A1 to EN 1990.

ii. There is no reduction in the horizontal tying force requirement for buildings with less than five storeys.

iii. The resistance required for effective vertical tying is the tensile force equal to the largest total design load applied to the column at any floor level and not only between two column splices.

iv. The maximum acceptable area of collapse is the minimum between 15% of the floor area and $100m^2$, rather than $70m^2$.

The SCI has published a guide (Way, 2004) which focuses more closely on the tools and methods available for fulfilling the requirements of Document A. Its scope is similar to the more general SCI guide for the Eurocodes, however it includes additional practical details and design guidance for each building class, as well as worked examples. The current 2004 version of Approved Document A refers to national design standards and not to the Eurocodes. Following revision in 2013, it now includes references to the Eurocodes, as well as more detailed guidance for Class 3 structures.

2.2.3 United States of America Standards and Guidelines

Provisions in the U.S. are amongst the most detailed and comprehensive, with an intense drafting activity being recorded in the aftermath of the WTC collapse (Section 1.1.3.4). Essentially, they follow one or more of the original 3 approaches (effective tying, key element design and the alternate load path method), often embedded within a risk based assessment framework. In certain cases, they also contain some - partially complete - guidance on how to carry out the analysis of the damaged structure required by the alternate load path approach.

The design and construction of private and Federal structures is usually governed by separate codes and standards. Private construction is controlled by the National Standards and the building codes that each State adopts, with the most common one being the International Building Code (IBC, 2012). Material specifications follow the ASTM standards and design loads most often follow the ASCE 7-10 Standard (ASCE, 2010). Federal buildings on the other hand, are controlled by the General Services Administration requirements (GSA, 2003, GSA, 2005), which prevail over the National Recognised Codes (although they usually overlap). For all other structures, often built outside the U.S., the Department of Defence (DoD) United Services Facilities Criteria (UFC) (DoD, 2009) should be followed. For steel seismic design, FEMA publications (FEMA-350, 2000) are used.

2.2.3.1 GSA guidelines

2.2.3.1.1 General guidance in the Facilities' Standard P100

The Facilities' Standard (P100) document (GSA, 2005) separate chapter on security stresses that security is not an issue that should be considered on one level but on all, including for example the selection of materials and the designing of redundant electrical systems.

Especially concerning progressive collapse, it lays out as a minimum requirement that all new buildings need to be able to withstand loss of a ground level column. Although it dictates the use of alternative load path methods to examine the resistance in the perimeter, it allows designing of the interior of the building based on the key elements approach, stating that if columns are sufficiently protected or reinforced to avoid being critically damaged, then the interior column removal scenarios need not be considered. All primary structural and non-structural systems should be given priority in the progressive collapse analysis. Finally, if any structural changes are made to a building, including upgrading for seismic forces, then a progressive collapse analysis must be performed to examine potential vulnerabilities.

Nonetheless, it does not offer a comprehensive assessment framework or a methodology to upgrade the design of a building, apart from citing certain prescriptive rules of thumb, such as recognizing that components may act in other directions than those they were designed for and that ductile detailing should be used for connections.

2.2.3.1.2 Specific guidelines for progressive collapse analysis and design

Originally, the GSA drafted specific guidelines in 2000 for reinforced concrete structures, which were later (2003) supplemented by guidelines for steel and composite structures (GSA, 2003). Although the design scenario is described as event-independent, it is obvious that the document has been drafted with a possible attack in mind. This becomes evident as the reader comes across terms such "defended standoff distance/perimeter" and "qualified blast engineer" as the focus of the security of federal buildings. The governing philosophy can be summarised in four steps:

i. Identify if the building is important and/or big enough to be targeted
ii. Identify if the design of the frame is prone to progressive collapse
iii. Conduct the analysis to verify the building sensitivity to progressive collapse
iv. Apply structural modifications and restart from step ii.

The designer must identify whether the facility should be thoroughly designed against progressive collapse (largely based upon the alternate load path design method) by examining certain building attributes, the most important of which are summarised in Table 2-2.

Table 2-2: GSA criteria for detailed building progressive collapse design

Attributes	Details	Comments
Local	Connection resilience	The ability of the connection not to fail under the circumstances that caused the column to fail. It is measured by the connection's torsional and weak-axis flexural strength, its robustness and available ductility.
	Discrete beam-to-beam continuity	The ability of the connection to transfer gravity loads to the beam regardless of the state of the column.
	Connection rotational capacity	Crucial for the beam to satisfy the double-span condition.
Global (significant)	Examples include single point failure mechanism(s) and structural irregularities	

The text suggests conducting a linear finite element analysis - either elastic static (LS) or elastic dynamic (LD) - for low rise structures and a non-linear dynamic (ND) for those higher than 10 storeys. The combined loading for dynamic analysis is given in equation 2-3; for static analysis, a load factor of 2 should be adopted (equation 2-4) to account for dynamic effects.

Static: $DL + 0.25LL$ (2.3)

Dynamic: $2\ (DL + 0.25LL)$ (2.4)

Where: DL is the dead load and LL is the live load

Special attention is given to atypical structural configurations, in which case the notional removal tests of the following column locations should be carried out:

i. One interior to the perimeter column lines column, at the underground parking or any uncontrolled ground floor areas.

ii. Three ground floor columns, at the exterior (perimeter) of the structure.

2.2.3.2 ASCE 7-10 Standard

The ASCE 7-10 Standard (ASCE, 2010) aims to provide provisions for maintaining structural integrity after an unforeseen scenario. Indirect measures (commentary to Clause 1.4) focus on enhancing the ductility, continuity and redundancy of the structure. However, their application is open to interpretation (perhaps by referring to more detailed guidance) as there are no minimum load or strength criteria specified.

Direct measures involve designing certain elements as key elements, able to resist the load combination of Equation 2.5, or checking the ability of the structure after notional removal of an important structural element to provide the required resistance to withstand the gravity load combination of Equation 2.6 and to provide the required lateral stability resistance for the notional lateral force of Equation 2.7.

Alternate load path method: $(0.9 \text{ or } 1.2)\ D + 0.5L + 0.2(L_r \text{ or } S \text{ or } R)$ (2.5)

Key elements designs: $(0.9 \text{ or } 1.2)\ D + A_k + 0.5L + 0.2S$ (2.6)

Lateral stability: $N_i = 0.002\ \Sigma P_i$ (2.7)

Where: D, L, L_r, S and R are the dead, live, roof live, snow and rain loads respectively
A_k is the load effect arising from an abnormal event
ΣPi is the gravity force acting at level i, defined by Eq. 2.7 or Eq. 2.8

2.2.3.3 United Facilities Criteria of the Department of Defence

This document (DoD, 2009) provides detailed provisions for all types of construction (concrete, structural steel, masonry, timber and cold-formed steel) along with examples of application for each. Similar to the Approved Document A (ODPM, 2004a), depending on the size and importance of a structure, the three basic methods are employed: tying forces, alternate load path and specific local resistance design.

Horizontal ties need to be provided by the floor systems unless the comprising beams and connections are able to carry part or all of the forces while undergoing very large deformations. The minimum connection rotational capacity should be 0.2 rad. The required tying strength in the longitudinal or transverse direction (KN/m) are:

Edge (peripheral) ties per unit length $3\ w_f\ L_l$ (2.8)

Internal ties $6\ w_f\ L_l$ (2.9)

Where: w_f is the floor load per unit area (1.2D + 0.5L)
L_l is the greatest distance between any of two adjacent columns in the direction of the tie force

Similar to the ASCE (ASCE, 2010) requirements for checking the structure's ability to resist gravity (Equation 2.5 and 2.6) and lateral (Equation 2.7) loading after notional removal of a structural element, the UFC Criteria require performing a dynamic (nonlinear) or an appropriately increased static (linear or nonlinear) analysis. For each analysis type different failure criteria are specified for the structural components.

The dynamic increase factor (DIF) used to run the nonlinear static analysis for framed buildings is less conservative than the corresponding load increase factor specified in the GSA requirements (GSA, 2003). The connections' rotational capacity (specific limits are set for each connection type and beam size) defines the DIF, which is typically less than 2.

The use of a linear static procedure is only allowed when the structure meets certain structural configuration regularity and component capacity-demand ratio criteria. The increase factor for the applied gravity load, which depends on the properties of the connection and on the size of the beam, is similar to that in Equation 2.7.

The analysis should be carried out for each direction separately. The column scenarios considered are the same as those put forward by the GSA requirements (GSA, 2003) in Section 2.2.3.1 (one interior and three peripheral) for the following floors:

i. Underground parking (interior)
ii. Ground floor (interior and exterior)
iii. Mid-height of the building (interior and exterior)
iv. Below the roof (interior and exterior)
v. Above column splices (interior and exterior)

Finally, in the case where notional removal of an element is critical and may lead to progressive collapse of the structure, this element should be designed to be able to provide enhanced shear and flexural capacity. The Guidelines recommend that this approach should be carried out for all accessible perimeter columns in order to decrease the probability and intensity of abnormal loads or initial damage able to serve as a triggering event.

2.2.3.4 IBC 2012

The latest version of the International Building Code (IBC, 2012) requires all high-rise buildings of Risk Category III or IV (Clause 1604.5) to comply with structural integrity requirements. Clause 1613.3 presents the provisions for frame structures and Clause 1615.4 for bearing wall structures, which are based on effective longitudinal, transverse and perimeter tying.

For frames, this is expressed by specifying a minimum nominal tensile resistance for the beam-column and slab-wall or slab-girder connections. In addition, column splices are required to be able to provide the minimum design strength in tension needed to transfer the design dead and live load tributary to the column between the splice and the splice or base immediately below.

Generally, end connections of all beams and girders are required to be able to resist a minimum nominal axial tensile strength equal to the required vertical shear strength for allowable stress design (ASD) or two-thirds of the required shear strength for load and resistance factor design (LRFD) but not less than 45 kN, while the shear force and the axial tensile force need not be considered to act simultaneously (Clause 1615.3).

The new requirements, which were incorporated in the 2012 edition of the IBC, were largely dependent on and influenced by the outcomes of the National Institute of Technology (NIST) Measures of Building Resilience and Structural Robustness Project (NIST, 2011). Another document, which has had a major impact on codifying formal guidelines in the IBC, is the NIST report (NIST, 2007) on Best Practices for Reducing Progressive Collapse in Buildings, which documents a review of the best practices and requirements as identified in British, European and American codes as well as of relevant literature. It identifies indirect (tying capacity) and direct (alternative load path and key members) provisions that have been used up to 2007. In addition, it examines the case studies of collapse scenarios like the WTC, Ronan Point and the Murrah Building.

2.2.4 Critical appraisal of current provisions

Provisions fall into one of the main 4 categories: tying, key elements, alternate load path and risk based. Table 2-3 summarises their main characteristics, advantages and disadvantages. These approaches contrast with the conventional process of modern structural design, which is based on conducting an analysis on a representation of the structure using an idealised set of loads and then comparing the findings with certain limits. In addition, most of them still remain prescriptive and do not permit quantitative comparisons between alternatives or the identification of the margins against failure provided by those alternatives.

Table 2-3: Summary of current provisions' appraisal

Type	Description	Advantages	Disadvantages
Tying	Ability of beam to column connections to transmit an axial force from the beam into the column.	Simple, readily applicable and achievable.	Not possible to compare alternative designs. Do not account for ductility. Prescriptive. Cannot provide information on the reserve capacity of the damaged structure.
Key elements	Members are designed for higher and/or abnormal loads.	Use in combination with other methods can lead to cost-effective solutions.	
Alternate load path	Ability of the structural system to redistribute forces after notional removal of a member to resist normal loads.	Allows comparing alternative designs. Can provide quantitative results.	Application is often more complicated than prescriptive methods. Limited precision if coupled with non-sophisticated analysis.
Risk based	Measures focused on preventing or minimizing local damage.	Can minimise the intensity and/or probability of abnormal loading.	Do not enhance structural robustness. Liable to judgement.

Tying capacity is the most common amongst provisions employed by present regulations and building design offices in order to evaluate the resistance to progressive collapse. Although tying provisions are of a prescriptive nature, their main advantages are their simplicity to be appreciated and the simple calculations required for their application. Also, they do not require sophisticated design practices except for non-continuous columns, long spans and other factors, which lead to considerable tying forces requirements.

However, it is unclear whether there is a secure link between tying capacity and actual resistance (Byfield, 2006). Furthermore, recent studies (Nethercot et al., 2010, Izzuddin et al., 2007, Stylianidis, 2011, Vlassis et al., 2008a) using the Imperial College London assessment framework, indicate that although increased tying capacity has been effective is some cases, no general rule can be confirmed as per its ability to set the standard for designing robust structures. Instead, the lack of account for other occurring complex phenomena leads to overestimations of the capacity against sudden column removal.

Moreover, the exclusion of ductility considerations at all levels of the tying provisions can lead to unrealistically large ductility demands, which render them unsafe (Vlassis et al., 2008b, Ove Arup & Partners Ltd., 2003). In fact, Nethercot and Stylianidis (Nethercot and Stylianidis, 2011) have studied cases where an increase in the ability of the system to resist collapse is unaccompanied by an increase in tying force resistance, since large deformations and increased tensile capacity of the connections are needed for the necessary catenary action to develop. A recent study (Blundell et al., 2010) on a bare steel frame's resistance to progressive collapse also suggested little if any direct correlation. Finally, another study (Vidalis and Nethercot, 2012) demonstrated that the use of tying capacity provisions is in most cases not an efficient means of increasing resistance against progressive collapse.

On the other hand, designing certain members as **key elements** may be not readily achievable due to architectural constraints. In addition, the rapid evolution of material properties and new means of malicious acts may mean that their properties may not be pertinent in the future. However, this approach can still be very useful for reducing costs if applied in conjunction with other approaches.

Notional removal is based on a deterministic approach and – coupled with the right analysis – framework, has the ability to yield quantitative results and to permit the comparison of different designs. Its drawbacks, apart from being a more complicated and onerous process (especially if a detailed numerical model is required), mainly arise from the analysis method with which it is coupled. For example, in the case of static or dynamic linear analysis, system response cannot benefit from non-linear behaviour, such as catenary action and compressive arching action, and is in danger of not taking dynamic effects such as the dynamic stress-stress behaviour of the components into account. Characteristically, the GSA Guidelines (GSA, 2003) state that:

"The use of a Linear Procedure, as provided for in these Guidelines, is not intended for and not capable of predicting the detailed response or damage state that a building may experience when subjected to the instantaneous removal of a primary vertical element".

As a result, the GSA Guidelines' approach aims at identifying if the building has a "high" or "low" potential for progressive collapse, before conducting more detailed analysis. While similar approaches have the advantage of being relatively simple to use, more sophisticated levels of structural analysis can provide more realistic representation of performance (Sadek et al., 2011, Kwasniewski, 2010, Izzuddin et al., 2008, Marjanishvili and Agnew, 2006, Kim et al., 2009).

Finally, **risk based** approaches can provide decision support on the strategic, normative and operational levels but their main use is within a consulting role towards assessing or identifying priorities for upgrading a structure to resist progressive collapse, rather than assisting towards design recommendations. They can be used to establish non-structural protective measures with the aim of minimising the occurrence of abnormal events or reducing the intensity or effect of abnormal loads on the structure. Nevertheless, they cannot be used to enhance robustness outside of the scope of scenarios for which they have been drafted for.

2.2.5 Concluding remarks

During the past years, there has been significant research activity on reviewing the formal provisions in Europe and the U.S., which is expected to continue or even rise in the next years. While practicing engineers still need to react to certain inconsistencies of the present guidelines, they can now refer to a number of recently published references which can provide a review of present provisions as well as additional guidance on how to apply them (NIST, 2007, IStructE, 2010, Cormie, 2011).

As present advances continue to close the gap between a deterministic and realistically applicable assessment framework, attention is expected to shift towards how will a designer, once he or she has completed the assessment process, decide on the design changes necessary to make the structure able to withstand an unforeseen scenario. The lack of such knowledge currently prohibits the designers from addressing the needs of the structure using a streamlined process instead of a trial and error approach, limiting their ability to choose how to enhance the robustness of their design using an efficient method.

2.3 Analysis frameworks for progressive collapse

2.3.1 Introduction

As mentioned in Section 2.2.4, the alternate load path method, properly applied, can allow taking all the essential features of progressive collapse into account and can provide a quantitative evaluation of the structure’s resistance. The accuracy, ease and speed of application, pertinence and applicability for design (ability to evaluate the reserve capacity or to compare alternative designs) of an analysis framework depends on the choice of its constituent methods, which can be divided in three main categories:

i. The design scenario, which can be threat depended or independent (see Section 2.3.2).
ii. The modelling approach, which can be numerical or analytical.
iii. The type of analysis, which can be static or dynamic, linear or nonlinear (2.3.3).

Depending on the type of the application and the resources available, a different combination may be appropriate, which is discussed in the next sections.

2.3.2 Choice of the design scenario

2.3.2.1 Threat dependent

Threat dependent scenarios take into account the special loading or other conditions generated from the triggering event, which have a direct or indirect impact on the behaviour of structural components. The most commonly documented threats are blast, impact and fire.

In some threat-dependent scenarios, structural members also need to be able to dampen the motions caused by abnormal loading, which may reduce their ability to carry loads in their damaged state (Szyniszewksi and Krauthammer, 2012).

2.3.2.1.1 Blast

Marchand and Alfawakhiri (Marchand and Alfawakhiri, 2004) have published an extensive review on the relationship between explosive loads and progressive collapse. Their work summarises the general principles governing this type of loads (blast loads) and the methods to predict response and discusses the recommendations for designing a structure to resist effects from a blast and subsequently mitigate the risk for progressive collapse in the context of the GSA and DoD requirements, which are currently in force in the U.S. (GSA and DoD Guidelines). Byfield examined the behaviour of simply designed multi-storey buildings (Byfield, 2006) and proposed a series of general design recommendations, including strengthening beam to column connections located near potential vehicle access points. Karns et al. (Karns et al., 2007) examined the resistance of moment frame connections to blast attack, taking into account effects such as high strain rates in critical components. The main aim of the study was to evaluate the post-blast integrity of these connections and examine whether it allowed arresting the necessary structural mechanisms to resist progressive collapse, as set forth by the GSA Guidelines. Finally, a recent comprehensive review by Cormie (Cormie D et al., 2009) summarised the main research advancements and conclusions concerning blast effects on buildings.

2.3.2.1.2 Fire

Fire can influence material properties and reduce the resistance of members. Fang et al. (Fang et al., 2011) compared the use of two alternative approaches, one temperature-dependent and one temperature-independent to predict the fire response of structures. The temperature dependent approach, albeit more complicated, can be more effective for limiting the progression of local damage under unforeseen events and is more readily applicable in design practice. Also, results show that fire affected members need to be able to provide residual resistance at elevated temperatures in order for the system to be able to prevent collapse. However, since the effects of fire vary depending on a great number of parameters, including the column size, loading level, floor configuration, location of fire and the number of ambient floors above the affected floor, it is often hard to predict the maximum temperature leading to overall collapse.

Burgess and Davison (Burgess and Davison, 2012) discuss the influence of thermal expansion and strength degradation in connections within simply designed frames during progressive collapse in fire. These properties influence rotational capacity and induce beam-end movements generating high normal forces. Heat expansion causes additional compressive loading at the connections if axially restrained, which makes them more likely to fail in local inelastic buckling at the compressive flange instead of tensile rupture of the other flange. The study suggests that a combined component-based connection element and temperature-dependent analytical models solution process will aid performance-based structural fire engineering modelling for progressive collapse in the future. Subsequent experimental tests (Huang et al., 2013) examined the performance of reverse-channel connections and concluded that they can provide satisfactory levels of strength and ductility.

Haremza et al (Haremza et al., 2012) performed experiments with steel-concrete composite connections under combined bending and axial loads and high temperatures. Their loading conditions corresponded to the loss of a column and the presence of a localised fire. Their results determined that beam axial restraints can increase the capacity of the joint as the compression of the concrete reduces the load at the bolt rows and the endplate, which is affected by high temperatures. On another note, connection behaviour in fire, including response to tension, compression and deformation reversal, can be described using a component-based model (Block et al., 2013).

2.3.2.1.3 Impact

In this scenario, impact forces can be generated by either debris impact from the damaged part of the structure or by debris, vehicle or aircraft impact originating from non-native causes. In the first case, satisfying the admissible floor area of collapse criteria of most codes requires that all lower floors can provide the required resistance to withstand impact from the collapsed portion of the upper floor(s), unless the damaged floor corresponds to the first level of the building. Nevertheless, recent studies (Vlassis et al., 2009) indicate that the ability of the lower floors to sustain the impact forces is questionable.

Lynn and Isobe (Lynn and Isobe, 2007) constructed a beam-element based finite element model to study the behaviour of a framed structure subject to extreme loads originating from the impact of a small aircraft. The study proposes a technique, based on adaptively shifted integration (ASI) with Gaussian points, for use within finite-element codes. Their results showed information on propagation phenomena of impact loads and shock waves and indicate that the mass of the aircraft has a stronger influence on impact damage than its velocity.

More recently, research at the University of Liege suggested a procedure for the appraisal of the structural robustness of plane frames under impact loading (Comeliau et al., 2012).

2.3.2.2 Threat independent (sudden column loss)

The advantages of an event-independent scenario should include simplicity of investigation, aptitude for comparative evaluation of different structural designs and compatibility with the simplified dynamic assessment. Since an event-independent scenario cannot be fully accurate compared to individual event based scenarios (2.3.2.1), it should therefore be based on measurements that are directly linked with the progressive collapse limit state, such as vertical displacements of upper floors and ductility demands of connections.

The "sudden column loss" idealization is a suitable design scenario, which displays all the above characteristics (Gudmundsson and Izzuddin, 2010). A study on the investigation of progressive collapse of multi-storey frames (Izzuddin et al., 2008) has also presented a correlation factor for linking sudden column loss with different levels of blast loading, which implies that a similar approach can be carried out with other extreme scenarios. Hence, this could permit the structural engineer to isolate the causing effect from the structural design of the building and examine each one's contribution to progressive collapse separately.

The vast majority of studies employing the threat independent design scenario of member notional removal consider sudden loss of one column, although some exceptions exist; for example, the study of the response of a structure for a dual (Fu, 2010) or a multiple column loss (Pereira M. and Izzuddin B.A., 2011) scenario.

2.3.3 Structural analysis methods

2.3.3.1 Linear

Simple and easy to perform, linear static analysis for progressive collapse is conducted with the use of a combination of dead and live service loads amplified by a dynamic increase factor (DIF) of 2. The response is evaluated based on demand to capacity ratios, which should not exceed a prescribed value (usually 3). It is limited to relatively simple structures with insignificant or easily predictable nonlinear and dynamic effects.

Santafé et al (Santafé et al., 2011) determined that elastic analysis for progressive collapse is very conservative compared to elastic-plastic. Their results showed that inertial effects can have an important influence: higher removal times (lower accelerations) can lead to no collapse after the column loss, thus sudden column loss offers an upper bound on the deformations obtained.

2.3.3.2 Nonlinear static

Structural performance in progressive collapse can be evaluated more accurately by taking into account nonlinear effects. Although it does not consider dynamic effects and may become time-consuming due to convergence issues, this procedure is useful in determining elastic and failure limits of the structure.

Izzuddin et al (Izzuddin et al., 2008) proposed a novel method, which uses the nonlinear static response combined with the maximum dynamic response of the structure to account for dynamic effects without the need for dynamic analysis, in order to evaluate the pseudostatic capacity of a structure, which can be used as a measure for progressive collapse resistance.

Lee et al (Lee et al., 2009) studied two simplified nonlinear analysis methods for double span beam systems using welded connections. Based on a tri-linear model for predicting vertical resistance against chord rotations at the conections, their study proposes an energy-based nonlinear static progressive collapse analysis approach, with the aim of being able to quickly assess whether a structure's response is within the collapse spectrum or not based on ductility criteria.

Khandelwal and El-Tawil (Khandelwal and El-Tawil, 2011) used a pushdown analysis technique, similar to the pushover method used in earthquake engineering, to calculate the residual capacity of two frames designed for different seismic regions to resist progressive collapse under a missing column scenario. The study employs the three variants of "pushdown analysis": Uniform, bay and incremental dynamic. The first two consider an incremental uniform gravity load; the last considers the maximum dynamic load for the undamaged bay. Results showed that the dynamic increase factor of 2 proposed by the GSA (GSA, 2003) is rather conservative, as suggested by previous studies (Ruth et al., 2006). For this study in particular, it is located within the range of 1.06 to 1.45, which suggests that dynamic effects vary depending on the type of structural system.

Along the same lines, Meng-Hao Tsai (Tsai, 2012) proposes an analytical expression to replace the empirical formula used for calculating the load increase factor (LIF) and dynamic increase factor (DIF) in UFC 4-023-03 for progressive collapse analysis. The analytical formula takes into account the post-yield stiffness of plastic hinges which gives more accurate and less conservative ductility requirements for weaker joints.

2.3.3.3 Nonlinear dynamic

Progressive collapse is an essentially dynamic event; instantaneous loss of a column releases significant internal energy that disturbs the initial load equilibrium of external loads and internal forces, which need to be absorbed by the ductile members of the remaining structure, mainly the connections, in order for the structure to reach a new equilibrium position, otherwise it collapses. For this reason, nonlinear dynamic analysis procedures, albeit being the most complex and resource demanding, are the most accurate since they inherently incorporate dynamic amplification factors, inertia, and damping forces.

Marjanishvili and Agnew compared four analysis approaches (Marjanishvili and Agnew, 2006) static linear, static nonlinear, dynamic linear, and dynamic nonlinear by analyzing a nine-story steel moment-resistant frame building with a loss of one primary column. Their recommendations include using the nonlinear static procedure to supplement the nonlinear dynamic in determining the first yield and ultimate capacity limits, as well as in verifying and validating dynamic analysis results.

Certain guidelines (GSA, 2003, DoD, 2009) still do not seem to encourage using nonlinear dynamic analysis on the grounds of its perceived complexity, apart from special scenarios.

2.4 Imperial College London Method

Using the concepts of the alternative load path and energy conservation, as well as the sudden column loss design scenario, the Imperial College London Method (ICLM) provides a quantitative assessment of the structure's ability to reach a new equilibrium position. Its main features are that it does not require heavy non-linear dynamic analysis although it accounts for dynamic effects, that it recognises all the important complex physical phenomena, that it employs a realistic criterion of failure, that it can implemented at various structural idealisation levels and that it has been recently simplified in order to streamline the process.

Recent work at Imperial has considered the behaviour of bare steel and composite simple, semi-continuous and continuous construction (Nethercot et al., 2011, Vidalis and Nethercot, 2012, Nethercot and Vidalis, 2013, Stylianidis, 2011). Other studies have examined the combined effects of possible progressive collapse following a fire (Fang et al., 2011) and the contribution of the floor slab (Zolghadr Jahromi et al., 2012 -a) over the resistance obtained using the basic method presented herein. Notwithstanding, integrating additional features increases complexity, which means that the necessary analysis cannot utilise the simplified approach of Section 2.4.2.

2.4.1 Assessment framework for multi-storey buildings

2.4.1.1 Introduction

The ICLM is based on the simplified framework for assessing the structural performance of multi-storey framed buildings developed by Izzuddin et al. (Izzuddin et al., 2008, Izzuddin et al., 2007). Its three main stages are: determination of the nonlinear static response (by a detailed finite element or a simplified analytical model), dynamic assessment using a novel simplified approach and ductility assessment.

2.4.1.2 Simplified dynamic assessment

In order to account for the effects following sudden column removal, the response of the structural system is calculated for λ*Po and the dynamic effects can be reasonably accurately evaluated using a simplified energy-equivalence approach coupled with the nonlinear static response. The base of the approach is that the effect of sudden column loss is very similar to instantaneous application of a gravity load to the damaged structure. If a single deformation mode dominates the response, then the maximum dynamic response is achieved when the work done by the gravity load is equal to the energy absorbed, resetting the kinetic energy balance back to zero.

The approach, illustrated in Figure 2-1, has been validated based on its excellent agreement with detailed dynamic finite element analyses' results (Vlassis, 2007). The maximum dynamic displacements ($w_{d,1}$ and $w_{d,2}$) for two levels of suddenly applied gravity load ($\lambda_1 P_o$ and $\lambda_2 P_o$) are calculated by equating the hatched areas of the two nonlinear static responses (Figure 2-1a and Figure 2-1b). The maximum nonlinear dynamic, also referred to as "pseudostatic" (Izzuddin, 2004), response can be determined by plotting the suddenly applied gravity load against the maximum dynamic displacement (Figure 2-1c). This allows obtaining the maximum dynamic displacement from the actual gravity load (Figure 2-1c).

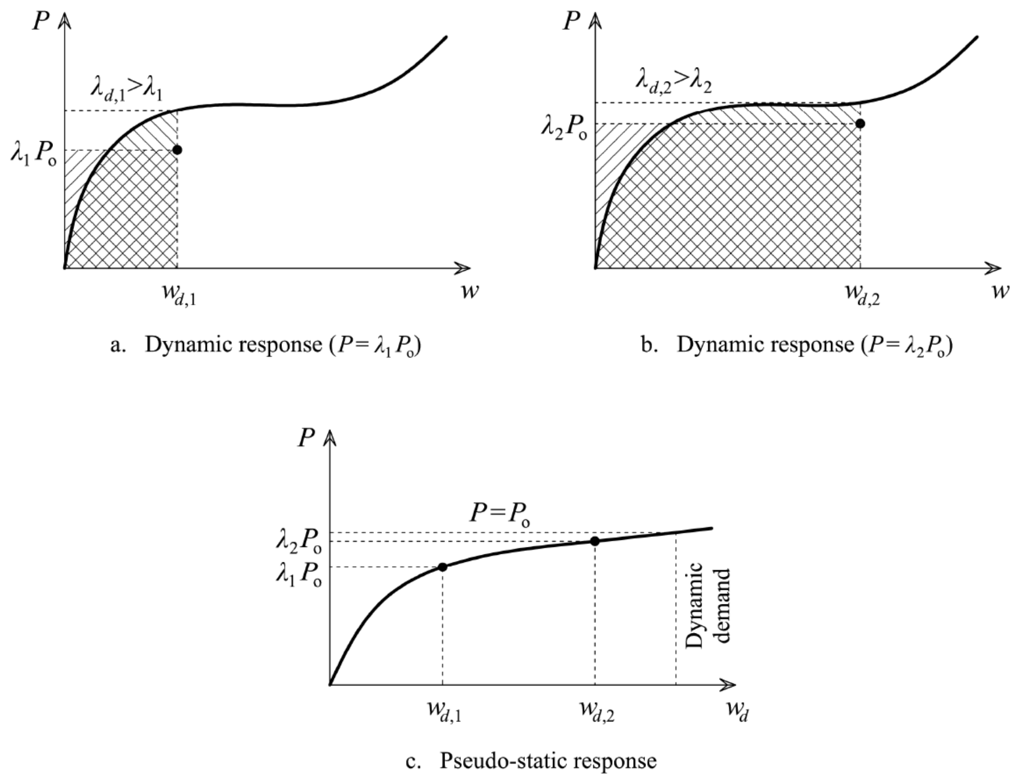


Figure 2-1: Simplified dynamic assessment (Izzuddin et al., 2008)

2.4.1.3 Measure of robustness

Robustness is measured by comparing the supply and demand of pseudostatic capacity, an indicator which takes into account the ductility, strength and energy absorption capacity of the system. The failure criteria in the framework are associated with the ductility assessment of the structure, i.e. whether it can provide the required pseudostatic capacity before component deformations exceed the allowable limits.

2.4.1.4 Multi-level assessment (structural idealisation)

The ICLM allows a significant reduction in the complexity of the multi-storey frame model since in most cases it is reasonable to assume that the deformation of structural components will be concentrated in the bay of the lost column (Figure 2-1a). Moreover, provided that the remaining column has an adequate capacity to carry the redistributed loading, the model can further be reduced to the floors above the removed column (Figure 2-1b) Since each floor works to redistribute the load applied at that level and all the storeys are subject to the same loading and design, only one floor needs to be considered in order to assess the capacity of the structure in resisting progressive collapse (Figure 2-1c). If slab membrane effects are ignored, then the response is only influenced by the individual beam models (Figure 2-1d). This allows for the response at higher levels to be deducted from the responses at lower levels using any type of analysis, be it detailed finite element or simplified analytical models.

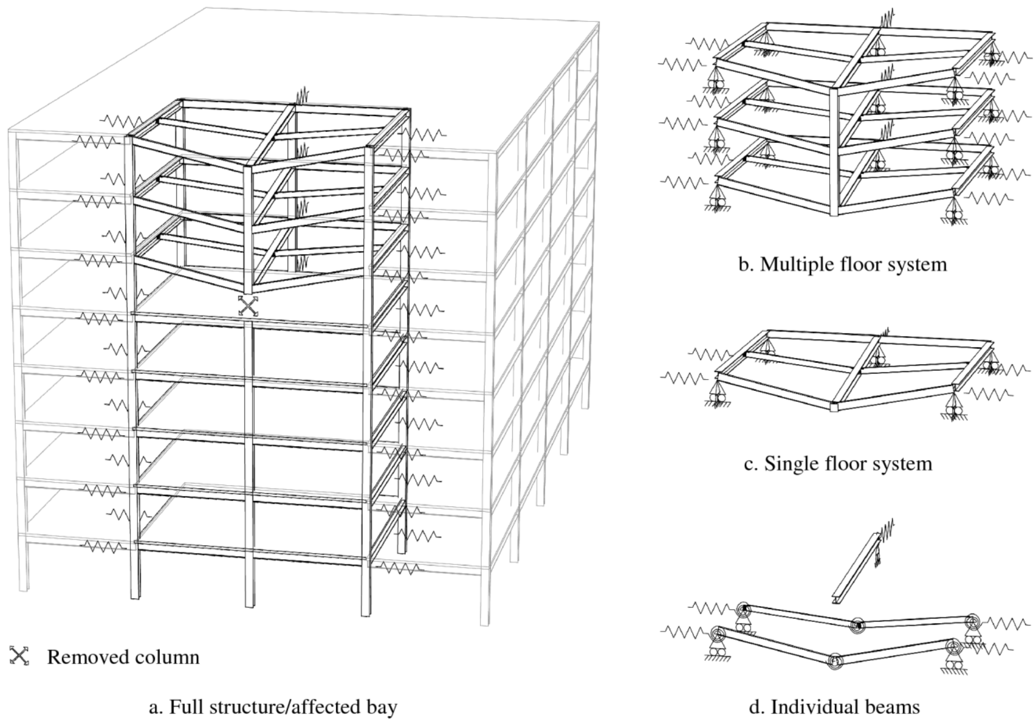


Figure 2-2: Simplified multi-level approach, source (Stylianidis, 2011)

2.4.2 Simplified method

Although the method originally relied on numerical analysis, in order to streamline the process, a simplified hand-calculation method for the prediction of the beam nonlinear static response following column loss was developed (Stylianidis, 2011, Stylianidis et al., 2009).

It provides a set of explicit equations that link the connection bending moments and deformations, the beam axial load and axial deformation as well as the beam deflection with the beam loading. By employing the appropriate deformation failure criteria for each connection component, the ultimate ductility and pseudostatic capacity of the system can be predicted.

It can be applied to either bare steel or composite frames and allows for representation of the basic features of beam behaviour such as material and geometric nonlinearity and connection bending moment-axial load interaction (Stylianidis and Nethercot, 2009). Its accuracy has been successfully verified with the use of the ADAPTIC finite element analysis software (Izzuddin, 1991). Hence, it is perfectly suitable for conducting rapid parametric studies which can be used to understand the mechanics of the problem.

2.4.2.1 Connection modelling

Previous work (Stylianidis and Nethercot, 2009, Stylianidis and Nethercot, 2010) has extended the component method of EC3 and EC4 to incorporate the connection bending moment-axial load interaction. Figure 2-3 shows the connection mechanical spring model used. The connection rotation capacities define the beam capacity and are associated with the deformations of the connection compressive and tensile components (θ'_2 and θ'_1 respectively). Figure 2-4 illustrates the support and mid-span connection in a double span semi-continuous beam system.

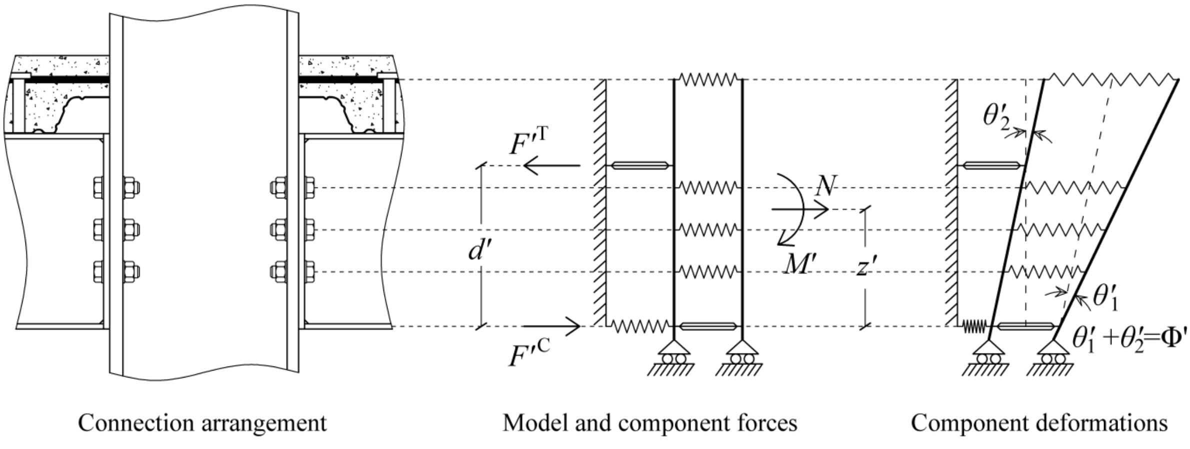


Figure 2-3: Connection mechanical spring model (Stylianidis, 2011)

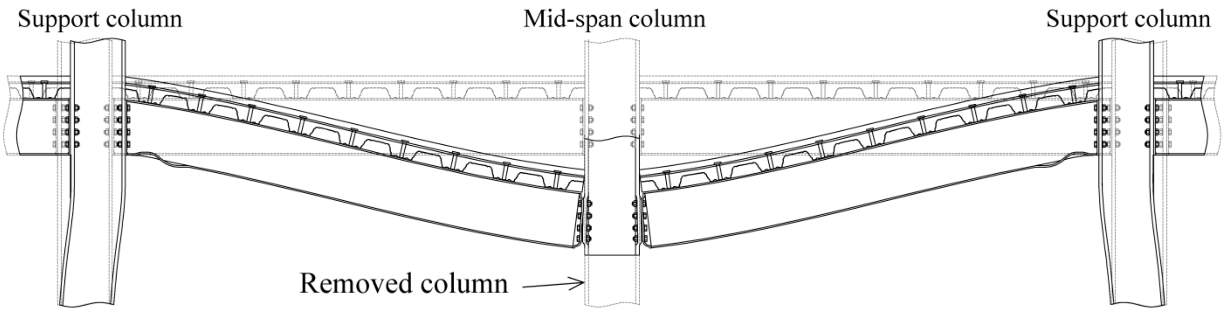


Figure 2-4: Axially restrained double-span beam system with removal of the midspan column

2.4.2.2 Application of the simplified method

Stylianidis and Nethercot (Stylianidis, 2011, Nethercot et al., 2011) studied the mechanics of axially restrained and unrestrained bare steel and composite beam and floor grillage arrangements with partially restrained connections. Among the parameters examined were he beam length and depth, the degree of axial restraint and variations in the tensile, compressive and shear components' capacity. Results have revealed a significant interplay amongst the main influencing parameters: strength (especially the balance between the tensile and compressive capacities), stiffness and ductility. The main conclusions of the studies were:

i. Decreasing the beam span and/or increasing the connection strength (especially the tensile capacity), stiffness and ductility will enhance the capacity of axially unrestrained beams.

ii. Decreasing the beam span for the axially restrained beams will make the following considerations for enhancing performance more effective:

    a. Shallow beams: use of more rigid support connection tensile components, or simultaneously increasing the connection tying capacity and ductility (the latter may also be achieved by decreasing the connection compressive capacity).

    b. Deep beams: the stiffness of the compressive components and the connection compressive capacity should be increased in order to enhance performance.

iii. The connection and beam parameters that control beam responses have an analogous influence on grillage performance.

iv. Floor performance is dominated by the beams that exhibit the higher response, which generally are the short-span and/or axially restrained beams, while it is limited by the failure of the least ductile beam, which are often the shorter ones.

2.4.3 Summary

One of the main advantages of the Imperial College method is that it calculates and takes into account the nonlinear static and pseudostatic response using the equivalent absorbed energy concept. Thus, the maximum dynamic response can be estimated with reasonable accuracy from the nonlinear static response under amplified gravity loading, which eliminates the need for detailed nonlinear dynamic analysis. In addition, the recently developed simplified hand-calculation version of the method facilitates extensive parametric studies and hence allows the structural engineer to calculate and compare the merits of alternative designs.

2.5 Building behaviour in progressive collapse

2.5.1 Introduction

As noted previously, research on the progressive collapse of structures has almost exponentially intensified following high profile incidents like the WTC collapse in 2001, the evolution of the numerical capabilities of computers and the availability of the advanced analysis and assessment frameworks and methods mentioned in the previous section. The new studies and reviews, which are constantly being reported in Structural Engineering journals and Structural conferences, generally fall into one of the following categories:

i. Numerical and analytical studies, which employ the principles of mechanics to analyse or represent the physical process of progressive collapse.

ii. Risk or probability studies, which focus on devising philosophies and frameworks to address progressive collapse scenarios and their impact.

iii. Targeted studies, which apply analytical, numerical or experimental methods in order to explain specific features of progressive collapse.

2.5.1.1 Numerical / analytical

This research is primordial for investigating particular problems and improving our understanding of the problem. It is divided between two types of studies:

i. **Model based**, which are in most cases based on the Alternate Load Path concept, employing microscopic or macroscopic models.
ii. **Experimental,** which examine the behaviour of structural components, subsystems and systems by monitoring their behaviour under conditions which arise in a progressive collapse scenario.

On the one hand, conducting an extensive real scale study of the behaviour of an entire building in progressive collapse is not realistically achievable due to the associated cost and monitoring challenges. Thus, a small number of tests – involving only parts of the structure – have been conducted so far.

On the other hand, as the capacity of modern computers and the sophistication of numerical and analytical analysis has exponentially increased in the 21st century, it is now possible to examine potentially critical scenarios and obtain reliable results for certain illustrative problems. This is particularly useful in the field of forensic engineering: in the analysis of the WTC collapse, for example, these studies have been essential in forming an understanding of the main collapse mechanism, despite the chaos during the incident.

However, the usefulness of these studies is subject to certain conditions. For example, their accuracy is significantly limited – to the point of being misleading and potentially dangerous – if a simulation omits certain key effects, such as the deformation capacity of the connections. Also, their results are most often solely applicable for the circumstances related to the particular case study. For example, instead of reporting on the influence of changing certain parameters on the main response resistance mechanisms, focus is usually narrowed in explaining a particular event. Hence, the understanding of the problem arising from such an analysis is not necessarily relevant for another structure and offers a limited contribution to a better general understanding.

Nevertheless, such "targeted" research seeks to answer specific questions and results and when combined with those from parallel and complementary studies, can lead to the development of advanced tools for assessing the resistance of a structure or for prioritising between alternative configurations.

2.5.1.2 Risk or probability based

As the cause or "trigger event" of progressive collapse is, by definition, extreme and unexpected, designing for a scenario characterised by "high consequences but low probability" requires not only careful treatment but also an approach different from that adopted for conventional structural design. It can perhaps be compared to creating seismic hazard zones, which analyse the probability of the "maximum considered earthquake" for a specific area, expected to occur with a maximum of 2% probability every 50 years.

Risk or probability based research can follow pragmatic and judgemental rules and procedures, which are inherently subjective, or entirely probabilistic frameworks, which rely on objective decision making methods but require input of a quality and extent that is never likely to be available. Nevertheless, work on the matter can help the designer decide on what scenarios to consider, on the required degree of sophistication for addressing them and on the target reliability of a building's resistance against progressive collapse.

Work on the subject has been carried out by the members of the COST Action TU601 research network (COST Action TU0601 homepage, 2011). The project's main suggestions, methods and conclusions, which arise from the research of its members, have been summarised in the final report of the Action (COST Action TU0601, 2011a). A recent application of a probabilistic methodology for multi-storey buildings (Izzuddin et al., 2012) reported on the conditional probability of failure for specific local damage scenarios (single column loss). The study concluded that the framework is most useful when coupled with deterministic methods.

2.5.2 Steel and composite steel framed structures

2.5.2.1 Introduction

Among the favourable characteristics of steel construction for resisting progressive collapse is the provision of various alternative load paths after loss of a key member, the good ductility, over-strength and strain rate sensitivity properties (Kuhlmann et al., 2012), as well as satisfactory energy absorption capacity levels, especially in the case of composite construction.

Almost all modern studies consider sudden column loss (Section 2.3.2.2) as a standard approach for evaluating the structure's resistance in progressive collapse. Although the majority of studies examine the behaviour of buildings, a few exceptions focus on isolated structural systems, like the response of cable-stayed steel roofs following sudden cable loss (Gerasimidis and Baniotopoulos, 2011). In the case of framed structures, as mentioned in

Section 2.4.2.1, connection modelling aiming to accurately simulate response should be able to take into account the interaction between axial force and bending moment under large deformations (Stylianidis and Nethercot, 2009, Del Savio et al., 2009).

In the case of non-continuous construction, simple (pinned) or partially restrained connections are employed. The former, which include connections with flexible endplates or fin plates, are mainly used to resist shear forces (gravity loading), while the latter, which include full-depth endplate bolted connections, can also resist variable levels of bending moment. Continuous construction (plastic design) on the contrary, requires the use of fully restrained and often full-strength connections, like welded or full depth endplate.

2.5.2.2 Model behaviour vs. actual behaviour

Simoes da Silva (Simões da Silva et al., 2002) et al have compared the theoretical and experimental post-limit stiffness and ductility of the components for end-plate connections. Their study suggests that 1% strain hardening might be a conservative value for certain components like the column web panel in shear and column web in compression but accurate for most components in bending or tension.

Another comparison between experimental results and a nonlinear dynamic simulation (Kwasniewski, 2010) has shown that FE models may not always accurately represent connection response in progressive collapse. One of the main parameters of uncertainty is the failure strain, especially for bolts. FE models do not always explicitly take this account, as material failure leads to the component disintegration by deleting (eroding) a finite element from further calculations, while in actual connections failure is usually initiated by the rupture of fillet welds or by indirect bolt failure, such as shear stripping of the threads. Another example is the underestimation of the initial stiffness of the connection, which is usually attributed to the bending of the end plate due to imperfections and the realisation of contact between the flush plate and the column flanges, which is not included in the model.

2.5.2.3 Simple construction

Researchers at the University of Washington (Weigand and Berman, 2009) have tested 34 different single plate shear and bolted web angle connections via experimental tests. The results were used for the development of detailed and simplified analytical models for use in earthquake engineering.

An assembly developed in the University of Alberta (Oosterhof and Driver, 2012) simplified the testing of double span simple beam system by eliminating the need for construction of a two-bay frame. A series of 45 full-scale experimental tests with common steel shear connections has been conducted under static proportional combinations of moment, shear and tensile loads. The results show that in the majority of cases catenary tension eventually dominates the axial stresses in the connection, effectively decreasing the moment to zero before the connection fails due to bolt tear-out. However, it is still possible for the response to reach a peak vertical load after rupture thanks to the remaining of catenary forces. Additional research (Oosterhof, 2013) supports that the design of shear tab connections appears to have superior properties compared to the other simple connections, such as those employing a single angle or a fin plate, because they maintain some of their stiffness under large deformations. Another experimental study (Schwindl and Mensinger, 2012) on secondary girder connections with long fin plates concluded that long fin plates have slightly enhanced properties compared to standard fin plate connections.

Using the previously mentioned experimental data, Daneshvar and Driver (Daneshvar and Driver, 2012) have developed a numerical model for bolted-bolted WT connections (structural “T” section cut from a wide flange cross section), commonly used in gravity frames. The variable examined was connection depth for three, four and five bolt row connections. The failure mode observed was bolt shear failure at the bottom hole while the largest contributor to ductility was the local deformation in the web adjacent to the bottom rows.

Currently, a study (Oosterhof and Nethercot, 2014) using the results from the physical tests conducted in the University of Alberta together with the Imperial College London simplified analysis framework will allow examining the performance of a bare steel gravity frame in various column removal scenarios and will identify general vulnerabilities which affect the robustness of such structures.

2.5.2.4 Partially restrained construction

Semi-continuous construction, if efficiently used, can result in smaller sections reducing the total cost of construction. The designer chooses the maximum resistance of the connection detail in order to minimise the bending moment resistance requirements in the beams. The behaviour in progressive collapse of these systems is amongst the most complicated because of the complex resistance mechanisms that participate.

Early studies include an extensive project encompassing experimental tests and simulation with numerical and analytical models completed in Europe in 2007 on the role of joint ductility in structural robustness (Kuhlmann et al., 2009). The main experimental tests were carried out at Liege University (Demonceau and Jaspart, 2010), Stuttgart University and the University of Trento (Baldassino and Zandonini, 2009). The results were analysed in order to extract a series of requirements for enhancing joint ductility, such as increases in the bolt resistance and gauge, as well in the spacing between the beam flanges and the adjacent bolt-rows. The outcome of the numerical studies was the development of a simplified method for representing the beam tensile catenary behaviour after column loss in terms of required ductility for different levels of beam gravity loading. However, it only accounts for the tensile catenary phase and is thus applicable only if large rotations can be attained, while other phases such as compressive arching and elastic are disregarded. Nevertheless, it can be used to describe the connection bending moment-axial load post-limit behaviour during that phase while also accounting for the influence of axial restraint in the beams.

Researchers at Imperial College London (Vlassis et al., 2008a) examined the behaviour of fin-plate connections for both bare steel and composite arrangements. For the bare steel arrangement, results agreed with the aforementioned observation that the presence of axial restraint can enhance connection performance during the catenary action phase. On the contrary, for composite fin plate connections, the reduced ductility supply and the increased effective cross-section depth resulted in a poor demonstration of tensile catenary action.

The opposite is true for full depth endplate connections. Also, composite construction can be advantageous compared to bare steel arrangements (Stylianidis, 2011, Nethercot et al., 2011, Vidalis and Nethercot, 2012), provided that the bare steel components are strong enough to ensure a constructive balance of component capacity within the connections in both hogging and sagging bending moment loading.

Examination (Vlassis et al., 2008a) of the rotational capacity of partial-depth flexible end-plate connections for composite beams demonstrated that the maximum rotational capacity provided (around 70 mrad) is insufficient for allowing the beam system to enter the tensile catenary action phase. A comprehensive parametric study at Imperial College London (Stylianidis, 2011, Nethercot et al., 2011), presented in Section 2.4.2.2, has shown that the performance and resistance actions of a partially restrained beam system mainly depend on the presence of axial restraint, the beam length to depth ratio and the balance between the capacity of the connection tensile (bolts, endplate, reinforcement bar, column flange), compressive (beam flange, column web) and shear components (column web), while other parameters like bolt row positioning and the beam axial and bending stiffness do not play an influential role in this type of construction (Blundell, 2010).

The conclusion of the ICL research that the combined effect of strength, ductility an energy absorption capacity should be taken into account has been corrugated by other case studies. Moreover, the robustness assessment of steel building frames with partially restrained connections fabricated from bolted T-stubs after column loss (Xu and Ellingwood, 2011a) concluded that the performance of full-strength (FR) connections in a progressive collapse scenario was superior to partial-strength (PR) connections. The connections were represented in a non-linear finite element model with a macro-model validated against experimental data. The outcome of another study (Dubina and Dinu, 2012) pointed towards the important role that the connections have towards assuring the redistribution of forces after the loss of a column. In order for them to do so, they need to be ductile, allowing both the attainment of the beam plastic moment and – after the plastic hinges are formed – of the beam axial capacity.

2.5.2.5 Continuous steel framed structures

Continuous construction requires the use of ductile moment resisting connections. Analytical modelling of their behaviour in conditions that simulate progressive collapse is a complicated task and most studies rely on numerical analysis or laboratory tests with simplified assemblies.

Researchers at NIST (NIST, 2011) have made a considerable effort to understand the behaviour of buildings with moment resisting assemblies. A series of tests (Sadek et al., 2010, Sadek et al., 2011) on beam-column assemblies with welded unreinforced (WF) and reduced beam section (RBS) connections under monotonically increasing vertical displacements of the unsupported centre stub column were conducted. Also, the arrangements' behaviour was simulated with both detailed and reduced FE models. The results suggest that the ultimate loads are resisted through catenary action until connection capacity under combined bending and tension is exhausted.

However, a previous study (Khandelwal and El-Tawil, 2007) on similar arrangements with a calibrated micro-model for estimating the connection ultimate rotational capacity suggested that catenary action does not play a critical role in resisting progressive collapse for moment connections because of the small deformations occurring. A later study (Khandelwal and El-Tawil, 2011), employing a "pushdown" analysis technique to calculate the residual capacity of two frames designed for different seismic regions, again claimed that for continuous construction, tying capacity is less relevant due to the de facto strong and very stiff connections.

Another study (Lee et al., 2009), which did not consider failure at the connections but rather that they are capable of providing all the necessary strength and ductility required, identified the beam span to depth ratio as the governing factor of resistance. The researchers examined the progressive collapse resistance of multi-storey steel moment frames with fully-restrained ductile moment-resisting frame connections using two methods: a non-linear static push-over and a pseudo-dynamic analysis, in which the energy equivalence approach is used to determine the "collapse spectrum" or the maximum chord rotation of a double span beam system.

The behaviour of special concentrically braced frames (SCBF) and eccentrically braced frames (EBF) in progressive collapse was examined with the use of two-dimensional FE macro models together with the alternative load path approach after sudden column and/or brace loss (Khandelwal et al., 2009). For corner column removal of the SCBF, the adjacent bay completely collapses, while the system can withstand the removal of an edge column and its attached brace member. The main contribution to resistance comes from the massive corner column rather than the bracing system. The EBF resists an edge column removal much better that the SCBF, though a corner removal scenario is not examined. The study highlights that the bracing system does not substantially contribute to the robustness of the system.

In addition, a number of studies (Khandelwal and El-Tawil, 2011, Szyniszewksi and Krauthammer, 2012, Khandelwal et al., 2008, Park and Kim, 2010, Kim et al., 2009, Kim and Kim, 2009) studied the response of moment frames designed to resist an earthquake. The majority of these studies demonstrate that the continuous beam systems are less likely to be critical compared to the non-continuous ones used in the same frame.

2.5.3 Role of the composite floor

In the case of composite construction, the combined contribution of the metal decking and the concrete slab can provide additional capacity towards the progressive collapse resistance of floors, either via membrane or catenary action. Numerical simulations with full floor models including the 2D reinforced concrete slab (Yu H. et al., 2010) claim resistance can be enhanced by over two fold, though the computational demands are typically much more extensive than those associated with the assembled beam and grillage models.

A finite elements study of a 10 storey composite frame with shear tab connections (Alashker et al., 2010) determined that for the peak load, after the connections had failed, the steel decking was the most influential component in resisting collapse through the development of membrane (catenary) forces. Since the initial design is unable to resist a progressive collapse scenario, increases in the steel decking thickness and/or in connection strength are proposed as a remediating solution. However, since the deck's full membrane resistance develops at significantly higher displacement levels, connections and floor deck strength cannot be considered to be additive.

Hoffman and Fahnestock (Hoffman and Fahnestock, 2011) used three-dimensional nonlinear finite element models and explicit dynamic analysis to study column loss scenarios for two typical multi-storey buildings with perimeter moment frames and composite floors. Again, results showed that composite flexural response is a good load redistribution mechanism. However, large demands on the connections, steel deck and concrete slab could make them individually liable to localized failure instead of them exhibiting a combined response.

Researchers at the University of Texas, Imperial College, PEC, and Walter P Moore have studied the response of composite floor systems in typical steel framed structures to determine their contribution to collapse mitigation (Williamson and Stevens, 2009). The large-scale testing of a 2-bay by 1-bay and a 2-bay by 2-bay test specimen, constructed to evaluate a perimeter column-removal scenario, is expected to provide analysis and design recommendations. Results to date demonstrate the importance of the corrugated metal decking in developing membrane forces that can mitigate collapse. The numerical study of a 4x4 bay with beams attached to the slab with shear tabs (Zolghadr Jahromi et al., 2012 -b) suggests that the composite slab can provide enough capacity to resist progressive collapse by itself, in the case of simple connections, despite the fact that the development of compressive arching action is limited by the fact that the composite floor can provide only little rotational restraint to the edges.

A similar experiment, focusing more on how the concrete slab interacts with fire in a progressive collapse scenario, used sub-assemblage specimens with typical configuration of composite slabs designed and built at Purdue University (Pakala et al., 2011).

Currently, large-scale system tests are planned by the University of Illinois (Stevens, May 2012) for investigating the integrated connection and slab behaviour when these elements are combined together in a typical composite floor system configuration. The 3-bay by 3-bay configuration (with 9m bays) was chosen so that four distinct column removal scenarios (one corner, one interior, one exterior with beams parallel to the perimeter, and one exterior with beams perpendicular to the perimeter) could be conducted using one structure.

2.5.4 Other materials

2.5.4.1 Reinforced concrete buildings

Although the thesis focuses is on steel and composite steel structures, there has been a noticeable research activity around reinforced concrete structures, also spurred by the occurrence of high profile incidents, like the 1971 collapse of the concrete high-rise at 2000 Commonwealth Avenue in Boston during its construction (Granger et al., 1971) and the more recent WTC Twin Towers collapse (Section 1.1.3.4). Guidance for applying the robustness requirements in concrete construction of the Approved Document A (BSI, 2005b) and in BS EN 1992-1-1:2004 (BSI, 2008b) exist in various publications (IStructE, 2006, Brooker, 2008). Similar references (Portland Cement Association, 2005, NIST, 2007) exist for the GSA Guidelines (GSA, 2003).

Most guidelines and guidance documents focus on the provision of horizontal and vertical ties as a means of complying. Although certain studies based on the Alternate Load Path Approach (Mohamed, 2009, Sasani et al., 2007) exist, their number and depth is significantly lower compared to steel structures (Cormie, 2011). This limits the practical application of the Alternate Load Path analysis in design, despite the fact that findings (Merola, 2009, Merola and Clark, 2009) indicate that tying capacity provisions may be inadequate for ensuring collapse prevention.

2.5.4.2 Timber structures

Prior to recent studies, most of the data available on the robustness of timber framed construction came from the Timber Frame 2000 project carried out by the Building Research

Establishment and TRADA Technology, which reported results from full-scale tests on a six storey model building constructed at Cardington in 1998 (Milner, 2005).

Of particular interest are studies in which the potential advantages and disadvantages of implementing seismic design requirements in timber structures are discussed (Branco and Neves, 2011, Dalsgaard, 2011). Their findings suggest that although the removal of weak links and increase in transversal stiffness might allow damage to propagate through the structure, like in the case of the Siemens Arena roof failure in 2003 (Hansson and Larsen, 2005), the increased redundancy and closer attention to the detailing of the connections can reduce the global damage sustained by the structure, like in the case of the WTC car bombing in 1993. Sorensen has proposed a theoretical framework (Sørensen, 2011) for the design and analysis of advanced types of timber structures with limited redundancy.

2.5.4.3 Cold-formed steel

Academic publications on the behaviour of cold-formed steel are harder to come across. Bae et al (Bae et al., 2008) investigated the vulnerability in progressive collapse of a cold-formed steel framed structure. Five different cases were considered, including an exterior wall and a corner wall column removal as specified in the GSA and DoD guidelines, as well as successive removal of columns. The results showed that the removal of corner wall columns appeared to cause progressive collapse of a portion of the second and third floor of the end bay but not of the entire building.

Generally, findings agree that while hot rolled steel arrangements satisfy formal requirements generally using tying forces, redistributing loads from damaged areas in cold-formed steel arrangements is done using catenary action in the floors. Lawson (Lawson et al., 2008) points out that the properties of the connections in this type of construction are relatively inferior to those of the composite metal decking floor with in situ concrete for resisting catenary forces. Another study (Way et al., 2007) gives some typical connection details of floors to beams for this type of construction for robustness, although they are more applicable for precast floor construction.

2.5.5 Discussion and comments

Findings agree that simple frame arrangements do not have the connection strength required to resist progressive collapse by tying of members but instead employ catenary action. Until recently, the number of studies looking into the behaviour of this type of construction under loading consistent with vertical collapse and development of catenary-type action, including large rotations and axial deformations, has been relatively low. While research on the behaviour of simple frames mainly focuses on understanding and being able to predict the behaviour of the structural components at the lowest level of idealisation (connections and beam systems), new studies should examine the robustness of this type of frames based on an assessment exercise of representative structures.

In the case of semi-continuous construction, the connection global properties (strength, ductility and stiffness) and their local, asymmetrical distribution, vary significantly, even for connections within the same frame. Connection components behave differently in tension, compression and shear. The interplay between their properties influences the resistance mechanisms that the beam system develops under sudden column loss, as well as its ultimate ductility and capacity (Nethercot et al., 2011). In addition, behaviour of the structural systems at higher levels of structural idealisation (i.e. the floors) cannot be predicted from that of the comprising subsystems in a straightforward way (Stylianidis, 2011, Vidalis and Nethercot, 2012). This is one of the reasons why tying capacity is not an entirely useful measure of robustness for this type of construction, the other being the very large rotational capacity it requires at the connections.

For both types of construction, while the contribution of the composite slab is recognised, the floors deck's full membrane resistance develops at significantly higher displacement levels and thus cannot be considered to be additive to the resistance provided by the connections. Hence, the weaker and more flexible the connections, the more likely it is that the floor slab may effectively enhance resistance in progressive collapse. For this reason, for partial and full-strength connections, increasing resistance should rely primarily on the connections themselves having sufficient capacity to carry the collapse loads. This is aligned with the view of the ICLM which considers the connections as the controlling components.

Finally, studies on continuous arrangements focus more on their performance within moment resisting frames. Although the increased strength of these connections is beneficial in the tying of members, there are still uncertainties including which are the main resistance actions for this type of systems, how design changes to moment connections will affect beam system behaviour and how this will impact the overall robustness of the frame.

2.6 Summary and conclusions

The work presented herein demonstrates that there has been a considerable increase in research activity around the issue of robustness and progressive collapse recently, loosely followed by publication of new guidance documents and updating of practice codes, especially in the U.S. Nevertheless, the majority of the studies agree that current provisions appear to oversimplify the problem with the suggestion of purely prescriptive measures. Moreover, a careful examination of the recommendations and conclusions found in the literature suggests that any credible analysis or design approach must include ways of addressing the following key features, not necessarily by modelling each of them explicitly but in a way that captures the essential aspects:

- Progressive collapse is essentially associated with dynamic effects, as failure propagates rapidly throughout the structure; these effects need to be taken into account.
- Preserving structural integrity, especially avoiding separation at the beam to column connections, is necessary to avoid collapse; failure criteria must be able to reflect that.
- Structural mechanisms for resisting progressive collapse mobilise phenomena such as gross deformations, inelastic material and connection response under complex loading; the modelling of the structural elements has to account for their combined effect.
- Failure prevention is associated with the damaged structure's ability to reach a new equilibrium position, thus strength alone cannot be a measure of robustness but needs to be combined with the concepts of ductility and energy absorption capacity.

The sophistication and widespread availability of structural analysis software makes using detailed finite element analysis to model progressive collapse more attractive. However, detailed finite element models allowing to comprehensively model each of the above key features is an obvious but onerous – in some extent – way of gaining insights, let alone restricted to each particular case study instead of providing a comprehensive evaluation of the phenomenon.

Nevertheless, the number of studies of both individual systems and entire structures presented in Section 2.5 is expected to grow even more. These studies are extremely useful in developing a better understanding of particular features of progressive collapse. However, for their use to be worthwhile in routine design, they need to include considerable detail of the diverse structural phenomena involved, which makes them prohibitively resource demanding both in terms of work hours and computing power. Thus, their role is perhaps better suited to assist forensic studies or investigations of specific aspects of the problem.

Due to the complex interplay between features, the influence of connection and frame design on local resistance mechanisms is still a subject of research, rendering any effort to provide even simple design guidance very difficult. In fact, there is no documented method in the literature that allows designers to address the needs of the structure using a streamlined process instead of a trial and error approach.

Assessing a structure is now possible via various means, an example being the Imperial College London Method, which offers a simplified framework for quantitatively evaluating structural robustness on the basis of pseudostatic capacity supply and demand. However, there appears to have been little work towards exploring how this or any other capacity indicator can be translated into specific remediating recommendations and thus provide a tool for determining effective structural modifications to ensure robustness.

Chapter 3

Welded connections modelling for progressive collapse analysis

3.1 Introduction

The prominent role of connections in defining the behaviour and controlling the performance of a frame is widely acknowledged. Developing accurate models is a continuously challenging task because of the need to allow for complex loading conditions and component interaction. However, incorporating connection design in modern building codes requires a consistent, quantitative and widely accepted method, able to take into account the contribution of each individual component and its influence on overall connection behaviour. In response to this need, the Component Method has been developed over a number of years and is now included in Eurocode 3 (EN 1993, 2010). It represents a major technical improvement because it is practical to apply and can facilitate the calculation of the internal distribution of forces, realistic moment-rotation response, rotational capacity based on component deformation and global connection properties under varying loading conditions.

In the extreme event of progressive or disproportionate collapse, local damage is not arrested locally but propagates to the rest of the building. Its main features are gross deformations, dynamic effects and inelastic material behaviour. As mentioned in the previous chapter, the ICL Method proposes a multi-level structural idealisation, which allows for the response at higher levels to be deduced from the responses at lower levels. As ultimate capacity and ductility depend strongly on connection strength, stiffness and rotational capacity, connection modelling is considered a priority. This is a necessary prerequisite for conducting rapid parametric studies that will allow the relative merits of alternative connection designs to be compared.

For a welded moment resisting beam to column connection in a frame subject to column removal, substantial axial forces will develop as the system passes through the compressive membrane, tensile and, eventually, catenary stage; existing models do not cover this loading regime (Nethercot et al., 2007, Simões da Silva, 2008). Unless extended to incorporate the connection bending moment – axial load interaction, analytical models are unfit for progressive collapse analysis.

However, current guidance in the European and British codes on how to calculate the M-N-Φ response and rotational capacity only considers end-plate partially restrained connections. Simple connections have initially received less attention because of their limited contribution to the overall frame response but as they are very common in frame construction, recent efforts have allowed constructing models able to accommodate the extreme conditions of progressive collapse (Oosterhof and Driver, 2012, Daneshvar and Driver, 2012). Depending on the type of the simple shear connection, it is sometimes possible to approximate their behaviour with the use of partially restrained connection models (Stylianidis, 2011, Vidalis and Nethercot, 2013b).

Moment resisting connections have a direct influence on the overall frame response and are employed to resist special loading conditions such as earthquakes. Although the behaviour of moment resisting endplate bolted connections can be described with approximation by models employed for partially restrained connections (Nethercot et al., 2011), modelling the behaviour of fully welded moment connections requires overcoming certain uncertainties and challenges, which are outlined in Section 3.2. Most of the work available appears to rely on the use of numerical models (Braconi et al., 2008, Hedayat and Celikag, 2009, Khandelwal and El-Tawil, 2007, Kim and Kim, 2009, Kim et al., 2009, Lee et al., 2009, Lignos et al., 2011, Park and Kim, 2010, Sadek et al., 2010, Xu and Ellingwood, 2011b, Yim, 2007), which albeit being more accurate, do not readily allow comparing alternative designs and often tether significant computing and manpower resources. Progress towards modelling their behaviour with the Component Method has not yet been documented, to the extent of the author's knowledge. Existing efforts are limited to predicting the initial response of the connection within the elastic phase of the response (Simões da Silva and Girão Coelho, 2001).

The work reported in this chapter extends previous provisions to consider the response of connections under bending and substantial axial forces and also accounts for the influence of the support columns. The following fully welded steel connections used in seismic design are considered: welded unreinforced flange bolted (WUF-B), reduced beam section (RBS) and welded reinforced with coverplates flange bolted (WCF-B) connections. The solution approach, assumptions and failure criteria for the proposed component models, which are suitable for analytical or numerical investigations of progressive collapse behaviour, are presented in detail for each of the above connection designs. Validation is by comparison against both experimental and detailed numerical results.

3.2 Steel moment resisting connections

3.2.1 Use in seismic design

In order to maintain its stability under earthquake loading, a structure must be able to provide resistance to lateral loads while absorbing the kinetic energy of the earthquake without collapsing. A common solution is the use of a moment resisting frame arrangement. The bays providing lateral resistance are usually located in the perimeter and sometimes in the core of the building. Continuous, rigid construction is achieved with the use of moment resisting connections with welded flanges or endplates. In most cases, it can lead to significant weight and cost reductions (Blodgett, 1966, Engelhardt and Sabol, 1998).

The natural occurrence of earthquakes and the threat they pose for the built environment has spurred research activity around the field of earthquake engineering. In particular, the 1994 Northridge earthquake brought considerable attention to the role of the connections in defining the behaviour of a framed structure. As a result of this incident, extensive research was conducted not only on upgrading or repairing existing welded connections but also on agreeing on appropriate specifications for new steel moment frame structures. The U.S. SAC Joint Venture (FEMA-355C, 2000, FEMA-355D, 2000, SAC Steel Project, accessed August 2013), also referred to as the “SAC Project”, examined and laid out recommendations for current and future buildings. The project was divided into several subprojects, which involved extensive connection experimental tests, numerical and analytical modelling of their behaviour and building case studies. Laboratory testing of the connections incorporating the proposals of the project have demonstrated in most cases a dramatic increase in performance and reliability compared to the “pre-Northridge” arrangements and hence they have since been widely used in new frame construction in seismic regions. Further independent studies (Miller, 1998, Popov et al., 1998, Roeder, 2002, Righiniotis and Imam, 2004, Stojadinović et al., 2000) on the behaviour of post-Northridge welded connections have suggested that weld fracture mitigation measures, such as better welding practice and notch-tough weld metal, are the key in improving connection performance and reliability.

3.2.2 Fully welded connections

3.2.2.1 Unreinforced

The ductile performance and reasonable production cost of fully welded connections make them a popular choice in seismic design. The design of the unreinforced version (Figure 3-1a) is basic: the flanges are fully welded to the column flange and the web is welded or bolted to a shear tab with slip-critical high-strength bolts and fillet welded to the column flange. Its low fabrication cost and easy erection procedure make it a very popular choice, especially for areas of low seismicity. Failure is expected to occur by flange tensile rupture without a bias towards either the top or the bottom flange fracturing first (Stojadinović et al., 2000).

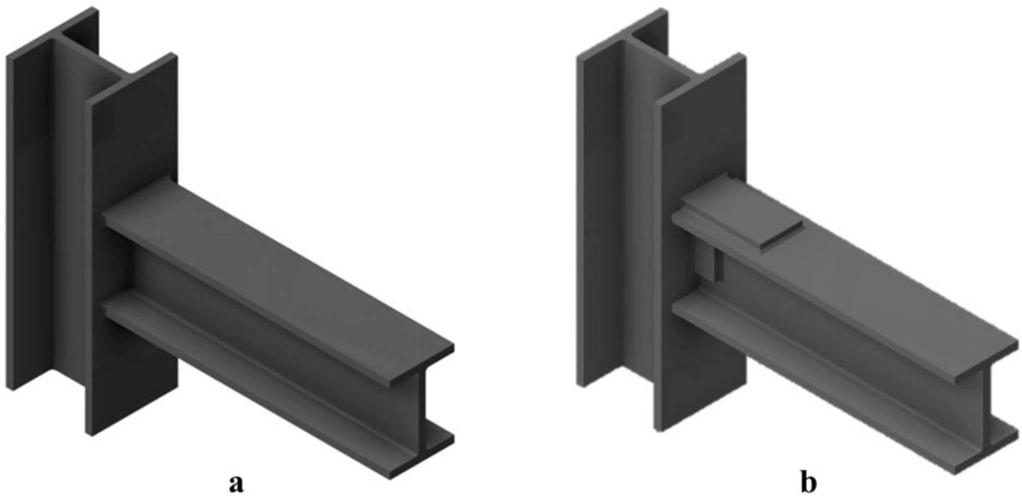


Figure 3-1: Welded unreinforced and reinforced with cover plates steel beam to column connection

Laboratory experiments (Kato, 2003) have showed that it does not make a difference whether the shear tab is bolted or welded. It is preferable that it is welded otherwise the flanges might have to bear most of the shear loading. In fact, overstressing of the flanges is one of the critical potential causes for premature failure. Hence, beam systems should be designed with connections of equal or superior moment resistance to the connected beam, in order to allow for the plastification of the beam to occur first, since little plastic deformation occurs in the connection apart from the shear panel zone deformation.

Further testing of these connections has demonstrated that they cannot consistently attain increased levels of rotational capacity, i.e. >30 mrad (Stojadinović et al., 2000, FEMA-355D, 2000), which is a required level of performance against stronger earthquake actions. In this case, variants can be used. These modifications also ensure that the location of the plastic hinge is away from the face of the column, where potential weld defects, stress concentration at weld access holes or unfavourable states of triaxial tension can cause premature and in most cases brittle fractures. The most popular approaches include:

3.2.2.2 Reduced beam section

The RBS or "dog bone" connection employs circular radius cuts in both top and bottom flanges of the beam to reduce the flange area over a length of the beam near the ends of the beam span. The flanges are fully welded to the column, while web joints may be either butt welds or bolted or welded shear tabs. These connections are common in special moment resisting frames since their very ductile performance and reasonable fitting cost make them a popular choice (Jones et al., 2000). Expected failure modes include flange tensile rupture, web buckling and flange local buckling (Jin and El-Tawil, 2005). Detailed design guidance for use in moment frames can be found in Chapter 3.5 of FEMA 350 (FEMA-350, 2000).

3.2.2.3 Reinforced with cover plates

On the other hand, various components can be used to reinforce a connection, including upstanding ribs, haunches, side plates and cover plates. The latter are the most common, least costly (The Herrick Corporation, 1994) and have demonstrated high levels of cyclic ductility (Engelhardt and Sabol, 1998). This type of connection offers certain key advantages for seismic resistant steel construction, including a highly ductile response and lower cost compared to other reinforcement options, as the rectangular shape of the coverplated connection can facilitate field construction ((Figure 3-1b).

Despite their excellent performance in a number of laboratory tests of the SAC project (SAC Steel Project, accessed August 2013), cover plated connections have also experienced some failures and introduce some difficulties in welding and inspection. Research has shown (Engelhardt and Sabol, 1998, Engelhardt et al., 1996) that very thick or long cover plates should be examined with care, as they might increase the triaxial stress state at the column face. Also, welding practices should be as stringent as possible to avoid brittle failures. The arrangement providing the best performance for this type of connection is when rectangular reinforcing plates and three-sided fillet welds joining the plate to the beam are used.

3.2.3 Studies on the behaviour in progressive collapse

Research on the behaviour of moment resisting frames has mainly focused on modelling entire or parts of a structure in order to identify the possible alternative load paths and observe the general behaviour. Although moment resisting connections have been extensively studied in earthquake loading, behaviour under the loading and deformation conditions of progressive collapse has only relatively recently been examined more closely.

Among the first points to become clear from a series of field experimental tests on the behaviour of WUF-B & sideplate connections (Karns et al., 2007) was the necessity of adequate connection rotational capacity to arrest progressive collapse. The study also warned that moment connections prequalified for rotational capacity due to bending alone might not perform equally under combined bending moment and axial loading. Failure for the WUF-B connection was brittle and it was observed at the compressive beam flange; after that, the flexural demand in the connection interface had to be resisted by the beam's bolted web, which in turn quickly deteriorated. The sideplate connection was able to maintain stability under much higher loading and failure in the system was observed in the beam instead of the connection. Results identified deep rolled wide-flange steel sections as a cost-effective solution for enhancing progressive collapse resistance because of their ductility.

Another study (Khandelwal and El-Tawil, 2007) on the ductility and strength of steel special moment resisting frame connections in column loss, performed using a micro-model based computational simulation of a two-bay sub-assemblage, showed that beam depth, yield to ultimate strength ratio and beam web-to-column detail affect the response.

Park and Kim (Park and Kim, 2010) examined how uncertainties in material properties such as yield strength, live load and elastic modulus can affect the behaviour of welded moment resisting connections in progressive collapse. They studied the behaviour of beam systems with unreinforced, coverplated and RBS connections. Although the WCF-B connection is the strongest one, the ductility of the RBS connection helps it achieve an enhanced performance. The fragility analysis showed that although uncertainties in material properties influence the initial response of coverplated connections less, it is the RBS connections that are more reliable in providing the estimated non-linear static capacity.

The U.S.A. National Institute of Standards and Technology (NIST) has recently (November 2011) initiated a project on building resilience and structural robustness. The initial phase involves testing of full-scale subsystems to validate detailed computer models (Sadek F. et al., 2010) and includes testing an individual beam under conditions that simulate column loss. The published experimental results are used herein for reference purposes and comparison with the Imperial College London simplified analysis results.

3.2.4 Representation of connection response in progressive collapse

Progressive collapse loading conditions are not only different from conventional cases but also vary significantly throughout the response (Vidalis and Nethercot, 2013b). A beam system satisfying the double span condition (Figure 2-4) develops the following main resistance action mechanisms as the vertical deflection at the point of the lost column increases:

- Compressive arching: connections loaded in compression and bending moment.
- Transient catenary: connections loaded in tension and bending moment.
- Tensile catenary (requires large rotations): connections loaded in tension.

Modelling the connection using the Component Method allows using the ICL Simplified Method for predicting the progressive collapse behaviour of individual beam systems. The step-by-step analysis of the ICLS Method allows taking into account the changes in component stiffness during the different phases of the progressive collapse response as the loading conditions (axial and bending moment) evolve.

3.2.4.1 The Component Method

The Component Method aims to provide a practical tool for analysing the rather complex behaviour of structural steel connections. In essence, it simulates the connection arrangement, properties and loading reactions with a simplified mechanical model composed of extensional springs and rigid links, which assembled together, define the connection structural properties in the same way as the original arrangement.

Current guidance limits the application of the Component Method to loading conditions where the axial load does not exceed 5% of the axial resistance of the supported beam (EN 1993-1-8, 2005). This may be applicable under conventional loading conditions but in a progressive collapse scenario, axial loading will almost certainly surpass this level.

The majority of the solutions that include the additional effect of the axial load in a connection apart from the bending moment assume either that the applied axial load is constant or that it varies proportionally to the variation in the connection bending moment (Del Savio et al., 2009, Jaspart et al., 1999). Another approach (Simões da Silva and Girão Coelho, 2001), which leads to an equivalent elastic model with bilinear springs with properties calculated using an energy formulation based on a post-buckling stability analysis, assumes that steel joints are only loaded in combined compression and bending moment. Although this may be accurate for predicting connection behaviour during the compressive arching action phase of the beam system, where both flanges of the fully welded connection are loaded in compression and bending moment, it does not account for the reversal of the axial loading to tensile, which occurs as the system deforms and forms a catenary.

Previous work at Imperial College London (Stylianidis and Nethercot, 2009) has successfully extended the component method of EC3 (EN 1993-1-8, 2005) for bare steel and of EC4 (EN 1994-1-1, 2004) for composite endplate bolted connections. This opens the way for investigating solutions for welded connections, for which supplementary information does not currently exist in the Eurocodes.

3.2.4.2 Mechanical spring model

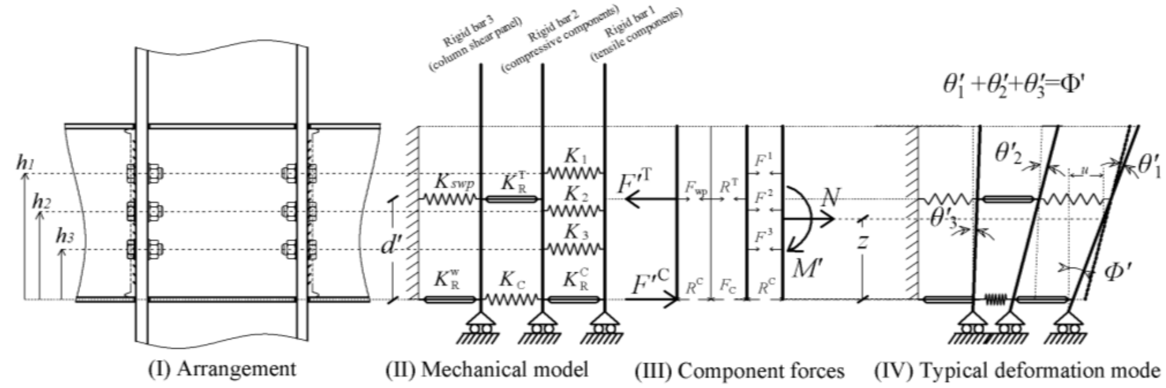
The connection mechanical spring model proposed by Del Savio (Del Savio et al., 2009) for bare steel connections has been expanded for bare steel and composite bolted endplate connections by Stylianidis and Nethercot (Stylianidis and Nethercot, 2009, Stylianidis, 2011) in order to take into account certain key additional features, including:

i. The influence of the shear behaviour of the column web in major axis beam-to-column connections in the presence of axial load. The column web shear force is equal to the minimum between the connection tensile and compressive internal forces.

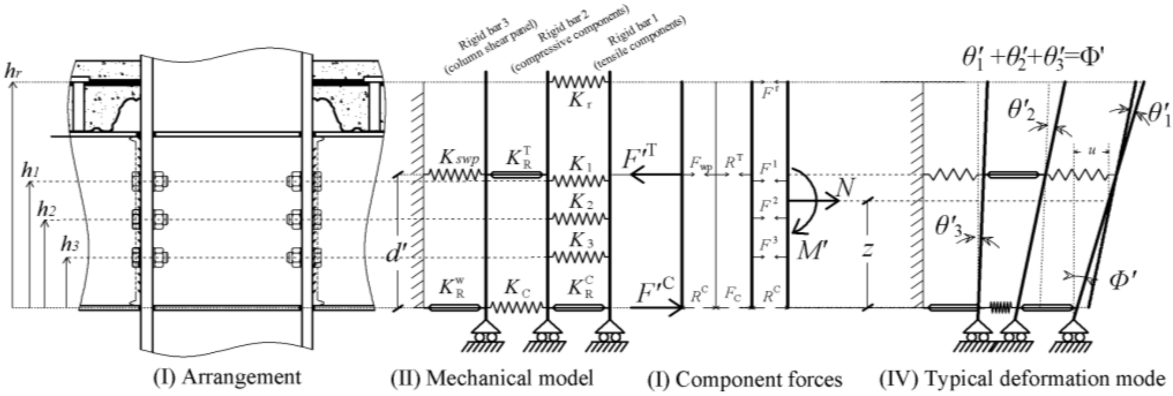
ii. Necessary adjustments to capture composite action, including taking into account the contribution of the reinforcement bar and shear studs by introducing an additional tensile extensional spring (K_r).

iii. The explicit relationship between the axial deformation (u) of the connection for axially restrained systems and the deformations in the compressive and tensile zones.

The mechanical spring model, illustrated in Figure 3-2, consists of three rigid bars (rigid bars 1, 2 and 3) associated with the connection tension, compression and shear zones respectively. The tension zone consists of a series of springs (K_i) representing the behaviour of the "tensile components". The total tensile force is transmitted to the support via a tensile rigid link (K^T_R), positioned at the level of the connection equivalent lever-arm (d), which is equivalent to z_{eq} used in the Eurocode component method (EN 1993-1-8, 2005) for calculation of the connection initial rotational stiffness. In the case of composite action, the position of the lever-arm of the tensile rigid link (K^T_R) needs to reflect the presence of the additional tensile row (K_r) as well, in order to account for the contribution of the reinforcement bar and shear studs. The transmission of the connection compressive and shear force is done a similar way - via a compressive (K^C_R) and a shear rigid link (K^S_R), which represent the behaviour of the compressive (K_c) and shear (Szyniszewksi and Krauthammer, 2012) components respectively. The former elements are located at the level of the compressive centre, which is considered fixed on the centre of the beam compressive flange in order to simplify the problem.



a. Bare steel connection subject to hogging bending moment and axial load (based on Stylianidis, 2011)



b. Composite steel connection subject to hogging bending moment and axial load (based on Stylianidis, 2011)

Figure 3-2: Connection modelling for bare steel and composite partially restrained connections (based on Stylianidis, 2011)

The deformation modes of the mechanical model depend on the component properties and the connection bending moment (M) and beam axial load (N); a typical example is illustrated in Figure 3-2a-IV. The centres of relative rotation between the tensile, compressive and shear rigid bars are defined as follows: the compressive rigid link acts as a centre (associated with θ_1) between rigid bars 1 and 2, the tensile rigid link acts as a centre (θ_2) between bars 2 and 3 and the shear rigid link acts as a centre (θ_3) between the support bar and the rigid bar 3. The sum of the relative rotations (θ_1, θ_2 and θ_3), which simulate the deformations of the connection tensile, compressive and shear components respectively, is equal to the total connection rotation (Φ). The arrangement suggests that the compression zone has a single row of components and that the behaviour of the tensile components can be expressed with a linear relationship to each other. The connection loading for bare steel connections is applied at the level of the neutral axis of the supported beam, which is positioned at a vertical distance z from the compressive centre (Figure 3-2a-III). For composite connections, the effective cross-sectional area within the region of hogging bending moment needs to be considered. The connection axial deformation (u) is calculated based on the displacement of rigid bar 1 with respect to the support bar at the neutral axis level (Figure 3-2a-IV).

The reversal of the bending moment at the point of the removed column requires modifying the model to accurately represent behaviour under sagging bending moment loading by:

i. Repositioning of the compressive centre and the compressive components of the bare and composite steel connections at the centre of compression of the beam top flange and the effective cross-sectional area of the concrete slab respectively.

ii. Redefining the connection equivalent lever-arm and the location of the tensile rigid and shear links based on their respective lever-arms from the new connection compressive centre.

Further expansion of the model in order to account for the additional influence of the supporting columns in continuous construction is presented herein. The connection components for fully welded and endplate bolted connections are summarised in Table 3-1.

Table 3-1: Common component types for endplate bolted and fully welded connections

	Endplate bolted	Fully welded
Tensile	Bolt rows in tension	Beam flange in combined bending and tension
	Endplate in bending	-
	Reinforcement bar (composite arrangement)	-
	Column flange in bending	
	Column web in transverse tension	
Compressive	Beam flange in compression	
	Column web in transverse compression	
Shear	Column web in shear	

3.2.5 Analytical representation of the connection M-N-Φ response

In order to predict the beam system response following sudden column loss, the connection modelling approach presented herein uses the analytical representation of the connection M-N-Φ response by Stylianidis (Stylianidis, 2011). This approach accounts for the combined effect of connection axial deformation, bending moment and axial force loading, which can vary in sign and magnitude during the arching or catenary resistance phase of the response of a double span beam system (Figure 2-4). A brief summary of the solution is presented below and a more detailed description is featured in the next chapter, where the analytic response representation is expanded in order to be able to encompass the complex behaviour of irregular beam systems.

Examination of the system equilibrium allows extracting the necessary equation for representing the relationship between the tensile, compressive and shear component forces and the external loading. By studying the different connection deformation modes and zones (tensile, compressive and shear) activated for each, it is possible to identify the axial loading levels corresponding to the limits between the different forces of connection behaviour. The corresponding resistance mechanisms and their properties are examined in detail in Chapter 5.

In order to be able to fully represent connection behaviour up to failure, a simplified bi-linear or multi-linear characteristic for the components' force-deformation behaviour is employed. By combining the resulting equilibrium equations, the component force-deformation equations and the compatibility equations of the system, a set of closed form expressions linking the connection component deformations with the loading conditions (i.e. axial load and bending moment) for the possible forms of behaviour expected in progressive collapse can be deducted. Assembly of the aforementioned expressions can lead to an explicit formulation of the connection M-N-Φ response of the following type:

$$\Phi = M\alpha + Nz\beta - \gamma \tag{3.1}$$

where α, β, γ are associated with geometric and material properties of the connection, while z corresponds to the level of application of N, from the centre of compression of each connection respectively.

The resulting solution is a representation of the connection moment-rotation response based on a tri-linear curve approximated by the connection tensile, plastic and ultimate stiffness as well as the yield, ultimate and final strength, as defined by the corresponding parameters of the individual components (Stylianidis, 2011). The most effective simulation of performance can be achieved with the use of a step-by-step analysis, in order to allow for the variations in stiffness of the connection components and the external loading.

3.2.6 Influence of the support columns

Depending on the position of the lost column in the frame arrangement, the connections at the supporting columns might be axially restrained or unrestrained. Connection axial restraint is defined as “the effective axial stiffness of the beam system on the opposite side of the support joint which is considered as identical to the system under consideration. The effective axial stiffness may be different in tension and compression due to the different compressive and tensile stiffness of the connection on the opposite side of the support joint" (Stylianidis, 2011).

For fully restrained connections, commonly used in moment resisting frames, the definition of axial restraint needs to be expanded to take into account the resistance against deformations of the column in bending due to the catenary pull-in effect.

Moreover, the absence of bracing supports in some floors combined with significant catenary axial forces following column removal can lead to small rotations and translatory displacement of the column flange at the end of the column. In the case of simple or partially restrained connections, this effect is insignificant due to the large difference between the connection’s axial stiffness and the lateral stiffness of the column in bending. However, in the case of fully restrained, full-strength connections, accounting for this effect is an essential addition to the model in order to be able to:

i. Improve the accuracy for predicting the connection response: deformations may be insignificant for support columns in the interior of the frame, however, in the case of edge and corner support columns, the aforementioned deformations need to be considered additive to the beam flange components deformations in order to define the strains at the connection tensile zone.

ii. Account for common laboratory conditions: without the rest of the structure, the test assembly bracing is not realistic, which can influence test observations. For example, it may lead to the reduction of the strains of the support column connection beam flange, while increasing the equivalent for the centre connection (point of removal column), resulting in failure being observed more commonly at this connection first.

iii. Arrest buckling under combined compression and bending moment of unbraced columns for double span beam systems with unreinforced or reinforced fully welded connections (see Section 3.2.3.2).

The term "**bracing**", arbitrarily used in this study to take the contribution of this factor into account, is thus expressed as the stiffness of the support connections against inward horizontal displacement due to the strong axial forces applied during catenary action. It is represented in the connection mechanical spring model with an additional extensional spring at the level of the neutral axis of the connection (Figure 3-3) with a linear component characteristic, defined by the combined influence of the following:

- Flexural stiffness of the column.
- Axial or flexural stiffness of diagonal braces, if any.
- Presence of above floors and/or of adjacent structure.

For symmetrical connections, like the fully welded connections examined herein, the effective axial stiffness is equal for their compressive and tensile zones. In the case of cantilever girder systems, the degree of axial restraint is considered negligible as the edge connection is almost free to move in the axial direction. In a similar manner, for double span girder systems with connections free to move outside the perimeter of the frame, only the contribution of the supporting column bending stiffness is taken into account. In this case of unbraced support columns, very low levels of axial restraint can be considered, despite the absence of a beam at the opposite side of the support connections.

Thus, the position in the frame of the supporting columns– which depends on the position of the removed member - influences the degree of restraint and "bracing" at the connections. Table 3-2 presents the combined axial restraint and bracing conditions for the various column removal scenarios. These conditions have a major influence on the type of loading at the connections; different resistance mechanisms are activated, as explained in Chapter 5. The examples correspond to the case study of the SDC-D frame examined in Chapter 6, where the simplified frame arrangement is illustrated in Figure 6-1.

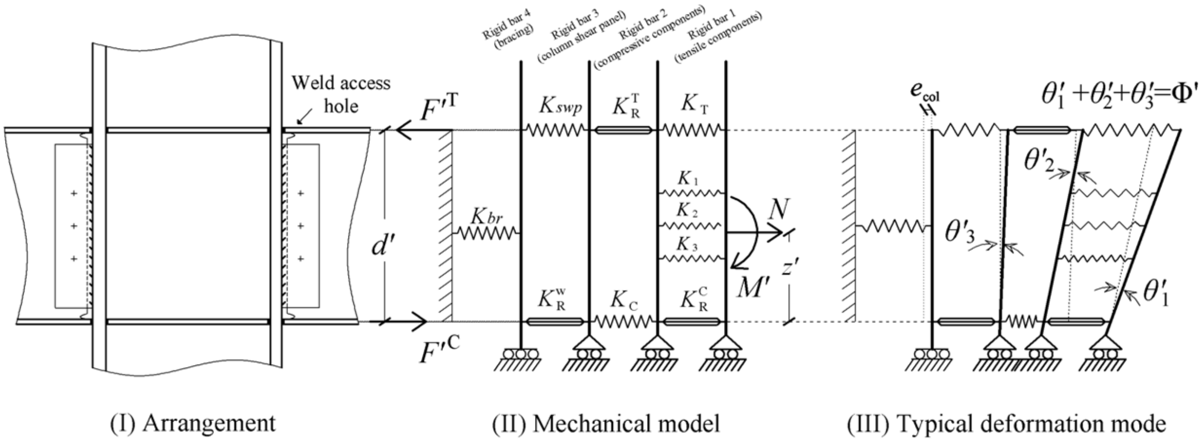


Figure 3-3: Connection modelling including the full influence of the supporting columns

Table 3-2: Girder system types according to the position of the lost column

Support column position	Degree of axial restraint / bracing	Connection loading	Examples (Chapter 6)
Interior	Full because of the girder at the opposite side of the connection	Average bending moment Significant axial forces	Ey3, Ex3
Edge	Low because only the corner column contributes (≈ unrestrained)	High bending moment Low axial forces	Ey4, Ex4
Corner	Negligible (≈ unrestrained)	Bending moment only	C1, C2

3.3 Welded moment resisting connection modelling

3.3.1 Introduction

Three popular types of welded moment resisting connections are modelled using the Component Method. The member sizes correspond to those used in the case studies of Chapter 6 and are considered representative. The associated prototype frames are taken from widely used U.S. publications (FEMA-355C, 2000, NIST, 2011):

i. For the WUF connections: the SAC Project “Boston” and the NIST Robustness Project SDC-C frames.

ii. For the RBS connections: the NIST Robustness project SDC-D frame.

iii. For the coverplate connections: the SAC Project “Los Angeles” and “Seattle” frames.

Connection design and dimensions are calculated according to the recommendations of FEMA 350 (FEMA-350, 2000) and the Eurocode (EN 1993-1-8, 2005).

3.3.2 Assumptions

In order to maintain simplicity while preserving sufficient accuracy, the following assumptions are made:

i. All welds are considered as rigid, given the existing stringent welding specifications.

ii. As the column web in shear is stiffened by doubler plates in the prototype moment resisting frames, its behaviour is considered rigid. Also, stress concentrations have been found to be very low at the corresponding welds (Sadek F. et al., 2010).

iii. The column flange and web will be considered to behave as rigid, as the stiffening with continuity and doubler plates (common in moment resisting connections used in seismic design) minimises their contribution to the rotation of the connection.

iv. The beam web contribution to bending moment and axial loading resistance is limited to a maximum of 20% (EN 1993-1-8, 2005), in order to account for the combined axial and bending moment loading with shear forces.

v. The shear tab or sideplate bolted to the beam web is only considered in the component model of the unreinforced connection, as in the RBS and in the reinforced arrangement the critical plane is located away from the face of the column.

The models presented in this chapter aim to approximate behaviour rather than to provide an exact prediction. Certain uncertainties, mentioned below, exist by default; accommodating them would disproportionately increase the complexity of the problem.

i. The exact contribution of the beam web in bending moment resistance is not known because of the combined axial, shear and bending moment loading.

ii. The complex stress distribution and state in the connection region at the face of the column cannot be accurately predicted using simple bending theory. The uncertainties involved include the participation of the flange and web connection region in shear and moment resistance and the stress concentrations in the full penetration weld and at the beam region at the end of the coverplate.

iii. In addition, the component models being deterministic, they cannot account for variations and inconsistencies in the material properties. Although element properties used in numerical modelling are sometimes calibrated by coupon tests, this does not guarantee compliance for the actual tested sections.

3.3.3 Failure modes

3.3.3.1 Tensile rupture

When the elongation of the beam flange in bending under combined bending and tension exceeds maximum deformation capacity, the flange begins to rupture with failure spreading into the web almost instantly. The allowed maximum strain is 18%, which is the minimum accepted value for ASTM A992 steel based on a 200 mm specimen. Arguably, this is a rather conservative measure as failure strains have been reported to go up to ≈ 28%.

This failure mode is associated with the uncertainties mentioned in Section 3.3.2. In addition, the component model does calculate true strain values, which might be different than plastic strain due to changes in the cross-section area of the flange in tension for very large deformations. However, it offers a simplified approximation of behaviour which can be improved in future studies.

3.3.3.2 Inelastic local buckling

3.3.3.2.1 Introduction

In progressive collapse, the connections suffer significant overstress and at a critical strain, local buckling of the plastic sections may initiate, limiting the connection's ability to provide resistance or additional rotational capacity.

3.3.3.2.2 Solution approach

Inelastic local buckling of a beam flange is a complex issue and the aim of several research efforts has been to describe, predict and provide failure criteria for it. Seismic design requires limiting plate and lateral slenderness to critical limits, often considering that a plastic section will reach strain hardening before local buckling occurs (Lay, 1965b).

Lay and Galambos (Lay, 1965a) assumed failure to occur when the length of the yielded portion of the compression flange is equal to or greater than the full wave length over which the local buckle would develop in the flange. Their study outlines the limiting conditions for a beam section to be used in the inelastic range in order to avoid occurrence of local buckling in the fully yielding case. Some of the most influencing factors are the length of the fully yielded region, the b/t ratio, the moment gradient of the beam and the steel strain-hardening stiffness. An analytical method for predicting the section inelastic shear modulus shows that this property is negatively affected by the presence of shear stresses, mainly because of material imperfections and eccentricities. Lay's local buckling slenderness limit (for steel up to 345 MPa) is equal to:

$$\lambda\rho = 0.38\sqrt{E/f_y} \tag{3.2}$$

Kato (Kato, 1990) examined the behaviour of members in tension and compression affected by local buckling in part of the section or by buckling of the section. The results showed that strength and deformation capacity are controlled by the steel yield ratio; members with very high ratios exhibit a decreased deformation capacity. Furthermore, a series of tests for different steel grades was studied in order to provide an expression for calculating the wave length of the yielded section that corresponds to the inelastic local buckling initiation (Kato, 2003). Both studies verified that inelastic local buckling of the flange in compression in a beam-to-column connection limits the rotational capacity of the connection but reaching an explicit analytical solution represented a complex problem.

A review (Daali and Korol, 1995) of existing research on local buckling and strain hardening of a plasticised flange in compression compared the analytical solutions with experimental tests. Upon establishing agreement between the two, the validated formulas were used to develop interaction diagrams describing the relationship between the flange, web and lateral slenderness. Using the solutions by Lay and Galambos (Lay, 1965a), Kato (Kato, 1990) and Kuhlmmann (Kuhlmann, 1989), Daali and Korol proposed a formula for determining the yielded length at which buckling occurs for a wide flange section in flexure:

$$l = 0.3997 - \frac{a_f^2}{\left(E/480\right)} - \frac{a_w^2}{\left(E/480\right)}, \tag{3.3}$$

Where:

$$a_f = b/T\cdot\zeta \tag{3.4}$$

$$a_w = D/t\cdot\zeta \tag{3.5}$$

$$\zeta = \sqrt{300/\sigma_y} \tag{3.6}$$

This solution however, needs to be used within a connection modelling approach that can take into account the combined effect of the connection bending moment and axial loading (Section 3.2.4). In the component models developed herein, failure is considered to occur when the yielded length of the beam flange reaches the critical value of equation 3.3. A linear moment gradient is used to calculate the bending moment loading in the RBS and WCF-B arrangements where the connection region is located away from the face of the column.

Cantilever beam systems are expected to be more vulnerable compared to axially restrained systems, as the latter develop important tensile catenary forces that reduce the strains in the connection compressive components. Nevertheless, the case studies of the next chapters suggest that it is not common for inelastic local buckling to occur prior to flange rupture. This is due to the fact that the realistic geometry of the flange in compression differs: a free flange in the region of the access hole followed by a complete section creates a discontinuity point with increased stress concentrations. This is where local buckling has been observed in experimental tests (Sadek F. et al., 2010).

As it is not possible to capture this failure mode with the use of bilinear elasto-plastic elements with strain hardening, validation of the accuracy of the proposed analytical solution is only by comparison with the experimental studies of Section 3.4.4.3 and Section 3.5.4.3.

It is possible for the connection to be able to still provide tensile resistance after the compressive flange has buckled, depending on whether and how failure propagates through the beam web. Predicting this complex behaviour is beyond the scope of the present study.

3.3.3.3 Column buckling

Significant axial forces develop in full-strength connections during a progressive collapse scenario. The connected column is subsequently loaded in bending and compression, which can cause buckling. A recent study (Khandelwal and El-Tawil, 2011) has observed a similar phenomenon for an intermediate moment resisting frame, leading to disproportionate collapse of the structure. Despite its severity due to its propagating nature to the rest of the structure, this failure mode is uncommon. However, in the case of stiff, full-strength connections, column buckling is possible because:

i. Very strong tensile axial forces develop in the connection prior to its failure.

ii. The horizontal force component of connection axial loading is higher compared to other cases, due to the high rotational stiffness of welded connections.

An additional failure mode is thus considered for beam systems with welded unreinforced and reinforced connections: column buckling under combined bending and axial load, based on the values provided in clause 6.3.3 of the EC3 (EN 1993-1-1, 2005).

3.4 Welded RBS connection modelling

3.4.1 Introduction

A component-based model able to predict the behaviour of a beam system with RBS connections in progressive collapse requires overcoming certain challenges, which include:

i. The connection region is not located at the face of the column but slightly afar from that. More specifically, the thinnest flange section, where failure usually occurs, is often located at approximately 300 - 400 mm from the face of the column.

ii. The properties of the RBS section after yielding of the first plane cannot be accurately described by a bilinear or tri-linear curve as there is a significant spread of inelasticity (expanding plastic hinge) in the cut area.

iii. There is no analytical model available in the literature, to the extent of the author's knowledge, capable of predicting the behaviour of the RBS section under combined bending and axial load and large rotations.

3.4.2 Solution approach

The radius-cut region is replaced with a section of equal length and equivalent width (B_{eq}) as shown in Figure 3-4, based on the formula initially proposed by Lee (Lee and Chung, 2007) for calculating storey drift of a steel frame with RBS connections:

$$B_{eq} = \frac{X1}{X2} \tag{3.7}$$

with

$$X1 = b \cdot \left(\frac{L_b}{2} - (a + \frac{b}{2})\right) \tag{3.8}$$

$$X2 = \frac{(L_b - 2a - b)}{\sqrt{\frac{b_f - 2c}{R}}} * \tan^{-1}\left(\frac{b}{2R\sqrt{\frac{b_f - 2c}{R}}}\right) \tag{3.9}$$

It considers a linear moment profile over the radius-cut region, which is an accurate simplification for centre connection displacements of up to at least the beam depth (w = D) and a reasonable approximation for up to w = 1.5D for beam systems with length over depth ratio over 10. For larger deformations, the spread of inelasticity in the reduced section significantly alters the moment profile and use of the equivalent standard width section overestimates the response. However, for the case studies examined in the next chapters, failure is observed before the simplified model begins to diverge from the finite element model. This model is only valid for connections with a double axis of symmetry.

Although the equivalent section can be used to calculate the properties of the tensile and compressive components, calculating the maximum capacity of the system requires considering a reduced beam length, in order to produce an accurate estimate of the connections' ultimate rotational capacity. This length is equal to the distance between the thinnest regions of the RBS connections:

$$\text{Reduction} = \frac{(a + b)}{2} \tag{3.10}$$

With

$$a = 0.55\ B \tag{3.11}$$

$$b = 0.70\ D_b \tag{3.12}$$

$$c = 0.25\ B \tag{3.13}$$

For a 6m beam average sized beam, this value is around 0.5 m to 0.8 m. The ultimate ductility obtained from the analysis is used with the response of the original system in order to calculate the corresponding maximum capacity obtained at the point of failure.

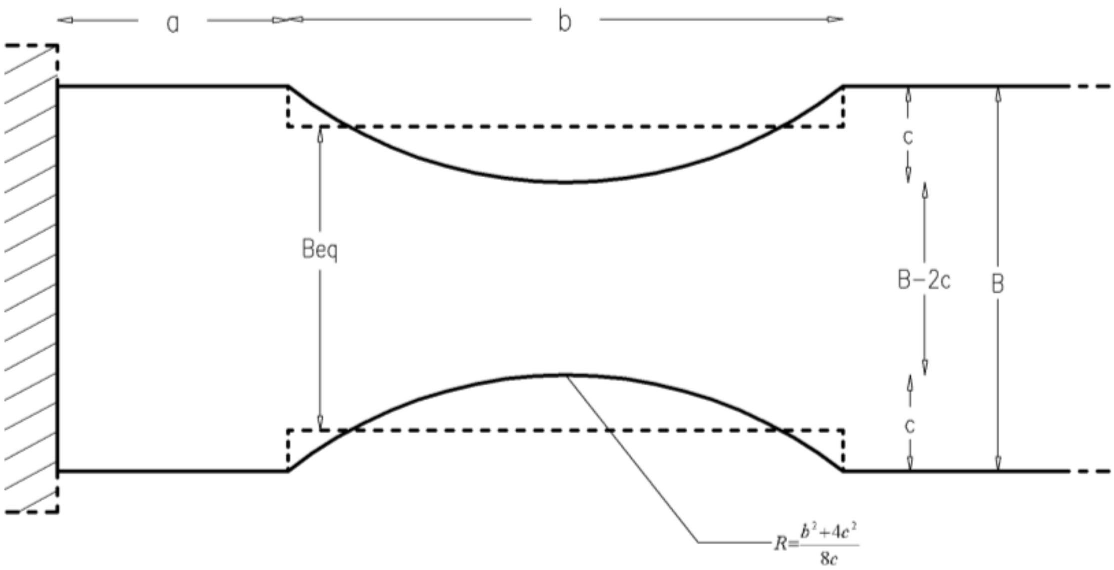


Figure 3-4: Equivalent section with standard width

3.4.3 Component failure modes

Flange tensile rupture under combined tension and bending is the most common expected failure mode for this connection. The failure criteria used in this model are expected to yield conservative results, as the ductility of the radius-cut region is higher than that of an equivalent width region because of the beneficial effect of the spread of inelasticity.

Notwithstanding, they provide a reasonable quantitative estimate (within 10% compared to FE results) and an excellent qualitative estimate for the mode of failure and the location of the critical connection. For the section size of the systems considered in the validation exercises and the case studies in the next chapters, the 18% maximum strain criterion corresponds approximately to 70-90 mm of the tensile component deformation (calculated based on the length of the equivalent section). Depending on the reduction percentage of the flange and the beam slenderness, failure of the compressive flange in inelastic buckling under combined compression and bending may also be observed. Failure of a double-span beam due to buckling of the support columns under large system deformations is less likely expected to occur for the RBS arrangement, as this is not an entirely full-strength connection. Table 3 summarises the failure modes for the main tensile and compressive components of the spring model presented in Figure 3-5.

Table 3-3: RBS connection - component failure modes and loading

	Tensile	Compressive	Shear
Beam flanges	Rupture under combined tension & bending (critical for most beam systems)	Inelastic local buckling under combined compression & bending (critical for cantilever systems)	Does not participate
Beam web	20% participation - not critical		Not critical
Column web	Stiffened by doubler plates –considered rigid		
Column flange	Stiffened by continuity plates –considered rigid		
Welds	Stringent welding requirements – considered rigid		
Bolts	Because of the high stiffness of fully welded flanges, the shear tab connection components are not considered critical, which is supported by experimental data (Sadek F. et al., 2010)		

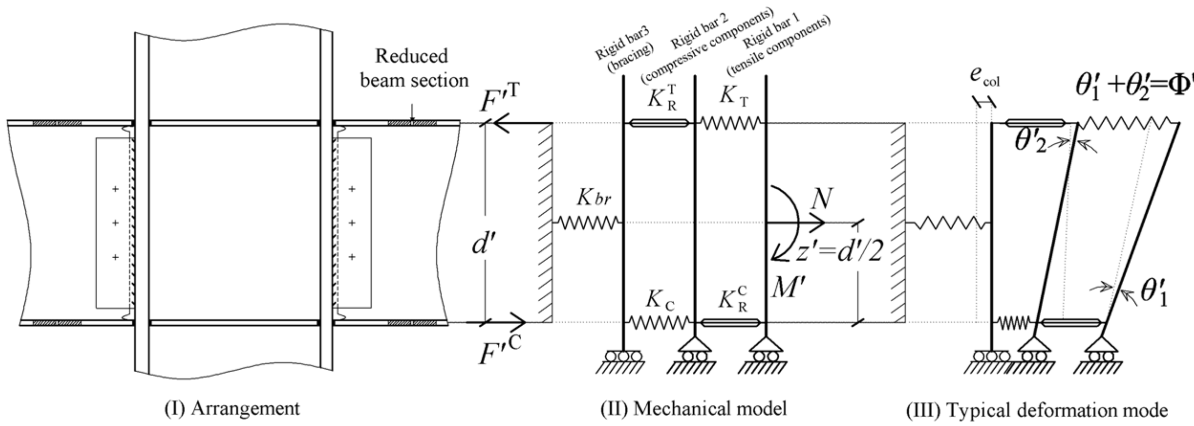


Figure 3-5: RBS connection spring model

3.4.4 Validation of the RBS connection model

3.4.4.1 Verification against ADAPTIC analysis

A finite element model of a beam system with RBS connections was constructed using the ADAPTIC (Izzuddin, 1991) analysis software in order to validate the accuracy of the simplified ICL model. The results obtained from the 2D elasto-plastic static analysis were converted to pseudostatic using the ICL Method (Izzuddin et al, 2007) . The control phases used to draw the load deflection curve were displacement and load control at the centre connection.

In the bilinear steel model with kinematic strain-hardening of 1%, the sections were modelled using cubic elasto-plastic formulations to utilise the full inelastic characteristics of the steel beam. The boundary conditions were modelled using 2D joint elements with uncoupled axial, shear and moment actions for taking into account the axial restraint at the beam ends due to the presence of the adjacent structure and using rigid links at the welded regions.

Figure 3-6 shows the deformed shape of the model equivalent to a double-span beam system in which symmetry was used to reduce it to an equivalent single span, which is less computationally demanding. The radius-cut region was approximated using a series of sections with decreasing flange sections (Figure 3-7). In addition, for the sake of simplicity, the beam web is considered to be fully welded in the FEM model instead of being connected with shear tabs; this is not expected to have a significant bearing in the final response of the system (see Section 3.2.2.1). Also, the adjacent structure and the columns were introduced in the model via appropriate axial restraint boundary conditions at the beam edges.

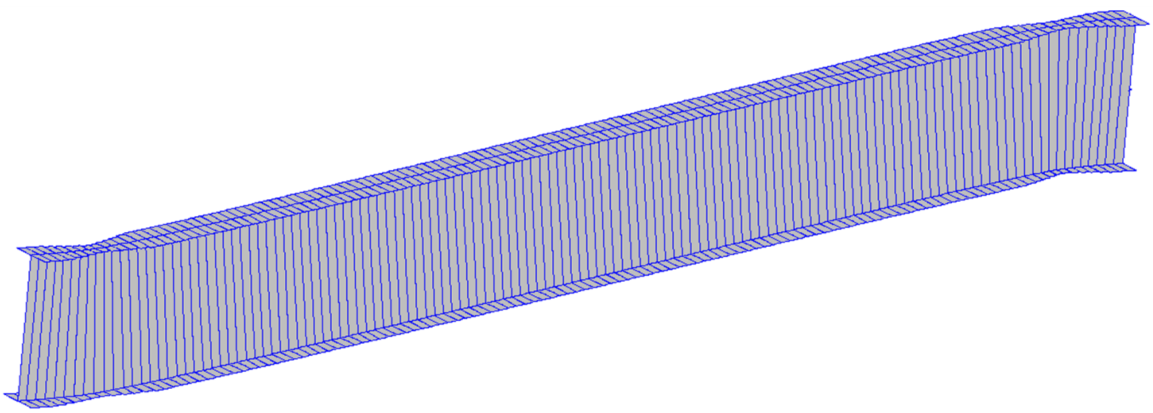


Figure 3-6: FEM model of a 6m W24x107 beam system at failure point

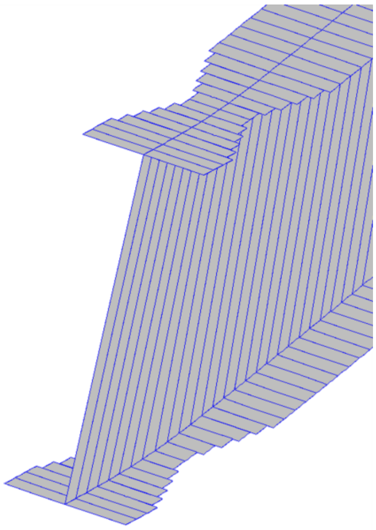


Figure 3-7: RBS FEM modelling detail

The response comparison for the 10 beam systems with varying length over beam depth ratios of Table 3-4 shows a very good agreement between the finite element analysis results and those from the proposed model. For the static response, the average standard deviation for all the beam systems is D_{Pd} = - 4.42%. Figure 3-8 shows the response comparison for 4.5m, 6m and 9m double span W24x94 beam systems after column loss at the centre connection. The conditions of interior column loss are simulated with the presence of appropriate axial restraint conditions at the support connections. For the pseudostatic response, the standard deviation between the results obtained with the equivalent width and the numerical model is D_{Pd} = - 5.29%; Figure 3-9 shows the response comparison for the same beam length and degree of axial restraint of a double span W27x102 beam system. The model's failure point estimate is conservative, as expected. The detailed analysis results and the complete set of figures comparing the model and the numerical analysis responses are included in Appendix A.

Table 3-4: Beam L/D ratio information for the parametric FE validation exercise

Beam L/D	W24x76	W24x94	W27x102	W30x108
L = 4.5 m	7.4	7.3	6.5	-
L = 6.0 m	9.9	9.7	8.7	7.9
L = 9.0 m	-	14.6	13.1	11.9

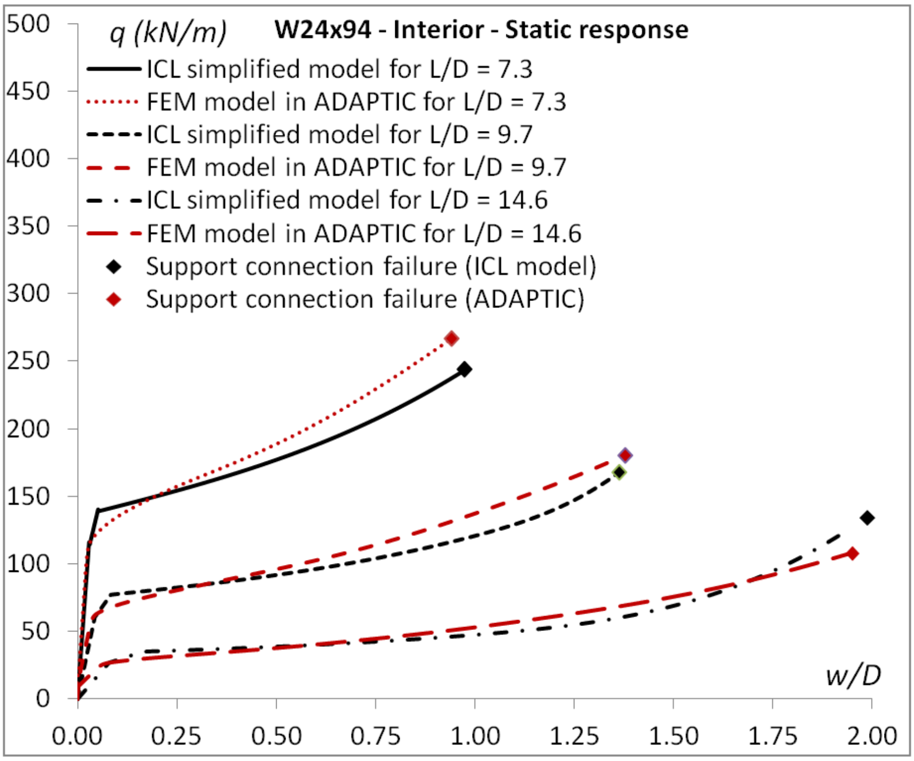


Figure 3-8: Static response comparison for the W24x94 axially restrained beam system

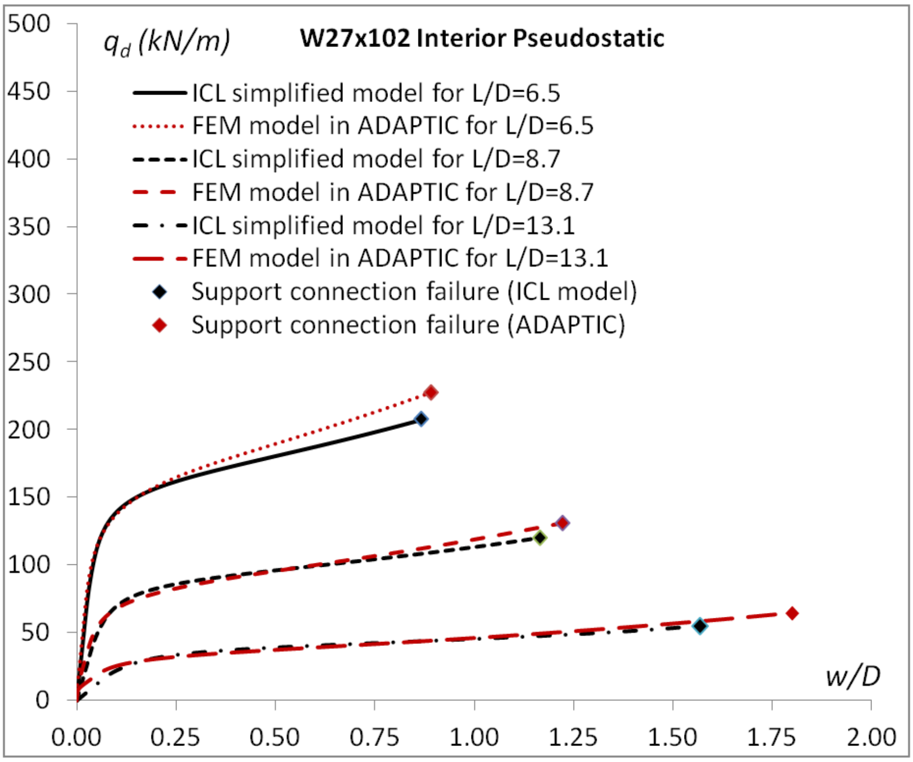


Figure 3-9: Pseudostatic response comparison for the W27x102 axially restrained beam system

3.4.4.2 Verification against independent numerical studies

In their study (Khandelwal and El-Tawil, 2007), Khanderwal and El-Tawil examined the ductility and strength of steel special moment resisting frame connections in column loss using a micro-model based computational simulation of a two-bay sub-assemblage. Several systems were analysed to determine whether the beam depth, yield to ultimate strength ratio and beam web-to-column detail affect connection response. The failure criteria were based on the Gurson micromechanical fracture model, which was calibrated using experimental data.

The comparison will be performed for the dynamic and pseudostatic response of the "S-5-RBS" system, which is similar to the beam systems that the ICL simplified model was developed for. It corresponds to a 9.14 m double-span beam system employing a W27x102 section with RBS connections of a 40% flange reduction. Figure 3-10 illustrates the good agreement between the results both in terms of the form of the response and failure point. The simplified method was expected to slightly overestimate ductility, although the difference observed in this case is -2% (ICL results more conservative).

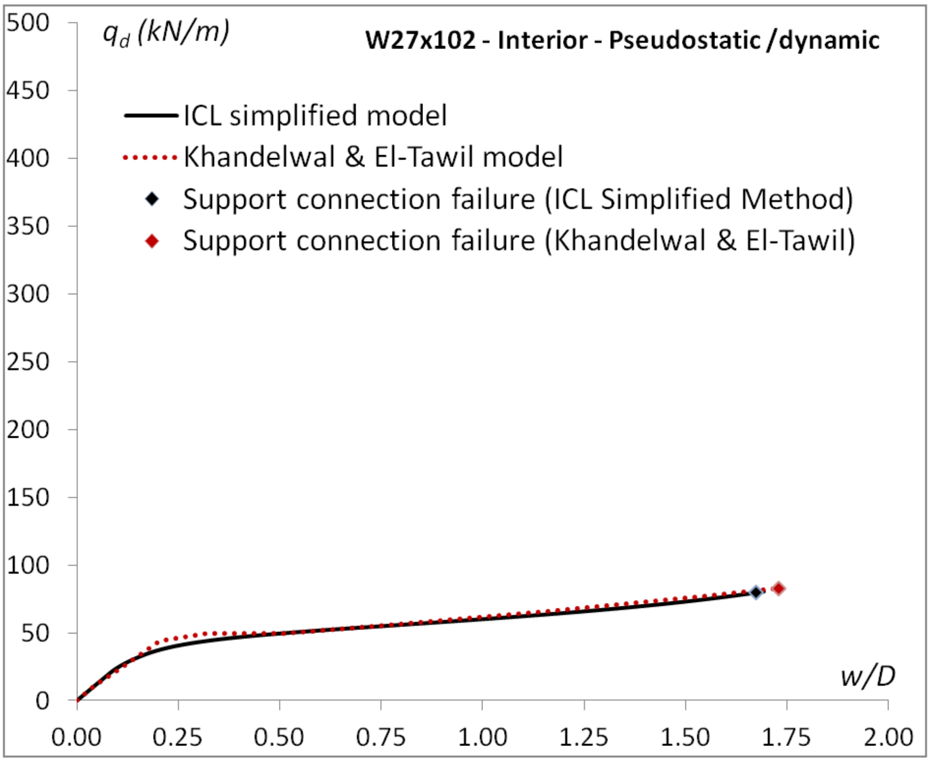


Figure 3-10: Response comparison for the ICL and the Khandelwal and El-Tawil model

3.4.4.3 Verification against experimental tests

As mentioned in the previous chapter, the U.S. National Institute of Standards and Technology (NIST) has recently (November 2011) initiated a project on building resilience and structural robustness. The primary phase involves testing of full-scale subsystems to validate detailed computer models (Sadek F. et al., 2010) and includes testing of an individual beam under conditions that simulate column loss.

The assembly consists of a double span W24x94 girder connected to a W24x131 column using radius-cut RBS connections with a 50% reduction. The system is monotonically loaded at the centre in order to simulate column loss. In order to compare both the static and dynamic responses of the beam system, the ICL method was used to calculate the pseudostatic response from the static response of the test-setup (Izzuddin et al., 2008).

However, there are certain differences between the ICL simplified model and the experimental setup which might affect the comparison exercise; they are presented in Table 3-5. Notwithstanding, there is a very good agreement between the static and the pseudostatic response as shown in Figure 3-11 and Figure 3-12 respectively.

Table 3-5: Differences between the NIST experimental assembly and the ICL simplified model

Parameter	Comments
Loading type	Since it is very hard to simulate sudden column loss, the experiment is carried out by gradually applying a static load at the centre connection. The loading process however is not continuous but instead interrupted three times because of the limited stroke of the hydraulic actuator, during which time the structure is unloaded and re-loaded. This is only allowable in static testing. On the other hand, the ICL method can predict not only the static but also the pseudostatic response by taking into account dynamic effects in order to evaluate the system pseudostatic capacity, since progressive collapse is essentially a dynamic scenario.
Strain measurements	Because of practical limitations, the strain gauges in the test are not located at the exact component locations but at adjacent locations.
Bracing & axial restraint	In an actual frame, the columns are predominantly loaded in compression throughout most of the response. The bending moment due to the beam “pulling” the column via catenary action becomes significant for deformations usually exceeding the 150% of the depth of the beam (Nethercot and Stylianidis, 2011), given that the connection rotational capacity is not exceeded beforehand. In the case of the test assembly, there is no compressive load imposed on the columns because of the absence of the above floors. Instead, because of the bracing setup, they are loaded in tension rather than compression after a certain point in the experiment. The horizontal displacement and the rotation of the column edge are not negligible in this case, since the difference in stiffness in the horizontal direction (along the beam) is quite different between the experimental setup and the model. In the NIST laboratory, the support joints attain an inward displacement of 27mm - approximately ten times larger than the ICL analysis estimates for internal column loss in a seven storey frame or higher. This makes the results of the ICL model appear more conservative.

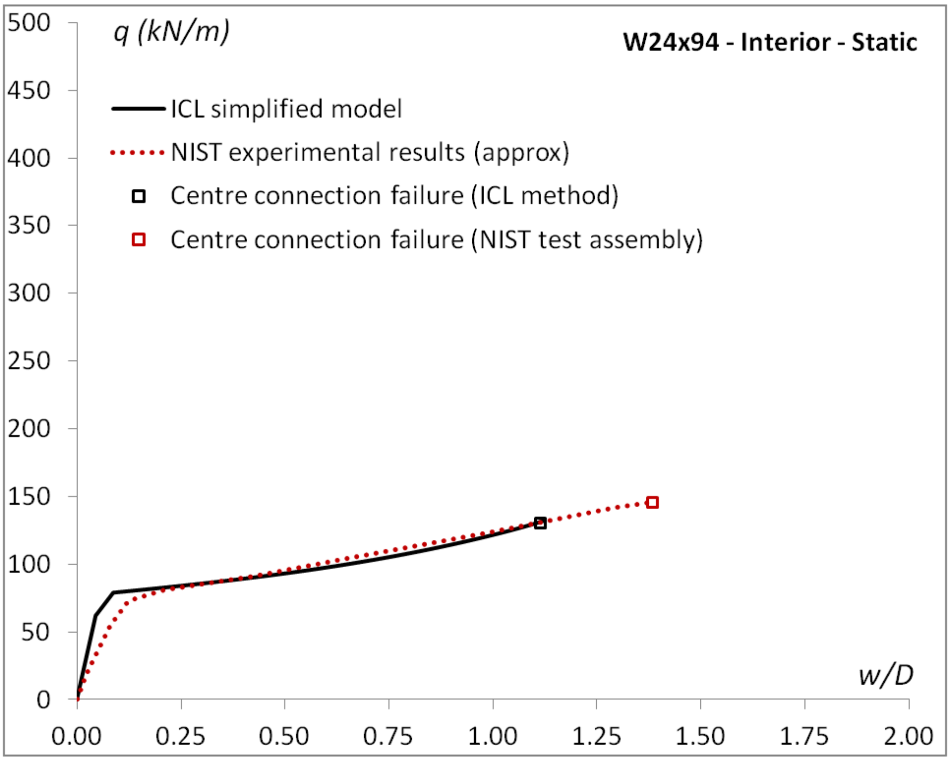


Figure 3-11: Static response comparison for the ICL analytical and NIST experimental results

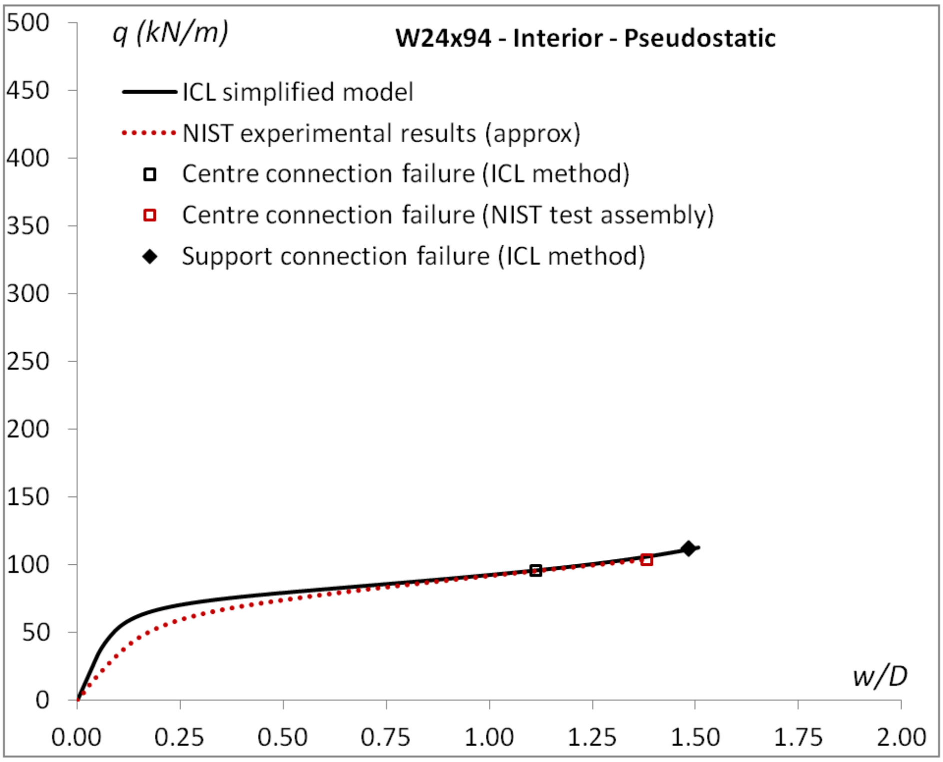


Figure 3-12: Pseudostatic response comparison for the ICL analytical and NIST experimental results

3.5 Welded unreinforced flange-bolted connection modelling

3.5.1 Introduction

Despite the WUF-B's relatively simpler design compared to the RBS connection, its plastic behaviour under large deformations is rather complicated, due to:

i. Bolt irregular behaviour, which is hard to model analytically because of the combined effects of bolt shear deformation, bolt shank rotation and stub plate deformation induced by bearing. As bolt participation increases with connection rotation, the bolt row may attain its maximum capacity before failure of the flanges in some cases.

ii. The important difference in stiffness between the bolted web and the welded flanges causes the latter to become significantly overstressed as they are expected to bear the shear forces imposed on the connection. In fact, the "deformation compatibility" of the flanges and the web can define the critical component (Kato B., 2003). For larger sections, failure is expected to be triggered by local buckling of the flange in compression, followed by bolt shear failure or rupture of the flange in tension. Average sections are expected to undergo the opposite failure sequence. In smaller sections, although the beam flanges are within the plastic zone, bolt rupture is expected to trigger failure of the connection.

iii. Determining the initial stiffness of the connection requires evaluating the effective length of the participating beam before a region reaches its yielding stress limit. Even after the elastic phase, evaluating the true stress vs. plastic strain characteristic requires calibration with experimental results and it is expected to be significantly influenced by the material properties (Simões da Silva and Girão Coelho, 2001).

3.5.2 Solution approach

For the simplified model of the connection, the bolts will be considered to be in the same plane as the column flange. Their contribution will be modelled with springs in tension at different lever arms and their stiffness and resistance in shear will be used to calculate their properties. The hypothesis that the resistance of the bolts in the vertical loading direction is negligible will be validated from the numerical model.

Upon studying the exact geometry of the connection, it becomes apparent that the weld access holes play an important role in defining behaviour: the critical region is expected to be between the full penetration welds and the end of the weld access holes, as this is the weakest part of the arrangement. The components' properties will be calculated based on the geometrical characteristics of this region.

3.5.3 Component model and failure modes

Critical modes include connection failure and column buckling. The first is more common, triggered either by local buckling of the flange in compression or fracture of the flange in tension and followed by shear failure of the bolts and immediate failure of the intact flange. Table 3-6 summarises the potential failure modes for the main tensile and compressive components. The spring model of the connection is presented in Figure 3-13.

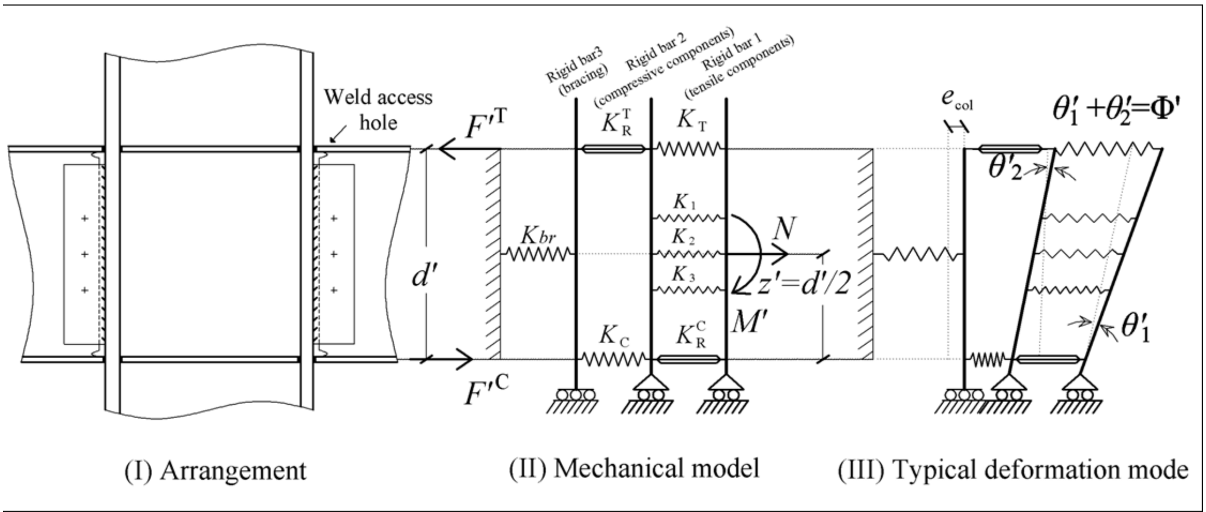


Figure 3-13: WUF-B connection spring model

The failure mode for the flange in tension is similar to the RBS connection: rupture is considered to occur at the reduced flange section in tension for an allowed maximum strain of 18% at least, calculated on the same length of a coupon experiment of 200 mm. Due to the uncertainties associated with the participation of the flange and web connection region in shear and moment resistance and with the stress concentrations at the full penetration weld, beam region and end of the beam, this simplified model should be used for sections that are close to the ones it is validated against: D = 500-700 mm and B = 170-340 mm.

The support columns are potentially critical in buckling under combined compression and bending moment. The bending moment loading is calculated based on the axial forces in the connections generated during the tensile catenary action phase. The WUF-B connection arrangements are more likely to suffer this failure mode than the RBS arrangements.

Table 3-6: WUF-B connection - component failure modes and loading

Components	Tensile	Compressive	Shear
Bolts	Bolt in shear failure		
Girder flanges	Fracture near the weld access hole under combined tension and bending	Local buckling under combined compression and bending	
Column web panel	Stiffened	Stiffened	Stiffened
Column	Potentially critical in buckling under combined compression and bending due to strong catenary forces		

3.5.4 Validation of the WUF-B connection model

3.5.4.1 Verification against ADAPTIC analysis

A finite element model of a beam system with WUF-B connections was constructed using the ADAPTIC analysis software in order to validate the simplified ICL model. The procedure, section sizes and beam lengths were the similar to the RBS validation exercise. The high strength ASTM A490 bolts' shear tri-linear characteristic was constructed from experimental data (Kulak G.L. et al., 1986): $F_{\tau,y} = 232$kN, $F_{\tau,u} = 330$kN, $\Delta_{\tau,y} = 1.3$mm and $\Delta_{\tau,u} = 12.7$mm.

The response comparison for 10 beam systems (Table 3-7), shows a very good agreement between the finite element analysis and the proposed model's results. For the static response, the average standard deviation for all the beam systems is $D_{Pd} = -7.78\%$. For the pseudostatic response, the standard deviation is $D_{Pd} = -5.02\%$; Figure 3-14 shows the response comparison for the same beam length and axial restraint conditions of a double span W21x73 beam system. The WUF-B systems appear to be less sensitive to variations in beam length than their RBS counterparts. The detailed analysis results and the complete set of figures comparing the model and the numerical analysis responses are included in Appendix A.

Table 3-7: Beam L/D ratio information for the parametric FE validation exercise

	W21x68	W21x73	W24x62	W24x76
L = 4.5 m	8.4	8.3	7.5	-
L = 6.0 m	11.2	11.1	10.0	9.9
L = 9.0 m	-	16.7	14.9	14.8

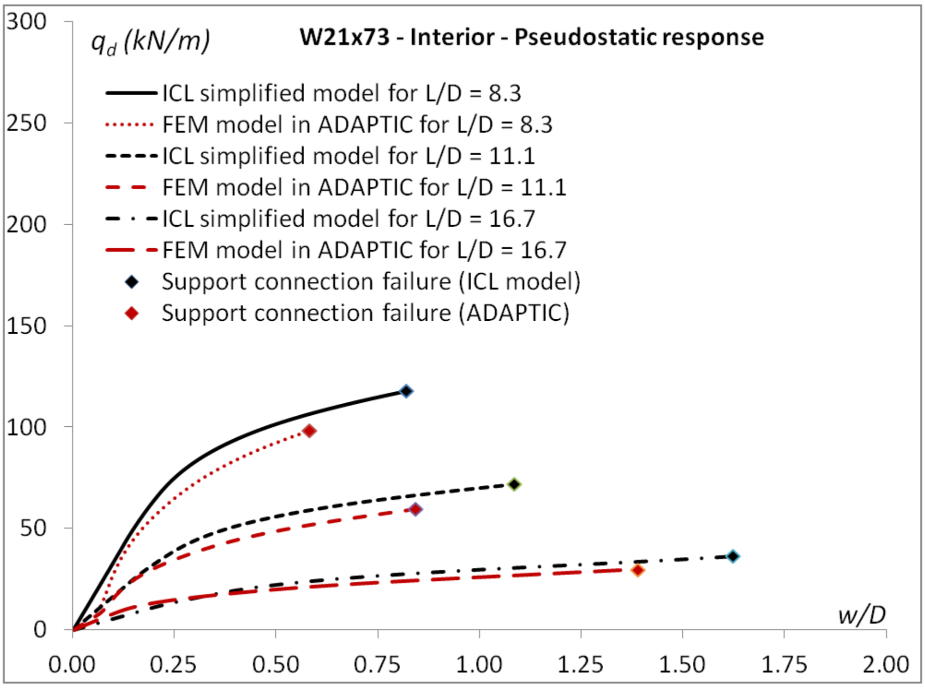


Figure 3-14: Pseudostatic response comparison for the W21x73 axially restrained beam system

Ultimate ductility and capacity

Both approaches suggest that decreasing the length of the beam has a negative effect on the maximum rotational capacity of the connection. Potential explanations are that:

i. Shear effects are much higher for shorter beams, which affects the deformation capacity of the beam flanges.

ii. Fully rigid connections require unrealistically high component deformation capacities to achieve larger rotations.

Thus, it is unlikely that a short (L/D < 10) beam with fully welded unreinforced connections will have the necessary joint ductility to allow entering the tensile catenary phase. The agreement on the failure point suggests an overestimation of ductility for beam systems. However, comparison against other numerical studies in the next section suggests the opposite.

Response form

The system initially enters the transient catenary phase, where the flanges are predominantly loaded in bending moment (Figure 3-15a, Figure 3-15b). Past certain levels of rotational deformation of the connections, the system enters the tensile catenary phase, during which the bending moment decreases while tensile axial forces take over, as shown Figure 3-15c.

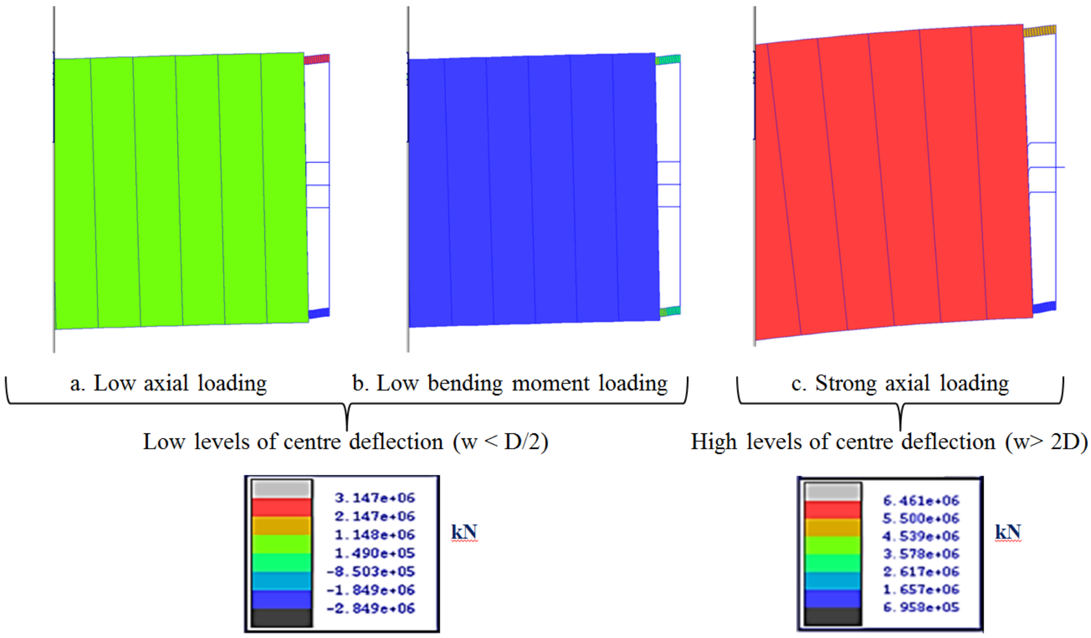


Figure 3-15: Axial loading for varying levels of centre deflection (W21x73, L=6m)

The critical element of the finite element model, for which the largest strains are observed, is in all cases adjacent to the welding region of the top flange, within the region of the weld access hole. The strains were calculated based on the 1st Gauss integration point at the top of an element just outside the welding region. However, the failure point estimate is expected to be conservative when compared to results from the micro-model and experimental studies discussed next, as the ADAPTIC model considers a perfect material behaviour.

Agreement in general is very close, while the difference in the elastic phase (less pronounced "knee") is attributed to the reduced initial stiffness of the flanges in tension. On the other hand, the constant post-yielding stiffness of the component manages to partially offset the reduction in rigidity due to the spread of inelasticity.

Behaviour of individual components

Additional finite element models were constructed to examine the influence of the bolts:

i. The bolts were completely removed (no web connection, just welded flanges).

ii. The bolts were considered to act only in tension (no 2D resistance).

Results showed that the contribution of the bolts is relatively low, i.e. between 4% and 8% towards the overall resistance throughout the response. They work mainly in tension as the relationship between loading in the horizontal and the vertical direction was found to be 50:1. This contrasts with partially restrained bolted endplate connections, where considering the two-dimensional behaviour of the bolts is expected to be more beneficial. Strain rate effects were not considered.

The important difference between the stiffness in the beam section and the welded flanges makes the free flanges bend and “twist” as the centre connection deflection increases. Figure 3-16 illustrates the final shape of the four top and bottom flanges of a support and a centre connection in a regular (symmetrical) beam system for large displacements (elements have been enlarged for demonstration purposes). Modelling this localised rotation of the flanges with a component model is very challenging. The localised bending and “twisting” of the flanges means that after their yielding, the welded edges are loaded under combined bending moment, shear and tension as the system progresses in the tensile catenary phase, reducing their deformation capacity prior to rupture.

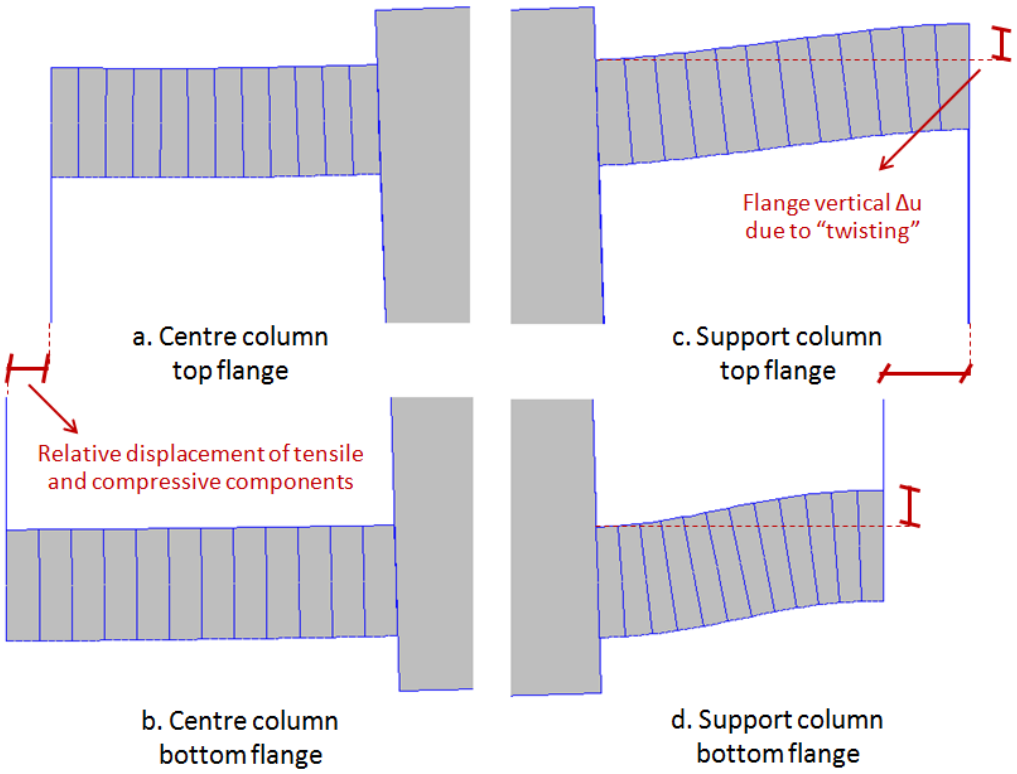


Figure 3-16: Details of the WUF-B connection FE model during the response

3.5.4.2 Verification against independent numerical studies

According to Figure 3-17 and in Figure 3-18, there is a very good agreement between the ICL method and the numerical solution of Khandelwal and El-Tawil (Khandelwal and El-Tawil, 2007), both in the form and the general order of magnitude of the response.

However, the agreement on the failure point is less helpful. The micromechanical model's prediction of ultimate deflection differs more than 100% for the two sections examined in the numerical study ($w_{max,W27x102}$ = 1190 mm and $w_{max,W30x124}$ = 570 mm). The authors attribute this to the geometrical differences between the two sections: the W30x124 is by 11% deeper and by 5% wider than the W27x102. On the other hand, the ICL model's prediction, which is based on the ultimate strain capacity of the flange in tension, appears to be less sensitive to changes in the beam section size; it differs by about 12%.

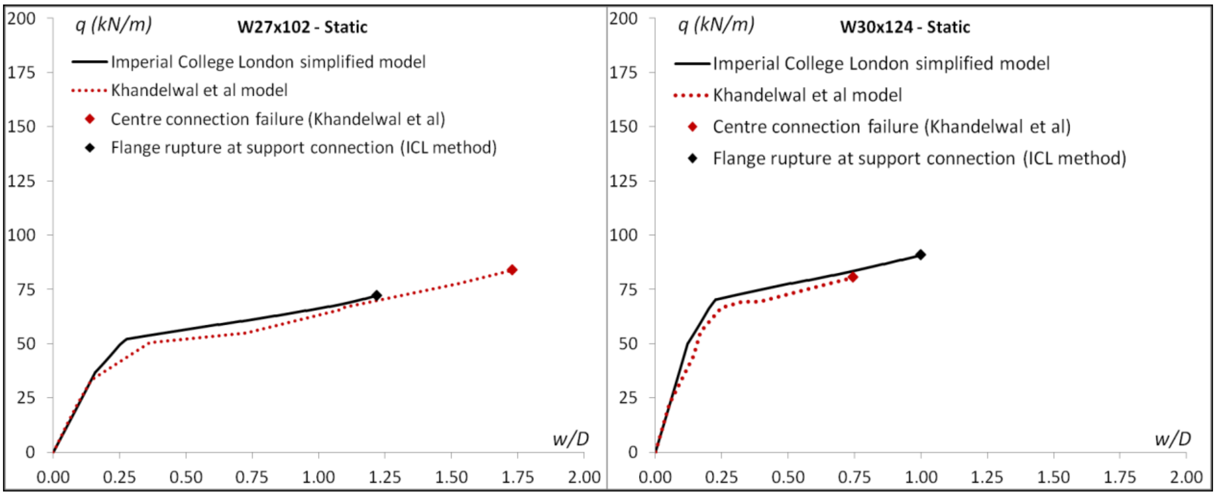


Figure 3-17: Comparison of the static response with the model of Khandelwal et al

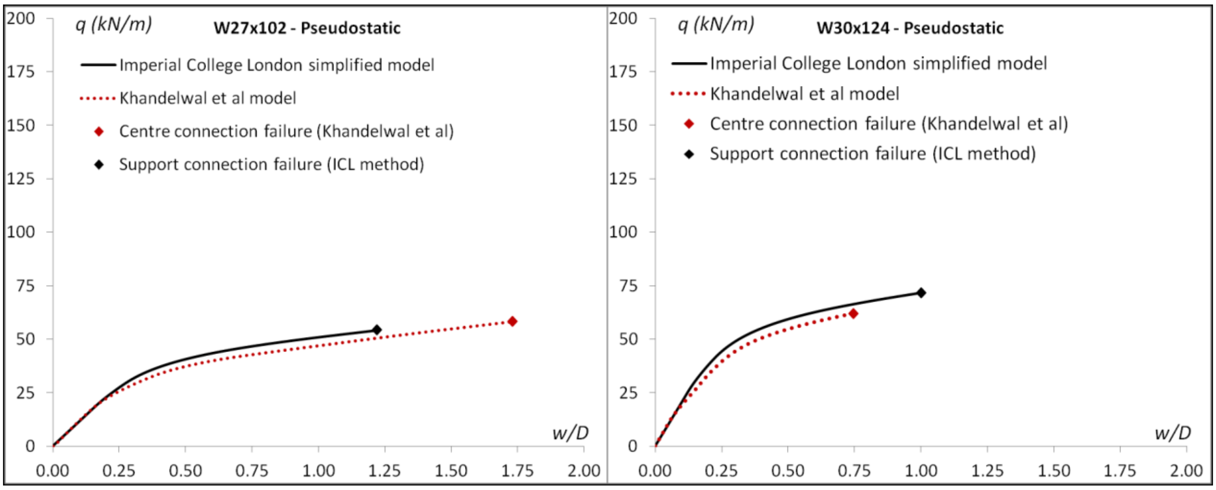


Figure 3-18: Comparison of the pseudostatic response with the model of Khandelwal et al

3.5.4.3 Verification against experimental results

The comparison between the NIST experimental results and those obtained with the ICL simplified model is illustrated in Figure 3-19. The general form of the response and the ultimate ductility are very close. The ICL model estimates a higher system resistance throughout the response (approx. +7%) and a higher ultimate capacity (approx. +20%).

In the NIST experiments, failure is triggered by local buckling of the top flanges of the beams near the centre column for a centre point deflection of w = 0.78·D, followed by successive shear failure of the bottom and middle bolts connecting the beam web to a shear tab at the centre column at w = 0.85·D and ultimately, fracture of the bottom flange near the weld access hole at w = 0.92·D. The ICL model predicts that bolt shear will occur for a similar level of displacement (w = 0.77·D) but that the beam system will retain its ability to withstand further loading until the fracture of the flange in tension (w = 0.99·D) at the support connection. Inelastic local bucking is not detected during the response because the development of important catenary forces offsets the load at the flanges in compression.

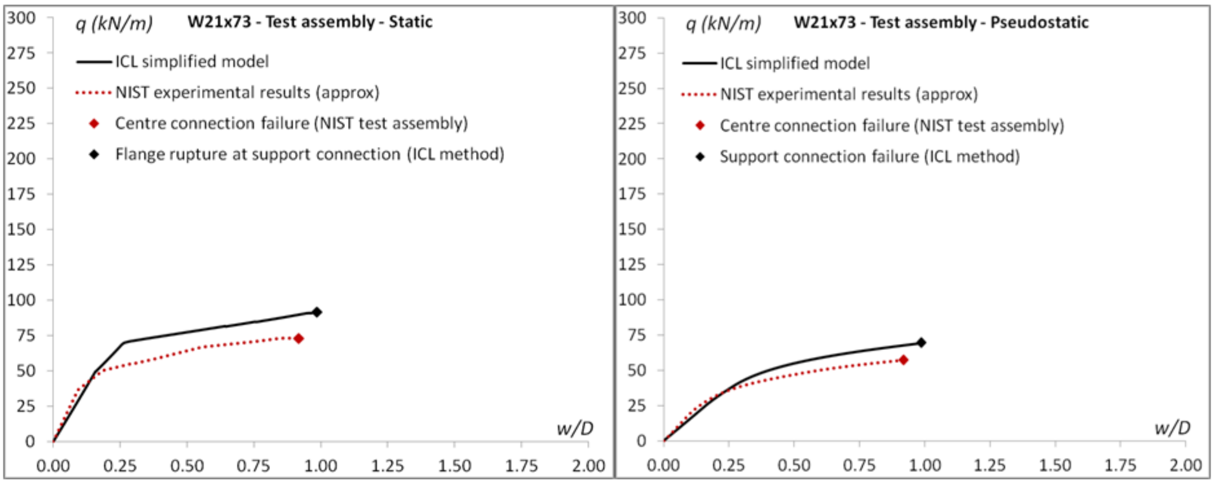


Figure 3-19: Beam system response comparison between the ICL model and the NIST experiment

3.6 Welded connections reinforced with cover plates

3.6.1 Introduction

Reinforcing the previous type of connections with rectangular cover plates, fillet welded to the flanges, offers certain key advantages for seismic resistant steel construction, including a highly ductile response and lower cost compared to other reinforcement options. The cross-sectional area of the cover plates is chosen based the criterion of allowing the region of the connection at the face of the column to remain elastic under the maximum bending moment and shear forces developed by the fully yielded and strain hardened beam. Research has shown (Engelhardt and Sabol, 1998, Engelhardt M. D. et al., 1996) that very thick or long cover plates should be examined with care, as they might increase the triaxial stress state at the column face, as well as that welding practices should be as stringent as possible to avoid brittle failures.

The main sources of uncertainty are the variation in the input values of material properties, which can result in significant differences in the prediction of the connection's performance, as well as the exact stresses and their maximum acceptable level for "essentially elastic" behaviour" of the connection in the region at the end of the coverplate.

3.6.2 Solution approach and component model

The modelling approach, failure modes and critical components are similar to those of the unreinforced connection, while the initial stiffness is calculated by considering a series of two springs, one for the unreinforced flange and the other for the reinforced, as shown in Figure 3-20. The equivalent component for both regions is calculated based on the gauge length of $5.65 \cdot \sqrt{A_o}$ (A_o is the original cross section area) as suggested in clause 3.2.2. of EC3 (EN 1993-1-1, 2005). The post-yielding properties of the unreinforced region control the behaviour, as the reinforced region remains in the elastic phase. Similar to the RBS arrangement, using the connection model within a beam system requires considering the axis of rotation of the connection at the beginning of the reinforced region instead of the end.

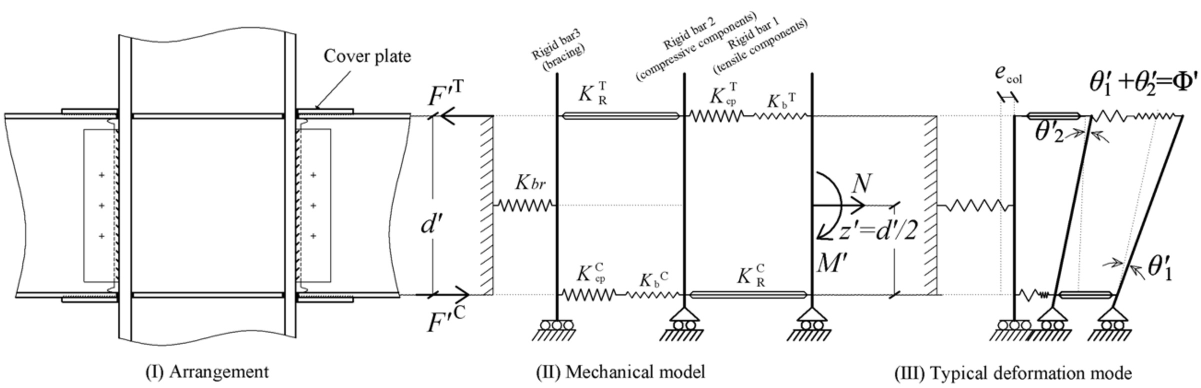


Figure 3-20: Welded unreinforced flange-bolted (WUF-B) connection spring model

3.6.3 Behaviour and failure modes

Parametric tests show that if the column web and flange are not reinforced, rotational capacity depends on the interplay between the capacity of the three types of components: compressive, tensile and shear; yielding of either reduces the loading in the other, until one of the components surpasses its deformation capacity. Reinforcing the connection with welded coverplates, continuity and doubler plates moves the critical region of the connection to the beam, at the end of the coverplates.

3.6.4 Validation of the welded reinforced with coverplates connection model

3.6.4.1 Verification against ADAPTIC analysis

Comparison with numerical models for the same beam systems of Table 3-7 has shown a good agreement (average deviation < 8%) in the response.

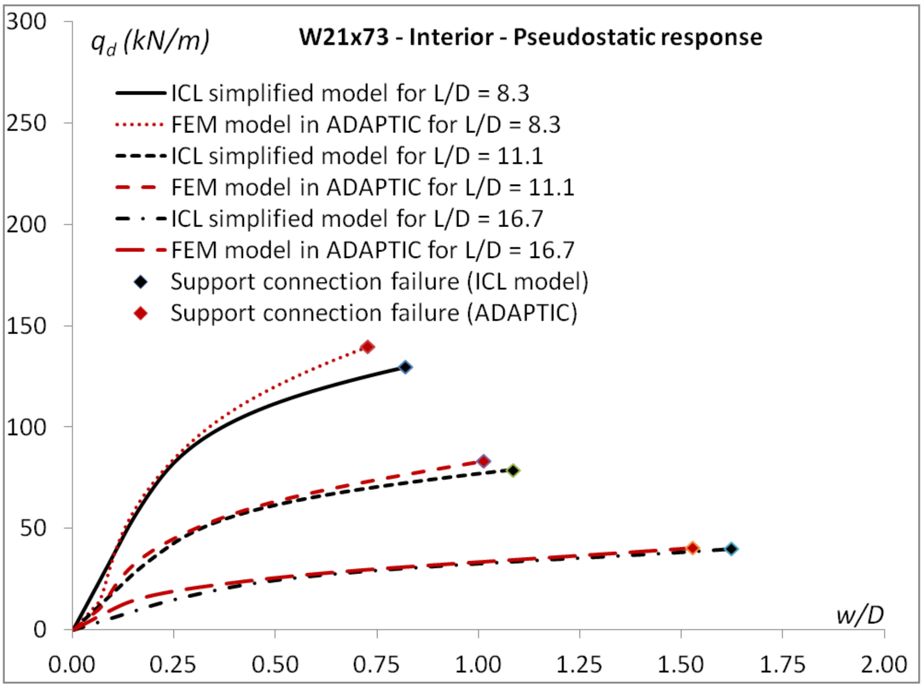
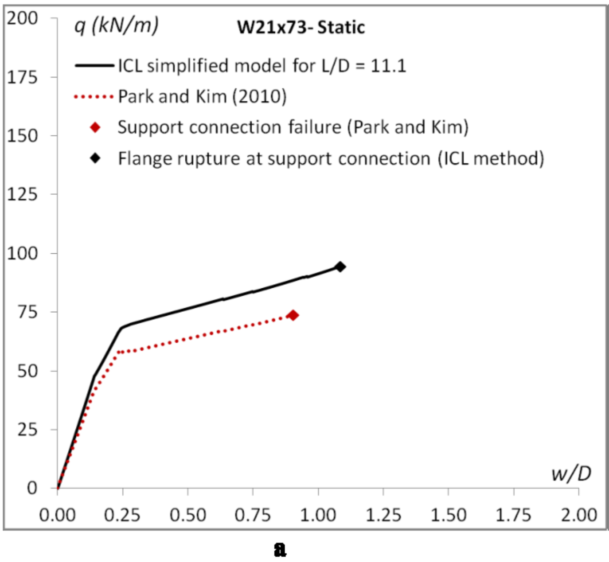


Figure 3-21: Pseudostatic response comparison for the W21x73 axially restrained beam system

3.6.4.2 Validation against independent numerical studies

Park and Kim (Park and Kim, 2010) studied the behaviour of beam systems with unreinforced, coverplated and RBS connections. Their results showed that although the WCF-B connection is the strongest one, the ductility of the RBS connection helps it provide the highest resistance. The findings of the fragility analysis also suggested that although uncertainties in material properties influence the initial response of coverplated connections less, it is likely that the RBS connections are more reliable.

According to Figure 3-22, there is a good agreement between the ICL method and the numerical solution in terms of the response form. However, the fragility analysis offers a much more conservative evaluation of the ultimate capacity, which can be attributed to the GSA guidelines' (GSA, 2003) load factor based analysis procedure that was employed for the study. As previously noted in Section 2.2.4, this approach offers a conservative evaluation.



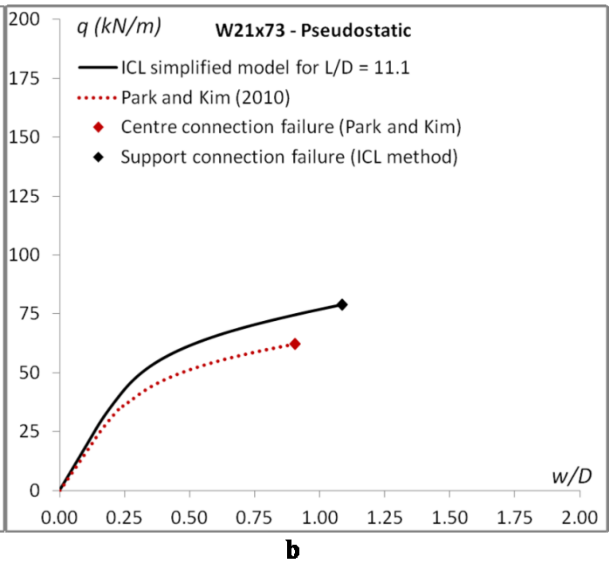


Figure 3-22: Comparison of the static and pseudostatic response with the model of Park and Kim (2010)

3.7 Summary and conclusions

Three types of popular welded moment resisting connections were modelled using the Component Method and appropriate failure modes and criteria for approximating their performance under large rotations were introduced. This allows modelling the behaviour of individual beam systems with these connections under progressive collapse loading conditions.

Based on an analytical solution that allows taking into account the combined effect of axial forces and bending moment loading in the connections, the models for reduced beam section, welded unreinforced and welded reinforced connections can be used to examine the behaviour of assemblies following loss of a column.

Modelling the behaviour of this type of connections is greatly dependent on material properties, which might not always allow for explicit simulation of experimental tests. It does, however, allow examining the controlling components and basic aspects of behaviour.

For equal beam depths and flanges sizes, results showed that the RBS arrangement is the most ductile. Welded unreinforced connections have almost half the rotational capacity, which makes them unlikely to have the necessary ductility for the beam system to enter the tensile catenary phase when resisting a progressive collapse scenario. In progressive collapse, coverplated connections are expected to have a relatively superior behaviour against other types of reinforced connections, based on the fact that the reinforcing component does not have a critical brittle failure mode like local buckling, in which haunches can be critical. This model also demonstrated the best agreement with the finite element verification analysis.

The use of these models opens an important number of future research possibilities, including running parametric tests for identifying the critical components and their influence on the ultimate rotational capacity under complex loading conditions, studying the relationship between seismic provisions and robustness, examining the ability of moment resisting frames to withstand progressive collapse and comparing the merits of alternative connection designs based on their performance against progressive collapse.

Chapter 4

Representation of bare steel irregular beam systems static nonlinear response following column removal

4.1 Introduction

As noted in Section 2.4**,** one of the advantages of the Imperial College London Method is that it accounts for dynamic effects without the need for a detailed dynamic analysis; instead, these effects are defined using a simplified energy-equivalence approach directly from the nonlinear static response. This response can be obtained by any type of analysis: either by detailed finite element models, which are the most accurate, or by simplified analytical "hand calculation" methods, which provide significant advantages in terms of streamlining the process and making it simpler, while still taking into account the essential features of performance (gross deformations, material and geometric nonlinearity and the development of compressive arching and tensile catenary action in the presence of axial restraint).

This approach of the ICL Method can be applied at any level of structural idealisation, such as the individual beam systems, floor grillage systems or frame bays. For regular structures and loads, the behaviour of the directly affected subsystem plays a crucial role in the ability of the frame to withstand progressive collapse. In order to be able to determine the behaviour of the floor grillage from the responses of the comprising beams, it is necessary to accurately represent the individual beam nonlinear static response following column removal. This is principally associated with the behaviour of the connections. As the system undergoes very large deflections in order to arrest progressive collapse, the connections are loaded under extreme conditions and the corresponding gross deformations are typically well beyond the design limits. Furthermore, the presence of components of the surrounding structure may provide axial restraint to the beams, leading to the development of very strong axial forces.

As mentioned in the introduction of the previous chapter, appropriate simplified models are available for simple connections (Oosterhof and Nethercot, 2014), partial strength (Stylianidis and Nethercot, 2009, Stylianidis, 2011) connections and moment resisting full strength connections (Vidalis and Nethercot, 2013a).

A solution for regular beam systems satisfying the double-span condition has been developed at Imperial College London (Stylianidis, 2011), providing an analytical method for predicting the nonlinear static response following removal of an intermediate column (Figure 4-1). By applying reasonable simplifications and assumptions, the model permits the interaction between beam deflection (**w**) due to lateral loading (**q**) with beam bending, support axial deformation (**Δs**) as well as rotation (**Φ**) of the connections to be analysed. It considers a reduced cracked stiffness (EI') for the region of the hogging bending moment, in order to accommodate composite beams. The model is simplified by assuming a fixed point of inflexion at the midpoint of the span. In the mechanical spring model of Figure 3-2, connection rotations are derived from the level of bending moment and axial load interaction from the relationship of equation 3.1.

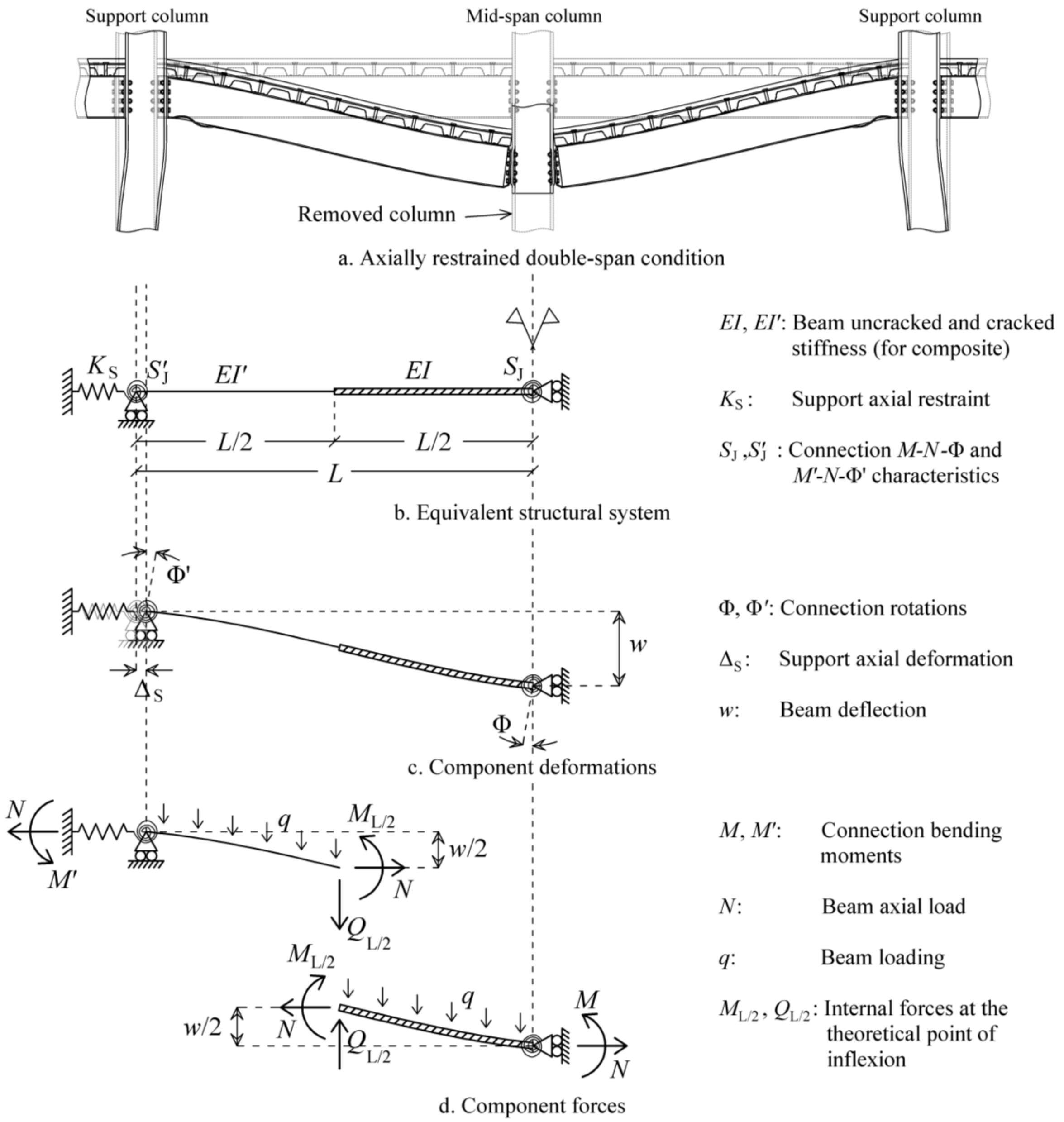


Figure 4-1: Representation of beam response following column loss (Stylianidis, 2011)

The solution by Stylianidis (Figure 4-1) describes the behaviour of a “regular” beam system, which satisfies the double-span condition and has the following properties:

i. The two support connections are identical.
ii. The two connections at the point of the removed column are identical.
iii. The length of the beams at each side of the removed column is equal.

Subsequently, due to the symmetry of the system in Figure 4-1a with respect to the centreline of the removed column, the response is equal for the comprising beam systems, which allows consideration of only half of the system. This condition significantly reduces the complexity of the “hand calculation” method. Although study of regular systems is primordial in understanding the mechanisms of progressive collapse, framed structures often also employ “irregular” beam systems. Some examples include:

i. Architectural irregularities; for example, external bays are usually shorter.
ii. Presence of reinforced concrete walls for fire protection, services’ infrastructure or lateral stability requirements.
iii. Frame action; for example, moment resisting frames have full strength connections in their perimeter but simple connections in the interior of the frame.

Thus, expansion of the analytical solution will allow predicting the response of those systems.

4.2 Beam structural model

The beam structural model for response representation is illustrated in Figure 4-2: part a and b show an irregular beam system subject to removal of the intermediate column and part c depicts how the boundary conditions of the two single beams, left and right of the point of the lost column, can be represented. The subscript “L” is used for the properties and forces corresponding to the beam system at the left of the lost column, while “R” is used for the equivalent properties and forces of the system at the right. An apostrophe is used for support connections. For example, $S'_{J,L}$ is the rotational stiffness of the support connection of the beam at the left side of the lost column. The rotation of the intermediate (removed) column web panel due to the different connection properties on each side is assumed negligible; hence the deflection (w) is equal for both sides. For non-stiffened connections, the axial stiffness of the column panel’s tensile and compressive zones at the intermediate connections should be considered. The horizontal dimension of the intermediate (removed) column web panel is considered as negligible in the structural model. However, its effective length is used when determining the column panel component properties.

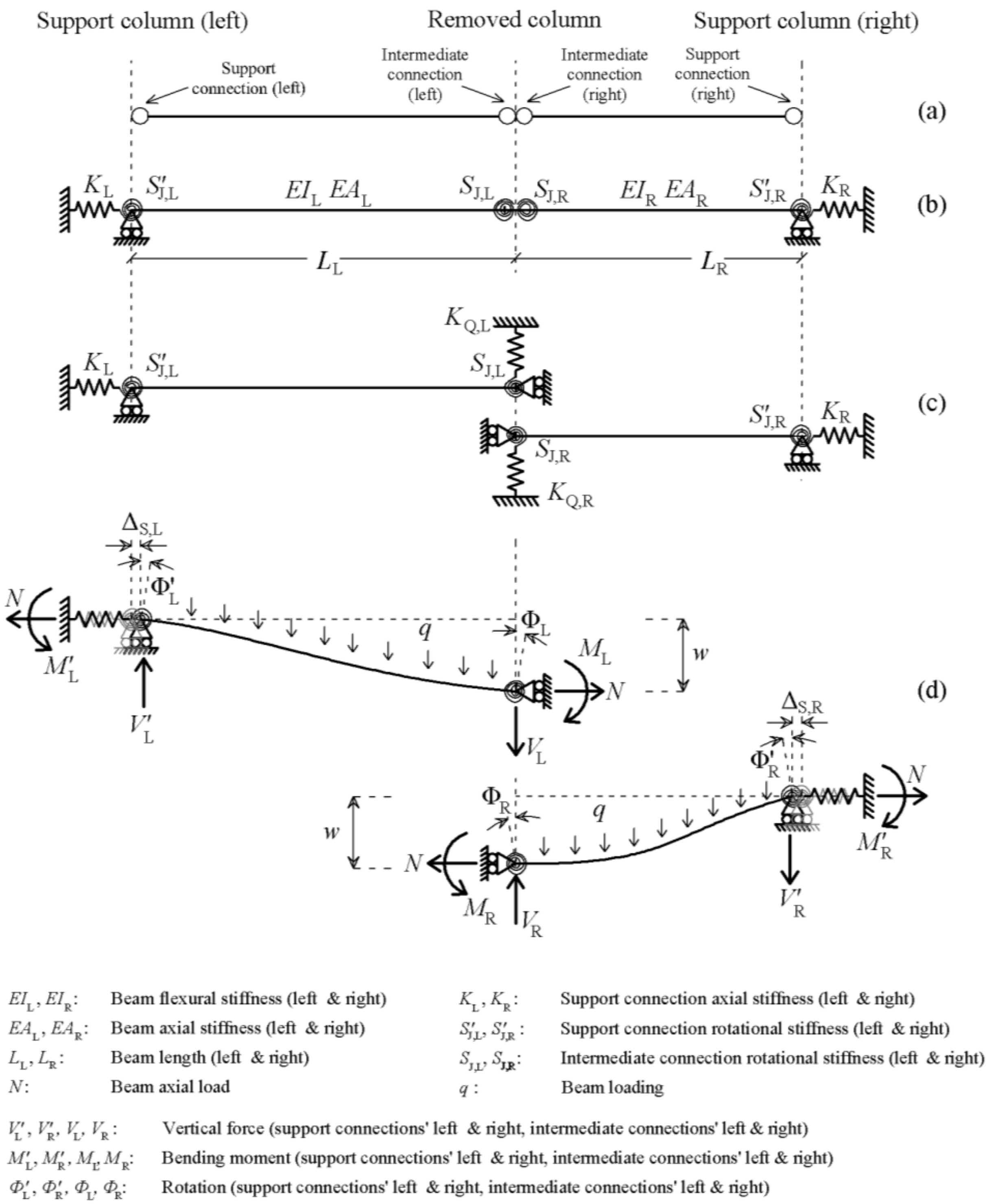


Figure 4-2: Structural model and equilibrium of the irregular beam system arrangement

4.3 Analytical representation of the irregular beam system performance

The analytical solution for the nonlinear static response following column loss requires extrapolating an explicit expression between the beam gravity load and deflection (q-w). This can be done by solving the equations derived from:

i. The equilibrium of the system, illustrated in Figure 4-2d, which provides a relationship between the beam axial and gravity loads: Section 4.3.1.

ii. Application of the stiffness method, which provides a relationship between the four connection bending moments, the beam lateral loading and the geometry and structural properties of the system: Section 4.3.2.

iii. The compatibility equation of the system, which provides a relationship between the deformation values of the system, its properties and the axial loading: Section 4.3.3.

4.3.1 Connection bending moments

The beam flexural behaviour defines the connection bending moments. The length and the second moment of area of beam at the right hand side of the lost column can be expressed as:

$$L_R = \lambda \cdot L_L \tag{4.1a}$$

$$I_R = \eta \cdot I_L \tag{4.1b}$$

In bare steel construction, beam stiffness is equal in both the hogging and sagging bending moment region. Application of the stiffness method for the structural system of Figure 4-2 leads to the nodal forces for each element of the equivalent clamped structure in Figure 4.3.

Figure 4-4 illustrates the basic displacement modes. The sum of the nodal forces caused by the deformations of the released structure corresponding to these modes, together with the equivalent nodal forces of the clamped structure, define the total nodal forces of each section. For the left beam system, the stiffness-displacement-loads matrix will be:

$$\begin{bmatrix} -M'_L \\ -M_L \\ Q_L \end{bmatrix} = \begin{bmatrix} \frac{4EI_L}{L_L} & \frac{2EI_L}{L_L} & -\frac{6EI_L}{L_L^2} \\ \frac{2EI_L}{L_L} & \frac{2EI_L}{L_L} & -\frac{6EI_L}{L_L^2} \\ -\frac{6EI_L}{L_L^2} & -\frac{6EI_L}{L_L^2} & -\frac{12EI_L}{L_L^3} \end{bmatrix} \cdot \begin{bmatrix} \Phi'_L \\ \Phi_L \\ w_L \end{bmatrix} + \begin{bmatrix} -\frac{q\,L_L^2}{12} \\ \frac{q\,L_L^2}{12} \\ -\frac{q\,L_L}{12} \end{bmatrix} \tag{4.2a}$$

For the right beam system, the equivalent equation is:

$$\begin{bmatrix} M_R \\ -M'_R \\ -Q_R \end{bmatrix} = \begin{bmatrix} \frac{4EI_R}{L_R} & \frac{2EI_R}{L_R} & \frac{6EI_R}{L_R^2} \\ \frac{2EI_R}{L_R} & \frac{4EI_R}{L_R} & \frac{6EI_R}{L_R^2} \\ \frac{6EI_R}{L_R^2} & \frac{6EI_R}{L_R^2} & \frac{12EI_R}{L_R^3} \end{bmatrix} \cdot \begin{bmatrix} \Phi_R \\ \Phi'_R \\ w_R \end{bmatrix} + \begin{bmatrix} -\frac{q\,L_R^2}{12} \\ \frac{q\,L_R^2}{12} \\ -\frac{q\,L_R}{12} \end{bmatrix} \tag{4.2b}$$

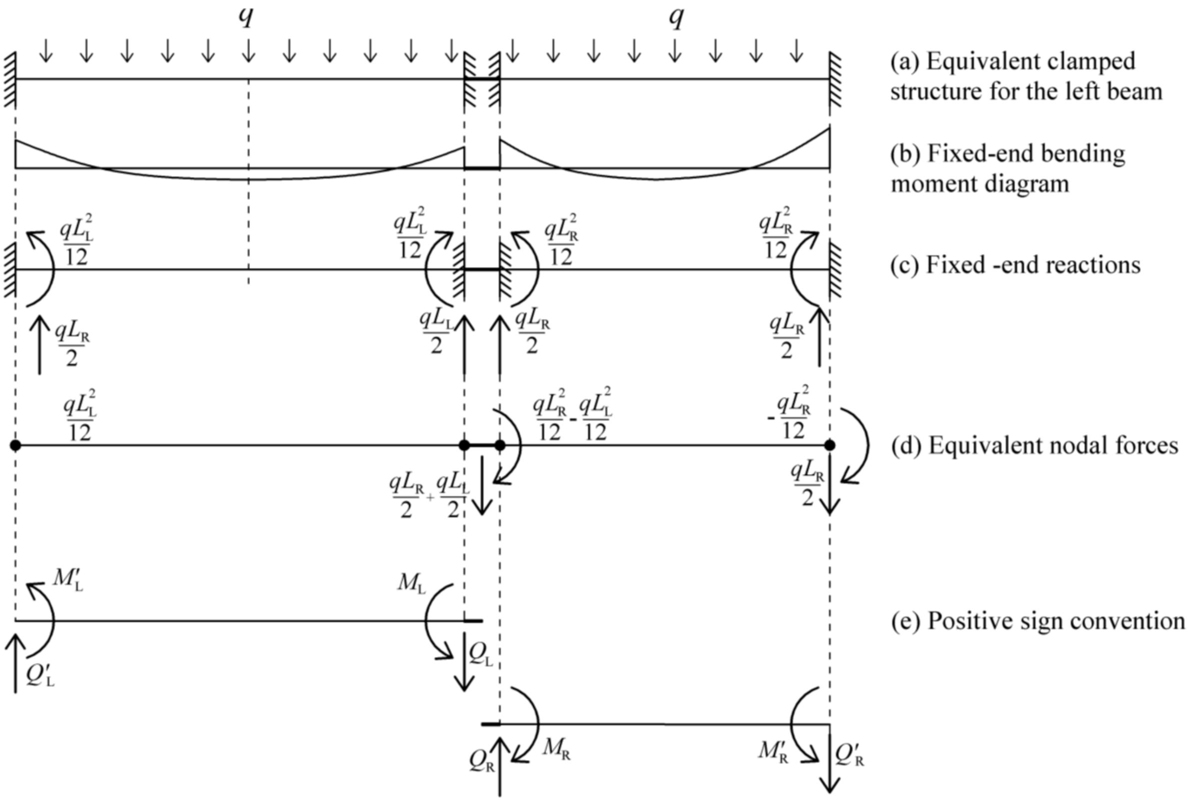


Figure 4-3: Nodal forces of the clamped structure

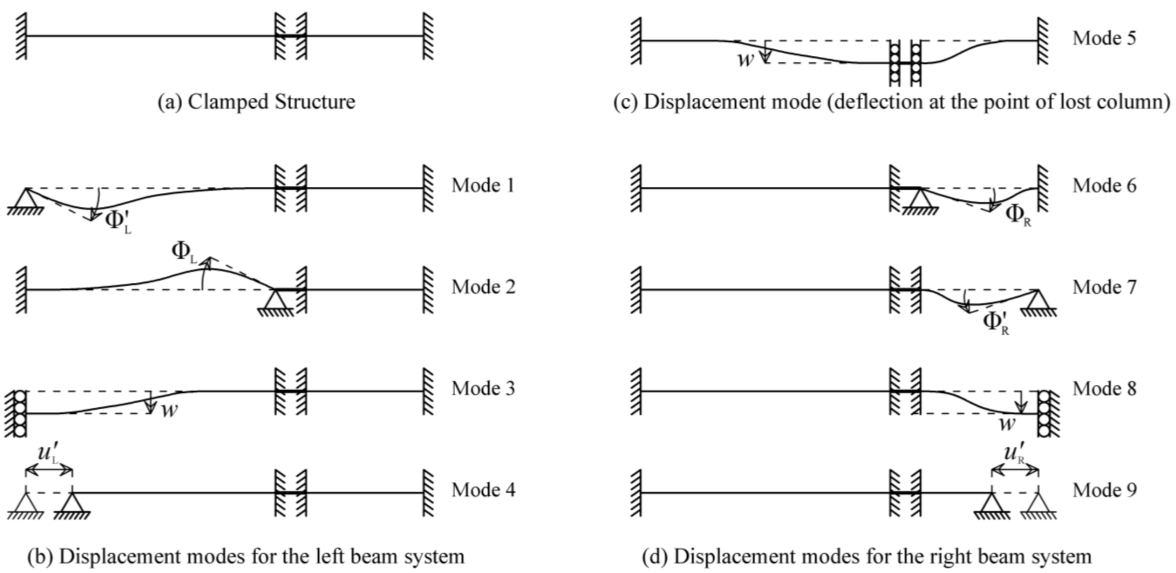


Figure 4-4: Displacement modes of the released structure

Using equations 4.1a, 4.1b, 4.2a and 4.2b, the following relationships can be extracted:

$$M_L' = -\frac{4EI_L}{L_L}\Phi_L' - \frac{2EI_L}{L_L}\Phi_L + \frac{6EI_L}{L_L^2}w + \frac{qL_L^2}{12} \tag{4.3a}$$

$$M_L = -\frac{2EI_L}{L_L}\Phi_L' - \frac{4EI_L}{L_L}\Phi_L + \frac{6EI_L}{L_L^2}w - \frac{qL_L^2}{12} \tag{4.3b}$$

$$Q_L = -\frac{6EI_L}{L_L^2}\Phi_L' - \frac{6EI_L}{L_L^2}\Phi_L + \frac{12EI_L}{L_L^3}w - \frac{qL_L}{2} \tag{4.3c}$$

$$M_R = \frac{4\eta EI_L}{\lambda L_L}\Phi_R + \frac{2\eta EI_L}{\lambda L_L}\Phi'_R + \frac{6\eta EI_L}{\lambda^2 L_L^2}w - \frac{q\lambda^2 L_L^2}{12} \tag{4.3d}$$

$$M_R' = -\frac{2\eta EI_L}{\lambda L_L}\Phi_R - \frac{4\eta EI_L}{\lambda L_L}\Phi'_R - \frac{6\eta EI_L}{\lambda^2 L_L^2}w - \frac{q\lambda^2 L_L^2}{12} \tag{4.3e}$$

$$Q_R = -\frac{6\eta EI_L}{\lambda^2 L_L^2}\Phi_R - \frac{6\eta EI_L}{\lambda^2 L_L^2}\Phi'_R - \frac{12\eta EI_L}{\lambda^3 L_L^3}w + \frac{q\lambda L_L}{2} \tag{4.3f}$$

Based on the equilibrium of the system in Figure 4-2d, V_L is equal to the sum of the resistance due to bending and the resistance from the vertical spring (similar for the right side):

$$V_L' = Q_L + Q_{s,L} \tag{4.4a}$$

$$V_R' = Q_R + Q_{s,R} \tag{4.4b}$$

At the same time, the vertical springs at the intermediate point represent the contribution of the bending resistance of the beam of the opposite side:

$$Q_{s,L} = Q_R \tag{4.5a}$$

$$Q_{s,R} = Q_L \tag{4.5b}$$

Inserting Equation 4.5a to 4.4a and 4.5b to 4.4b leads to the following relationship for the shear resistance at the intermediate connections:

$$V_L = V_R = Q_L = Q_R = V_m \tag{4.6}$$

The equilibrium of the system (Figure 4-2d) allows deducing the following additional expressions, which also incorporate the effects of the beam axial load:

$$M_L' = \frac{qL_L^2}{2} + V_m L_L - M_L - Nw \tag{4.7a}$$

$$M_L = -\frac{qL_L^2}{2} + Q_L^{'} L_L - M'_L - Nw \tag{4.7b}$$

$$M_R = \frac{qL_R^2}{2} + Q_R^{'} L_R - M'_R + Nw \tag{4.7c}$$

$$M_R^{'} = -\frac{qL_R^2}{2} + V_m L_R + M_R + Nw \tag{4.7d}$$

$$Q_L^{'} + Q_R^{'} = qL_L + qL_R = qL_L\,(1+\lambda) \tag{4.7e}$$

By substituting M_L, Q_L and Q_R, from the expressions 4.3b, 4.3c and 4.3f respectively, into 4.7a, M'_L can be expressed as follows:

$$M'_L = -\frac{4EI_L}{L_L}\Phi'_L - \frac{2EI_L}{L_L}\Phi_L - \frac{6\eta EI_L}{\lambda^2 L_L}\Phi'_R - \frac{(6\lambda+1)L_L^2}{12}q + \left[\frac{6EI_L}{L_L^2} - \frac{12\eta EI_L}{\lambda^2 L_L^2}\right]w - Nw \tag{4.8}$$

By substituting M_R, Q_L and Q_R, from the expressions 4.3d, 4.3c and 4.3f respectively, into 4.7d, M'_R can be expressed as follows:

$$\begin{aligned} M'_R = & -\frac{4\eta EI_L}{\lambda L_L}\Phi'_R - \frac{2\eta EI_L}{\lambda L_L}\Phi_L - \frac{6\lambda EI_L}{L_L}\Phi'_L - \frac{6\lambda EI_L}{L_L}\Phi_L - \frac{\lambda L_L^2(6+\lambda)}{12}q \\ & + \left[\frac{12\lambda EI_L}{L_L^2} - \frac{6\eta EI_L}{\lambda^2 L_L^2}\right]w + N\,w \end{aligned} \tag{4.9}$$

By substituting M'_L from Equation 4.8 to 4.3a, Φ'_R can be expressed as a function of Φ_R:

$$\Phi_R^{'} = -\Phi_R + \frac{\lambda^2 L_L^3(3\lambda+1)}{36\eta EI_L}q - \frac{2}{\lambda L_L}w - \frac{\lambda^2 L_L w}{6\eta EI_L}N \tag{4.10a}$$

$$\Phi_R = -\Phi'_R + \frac{\lambda^2 L_L^3(3\lambda+1)}{36\eta EI_L}q - \frac{2}{\lambda L_L}w - \frac{\lambda^2 L_L w}{6\eta EI_L}N \tag{4.10b}$$

By substituting M'_R from equation 4.9 to 4.3e, Φ'_L can be expressed as a function of Φ_L:

$$\Phi_L^{'} = -\Phi_L - \frac{L_L^3}{12EI_L}q + \frac{2}{L_L}w + \frac{L_L w}{6\lambda EI_L}N \tag{4.11a}$$

$$\Phi_L = -\Phi'_L - \frac{L_L^3}{12EI_L}q + \frac{2}{L_L}w + \frac{L_L w}{6\lambda EI_L}N \tag{4.11b}$$

By substituting the expressions of Φ'_R, Φ_R, Φ'_L and Φ_L from 4.10a, 4.10b, 4.11a and 4.11b respectively, into the expressions 4.3a, 4.3b, 4.3d and 4.3e for the connection bending moment can be written in the following way:

$$M'_R = -\frac{2\eta EI_L}{\lambda L_L}\Phi'_R - \frac{\lambda L_L^2(3\lambda+2)}{36}q - \frac{2\eta EI_L}{\lambda^2 L_L^2}w + \frac{\lambda w}{3}N \quad (4.12a)$$

$$M_R = \frac{2\eta EI_L}{\lambda L_L}\Phi_R + \frac{\lambda L_L^2(3\lambda+2)}{36}q + \frac{2\eta EI_L}{\lambda^2 L_L^2}w - \frac{\lambda w}{3}N \quad (4.12b)$$

$$M'_L = -\frac{2EI_L}{L_L}\Phi'_L + \frac{L_L^2}{4}q + \frac{2EI_L}{L_L^2}w - \frac{w}{3\lambda}N \quad (4.12c)$$

$$M_L = -\frac{2EI_L}{L_L}\Phi_L + \frac{L_L^2}{12}q + \frac{2EI_L}{L_L^2}w - \frac{w}{3\lambda}N \quad (4.12d)$$

In order to simplify the previous equations, the following coefficients are introduced:

$$K_1 = \frac{2EI_L}{L_L} \quad (4.13a)$$

$$K_2 = \frac{\lambda L_L^2(3\lambda+2)}{36} \quad (4.13b)$$

$$K_3 = \frac{2EI_L}{{L_L}^2} \quad (4.13c)$$

$$K_4 = \frac{w}{3} \quad (4.13d)$$

$$K_5 = \frac{L_L^2}{12} \quad (4.13e)$$

Thus, equations 4.12a to 4.12d can be now written as:

$$M'_R = -\frac{\eta}{\lambda}K_1\Phi'_R - K_2 q - \frac{\eta}{\lambda^2}K_3 w + \lambda K_4 N \quad (4.14a)$$

$$M_R = \frac{\eta}{\lambda}K_1\Phi_R + K_2 q + \frac{\eta}{\lambda^2}K_3 w - \lambda K_4 N \quad (4.14b)$$

$$M'_L = -K_1\Phi'_L + 3K_5 q + K_3 w - \frac{K_4}{\lambda}N \quad (4.14c)$$

$$M_L = -K_1\Phi_L + K_5 q + K_3 w - \frac{K_4}{\lambda}N \quad (4.14d)$$

The moment – axial load – rotation relationships of the connections can also be defined as:

$$\Phi'_R = M'_R \cdot \alpha'_{1,R} + N'_{Z,R} \cdot \beta'_{1,R} - \gamma'_{1,R} \quad (4.15a)$$

$$\Phi_R = M_R \cdot \alpha_{1,R} + N_{Z,R} \cdot \beta_{1,R} - \gamma_{1,R} \quad (4.15b)$$

$$\Phi'_L = M'_L \cdot \alpha'_{1,L} + N'_{Z,L} \cdot \beta'_{1,L} - \gamma'_{1,L} \quad (4.15c)$$

$$\Phi_L = M_L \cdot \alpha_{1,L} + N_{Z,L} \cdot \beta_{1,L} - \gamma_{1,L} \quad (4.15d)$$

The calculation of coefficients $\alpha'_{1,R}$, $\alpha_{1,R}$, $\alpha'_{1,L}$, $\alpha_{1,L}$, $\beta'_{1,R}$, $\beta_{1,R}$, $\beta'_{1,L}$ and $\beta_{1,L}$ is based on the analytical solution for progressive collapse connection modelling proposed by Stylianidis (Stylianidis and Nethercot, 2009); more detailed information is available in Chapter 3 of his Thesis (Stylianidis, 2011).

By substituting the expressions of Φ'_R, Φ_R, Φ'_L and Φ_L from 4.10a, 4.10b, 4.11a and 4.11b into the above four expressions, the connection bending moment can be expressed as a function of beam deflection, beam gravity load, beam axial load and the dimensions and properties of the system, in the following way:

$$M'_R = \lambda'_R N + \mu'_R q + V'_R \quad (4.16a)$$

$$M_R = \lambda_R N + \mu_R q + V_R \quad (4.16b)$$

$$M'_L = \lambda'_L N + \mu'_L q + V'_L \quad (4.16c)$$

$$M_L = \lambda_L N + \mu_L q + V_L \quad (4.16d)$$

Where:

$$\lambda'_R = \frac{\lambda K_4 - \left(\eta/\lambda\right) K_1 \cdot z'_R \cdot \beta'_{1,R}}{1 + \left(\eta/\lambda\right) K_1 \cdot \alpha'_{1,R}} \quad (4.17a)$$

$$\lambda_R = \frac{-\lambda K_4 + \left(\eta/\lambda\right) K_1 \cdot z_R \cdot \beta_{1,R}}{1 - \left(\eta/\lambda\right) K_1 \cdot \alpha_{1,R}} \quad (4.17b)$$

$$\mu'_R = -K_2 \quad (4.17c)$$

$$\mu_R = K_2 \quad (4.17d)$$

$$V'_R = -\frac{\left(\eta/\lambda^2\right) K_3 \cdot w - \left(\eta/\lambda\right) K_1 \cdot \gamma'_{1,R}}{1 + \left(\eta/\lambda\right) K_1 \cdot \alpha'_{1,R}} \quad (4.17e)$$

$$V_R = \frac{\left(\eta/\lambda^2\right) K_3 \cdot w - \left(\eta/\lambda\right) K_1 \cdot \gamma_{1,R}}{1 - \left(\eta/\lambda\right) K_1 \cdot \alpha_{1,R}} \quad (4.17f)$$

$$\lambda'_L = -\frac{K_1 \cdot \beta'_{1,L} \cdot z'_L + \left(K_4/\lambda\right)}{1 + K_1 \cdot \alpha'_{1,L}} \quad (4.17g)$$

$$\mu_L' = \frac{3K_5}{1+K_1 \cdot \alpha_{1,L}'} \tag{4.17h}$$

$$V_L' = \frac{K_3 + K_1 \cdot \gamma_{1,L}'}{1+ K_1 \cdot \alpha_{1,L}'} \tag{4.17i}$$

$$V_L = \frac{K_3 + K_1 \cdot \gamma_{1,L}}{1+ K_1 \cdot \alpha_{1,L}} \tag{4.17j}$$

$$\lambda_L = - \frac{K_1 \cdot \beta_{1,L} \cdot z_L + \left({K_4}/{\lambda} \right)}{1+ K_1 \cdot \alpha_{1,L}} \tag{4.17k}$$

$$\mu_L = \frac{K_5}{1+K_1 \cdot \alpha_{1,L}} \tag{4.17l}$$

4.3.2 Beam axial load

By solving the system equilibrium equations 4.7b and 4.7c in respect to Q'_L and Q'_R and by substituting their expressions into expression 4.7e, the beam axial load can be expressed as:

$$N = \frac{3L_L^2}{(1+\lambda)\ w} q - \frac{2}{\lambda(1-\lambda)\ w}\left[M_R'+M_R\right] - \frac{2}{(1-\lambda)w}\left[M_L'+M_L\right] \tag{4.18}$$

Furthermore, by using the connection bending moment expressions of 4.14a, 4.14b, 4.14c and 4.14d, the beam axial load can be written as a function of the system properties, intermediate point vertical deflection and beam load:

$$N = \mu_N \cdot q + V_N \tag{4.19}$$

Where:

$$\mu_N = \frac{{3}/{2} \cdot \lambda \cdot (1-\lambda) \cdot L_L \cdot w + 2 \cdot (\mu_R' + \mu_R + \lambda \cdot \mu_L' + \lambda \cdot \mu_L)}{\lambda \cdot (1-\lambda) \cdot w - 2 \cdot (\lambda_R' + \lambda_R + \lambda \cdot \lambda_L' + \lambda \cdot \lambda_L)} \tag{4.20a}$$

$$V_N = \frac{2 \cdot (V_R' + V_R + \lambda \cdot V_L' + \lambda \cdot V_L)}{\lambda \cdot (1-\lambda) \cdot w - 2 \cdot (\lambda_R' + \lambda_R + \lambda \cdot \lambda_L' + \lambda \cdot \lambda_L)} \tag{4.20b}$$

4.3.3 Component axial deformations

Axial deformation, defined along the beam neutral axis, can be estimated by a second-order approximation (Izzuddin, 2005).

$$\Delta = L_i \left[1 - \cos\frac{w}{L_i}\right] \tag{4.21a}$$

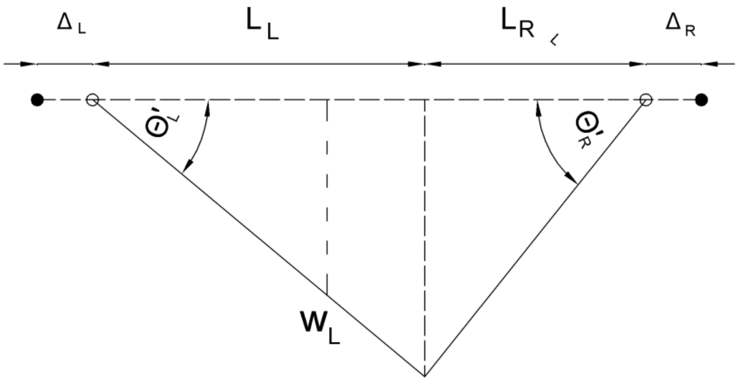


Figure 4-5: Second-order approximation of the beam axial deformation

Based on Figure 4-5, the axial displacement of the two beam subsystems can consist of the following component deformations:

$$\frac{w^2}{2L_L} = \Delta_L = u'_L + u_L + \Delta_L^\alpha + \Delta_{b,L} \tag{4.22a}$$

$$\frac{w^2}{2L_R} = \Delta_R = u'_R + u_R + \Delta_R^\alpha + \Delta_{b,R} \tag{4.22b}$$

The connection moment – axial load – axial displacement relationships can be expressed as:

$$u'_L = M'_L \cdot \alpha'_{2,L} + N \cdot z'_L \cdot \beta'_{2,L} - \gamma'_{L,2} \tag{4.23a}$$

$$u_L = M_L \cdot \alpha_{2,L} + N \cdot z_L \cdot \beta_{2,L} - \gamma_{L,2} \tag{4.23b}$$

$$u'_R = M'_R \cdot \alpha'_{2,R} + N \cdot z'_R \cdot \beta'_{2,R} - \gamma'_{R,2} \tag{4.23c}$$

$$u_R = M_R \cdot \alpha_{2,R} + N \cdot z_R \cdot \beta_{2,R} - \gamma_{R,2} \tag{4.23d}$$

The beam axial displacement (Δ_b) due to bending for bare steel beams is negligible. The axial displacement of the beam and of the axial support can be expressed as:

$$\Delta_L^\alpha = \frac{N}{K_L^\alpha} \tag{4.24a}$$

$$\Delta_R^\alpha = \frac{N}{K_R^\alpha} \tag{4.24b}$$

Where:

$$K_L^{\alpha}= \left[\frac{L_L}{EA_L}+\frac{1}{K_L}\right]^{-1} \quad (4.25a)$$

$$K_R^{\alpha}= \left[\frac{L_R}{EA_R}+\frac{1}{K_R}\right]^{-1} \quad (4.25b)$$

4.3.4 Nonlinear static load-deflection relationship

By substituting the component axial deformations as defined by the equation set of 4.23, 4.24 and 4.25, the connection bending moments and axial load can be expressed as follows:

$$\frac{w^2}{2L_L}= \alpha'_{2,L}M'_L + \alpha_{2,L}M_L + \left[z'_L\beta'_{2,L}+z_L\beta_{2,L}+\frac{1}{K_L^{\alpha}}\right]\cdot N - \left(\gamma'_{2,L}+\gamma_{2,L}\right) \quad (4.26a)$$

$$\frac{w^2}{2L_R}= \alpha'_{2,R}M'_R + \alpha_{2,R}M_R + \left[z'_R\beta'_{2,R}+z_R\beta_{2,R}+\frac{1}{K_R^{\alpha}}\right]\cdot N - \left(\gamma'_{2,R}+\gamma_{2,R}\right) \quad (4.26b)$$

By substituting the expressions of M'_L, M_L and N from 4.16a, 4.16b and 4.19 respectively, the following relationship between the beam gravity load and the beam deflection (q-w) can be obtained:

$$q = \frac{w^2 - 2L_L\cdot(K_6V_N - K_8)}{2L_L\cdot\left(K_7 + K_6\mu_N\right)} \quad (4.27)$$

Where:

$$K_6 = Z'_L\cdot\beta'_{2,L}+Z_L\cdot\beta_{2,L}+\frac{1}{K_L^{\alpha}}+\alpha'_{2,L}\cdot\lambda'_L+\alpha_{2,L}\cdot\lambda_L \quad (4.28a)$$

$$K_7 = \alpha'_{2,L}\cdot\mu'_L+\alpha_{2,L}\cdot\mu_L \quad (4.28b)$$

$$K_8 = \alpha'_{2,L}\cdot V'_L+\alpha_{2,L}\cdot V_L-\gamma'_{2,L}-\gamma_{2,L} \quad (4.28b)$$

Equation 4.27 can be used to explicitly model the non-linear beam static response following column loss, while the component forces and deformations can be derived from the corresponding equations developed in this section. The absence of axial restraint can be accommodated with a very small value of axial stiffness K_L or K_R in Expression 4.25.

Similar to regular systems (Stylianidis, 2011), this set of equations can be programmed into a spreadsheet software platform. Thus, the performance of irregular beam systems can be predicted on a step-by-step analysis by gradually increasing the deflection at the point of the removed column up until failure.

4.4 Numerical verification of the analytical relationship with ADAPTIC

In order to verify the validity of the analytical solution for irregular beam systems, detailed numerical models were constructed using the nonlinear structural analysis program ADAPTIC (Izzuddin, 1991). The current exercise essentially follows the verification study of Section 3.3.4.1 since the exemplar systems considered herein employ the connection arrangements as well as the column and beam sections adopted in that study. Therefore, the beam system models are essentially based on the assumptions made for the connection models (see general assumptions in Section 3.2.2 as well as assumptions for each type of connection in Sections 3.3.2, 3.4.2 and 3.5.2).

4.4.1 Beam arrangements and modelling

Table 4-1 summarises the arrangements that were considered. The models used for full-strength resisting connections are based on those in Chapter 3; bare steel welded unreinforced flange-bolted and RBS (reduction 50%, Figure 3-5) connections are modelled based on the assumptions, simplifications and failure criteria presented in the previous chapter.

Table 4-1: Beam arrangement constituent length and connection properties

No	Beam length (m)		Beam sections		Connection arrangement			
	Left	Right	Left	Right	Left		Right	
					Support	Centre	Centre	Support
1	6.0	4.5	W27x102	W27x102	RBS	RBS	RBS	RBS
2	6.0	6.0	W27x102	W27x102	RBS	RBS	RBS	RBS
3	6.0	9.0	W27x102	W27x102	RBS	RBS	RBS	RBS
4	6.0	6.0	W27x102	W21x73	RBS	RBS	RBS	RBS
5	6.0	6.0	W27x102	W24x94	RBS	RBS	RBS	RBS
6	6.0	6.0	W27x102	W27x102	RBS	RBS	RBS	WUF-B
7	6.0	6.0	W27x102	W27x102	RBS	WUF-B	WUF-B	RBS
8	6.0	6.0	W27x102	W27x102	RBS	WUF-B	WUF-B	WUF-B

4.4.2 Comparison of response load-deflection curves

For the arrangements of Table 4-1, the load-deflection curves obtained by the numerical ADAPTIC analysis and the analytical solution (calculated with the help of a spreadsheet, i.e. MS Excel) are compared in Figures 4-6, 4-7 and 4-8. Specifications for each arrangement are given in the corresponding figures.

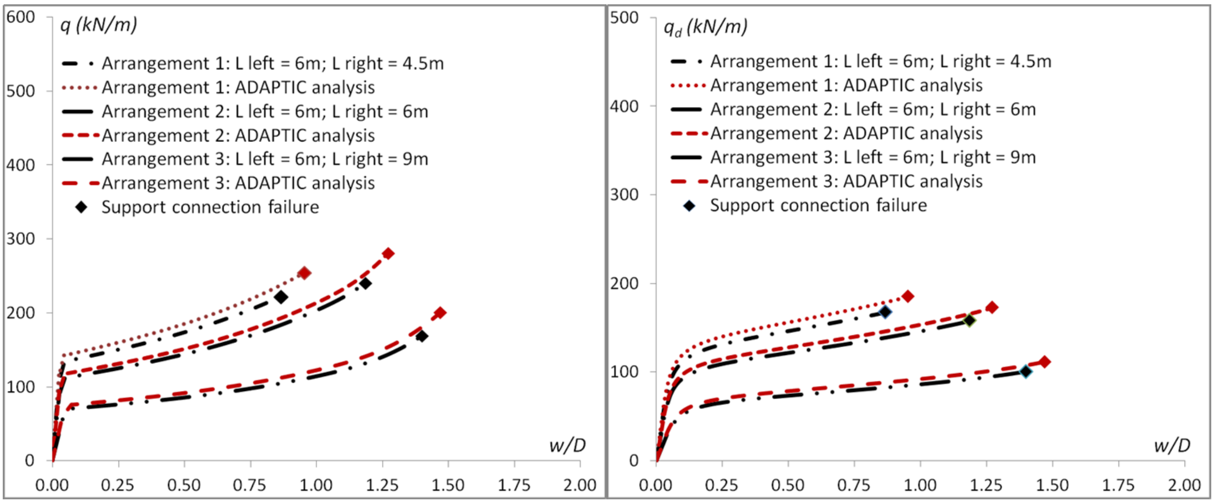


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Figure 4-6: Static and pseudostatic responses of axially restrained irregular beam systems with W27x102 sections and RBS connections

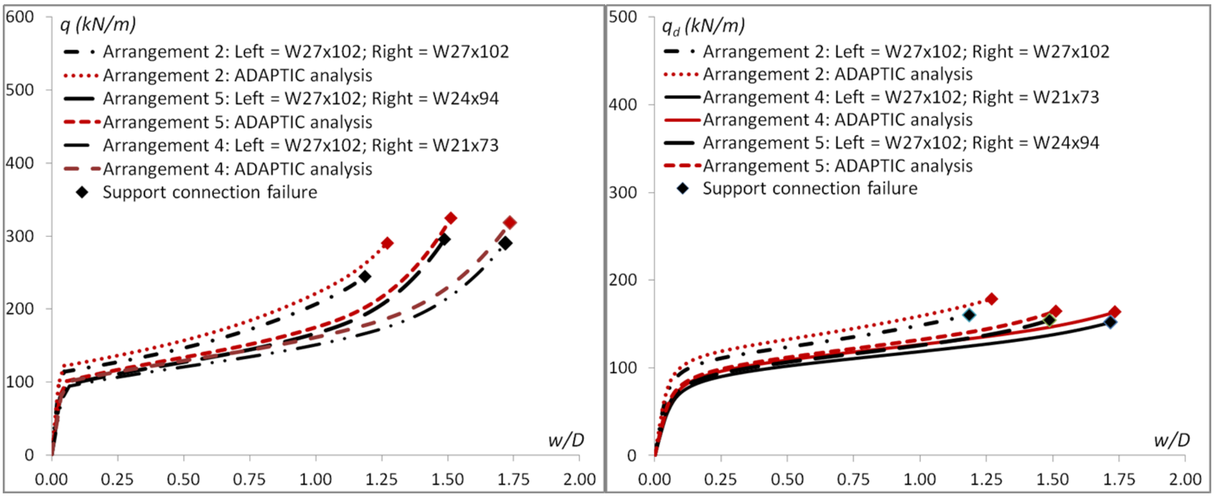

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Figure 4-7: Static and pseudostatic responses of axially restrained irregular beam systems with beam length equal to 6m and RBS connections

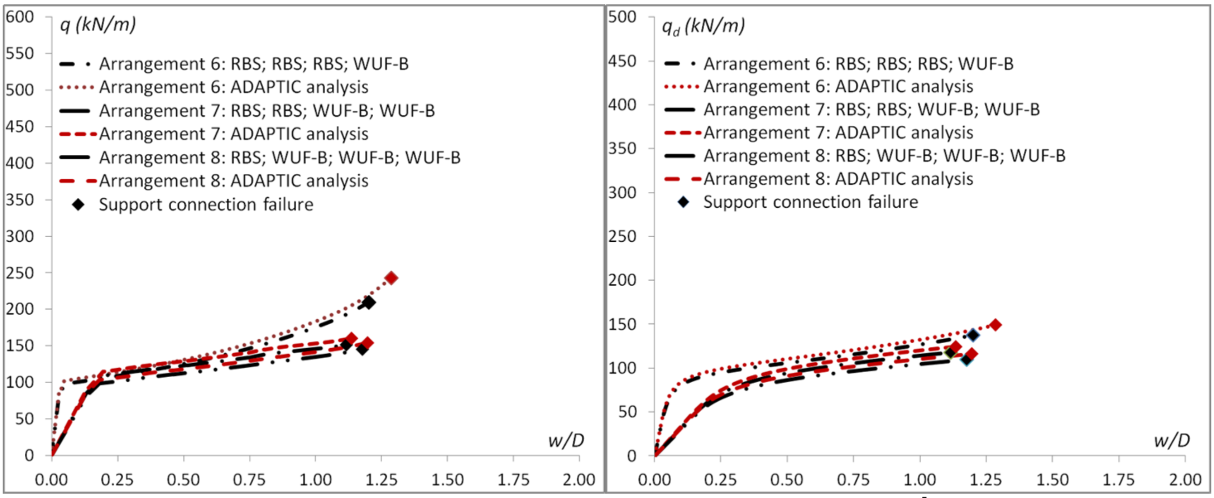

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Figure 4-8: Static and pseudostatic responses of axially restrained irregular beam systems with W27x102 sections and beam length equal to 6m

It is confirmed that the agreement between the results obtained from the two analysis methods is very good. In particular, both the response curves and the ultimate capacities of the systems are predicted with the very good accuracy by the analytical method as compared to the numerical results. However, verification of the solution was not possible in the case of irregular beam systems with very different sections or connection normalised properties.

Minor discrepancies are observed in some cases, in a manner consistent with the discrepancies observed for the welded connection models in the previous section, which largely represent the nonlinear behaviour of the beam elements in the vicinity of the connections due to high tensile forces and the spread of inelasticity. These effects are an inherent limitation of any hand-calculation method and can only be captured by the numerical models. On the other hand, verification of the solution for irregular beam systems with very different sections and with centre connections with large (> 40%) differences in stiffness has shown poor agreement because both conditions invalidate the assumption that the intermediate column web panel does not rotate.

4.5 Preliminary observations on irregular beam system behaviour

Although the aim of this study is not to exhaustively examine the behaviour of irregular beam systems, some general points based on the results of the validation exercise and the findings from the rest of the chapters can be made:

For irregular beam systems with different sections for each span:

i. For major axis connections to the removed column, the column web panel is likely to be critically loaded in shear.

ii. The beam with the lowest length to depth (L/D) ratio, given that all other properties are identical, is expected to bear most of the bending moment and axial force loading because of its relative higher stiffness.

For irregular beam systems with different connections at the supports:

i. More rigid connections are expected to bear an increased share of the bending moment.

ii. Simple connections, as long as they can transfer the shear loading effectively and can provide the necessary rotational ductility, are expected to fail for higher levels of deformation than the participating full strength connections.

iii. For edge and cantilever column removal scenarios, taking into account the low level of axial restraint provided at the support connection by the adjacent structure is expected to increase the total system capacity predicted by the analysis.

iv. The maximum capacity of beam systems connected to reinforced concrete cores, which are considered fully rigid and full-strength, is expected to be strongly influenced by the size and depth of the beams.

v. A connection's share of bending moment loading within the system will be proportional to its relative stiffness compared to other joints. Thus, the following priorities should be considered, depending on the stiffness of the connection:
   a. Most rigid connection's key property: strength
   b. Least rigid connection's key property: ductility

4.6 Summary and conclusions

This chapter presents an analytical method for the prediction of the nonlinear static response following column removal of a double span irregular beam system. By incorporating similar connection models to those developed in the previous chapter into an extended slope-deflection model, which accounts for the interplay between the beam and connection structural parameters at the various stages of the response, it is possible to capture the essential features of progressive collapse in an explicit manner. A verification exercise was performed by comparing the results obtained from detailed numerical models and from the analytical solution, which demonstrated very good agreement. The solution will be employed in chapter 7, in order to identify the impact of shortening the length of the last girder in a moment resisting frame on the response of the edge bay.

Preliminary examination of the behaviour of irregular beam systems shows that the interplay between the increased number of structural elements - compared to regular systems - may lead to different considerations for their behaviour. In general, the stronger and stiffer connections and the shorter and deeper beams of the system dominate the response. Future studies can reveal how these elements interact with another and which is the optimum design in order to resist progressive collapse.

Chapter 5

Improving progressive collapse resistance in simply designed steel and composite frames

5.1 Introduction

Explicit evaluation and improvement of a building's robustness requires taking into account all the main resistance mechanisms, employing realistic failure criteria and obtaining quantitative results based on a suitable level of structural idealisation. The Imperial College London Method has made significant progress towards establishing a soundly based analysis methodology for calculating and comparing the performance of different designs; recent advances have facilitated the execution of extensive parametric studies that are quick to run and thus able to provide comprehensive results. Section 5.2 discusses the parameters that influence the critical resistance mechanisms of regular axially restrained beam systems, the understanding of which is necessary for enhancing resistance.

Section 5.3 introduces a step-by-step methodology to determine the most efficient and practically applicable changes, making it possible to redesign the composite frame in a way that it will be sufficiently robust to cope with any sudden column removal scenario. The application of this methodology within the context of a detailed study is presented in the rest of the chapter in order to demonstrate each step. Moreover, the Imperial College London Method is employed herein to examine the robustness of a simplified version of the Cardington test frame (presented in Section 5.4) and to compare its performance with that of a bare steel equivalent frame. Both arrangements fail to provide the necessary resistance, with the bare steel being inherently less robust.

The parametric analysis results in Section 5.5 for the lower levels of structural idealisation help determine the physical components that limit the response. In addition, by referring to a similar case study (Blundell D. et al., 2010) on an identical frame with equivalent bare steel beam sections, Sections 5.6 and 5.8.3 highlight the differences in the behaviour of composite and bare steel frames. The comparison of responses reveals that different priorities might need to be considered for composite and for bare steel frames.

Using a simple process, Sections 5.8 and 5.9 suggest and justify changes in the connection design that (for this case study) enhance floor grillage maximum capacity by between 117%

and 155%. In order to align the study with construction practice, certain parameters of the frame, beams and connections are considered as native and unalterable. Thus, focus is on the realistically alterable parameters of the connections and how their handling can influence the beam and hence the grillage systems' pseudostatic response.

The comparison of the methodology in Section 5.10 with simply increasing tying capacity reveals that the latter does not have a direct and proportional effect on the frame's resistance. This leads on to consideration of how tying capacity might be used within a more informed context. Consequently, the process is identified as a method suitable for determining a more efficient and acceptable alternative design configuration, compared to simply increasing connection tensile or compressive resistance.

5.2 Beam system resistance action mechanisms

5.2.1 General

As mentioned in Chapter 2, the Imperial College London Method permits a significant reduction in the complexity of the multi-storey frame model since it makes the reasonable assumption that the response at the higher levels of structural idealization can be constructed from the response at the lower levels, which are the beam systems (Izzuddin B.A. et al., 2008). An example of a double span beam system is illustrated in Figure 4-1.

Following sudden column loss, the loading conditions at the connections evolve throughout the response of the system. More specifically, depending on the ratio between the bending moment and the applied axial load at the connections, different component types, such as compressive, shear and tensile, are activated. Previous work at Imperial College London (Stylianidis, 2011) has identified the connection component characteristics, deformations and limit states. Thus, for each different loading combination, the pseudostatic response of semi-continuous beam systems benefits from a series of non-linear resistance mechanisms, thanks to the presence of axial restraint at the boundary joints.

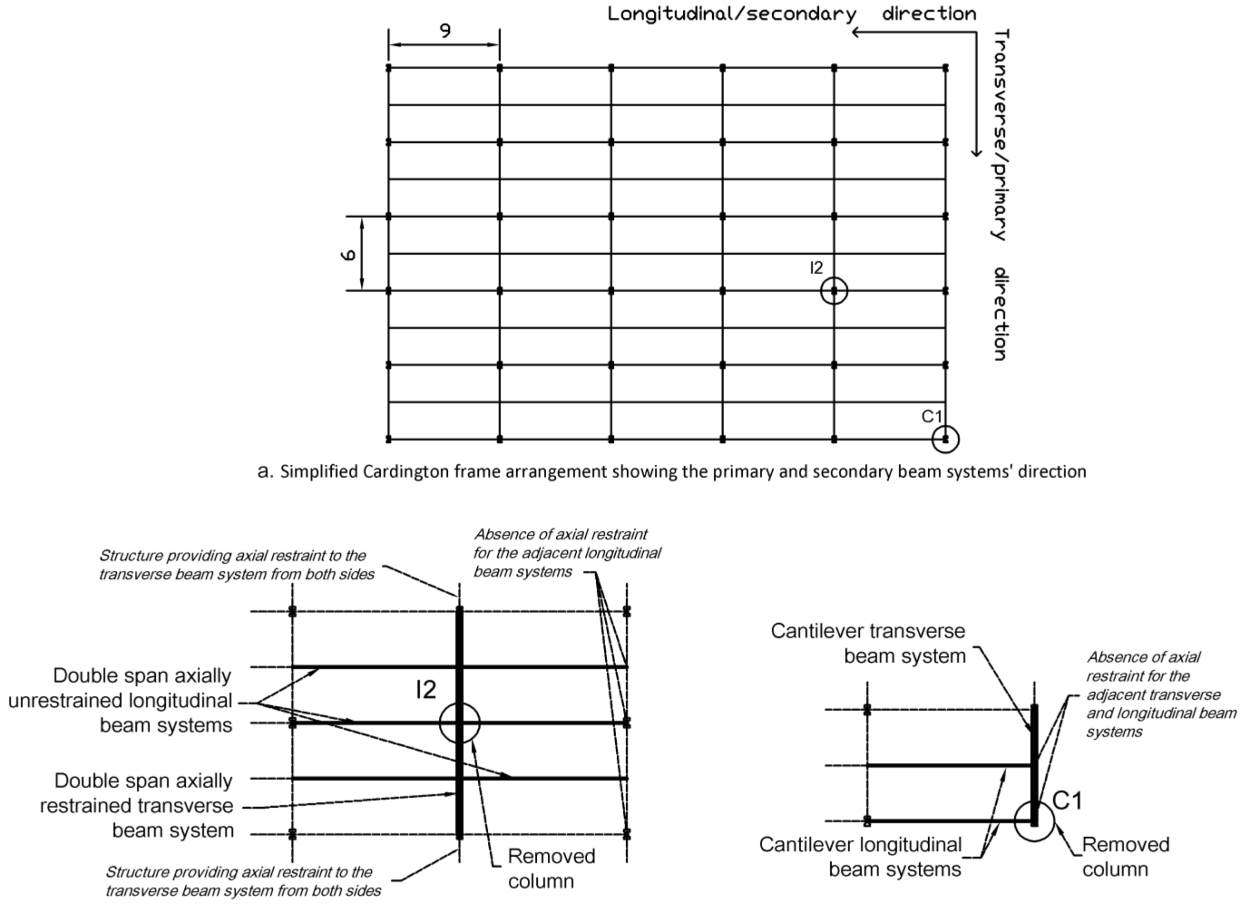
5.2.2 Influence of support axial restraint

According to the ICL Method, the response is governed by the rotational capacities of the connections, which depend on the relative resistance and stiffness of the tensile and compressive components. Nevertheless, this interplay is defined differently when the boundary joints are axially restrained (Stylianidis and Nethercot, 2009), which is usually determined by the position of the beam system in the frame.

The beam systems are categorised in Table 5-1 based on the presence of axial restraint and on whether or not they can be considered semi-continuous (or continuous) over a double span after the loss of the middle column. Figure 5-1b and 5-11c illustrate an example of the beam types considered for the loss of an internal and a corner column, along with the conditions of axial restraint for the beam systems constituting the corresponding floor grillage.

Table 5-1: Beam system types based on axial restraint conditions at the boundary joints

Beam system type	Axially restrained joints	Axially unrestrained joints
Single span		"Cantilever"
Double-span, semi-continuous	"Axially restrained"	"Axially unrestrained"



b. Axial restraint conditions for removal of column I2 (floor grillage constituent beam systems)

c. Axial restraint conditions for removal of column C1 (floor grillage constituent beam systems)

Figure 5-1: (a) Simplified Cardington frame arrangement showing (b) Axial restraint conditions for removal of column I2 (floor grillage constituent beam systems); (c) Axial restraint conditions for removal of column C1 (floor grillage constituent beam systems)

5.2.3 Response action phases

As described in Table 5-2, for low deflection levels, the system enters the compressive arching action phase and the connections are loaded under compressive axial forces and bending moment. For larger deflections, the connection axial load becomes tensile and the system enters the transient catenary phase, during which the connections are still under some bending moment loading. The internal tensile loading is applied at a lever arm distance from the connection rotational centre, generating an internal bending moment which at some stage cancels out the external bending moment loading. Hence, for substantial deflection levels, the system enters the pure tensile catenary action phase and the connections support the system with their tensile resistance, similar to a catenary. The ductility reserve of the system defines the critical action phase during which the system will fail.

The “deflection level” is measured as the beam system midspan vertical deformation over twice the beam depth (w/2D). It is influenced by the same parameters affecting the pseudostatic response of the system. More precisely, beam deflection (w) due to lateral loading (q) is associated with beam bending, support joint axial deformation (Δs) as well as with the rotation of the centre (Φ) and support (Φ’) connections of the double span beam.

Table 5-2: Connection loading conditions for each beam system resistance action phase

Beam system type	Combined bending & compressive arching	Combined bending & tensile effects	Pure catenary action
Simply supported axially restrained	No bending effects - carries lateral loading through compressive arching		Tensile forces – performance depends on tying capacity
Semi-continuous axially restrained	Bending effects - performance depends on connection stiffness and strength		Tensile forces – performance depends on tying capacity
Semi-continuous axially unrestrained	Bending effects only		

5.2.4 Failure criteria

Failure of the system essentially begins when the key column to beam connections exhaust their maximum rotational capacity, which corresponds to when their components reach their limiting deformation. After that point, the system significantly loses the ability to redistribute the loading (the contribution of the floor slab is ignored). Even if in some cases it is able to continue providing additional resistance, the approach used in this study does not account for that and thus can be considered as conservative, offering the lower bound for resistance to progressive collapse, which is the minimum expected pseudostatic capacity.

5.3 The Imperial College London redesigning methodology

The methodology developed at ICL can be used to determine - explicitly - how to improve the design of a structure based on:

i. Enhancing the response of the structural subsystems.

ii. Achieving the most constructive interaction of the subsystems, in order for them to be able to provide the additional pseudostatic resistance for withstanding collapse.

Although it can be applied to any type of frame or type of construction, the solution type may vary depending on the different nature of vulnerabilities and acceptably alterable parameters. The approach focuses on examining the beam and floor grillage systems' pseudostatic responses by performing a series of parametric studies on selected connection parameters. Using these results, it suggests and justifies changes in the connection design that will improve the system response. It is conducted in five phases, outlined below:

Initially, each beam system's pseudostatic response to sudden column loss is calculated, in this case using the simplified ICL Method. Based on the findings, it is possible to identify the basic resistance actions for each system (compressive arching, strain hardening phase, transient catenary or tensile catenary) and the weaknesses of the original configuration.

In the second phase, a series of parametric tests is conducted for each beam system and a maximum of three alternative connection design configurations are chosen based on the following criteria: most enhanced system ductility, most enhanced system capacity and the optimum combination of these two properties. Subsequently, the beam systems are categorized according to the critical resistance action both for their original as well as for their alternative configuration.

Moving on to the next level of structural idealisation (the grillage), a separate analysis for each column removal scenario is carried out and the responses are compared with those of the equivalent bare steel frame. Together with the conclusions from the previous phase, this highlights the advantages and the disadvantages of the composite frame compared to the bare steel arrangement. At the same time, when the demand in pseudostatic capacity is found to be higher than the supply, the corresponding grillage assemblies are examined in order to find out which beam systems limit the response and how. This investigation reveals whether poor performance is a result of insufficient ductility, capacity or a combination of both and whether this occurs at the beam system or floor grillage structural idealisation level. When possible, these weaknesses are quantitatively determined with a percentage ratio based on the projected increase required to assist the grillage meet the demand.

Consequently, this percentage ratio is used as a criterion for matching the weak floor grillages with the most appropriate alternative configurations based on the candidate's impact on increasing the system ductility and capacity. If more than one configuration is found to provide the necessary features, then the one closer to the original is chosen.

After that, the frame is considered with the new connection design and the grillage assemblies are examined in order to evaluate whether the proposed changes have been effective in ensuring the frame meets the demand for any potential column loss scenario.

5.4 Exemplar case study outline

The merits of the redesigning methodology are demonstrated with its application to a representative composite and its equivalent bare steel frame. The case study examines the simplified version of the 8-storey composite structure built at the BRE large scale test facility at Cardington UK, originally constructed to investigate the behaviour of modern composite structures subject to fire (British Steel, 1998). In order to simplify the process as well as to be able to examine the influence of certain parameters independently from any interplay with others, the following simplifications and assumptions are made:

- The frame arrangement is simplified from the layout of Figure 5-2 to that of Figure 5-1a: the cores are omitted, 2 extra bays are added in the transverse direction and a uniform beam length is employed.
- Loading is simplified to $g_k + 0.25*q_k / m^2$ for the area supported by the removed column, ignoring the facade loading caused by the cladding and any additional loading on the roof.
- The section used for all beams corresponds to that of the perimeter secondary beam; using a universal section will allow determining the influence of the beam length.
- All beam to column and beam to beam connections are assumed to be full depth end plates. The longitudinal beam connections are minor column axis while those for the transverse beams are major axis connections that include the effect of the shear deflection of the column web.
- A single section is used for both edge and internal columns.

- Beam flange stiffening using horizontal endplates welded on the beam flange of the support connection is considered at a theoretical level only and does not take into account any additional effects of such modifications, nor does it define the required dimensions and characteristics of such an endplate. Certain complications are ignored:
    - Differences in the flexural and axial stiffness of the beam section.
    - Differences in the position of the compression centre.
    - Variations in the lever arm of the tensile components.

The bare steel beam sections presented in Table 5-3 have been chosen with the criterion of maintaining an equivalent moment capacity $W_{b,Pl,Rd}$ and beam length to depth ratio (L/D) with the composite arrangement. Table 5-4 and Table 5-5 provide additional design information whereas Figure 5-2 and Figure 5-4 illustrate the composite and bare steel connections respectively. In an effort to replicate common construction practice, the case study examines changes in the realistically alterable design parameters presented in Table 5-6. In fact, changing the beam design or frame arrangement would in most cases be either incompatible with design provisions for other load cases, or impractical and expensive or unable to significantly influence the response.

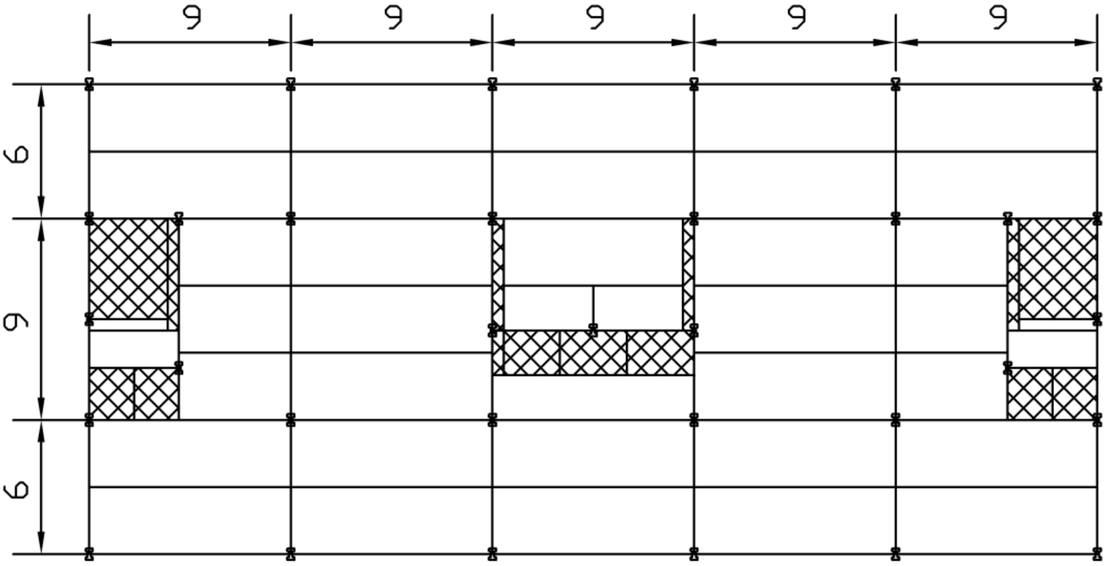


Figure 5-2: Original Cardington frame layout

Table 5-3: Beam system information (italic font denotes the equivalent bare steel frame)

Structural element	Type	L (m)	Beam Section	$W_{b,Pl,Rd}$ (kNm)	Centre deflection (mm)	L/D
Primary / transverse	Composite	6	356x171x51 UKB	559.5	3.89	13.4
Primary / transverse	Bare steel	6	457X152X82 UKB	577.6	5.53	13.5
Secondary / longitudinal	Composite	9	356x171x51 UKB	638.5	5.98	20.0
Secondary / longitudinal	Bare steel	9	457X152X82 UKB	643.3	8.34	20.1
Column			305x305x198 UKC			

Table 5-4: Composite and bare steel element dimensions, grade and type

Element	Dimensions	Grade	Type
Steel beam	*Varying*	S355	UKB
Composite slab	h_c = 130mm , h_c = 70mm, c = 50mm	35 (lightweight)	
Reinforcement	*Varying (default: 4 ∅16)*	S 460	

Table 5-5: Connection component information

Component	Type	Grade	f_y (kN/mm^2)	Dimensions (mm)
Plate	Full depth endplate	S275	0.275	T_p = 10, B_p = 150, D_p = D_{beam}
Bolts	Two M20 (22 mm holes), four rows	8.8	0.640	g = 90, e_1^T = 90, $p_{1,2,3}$ = 70
Welds	Fillet welds	-	-	6 x 6

Table 5-6: Unalterable and alterable frame, beam and connection parameters for this case study

	Unalterable (native to the frame)	Alterable	Range
Frame	Frame arrangement		
	Beam system length		
	Axial restraint		
Beam	Thickness of slab and profile height	Reinforcement ratio (ρ) (tensile component)	ρ = 0-3.57 %, step ≈ 0.45% (2∅16)
	Beam and column sections	Span/depth ratio (indirectly)	*See table 5-3*
	Beam moment capacity / axial stiffness	Beam length (indirectly)	*See table 5-3*
	Beam section depth		
Connections	Bolt size	Endplate thickness t_p (tensile component)	t_p = 8 – 20 mm, step = 2 mm
	Bolt row geometry and number	Beam flange stiffening (compressive component)	$F_{yc,bf,Rd}$' = 60-300% x $F_{yc,bf,Rd,}$, step ≈ 10%

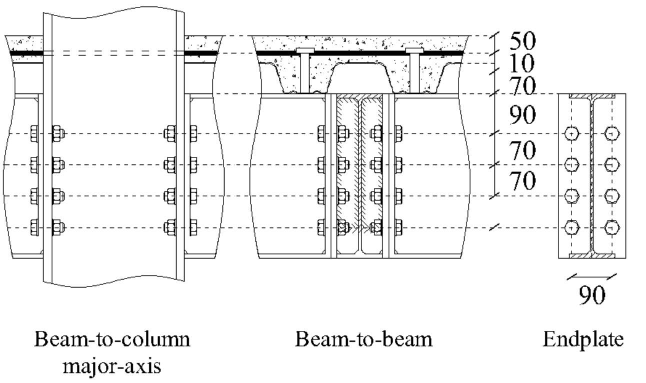


Figure 5-3: Composite arrangement connections

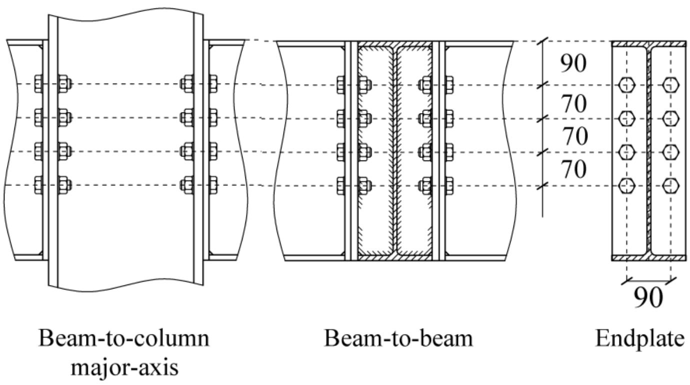


Figure 5-4: Bare steel arrangement connections

5.5 Parametric tests for individuation beam systems

5.5.1 Introduction

Previous investigations (Nethercot D.A. et al., 2009, Stylianidis et al., 2009, Nethercot et al., 2011) have helped gain better understanding of the parameters influencing the behaviour of the individual beam systems: response is principally governed by the rotational capacities of the connections, which depend on the relative resistance and stiffness of the tensile and compressive components. Examples of the tensile components for common connection types include the endplate, bolt rows and reinforcement bars whereas examples of compressive components include the beam flange in compression and the column shear panel. The failure criterion employed corresponds to the maximum deformation capacity of the connections, which depends on the strength, stiffness and ductility of key connection components. Once the rotational capacity reserve is exhausted, unloading begins in these connections.

Complementary to these investigations, a series of extensive parametric tests was carried out for both prototype frames with the focus in this case being on:

i) Identifying which parameters are most influential in improving resistance for common beam system arrangements and attempting to normalise their impact.

ii) Looking for patterns in the results, i.e. until what point will adding tensile reinforcement to the connection benefit the response and why.

iii) Examining whether the findings of the two previous considerations can be formalised in a methodology which will allow an "answer-first" design approach, i.e. a method that will highlight the most pertinent improvements based on the initial assessment results.

This section presents the main findings and conclusions from the studies along with the necessary supporting material to illustrate the key points. For detailed results and comments, the reader is invited to refer to Appendix B, which includes the parametric test results (which deformations correspond to component yielding and failure, identification of the critical components, ultimate capacity and ductility and the main resistance actions) for all beam systems, as well as the double parametric test results for simultaneous manipulation of the connection endplate thickness and the concrete slab reinforcement ratio.

As explained in Section 2.4.2, the ICL Method considers the first point of failure in order to calculate the maximum capacity of the system. The part of the response curve past that point is provided, where necessary (i.e. in Figure 5-5), in order to demonstrate the theoretical response of the system if the deformation capacity of all components was inexhaustible.

5.5.2 Cantilever beam systems

For the cantilever beams, the pseudostatic response tends to approach the maximum static response. When the support connection exhausts its capacity, then the beam's deflection continues to increase without any increase in the pseudo dynamic load.

As mentioned in Table 5-2, cantilever beam systems are subjected solely to bending moments. Also, according to the model presented in Section 3.2.4.1, because the connection compressive and tensile forces are equal, failure of either component means failure of the system. Thus, connection moment capacity will be equal to the lesser of F^{T}_{Rd}*d or F^{C}_{Rd}*d; if tensile capacity governs then increasing compressive resistance will clearly not enhance the connection moment capacity. For example, in Figure 5-5, higher percentages of stiffening push the yielding point of the compressive beam flange further in the response but this is only beneficial up to the stage at which the tensile components become critical before the compressive ones.

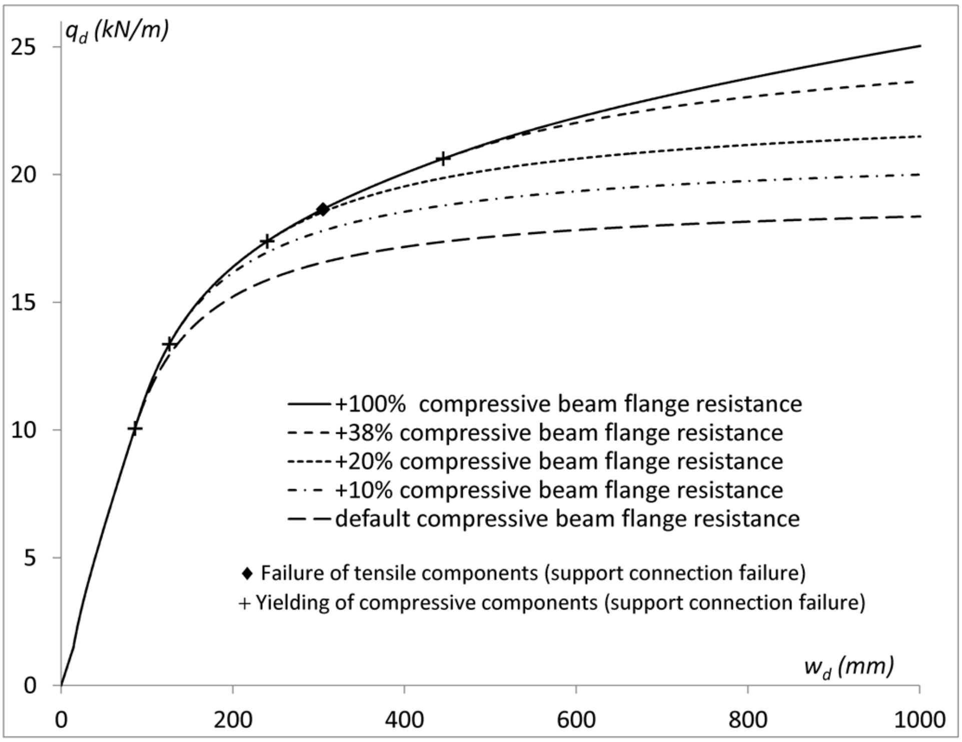


Figure 5-5: q-w response of transverse cantilever systems for increased connection compressive components' resistance (ρ=1.79%, +0% to +100% variation in comp. resistance)

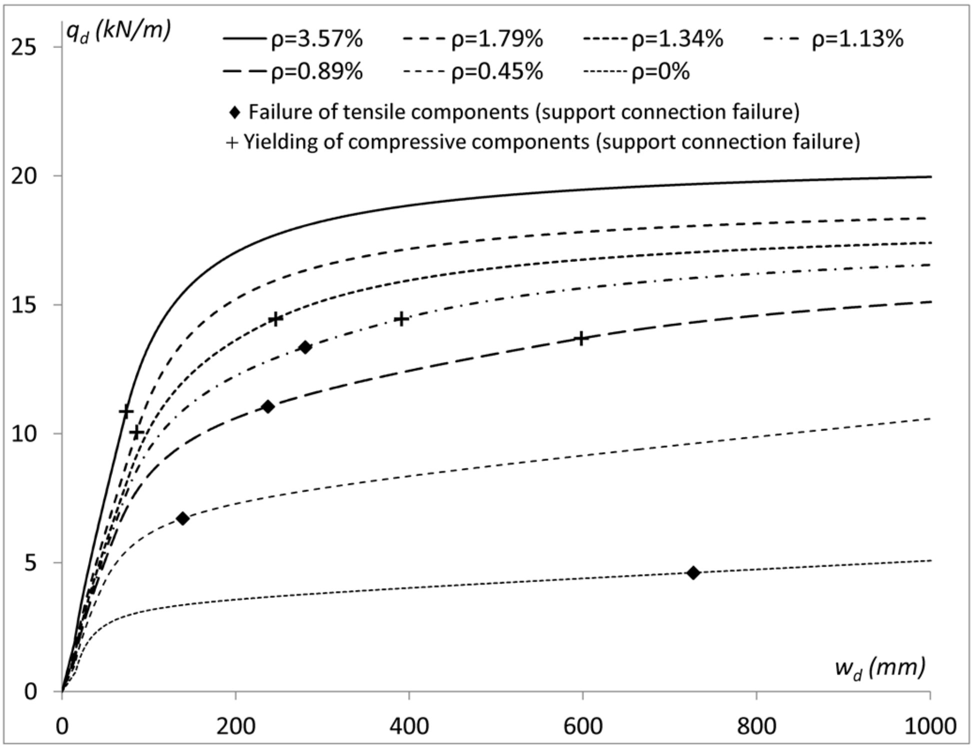


Figure 5-6: q-w response of transverse cantilever beam systems for varying reinforcement ratios

In this case, increasing the reinforcement ratio (Figure 5-6) provides additional resistance and deformation capacity for the connection tensile components, which thanks to the existing capacity of the compressive components, boosts system resistance and ductility. However, for high levels of tensile component capacity ($\rho >= 1.34\%$), the compressive components begin to yield before the reinforcement bars fail. The contribution of extra reinforcement past that point is not beneficial because the increased stiffness of the tensile components causes the compressive components to yield for lower deflection levels - at which the system has not yet developed most of its resistance. Thus, this point defines the maximum level at which the connection can be reinforced in tension without the sacrifice in ductility having an overall negative impact on ultimate capacity and it is different for each connection design.

5.5.3 Axially unrestrained double span beam systems

The axial load is negligible for the axially unrestrained beams and the double span system cannot reach full catenary action as the response is governed mainly by bending effects. Unlike the cantilever system, the tensile components' brittle failure governs system failure, since yielding of the compressive components does not limit the system's ability to redistribute the loading.

In general, the support and centre connections for double-span semi-continuous beam systems exhibit different rotation capacities for connection designs that behave differently under hogging and sagging bending moment loading (horizontally asymmetrical) such as the composite arrangements. This is due to the difference in the position of the lever arm and the compression centre as well as the different behaviour of concrete and steel in compression and tension. For example, while the failure point of the centre connection is unaffected by modifications of the reinforcement ratio in Figure 5-7, changes in the endplate thickness in Figure 5-8 can be beneficial until the bolt row becomes the critical component; thicknesses beyond $t_p = 12$mm limit the rotational capacity of the connection because of the increased stiffness of the endplate. Thus, increasing the tying capacity of the connections with additional reinforcement bars may be less effective than increasing the endplate thickness.

In addition, for responses in Figure 5-7 and Figure 5-8 corresponding to $\rho > 1.13\%$ and to $t_p > 14$ mm respectively, the yielding of the compressive components occurs before the failure of the tensile ones, which moves the support connection failure point well past that of the centre. In fact, as Stylianidis observed (Stylianidis, 2010), the increased deformation of the compressive component (θ_2') after yielding reduces the rate of increase in the deformation of the tensile components (since $\Phi' = \theta_1' + \theta_2'$), which benefits the connection's total maximum ductility. This can often lead to the centre connection becoming critical instead, which for the reasons outlined above can be accompanied by an increase in system maximum capacity and ductility. Thus, the beam system develops an enhanced response by "unlocking" the untapped rotational capacity of the centre connection.

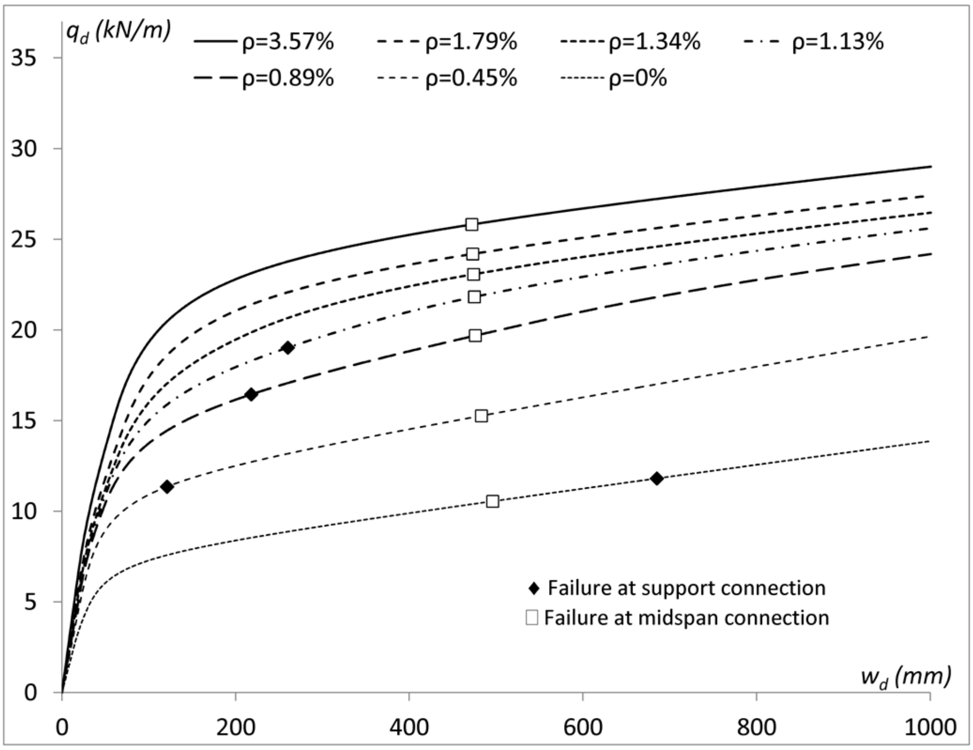


Figure 5-7: q-w response of transverse unrestrained beam systems for varying reinforcement ratios and t_p=10mm

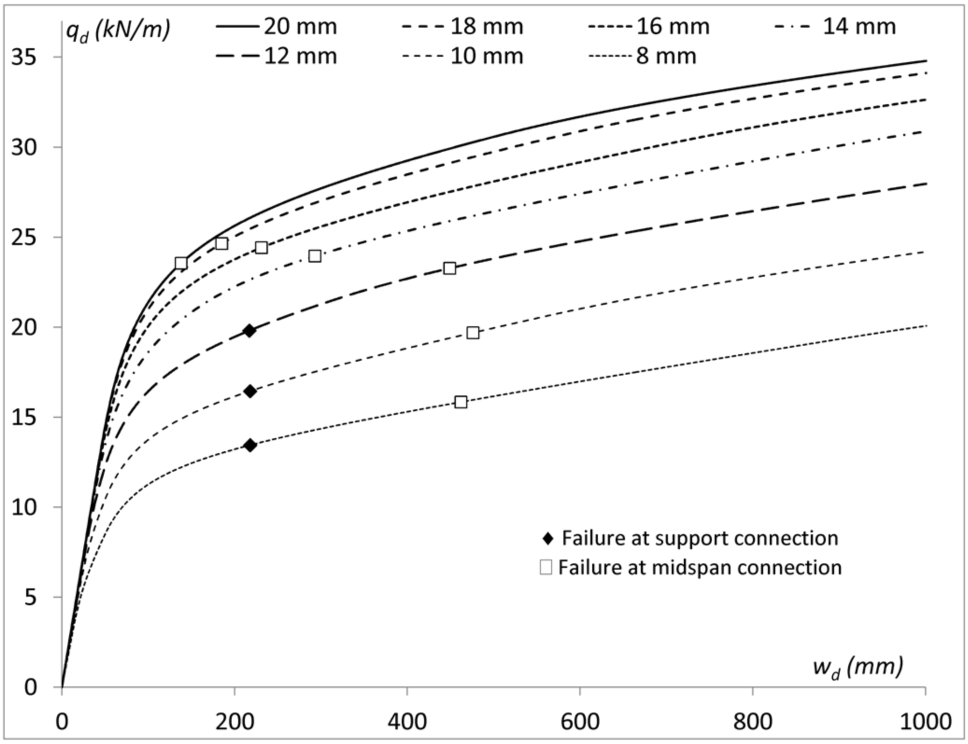


Figure 5-8: q-w response of transverse unrestrained beam systems for varying endplate thickness and ρ=0.89%

5.5.4 Axially restrained double span beam systems

Generally, axially restrained beam systems exhibit higher pseudostatic capacities because of their ability to develop compressive arching and transient catenary effects. These mechanisms are influenced by the level of axial restraint, L/D ratio, connection stiffness and moment resistance. For example, shorter beams demonstrate an enhanced catenary and compressive arching action phase, e.g. the transverse compared to the longitudinal system of Figure 5-9. Their lower L/D ratio increases the level of axial load in the system which assists the compressive membrane effect. However, they can be less ductile because of the larger rotations required for a given displacement, which explains the difference in the centre connection's failure points for the two arrangements.

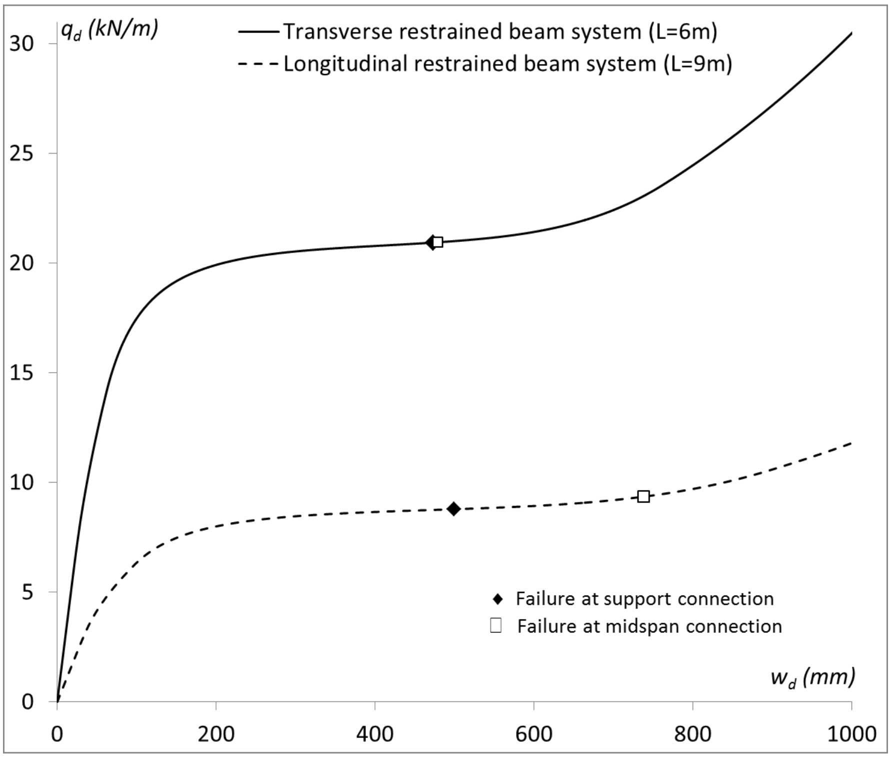


Figure 5-9: Comparison between transverse and longitudinal axially restrained double span systems

The balance between the support connection compressive and tensile components' capacity does not allow for a peak response during the compressive arching phase of the transverse system to take place; the compressive components yield for low deflections before the response can reach a local maximum. Whereas in Figure 5-10, high levels (+30%) of theoretical stiffening of the beam flange in compression significantly assist the compressive arching action, leading to a peak in response during that phase, followed by a softening phase. In this case, increasing the ductility of the system will only enhance the response if it is adequate in order to "push" the failure point into the tensile catenary phase. This agrees with recent research findings (Nethercot D.A. et al., 2011) which argue that “the performance of axially restrained long-span beams is similar to the performance of the corresponding axially unrestrained beams unless either the connection compressive capacity or the connection ductility is very high”; the latter is not true in the present case.

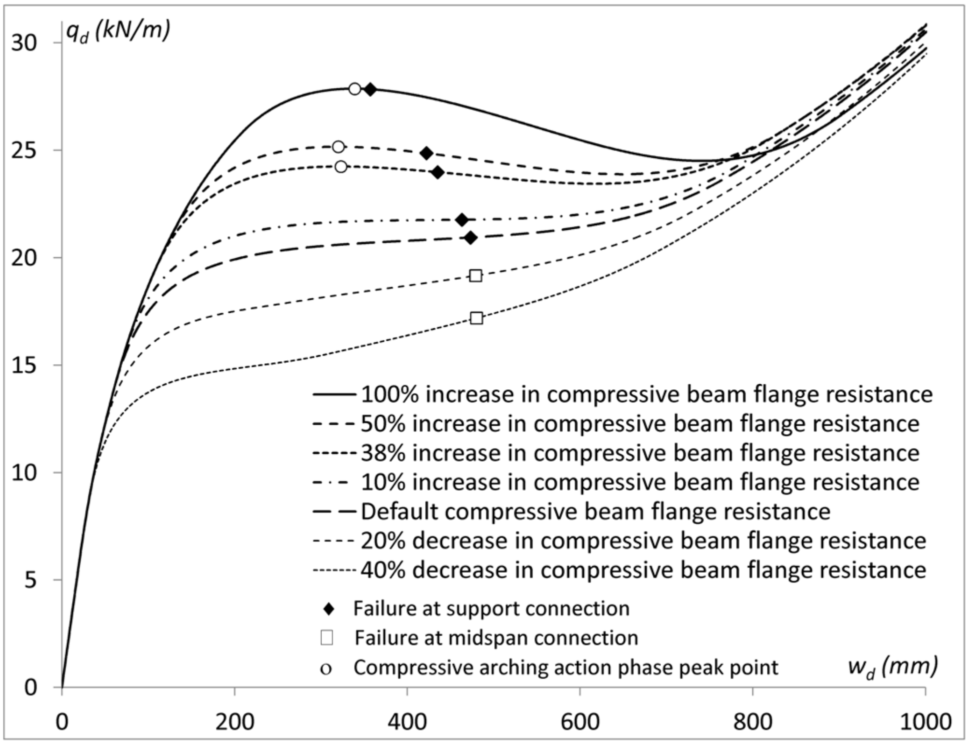


Figure 5-10: q-w response of restrained beam systems for increased connection compressive components' resistance (ρ=1.79%, -40% to +200% variation in compressive resistance)

5.5.5 Comparison between the transverse and longitudinal beam systems

The progressive collapse analysis study of the two beam systems is an opportunity to examine the influence of additional parameters in the pseudostatic response, which is summarized in Table 5-7 below:

Table 5-7: Impact of the transverse and longitudinal beam systems' differences on their response

Parameter	Beam system		Influence on system pseudostatic response
	Trans.	Long.	
Connection component design (bolt and reinforcement bar position, endplate t_p, bolt row geometry)	Same		Since the connection components and beam depth are the same, the tensile and compressive components' capacity is standard and so are the connection stiffness, strength and rotational capacity. Thus, the critical connection is expected to be the same for both systems.
Beam Section	Equal		

Beam length L	6 m	9 m	Higher L/D ratios decrease the level of axial load for the same levels of deflection. Thus, tying capacity is a more relevant provision for the transverse beam. Although shorter beams demonstrate an enhanced catenary and compressive arching action phase, they are usually less ductile; larger rotations for a given displacement are required and rotational capacity is quickly exhausted.
Span/depth ratio (L/D)	13.4	20.0	
Beam to column connection	Major axis	Minor axis	As minor-axis connections don't have column shear and compression panels, their yielding resistance doesn't limit the compressive components resistance for connections under hogging bending moment. However, this is also rarely the case for the full depth endplate major axis connections employed in this study. In general, since the critical compressive component is the beam flange, the different type of connection has a minor impact on the response.
Beam section moment capacity ($W_{b,Pl,Rd}$)	559.5 kN	638.5 kN	For cantilevered beams, beam moment capacity has little impact on the response; connection properties play the most important role in defining the maximum system capacity and ductility.

5.6 Comparison with an equivalent bare steel arrangement

5.6.1 Introduction

The mechanics of progressive collapse for the two chosen frames vary, despite the use of broadly equivalent beam and column sections. In order to further investigate which parameters are the cause of this, the study focuses on comparing responses at the first two basic levels of structural idealisation: the individual beam systems presented in Table 5-2 and Table 5-3 as well as the assembled floor grillage models presented in the next section.

5.6.2 Overview

Comparison of the pseudostatic responses in Figures 5-11 and 5-12 shows that the composite system attains a higher maximum pseudostatic capacity because of its higher connection bending moment resistance and stiffness as well as lower L/D. Nevertheless, the bare steel system is almost twice as ductile: $w_{composite}$ / 2D = 0.19 < w_{bare} / 2D = 0.4. The thicker compressive beam flange and higher deformation capacity of the bare steel connection tensile components (bolts vs. reinforcement bars) increase the available $\theta_{1,f}$, (rotation of the tensile rigid bar – Figure 3-2), which increases the available connection rotation capacity.

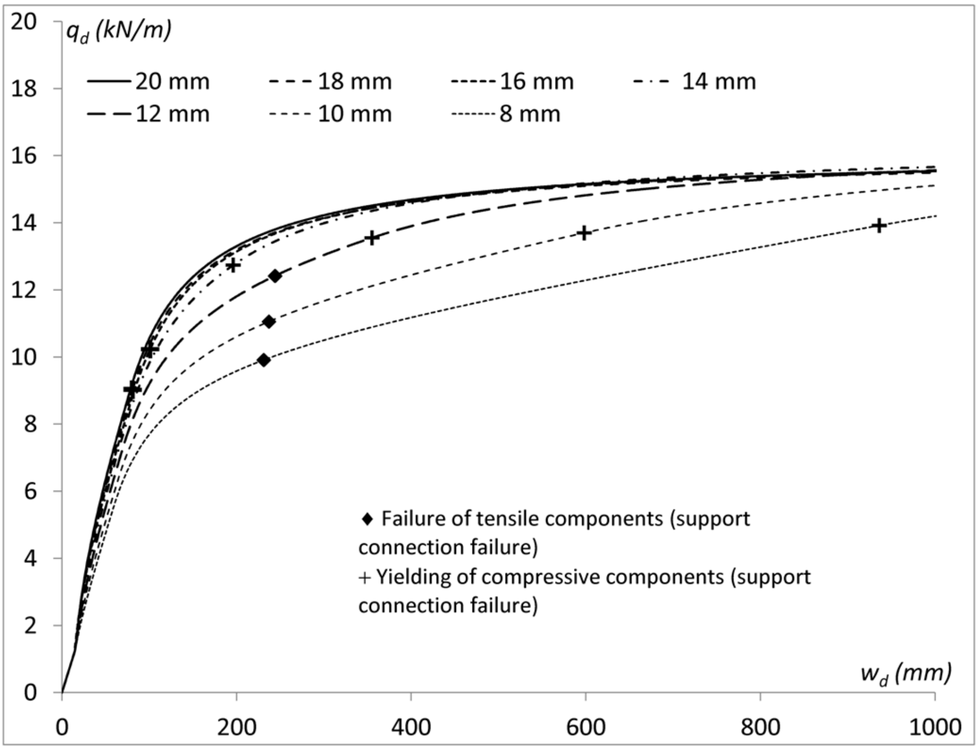


Figure 5-11: q-w response for cantilever composite beams for varying connection endplate thickness t_p

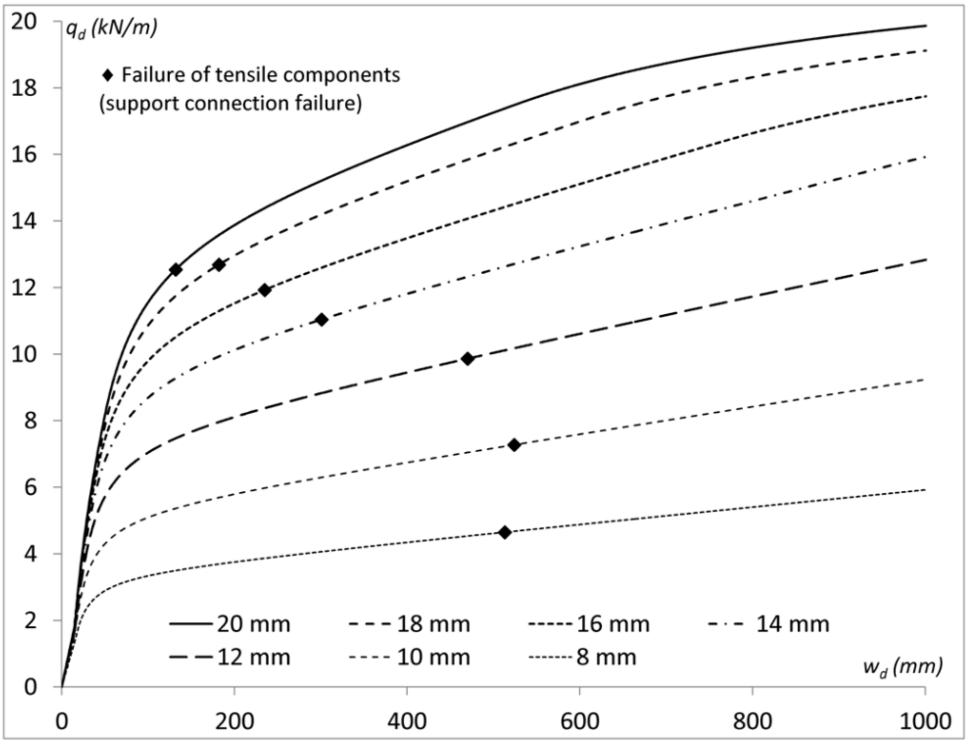


Figure 5-12: q-w response for cantilever bare steel beams for varying connection endplate thickness t_p

Regardless of the level of axial restraint, the inherently different connection design influences the balance of components' capacity. On the one hand, composite connections benefit from the additional deformation capacity of the reinforcement bars and the longer lever arm compared to the bolt rows in bare steel connections. Also, the centre of compression is raised from the beam top flange into the concrete slab, thereby increasing its moment capacity. On the other hand, the thicker beam flange of the equivalent bare steel beams increases their connection compressive components' capacity.

5.6.3 Connection design and response action phases

The comparison between axially restrained system responses in Figure 5-13 and Figure 5-14 stresses the influence of the balance in capacity between the connection compressive and tensile components in controlling system pseudostatic resistance.

Moreover, despite the higher connection strength and stiffness of the composite arrangement, the compressive components' low capacity (yielding occurs for $q \approx 14$ kN/m) prevents it from achieving a peak response during the compressive arching phase. On the contrary, because of the study's bare steel connections' increased compressive resistance (yielding occurs between $q \approx 20\text{-}15$ kN/m), lower tensile component resistance (absence of reinforcement) and reduced deformation capacity compared to the composite case, the beneficial effect of axial restraint is more pronounced: for t_p<16mm, the peak point of the response during the compressive arching phase corresponds to greater capacity than does the point of failure.

In addition, although the composite arrangement provides an increased capacity (≈ +10%) for the original thickness of the endplate, this difference disappears for the use of endplates thicker than 14mm. At that point, the bolt row becomes the critical component of the centre connection and increases in t_p only make the connections less ductile. Due to the more favourable balance in connection component capacity in the bare steel case, the response can reach higher values of pseudostatic resistance earlier in the response curve and thus both arrangements exhibit similar capacities at their failure points, despite the advantages of the composite case that were mentioned earlier. Furthermore, the bare steel system is more ductile by an average of +30% ($w_{composite} / 2D = 0.37 < w_{bare} / 2D = 0.49$) compared to the composite one, though the available rotational capacity of the connections is still not sufficient for the system to enter the catenary phase.

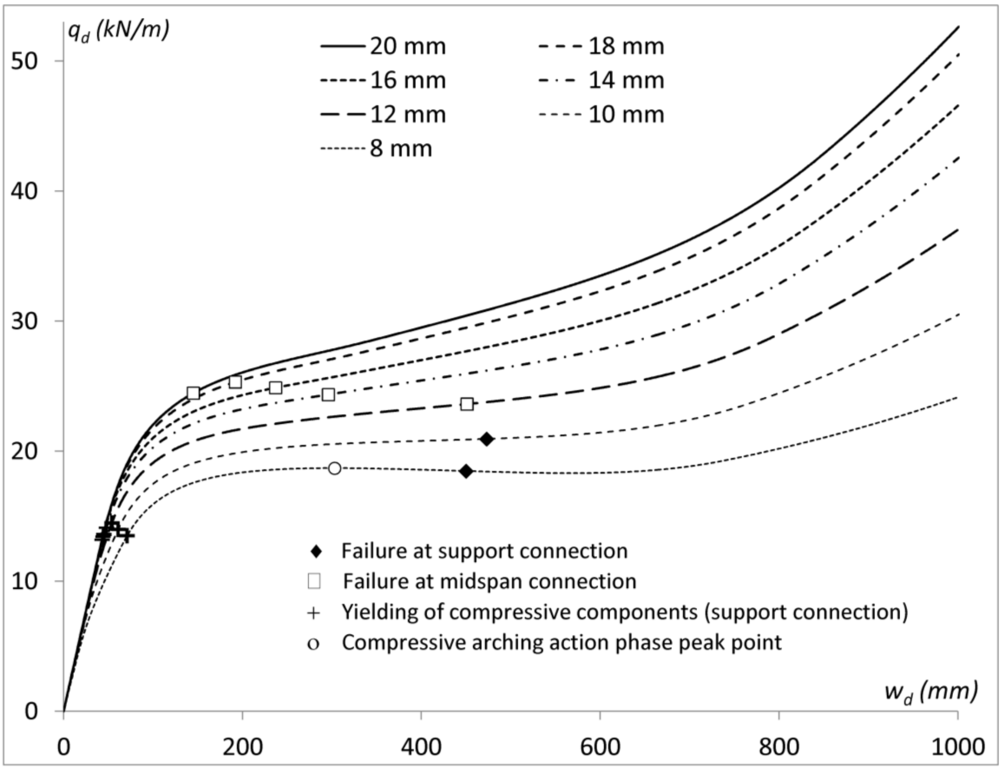


Figure 5-13: q-w response of composite restrained beam systems for varying endplate thickness

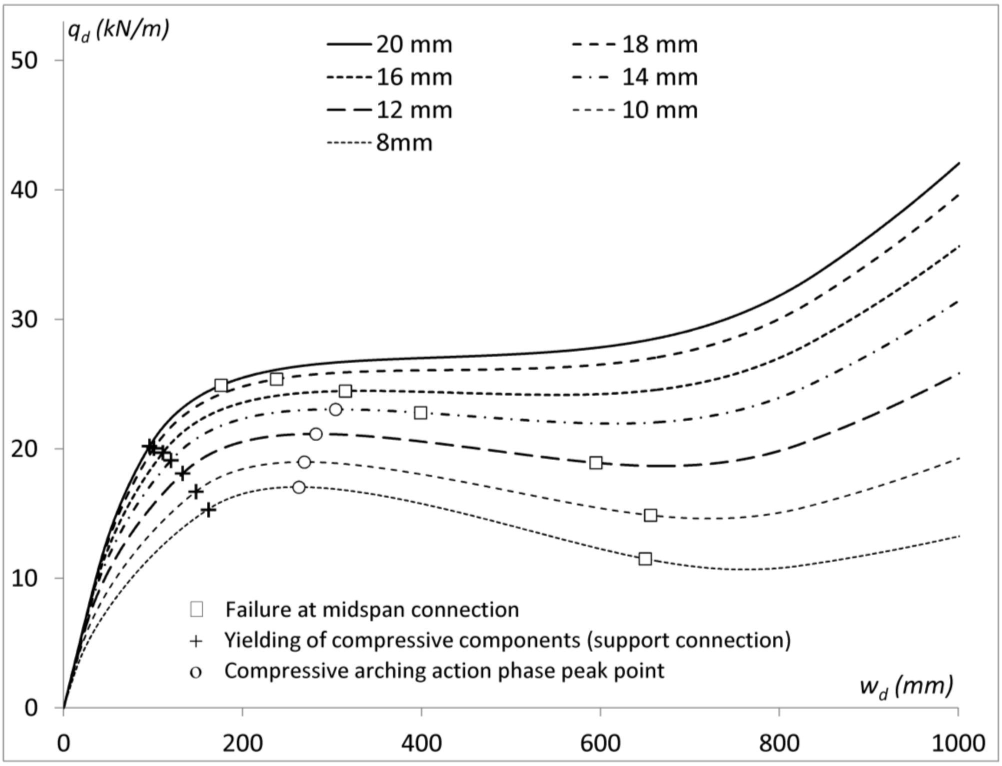


Figure 5-14: q-w response of bare steel restrained beam systems for varying endplate thickness

5.6.4 Sensitivity to beam geometry

The responses in Figure 5-15 and in Figure 5-16 correspond to the transverse and longitudinal beam systems, which only differ in their length ($L_{transverse}$ = 6m, $L_{longitudinal}$ = 9m) and in the use of major and minor axis beam to column connections respectively. The varying influence of beam length and span to depth ratio for the two section types is presented in Table 5-8. The bare steel systems appear to be much more "sensitive" to changes in these parameters, which can be attributed to their prominent influence on the compressive arching and catenary action phases' properties. These actions subsequently govern the response depending on which is critical; this forms the central point of discussion of Section 5.11.

Table 5-8: Influence of beam length and of length to depth ratio on system response

	Response characteristic	Influence on response: composite	Influence on response: bare steel
L and L/D	Ductility (% relative variation)	Longitudinal beam systems moderately more ductile: Cantilever: +30% Double span: +10%	Considerable gain in ductility for longitudinal systems: Cantilever: +45% Double span axially unrestrained: +43% Double span axially restrained: +17%
L and L/D	Compressive arching and tensile catenary action phases	Compressive arching and tensile catenary action more pronounced for the transverse beam systems.	Due to inherent low connection strength and lower L/D, catenary action is less pronounced for the shorter beam. On the other hand, because compressive arching effects are already more pronounced for the bare steel system, the shorter length boosts the peak capacity attained during this stage.
L and L/D	Capacity (% relative variation)	Noticeable loss in average for the longitudinal beam systems: Average loss: - 40%	Noticeable loss in average for the longitudinal beam systems, substantial loss for those axially restrained: Cantilever: - 35% Double span unrestrained: -37% Double span restrained: -52%
Beam to column connection		No effect on the response (critical compressive component: beam flange)	
Beam moment capacity ($W_{b,Pl,Rd}$)		Negligible effect on the response	

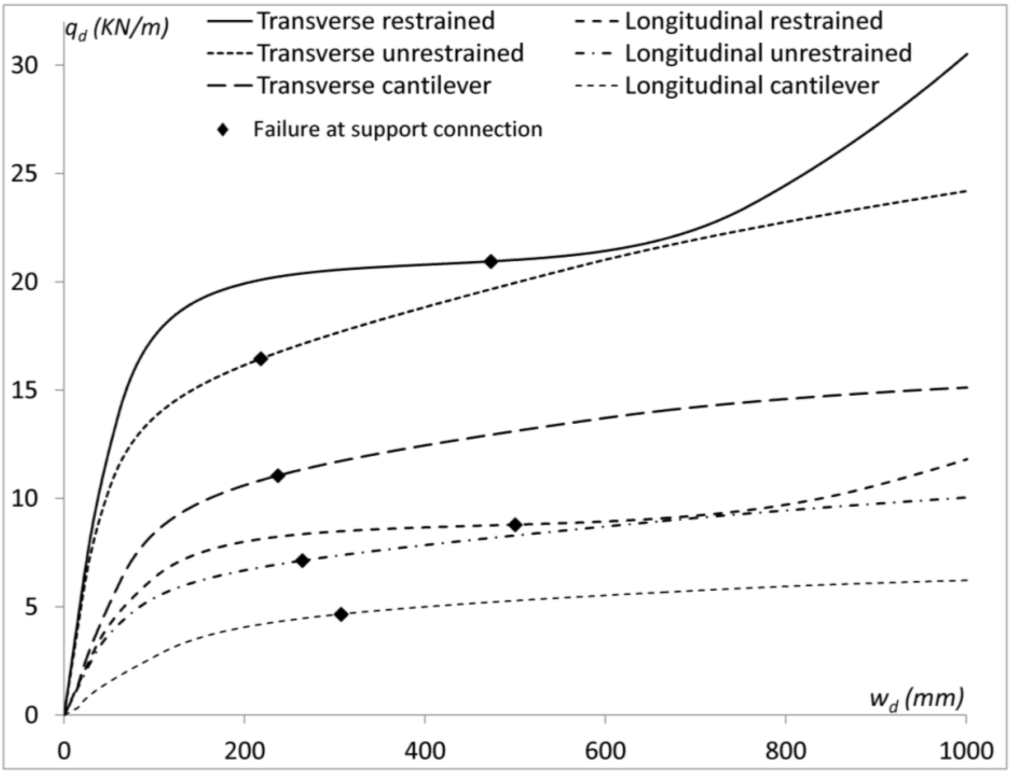


Figure 5-15: q-w response of the composite transverse and longitudinal beam systems

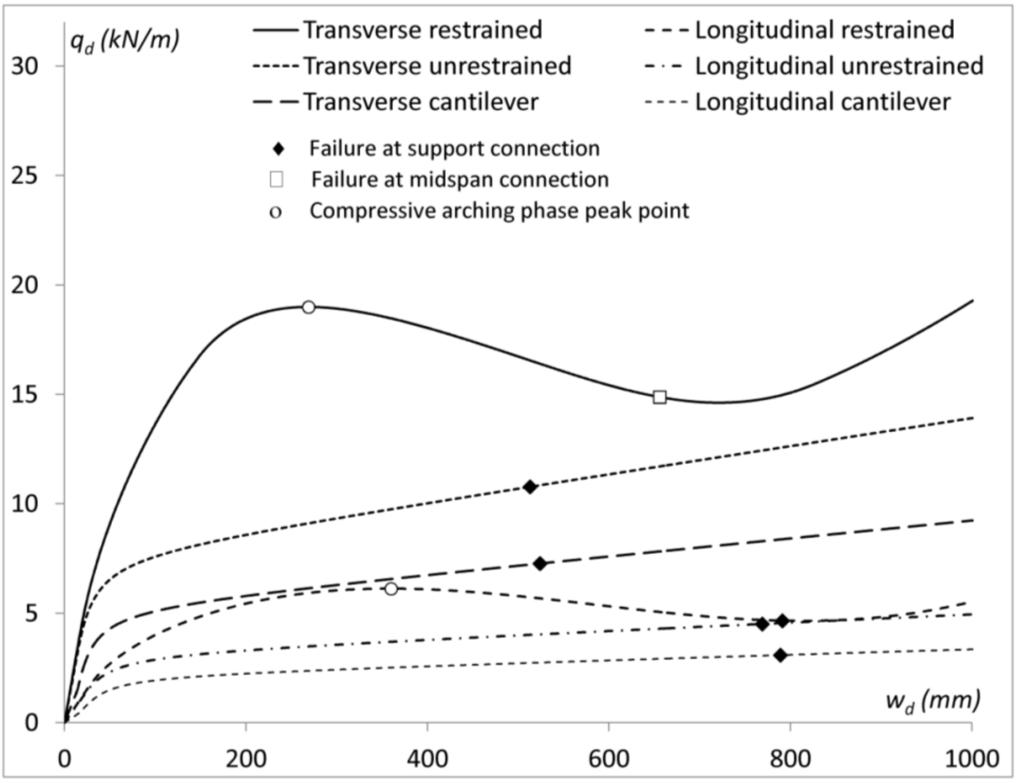


Figure 5-16: q-w response of the bare steel transverse and longitudinal beam systems

5.7 Original floor grillage collapse

5.7.1 Introduction

According to the multi-level idealization in Section 2.4.1.4, the floor response can be determined by the combined responses of the constituent beam systems using an appropriate displacement factor (β) related to the geometry of the frame to ensure compatibility (Vlassis et al., 2008a). The floor grillage approximation is shown in Figure 5-17.

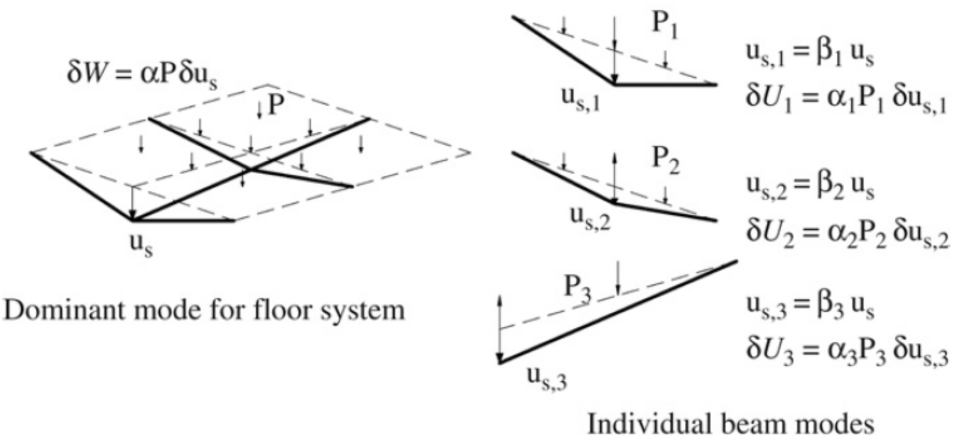


Figure 5-17: Grillage approximation for a floor system with three beams (Izzuddin et al., 2008)

Since the approximated floor response depends entirely on the responses of its constituents, the previously mentioned connection and beam parameters also govern the floor grillage behaviour. Recent work (Nethercot D.A. et al., 2011) has shown that the beam system with the lowest ductility provides the ultimate rotational capacity which defines the deformation at failure for the grillage. Up to that level of deformation, the beam with the highest pseudostatic resistance defines the overall response of the grillage. These are usually the girders or the axially restrained beams. Thus, when designing the beam systems, care must be taken to ensure that any "sacrifice" of ductility for a higher maximum beam system capacity does not have a much larger adverse effect if the beam is used in a grillage. This also applies when cantilever beams are used with restrained beams, where the formation of arching action requires larger displacements than those associated with yielding of the connection.

5.7.2 Column removal scenarios

Table 5-9 summarises the constituent floor grillage assembly elements corresponding to each column removal scenario, which are presented in Figure 5-18.

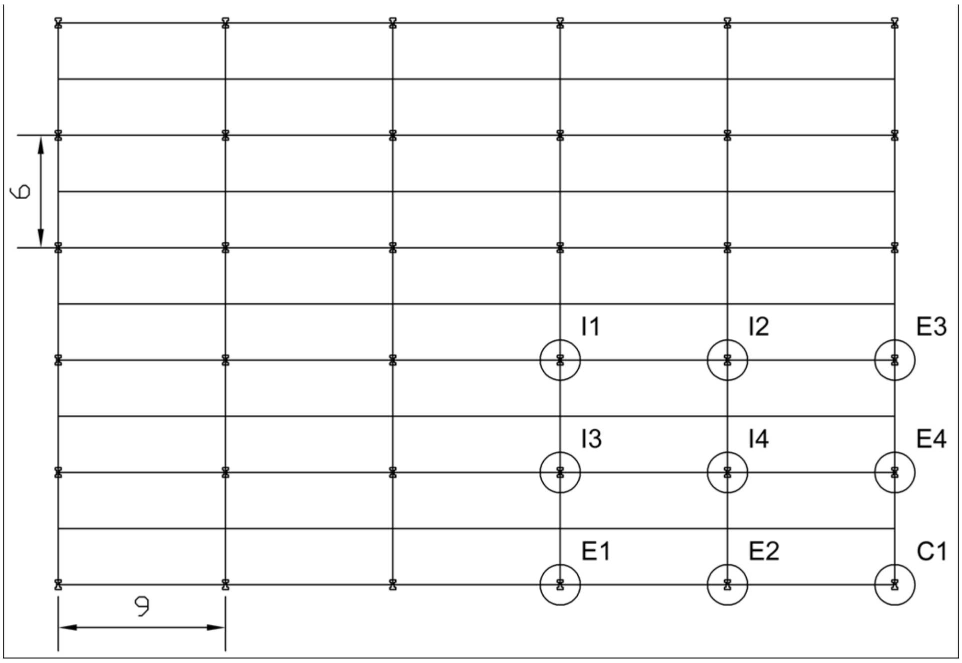


Figure 5-18: Column removal scenarios for the simplified Cardington frame arrangement

Table 5-9: Floor grillage assemblies depending on the column removal scenario

#	Description	Transverse beam systems			Longitudinal beam systems			Supported floor area (m^2)	Demand (kN)
		No.	β[a]	Axial restraint	No.	β	Axial restraint		
I1	Internal	1	1	yes	1	1	yes	216	1024
					2	0.5	yes		
I2[b]	Internal	1	1	yes	1	1	no		
					2	0.5	no		
I3	Internal	1	1	no	1	1	yes		
					2	0.5	yes		
I4	Internal	1	1	no	1	1	no		
					2	0.5	no		
E1	Edge	1	1	cantilever	1	1	yes	108	512
					1	0.5	yes		
E2	Edge	1	1	cantilever	1	1	no		
					1	0.5	no		
E3	Edge	1	1	yes	1	1	cantilever		
					2	0.5	cantilever		
E4	Edge	1	1	no	1	1	cantilever		
					2	0.5	cantilever		
C1[b]	Corner	1	1	cantilever	1	1	cantilever	54	256
					1	0.5	cantilever		

[a] *β is a displacement factor related to the geometry of the frame to ensure compatibility*
[b] *Refer to Figure 5-1 for illustration*

5.7.3 Analysis results

The progressive collapse analysis for the composite arrangement reveals the need for design interventions aimed at increasing robustness. More specifically, the frame cannot resist progressive collapse in internal column loss scenarios (Figure 5-19) with the exception of scenario I1 for which the corresponding grillage is composed entirely of axially restrained beam systems. On the contrary, for the edge and corner column removal scenarios (Figure 5-20), only the grillage assemblies with the cantilever transverse beam system (scenarios E2 and C1) lack the capacity to meet the demand. The transverse beam system is critical for most scenarios, as a result of its lower ductility compared to the longitudinal beams participating in the grillage assembly (Table 5-9). The least favourable scenarios appear to be the internal column removal scenarios closer to the edges of the building, instead of the edge column removal scenarios which might intuitively have been assumed to be the most critical due to the lack of axial restraint in their constituent beam systems. This can be attributed to the larger pseudostatic capacity demand on these areas combined with the low ductility of the double span primary beam systems. Nonetheless, façade loading has not been taken into account in this case study in order to simplify the analysis, leading to a conservative estimate of the loading demand at the perimeter floors of the frame.

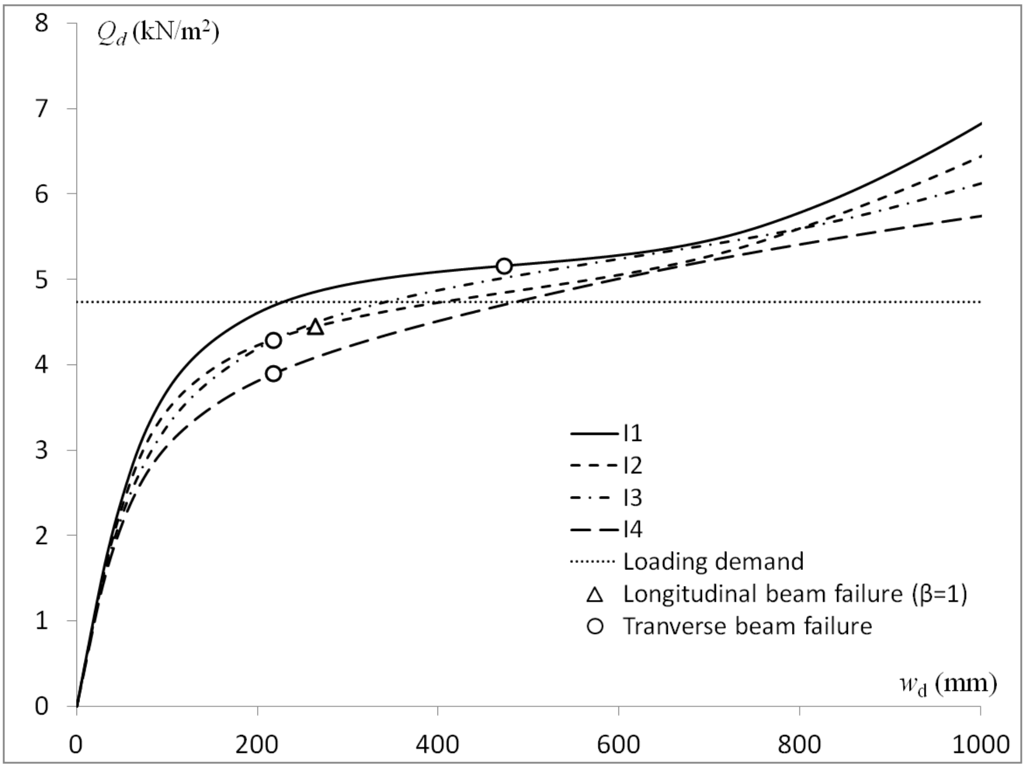


Figure 5-19: Q-w response of composite floor grillage assemblies for internal column loss scenarios

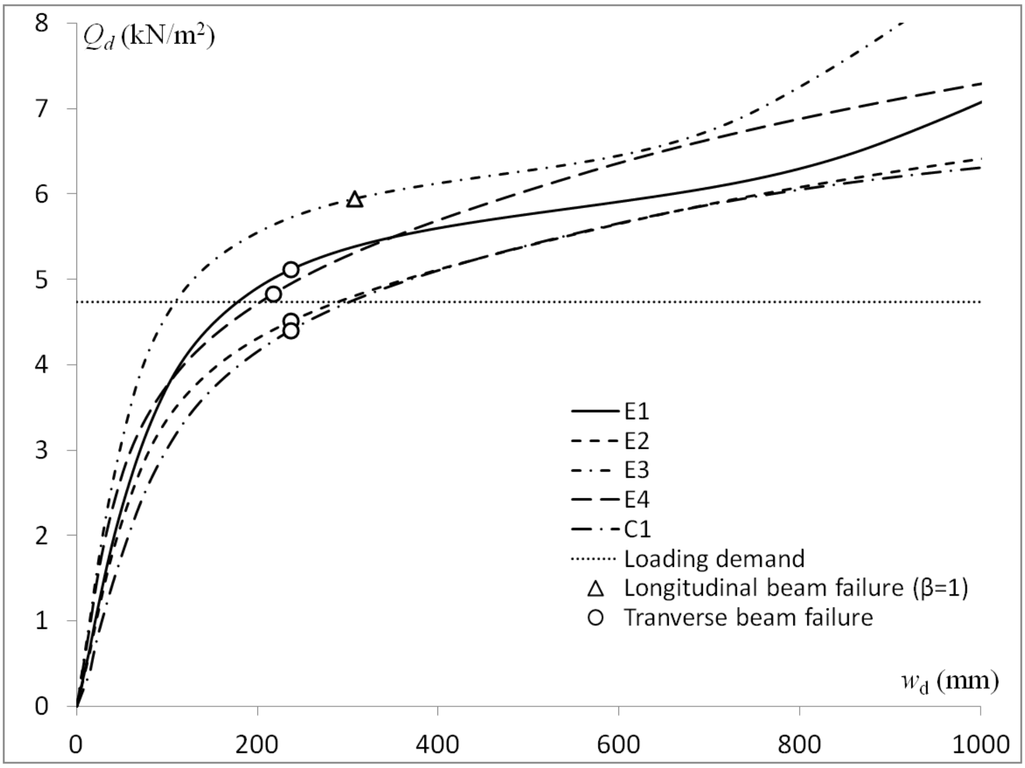


Figure 5-20: Q-w response of composite floor grillage assemblies for edge & corner column loss scenarios

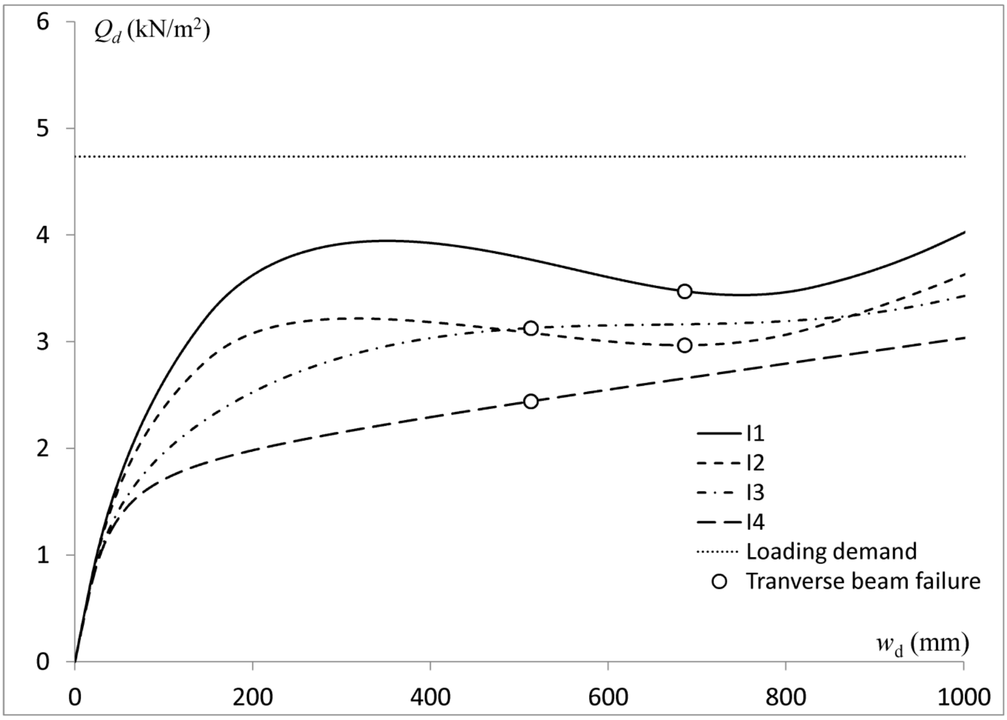


Figure 5-21: Q-w response of bare steel floor grillage assemblies for internal column loss scenarios

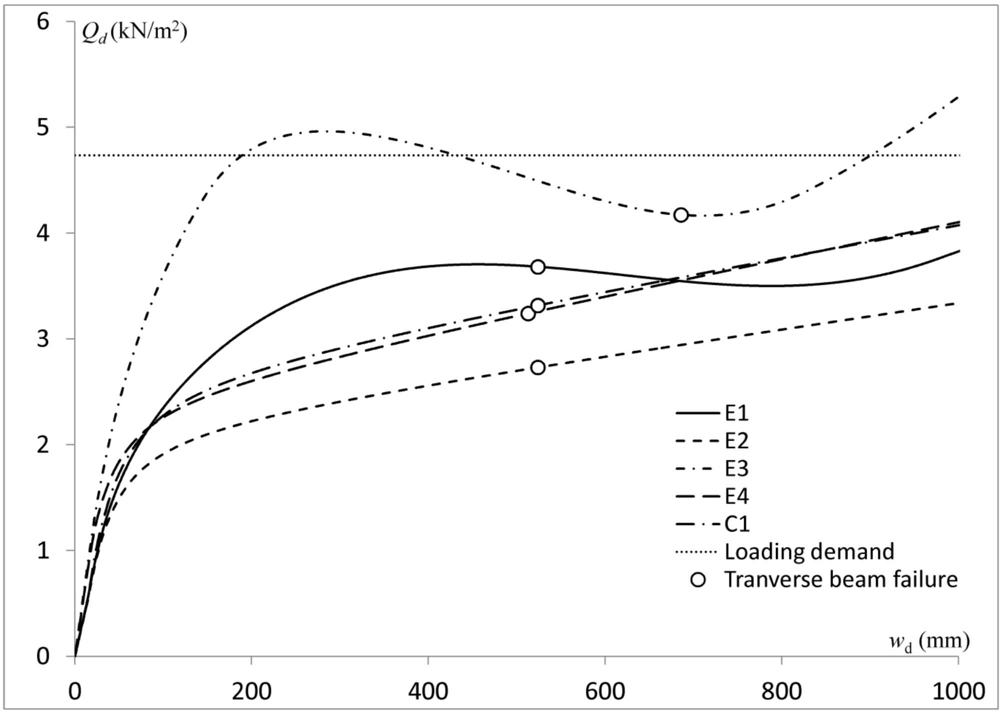


Figure 5-22: Q-w response of bare steel floor grillage assemblies for edge and corner column loss scenario

Concerning the equivalent bare steel floor grillage assemblies (Figure 5-21 and Figure 5-22), only the one corresponding to scenario E3 can meet the demand. The rest do not provide sufficient resistance at any stage, even during tensile catenary, so increasing system ductility cannot serve as a remediating solution by itself, contrary to the composite arrangement. Vlassis et al (Vlassis A.G. et al., 2008) also observed very low pseudostatic capacities of bare steel frames compared to composite, starting from 20% and amounting in average between 75% and 105% of the required demand respectively for the cases studied.

The difference between the resistance of the two arrangements was generally expected, given the lower capacities of the axially unrestrained and cantilever bare steel beam systems, whose responses are limited by the low strength and stiffness of the connections. Though, for grillages comprised mainly of axially restrained beam systems, such as those corresponding to scenarios I1, I2 & I3, this is less obvious. For these assemblies, unlike the small difference of 10% in capacities observed on the first level of structural idealisation (Figure 5-12 and Figure 5-13), the maximum capacity of the composite grillage is 40% higher on average compared to the bare steel equivalent.

The explanation for this difference is found in the form of the constituent beam systems' response. Moreover, as the compressive arching and tensile catenary action phases are more pronounced for bare steel, the response maximum capacity is achieved at the peak of the compressive arching action phase, followed by a significant drop during the subsequent softening phase. However, the composite system exhibits a more consistent response.

Therefore, for the bare steel assembly, the response peaks of the constituent beam systems are achieved for non-coinciding deformation levels as it can be seen in Figure 5-16. Thus, the average resistance of the floor system during progressive collapse does not reflect the highest pseudostatic capacities observed for the individual beam system responses.

5.8 Choice of the solution for improving resistance

5.8.1 Prioritising improvements for progressive collapse resistance

The combinations of alternative connection configurations can generate numerous different frame arrangements. Despite the fact that some may enhance the pseudostatic responses of the individual beam systems, this does not guarantee an equal effect for the floor grillage assembly, mainly because of the reasons outlined in Table 5-10.

Table 5-10: Potential complications concerning pseudostatic resistance when moving from the basic (beam system) to the next level of structural idealisation (floor grillage)

Potential complication / limiting factor	Examples from this study (Section 5.7)
The low ductility of one or more individual beam systems may still limit the floor response, even if the rest of the constituent beam systems are very ductile and exhibit significant capacities.	Internal column removal scenarios I2, I3 & I4 for the composite steel arrangement
Peak capacities of the constituent beam systems may be achieved for non-coinciding deformation levels, limiting the contribution of these systems to the floor system response.	Internal column removal scenarios I1, I2 & I3 for the bare steel frame

In order to avoid conducting unnecessary parametric tests, an appropriate methodology has been devised based on the following priorities:

a) Changes should be as minimal as possible so that the default configuration can be exploited in the best possible way.

b) Changes should be practical to apply according to common construction practice. For example, adding reinforcement bars and using thicker endplates is more feasible than significantly changing the bolt size or stiffening the beam flange.

c) Changes should aim at assisting the system achieve optimal system ductility, which is defined as the ductility demand on the subsystem at the point of realising the maximum contribution to the system pseudostatic capacity from the remaining subsystems, accounting for their ductility supply (Izzuddin B.A. et al., 2008).

Taking these priorities into account, the process for identifying the most appropriate configuration can be split into the four steps presented in the next section. They are described in detail for the composite frame and the final results are given for both arrangements.

5.8.2 Composite arrangement

5.8.2.1 Step A

The simplified frame comprises the beam systems presented in Table 5-9 in the transverse and longitudinal direction. From the alterable parameters in Table 5-4, the connection reinforcement ratio (ρ) and endplate thickness (t_p) are considered in the final alternative connection design. Other parameters, such as bolt horizontal gauge, bolt vertical position and beam section capacity have been excluded from this process because previous studies (Stylianidis, 2011, Blundell D. et al., 2010) have shown their influence on the response of the system to be negligible. For each beam system type, a series of double parametric tests examine the possible combinations of ρ and t_p within the ranges given in Table 5-5, calculating the impact on system ductility and capacity. The best results, which are determined from the parametric tests presented in Appendix B, are narrowed down using Priorities a and b (Section 5.8.1). After that, identifying those which are most beneficial for all the system types apart from the one examined reduces the candidate configurations to the ones presented in Table 5-11 according to their critical action phase.

Table 5-11: Critical action phases for the candidate connection configurations

	Transverse semi-continuous axially restrained beam system	Longitudinal semi-continuous axially restrained beam system
Default configuration: ρ = 0.89% (4∅16), t_p = 10 mm	Compressive arching	Compressive arching
Improvement 1: ρ = 1.34% (6∅16), t_p = 12 mm	Transient catenary	Transient catenary
Improvement 2: ρ = 1.79% (8∅16), t_p = 12 mm	Transient catenary	Transient catenary
Improvement 3: ρ = 1.34% (6∅16), t_p = 10 mm	Transient catenary	Transient catenary
Improvement 4: ρ = 1.79% (8∅16), t_p = 10 mm	Transient catenary	Transient catenary

5.8.2.2 Step B

The impact of each alternative configuration on the response is quantitatively evaluated and compared to the default connection design, which is ρ = 0.89% (4∅16) and t_p = 10mm. Table 5-12 and Table 5-13 present the percentage increase or decrease in ductility and capacity of the transverse and longitudinal beam systems for the configurations chosen in the previous step. Since in the case of yielding of the connection compressive components for cantilever systems there is no explicit failure point, the response is evaluated based on whether it can ultimately provide the resistance capacity required.

Table 5-12: Alternative configurations - impact on primary / transverse beam systems' response (rounded-up values)

Beam system type	Improvement 1 ρ = 1.34% (6∅16) t_p = 12 mm		Improvement 2 ρ = 1.79% (8∅16) t_p = 12 mm	
	Impact on ductility [b]: %	Impact on capacity %	Impact on ductility [b]: %	Impact on capacity %
Primary single span cantilever	n/a[a]	+60	n/a [a]	+70
Primary semi-continuous axially unrestrained	+ 35	+55	+200	+60
Primary semi-continuous axially restrained	- 5	+20	- 5	+30

[a] yielding of compressive components
[b] connection rotational ductility

Table 5-13: Alternative configurations - impact on secondary / longitudinal beam systems' response (rounded up values)

Beam system type	Improvement 3 ρ = 1.34% (6∅16) t_p = 10 mm		Improvement 1 ρ = 1.34% (6∅16) t_p = 12 mm		Improvement 4 ρ = 1.79% (8∅16) t_p = 10 mm	
	Impact on ductility [b]: %	Impact on capacity %	Impact on ductility[b] %	Impact on capacity %	Impact on ductility[b] %	Impact on capacity %
Secondary single span cantilever	n/a [a]	+65	n/a [a]	+60	n/a [a]	+70
Secondary semi-continuous axially unrestrained	+270	+45	+45	+50	+270	+50
Secondary semi-continuous axially restrained	+20	+15	0	+25	+30	+30

[a] yielding of compressive components
[b] connection rotational ductility

5.8.2.3 Step C

The pseudostatic responses of both levels of structural idealisation are examined in order to identify the factors that prevent the floor grillages from meeting the resistance demand. The left column of Table 5-14 displays the most important factors that limit the floor grillage response according to the column removal scenario. For each of them, the configurations that could resolve the issue are presented in the right column. The critical beam systems are solely those axially unrestrained, while the axially restrained beams participating in the grillage can provide significant additional resistance that remains unexploited because of the early failure of the rest of the systems. Thus, focus is on ductility rather than the maximum capacity.

Table 5-14: Factors limiting floor grillage response and potential remediating solutions

Scenario	Factors limiting response	Potential remedial improvements
I2	Very low ductility (-45%) of the axially unrestrained secondary beam system	Improvement 3: +270% ductility increase Improvement 4: +250% ductility increase
I3	Very low ductility (-40%) of the axially unrestrained primary beam system	Improvement 2: +250% ductility increase
I4	Very low ductility (-35%) of the axially unrestrained secondary beam system	Improvement 3: +270% ductility increase Improvement 1: +45% ductility increase Improvement 4: +250% ductility increase
	Very low ductility (-45%) of the axially unrestrained primary beam system	Improvement 2: +250% ductility increase
E2	Inadequate ductility (-5%) of the axially unrestrained secondary beam system	Improvement 3: +270% ductility increase Improvement 1: +45% ductility increase Improvement 4: +250% ductility increase
	Inadequate ductility (-10%) of the cantilever primary beam system	*Only possible to increase capacity for both improvements*
C1	Inadequate ductility (-25%) of the cantilever primary beam system	*Only possible to increase capacity for both improvements*

5.8.2.4 Step D

Using the information from the right column of Table 5-14, the configurations likely to be most appropriate are identified as these presented in Table 5-15. Improvement 4 will be examined only if Improvement 3 fails to efficiently enhance the grillage response. In addition, since beam flange stiffening is a complex and expensive modification compared to changing the endplate thickness and adding reinforcement, this will only be examined if the proposed solution fails to sufficiently enhance the grillage response.

Table 5-15: Final choice of alternative connection design configurations

	Final choice of alternative configurations
Primary beam system	Original: ρ = 0.89% (4∅16), t_p=10 mm Improvement 2: ρ = 1.79% (8∅16), t_p=12 mm
Secondary beam system	Original: ρ = 0.89% (4∅16), t_p=10 mm Improvement 3: ρ = 1.34% (6∅16), t_p=10 mm Improvement 4: ρ = 1.79% (8∅16), t_p= 10 mm

5.8.3 Bare steel arrangement

The methodology is based on the quantitative evaluation of the capacity and ductility deficit. Its use requires the original system to be able to provide capacity superior to the demand at some point during its response, even if this occurs past the failure point, otherwise the required improvements in the constituent beam systems' response cannot be determined with certainty. Thus, this prevents its application for the present equivalent bare steel section because the response does not meet or approach the demand even for the maximum level of deflection (Figure 5-21 and Figure 5-22).

Alternatively, since the bare steel arrangement has only one realistically alterable parameter, which is the thickness of the endplate, a simplistic remediating approach would be to determine the thinnest endplate for which all floor grillages can resist progressive collapse. The grillage response for column removal scenario I4 exhibits the lowest pseudostatic capacity over demand ratio and is thus chosen as the reference. The parametric study presented in Figure 5-23 determined that even for the maximum endplate thickness, t_p=20mm, the corresponding floor grillage is unable to muster the necessary resistance because of the transverse beam system's low ductility.

Since the connection critical component is the bolt row, changes to any other parameter will not address the issue. Potentially, the use of more bolts per row might be beneficial, though this is often not possible because of Eurocode limitations on minimum horizontal bolt gauge for full-depth endplates.

In conclusion, the bare steel frame is inherently less robust, despite the fact that it is more ductile. Its vulnerability against progressive collapse cannot be remediated in this case, at least not without having to perform more radical connection design changes, which are outside the scope of this study.

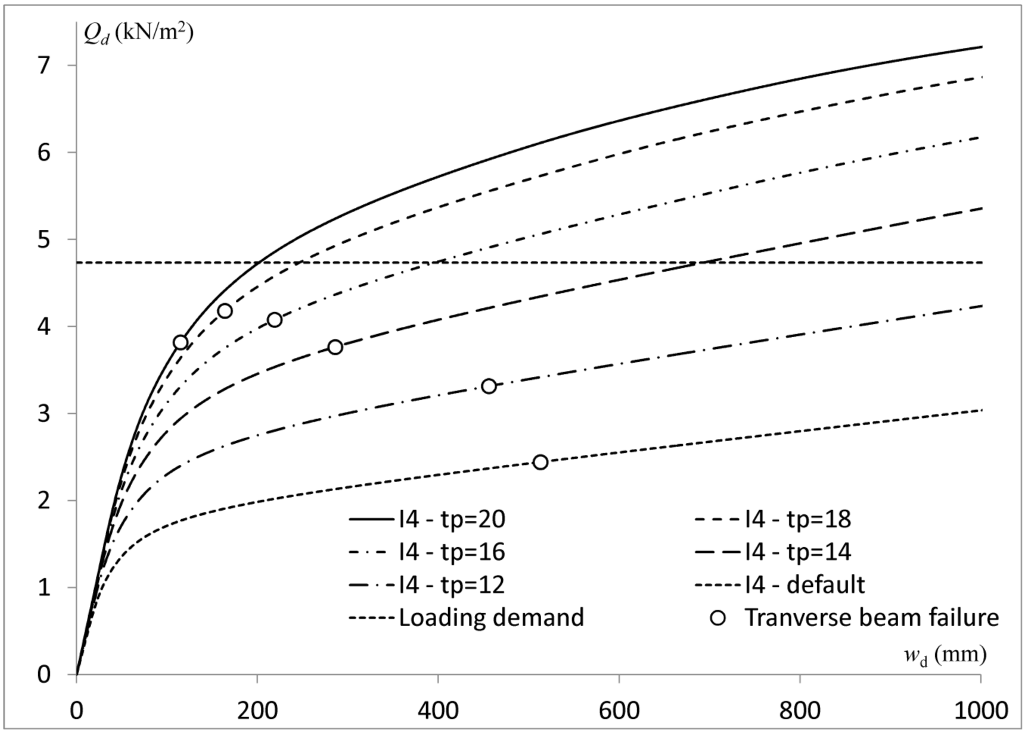


Figure 5-23: Q-w response of bare steel floor grillage assemblies for internal column loss scenarios

5.9 Enhanced floor grillage response for the composite arrangement

The progressive collapse analysis shows an increase in both ductility and resistance of the floor grillages using the new connection configuration, all of which now provide the required pseudostatic capacity.

Especially for scenarios I2 and I4 (Figure 5-24 and Figure 5-25), the enhanced ductility of the constituent beam systems (Table 5-12 and Table 5-3) is largely reflected on the floor grillage responses. For example, the substantial increase in the unrestrained beam system ductility (Table 5-13) has a clear impact on the response for the edge and corner removal scenarios (Figure 5-26 and Figure 5-27) and plays a key role in assisting the grillage meet the demand.

The most enhanced responses are those corresponding to column removal scenarios I2 (Figure 5-24), I4 (Figure 5-25), E2 (Figure 5-26) and C1 (Figure 5-27).

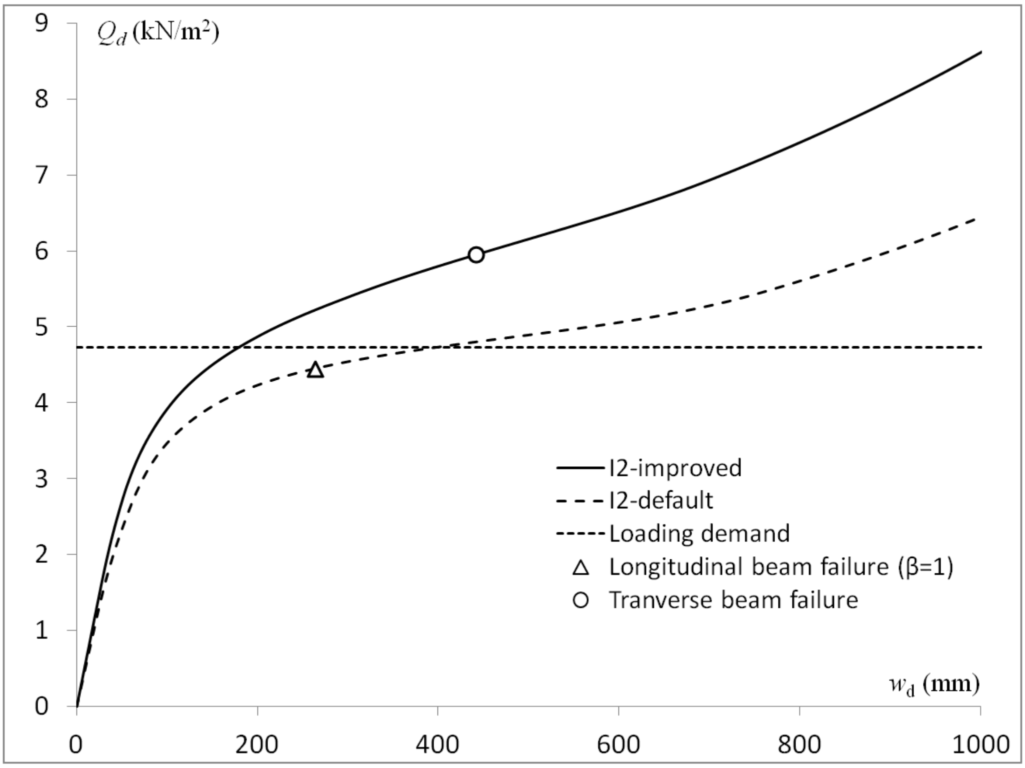


Figure 5-24: Q-w for the default and improved composite connection design for I2 column loss scenario

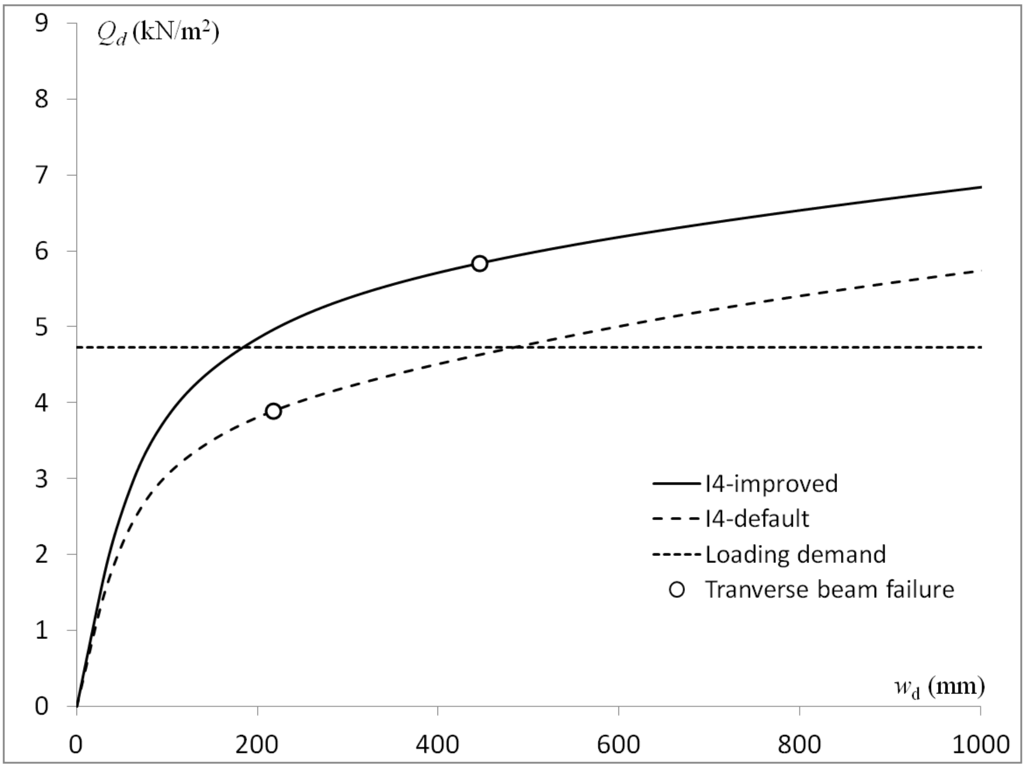


Figure 5-25: Q-w for the default and improved composite connection design for I4 column loss scenario

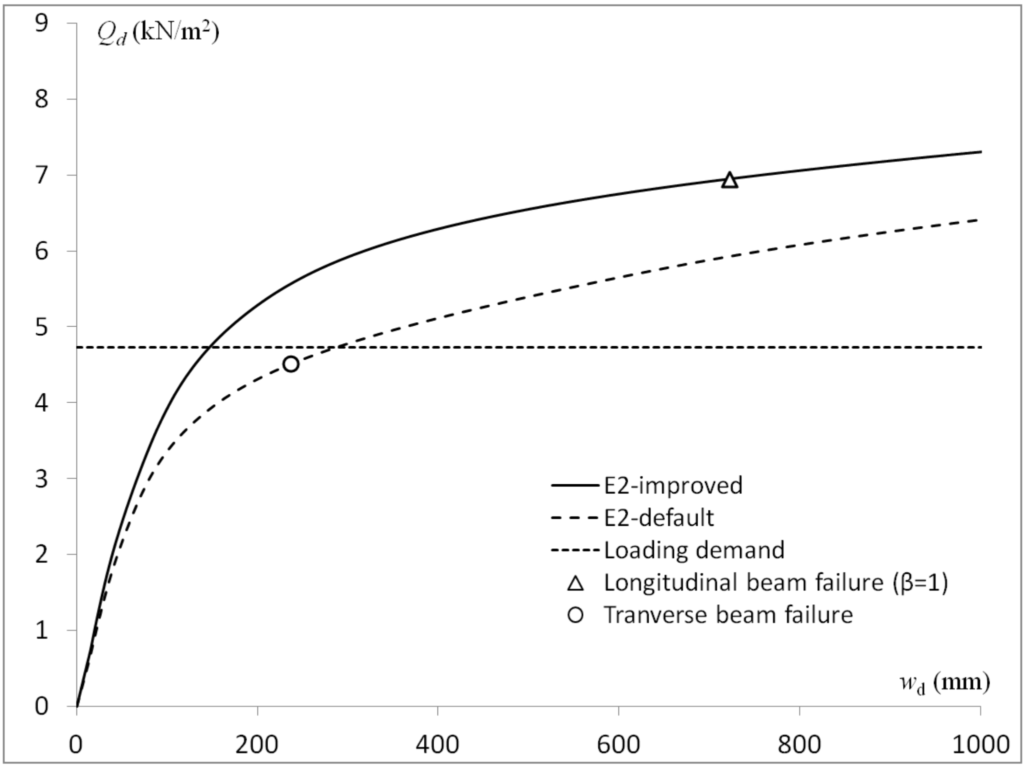


Figure 5-26: Q-w for the default and improved composite connection design for E2 column loss scenario

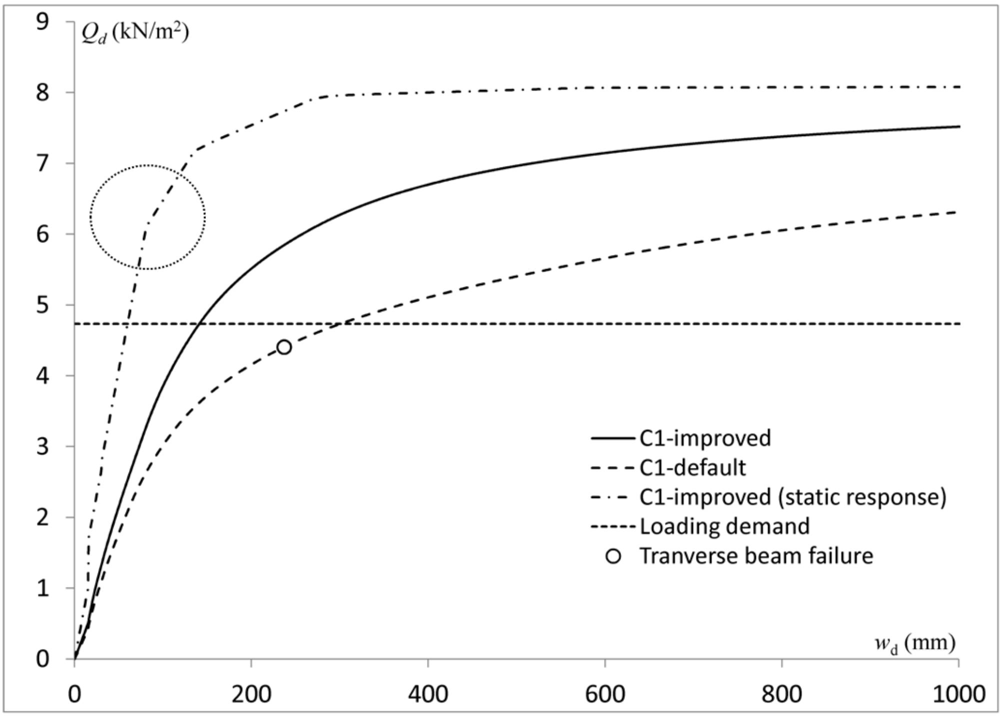


Figure 5-27: Q-w for the default and improved composite connection design for C1 column loss scenario

For the corner removal scenario C1 (Figure 5-27), the critical system is the cantilever beam and the critical component is the beam flange in compression. The first “knee” of the static response (highlighted within a circle), which tends towards the pseudostatic, defines the yielding point of the compressive components. Comparing this to the demand reveals that the system can provide the required pseudostatic capacity.

Table 5-16 shows the percentage gain in maximum capacity under sudden column loss of the improved configuration compared to the original configuration (see Table 5-15).

Table 5-16: Capacity-demand ratio and percentage of the gain in maximum capacity under sudden column loss of the improved configuration compared to the original

Scenario	Capacity-demand ratio, $r = q_{Rd}/q_{sd}$		Maximum capacity gain: %
	Original configuration	Improved configuration	
I1	1.09	1.27	+17
I2	0.94	1.26	+34
I3	0.82	1.23	+50
I4	0.90	1.25	+39
E1	1.08	1.43	+32
E2	0.95	1.47	+55
E3	1.25	1.70	+36
E4	1.02	1.55	+52
C1	0.93	1.09	+17

5.10 Comparison of the methodology with tying capacity provisions

5.10.1 Composite arrangement

Section 2.2.2.2 provides information about the tying capacity provisions in the UK for steel frame structures. The required tying capacity of the connections T_{Rd} is calculated based on Equation 2.1 and Equation 2.2. Table 5-17 shows that the provided connection tying capacity resistance considerably surpasses the provisions’ quota, especially for the composite arrangement.

Table 5-17: Tying capacity-demand ratio and additional required connection ductility

Beam system type			T_{sd}	T_{Rd}	T_{sd}/T_{Rd}
Transverse / Primary	Composite	Internal	668	263	2.54
		Edge	668	132	5.06
	Bare steel	Internal	319	263	1.21
		Edge	319	132	2.42
Longitudinal / Secondary	Composite	Internal	668	132	5.06
		Edge	668	75	8.91
	Bare steel	Internal	319	263	1.21
		Edge	319	132	2.42

Although the tying provisions are satisfied in all cases, most of the original floor grillage assemblies fail to provide the necessary pseudostatic resistance, which means that tying capacity cannot be used as a single measure for resistance in progressive collapse. This is also supported by recent studies (Vlassis A.G. et al., 2008, Stylianidis et al., 2009, Nethercot D.A. et al., 2011, Nethercot et al., 2010, Nethercot D.A. and Stylianidis P., 2011) which indicate that employing tying capacity provisions neglects the following:

- Influence of axial restraint on the compressive arching action, which affects the level of absorbed energy before the system enters the final catenary stage.
- Dynamic effects, which increase the system ductility requirements.
- Available connection rotational capacity, which may be exhausted before reaching the final catenary action phase.

For the semi-continuous axially restrained beams of this study, tying capacity is activated only under very large deflections within the range of 1.7D - 2.2D. This is significantly beyond the failure point by a difference in deflection of 0.4D - 0.9D, as shown in Table 5-18.

Table 5-18: Comparison of the failure point with initiation of tensile catenary stage

		D (mm)	Initiation of catenary (N = 0)		Failure point	
			w_i (mm)	w_i / 2D	w (mm)	w / 2D
Transverse / Primary	Composite	449.25	764	0.85	473	0.53
	Bare steel	446.90	830	0.93	656	0.73
Longitudinal / Secondary	Composite	449.25	896	1.00	500	0.55
	Bare steel	446.90	972	1.09	791	0.88

Increasing tying capacity is compared with the proposed method via the following process: either of the tensile components is enhanced without altering the other until the connection tying capacity becomes equal to that of the configuration determined in Section 5.8.2 (Table 5-15). The efficiency of the methodology is evaluated by comparing the floor grillage responses for the most critical column removal scenarios, which are I3, I4 and E2.

As shown in Figure 5-28 and more clearly in Figure 5-29, for approximately the same level of connection tying capacity, each connection configuration leads to a different response.

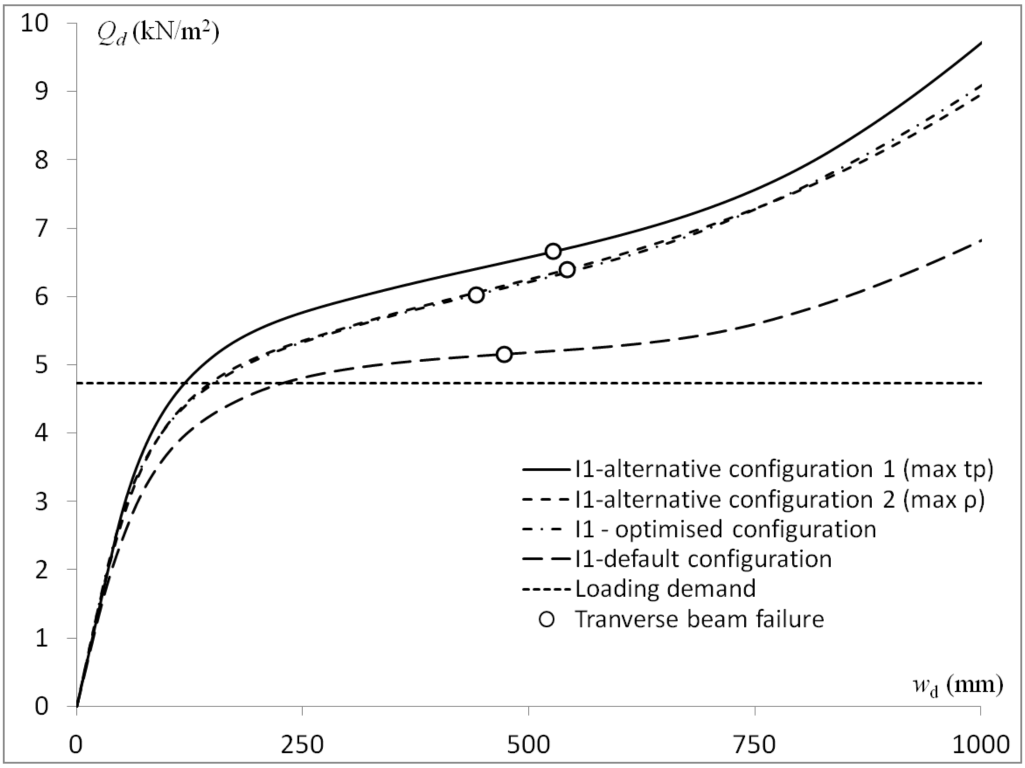


Figure 5-28: Q-w response comparison between alternative configurations for column loss scenario I1

The use of thicker endplates (max t_p) increases the connection tensile and bending resistance but its rotational capacity is still limited by either the reinforcement or the bolt row deformation capacity. For this reason, while it is almost equally efficient in improving the resistance for some scenarios e.g. I3, I4, E1 and E4 (Figure 5-28), it is less efficient for others e.g. I2, E2 and E3 (Figure 5-29). Thus, contrary to the optimised configuration, it only enhances beam system capacity and not ductility.

The endplate or the bolt row deformation capacity also constrains the maximum benefits of using just additional reinforcement (max ρ). Again, for scenario E2 in Figure 5-29, the lack of ductility causes a 15% negative difference in pseudostatic capacity compared to the system with the optimised connection configuration.

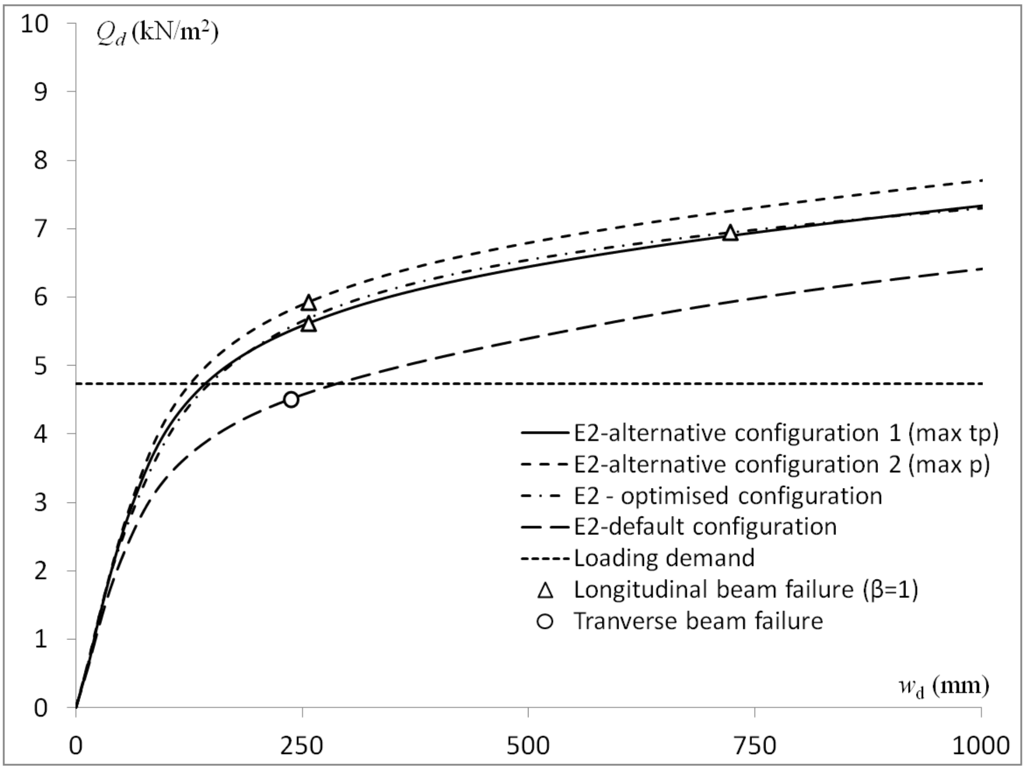


Figure 5-29: Q-w response comparison between alternative configurations for column loss scenario E2

Thus, the use of the proposed methodology can lead to a significantly more ductile and to some extent more resistant frame, compared to other arrangements with equal connection tensile resistance, as shown in Table 5-19. Moreover, the use of such arrangements, which constitute a "heavy connection design", might be impractical in construction and might conflict with the design provisions for other load cases.

Table 5-19: Comparison between connection designs with similar tying capacity

Scenario	Variation in floor ductility: %			Variation in floor pseudostatic capacity: %		
	Max. t_p	Max. ρ	ICL method	Max. t_p	Max. ρ	ICL method
I1	+13.5	+13.5	-6.5	+29.0	+24.0	+17.0
I2	-3.0	-3.0	+67.0	+25.0	+16.0	+34.0
I3	+146.0	+146.0	+105.0	+54.0	+41.0	+38.0
I4	+17.0	+17.0	+105.0	+42.0	+32.0	+50.0
E1	+127.0	+127.0	+150.0	+29.0	+36.0	+33.0
E2	+8.0	+8.0	+205.0	+24.0	+31.0	+54.0
E3	+4.0	+4.0	+77.0	+28.0	+15.0	+36.0
E4	+460	+46.0	+105.0	+58.0	+40.0	+52.0
Average	+45.0	+45.0	+101.0	+36.0	+29.0	+39.0

5.10.2 Bare steel arrangement

Similar to the composite frame, Table 5-18 and Table 5-20 show that tying capacity provisions are fully met for the bare steel arrangement. Nonetheless, its resistance in progressive collapse is still inadequate, as discussed in Section 5.7, even for the maximum endplate thickness considered in this study, as determined in Section 5.8.3.

In addition, the tying capacity and pseudostatic resistance supply over demand ratios presented in Table 5-21 suggest little if any correlation between the two; using a 20 mm instead of an 18 mm endplate increases average connection tying capacity by 7% but decreases grillage capacity for scenario I4 by 6%.

Table 5-20: Alternative connection configurations with similar tying capacity

	Connection configuration		Tying capacity		
			T_{sd}	T_{Rd}	T_{sd}/T_{Rd}
Transverse / Primary	Default configuration	ρ = 0.89% (4∅16), t_p =10 mm	668	263	2.54
	Methodology defined	ρ = 1.79% (8∅16), t_p = 12 mm	1160	263	4.41
	Increase in t_p only	ρ = 0.89% (4∅16), t_p =18 mm	1145	263	4.35
	Increase in ρ only	ρ = 2.23% (10∅16), t_p = 10 mm	1196	263	4.55
Longitudinal / Secondary	Default configuration	ρ = 0.89% (4∅16), t_p = 10 mm	668	132	5.06
	Methodology defined	ρ = 1.34% (6∅16), t_p = 10 mm	843	132	6.39
	Increase in t_p only	ρ = 0.89% (4∅16), t_p = 12 mm	806	132	6.11
	Increase in ρ only	ρ = 1.34% (6∅16), t_p = 10 mm	843	132	6.39

Table 5-21: T_{sd}/T_{Rd} and Q_{sd}/Q_{rd} for the bare steel non-continuous unrestrained beam systems

t_p (mm)	Beam system type	Tying capacity			Pseudo static capacity / demand for scenario I4 Q_{sd}/Q_{rd}
		T_{sd}	T_{Rd}	T_{sd}/T_{Rd}	
10	Transverse	320	263	1.2	0.51
	Longitudinal	296	132	2.3	
18	Transverse	804	263	3.1	0.90
	Longitudinal	796	132	6.0	
20	Transverse	860	263	3.2	0.85
	Longitudinal	855	132	6.5	

5.11 Response critical action phases for axially restrained systems

During a progressive collapse initiation scenario, such as sudden column loss, the pseudostatic response of semi-continuous beam systems benefits from a series of non-linear resistance mechanisms mentioned in Section 5.1.3, thanks to the presence of axial restraint at the boundary joints.

Each phase is influenced by different parameters due to the different nature of loading at the connections. The compressive arching action phase can provide a peak response for low deflections if the compressive components do not yield very early, allowing the further exploitation of the tensile components' deformation capacity, as has been discussed in Section 5.5. Failure within the transient catenary phase should be addressed by enhancing the tensile components' deformation capacity, which helps the connection develop sufficient bending moment resistance and rotational capacity to reach the tensile catenary phase. Past that point the resistance depends on connection tying capacity.

Thus, for the system to reach this final phase its connections must be very ductile; the corresponding rotational capacity is 110-120 mrad for the transverse beams of this study. Even if this is within the GSA (GSA, 2003) and the DoD (Department of Defense, 2005) guidelines' acceptance range (210 mrad) for nonlinear modelling of such connections, Table 5-21 shows that the connections for this study can only provide a maximum connection ductility of 76 mrad.

The capacity balance between the compressive and tensile components of the connections for each loading scenario is decisive in determining the critical action phase. It not only affects the connection rotational capacity but also the ultimate deflection corresponding to each action phase.

For example, in row 3 of Table 5-22, the use of enhanced support connection tensile components causes the compressive arching action phase to end for half the deflection level compared to the default configuration, due to the compressive components' early yielding. Subsequently, although the system ductility remains roughly the same ($w/2D = 0.49$ instead of 0.53), the failure point is now located within a different critical action phase, the transient catenary phase. Thus, changes in the connection components have a direct influence on the response critical action phase.

Table 5-22: Deflection levels and connection rotations for the critical action phases in axially restrained beam systems (italic font denotes the critical phase)

	Compressive arching			Transient catenary			Tensile catenary			Failure		
	$\frac{w}{2D}$	Φ' (mrad)	Φ (mrad)	$\frac{w}{2D}$	Φ' (mrad)	Φ (mrad)	$\frac{w}{2D}$	Φ' (mrad)	Φ (mrad)	$\frac{w}{2D}$	Φ' (mrad)	Φ (mrad)
Transverse (default connection configuration)	*0.02 to 0.66*	*2 to 95*	*1 to 94*	0.66 to 0.85	95 to 132	94 to 120	0.85+	132+	120+	*0.53*	*74*	*76*
Longitudinal (default connection configuration)	*0.03 to 0.72*	*1 to 66*	*1 to 67*	0.72 to 1.00	66 to 107	67 to 88	1.00+	107+	88+	*0.56*	*48*	*52*
Transverse (methodology determined configuration)	0.03 to 0.29	2 to 39	2 to 40	*0.29 to 0.78*	*39 to 118*	*40 to 109*	0.78+	118+	109+	*0.49*	*69*	*69*
Longitudinal (methodology determined configuration)	0.03 to 0.55	0.6 to 46	0.6 to 51	*0.55 to 0.93*	*46 to 95*	*51 to 83*	0.93+	95+	83+	*0.66*	*58*	*62*

5.12 Summary and conclusions

Based on the methodology presented in this chapter, it is possible to design a frame, otherwise prone to progressive collapse, in a way that it will be sufficiently robust to cope with any sudden column removal scenario. However, it is essential that:

- The critical mode of behaviour of the individual beam systems be identical in terms of ductility, otherwise the floor grillage response will not reflect the maximum resistance of the constituent systems.
- The connection critical component be realistically alterable, otherwise changes in other parameters will have a limited effect on the system pseudostatic capacity. Some components are more practical to modify, such as the endplate and the reinforcement, while others require more tenuous and complex work, such as the beam compressive flange stiffening.

Application of the methodology has revealed certain key observations:

- The balance of capacity between the connection compressive and tensile components is highly influential on its rotational capacity. If either component is too weak compared to the other, its premature failure limits the initial pseudostatic response.
- Axially restrained beam systems almost always exhibit an enhanced pseudostatic response compared to axially unrestrained ones. Thus, when choosing alternative configurations in order to improve floor response, priority should be given in increasing the ductility and - if possible - the capacity of the unrestrained and cantilever beam systems, so that the grillage can take full advantage of the compressive arching and catenary action of the axially restrained members.
- Increasing tying capacity does not have a direct and proportional effect on the frame's resistance to progressive collapse. In addition, most of the systems examined do not reach the tensile catenary action phase because of the extreme rotational capacity requirements at the connections.
- Whilst the response of the individual beam systems is of interest in order to understand which physical parameters influence the response to sudden column loss and how this is achieved, it is actually the response at the higher levels of structural idealisation, namely the floor grillage, that principally determines the resistance of the frame against progressive collapse.
- The column removal scenarios most sensitive to progressive collapse are not always the ones comprising axially unrestrained beams; internal columns, which usually comprise axially restrained systems, support larger areas and thus must provide an increased pseudostatic resistance.
- Bare steel arrangements are inherently less robust against a progressive collapse scenario because of the reduced connection resistance and initial stiffness.

The case study for the Cardington composite and its equivalent bare steel frame revealed that it is possible to determine common improvements to the connection configuration for all beam to column connections, instead of having to employ more than one configuration for each connection type in the frame, using the proposed methodology.

The proposed methodology is capable of highlighting the system weaknesses and efficiently remediating the simplified frame's robustness by taking into account on a case by case basis the needs for ductility, pseudostatic capacity and tying capacity of the connections. Compared to simply increasing tying capacity, which does not involve an adaptive process other than meeting certain quotas in connection and beam tensile resistance, it provides a significantly more ductile, lighter (in terms of connection component size) and to some extent more resistant frame.

Chapter 6
Progressive collapse resistance of steel moment resisting frames

6.1 Introduction

Framing systems designed according to the concept of 'simple construction' may be expected to occupy a particular space in the spectrum of behaviour for all framing types. In order to gain some indication of the behaviour in a progressive collapse situation of a conceptually different system, the methods outlined in Chapter 5 are employed in the study of moment frames, the use of which as the primary means of providing seismic resistance to steel and composite buildings is well established.

Study of actual incidences of progressive collapse, together with forensic investigations conducted in an attempt to explain the mechanics, have indicated that several phenomena not normally incorporated in studies of structural behaviour or utilised as the basis for structural design are often involved. Equally, behaviour under different sets of circumstances, e.g. seismic events, also provides helpful indications of the response of particular components when subject to unusual demands.

As the literature review in Section 2.5.2.5 and Section 3.1.3 highlights, assessment of these systems to potential progressive collapse is much less well understood in the general sense of how effective meeting the seismic requirements might be in terms of, inherently, providing substantial robustness. Thus, the aim of the present study is to:

- Examine the resistance mechanisms of continuous beam systems under progressive collapse loading conditions.
- Identify how the balance between the strength, stiffness and ductility and the provision of different combinations of these properties in beam to column connections through devices such as connection reinforcement and the use of reduced beam arrangements may influence behaviour.
- Study the interaction between the continuous and non-continuous systems of the structure, as well as compare their performance and behaviour.
- Investigate whether these structures, despite the perception of their superior performance, are potentially vulnerable to any column loss scenarios.

Therefore, a set of representative frames - the NIST (NIST, 2011) and SAC (FEMA-355C, 2000) moment frames - are assessed using the simplified Imperial College London Method and the results are presented in Section 6.5 and in Section 6.6 respectively. The response analysis for the lower levels of structural idealisation - the beam systems - but also for the floor grillage assemblies, will help identify the main influencing factors and critical resistance mechanisms in a progressive collapse scenario.

Further insight is gained by comparing the behaviour of different moment frame arrangements; these findings are discussed in Section 6.7. Section 6.8 compares the behaviour in progressive collapse with that of steel frames that constitute common UK practice and are therefore not designed to resist an earthquake. The increase in connection stiffness and strength strongly influences which factors control resistance and which priorities should be considered in order to design a more robust frame, which institutes the focus of the next chapter.

6.2 Study layout

The study employs the simplified Imperial College London Method to examine the ability of five model moment steel frames to withstand sudden loss of a ground floor perimeter column, which is the scenario deemed most likely (DoD, 2009). Study of the response of the subsystems, which form the floor grillage assemblies, is essential in identifying the main influencing factors and critical resistance mechanisms. Although the behaviour of the floors at the interior of the frame designed to resist gravity loads is not the principal focus of the current study, an approximation of their behaviour - largely based on the findings of a relevant study [a] (Oosterhof, 2013) - is presented in Appendix C.

Motivation for choosing the prototype structures is based on them being considered representative for different regions of seismic vulnerability. For example, the Los Angeles, Seattle and Boston 9-storey post-Northridge SAC project frames were designed for very high, high and average seismic vulnerability regions respectively. They employ coverplate connections and relatively deep beam sections. A summary of the frames examined is provided in Table 6-1.

[a] Dr. Oosterhof visited Imperial College London in 2013 in order to conduct joint research with Professor D.A. Nethercot. During the course of his stay as a visiting scholar, it was possible to collaborate within the frame of the current study in order to extend the application of the ICL Method to shear tab connections and also gain an understanding on the behaviour of these connections under progressive collapse loading conditions.

Table 6-1: Connection type and frame arrangement for the moment frames examined

Type	Name	Connections	Frame arrangement
SMF	“SDC-D” (NIST project)	Reduced Beam Section (RBS)	Perimeter connections: moment resisting Exception: 4 simple connections at one side of the corner columns; Figure 6-2
	“Los Angeles” (SAC project)	Reinforced with cover plates or RBS	*Same as above*
IMF	“SDC-C” (NIST project)	Welded unreinforced flange bolted /welded	Perimeter connections: moment resisting Exception: 8 simple connections at the corner and penultimate columns; Figure 6-3
	“Seattle” (SAC project)	Reinforced with cover plates	*Same as above*
OMF	“Boston” (SAC project)	Welded unreinforced flange bolted /welded	*Same as the SMF*

6.3 Moment resisting frames in the USA and EU construction practice

In the USA, private and federal structures’ construction is usually governed by different codes and standards. Moreover, private construction is controlled by the National Standards and the building codes adapted in each state. The most common one is the International Building Code (IBC, 2012). According to the requirements of frame ductility and toughness, bare steel and composite moment resisting frames are split in three classes: i) Ordinary moment-resisting frames (OMF); ii) intermediate moment-resisting frames (IMF) and iii) special moment-resisting frames (SMF). More information on relevant seismic provisions is available to the designer in ANSI-AISC 341-5 “Seismic Provisions for Structural Steel Buildings” (AISC, 2005). Federal buildings on the other hand, are controlled by the General Services Administration requirements (GSA, 2003), which prevail over the National Recognised Codes, although they usually overlap. All new buildings are classified as Category II structures according to Table 1604.5 of IBC. Material specifications follow the ASTM standards. For steel seismic design, FEMA publications (FEMA-350, 2000) are used.

Most braced frame construction is of structural steel (bare steel); exceptions include examples of concrete-braced framed frames in taller buildings designed to resist wind loads. Also, use of fully welded connections is more popular than bolted ones. The main energy dissipation mechanisms of structures with the latter arrangement include flexural yielding of beam end-plates and shear yielding of the column web panel zone.

In the EU, most structures designed according to the Eurocode (EN 1998-1, 2003) follow the capacity design approach and their behaviour is analysed using non-linear push-over analysis. They are categorised in ductility classes according to their dissipative structural behaviour:

i. Low dissipative behaviour (DCL)
ii. Average dissipative behaviour (DCM)
iii. High dissipative behaviour (DCH)

Also, Section 6 of EC8 identifies the following types of moment resisting frames:

i. Moment resisting (with or without concentric bracing), which can be ductile but usually have low lateral stiffness (prone to damage in high storey drifts)
ii. Concentrically braced, which can be stiff but can suffer from significant loss of ductility in case of buckling of the compression braces.
iii. Eccentrically braced, which can be stiff as well as ductile, however they require careful design of shear and / or bending links.

Figure 6-1 illustrates the two basic approaches to the provision of resistance horizontal loading in a multi-storey frame, the use of bracing or reliance on frame action, in simple diagrammatic form. Bracing may take the form of service cores, shear walls or actual braced bays, whilst frame action relies on the provision of moment connections between beams and columns. The two systems are often referred to as 'simple construction' and 'continuous construction'. Of course, other arrangements - often involving ingenious combinations of the two principles - are possible; indeed, it is one of the main challenges for the designers of tall buildings to find efficient ways to develop adequate lateral stiffness within their structures.

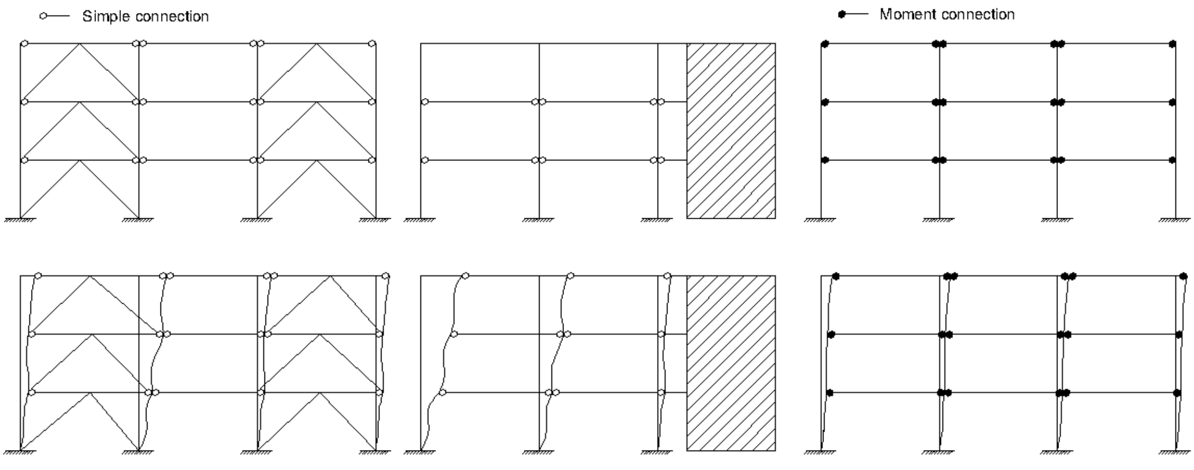


Figure 6-1: Bracing and frame action to resist sway

6.4 Modelling of moment resisting connections for progressive collapse

An essential prerequisite has been to extend previous provisions to consider connection behaviour under bending and substantial axial forces as well as the influence of the support joints. Using these analytical solutions, connection models with the Component Method were constructed for partially restrained (Stylianidis, 2011) and fully-restrained (Vidalis and

Nethercot, 2013a, Vidalis, 2014) connections, thus permitting the accurate capture of structural behaviour, as well as the running of extensive parametric tests to identify the critical components and their influence and to compare the merits of alternative connection designs based on their response in progressive collapse. Chapter 3 provides a detailed presentation of the connection models employed in this study.

6.5 Progressive collapse resistance of the NIST prototype frame structures

6.5.1 Introduction

In 2002, the USA National Institute of Standards and Technology launched a detailed investigation into the sequence of events leading to the WTC collapse (Section 1.1.3.4.2). Five years later, this was succeeded by a long-term project towards understanding and enhancing structural robustness. In 2011, the Materials and Structural Systems Division launched the Measures of Building Resilience and Structural Robustness Project (NIST, 2011). Its aims were to produce a research roadmap, best practice guidelines for assessing building resilience, a cost/benefit analysis for design or rehabilitation and computational methodologies to evaluate the progressive collapse potential of building structures, based on experimental and numerical case studies of subsystems and multi-storey frames.

6.5.2 Prototype structures

The 9-storey NIST structures model a typical special moment and an intermediate moment frame designed to resist very strong (SDC-D zone: Seattle, Washington) and strong (SDC-C zone: Atlanta, Georgia) earthquakes respectively. The SMF employs reduced beam section (RBS) connections while the IMF employs welded unreinforced flange bolted (WUF-B) connections. A summary of the frame and structural elements' design, available in Section 4 of the NIST report (Sadek F. et al., 2010), is presented in Table 6-2. Ground floor column and girder sections are considered.

In the frame layout in Figure 6-2 and Figure 6-3, column removal scenarios are indexed based on their position: "E" denotes a column at the "edge" or perimeter of the frame, "C" at the "corner" and "y" or "x" defines the direction of moment resisting beam systems that will support the bay with the lost column. C1 columns are connected with the beams with simple connections in the x direction and with moment connections in the y direction while C2 columns have the inverse connection configuration. The average dead floor and ceiling load is 3.64 kN/m^2 and the average live load is 4.79 kN/m^2.

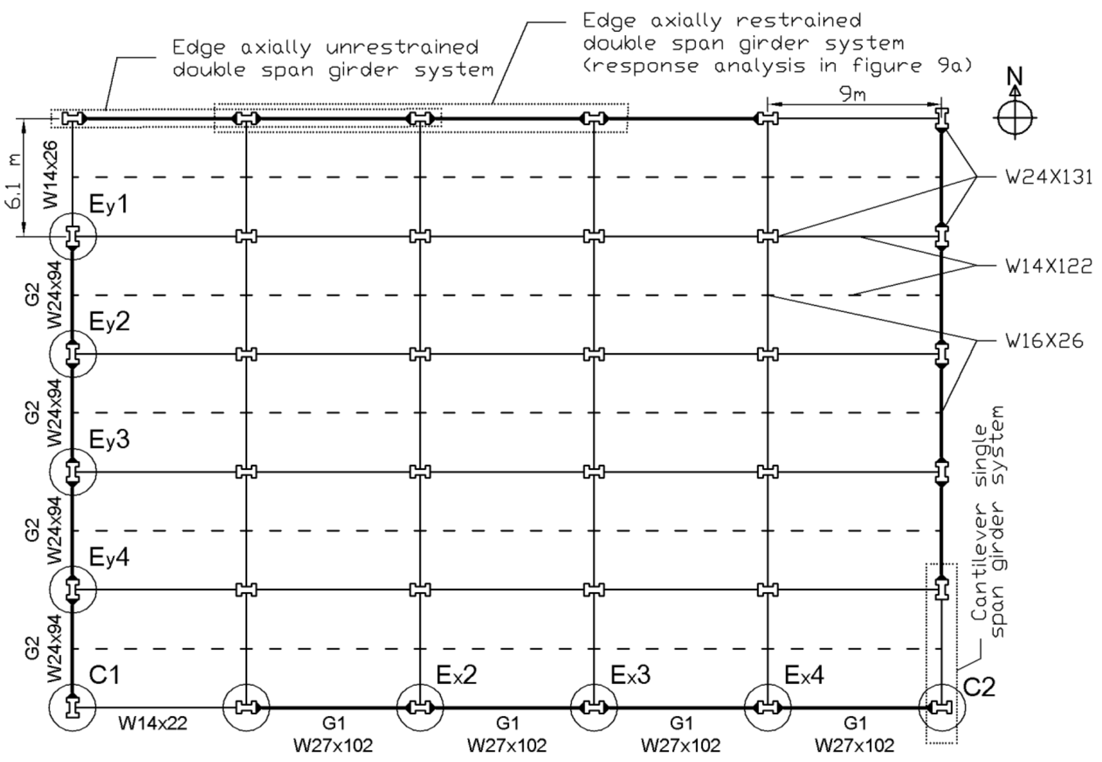


Figure 6-2: Plan layout for the SDC-D building with RBS connections; column and beam sections correspond to the ground floor

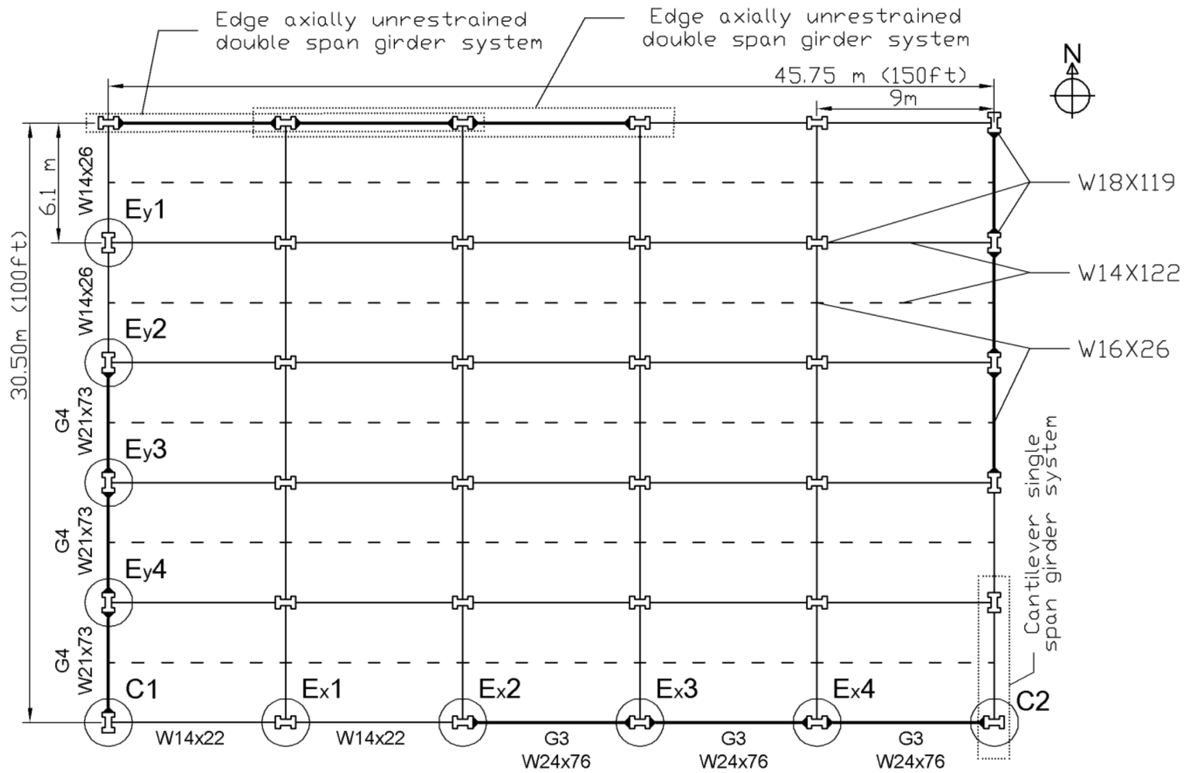


Figure 6-3: Plan layout for the SDC-C building with WUF-B connections; column and beam sections correspond to the ground floor

Table 6-2: Perimeter frame section and material information

Frame	Member	Section	Connections	Continuity plates [b] t_{cc} (mm)	Doubler plates t_{wc} (mm)	L (m)	L/D
SDC-D	Columns	W24x131		22.0	14.3	5.00	
	G1 girder	W27x102	RBS with 50% reduction [a]			9.14	13.3
	G2 girder	W24x94				6.09	9.9
	Typical x-x	*W16x26*	*Simple*			*9.14*	
	Typical y-y	*W14x22*				6.09	
SDC-C	Columns	W18x119		19.0		5.00	
	G3 girder	W24x76	WUF-B			9.14	15.2
	G4 girder	W21x73				6.09	11.5
	Typical x-x	*W16x26*	*Simple*			*9.14*	
	Typical y-y	*W14x22*				6.09	

[a] RBS connection dimensions: see Figure 3-4 and equations 3-11, 3-12 and 3-13.

[b] A36 grade steel is used for continuity plates

6.5.3 Response of beam systems with moment resisting connections

6.5.3.1 Analysis results for beam systems with RBS connections

The perimeter girder systems' response for different column removal scenarios in the special moment frame (SDC-D) is shown in Figure 6-4 and in Figure 6-5 for the longitudinal and the transverse girder systems G1 and G2 respectively. The pseudostatic load is divided over the length of the beam system and plotted against the deformation over beam depth ratio in order to allow the easy comparison of the two sets of responses.

Although double span systems remain in the elastic phase only for low deflections ($w/D \approx 0.1$), the higher stiffness of the shorter and deeper G2 beam system enhances its response at this early stage by approximately 150% compared to the G1. For the same reason, the ultimate pseudostatic capacity of the G2 is 85% higher than that of the G1, despite the fact that the latter is 25% more ductile on average. A similar observation can be made for the response of the single span cantilever system: the ultimate capacity of G2 is double that of G1.

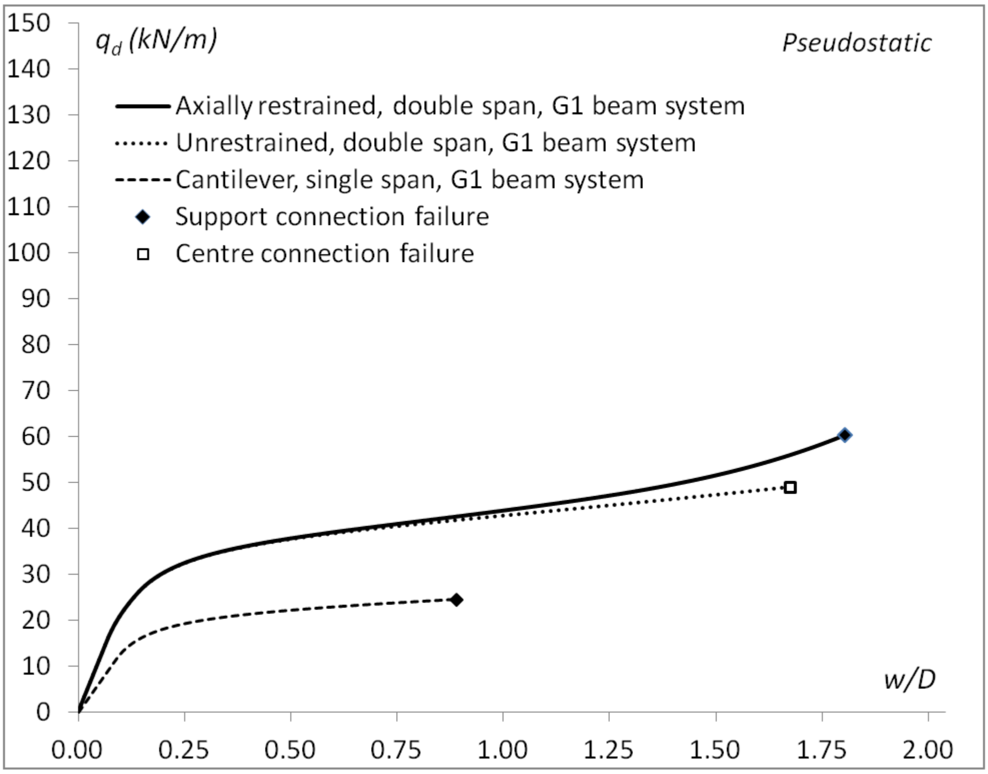


Figure 6-4: q-w response of the G1 beam systems with RBS connections (SCD-D frame)

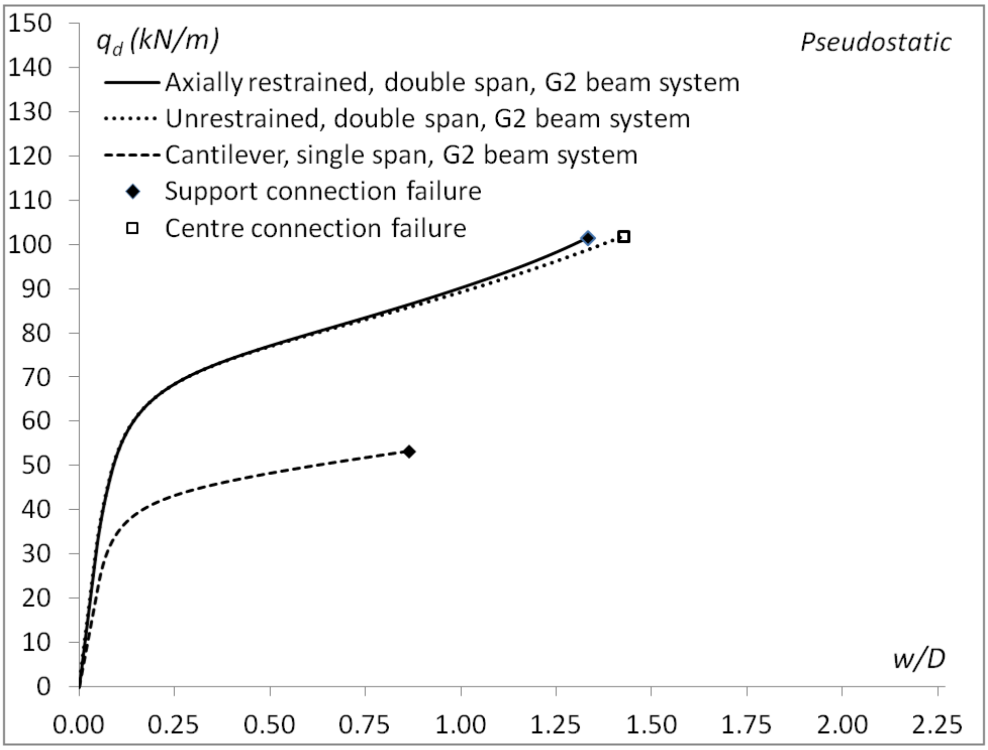


Figure 6-5: q-w response of the G2 beam systems with RBS connections (SCD-D frame)

The RBS connections in axially restrained beam systems fail after rupture of the reduced flange in combined tension and bending. The absence of axial restraint influences the location of the critical connection: failure initiates at the top flange of the support as opposed to the bottom flange of the centre connection for unrestrained systems. Figures 6-6a and 6-6b show the difference in horizontal displacement of the support connection due to the "pull-in" effect described in Section 5.2.2. This displacement is larger for axially unrestrained connections and can reduce tensile strains at the support connection flange while increasing those at the centre connection as the deformed system is pulled in at the point of the removed column.

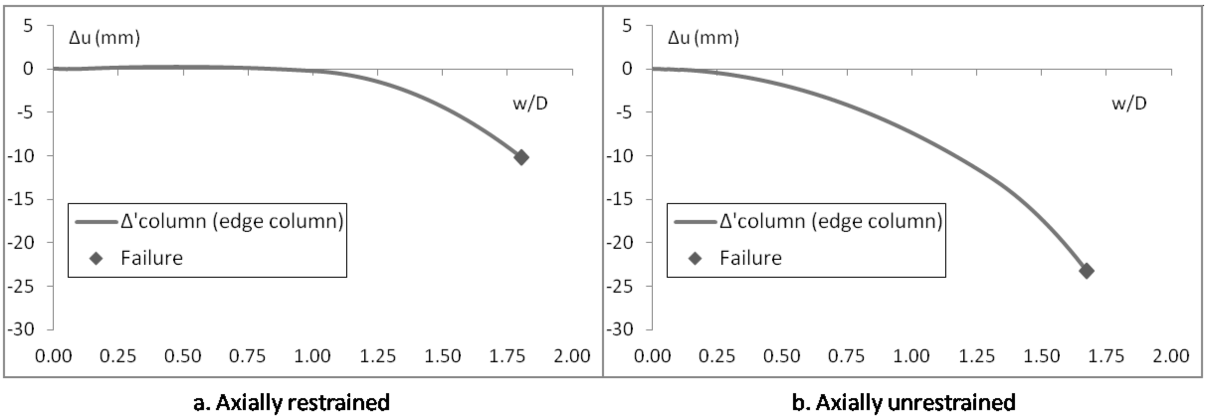


Figure 6-6: Horizontal displacement of the support connections in the G1 beam system (SDC-D frame) due to "pull-in" effect

The influence of axial restraint is limited by the rotational capacity of the welded connection, which is not adequate to allow the system to exploit the tensile catenary phase. The evolution of connection loading during the different action phases described in Section 5.2, is shown in Figure 6-7; the system fails before important tensile forces can develop at the connections.

Single span cantilever girder systems are vulnerable in inelastic local buckling of the flange in compression for relatively low levels of centre column vertical deflection. Table 6-3 summarises the relative ductility (w/D) of all SDC-D beam systems.

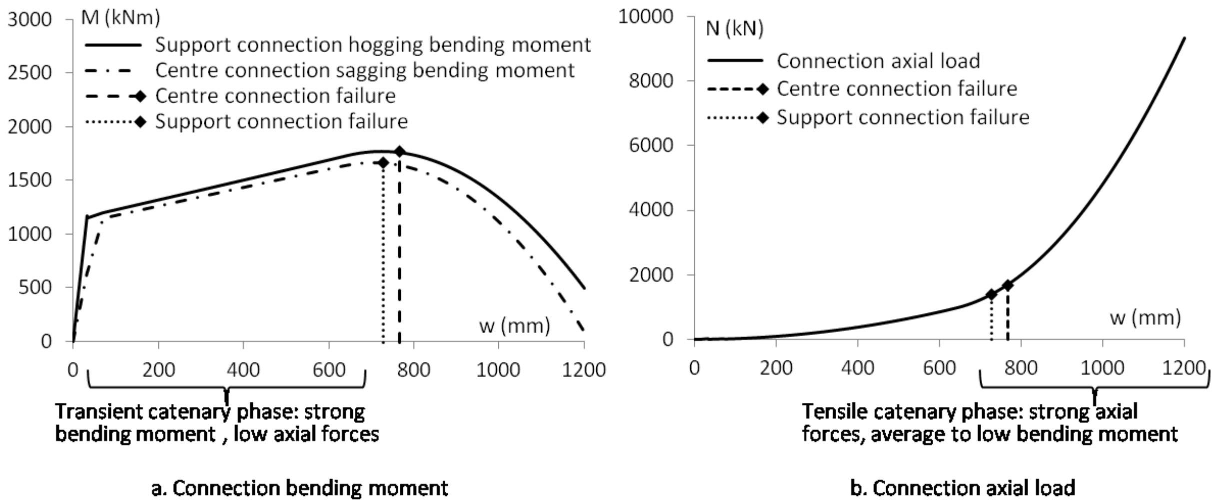


Figure 6-7: Connection loading and resistance mechanisms for the double span G1 girder systems

Table 6-3: Relative ductility for six main girder systems; NIST SDC-D frame

Beam system type	Corresponding column loss scenario in Figure 6-2	w / D	
		G1	G2
Restrained double span	Ex2, Ex3, Ey2, Ey3	1.80	1.34
Penultimate / unrestrained double span	Ex4, Ey4	1.68	1.44
Cantilever	C1, C2	0.90	0.86

6.5.3.2 Parameter sensitivity: beam systems with RBS connections

Although the frame assessment considers fixed values for certain parameters, such as beam length, section size and connection geometry, others, such as the degree of axial restraint and support joint bracing and the material properties (steel ultimate tensile strain and post-yielding resistance), are considered either in approximation or with a certain level of uncertainty. In both cases, theoretical values may differ from those in a realistic construction site. Thus, it is important to determine the sensitivity of the response analysis, in order to ensure that the modelling exercise is able to provide a lower bound for resistance against progressive collapse. It is also important to validate the assumption that increased sensitivity to any of these parameters will not affect the qualitative nature of the conclusions. Detailed results are available in Appendix D; the main conclusions are presented below.

According to the sensitivity analysis results in Figure 6-8 and Figure 6-9, axial restraint and in-plane bracing (see Section 3.1.6) are less influential on the response in the case of very stiff connections. On the contrary, material properties appear to have a direct and proportional effect on system response. Figure 6-10 illustrates how steel grade directly influences maximum capacity and Figure 6-11 demonstrates that the model's prediction of maximum ductility and capacity is very sensitive to the failure criteria employed (steel maximum allowable strain), at least for welded connections.

Table 6-4 presents the average model sensitivity to the degree of axial restraint, bracing, ultimate tensile stain and steel resistance. Negative percentages denote an inverse relationship. For example, a system ductility sensitivity of -5% to the degree of axial restraint for column loss at the interior of the frame perimeter means that a 100% increase in the degree of axial restraint will reduce system ductility at an average of 5% for this frame.

Results show that material properties have a substantial influence on system response, with ε_{max} having the largest impact on ductility and steel resistance having the largest impact on capacity. For this reason, the values used in this study are justifiably taken as the minimum required from construction codes, in order for the model to be able to offer a lower bound of the frame's resistance to progressive collapse. However, if the model is to be used in the future for predicting the behaviour of an experimental test assembly, the precision with which material properties will be used as input will affect its accuracy.

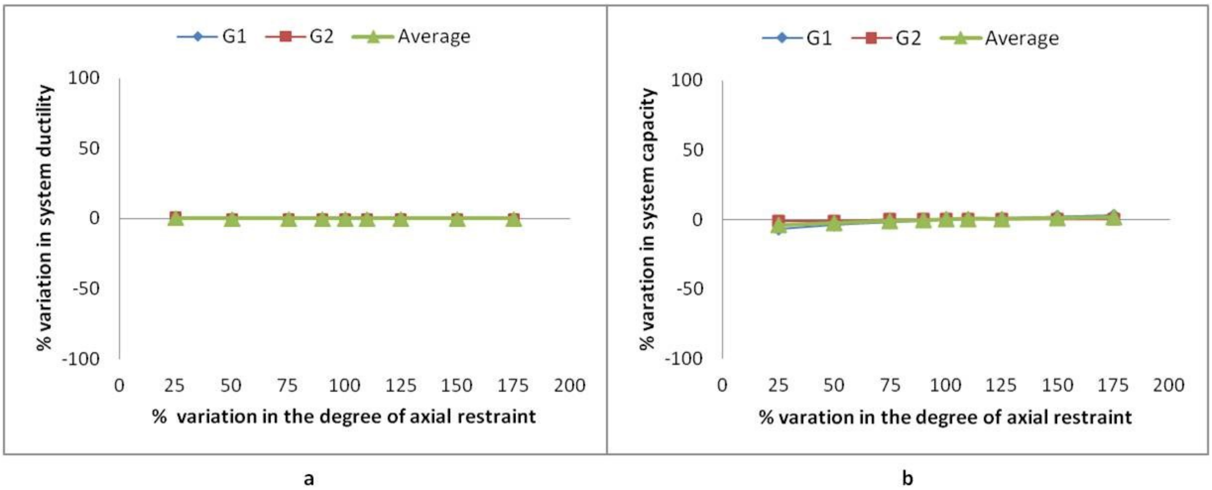


Figure 6-8: Axially restrained beam systems with RBS connections; sensitivity to axial restraint

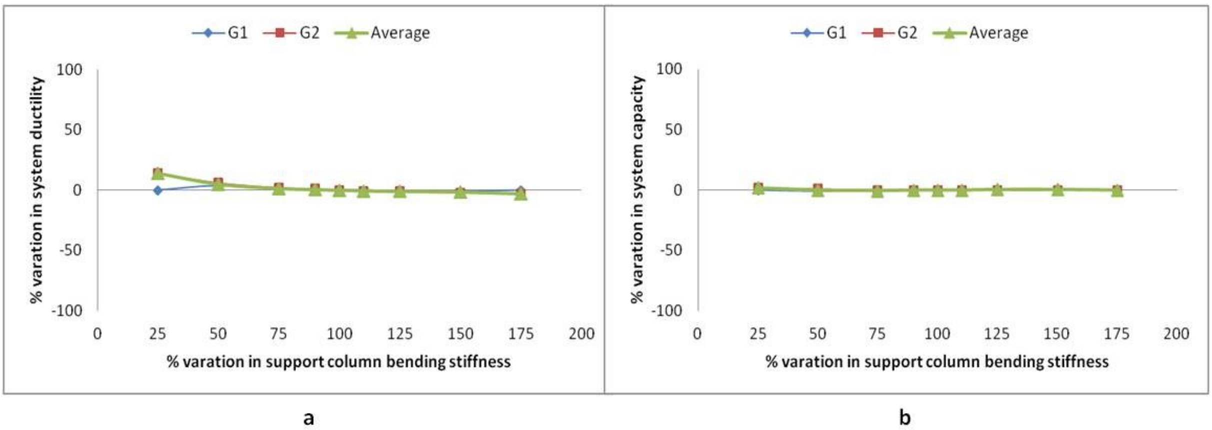


Figure 6-9: Unrestrained beam systems with RBS connections; sensitivity to support column bracing

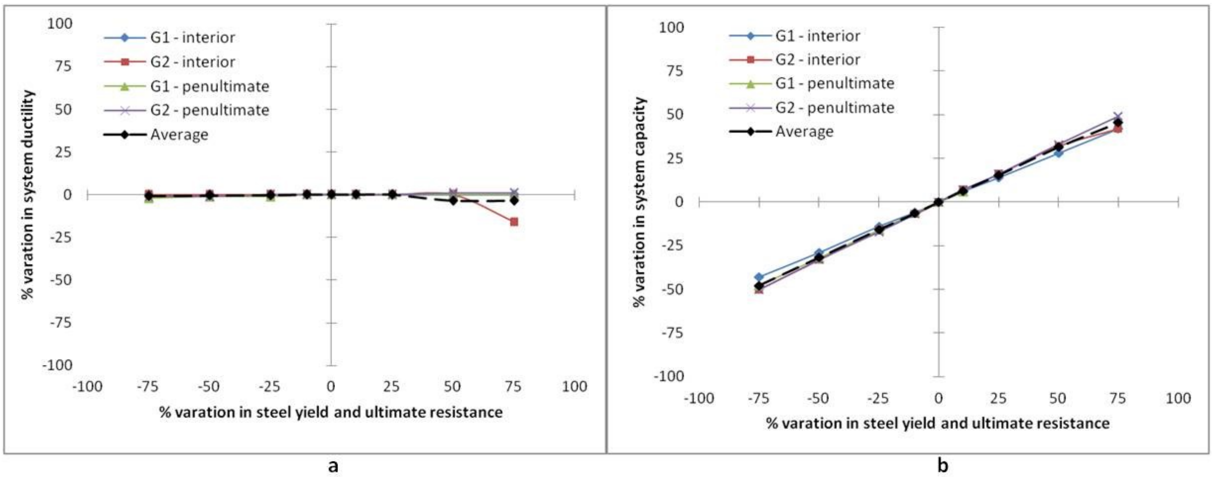


Figure 6-10: Axially restrained and unrestrained beam systems with RBS connections; sensitivity to steel yield and steel ultimate resistance

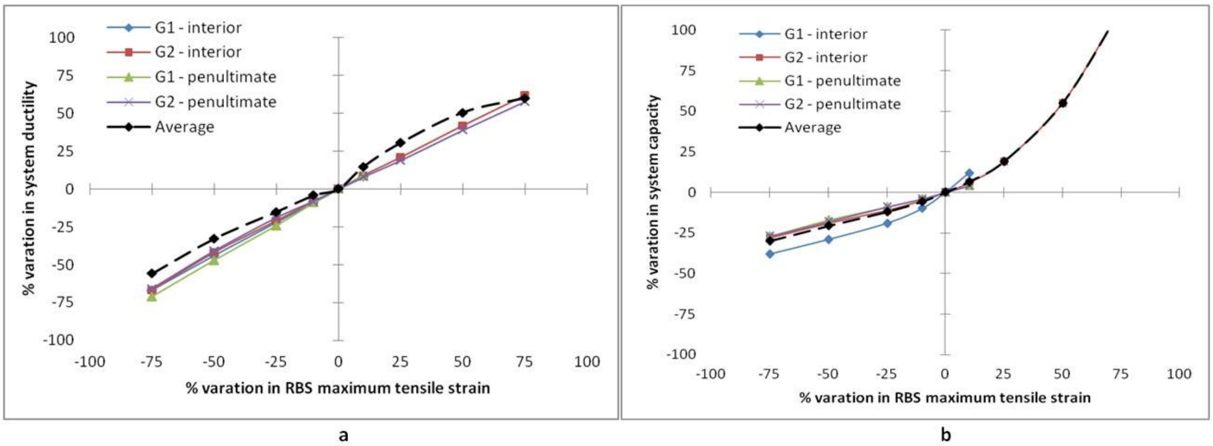


Figure 6-11: Axially restrained and unrestrained beam systems with RBS connections; sensitivity to steel maximum allowable strain

Table 6-4: Sensitivity of beam model to variations in influencing parameters for the NIST SDC-D frame

Parameter	% change	
	Ductility	Capacity
Degree of axial restraint (interior column removal)	*Insignificant*	+4%
Bracing level (edge column removal)	-7% [a]	*Insignificant*
Ultimate tensile strain (ε_u) before RBS flange rupture	+86%	+60%
Steel resistance (f_y and f_u)	+0%	+63%

[a] Negative percentages denote an inverse relationship

6.5.3.3 Analysis results for beam systems with WUF-B connections

The response of the perimeter girder systems upon loss a column is shown in Figure 6-12. In order to compare its complete spectre, Figure 6-13 illustrates the theoretical case in which the beam system response continues past the failure point. Comparison of the response of G3 and G4 beam systems suggests that the geometry of the system influences maximum capacity.

Furthermore, results show that axial restraint has a small influence on system capacity and that the compressive arching action phase is negligible. However, the increased ductility of axially unrestrained systems in the transient catenary phase appears to counter balance the absence of this action phase in terms of maximum capacity.

For double span beam systems of the SDC-C frame, failure was caused by tensile rupture of the flange in tension at the connection. The critical connection for axially restrained systems was located at the supports. However, in the absence of restraint, the “pull-in” effect at the supports lightly reduces local strains, leading the centre connection to exhaust its rotation capacity first in this case study. In the case of the cantilever systems, the flanges in compression fail in inelastic local buckling very soon after bolt rupture is detected.

Table 6-5 summarises the relative ductility (w/D) of the beam systems examined. The comparison with Table 6-3 suggests that the WUF-B arrangement is less ductile compared to the RBS one. A more thorough comparison is presented at the end of this chapter. The component loading and unloading sequence follows a similar pattern as that for the RBS arrangement presented in Figure 6-7. However, due to the fact that the WUF-B connection is less ductile, the contribution to system capacity of the tensile catenary phase is less than 10% of the ultimate resistance.

Table 6-5: Relative ductility for six main girder systems; NIST SDC-C frame

Beam system type	Corresponding column loss scenario in Figure 6-3	W / D	
		G3	G4
Restrained double span	Ex2, Ex3, Ey2, Ey3	1.38	1.04
Penultimate / unrestrained double span	Ex4, Ey4	1.86	1.56
Cantilever	C1, C2	1.02	0.64

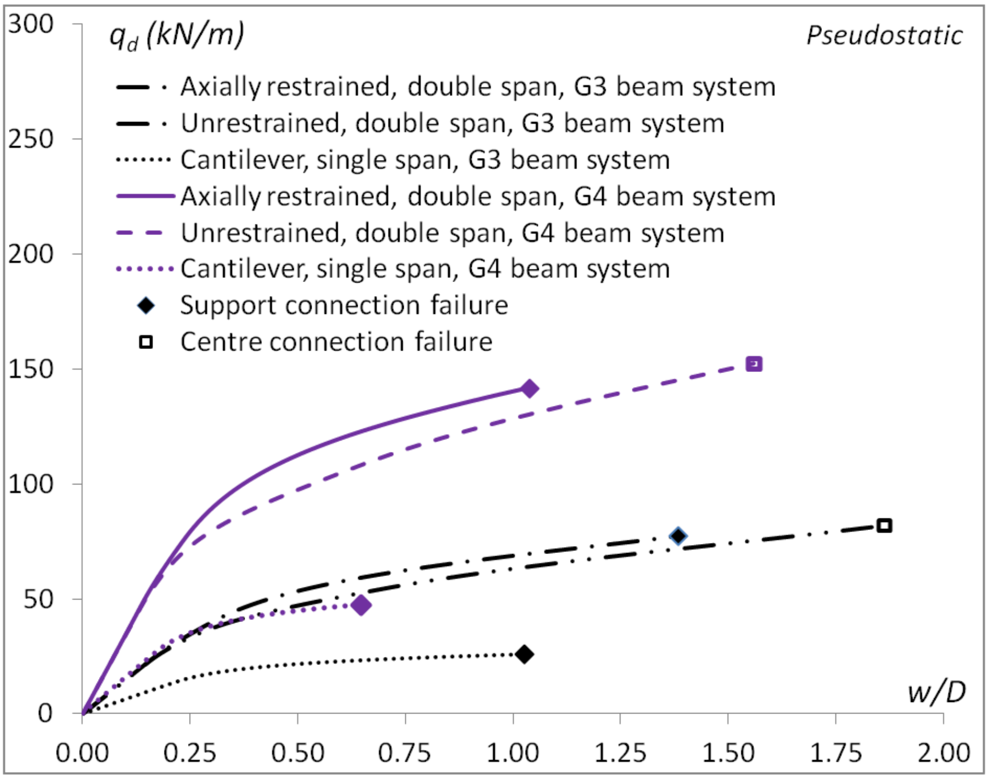


Figure 6-12: q-w response of the G3 & G4 beam systems with WUF-B connections (SCD-C frame)

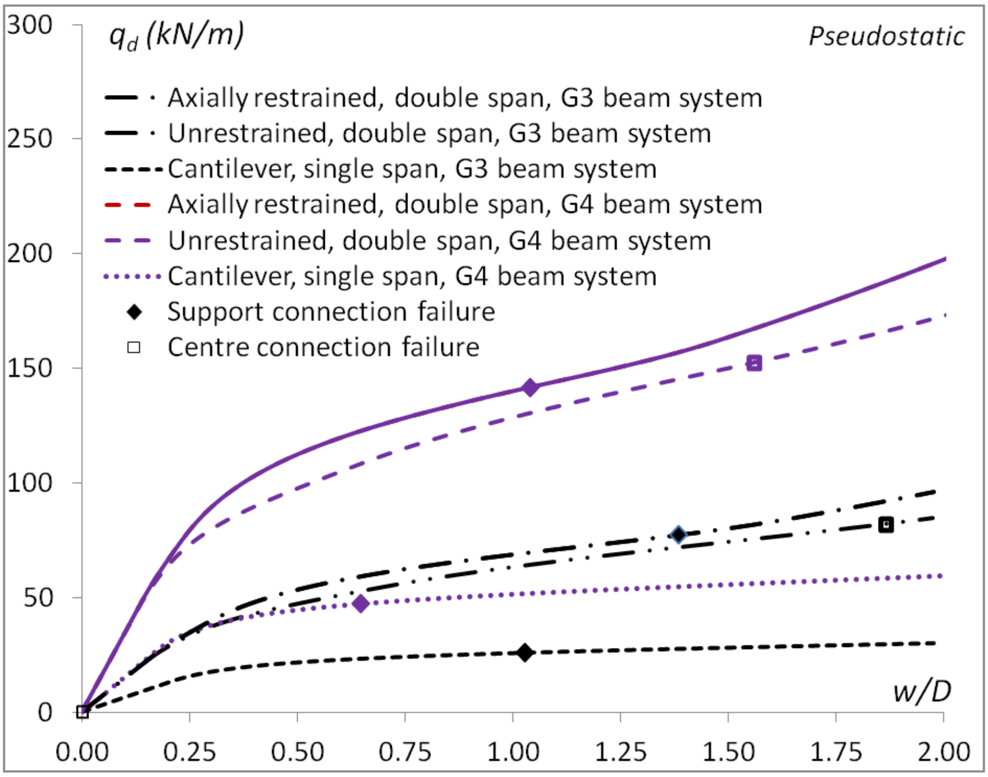


Figure 6-13: Theoretical response of the G3 & G4 beam systems with WUF-B connections (SCD-C frame)

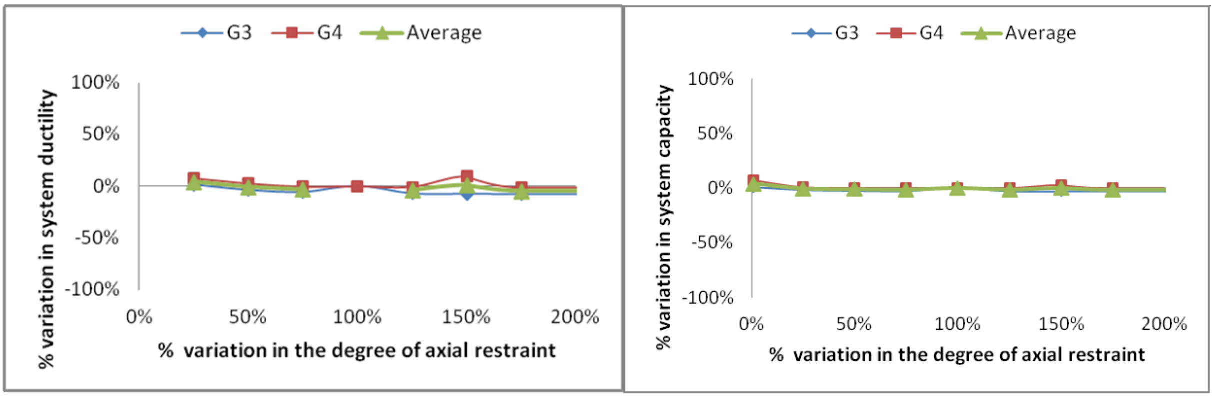
6.5.3.4 Parameter sensitivity: beam systems with WUF-B connections

An additional set of parametric studies is conducted in order to evaluate the sensitivity of the beam system model with WUF-B connections to the most pertinent parameters. Since the WUF-B design does not offer any practically alterable components, the range of examined parameters will be expanded to include, apart from the material properties and the degree of axial restraint, the beam section, length and length to depth ratio.

Although the influence of the degree of axial restraint is small (Figure 6-14), axially restrained systems are less sensitive to changes in other parameters related to the frame layout, such as the beam depth, length and their ratio, as shown in Table 6-6.

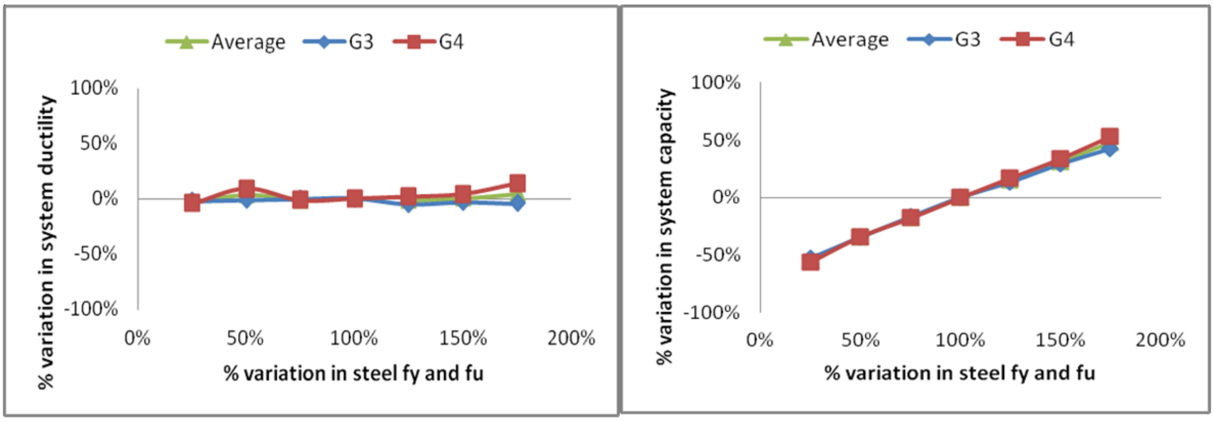
Material properties have a highly influential role on the response of beam systems with fully welded unreinforced connections and the relationship is similar to that observed for the beams with RBS connections. Although steel strength does not directly affect ductility, it has a direct and proportional impact on capacity, as shown in Figure 6-15. On the contrary, Figure 6-16 shows that the steel ultimate strain directly affects the ductility of the system and might also influence capacity.

All beam systems are directly affected by changes in beam geometry. The results presented in Figures 6-17, 6-18 and 6-19 illustrate that deeper and shorter beams behave better than longer and shallower ones.



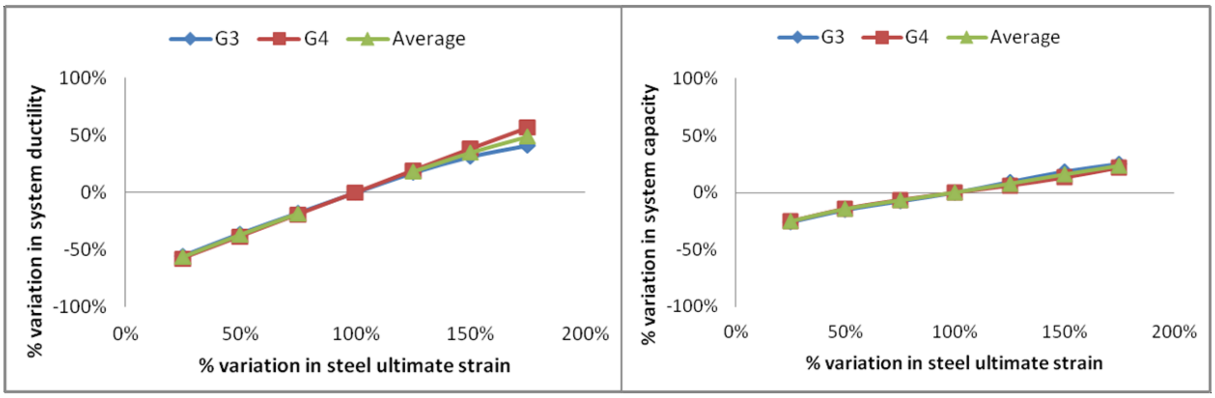
a **b**

Figure 6-14: Axially restrained beam systems with WUF-B connections; sensitivity to axial restraint



a **b**

Figure 6-15: Axially restrained beam systems with WUF-B connections; sensitivity to steel yield and ultimate resistance



a **b**

Figure 6-16: Axially restrained beam systems with WUF-B connections; sensitivity to steel maximum allowable strain

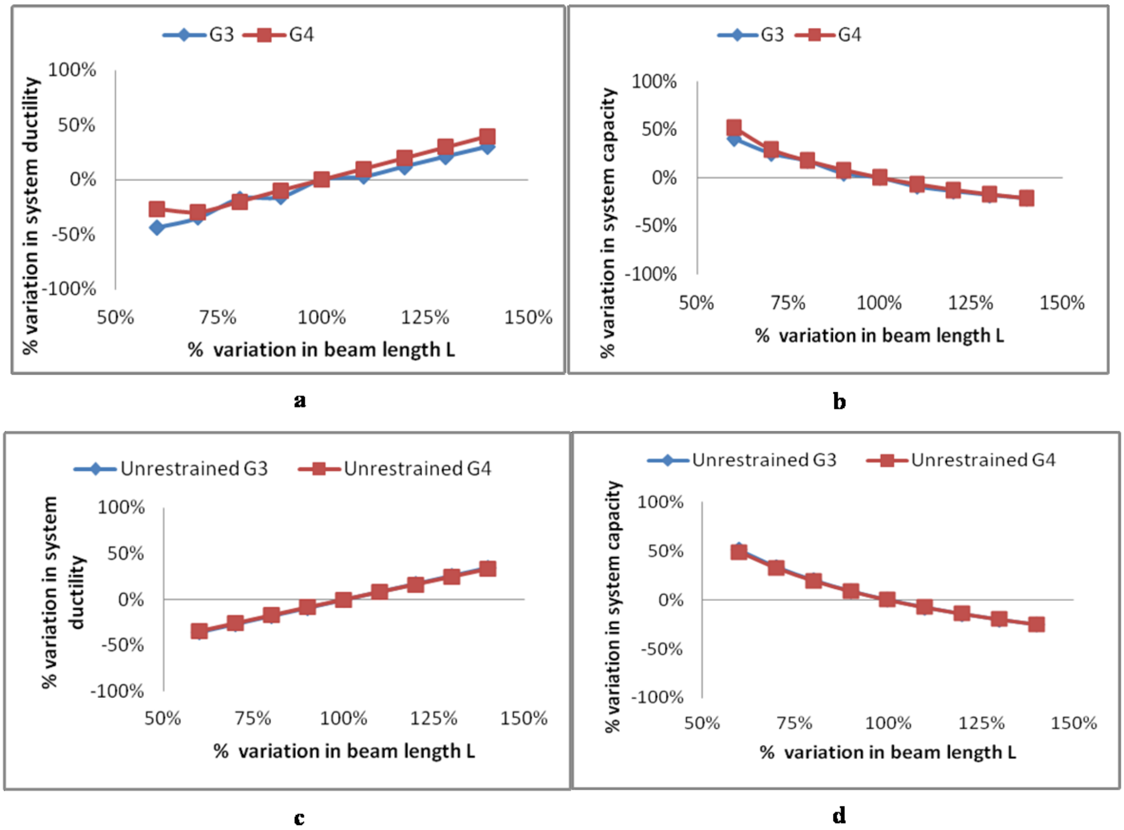


Figure 6-17: Axially restrained and unrestrained beam systems with WUF-B connections; sensitivity to beam length

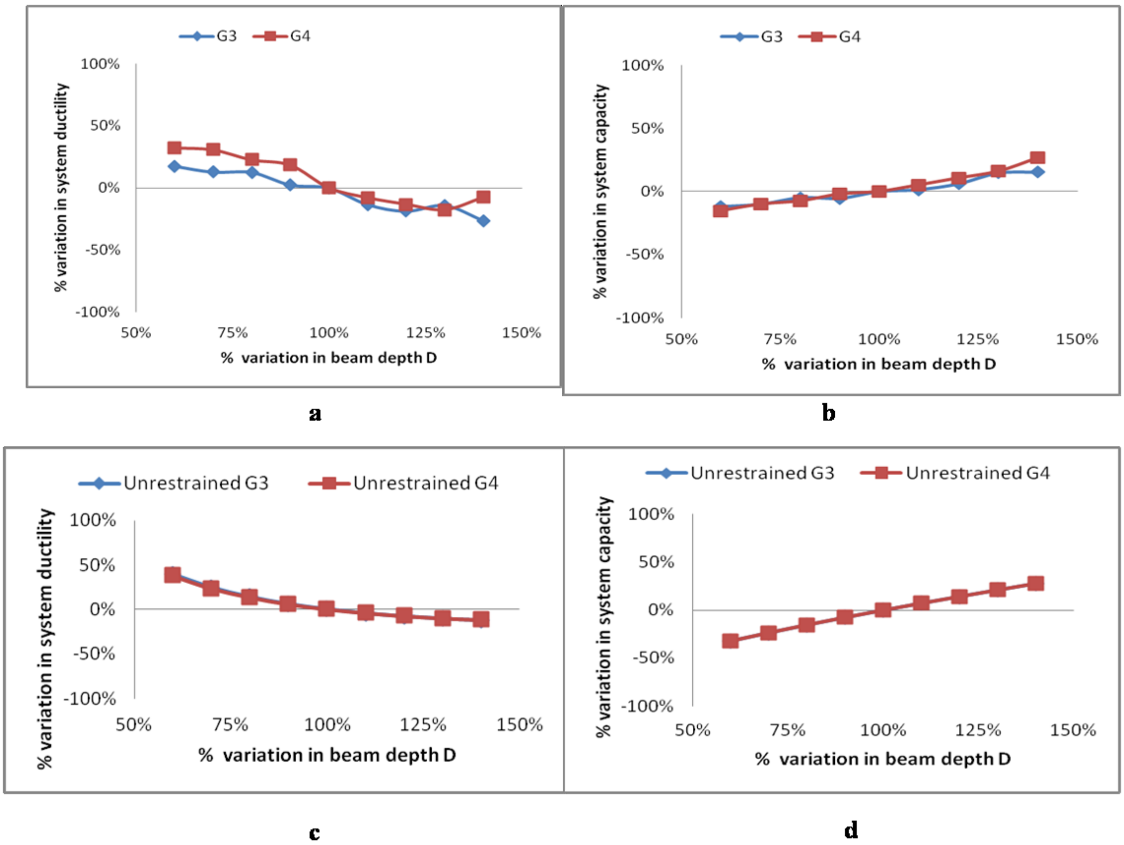
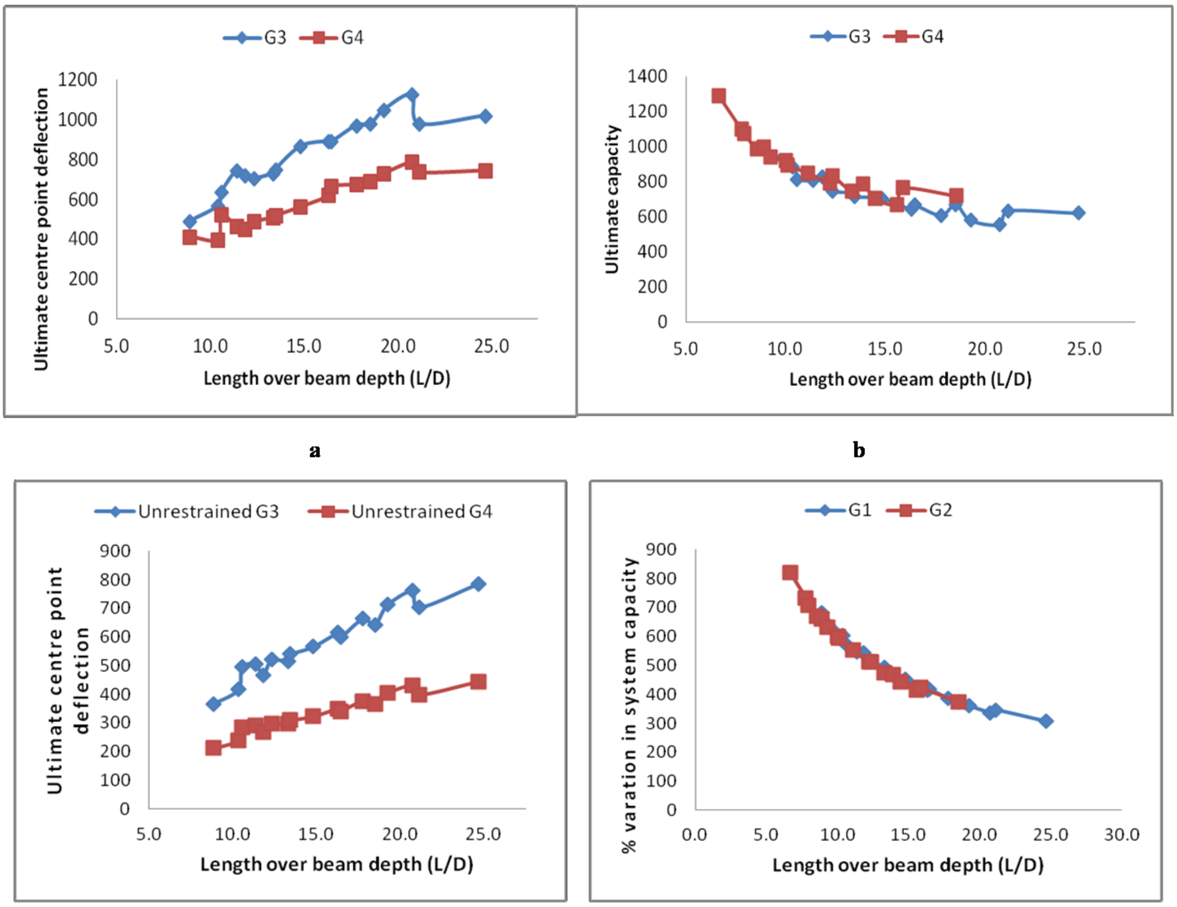


Figure 6-18: Axially restrained and unrestrained beam systems with WUF-B connections; sensitivity to beam depth



a b

c d

Figure 6-19: Axially restrained and unrestrained beam systems with WUF-B connections; sensitivity to beam depth over length ratio

Table 6-6: Sensitivity of beam model to variations in influencing parameters; NIST SDC-C frame

Parameter	Column within the interior of the frame perimeter		Column within the edge of the frame perimeter	
	% change		% change	
	Ductility	Capacity	Ductility	Capacity
Degree of axial restraint	*insignificant*	+7%	-	-
Ultimate tensile strain (ε_u)	+72%	*insignificant*	-	-
Steel resistance (f_y and f_u)	*insignificant*	*insignificant*	-	-
Beam length L	+91%	-77%	+34%	-34%
Beam depth D	-76% [a]	+40%	-22%	+30%
L / D	+29%	-59%	+68%	-72%

[a] Negative percentages denote an inverse relationship

6.5.3.5 Conclusions on the behaviour of individual beam systems

For axially restrained systems with stiff, symmetrical connections, compressive arching action does not make a noticeable contribution to either capacity or ductility. Although catenary action in double span beam systems with moment resisting connections is enhanced by the presence of axial restraint at the supports, its contribution depends on the system's ductility; without significant rotational capacity at the connections, the system fails before or very shortly after entering the catenary action phase, as illustrated in Figure 6-20.

In axially unrestrained systems, connections are predominantly loaded in bending moment rather than catenary forces. These systems reached at least 70% of their maximum capacity before the connections' bending resistance was exhausted, highlighting the bending moment catenary (transient catenary) as the main resistance mechanism (Figure 6-20).

The behaviour of girders supporting a lost corner bay is very close to that of a cantilever. Thus, the connection bending moment resistance and stiffness is still the main parameter that defines the ability of the system to withstand progressive collapse.

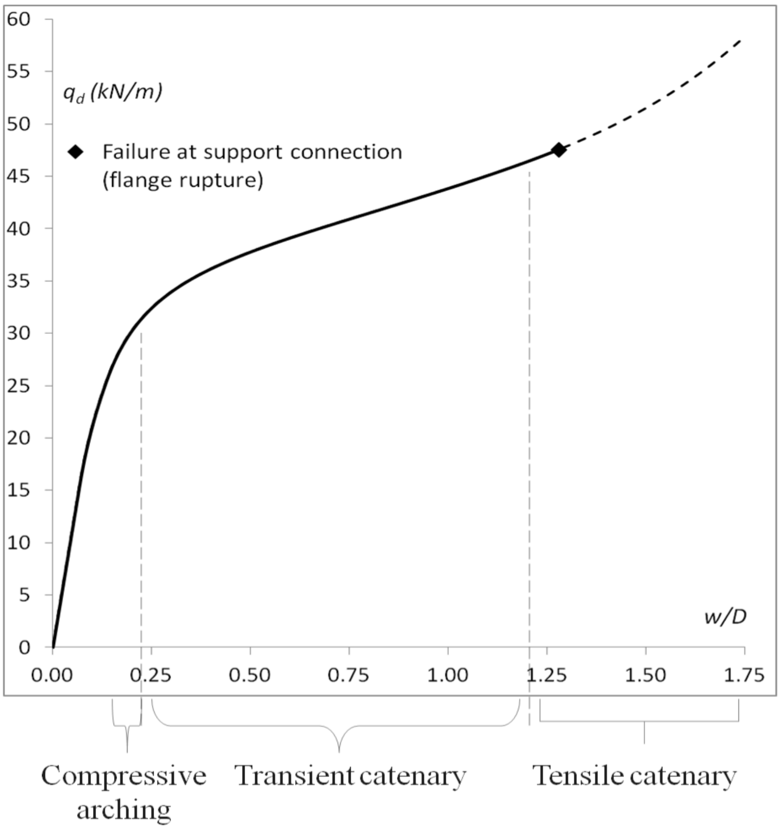


Figure 6-20: Response action phases; NIST SDC-D G2 double span axially restrained beam system

Results also showed that for beam systems with fully welded moment connections, material properties (steel ultimate tensile strain and yielding resistance) define the connection failure criteria and have a direct and proportional influence on the maximum capacity and ductility predicted by the analysis. This should be carefully considered when comparing results with experimental tests, as it was also highlighted in Chapter 3. Also, shorter and deeper beam systems perform better in progressive collapse.

6.5.4 Floor system response to perimeter column loss

6.5.4.1 Interaction between continuous and non-continuous systems

Moment frames designed to resist an earthquake usually rely on the frame action of perimeter bays, as shown in Figure 6-1 and 6-21, while the rest of the structure uses simple connections to resist gravity forces. Table 6-7 summarises the different types of interaction depending on the position of the removed column. Interior column removal scenarios (see Figure 6-3) Ex2, Ey2, Ex3 and Ey3 activate a fully axially restrained, double span continuous beam system. Penultimate scenarios Ex4 and Ey4 activate one axially unrestrained double span continuous system and corner scenarios C1 and C2 activate two single span cantilever systems.

Table 6-7: Interaction between beam systems based on column loss scenarios; NIST SDC-D

Scenario	Position of lost column	Non-continuous		Continuous		Figure
		Double span	Cantilever	Double span	Cantilever	
1	Interior	2	-	-	-	6-21d
2	Perimeter	-	1	1	-	6-21c
3	Corner	-	1	-	1	6-21b

In moment resisting bays (Scenarios 2 and 4), due to the significant difference in pseudostatic supply, the interaction between the two systems is not constructive: simply supported systems reach their peak response values for very large vertical deflections (≈2D) compared to continuous systems. An example is illustrated in Figure 6-22. In general, results show that the contribution from less rigid participating beam systems, such as those with pinned connections or the floor slab steel decking, is minimal (approximately 5% of total capacity of moment resisting bays).

In Scenario 1 (Figure 6-21d), "gravity" beam systems are connected to columns with shear tab connections. Simple connections, due to their low rotational stiffness and very low resistance in bending moment, mainly resist column loss by catenary action. Compared to the partially restrained connections examined in Chapter 5, the main difference lies in the very low capacity of the compressive components of a shear tab connection, which prevents developing an effecting compressive arching or even transient catenary action. Although the very high ductility of the connection arrangement allows the development of significant capacity via catenary action, its low stiffness makes it incompatible with a constructive interaction with the moment resisting connection at the other end of the beam system. Thus, a beam system simply connected to a corner column behaves as a double span cantilever supported by the moment resisting connection.

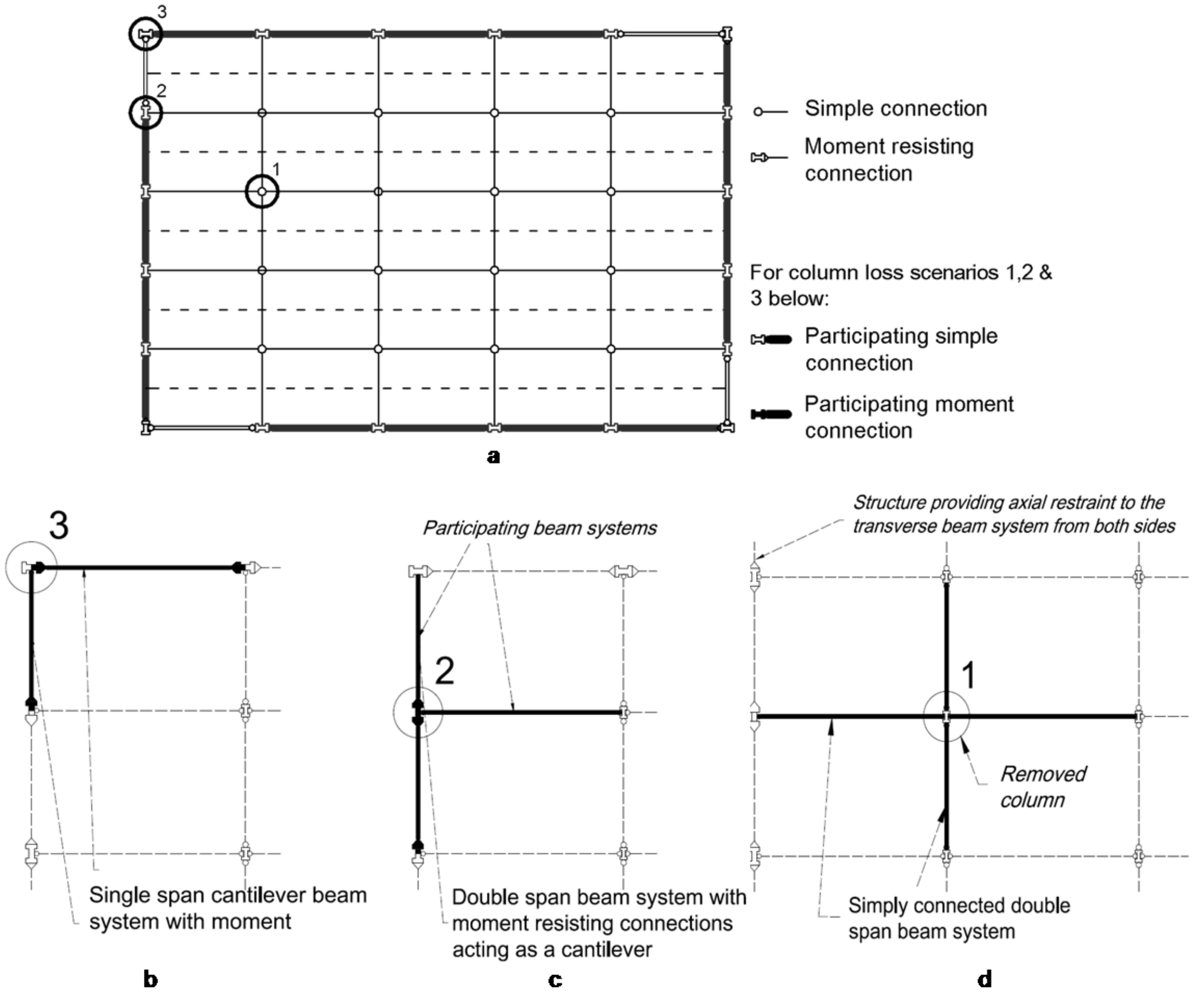


Figure 6-21: Special moment frame layout; b) Interior floor grillage c) Perimeter floor grillage d) Corner floor grillage

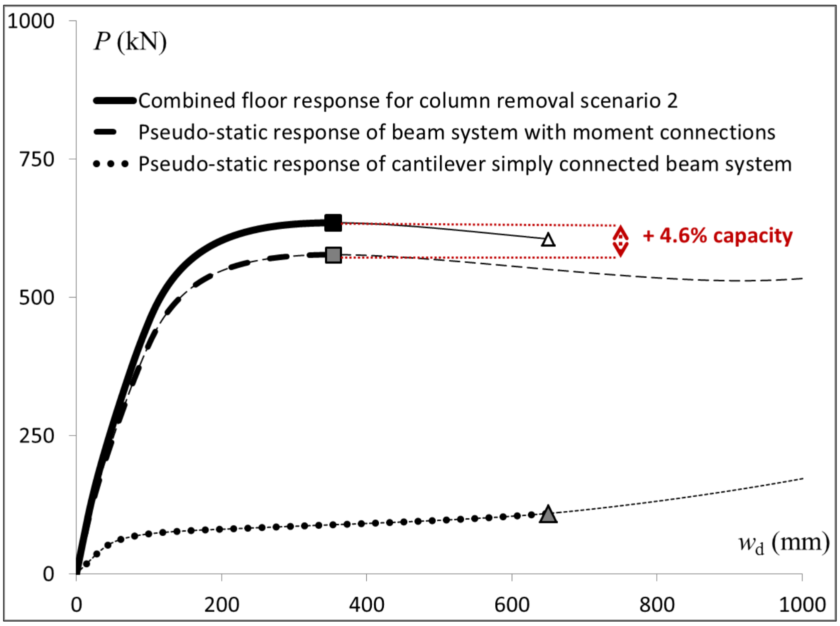


Figure 6-22: Contribution of non-continuous systems to total floor capacity for perimeter column loss

6.5.4.2 Analysis results for the NIST SDC-D frame

This section examines the response of the column removal scenarios presented in Figure 6-2. As described in Chapter 5, upon removal of each column, the participating perimeter beam systems have to provide the required pseudostatic capacity to support the floor from collapsing. The position of the lost column also determines:

- The degree of axial restraint at the support connections
- Whether participating beam systems can be considered continuous over a double span
- Whether the participating beam systems are connected to columns with moment resisting or simple connections (also see Figure 6-21)

Figure 6-23 shows the floor assembly responses for each column removal scenario and also indicates which participating beam system dominates the response. The y-axis represents the resistance, which is equal to a uniform per square meter load and the x-axis represents the deformation, which is equal to the vertical deflection at the removed column position divided by the depth of the controlling beam system. This normalisation allows the comparison between the behaviour of interior, penultimate and corner removal scenarios for different beam sections and length. Although the required resistance varies for each bay, i.e. a grillage resisting the removal of an interior column will have to provide two times more resistance compared to when resisting a corner column, the uniform load demand to withstand collapse is standard for all scenarios.

Results show that the special moment frame is able to withstand most column removal scenarios. In fact, double-span girder systems that are supported by two moment connections can provide 66% - 130% reserve capacity. The SW and NE corner removal scenarios (C1) are also capable of offering an adequate margin of safety of +21%. The four column removal scenarios, two corresponding to the E_y1 case and two to the C2 case, are potentially critical as they are able to provide 97% and 95% of the required resistance. However, given the fact that the present method is conservative, the frame is expected to most likely resist collapse in this scenario even if the margin of safety is not acceptable under the current assessment.

On the other hand, the frame is found vulnerable against the Ex1 loss scenarios. Moreover, Ex1 corresponds to a girder system that is simply connected to the corner column in order to avoid biaxial bending. The difference in connection stiffness significantly limits the contribution of the simple connection and forces the system to act as a double span cantilever until the activation of the tensile catenary phase; however, the moment connection fails before that point due to inelastic buckling of the flange in compression.

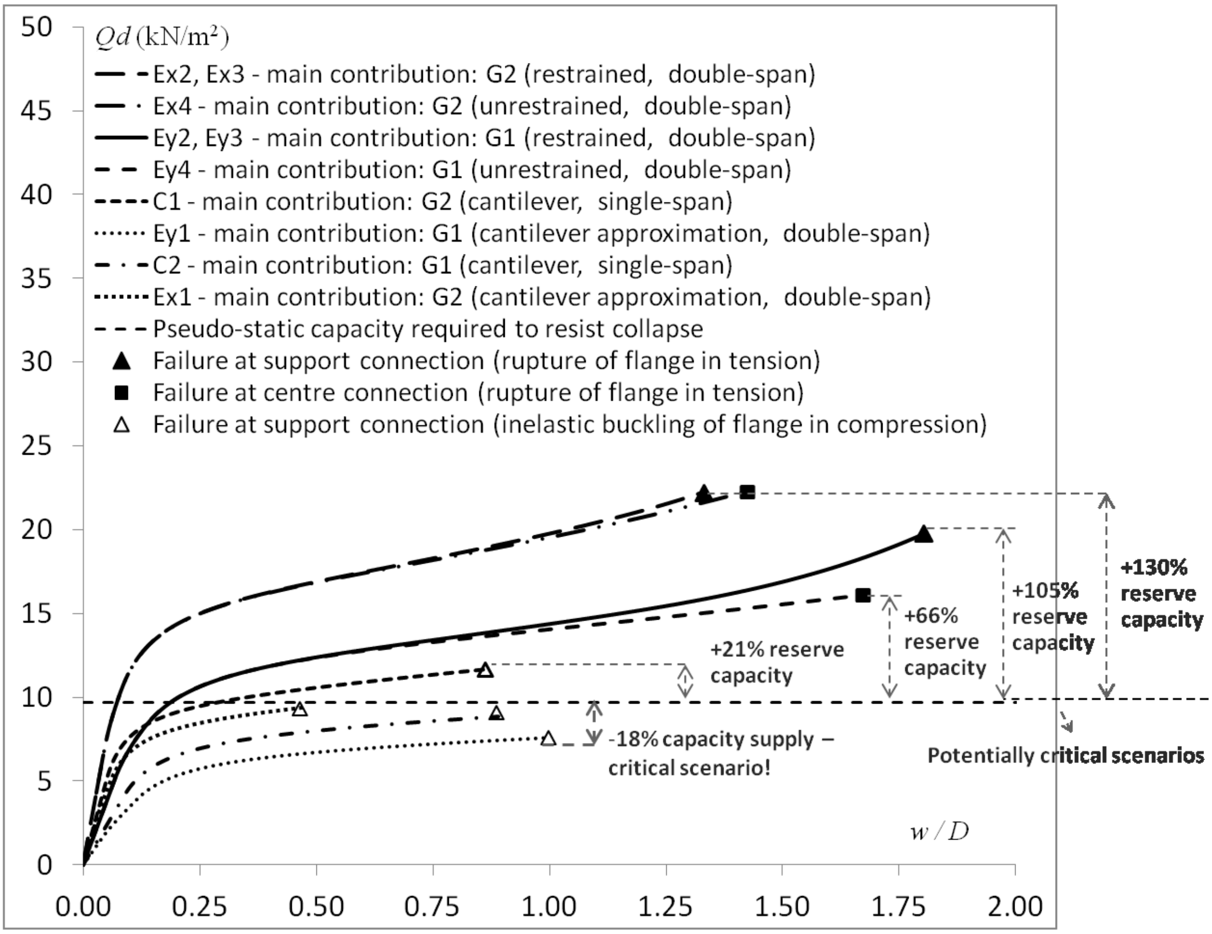


Figure 6-23: Floor grillage system responses for perimeter column removal scenarios of the SCD-D frame

6.5.4.3 Analysis results for the NIST SDC-C frame

The analysis results for the IMF frame are presented in Figure 6-24 and show that the frame is able to withstand most column removal scenarios. In fact, double span girder systems that are supported by two moment connections can provide a reserve capacity of approximately 30% - 60%. All corner removal scenarios (C1 and C2) are potentially critical as they are able to provide 94% of the required resistance. However, given the fact that the present method is conservative, the frame is expected to most likely resist collapse in this scenario even if the margin of safety is not acceptable under the current assessment. Similar to the SDC-D frame with RBS connections, for the Ex2 and Ey2 removal scenario, the difference in connection stiffness between the fully welded and simple connections at the corner columns and the gravity beams significantly limits the contribution of the less rigid connection. On the other hand, the Ex1 and Ey1 columns are supported by simple connections and an approximate evaluation of the system's response shows that the bay will be unable to withstand collapse.

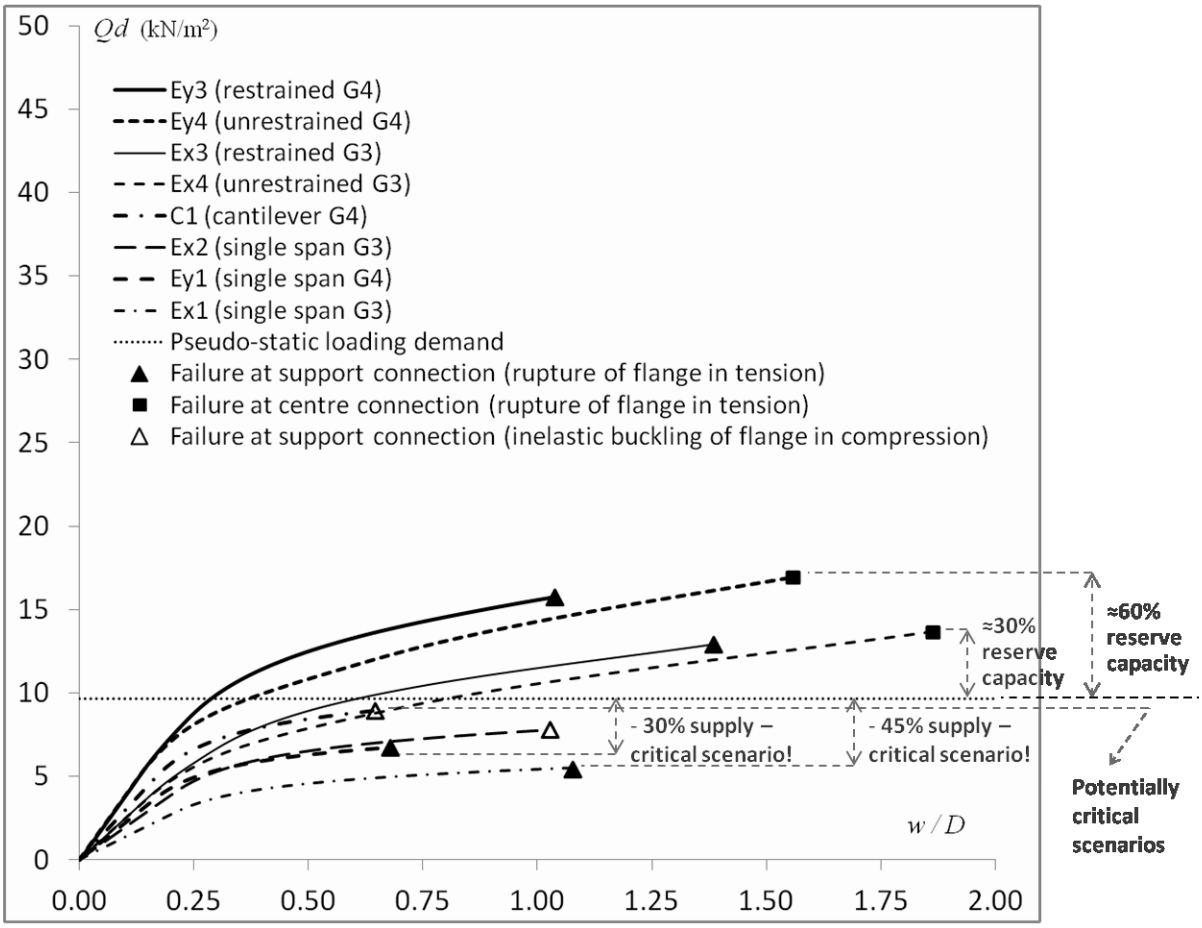


Figure 6-24: Floor grillage system responses for perimeter column removal scenarios of the SCD-C frame

6.5.5 Summary for the NIST prototype frames' case study

All moment frames that were examined were found capable of resisting the majority of perimeter column removal scenarios, for which they provided 20% to 130% reserve capacity. This is chiefly attributed to the increased strength, stiffness and ductility of the connections.

However, these frames are still vulnerable against certain column removal scenarios: loss of the perimeter columns connected via a simple connection to the adjacent corner or penultimate column may lead to disproportionate collapse of one or even two corner bays of the structure.

Contribution from less rigid participating beam systems, such as those with shear tab connections, is minimal in moment resisting bays (a maximum of 5% of total capacity), as the latter reach their peak response values for very small (compressive arching peak at about $w = D/2$) or very large deformation levels ($\approx 1.5D$).

Similarly, the contribution from the steel decking in the floor slab is excluded from the analysis in order to simplify the problem, as studies (Alashker et al., 2010) suggest that it only becomes noticeable for very high levels of deformation (larger than the depth of the beams used in this study), which does not allow a constructive interaction with the other subsystems in this case.

6.6 Progressive collapse resistance of the SAC project frame structures

6.6.1 Prototype structures

The extensive and severe damage caused by the 1994 Northridge earthquake to a large number of welded steel moment resisting frame buildings greatly underlined the need for the academic and professional community to improve design procedures and connection details that would ensure better performance in future earthquakes. The major effort in response was called SAC Joint Venture, a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), formed specifically to address both immediate and long-term needs related to solving performance problems with welded, steel moment frame connections (FEMA-355D, 2000). In its second phase, a number of prototype moment frames were studied in order to evaluate the efficiency of certain connection designs within different grades of moment frames.

Information about the prototype "Los Angeles", "Seattle" and "Boston" post-Northridge 9-storey model frame is available in Appendix B of FEMA-355c (FEMA-355C, 2000). Each frame has been designed for very high, high and average seismic vulnerability regions and corresponds to a special, intermediate and ordinary moment frame respectively. The study also considers the pre-Northridge version of the OMF in Section 6.7.2. Figures 6-25, 6-26 and 6-27 present the layout for each frame. All connections are fully welded. Information about the beam sections, connection cover plate reinforcement and column web stiffener "doubler " plates is presented in Table 6-8. The average dead floor and ceiling load is 4.6 kN/m^2, the average dead perimeter floor load is 4.75 kN/m^2 and the average live load is 2.4 kN/m^2. Girder length is considered equal to 9m (29.5 ft).

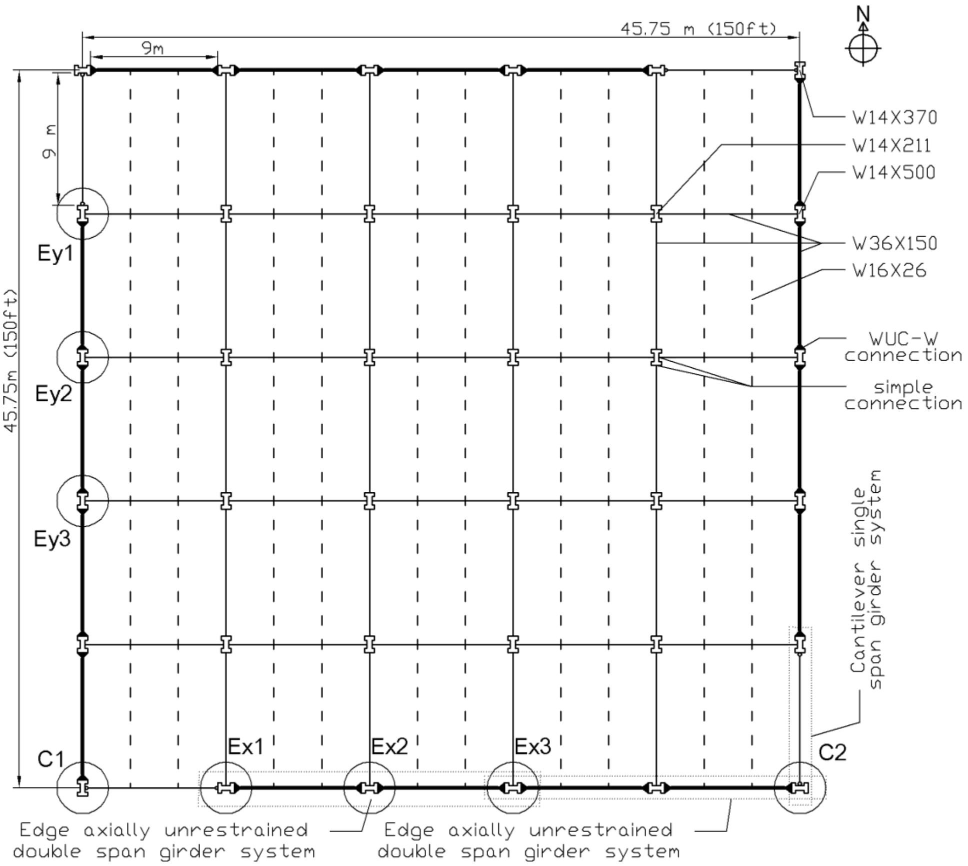


Figure 6-25: SAC Joint Venture prototype "Los Angeles" frame layout

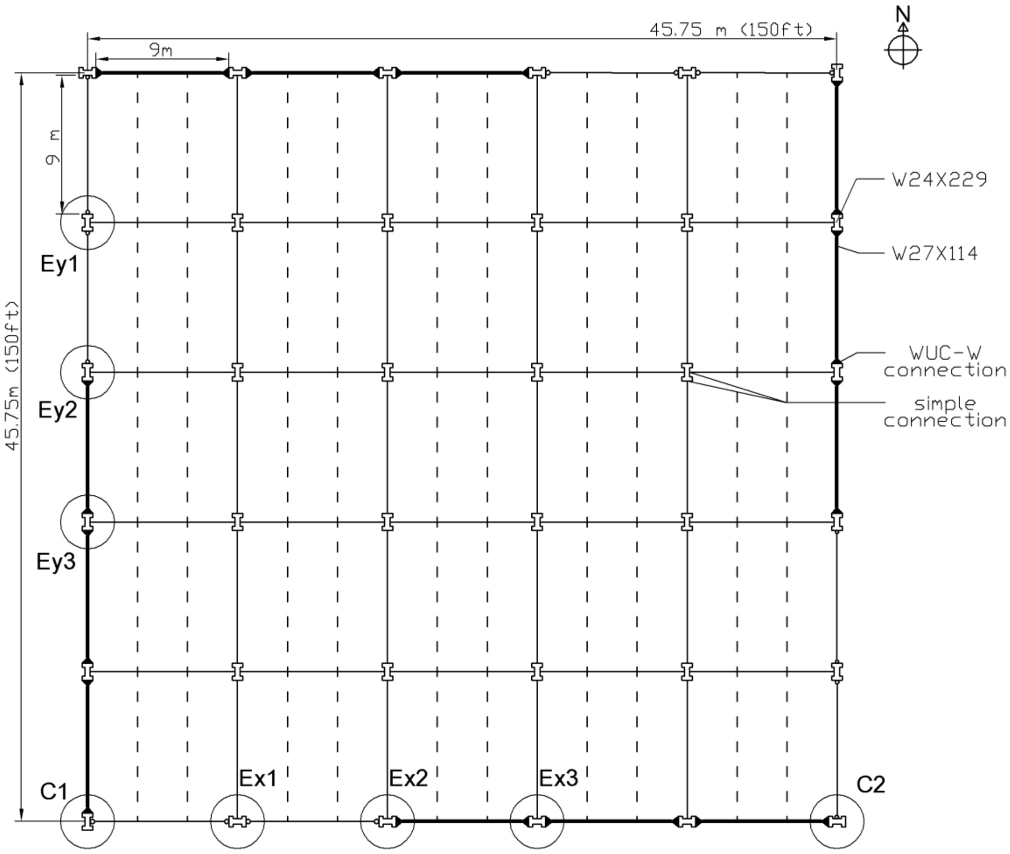


Figure 6-26: SAC Joint Venture prototype "Seattle" frame layout

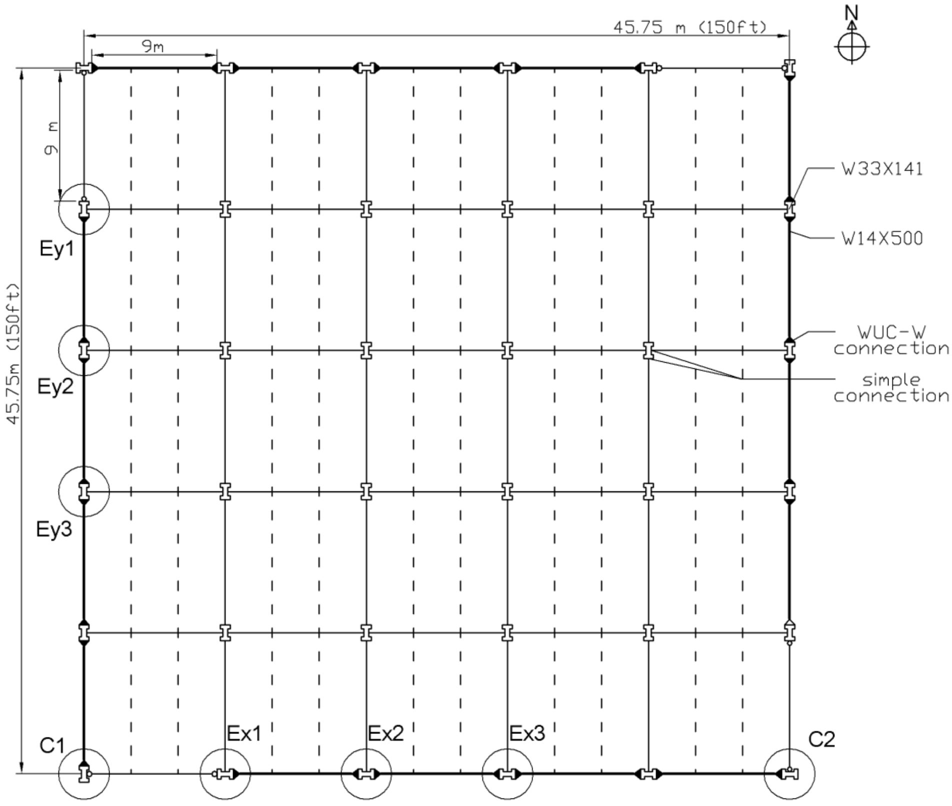


Figure 6-27: SAC Joint Venture prototype "Boston" frame layout

Table 6-8: Girder section and connection reinforcement information for the SAC prototype frames

Frame	Member	Perimeter	Grade	Doubler plates t_{wcd} (mm)	Cover plates (mm) Length – Width -Thickness
Los Angeles	Column	W14x500	Gr 50	-	
	Girder	W36x150	Gr.36 [a]		355 x 305 x 19
Seattle	Column	W24x229	Gr.50	24.4	
	Girder	W27x114			345 x 305 x 22
Boston	Column	W33x141	Gr.50	55.6	
	Girder	W14x500			508 x 293 x 25.4
Boston pre-Northridge	Column	W36x135	Gr.50	-	
	Girder	W14x283			-

[a] Although only Gr.50 (A992 or A572) is used in modern steel construction, Gr36 is kept in order to maintain the strength relationship between the beams and the columns, given the large section sizes.

The SAC frames satisfy all the prerequisites of the multi-level idealisation presented in Section 2.4.1.4, thus global conclusions can be drawn from local joint, girder and floor grillage behaviour analysis. Column removal scenarios are considered for the ground floor. The contribution of the gravity secondary beams simply connected to the columns, which are represented with a dashed line in the plan layout, is considered negligible, based on the findings of Section 6.5.4.1.

6.6.2 Beam system response to perimeter column loss

6.6.2.1 Response characteristic

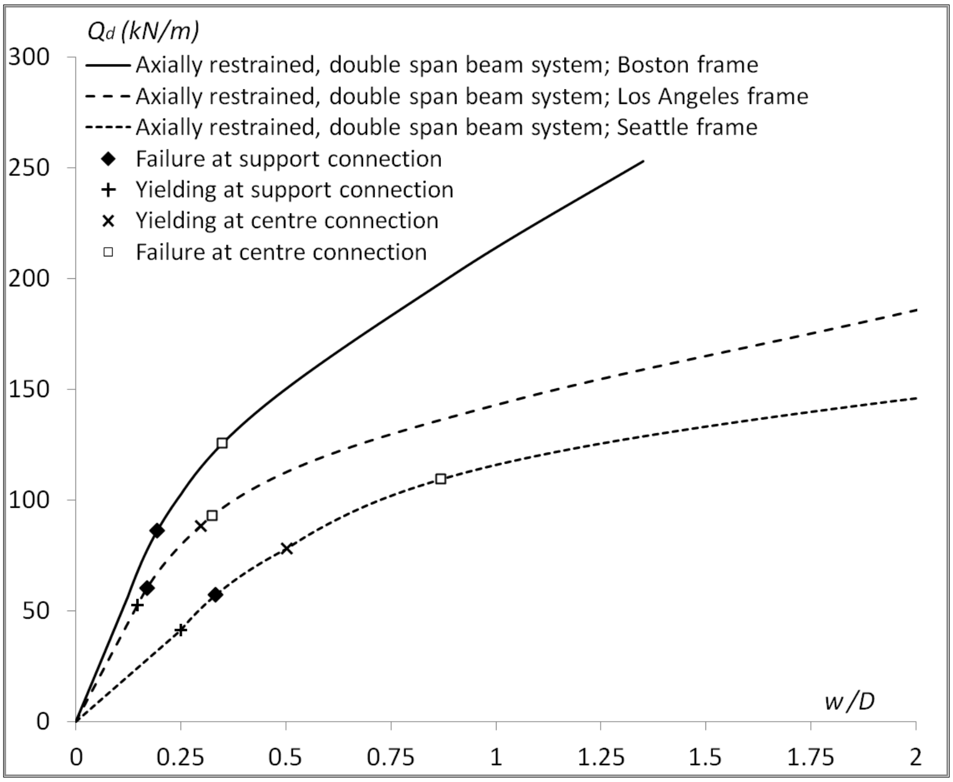


Figure 6-28: Q_d-w response of the axially restrained girder systems (SAC frames)

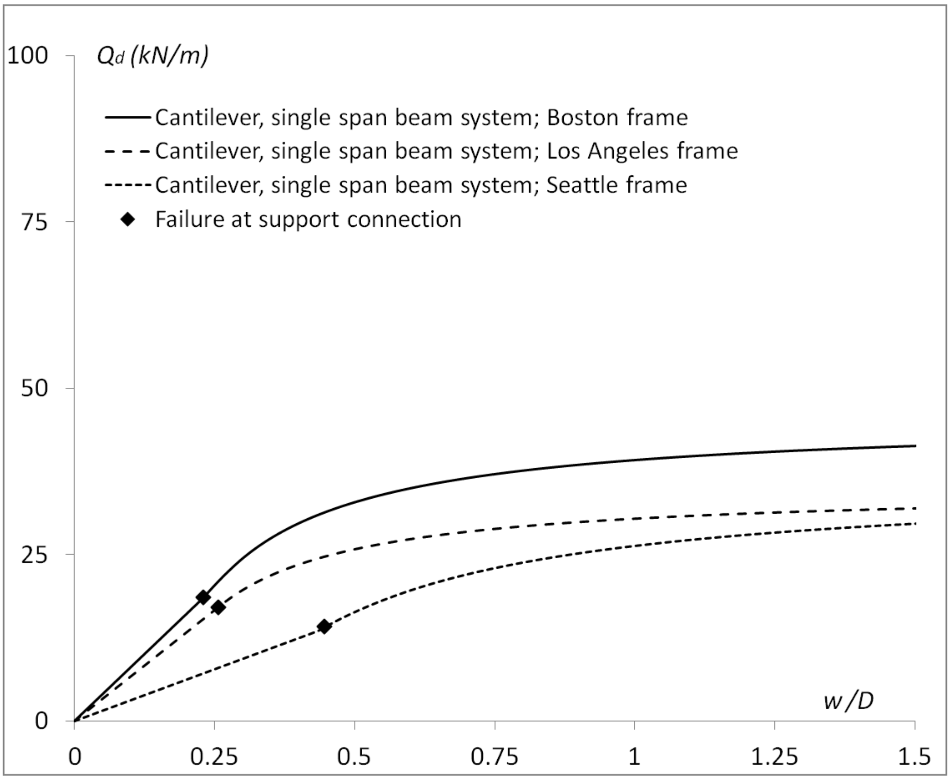


Figure 6-29: Q_d-w response of the cantilever girder systems (SAC frames)

The response of the axially restrained and the cantilever beam systems of each frame is presented in Figure 6-28 and Figure 6-29 respectively. In the case of axially restrained double span beam systems, information on the support and centre connection components' yielding and failure is provided. Although the form of the response is similar, Table 6-9 summarises the parameters that are responsible for the differences in ductility and capacity. The fact that the Boston frame beam system, despite being designed for lower levels of seismic vulnerability, can provide high levels of resistance against progressive collapse for perimeter column loss is discussed in Section 6.6.4.

Table 6-9: Effects of parameters on response

Indicator	Frame			Comments
	LA	Seattle	Boston	
Connection normalised tensile strength (m')	1.37	1.74	1.61	Although the Seattle frame connections have the highest tying capacity, its beam systems provide the lowest pseudostatic resistance. Since the major resistance action is transient catenary (connections highly loaded in bending moment), connection tensile strength is not an entirely useful indicator of capacity.
Beam section moment capacity ($M_{b,Pl,Rd}$)	2065 kN/m	1690 kN/m	2533 kN/m	
Beam length over depth ratio (L/D)	10.1	13.4	11.0	The LA and Boston frame employ deeper beams than the Seattle one, which decreases the rotational stiffness of their joints.
Connection reinforcement (column web plates and beam flange coverplates; Table 6.8)	Low	Average	High	The heavy cover and doubler plates used in the Boston frame's connections increase the bending moment capacity and ductility of the critical region of the connection under progressive collapse loading conditions.

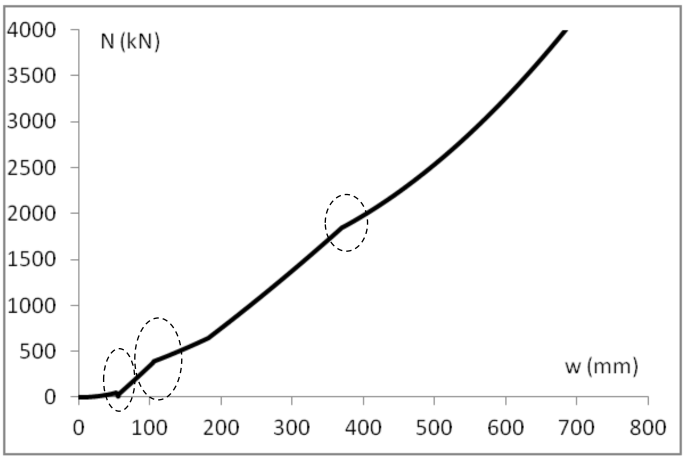


Figure 6-30: Connection axial loading; SAC Los Angeles axially restrained beam system

The high rotational stiffness of fully welded connections does not allow the development of compressive arching action. As the circled region of Figure 6-30 shows, the axial loading is initially low and remains tensile. On the contrary, bending moment loading increases at a steady rate throughout the response, along with the tensile axial load. In this case, the system enters the transient catenary phase but its ductility is exhausted before it can enter the tensile catenary phase.

6.6.2.2 Failure mode and critical components

The mode of failure depends on the class of the section and the flanges. In this case, all sections are Class 1 wide flange sections and the mode of failure is flange tensile rupture or inelastic buckling under combined bending and axial load. The critical components are the column web and beam flange in combined tension and bending and the beam flange in combined compression and bending for the tensile and compressive components respectively. Specifically for the Los Angeles frame, Figures 6-31 and 6-32 illustrate that:

- In Figure 6-32, following the first yielding of components (w = 52 mm), the tensile rigid bar (see Chapter 3) starts rotating, while yielding at the centre connection is observed for twice the deflection.
- Yielding of the column web panel in shear (Figure 6-31) causes a decrease in the rate of rotation of the tensile components.
- In a similar fashion, Figure 6-32 shows that the rotation of the compressive rigid bar (see Chapter 3) initiates after yielding of the compressive beam flange.

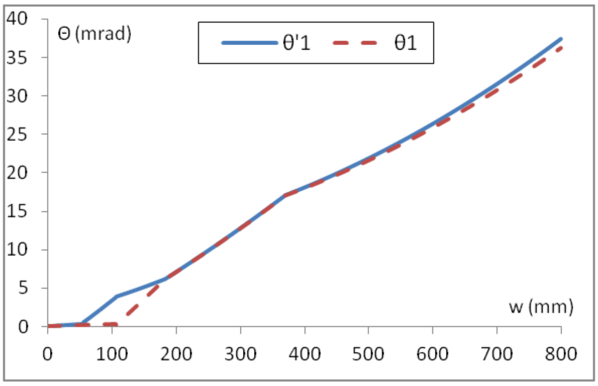


Figure 6-31: Connection tensile component rotation

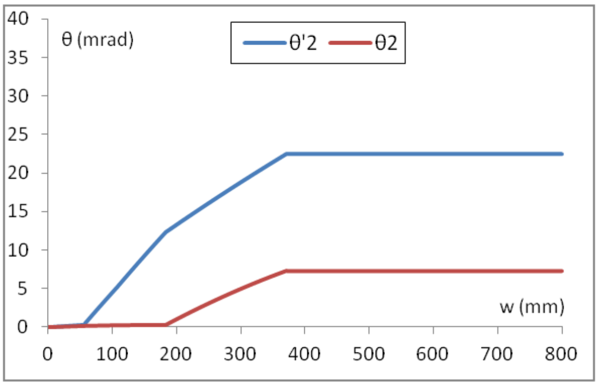


Figure 6-32: Rotation of compressive components

6.6.2.3 Influence of axial restraint

Similar to the results for the NIST frames in Section 6.5.3, despite the fact that axial restraint has a direct impact on ductility, increased levels do not lead to an effective compressive membrane effect. The comparison between the responses of beam systems with different degrees of restraint in Figure 6-32 and in Figure 6-33 show that although axial restraint has little effect on the yielding point of the connection components, it significantly influences the ultimate rotation before failure since it affects the redistribution of forces after yielding of the components. However, if assumed entirely absent, then connection axial loading becomes almost zero, leading to unrealistically large rotation capacities and thus increased system resistance; the opposite effect of what was observed for the non-continuous beam system tests of Chapter 5.

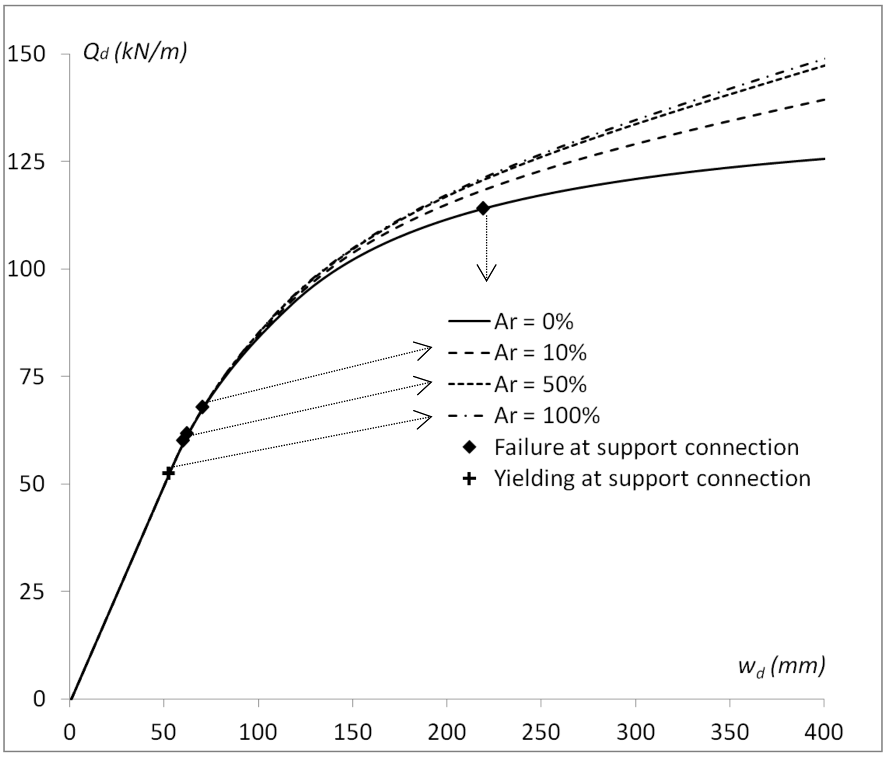


Figure 6-33: Influence of the degree of axial restraint on the Q-w response of the double span beam systems (Los Angeles frame)

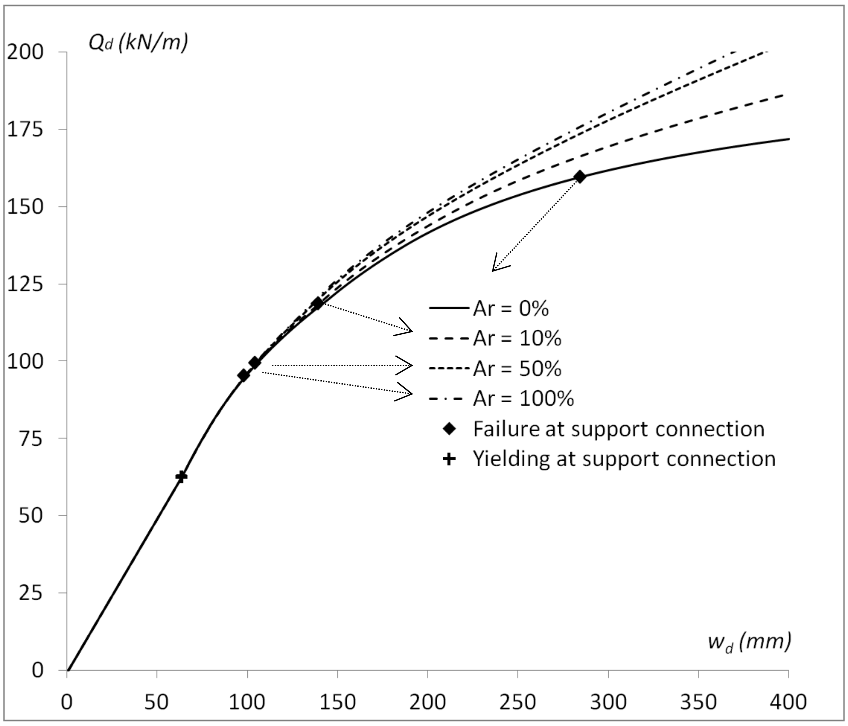


Figure 6-34: Influence of the degree of axial restraint on the Q-w response of the double span beam systems (Los Angeles frame)

6.6.3 Floor grillage system response to perimeter column loss

All frames were found sufficiently robust to resist most perimeter column removal scenarios (Figures 6-35 and 6-36), with the exception of:

- For all frames: scenarios E_x1 and E_y1 (Figure 6-37)
- For the Seattle frame, in addition to the above, scenarios E_x2 and E_y2.

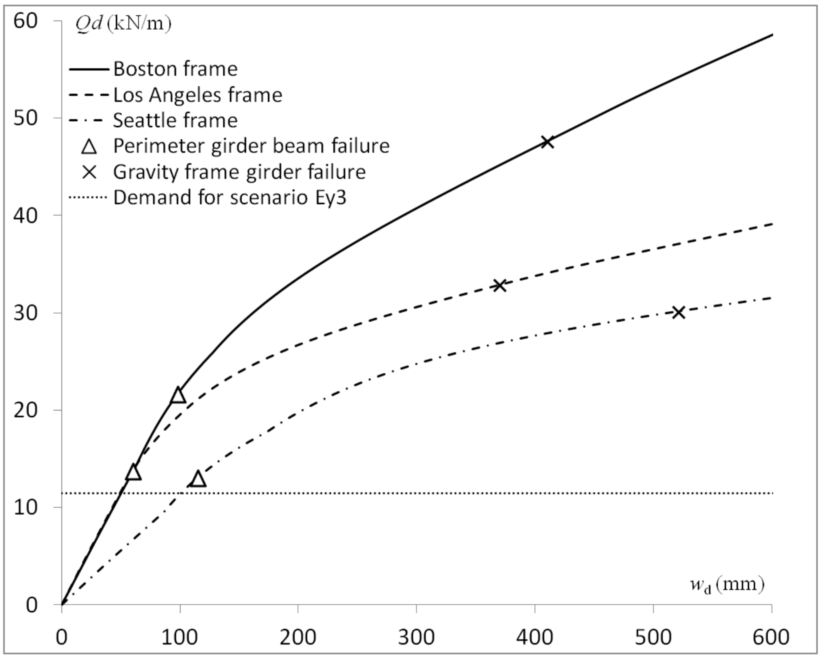


Figure 6-35: Floor system responses for Ey3 column loss scenario (all SAC frames)

Similar to the NIST frames, the above critical column removal scenarios involve a girder simply connected to the corner column to avoid bi-axial bending. Due to the difference in connection stiffness, the element is considered as a cantilever beam system submitted to the equivalent double span loading.

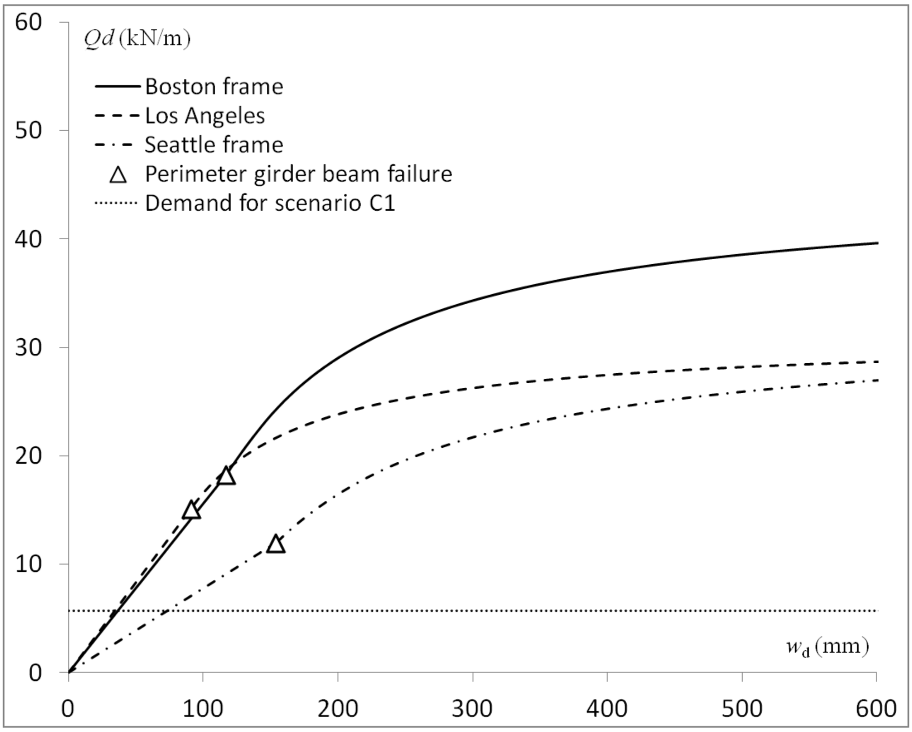


Figure 6-36: Floor system responses for C1 column loss scenario (all SAC frames)

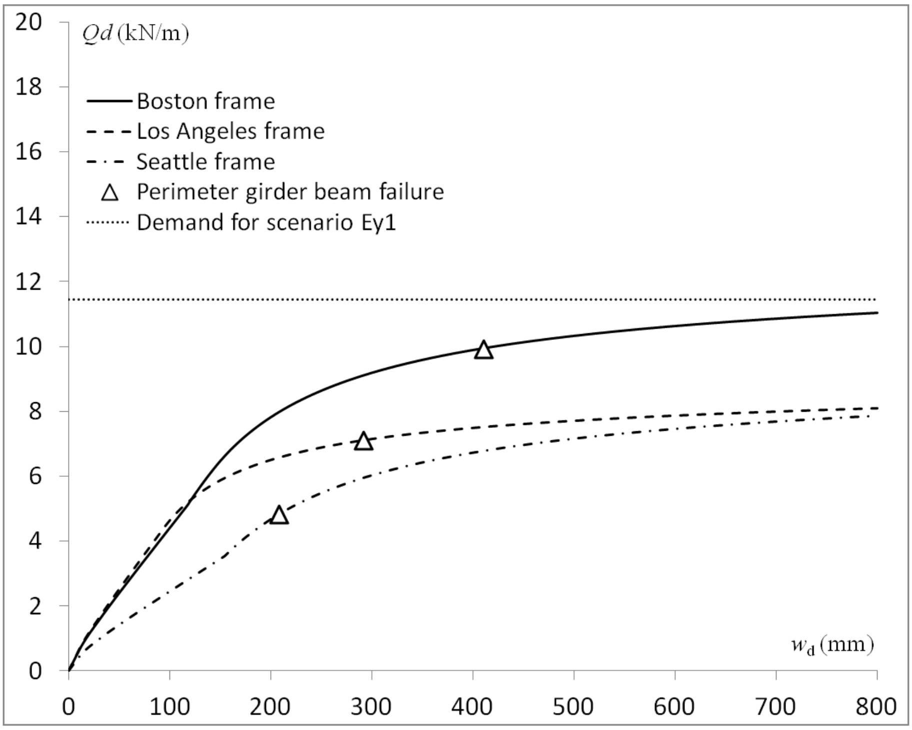


Figure 6-37: Floor system responses for Ey1 column loss scenario (all SAC frames)

Figure 6-38 compares the response of different floors of the Los Angeles frame. Although floor response at the interior of the frame's perimeter is very similar, corner bays with fully welded cantilever systems are significantly less ductile.

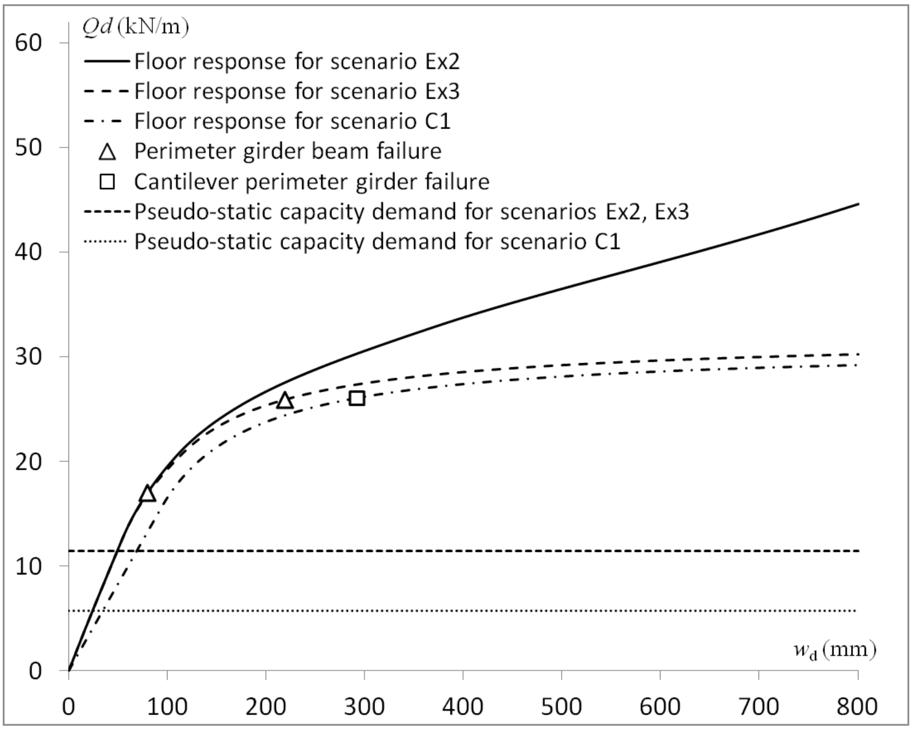


Figure 6-38: Floor system responses for Ex2, Ex3 and C1 column loss scenarios (Los Angeles frame)

6.6.4 Comparison between pre-Northridge and post-Northridge designs

The Boston frame employs particularly heavy column sections and extensive cover and doubler plates. The initial pre-Northridge frame was not designed against the event of an earthquake. As the SAC project involved redesigning pre-Northridge frames, the post-Northridge frame had to comply with low-vulnerability seismic provisions. In order to satisfy the strong column weak beam concept, heavier column sections and reinforcement were used instead of reducing the beam sections.

Figure 6-39 compares the response of pre and post Northridge designs, while the effect of different parameters is discussed in Table 6-10. There is a significant increase in the pseudostatic loading capacity with the use of supplementary column web plates, without which the column web is the critical component. Only then, the addition of coverplates also has a positive impact on system capacity and ductility.

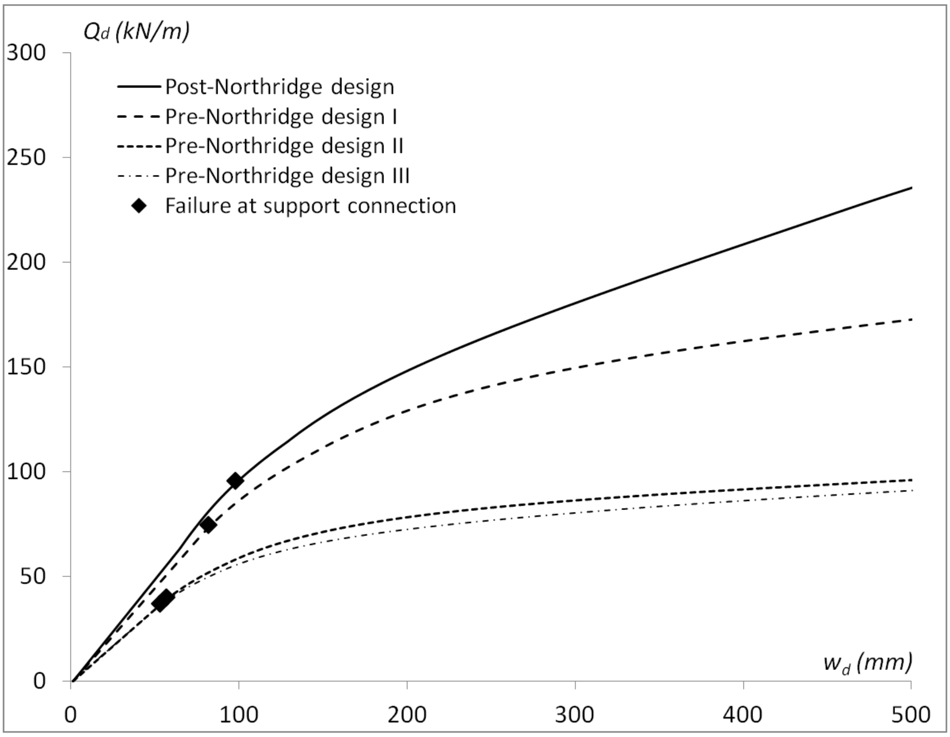


Figure 6-39: Comparison of the performance of the pre and post Northridge Boston frame design; Q-w response of an axially restrained double span continuous beam system

Table 6-10: Impact of different design features on the Boston frame beam system response

Design type	Features	Comments
Pre-Northridge III	× *Original column sections* × *Original girder sections* × *No doubler plates* × *No coverplates*	Failure is localised at the welded beam flange, which ruptures very early in the response.
Pre-Northridge II	× *Original column sections* × *Original girder sections* × *No doubler plates* ✓ Girder flange coverplates	Failure is localised at the column web, which fails in combined tension and shear very early in the response
Pre-Northridge I	× *Original column sections* × *Original girder sections* ✓ Doubler column web plates ✓ Girder flange coverplates	Although failure is localised at the welded beam flange, the response is enhanced and the system is more ductile.
Post-Northridge	✓ Heavier column sections ✓ Slightly larger girders ✓ Doubler column web plates ✓ Girder flange coverplates	The lower L/D ratio of the beam system further contributes to enhancing the response and thus the ultimate capacity of the system.

6.7 Performance of special and intermediate moment frames

All frames examined were found able to resist the majority of column removal scenarios, although the increased number of moment resisting bays in the special moment frames reduced the number of critical column loss scenarios by half. This is illustrated in Figure 6-40, which allows comparing the number of critical column removal scenarios between an SMF, which can be either the NIST SDC-D or the SAC Los Angeles frame, with that for the IMF, which can either the NIST SCD-C or the SAC Seattle frame, showing that the first are more robust in terms of the number of critical perimeter ground level column loss scenarios.

For all five frames, the resistance mechanisms and critical components are similar to those identified for the NIST frames in Section 6.5.3. In addition, the contribution of the simply connected beam system at the interior of the frame is negligible - the floor grillage response is dominated by the rigid beam systems with moment connections. For a comparison between the performance of beam systems with RBS, WUF-B (unreinforced) and WCF-B (reinforced with coverplates) connections, please refer to Chapter 3.

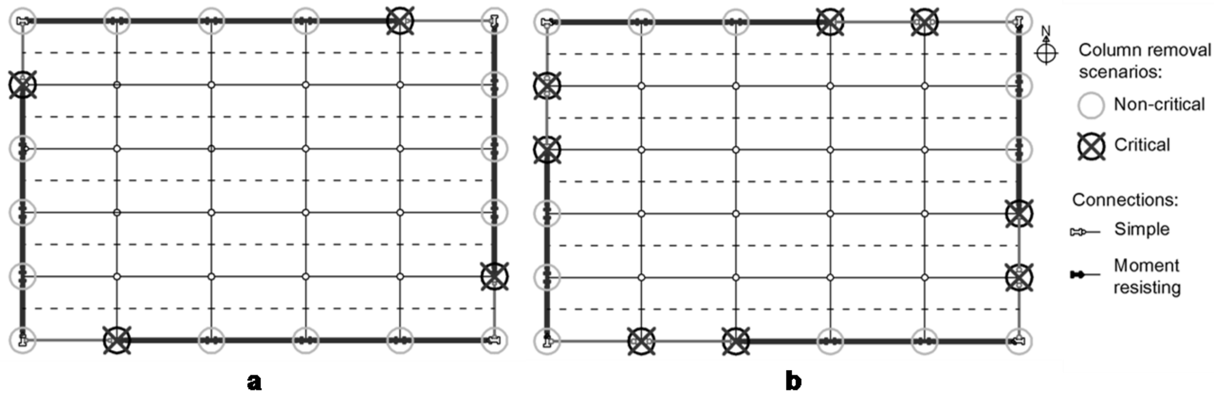


Figure 6-40: Critical column loss scenarios for a) prototype SMF; b) prototype IMF

6.8 Comparison with the behaviour of simply designed frames

6.8.1 On the response of individual beam systems

6.8.1.1 Influence of axial restraint

The influence of axial restraint can potentially be more beneficial for non-continuous systems than for systems with moment resisting connections. In the first case, it has a direct impact on the form of the response with the activation of the compressive arching action and enhances ultimate capacity during the tensile catenary phase. The peak point during the compressive arching phase is more beneficial to ultimate capacity of certain beam systems as the rest lack the necessary connection ductility to activate the tensile catenary phase. On the other hand, the presence of axial restraint in fully welded connections has a lesser effect; moment connections are usually symmetrical and very rigid, which makes the contribution of the compressive membrane effect negligible.

6.8.1.2 Influence of the beam length over depth ratio

For either frame design, shorter and deeper beams mean that smaller rotations are required at the connections for the same levels of pseudostatic resistance. This can be beneficial for the system response when matched with adequate connection strength. Otherwise, it will lead to an important decrease in connection rotational capacity, hence negatively affecting system ductility and capacity. While the strength of simply designed connections varies greatly, moment resisting connections are generally very strong, so lower L/D ratios will enhance system capacity.

6.8.1.3 Influence of beam section moment capacity

Although the beam section moment capacity has a minimal effect on the response of non-continuous systems since the beam flange is not a potentially critical component, it has a significant effect on that of continuous systems, especially if the beam section is classified as Class 1, in which case it is expected to fail under combined bending and axial loading.

6.8.1.4 Influence of global connection stiffness and strength

The interplay between connection stiffness and strength plays a significant role in defining the system's ductility and capacity in the case of non-continuous systems. Increased rotational stiffness is beneficial for partial strength connections and can enhance the peak response point attained during the compressive arching action phase. Fully welded connections are normally regarded as rigid, so increased connection strength will enhance the response, though increased stiffness will not affect it. In most cases, increasing the strength of the connection will also increase its bending moment capacity.

6.8.1.5 Influence of connection tying capacity

For both frame types, increased tying capacity will not directly increase the system's ultimate capacity if catenary action is not the critical resistance mechanism, which requires very high levels of connection rotational capacity (see Chapter 3, Section 6.5.3 and Section 6.6.2). In practice however, reinforcing the connection will also affect bending moment resistance, which will enhance the transient catenary phase of the response. Thus, although there is an indirect benefit when tying capacity is increased, it is not a directly useful measure of progressive collapse resistance.

6.8.1.6 Failure modes and critical components

The mode of failure is defined by the type of load, rotational capacity and stiffness of the connections. Generally, simple or partial-strength connections fail in a ductile manner once the deformation capacity of one or more components is exhausted. For fully welded connections, which are a popular approach to moment resisting connections, attention has to be given to the welding procedure of the latter to avoid a brittle type of failure which could leave the structure vulnerable to progressive collapse.

As discussed in Chapter 5, the critical component of simple or partial-strength connections varies greatly depending on the beam section and length, column section, t-stub arrangement, bolts and endplate used. Any remediating solution should take into account that reinforcing a weak component will only yield positive results up to the stage at which another component becomes critical. Thus, connection design interventions should provide the optimal ductility and capacity for the connections.

In moment frames, because the column web panel is usually stiffened and there are no other connection components, the beam section is often the critical component and fails either in rupture under combined bending and tension or in inelastic buckling under combined bending and compression.

6.8.1.7 Relationship between ductility and ultimate capacity

The impact of changes in ductility on system capacity in a progressive collapse scenario is influenced by the resistance mechanisms activated during the system's response. For simply designed frames, additional ductility will enhance system capacity during the tensile catenary action phase, although high levels of connection rotation capacity are required to reach this phase. For axially restrained systems, the response peaks at lower levels of centre point deflection because of membrane action, so unless the connections are very ductile, small increases in connection rotational capacity will have little effect. In the case of moment frames examined in this chapter, ductility has a direct impact on capacity because rigid systems enter the catenary phase for relatively low levels of beam system deformation.

6.8.2 On the response of floor grillage assemblies

For simply designed frames, the relative ductility of the subsystems influences the response of the floor to an important extent. On the one hand, if the subsystems do not attain their peak resistance for the same deflection levels at the point of the lost column, then the combined response will be less than the sum of the individual capacities (non constructive interaction). On the other hand, the least ductile subsystem defines the floor's maximum ductility.

On the contrary, the contribution of non-continuous systems in floor systems with participating continuous systems is very low (see Section 6.5.4.1); the perimeter moment resisting beam systems dominate the response. This difference, as well as its implications on improving progressive collapse resistance in the two types of frames, is discussed in detail in the next chapter.

6.9 Summary and conclusions

A series of model moment frames, based on the NIST Building Resilience and Structural Robustness Project and the SAC Joint Venture, were extensively studied using the simplified ICL Method in terms of their ability to withstand progressive collapse following loss of a level column.

Results showed that the stiffness of the connections prevents the system from developing a compressive arching resistance mechanism, making axial restraint at the support joints less influential to maximum capacity, while the connection bending stiffness and resistance play the most influential role. However, catenary action requires both an important degree of support axial restraint and substantial connection rotational capacity in order to be activated. Without both, the system fails before or very shortly after entering the tensile catenary action phase. In the absence of axial restraint, the system has already achieved at least 70% of its maximum resistance before significant axial forces develop in the connection activating the tensile catenary mechanism.

Deep sections and short beam spans enhance the ability of the system to provide the necessary resistance. Also, the connection failure criteria (steel ultimate tensile strain and yielding resistance) have an almost linear influence on the model's predicted maximum capacity and ductility. This provides additional challenges when comparing results from experimental tests for continuous beam systems compared to non-continuous assemblies.

Floors with continuous beams are likely to be substantially more resistant against progressive collapse compared to non-continuous systems, mainly because of the:

- Enhanced connection stiffness and strength in bending and not just in tension;
- Increased connection ductility;
- Connection ability to resist both hogging and sagging bending moments.

All frames that were examined were found capable of resisting the majority of perimeter column removal scenarios while providing reserve capacity between 20% and 130%. Beam systems employing full strength connections dominated the floor response. On the contrary, the contribution of non-continuous systems was minor.

However, corner bays were found vulnerable to progressive collapse, as the simple connections at the edge (connecting either edge or internal beams) were unable to provide the required pseudostatic capacity. The reduction in the number of moment resisting connections (SMF to IMF) increased the number of critical column removal scenarios from 4 to 8.

Comparison with the previous study of simply designed frames highlighted some differences in their behaviour with that of moment frames. On the one hand, the behaviour of simply designed frames with partially restrained connections is influenced by a rather complex interplay between various parameters: axial restraint, connection stiffness and strength, beam depth to length ratio and the balance between the resistance of the connection tensile and compressive components. The fact that the connections are usually asymmetrical - potentially weaker in resisting hogging bending moments – also means that there is a large difference between the behaviour of the support and centre connections in a double-span beam system. The response comprises four main action phases: elastic, compressive arching, transient catenary and tensile catenary. The ductility of the system defines which of these will be critical and different priorities should be considered in each case. Finally, the behaviour of the floor grillage assembly system is influenced not only by the maximum capacity of the participating beam systems but also by their relative ductility and the form of response of each subsystem.

On the other hand, the relationship between the influencing parameters in the case of moment frames with fully welded connections is more simple and straightforward. Transient catenary action is usually the critical action phase, during which the connection’s bending resistance is more important than its tying capacity. Also, the connection behaviour is usually dominated by one component, which is the beam flange in combined bending and tension or compression. Finally, floor grillage behaviour is dominated by the perimeter moment frame.

Both types of frames that were examined were found to lack the necessary ductility to fully activate tensile catenary action, thus tying capacity is not a directly useful measure of resistance to progressive collapse. In practice however, reinforcing the tensile capacity of any component in the connection will also enhance its ability to resist compression, shear and bending moment.

Chapter 7

Improving progressive collapse resistance in steel moment frames

7.1 Introduction

Seismically designed moment frames are often instinctively considered to be more robust, as they employ strong and ductile connections. Previously, researchers have argued that earthquake design principles regarding joints and continuity might be useful for mitigating progressive collapse (Hayes J.R. et al., 2005, Gurley, 2008). This was also mentioned at an earlier time in the FEMA report on the aftermath of the Murrah building collapse (FEMA 277, 1996), which proposes that earthquake design principles regarding joints and continuity might be useful for mitigating progressive collapse. More recently, the USA NIST (NIST, 2007) has suggested that building standards could recommend minimum detailing requirements to ensure general structural integrity, and engineers would not have to directly consider abnormal loads or progressive collapse.

However, the majority of available studies focus on modelling the entire or parts of a structure, rather than on understanding how effective meeting seismic requirements might be in terms of, inherently, providing substantial robustness.

In the work report herein, the findings of Chapter 6, which determined the beam system and floor response characteristics of moment frames and highlighted the vulnerabilities due to seismic provisions under the loading and deformation conditions of progressive collapse, are used in conjunction with the redesigning methodology presented in Chapter 5. The methodology is employed in order to identify how best the design of the exemplar moment frames studied in Chapter 6 may be improved and to clarify the relationship between robustness and seismic resistance.

7.2 Study layout

7.2.1 Imperial College London redesigning methodology

The modifications determined by the ICL redesigning methodology (Section 5.3) are often related to connection component design and aim at either or both:

i. Enhancing the pseudostatic response of the individual beam systems, depending on the critical resistance action mechanisms (Section 5.2).
ii. Regulating their ductility for achieving their most constructive interaction within a floor system.

The redesigning process for non-continuous construction bears the additional complication of taking into account the interplay between an increased number of influencing parameters. The summary below recaptures the main steps (a detailed presentation of application for composite and steel semi-continuous frames is available in Chapter 5):

i. Extensive parametric tests are conducted for the chosen "alterable" parameters.
ii. The gain (or loss) in beam system capacity / ductility for each configuration is compared to the needs of the vulnerable floor grillage systems.
iii. The candidate configurations satisfying the above point are prioritized based on:
    a. Best use of the already existing configuration (minimal intervention).
    b. Ease and affordability of application based on common construction practice (practicality).
    c. Most constructive interaction for all subsystems (optimal performance).
iv. The final solution is implemented and the response of all floor systems (even those that were not initially vulnerable) is re-assessed against sudden column loss. If the first choice does not satisfy the required resistance demand, then the next candidate configuration, prioritized based on the above criteria, is examined.

Although the methodology can be applied to any type of frame, the solution for each type (non-continuous, semi-continuous and continuous construction) varies based on the different nature of vulnerabilities and of acceptably alterable parameters. However, the initial steps of the methodology are similar.

7.2.2 Prototype structures

The methodology will be applied for two exemplar frames: the NIST special moment frame (SDC-D) and the NIST intermediate moment frame (SDC-C), which are considered representative of moment frames commonly used to resist earthquake loading conditions.

7.3 Addressing moment frame vulnerabilities

7.3.1 Original beam and floor system response

Initially, the individual beam and floor system pseudostatic responses to sudden column loss need to be calculated, which has already been carried out and the results are available in Section 6.5.3. This permits the identification of the vulnerable systems (Figure 6-40) and of the main underlying factors that need to be addressed during the redesigning process.

As the findings of Section 6.5.4 indicate, the simple girder to column connections at the perimeter of the frame (necessary to avoid biaxial bending of the corner column during an earthquake) limit the ability of the supported floors to provide the required pseudostatic resistance. The shortfall in their main response characteristics (capacity, ductility and form of response) is quantified in order to provide a targeted instead of a prescriptive solution; this is illustrated in Figure 6-23 and in Figure 6-24 for the SMF and the IMF respectively.

Although the above information is summarised in Table 7-1, the reader is invited to refer to the previous chapter for a more detailed presentation of the results.

7.3.2 Identification of candidate remediating measures

In the case of the SMF frame with RBS connections, there is one main component (welded beam flange) which controls connection behaviour. Any change in the connection or beam section arrangement is likely to require verifying that the new design does not conflict with seismic design restrictions. In order to ensure that all candidate remediating measures do not contradict common construction practices, advice on popular moment frame arrangements as well as on the remediating measures often employed for enhancing robustness was provided by private communication with Mark Wagoner, a senior associate at Walter P Moore and Associates, (Mark Waggoner, 2012).

The conclusion of the above communication was that improving the performance of the vulnerable subsystem with fully welded moment connections will require either:

i. Drastic connection design interventions: using higher or more ductile steel grades, using a partially restrained or RBS connection or reinforcing the simple connection with coverplates, haunches or other means.

ii. Intervening in the geometry of the beam systems by either shortening the edge bay's dimensions or by installing an additional column near the corner of the frame to reduce the length of the last girder span.

7.3.3 Impact analysis of each candidate solution

7.3.3.1 Special moment frame

For the critical column removal scenarios of the SMF frame, Table 7-1 summarises the factors limiting response, presents the demand for capacity and ductility in a quantitative manner and compares them with those supplied by the two candidate interventions of the previous section. A decrease in ductility is acceptable, as long as the absolute increase in capacity satisfies the pseudostatic demand. For each alternative, a series of parametric tests has identified the optimal configuration able provide this additional resistance (Table 7-3):

- Figures 7-1 and 7-2 show how the floor responses corresponding to scenario Ey1 (dominated by the G1 girder) and Ex1 (dominated by the G2 girder) evolve for different levels of reinforcement of the simple connection tensile components (Improvement i). The minimum level of reinforcement required for the floor response to provide the needed supply of pseudostatic capacity is identified.
- Figures 7-3 and 7-4 show how the floor responses corresponding to scenario Ey1 (dominated by the G1 girder) and Ex1 (dominated by the G2 girder) evolve for shorter corner bay spans (Improvement ii). The response is calculated with the use of the analytical solution presented in Chapter 4 for irregular beam systems.

Table 7-1: Impact of limiting factors and proposed improvements on floor grillage response

<table>
<tr><th>Scenario</th><th>Factors limiting response</th><th>Shortfall</th><th colspan="2">Reserve capacity after improvements</th></tr>
<tr><td rowspan="2">Ex1</td><td rowspan="4">Low pseudostatic capacity of the perimeter beam
Very low strength (%) of the simple beam to corner column connection</td><td rowspan="2">-12% capacity</td><td>(i)</td><td>+8% capacity</td></tr>
<tr><td>(ii)</td><td>+3% capacity</td></tr>
<tr><td rowspan="2">Ey1</td><td rowspan="2">-18% capacity</td><td>(i)</td><td>+10% capacity</td></tr>
<tr><td>(ii)</td><td>+4% capacity</td></tr>
</table>

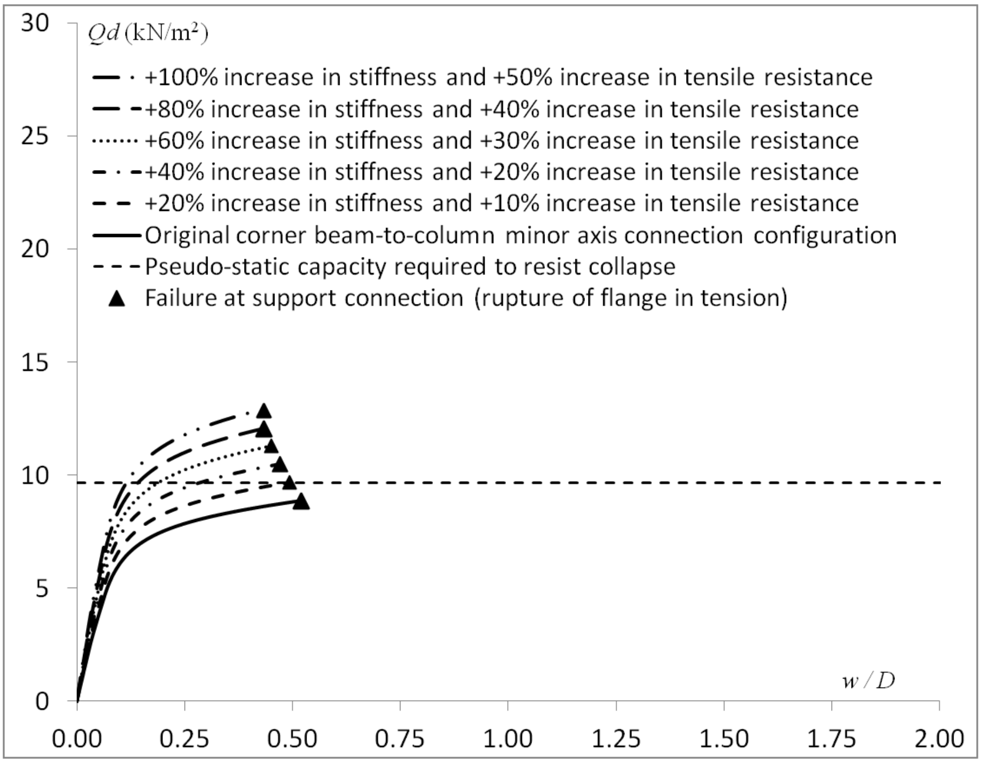


Figure 7-1: Enhanced floor response for scenario Ey1 (NIST SMF) following reinforcement of the simple connection to the corner column

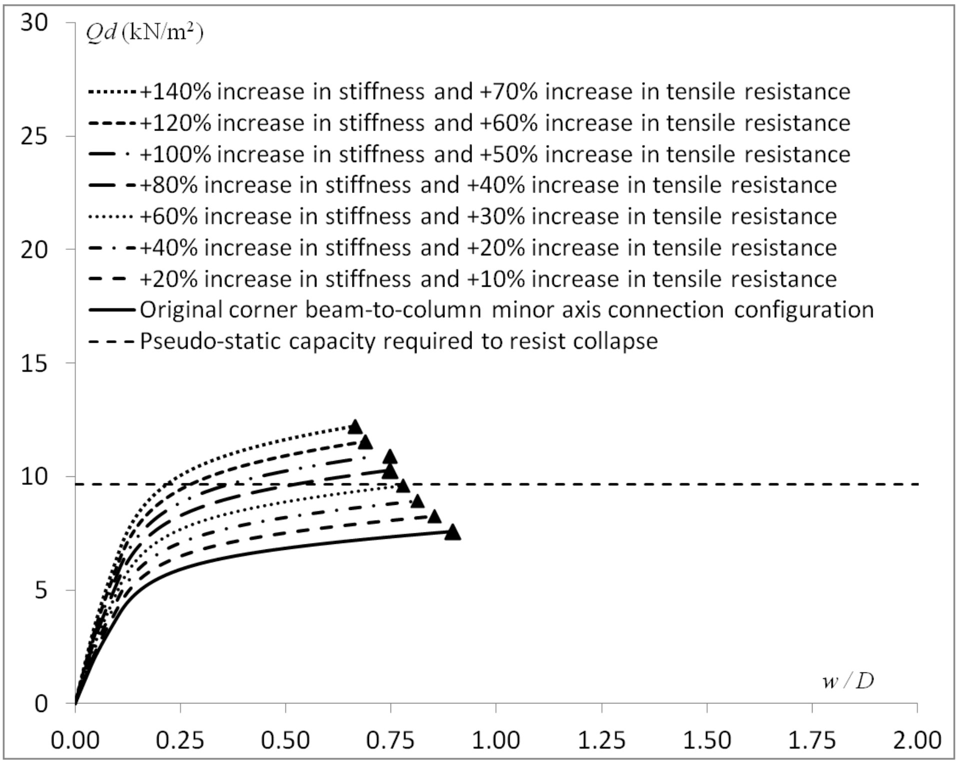


Figure 7-2: Enhanced floor response for scenario Ex1 (NIST SMF) following reinforcement of the simple connection to the corner column

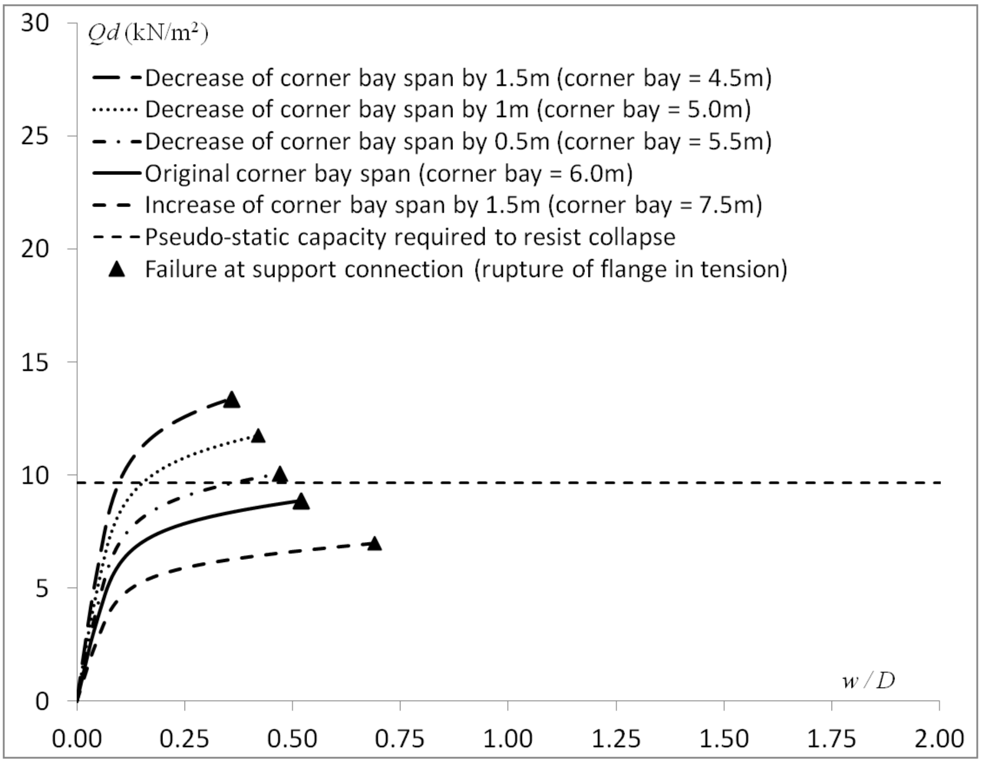


Figure 7-3: Enhanced floor response for scenario Ey1 (NIST SMF) following edge bay span reduction

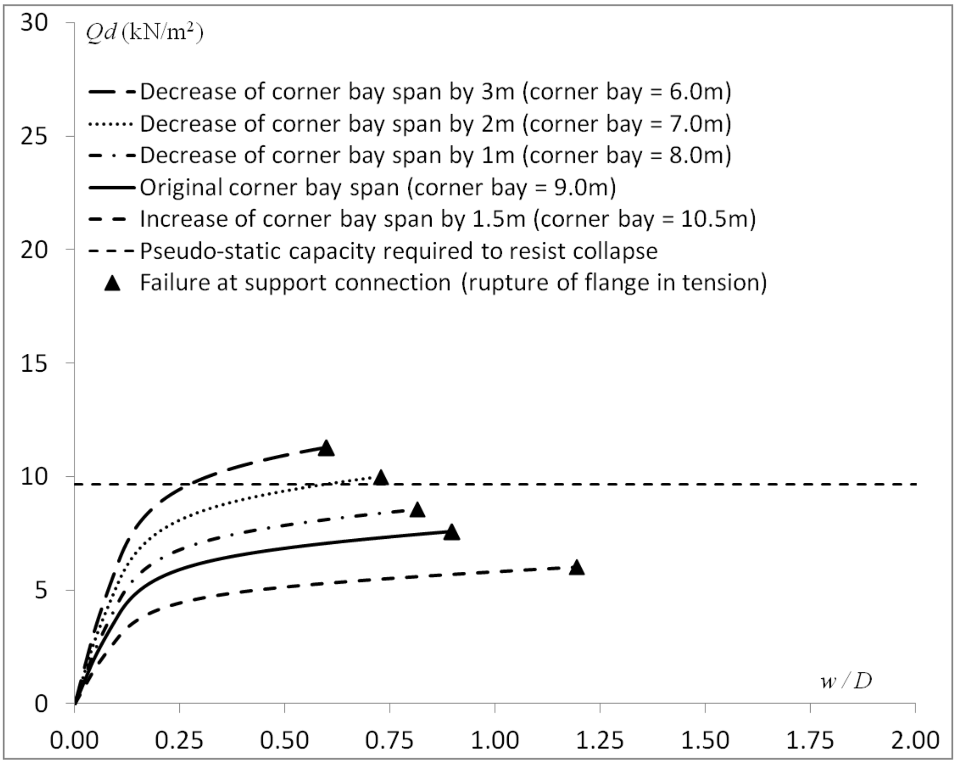


Figure 7-4: Enhanced floor response for scenario Ex1 (NIST SMF) following edge bay span reduction

7.3.3.2 Intermediate moment frame

For the critical column removal scenarios of the IMF frame, Table 7-2 summarises the factors limiting response, presents the demand for capacity and ductility in a quantitative manner and compares them with those supplied by the two candidate interventions of the previous section. In the case of the Ey2 and Ex2 column removal scenarios, reinforcing the simple beam-to-column connection at columns Ey1 and at Ex1 respectively is more practical, easy to apply and effective in enhancing progressive collapse resistance compared to shortening the penultimate bay. Thus, for these scenarios, only Improvement (i) will be considered.

For each alternative, a series of parametric tests has identified the optimal configuration able provide this additional resistance (presented in Table 7-3):

- Figures 7-5 and 7-6 show how the floor responses corresponding to scenario Ey1 (dominated by the G3 girder) and Ex1 (dominated by the G4 girder) evolve for different levels of reinforcement of the simple connection tensile components (Improvement i). The minimum level of reinforcement required for the floor response to provide the needed supply of pseudostatic capacity is identified.
- Figures 7-7 and 7-8 show how the floor responses corresponding to scenario Ey1 (dominated by the G1 girder) and Ex1 (dominated by the G4 girder) evolve for shorter corner bay spans (Improvement ii). The response is calculated with the use of the analytical solution presented in Chapter 4 for irregular beam systems.
- Figure 7-9 and 7-10 show how the floor responses corresponding to scenario Ex2 and Ey2, dominated by the G4 and G3 girder respectively, evolve for different levels of reinforcement of the simple connection tensile components (Improvement i).

Table 7-2: Impact of limiting factors and proposed improvements on floor grillage response

Scenario	Factors limiting response	Shortfall	Improvement	
Ex1	Low pseudostatic capacity of the perimeter beam Very low strength (%) of the simple beam to corner column connection	-30% capacity	(i)	+10% capacity
			(ii)	-4% capacity
Ey1		-45% capacity	(i)	+2% capacity
			(ii)	+4% capacity
Ex2	Very low strength & stiffness of the simple connection in the frame perimeter	-18% capacity	(i)	+11% capacity
			(ii)	+5% capacity
Ey2		-25% capacity	(i)	+8% capacity
			(ii)	+4% capacity

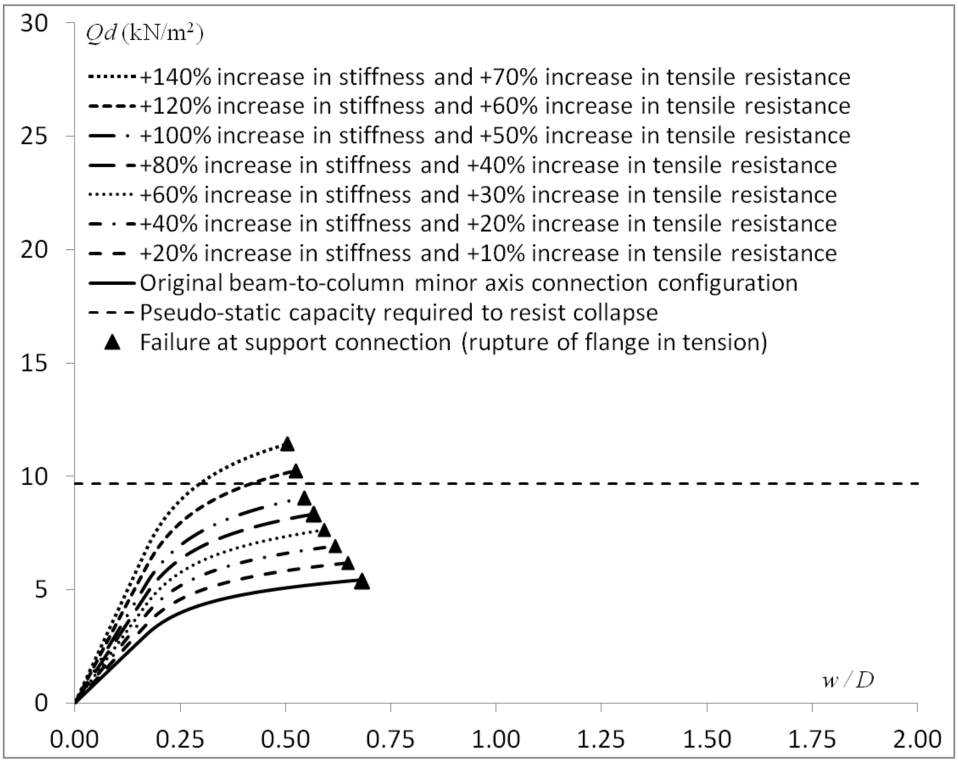


Figure 7-5: Enhanced floor response for scenario Ey1 (NIST IMF) following reinforcement of the simple connection to the corner column

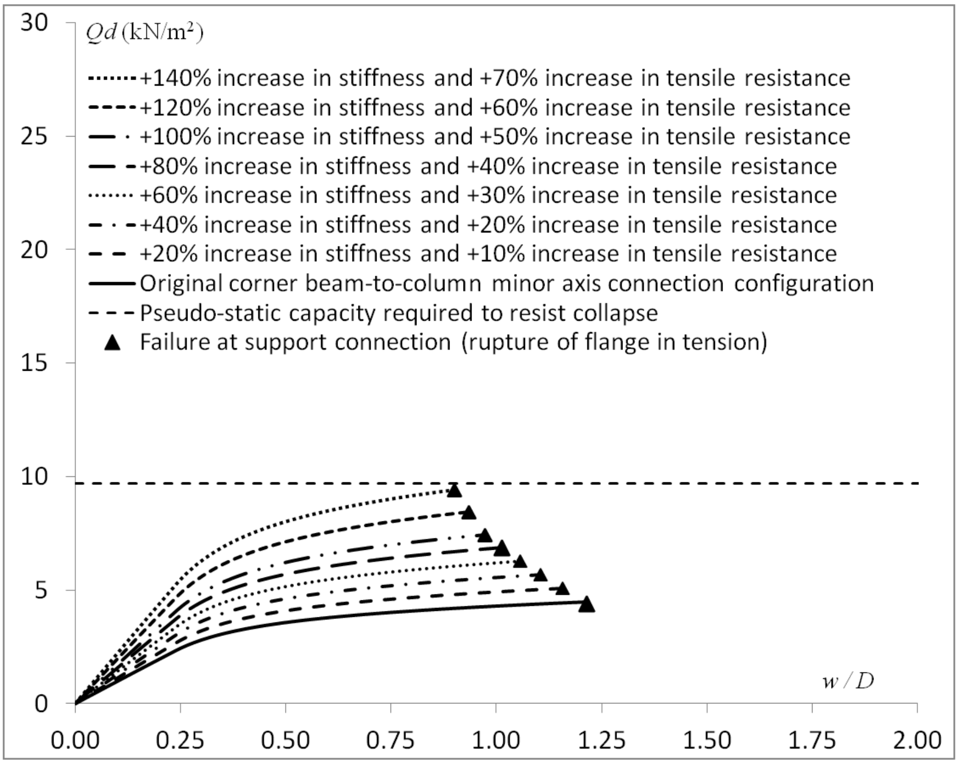


Figure 7-6: Enhanced floor response for scenario Ex1 (NIST IMF) following reinforcement of the simple connection to the corner column

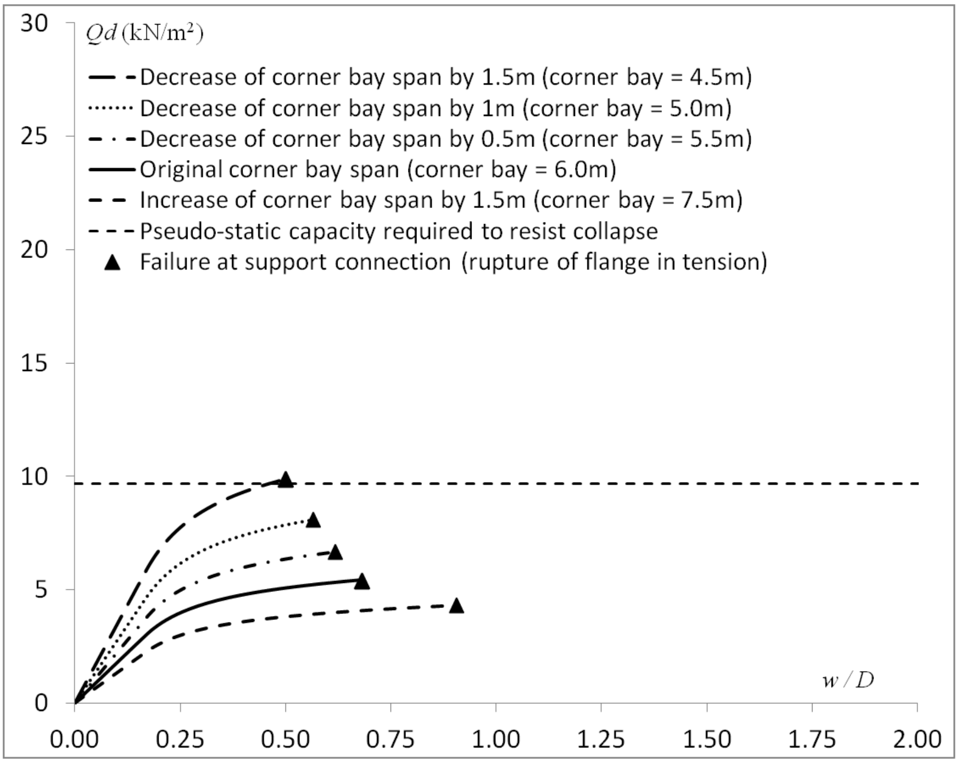


Figure 7-7: Enhanced floor response for scenario Ey1 (NIST SMF) following edge bay span reduction

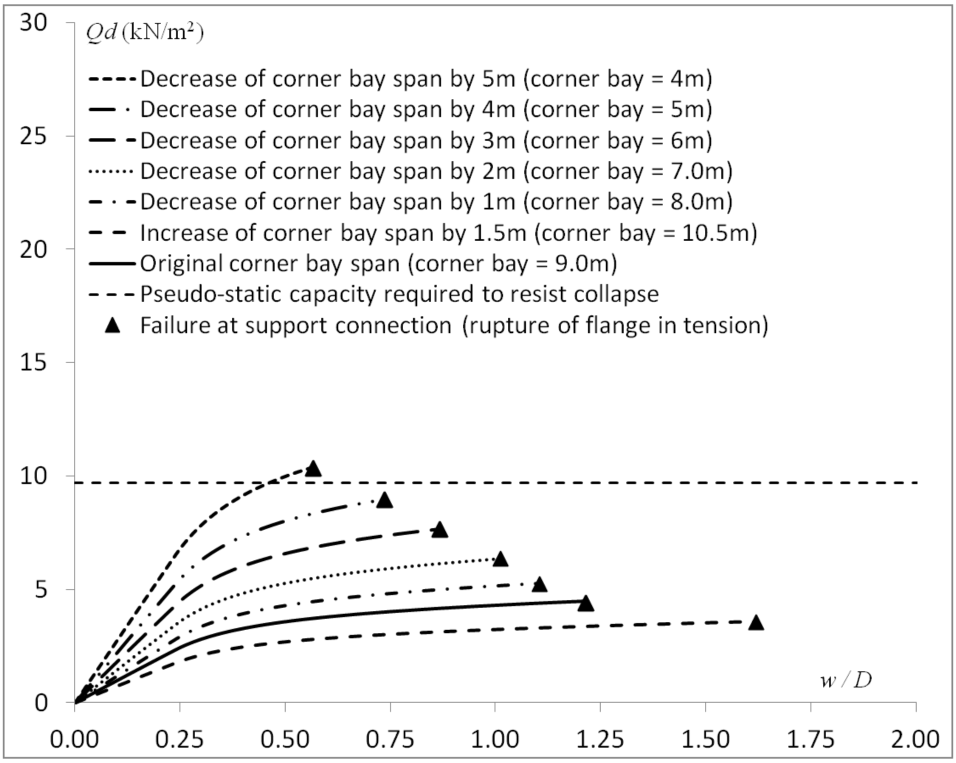


Figure 7-8: Enhanced floor response for scenario Ex1 (NIST SMF) following edge bay span reduction

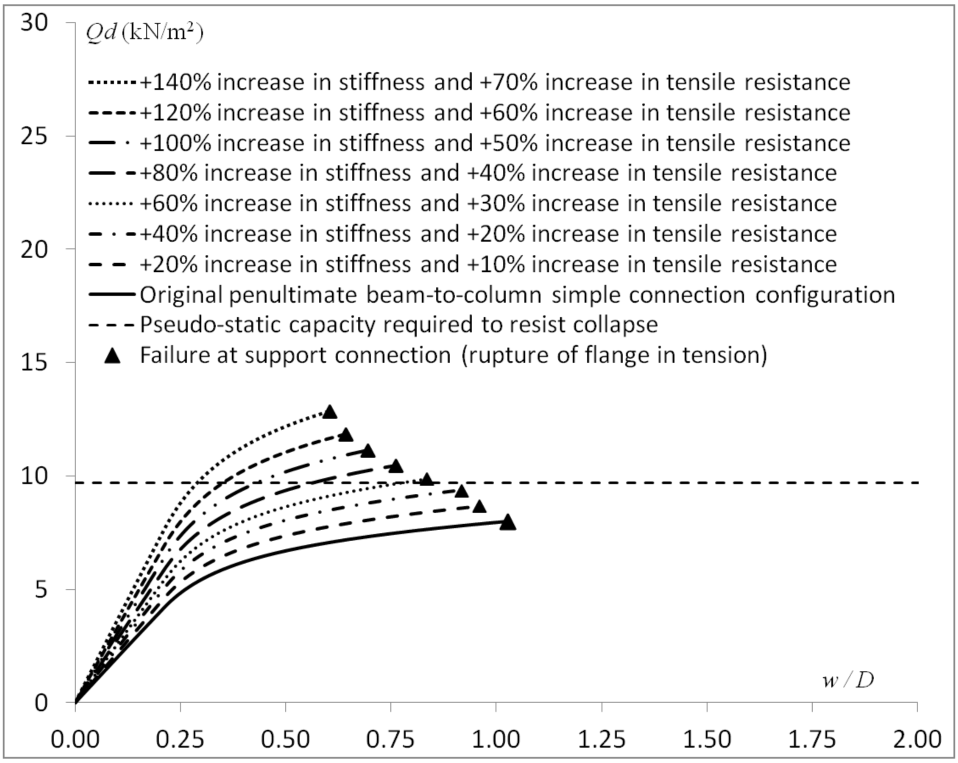


Figure 7-9 : Enhanced floor response for scenario Ey2 (NIST IMF) following reinforcement of the simple connection to the corner column

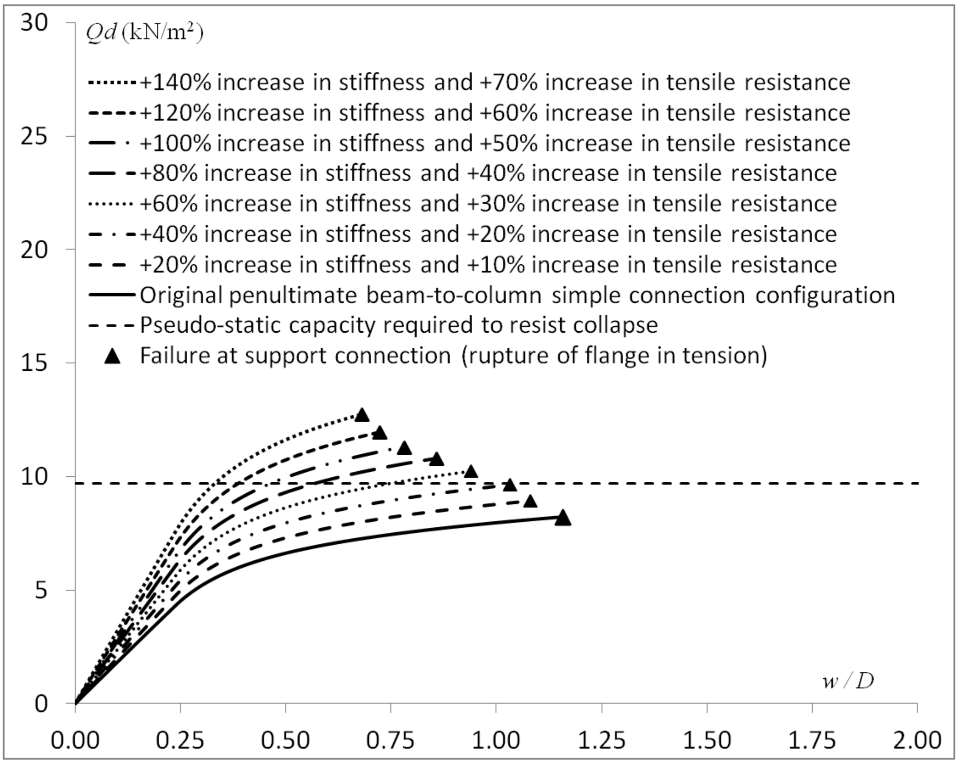


Figure 7-10: Enhanced floor response for scenario Ex2 (NIST IMF) following reinforcement of the simple connection to the corner column

7.3.3.3 Prioritising interventions

According to Section 5.8.1, interventions should be prioritised based on how well they exploit the default configuration (minimal changes), on how practical they are and on how much they contribute to a constructive interaction between subsystems, regardless of the column removal scenario considered.

Taking these priorities into account, the preferred improvement approach is the first one (reinforcement of the simple minor-axis connection), since it satisfies all three above conditions to a higher extent than the second approach, which involves altering the frame arrangement. However, in the case of the intermediate moment frame, reinforcement of the connections to the highest level possible (based on seismic detailing restrictions) cannot assist the vulnerable floors at providing the required pseudostatic capacity (Figure 7-6). Therefore, Table 7-3 summarises which design interventions will be applied to each frame before calculating the enhanced floor response in the next section.

Table 7-3: Final choice of alternative design interventions

Frame	Design intervention	Details
NIST SMF	Reinforcement of the beam minor axis connection to the C1 column	+80% stiffness; +40% tensile resistance
	Reinforcement of the beam minor axis connection to the C2 column	+40% stiffness; +20% tensile resistance
NIST IMF	Reinforcement of the simple connection to the Ey1 column Reduction of girder length in the last bay in the NS direction	+60% stiffness; +30% tensile resistance New corner bay span = 4.5m
	Reinforcement of the simple connection to the Ex1 column Reduction of girder length in the last bay in the WE direction	+60% stiffness; +30% tensile resistance New corner bay span = 4.0m

7.4 Enhanced floor response

In the case of critical scenarios of the SMF frame (Figure 6-40a), Figures 7-11 and 7-12 show the enhanced floor response to column loss scenario Ey1 and Ex1 respectively after reinforcing the simple connection at the corner column in hogging bending moment in order to avoid the complete cantilever behaviour of the original arrangement. Results show that although capacity is enhanced at the expense of ductility, the new arrangement is able to withstand collapse. In the case of the critical scenarios of the IMF frame (Figure 6-40b), Figures 7-13 and 7-14 illustrate the enhanced floor response to column loss scenarios Ey1, Ex1, Ey2 and Ex1 after reducing the length of the edge bay and after reinforcing the simple connection to the penultimate column. Results show that - similar to the SMF - capacity is enhanced at the expense of ductility.

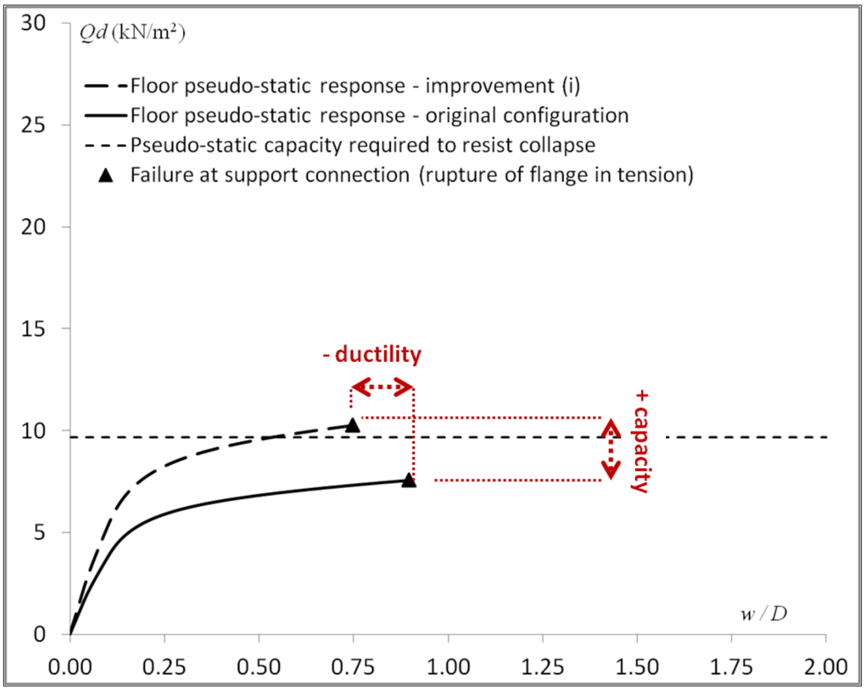


Figure 7-11 : Q-w for the default and improved floor response for the Ey1 column loss scenario (SMF)

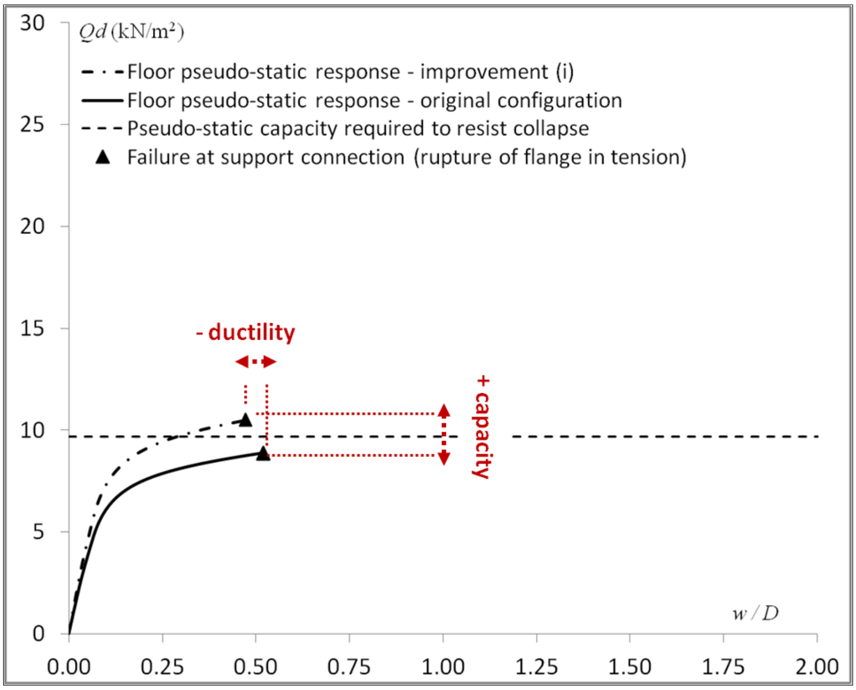
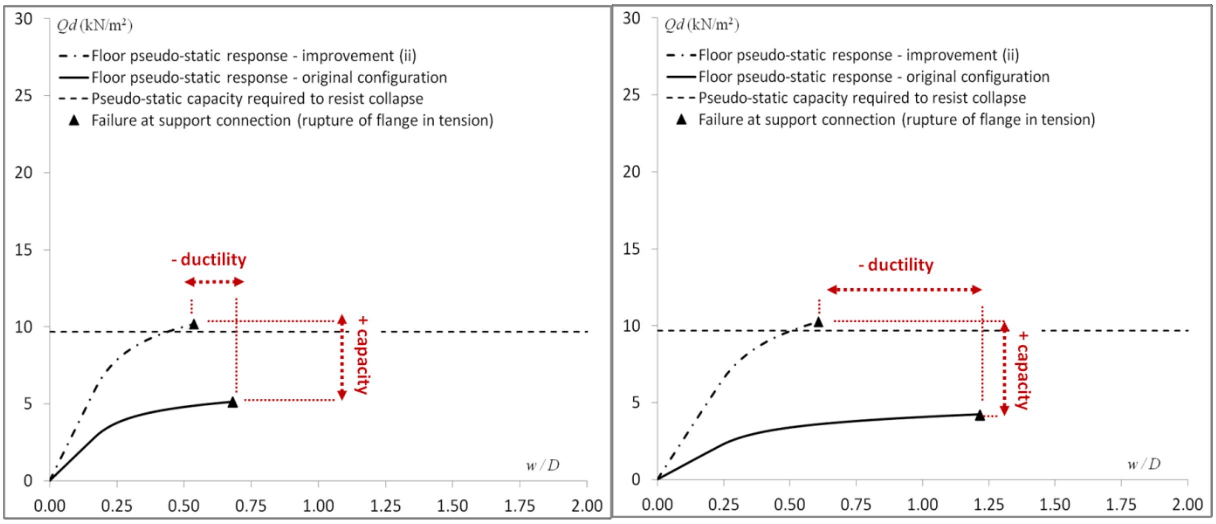
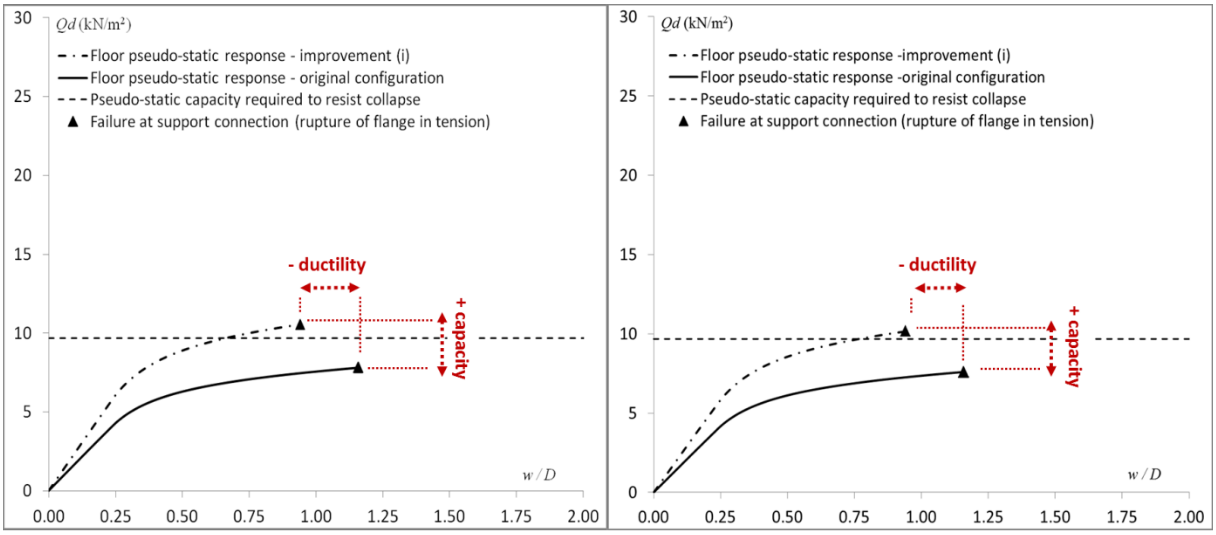


Figure 7-12 : Q-w for the default and improved floor response for the Ex1 column loss scenario (SMF)



a　　　　　　　　　　b

Figure 7-13 : Q -s for the default and improved floor response for the a) Ey1; b) Ex1 column loss scenario (IMF)



a　　　　　　　　　　b

Figure 7-14 : Q -s for the default and improved floor response for the a) Ex2; b) Ey2 column loss scenario (IMF)

For the vulnerable systems of the SMF and the IMF frame**,** Table 7-4 presents a comparison between the variations in capacity and ductility of the constituent individual beam systems and the floor grillage assemblies for each solution. The enhanced response (for both proposed improvements) is compared to that of the original arrangement for each column removal scenario for both redesigned frames.

Table 7-4: Capacity-demand ratio gain percentage under sudden column loss

Frame	Column removal scenarios	Capacity-Demand ratios; $r = q_{Rd}/q_{sd}$		
		Initial configuration	Improved configuration	
			(i)	(ii)
SMF (SDC-D)	Ey1	0.82	1.10	*n/a*
	Ex1	0.88	1.08	*n/a*
IMF (SDC-C)	Ey1	0.70	*n/a*	1.03
	Ex1	0.55	*n/a*	1.04
	Ey2	0.82	1.08	*n/a*
	Ex2	0.75	1.11	*n/a*

7.5 Comparison between improving resistance in moment and simply designed frames

Section 6-8 compares the behaviour of each type in progressive collapse and highlights the different influencing factors, which mainly depend on the connection performance and on the interaction between the participating beam systems in the floor grillage. By considering these findings together with the findings of the case studies presented in Chapter 5 and in Chapter 6 it is possible to confirm that:

i. Regardless of the level of average performance of the beam and floor systems, both types of frames were found to be vulnerable in progressive collapse under certain column loss scenarios.

ii. For both types of frames, it was possible to introduce effective and efficient design solutions by using the Imperial College London redesigning methodology, that will guarantee adequate performance under any scenario considered.

However, enhancing the resistance of different types of frames in progressive collapse highlights the differences in how the designer should approach the problem in each case:

i. Table 7-5 demonstrates - by comparing the fixed and alterable parameters for the two types of construction - that the range of alterable (non-native to the frame) parameters for simply designed frames offers considerably more freedom to the designer. On the contrary, continuous construction interventions (Section 7.3.2) are likely to represent an expensive, customised and intrusive solution.

Table 7-5: Fixed and alterable frame, beam and connection parameters

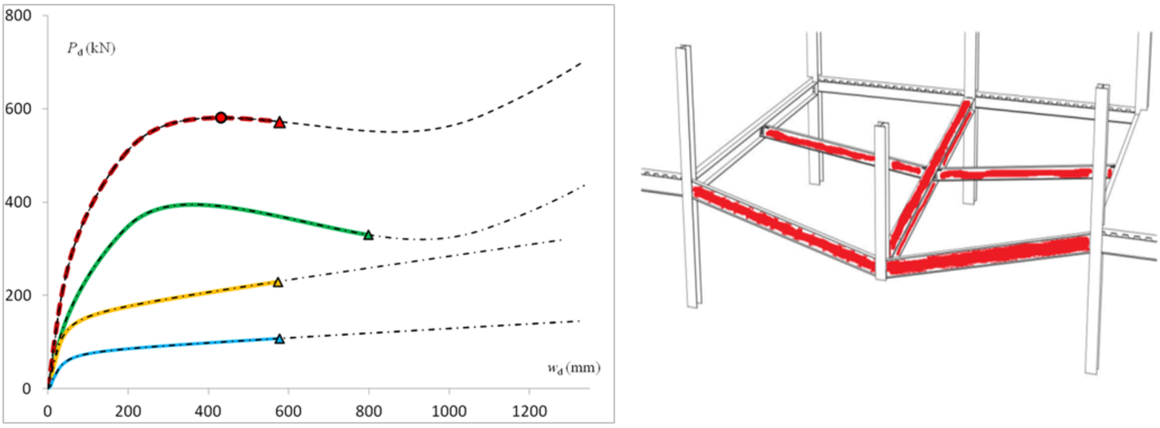
Parameters		Fixed (X) or Alterable (√)	
		Moment frames	Simple or P-R [a]
Frame	Frame arrangement (connection position)	(X)	(X)
	Bay dimensions	**(√)**	(X)
	Axial restraint	(X)	(X)
Beam	Thickness of slab and profile height	*N/A*	(X)
	Column section	(X)	(X)
	Beam moment capacity/ axial stiffness	(X)	(X)
	Beam Section depth	**(√)**	(X)
	Reinforcement ratio (ρ)	*N/A*	**(√)**
	Span/depth ratio (indirectly)	**(√)**	**(√)**
	Beam length (indirectly)	**(√)**	**(√)**
Connections	Bolt size	*N/A*	(X)
	Bolt row geometry and number	*N/A*	(X)
	Endplate thickness t_p	*N/A*	**(√)**
	Beam flange stiffening / reinforcement	**(√)**	**(√)**

[a] Partial strength (semi-continuous)

ii. Table 7-6 demonstrates that the underlying reasons for inadequate floor response in simply designed frames can be a combination of different factors. As mentioned in Section 6.8, behaviour in progressive collapse for this type of frames depends not only on a complex interplay of parameters at the subsystem level but also on the ability of the beam systems to interact constructively in the floor response. This is illustrated in Figures 7-15 and 7-16, which demonstrate that while floor response in moment frames is dominated by the perimeter girder system, the response of non-continuous floors is constructed from the contribution of each participating beam system.

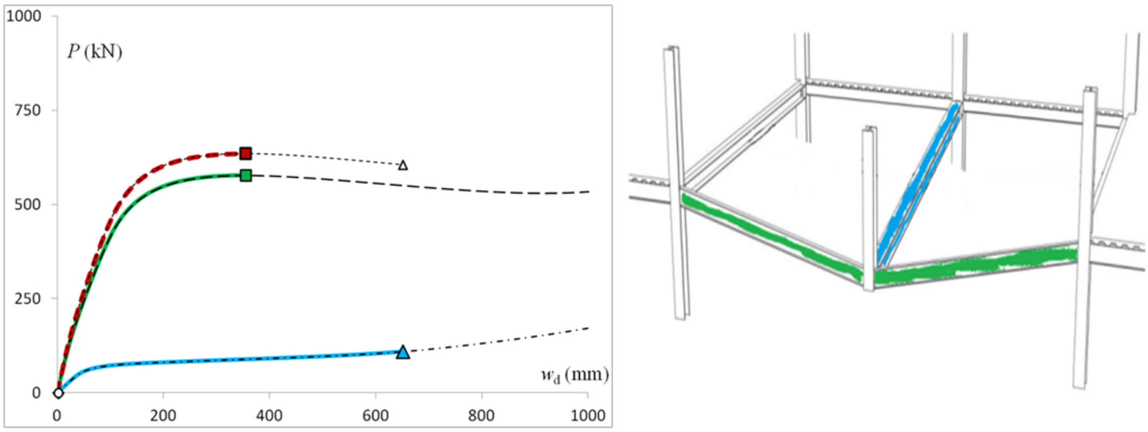
Table 7-6: Vulnerable floors depending on construction type

Type	Vulnerable bays	Figure	Underlying causes
SMF perimeter (continuous)	Edge (few)	7-19a	Simple connection at the corner column to avoid biaxial bending
PR or simple (non-continuous)	Corner (all) Edge (few) Interior (most)	7-19b	Beam system low maximum capacity
			Insufficient connection ductility
			Incompatible response form – beams do not interact constructively
			Combination of the above



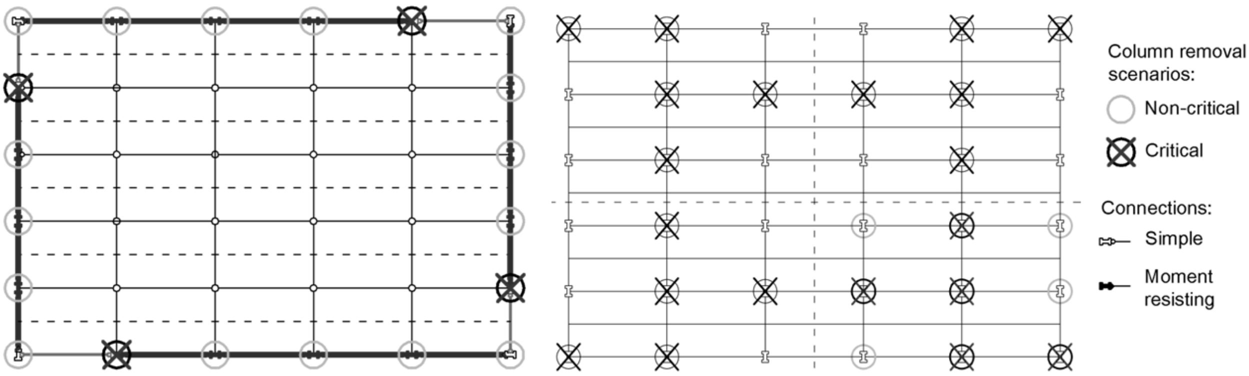
a b

Figure 7-15: Typical contribution of non-continuous beam systems to floor response



a b

Figure 7-16: Typical contribution of continuous beam systems to floor response



a b

Figure 7-17: Critical column removal scenarios for the SMF and the Cardington frame

iii. Due to the clear underlying cause of vulnerability in the case of moment frames, the distribution of critical column removal scenarios is standard. However, for simply designed frames, Figure 7-17 illustrates that each floor needs to be individually assessed to identify whether it can provide the necessary pseudostatic resistance or not. Thus, contrary to most non-continuous frames, there is no need to perform any changes to the other moment resisting connections and bays, since the design interventions are localised.

iv. Enhancing ductility in a seismically designed frame, typically with rigid, full-strength connections, is generally expected to reduce demands in strength. However, for resistance against progressive collapse, ductility can only be beneficial if matched with the necessary strength, as it mainly serves for redistributing the loading after yielding of a component. Also, in the case of welded connections used in this study, enhancing connection rotational capacity might not be readily achievable unless the girder section or connection type is changed.

v. On the other hand, the vulnerability of the non-continuous part of the frame is directly linked to the behaviour of the beam systems: although enhanced connection strength is key to increasing the level of the pseudostatic response, ductility plays a very important role because it controls the critical action phase in which the system fails. If the peak in response during the compressive arching phase is not adequate, very high levels of connection rotational capacity are usually required to provide enhanced resistance in the tensile catenary action phase.

7.6 Relationship between seismic provisions and progressive collapse resistance

The findings of the case studies presented herein have shown that:

i. Frames designed to resist earthquake motions with the use of moment resisting perimeter bays not only have less vulnerable floors within their perimeter but also that the average floor resistance to progressive collapse is higher by approximately 24%. Table 7-7 compares the reserve or lack of capacity for resisting progressive collapse for four frames: the NIST moment frames and the Cardington simplified composite and bare steel equivalent simply designed frames.

ii. Frames designed to resist stronger earthquakes are less vulnerable compared to those designed to resist weaker ones for two main reasons: the more ductile performance of the connections and the fewer “weak spots” (simple connections) at the perimeter of the frame.

iii. The number of critical perimeter column loss scenarios is the same for the composite semi-continuous steel frame and the intermediate moment frame.

iv. The average shortfall in capacity for the critical column removal scenarios is higher for the moment resisting frames than for the simply designed ones.

Table 7-7: Average capacity / demand ratio and number of critical column loss scenarios for each frame

Scenario type	Cardington composite frame		Cardington bare steel frame		NIST special moment frame		NIST intermediate moment frame	
	#	$r_{av} = q_{Rd}/q_{sd}$	#	$r_{av} = q_{Rd}/q_{sd}$	#	$r_{av} = q_{Rd}/q_{sd}$	#	$r_{av} = q_{Rd}/q_{sd}$
Non-critical perimeter column loss scenarios	12	1.14	4	1.05	16	1.98	12	1.32
Critical perimeter column loss scenarios	8	0.94	16	0.67	4	0.82	8	0.63

The aforementioned observations suggest that the relationship between the number of critical perimeter column loss scenarios and the ability of the frame to resist an earthquake is not direct, despite the fact that the average capacity of the floors within the non-critical moment resisting bays is significantly higher compared to that in simply designed frames due to the enhanced properties of their connections.

Thus, seismic provisions and robustness are not directly related, albeit the excellent performance of the moment connections. The main reason is that earthquake resistance requires providing ductility and capacity in the frame as a whole system, while satisfactory response to progressive collapse is largely dependent on local behaviour; adequate resistance must be provided locally in all vulnerable subsystems, i.e. designing “member by member”.

7.7 Summary and conclusions

Beam systems with moment connections can provide substantially higher levels of pseudostatic capacity because of their enhanced stiffness, strength and bending moment resistance. However, not all bays in the perimeter of a moment resisting frame are able to withstand sudden loss of a column.

Frames designed to resist stronger earthquakes are less vulnerable because of the more ductile performance of the connections usually employed and of the increased number of moment resisting connections (fewer “weak spots” / simple connections) at the perimeter.

Using the Imperial College London redesigning methodology, it has been possible to identify solutions that will allow a moment frames to withstand removal of any perimeter column. Compared to simply designed frames, more intrusive design alterations are required to enhance robustness, as care is required to avoid a conflict with seismic requirements. The proposed improvements, which involve small changes to the frame layout or selectively reinforcing connections, are thus likely to represent an expensive solution, even though no other changes are required to the rest of the structure.

Despite the excellent performance of moment connections, the findings of this study have concluded that the relationship between seismic provisions and robustness is not direct. While earthquake resistance is based on providing ductility and capacity in the frame as a whole system, resisting progressive collapse largely depends on local behaviour; adequate resistance must be provided locally in all vulnerable subsystems. Thus, seismic provisions are not the most effective means of improving resistance against progressive collapse.

Instead, frame robustness must be carefully examined with a quantitative rather than a prescriptive approach. An example is the ICL methodology, which is able to identify the optimum design for providing the connection strength, ductility and stiffness required to achieve the most constructive beam system interaction in a floor system.

Chapter 8
Closure

8.1 Summary and conclusions

The considerable increase in research activity around the issue of progressive collapse now not only offers a more comprehensive understanding of the phenomenon but also a method for how findings can be used to enhance the robustness of a structure. Research studies at Imperial College London over the past ten years have been oriented towards addressing a wide range of challenges: constructing a suitable analysis framework, producing appropriate structural analysis models, developing an understanding of the underlying mechanisms of progressive collapse and ultimately providing practical and relevant design guidance.

Assessing a structure is possible via various means, an example being the Imperial College London Method, which offers a simplified framework for quantitatively evaluating structural robustness on the basis of pseudostatic capacity supply and demand. Based on a simplified multi-level assembly approach, the response at higher levels of structural idealisation (i.e. full structure or substantial substructure) is obtained by assembling the responses at lower levels (i.e. individual beams). In addition, the dynamic effects are incorporated through a simplified energy equivalence approach which transforms the static response at any level of structural idealisation to pseudostatic. Although such an application normally requires detailed finite element analysis, previous work at Imperial has produced a simplified hand-calculation method for the prediction of the beam nonlinear static response following column removal, which provides a set of explicit equations that link the connection bending moments and deformations, the beam axial load and axial deformation as well as the beam deflection with the beam loading. By employing the appropriate deformation failure criteria for each connection component, the ultimate ductility and pseudostatic capacity of the system can be predicted.

Motivation for this study was principally based on the need to explore how the aforementioned existing quantitative capacity indicators can be translated into specific remediating recommendations and thus provide a tool for determining effective and efficient structural modifications to ensure robustness. This is an essential component for enhancing the present construction codes with guidelines capable of providing efficient and safe design provisions for routine design use.

The present research has also clarified on the effectiveness and efficiency of two popular strategies towards enhancing the resistance to progressive collapse for framed structures - the use of seismic provisions and the tying of members (a favourite among designers) - by comparing them with a novel redesigning methodology.

Progressive collapse resistance of steel moment resisting frames

The question of whether seismic provisions are an effective and efficient way of enhancing resistance against progressive collapse was considered. In order to assess the performance of a commonly employed structural system, the moment resisting frame, appropriate connection and beam analysis models were developed for continuous structural systems.

Welded connection and irregular beam system modelling

Three types of popular welded moment resisting connections (reduced beam section, welded unreinforced and welded reinforced with coverplates) were modelled using the Component Method and appropriate failure modes and criteria for large rotations and combined bending moment and strong axial forces' loading were introduced. Validation was achieved by comparison against both experimental and detailed numerical results. The following conclusions regarding the behaviour of beam systems with welded connections were defined:

i. The behaviour of this type of connections is greatly dependent on material properties, which might not always allow for explicit simulation of experimental tests. However, it is still possible to examine the controlling aspects and components of behaviour.

ii. For equal beam depths and flanges sizes, results showed that the RBS arrangement is the most ductile. Welded unreinforced connections have almost half the rotational capacity, which makes them unlikely to have the necessary ductility for the beam system to enter the tensile catenary phase when resisting a progressive collapse scenario. Coverplated connections demonstrated a relatively superior performance against other types of reinforced connections in progressive collapse.

In addition, an analytical method for the prediction of the nonlinear static response following column removal of a double span irregular beam system, commonly used in the corner bays of moment frames, was introduced. By incorporating the aforementioned connection models into an extended slope-deflection model, which accounts for the interplay between the beam and connection structural parameters at the various stages of the response, it was possible to capture the essential features of progressive collapse in an explicit manner. A verification exercise was performed by comparing the results obtained by the use of detailed numerical models and the analytic method. The preliminary findings showed that the interplay between the increased number of structural elements - compared to regular beam systems - may lead to different considerations for their behaviour. In general, the stronger and stiffer connections and the shorter and deeper beams of the system are expected to dominate the response.

Moment frames' case studies

A series of representative moment frames, based on the NIST Building Resilience and Structural Robustness Project and the SAC Joint Venture, was extensively studied with the ICL Method in terms of their ability to withstand progressive collapse following loss of a ground level column, which led to the following observations:

i. The contribution of non-continuous systems in continuous floors was minor.

ii. Deep sections and short beam spans enhanced the ability of continuous beam systems to provide the necessary resistance.

iii. The stiffness of moment connections prevented the system from developing a compressive arching resistance mechanism, making axial restraint at the support joints less influential to maximum capacity, while the connection bending stiffness and resistance played the most influential role.

iv. All moment frames that were examined were found capable of resisting the majority of perimeter column removal scenarios while providing reserve capacity between 20% and 130%. Beam systems employing full strength connections dominated the floor response.

v. Corner bays of these frames were vulnerable to progressive collapse, as the simple connections at the edge (connecting either edge or internal beams) were unable to provide the required pseudostatic capacity.

vi. Frames designed to resist stronger earthquakes were vulnerable to less column removal scenarios and employed more ductile connections.

Comparison of progressive collapse behaviour between simply designed and continuous systems

Comparison with a complementary study of two simply designed frames (the Cardington composite and its equivalent bare steel test frame) highlighted certain differences in their behaviour.

On the one hand, the behaviour of simply designed frames with partially restrained connections was influenced by a rather complex interplay between various parameters: axial restraint, connection stiffness and strength, beam depth to length ratio and the balance between the resistance of the connection tensile and compressive components. The fact that the connections were asymmetrical - potentially weaker in resisting hogging bending moments – also meant that there was a large difference between the behaviour of the support and centre connections in a double-span beam system. The response comprised four main action phases: elastic, compressive arching, transient catenary and tensile catenary. The ductility of the system defined which of these was critical and different priorities were considered in each case. Finally, the behaviour of the floor grillage assembly system was influenced not only by the maximum capacity of the participating beam systems but also by their relative ductility and the form of response of each subsystem.

On the other hand, the relationship between the influencing parameters in moment frames with fully welded connections was more simple and straightforward. Transient catenary action was usually the critical action phase, during which the connection's bending resistance was more important than its tying capacity. Also, connection behaviour was usually dominated by one component, which was the beam flange in combined bending and tension or compression. Finally, floor grillage behaviour was dominated by the perimeter moment frame and floors with continuous beams were substantially more resistant against progressive collapse compared to non-continuous systems, mainly because of the:

i. Enhanced connection stiffness and strength in bending and not just in tension;

ii. Increased connection ductility;

iii. Connection ability to resist both hogging and sagging bending moments.

Moreover, the column removal scenarios in simply designed frames most sensitive to progressive collapse were not always the perimeter ones; internal columns, which usually comprise axially restrained systems, supported larger areas and thus had to provide increased pseudostatic resistance. On the contrary, “weak spots” at the perimeter of moment frames were the underlying cause of their vulnerability.

Redesigning framed structures to resist progressive collapse

It appears that there is more than a single solution for improving resistance to progressive collapse, though most are limited by their cost and their incompatibility with common construction practices. Thus, the need to concentrate on determining the most efficient way to enhance robustness of a building for certain given design configurations was addressed with the introduction of a step-by-step methodology, which takes into account the complex interplay between connection strength, stiffness and ductility, as well as the frame arrangement. With the use of the proposed process, it is possible to redesign any frame in a way that it will be sufficiently robust to cope with any sudden column removal scenario.

Improving progressive collapse resistance in simply designed and moment resisting frames

The aforementioned study of exemplar frames showed that:

i. Regardless of the average performance of beam and floor systems, both types of frames were vulnerable to progressive collapse under certain column loss scenarios.

ii. For both types of frames, it was possible to introduce effective and efficient design solutions by using the Imperial College London redesigning methodology, able to guarantee adequate performance under any scenario considered.

By applying the methodology on non-continuous frames, it was possible to determine common improvements to the connection configuration for all beam-to-column connections, instead of having to employ more than one configuration for each connection type in the frame. The key observations on resistance enhancement for this type of frames study were:

- Whilst the response of the individual beam systems is of interest in order to understand which physical parameters influence the response to sudden column loss and how this is achieved, it is actually the response at the higher levels of structural idealisation, namely the floor grillage that principally determines the resistance of the frame against progressive collapse.

- In order for the floor grillage response to reflect the maximum resistance of the constituent systems, the critical mode of behaviour of the individual beam systems has to be identical in terms of ductility.
- If the connection critical component is not realistically alterable, changes in other parameters will have a limited effect on the system pseudostatic capacity. Some components are more practical to modify, such as the endplate and the reinforcement, while others require more tenuous and complex work, such as the beam compressive flange stiffening.
- Enhancing the resistance of simply designed frames efficiently and effectively requires taking into account a set of interconnected parameters. Nevertheless, if the designer had to choose one priority for enhancing robustness, the most common weakness of such frames is low connection strength.

For moment frames, the range of alterable (non-native to the frame) parameters offers less freedom to the designer. In this study, the proposed improvements involved small changes to the frame layout or selectively reinforcing connections; both are likely to represent an expensive and intrusive solution even though no other changes were required to the rest of the structure. Enhancing the resistance of moment frames efficiently and effectively requires taking into account these parameters. Nevertheless, if the designer had to choose one priority for enhancing robustness, the most common weakness of moment frames lies in the connection arrangement at the corner columns and in the weak connections in the gravity frame.

Relevance of tying capacity provisions

In general, connection rotational capacity and strength depends on the interplay between the properties of three types of components: compressive, tensile and shear; yielding of either reduces the loading in the other. The findings of this study concluded that taking into account solely the behaviour of the tensile components appears to oversimplify the problem and is not a directly useful measure of resistance to progressive collapse. In all cases, catenary action required both an important degree of support axial restraint and substantial connection rotational capacity in order to be activated. Without both, the system failed before or very shortly after entering the tensile catenary action phase. Most of the systems examined did not reach this phase because of the extreme rotational capacity requirements at the connections.

However, reinforcing the tensile capacity of most connection types also enhanced their ability to resist compression, shear and bending moment, hence often leading to the misunderstanding that increasing tying capacity will have a direct and proportional effect on the frame's resistance to progressive collapse.

Comparison of tying capacity provisions with the Imperial College London redesigning methodology has demonstrated that the latter provides a significantly more ductile, lighter (in terms of connection component size) and to some extent more resistant frame.

Relationship between seismic provisions and progressive collapse resistance

Enhancing ductility in a seismically designed frame, typically with rigid, full-strength connections, is generally expected to reduce demands in strength. However, based on the findings of this study, for resistance against progressive collapse, ductility can only be beneficial if matched with the necessary strength, as it mainly serves for redistributing the loading after yielding of a component.

Thus, despite the excellent performance of moment connections, the relationship between seismic provisions and robustness cannot be characterised as direct. While earthquake resistance is based on providing ductility and capacity in the frame as a whole system, resisting progressive collapse largely depends on local behaviour; adequate resistance must be provided locally in all vulnerable subsystems, i.e. designing "member-by-member".

This means that seismic provisions, albeit being towards the right direction, were not found to be the most effective means of improving resistance against progressive collapse. Instead, the study concluded that frame robustness should be carefully examined with a quantitative rather than a prescriptive approach, which is able to identify the optimum design for providing the connection strength, ductility and stiffness required to achieve the most constructive beam system interaction in a floor system.

8.2 Suggestions for future research

The outcomes of the current study, as summarised in the previous section and explained in detail in the previous chapters, can assist future studies with the development of appropriate tools and complete methods for improving buildings' resistance to progressive collapse. Some suggestions that could potentially facilitate the process are given in the first part of this section. The second part presents further suggestions for future studies into different features of the problem - complementary to the specific features explored in this study – that should also be considered in the design process.

8.2.1 Suggestions based on the outcomes of the current study

As noted in the presentation of welded connection models, there are still uncertainties that need to be overcome, in order to expand their applicability and improve their accuracy:

- *The strain rate effects* under dynamic loading should be studied, in order to ensure that the performance levels employed are not overly conservative. Numerical studies (Pereira, 2012) have confirmed that the increased dynamic strength of structural components may indeed enhance performance in progressive collapse and, therefore, these effects should be incorporated into the design methods.
- *The strain hardening effects* may cause the proposed design method to underestimate the ultimate capacity; a further sensitivity study should be carried out to ensure that the solutions do not provide conservative results.
- *The applicability range of the criteria for inelastic local buckling initiation* used in this study is relatively limited; additional data, preferably based on experimental tests, could help generalise the proposed models and validate the failure criteria employed.

The solution for the nonlinear response of irregular beam systems can be used to examine a wide range of new arrangements which have not been studied before; future studies can reveal how different elements (different types of connections, connections to concrete cores, etc.) interact with each other and which is the optimum configuration in order to resist collapse.

The choice of alterable parameters in the redesigning case studies of the Cardington, NIST and SAC frames was based on available existing information through communication with engineering professionals. Nevertheless, a broader study of candidate interventions might identify ingenious solutions and introduce novel design arrangements, which could potentially be better able at responding to progressive collapse loading conditions.

The proposed redesigning methodology is an important step towards enhancing current guidelines for designing progressive collapse resistance into structures. However, further case studies on simply designed frames, composite moment frames, earthquake resistant braced frames and reinforced concrete frames will not only contribute in its improvement and credibility but are also a necessary prerequisite for achieving acceptance from structural engineers.

8.2.2 Further suggestions

Behaviour of steel and composite structures in progressive collapse

The current study has explored how to improve the progressive collapse response of typical simply designed and moment resisting steel and composite buildings by focusing on the basic features of the behaviour of beam and grillage systems following sudden column loss. However, the topic of progressive collapse has many aspects and will, therefore, remain to the fore of research activity in the future. Some features of the problem that may need to be explored in subsequent research studies are outlined next:

- *Connection, beam and floor system post-limit behaviour:* The post-limit stiffness of the connection components should receive more systematic study and corresponding provisions should be explicitly introduced into the design codes, especially when performance can be significantly enhanced by the redistribution of forces to other structural elements. Available experimental data may be used in these studies as well as appropriate tests may be conducted where possible.
- *Connection resilience:* The ability of the connection not to fail under the circumstances that caused the column to fail is measured by the connection's torsional and weak-axis flexural strength, its robustness and available ductility; further studies are required to examine whether the contribution of the connections at the point of the removed column should be considered as reduced for safety reasons or not.
- *Steel decking membrane effects*: Although the contribution of the floor slab in partially and fully restrained systems is only pronounced in relatively large deflections, it might significantly enhance performance for very ductile arrangements

by facilitating the redistribution of forces and by providing additional resistance. In this regard, these effects should be further studied and incorporated into the assessment exercise.

- *Progressive collapse performance of steel structures in fire*: Ductility and strength are strongly influenced by temperature, especially in steel structures. Sudden column removal scenarios are often accompanied by a dramatic increase in temperature. For example, in the WTC collapse, the original structural study had taken into account the possibility of a plane crash but not the effects of extended fuel fires. Thus, the question of whether the effect of fire on the behaviour of the connections and beams should be included in the assessment should be considered.

- *Effect of sudden column loss removal on the surrounding structure:* The inability of the remaining columns – both at the same floor or those below and above - to sustain the redistributed load originally supported by the failed column may lead to horizontal propagation of failure which will most likely result in disproportionate collapse. Therefore, the multi-level approach of the Imperial College design framework should be developed accordingly in order to account for the resistance of those structural members and whether their stability might affect the redistribution of loading during the structural response.

Designing resistance in progressive collapse into framed structures

Connection ductility requirements for considering tying capacity provisions: The introduction of a globally applicable method, equivalent to current approaches for determining connection tying capacity and strength, will allow designers to compare arrangements based on their ability to effectively reach the tensile catenary action phase without the need of an in-depth assessment of the structural response under progressive collapse loading conditions.

Design provisions based on the interplay between tying capacity, ductility and connection component capacity balance: This is a necessary prerequisite for the introduction of a framework that identifies the most important design priorities to consider based on the ductility, tying capacity and compressive arching contribution of the lower level structural components. For example, if the connections employed are dominated by tensile component strength and are very ductile, it would be possible to limit the assessment process within consideration of only tying capacity provisions, given that the aforementioned ductility requirements are met.

It is believed that the basic assessment framework developed previously at Imperial College London and the new developments of the current study - particularly the introduction of a novel redesigning methodology for enhancing the current design approaches based on proper treatment of the mechanics of progressive collapse - will open the way for enhancing the present construction codes with more efficient and relevant guidelines. Future research studies can, therefore, build on these developments to arrive at safe design provisions for routine design use.

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Appendix A

Validation exercise for the welded connection models

Validation with ADAPTIC - results for the RBS connection (Chapter 3):

Table A-1: Deviation between the ICL model and FE static responses; RBS connections

D_{Pd} = -5.29%	Section size →				*Sensitivity*[1] to beam length*
Length ↓	W24x76	W24x94	W27x102	W30x108	
L = 4.5 m	-4.62%	-3.21%	-5.32%	-	*1.08%*
L = 6 m	-3.61%	-4.57%*[2]	-4.55%	-10.81%	*3.31%*
L = 9 m	-	-4.97%	-4.65%*[2]	-9.77%	*2.87%*
Sensitivity[1] to section size*	*0.96%*	*1.90%*	*0.94%*	*0.74%*	

Table A-2: Deviation between the ICL model and FE pseudostatic responses: RBS connections

D_{Pd} = -4.42%	Section size →				*Sensitivity*[1] to beam length*
Length ↓	W24x76	W24x94	W27x102	W30x108	
L = 4.5 m	-4.62%	-3.21%	-5.32%	-	*1.08%*
L = 6 m	-3.76%	-4.13%*[2]	-2.48%	-7.56%	*2.17%*
L = 9 m	-	-3.85%	-2.43%*[2]	-6.87%	*2.27%*
Sensitivity[1] to section size*	*0.61%*	*0.47%*	*1.66%*	*0.49%*	

*[1] Sensitivity = standard deviation

*[2] Beam systems in the moment resisting perimeter of the prototype framed structures examined in this study.

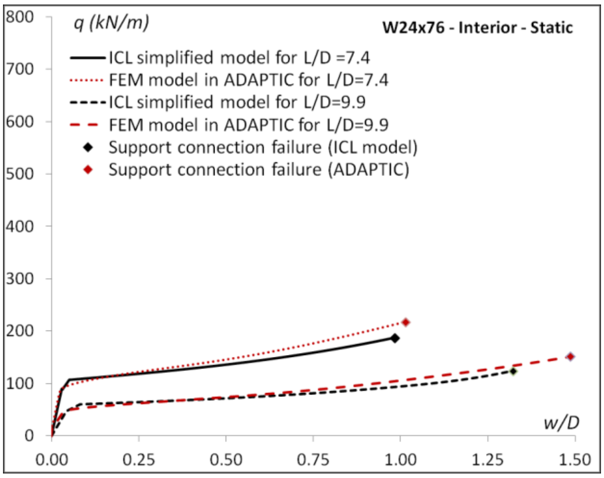

Figure A-1: Static response; W24x76 axially restrained with RBS beam-to-column connections

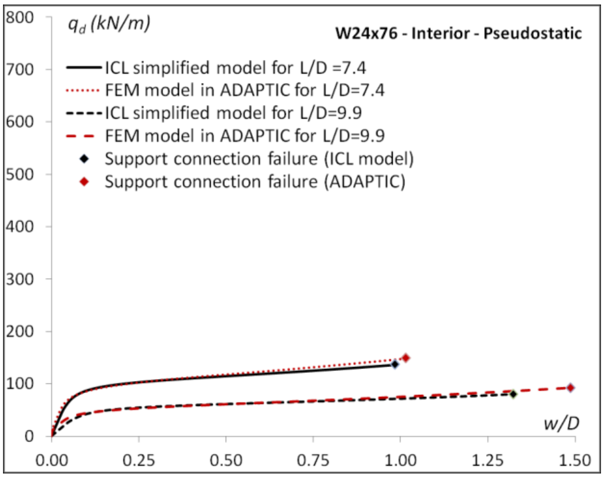

Figure A-2: Pseudostatic response; W24x76 axially restrained with RBS beam-to-column connections

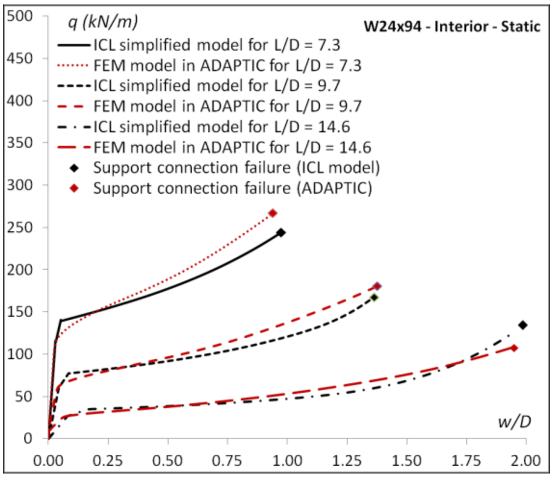

Figure A-3: Static response; W24x94 axially restrained with RBS beam-to-column connections

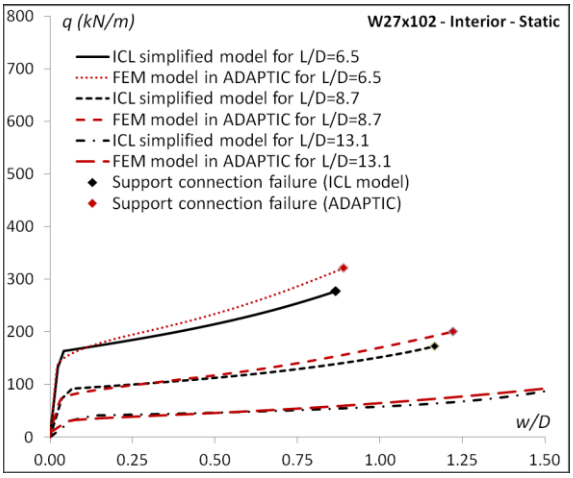

Figure A-4: Static response; W27x102 axially restrained with RBS beam-to-column connections

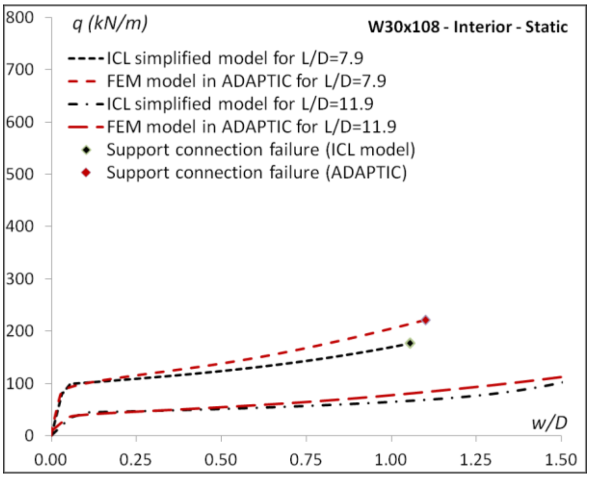

Figure A-5: Static response; W30x108 axially restrained with RBS beam-to-column connections

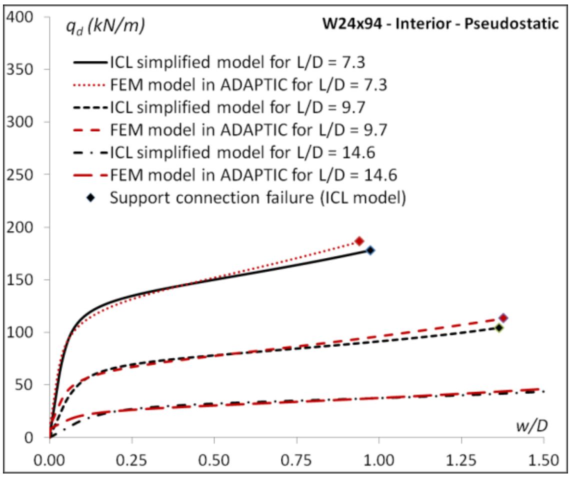

Figure A-6: Pseudostatic response: W24x94 axially restrained with RBS beam-to-column connections

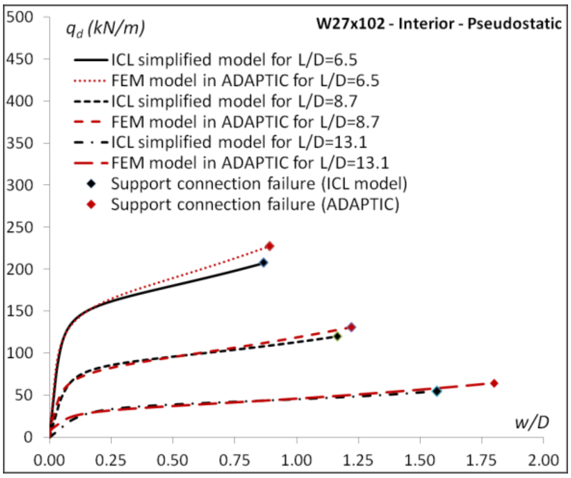

Figure A-7: Pseudostatic response; W27x102 axially restrained with RBS beam-to-column connections

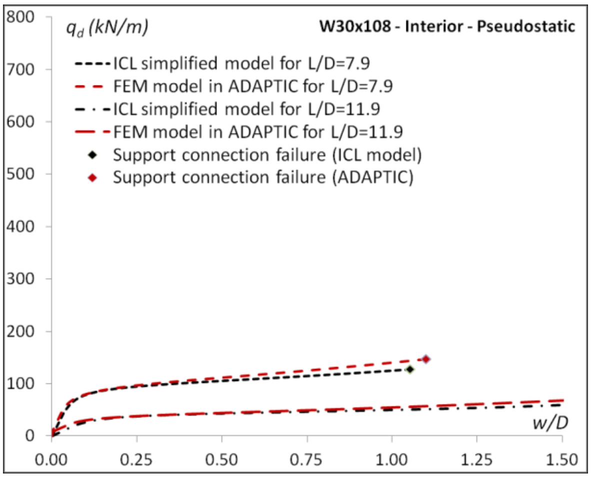

Figure A-8: Pseudostatic response; W30x108 axially restrained with RBS beam-to-column connections

Validation with ADAPTIC – results for the WUF-B connection (Chapter 3):

Table A-3: Deviation between the ICL model and FE pseudostatic responses

D_{Pd} = 7.78%	Section size →				Sensitivity*[1] to beam length
Length ↓	W21x68	W21x73	W24x62	W24x76	
L = 4.5 m	9.14%	9.01%	5.86%	3.97%	*2.52%*
L = 6 m	11.28%	10.34%*[2]	7.54%	7.16%	*2.04%*
L = 9 m	7.89%	7.99%	6.59%	6.55%*[2]	*0.79%*
Sensitivity[1] to section size*	*1.72%*	*1.18%*	*0.84%*	*1.69%*	

Table A-4: Deviation between the ICL model and FE static responses

D_P = 5.02%	Section size →				Sensitivity*[1] to beam length
Length ↓	W21x68	W21x73	W24x62	W24x76	
L = 4.5 m	2.63%	1.84%	-3.77%	0.00%	*2.85%*
L = 6 m	8.82%	7.90%*[2]	2.24%	2.69%	*3.43%*
L = 9 m	12.81%	11.96%	7.44%	8.43%*[2]	*2.62%*
Sensitivity[1] to section size*	*5.13%*	*5.09%*	*5.61%*	*4.31%*	

*[1] Sensitivity = standard deviation

*[2] Beam systems in the moment resisting perimeter of the prototype framed structures examined in this study.

Table A-5: Deviation between the ICL model and FE failure deformation and ultimate capacity

	$w_{cr,support}$	P_d
L = 4.5 m	-55.10%	-23.62%
L = 6 m	-51.08%	-26.24%
L = 9 m	-21.07%	-22.07%
Average for model	-42.42%	-23.98%

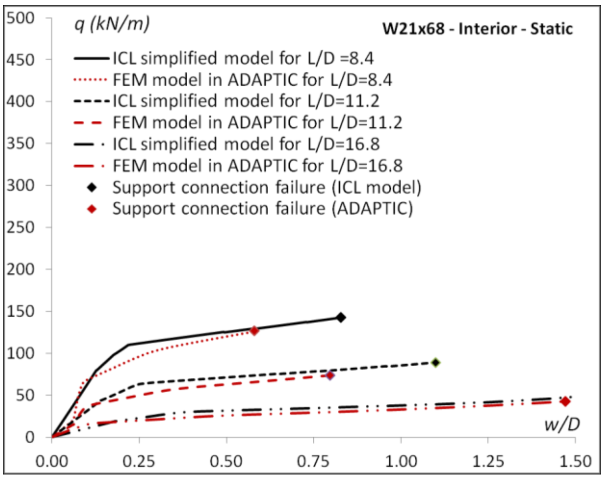

Figure A-9: Static response; W21x68 axially restrained with WUF-B beam-to-column connections

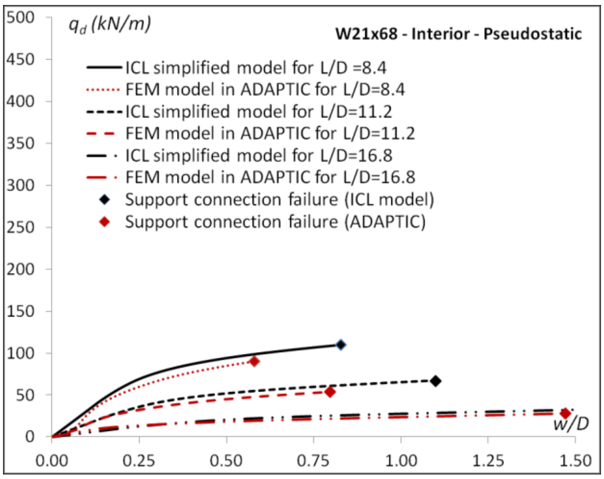

Figure A-10: Pseudostatic response; W21x68 axially restrained with WUF-B beam-to-column connections

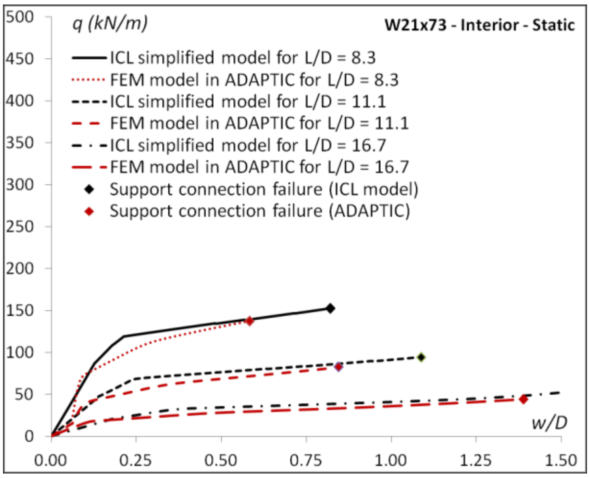

Figure A-11: Static response; W21x73 axially restrained with WUF-B beam-to-column connections

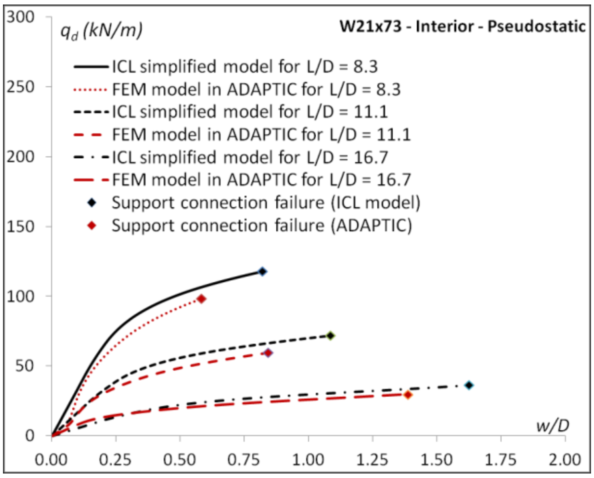

Figure A-12: Pseudostatic response; W21x73 axially restrained with WUF-B beam-to-column connections

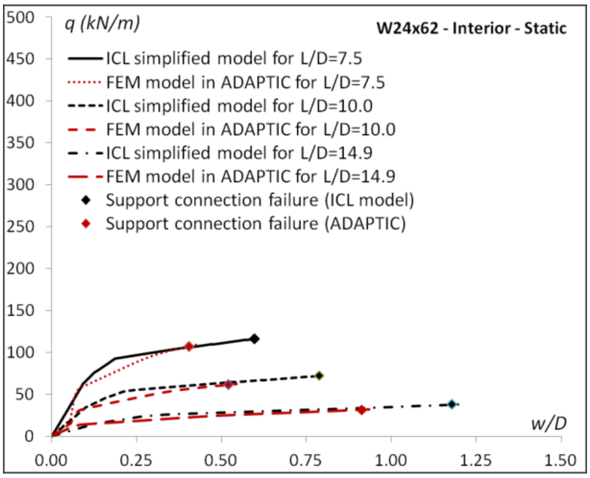

Figure A-13: Static response; W24x62 axially restrained with WUF-B beam-to-column connections

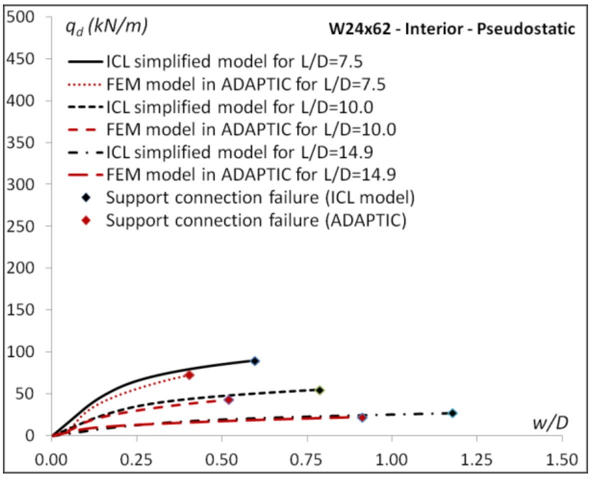

Figure A-14: Pseudostatic response; W24x62 axially restrained with WUF-B beam-to-column connections

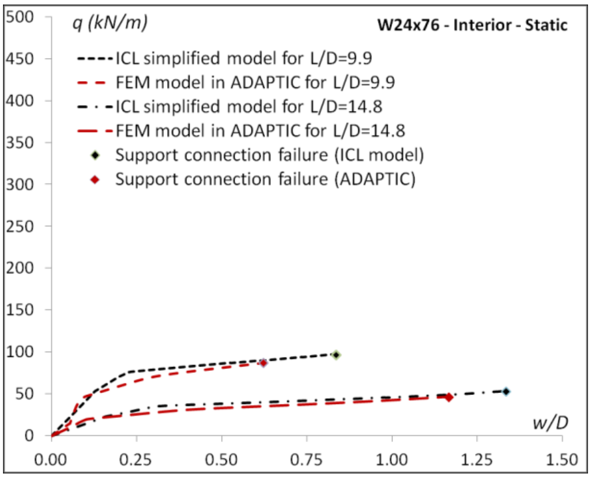

Figure A-15: Static response; W24x76 axially restrained with WUF-B beam-to-column connections

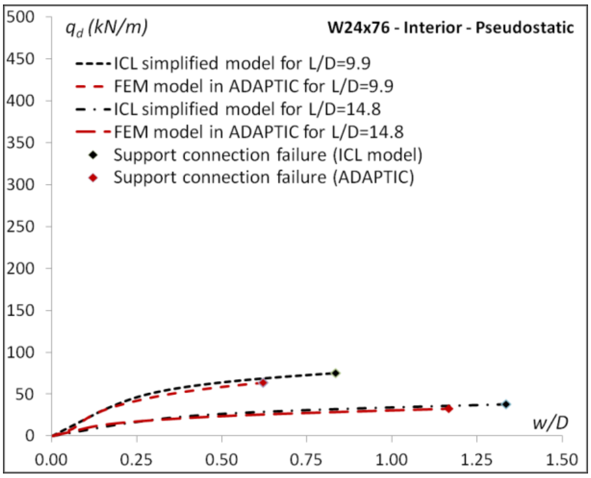

Figure A-16: Pseudostatic response; W24x76 axially restrained with WUF-B beam-to-column connections

Validation with ADAPTIC - results for the WCF-B connection (Chapter 3):

Table A-6: Deviation between the ICL model and FE pseudostatic responses

D_{Pd} = 7.78%		Section size →				Sensitivity*[1] to beam length
Length ↓		W21x68	W21x73	W24x62	W24x76	
	L = 4.5 m	7.31%	7.21%	7.03%	3.97%	1.61%
	L = 6 m	10.16%	5.17%	6.79%	8.59%	2.16%
	L = 9 m	8.68%	8.39%	4.61%	9.17%	2.09%
	Sensitivity[1] to section size*	1.42%	1.63%	1.33%	2.85%	

Table A-7: Deviation between the ICL model and FE static responses

D_P = 5.02%		Section size →				Sensitivity*[1] to beam length
Length ↓		W21x68	W21x73	W24x62	W24x76	
	L = 4.5 m	3.95%	2.76%	-5.65%		4.28%
	L = 6 m	7.50%	3.56%	1.90%	3.10%	2.42%
	L = 9 m	13.45%	11.96%	4.84%	11.38%	3.81%
	Sensitivity[1] to section size*	4.80%	5.10%	5.41%	5.89%	

*[1] Sensitivity = standard deviation

*[2] Beam systems in the moment resisting perimeter of the prototype framed structures examined in this study.

Table A-8: Deviation between the ICL model and FE failure deformation and ultimate capacity

	$w_{cr,support}$	P_d
L = 4.5 m	-44.08%	-14.17%
L = 6 m	-25.54%	-13.12%
L = 9 m	-14.75%	-17.66%
Average for model	-28.12%	-14.98%

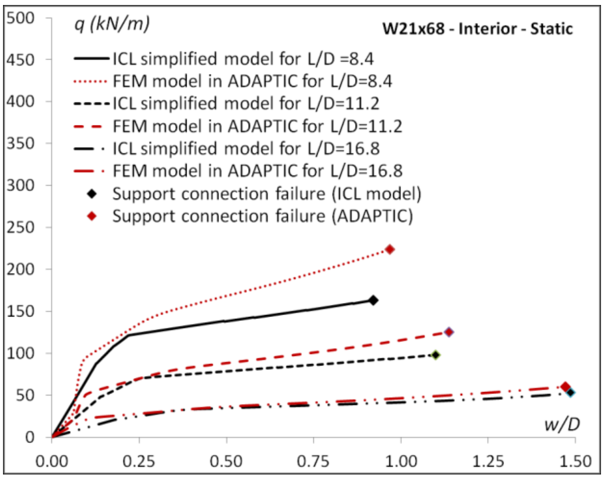

Figure A-17: Static response; W21x68 axially restrained with WCF-B beam-to-column connections

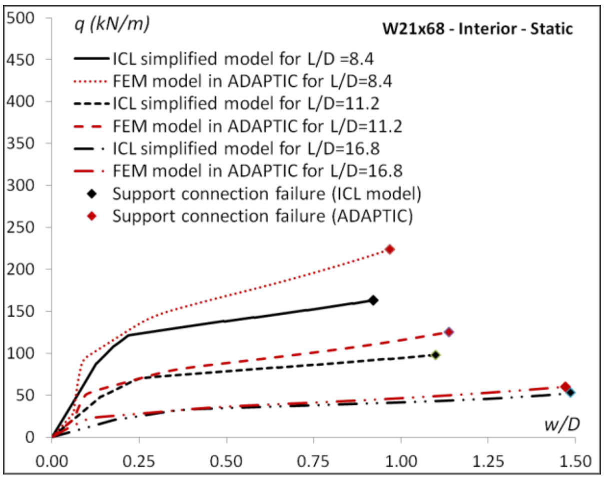

Figure A-18: Pseudostatic response; W21x68 axially restrained with WCF-B beam-to-column connections

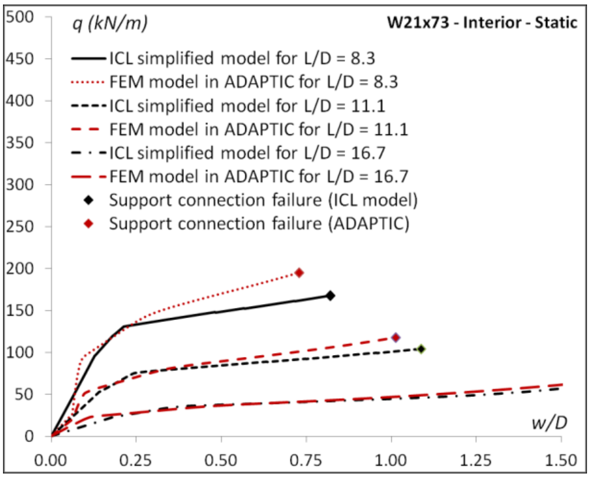

Figure A-19: Static response; W21x73 axially restrained with WCF-B beam-to-column connections

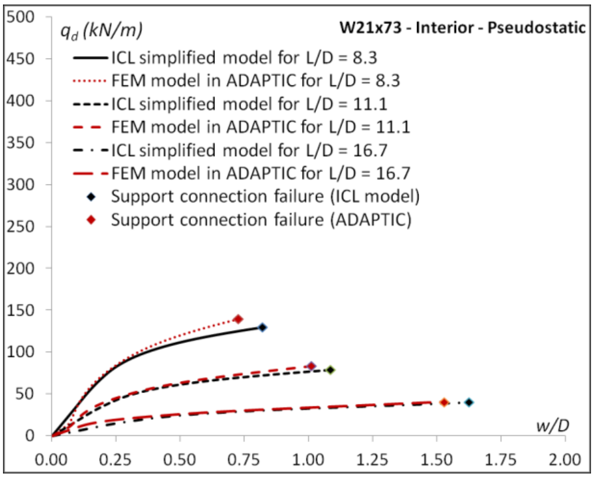

Figure A-20: Pseudostatic response; W21x73 axially restrained with WCF-B beam-to-column connections

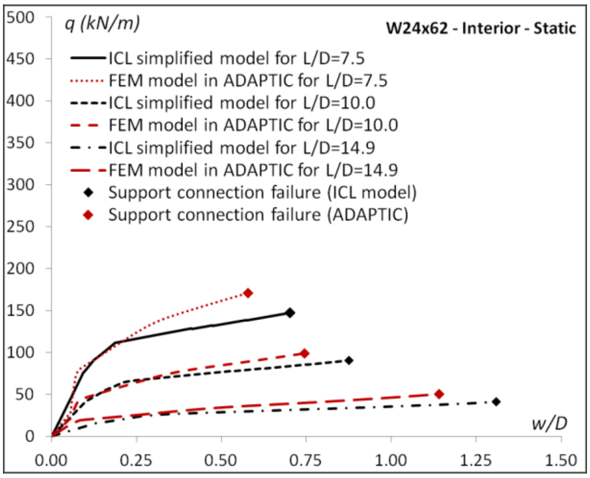

Figure A-21: Static response; W24x682 axially restrained with WCF-B beam-to-column connections

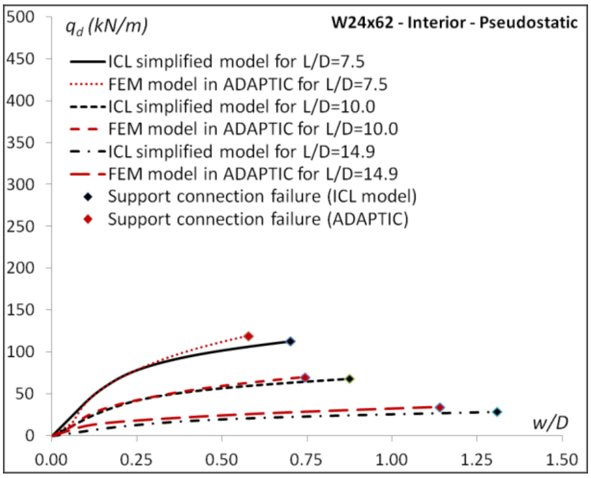

Figure A-22: Pseudostatic response; W24x62 axially restrained with WCF-B beam-to-column connections

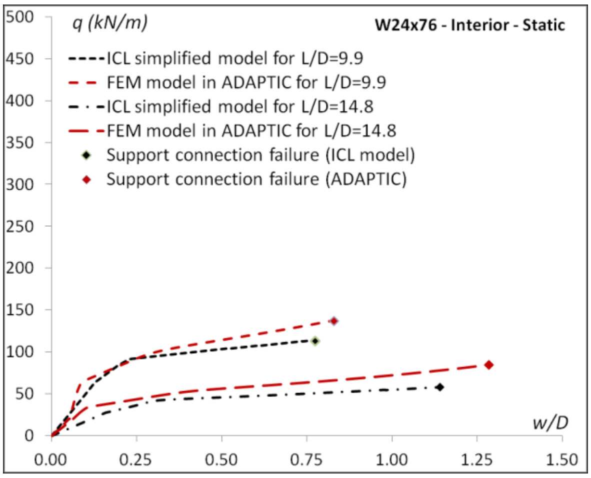

Figure A-23: Pseudostatic response; W24x76 axially restrained with WCF-B beam-to-column connections

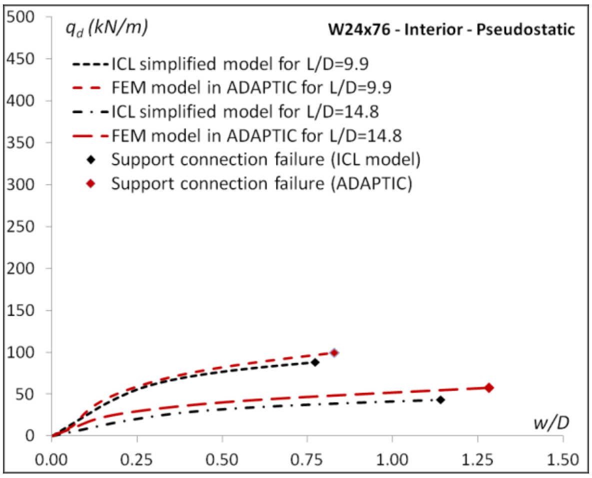

Figure A-24: Static response; W24x76 axially restrained with WCF-B beam-to-column connections

Appendix B

Cardington composite beam system parametric analysis test results

This appendix presents the results of the parametric tests for the composite and bare steel equivalent Cardington frame case study of Chapter 5. A summary of the data presented is provided below:

Table B-0: Summary of test results based on the beam system type

Table	Figure	Beam system type	Direction	Parameter under investigation
B-1	B-1	Cantilever (single span)	Transverse (primary)	Composite beam reinforcement ratio %
B-2	B-2	Cantilever (single span)	Transverse (primary)	Endplate thickness %
B-3	B-3	Cantilever (single span)	Transverse (primary)	Compressive beam flange reinforcement
B-4	B-4	Axially unrestrained (double span)	Transverse (primary)	Composite beam reinforcement ratio %
B-5	B-5	Axially unrestrained (double span)	Transverse (primary)	Endplate thickness %
B-6	B-6	Axially unrestrained (double span)	Transverse (primary)	Compressive beam flange reinforcement
B-7	B-7	Axially restrained (double span)	Transverse (primary)	Composite beam reinforcement ratio %
B-8	B-8	Axially restrained (double span)	Transverse (primary)	Endplate thickness %
B-9	B-9	Axially restrained (double span)	Transverse (primary)	Compressive beam flange reinforcement
B-10	B-10	Cantilever (single span)	Longitudinal (secondary)	Composite beam reinforcement ratio %
B-11	B-11	Cantilever (single span)	Longitudinal (secondary)	Endplate thickness %
B-12	B-12	Cantilever (single span)	Longitudinal (secondary)	Compressive beam flange reinforcement
B-13	B-13	Axially unrestrained (double span)	Longitudinal (secondary)	Composite beam reinforcement ratio %
B-14	B-14	Axially unrestrained (double span)	Longitudinal (secondary)	Endplate thickness %
B-15	B-15	Axially unrestrained (double span)	Longitudinal (secondary)	Compressive beam flange reinforcement
B-16	B-16	Axially restrained (double span)	Longitudinal (secondary)	Composite beam reinforcement ratio %
B-17	B-17	Axially restrained (double span)	Longitudinal (secondary)	Endplate thickness %
B-18	B-18	Axially restrained (double span)	Longitudinal (secondary)	Compressive beam flange reinforcement
B-19		Axially unrestrained (double span)	Transverse (primary)	Double variable: $\rho\%$ & t_p
B-20		Axially restrained (double span)	Transverse (primary)	Double variable: $\rho\%$ & t_p
B-21		Axially unrestrained (double span)	Longitudinal (secondary)	Double variable: $\rho\%$ & t_p
B-22		Axially restrained (double span)	Longitudinal (secondary)	Double variable: $\rho\%$ & t_p

Table B-1: Parametric test results on transverse cantilever beam systems: reinforcement ratio

ρ %	Rebar	Tensile components				Compressive components		$P_{d,max}$ (kN)	Main resistance action
		Yielding		Failure		Yielding / failure			
		Component	w_i	Component	w_i	Component	w_i		
0%	-	Top bolt row	13	Endplate	727			28.0	Elastic – plastic bending
0.45%	2 Φ16	Top bolt row	18	Rebar	139			40.0	
0.89%	4 Φ16	Top bolt row	24	Rebar	237			67.0	
1.13%	4 Φ18	Top bolt row	29	Rebar	280			80.0	
1.34%	6 Φ16	Top bolt row	33			Bolt flange	246	111.0	
1.79%	8 Φ16	Top bolt row	40			Bolt flange	86	1150	
3.57%	16Φ16					Bolt flange	74	125.0	

Table B-2: Parametric test results on transverse cantilever beam systems: endplate thickness

t_p (mm)	Tensile components				Compressive components		Max. P_d (kN)	Main resistance action
	Yielding		Failure		Yielding / failure			
	Component	w_i	Component	w_i	Component	w_i		
8	br1	26	Rebar	231			60.0	Elastic – plastic bending
10	br1	24	Rebar	237			66.0	
12	br1	44	Rebar	224			75.0	
14	br1	46			Bolt flange	196	99.0	
16	br1	47			Bolt flange	101	97.0	
18	br1	50			Bolt flange	81	97.0	
20	br1	51			Bolt flange	79	97.0	

Table B-3: Parametric test results on transverse cantilever beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %	t_p (mm)	Resistance of compressive components		Compressive components		Tensile components		$P_{d,max}$ (kN)
		R_d (kN)	%	Yielding		Failure		
				Comp.	w_i	Comp.	w_i	
0.89%	10	695	60	flange	96			74.0
0.89%	10	722	70	flange	230			82.0
0.89%	10	741	80			Rebar	237	66.0
0.89%	10	926	100			Rebar	237	66.0
1.34%	10	926	100	flange	86			
1.34%	10	1019	110	flange	126			126.0
1.34%	10	1111	120	flange	240			137.0
1.34%	10	1280	138			Rebar	284	112.0
1.34%	10	1852	200			Rebar	284	112.0
1.79%	10	926	100	flange	246			111.0
1.79%	10	1019	110			Rebar	284	90.0
1.79%	10	1280	138			Rebar	284	90.0
1.79%	10	1852	200			Rebar	284	90.0
0.89%	14	926	100	flange	196			99.0
0.89%	14	1280	138			Rebar	251	82.0
0.89%	14	1852	200			Rebar	251	82.0
0.89%	16	926	100	flange	101			97.0
0.89%	16	1019	110	flange	212			106.0
0.89%	16	1111	120			Rebar	256	87.0
0.89%	16	1280	138			Rebar	256	87.0
0.89%	20	926	100	flange	79			97.0
0.89%	20	1280	138			Rebar	213	900
0.89%	20	1852	200			Rebar	213	90.0

Table B-4: Parametric test results on transverse axially unrestrained beam systems: reinforcement ratio

ρ %	Rebar	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)
		Support		Centre		Support connection		
		Comp.	w_i	Comp.	w_i	Component	w_i	
0.45%	2 Φ16	Rebar	121					136.3
0.89%	4 Φ16	Rebar	218			Beam flange	565	197.4
1.13%	4 Φ18	Rebar	260			Beam flange	366	228.3
1.34%	6 Φ16			Endplate	474	Beam flange	225	276.8
1.79%	8 Φ16			Endplate	473	Beam flange	73	290.3
3.57%	16 Φ16			Endplate	472	Beam flange	62	309.8

Table B-5: Parametric test results on transverse axially unrestrained beam systems: endplate thickness

t_p (mm)	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)
	Support		Centre		Support connection		
	Comp.	w_i	Comp.	w_i	Component	w_i	
8	Rebar	218			Beam flange	907	162.0
10	Rebar	218			Beam flange	565	198.0
12	Rebar	217			Beam flange	321	238.0
14			Lower bolt row	293	Beam flange	163	288.0
16			Lower bolt row	231	Beam flange	69	293.0
18			Lower bolt row	185	Beam flange	48	296.0
20			Lower bolt row	138	Beam flange	47	283.0

Table B-6: Parametric test results on transverse axially unrestrained beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %		Resistance of compressive components		Support connection failure		Centre connection failure		$P_{d,max}$ (kN)
	t_p (mm)	R_d (kN)	%	Comp.	w_i	Comp.	w_i	
0.89%	10	695	60	Rebar	218			199.0
0.89%	10	722	70	Rebar	218			224.0
0.89%	10	741	80			Lower bolt row	480	198.0
0.89%	10	926	100			Lower bolt row	237	198.0
1.34%	10	926	100			Lower bolt row	237	290.0
1.34%	10	1280	138	Rebar	218			296.0
1.34%	10	1852	200	Rebar	218			296.0
1.79%	10	926	100			Lower bolt row	473	277.0
1.79%	10	1280	138	Rebar	286			249.0
1.79%	10	1852	200	Rebar	286			249.0
3.57%	10	926	100			Lower bolt row	472	310.0
3.57%	10	1280	138			Lower bolt row	466	386.0
3.57%	10	1852	200			Lower bolt row	460	422.0
3.57%	10	2778	300			Lower bolt row	460	422.0

Table B-7: Parametric test results on transverse axially restrained beam systems: reinforcement ratio

ρ %	Rebar	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)	Main resistance action
		Support		Centre		Support connection			
		Comp.	w_i	Comp.	w_i	Component	w_i		
0%	-	Rebar	692					195.0	Transient catenary
0.45%	2 Φ16	Rebar	381					229.0	Compressive arching
0.89%	4 Φ16	Rebar	473			Beam flange	78	251.0	Compressive arching
1.13%	4 Φ18			Lower bolt row	476	Beam flange	61	266.0	Compressive arching
1.34%	6 Φ16			Lower bolt row	474	Beam flange	60	281.0	Transient catenary
1.79%	8 Φ16			Lower bolt row	470	Beam flange	59	311.0	Transient catenary
3.57%	16 Φ16			Lower bolt row	404	Beam flange	52	550.0	Tensile catenary

Table B-8: Parametric test results on transverse axially restrained beam systems: reinforcement ratio

t_p (mm)	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)	Main resistance action
	Support		Centre		Support connection			
	Comp.	w_i	Comp.	w_i	Component	w_i		
8	Rebar	450			Beam flange	907	224.0	Compressive arching
10	Rebar	473			Beam flange	565	251.0	Compressive arching
12			Lower bolt row	451	Beam flange	321	283.0	Compressive arching
14			Lower bolt row	296	Beam flange	163	292.0	Compressive arching
16			Lower bolt row	237	Beam flange	69	299.0	Compressive arching
18			Lower bolt row	192	Beam flange	48	304.0	Compressive arching
20			Lower bolt row	145	Beam flange	47	294.0	Compressive arching

Table B-9: Parametric test results on transverse axially restrained beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %		Resistance of compressive components		Support connection failure		Centre connection failure		$P_{d,max}$ (kN)	Main resistance action
	t_p (mm)	R_d (kN)	%	Comp.	w_i	Comp.	w_i		
0.89%	10	695	60			Lower bolt row	480	206.0	Transient catenary
0.89%	10	741	80			Lower bolt row	479	230.0	Compressive arching
0.89%	10	926	100	Rebar	473			251.0	Compressive arching
0.89%	10	1019	110	Rebar	463			261.0	Compressive arching
0.89%	10	1280	138	Rebar	435			291.0[a]	Compressive arching
0.89%	10	1389	150	Rebar	422			302.0[a]	Compressive arching
0.89%	10	1852	200	Rebar	357			335.0[a]	Compressive arching
0.89%	10	2778	300	Rebar	375			335.0[a]	Compressive arching
1.34%	10	741	80			Lower bolt row	480	254.0	Transient catenary
1.34%	10	926	100			Lower bolt row	474	277.0	Transient catenary
1.34%	10	1019	110			Lower bolt row	474	288.0	Compressive arching
1.34%	10	1280	138			Lower bolt row	473	336.0	Compressive arching
1.34%	10	1852	200	Rebar	471			363.0[a]	Compressive arching
1.79%	10	926	100			Lower bolt row	470	300.0	Transient catenary
1.79%	10	1280	138			Lower bolt row	470	342.0	Transient catenary
1.79%	10	1852	200			Lower bolt row	468	382.0[a]	Compressive arching
3.57%	10	926	100			Lower bolt row	404	337.0	Transient catenary
3.57%	10	1280	138			Lower bolt row	450	409.0	Transient catenary
3.57%	10	1852	200			Lower bolt row	459	436.0	Transient catenary
3.57%	10	2778	300			Lower bolt row	459	436.0	Transient catenary

*[a] Peak response achieved during the compressive arching phase

Table B-10: Parametric test results on longitudinal cantilever beam systems: reinforcement ratio

ρ %	Rebar	Tensile components				Compressive components		$P_{d,max}$ (kN)
		Yielding		Failure		Yielding / failure		
		Component	w_i	Component	w_i	Component	w_i	
0%	-			Endplate	1011			20.0
0.45%	2 Φ16			Rebar	218			27.0
0.89%	4 Φ16			Rebar	307			42.0
1.34%	6 Φ16					Beam flange	291	71.0
1.79%	8 Φ16					Beam flange	143	74.0
3.57%	16 Φ16					Beam flange	122	81.0

Table B-11: Parametric test results on longitudinal cantilever beam systems: endplate thickness

t_p (mm)	Tensile components		Compressive components		Max. P_d (kN)
	Failure		Yielding / failure		
	Component	w_i	Component	w_i	
8	Rebar	299			37.0
10	Rebar	307			42.0
12	Rebar	318			48.0
14			Bolt flange	143	59.0
16			Bolt flange	121	59.0
18			Bolt flange	114	58.0
20			Bolt flange	110	58.0

Table B-12: Parametric test results on longitudinal cantilever beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %		Resistance of compressive components		Compressive components		Tensile components		$P_{d,max}$ (kN)
	t_p (mm)	R_d (kN)	%	Yielding		Failure		
				Comp.	w_i	Comp.	w_i	
0.89%	10	741	80	flange	298			41.6
0.89%	10	834	90			Rebar	307	41.9
0.89%	10	926	100			Rebar	307	41.9
0.89%	10	1280	138			Rebar	307	41.9
0.89%	10	1852	200			Rebar	307	41.9
1.79%	10	741	80	flange	113			32.3
1.79%	10	926	100	flange	143			39.6
1.79%	10	1280	138			Rebar	460	72.9
1.79%	10	1852	200			Rebar	460	72.9
1.79%	10	2778	300			Rebar	460	72.9

Table B-13: Parametric test results on longitudinal axially unrestrained beam systems: reinforcement ratio

ρ %	Rebar	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)
		Support		Centre		Support connection		
		Comp.	w_i	Comp.	w_i	Component	w_i	
-	-			Endplate	758			92.5
0.45%	2 Φ16	Rebar	176					92.5
0.89%	4 Φ16	Rebar	264					128.4
1.34%	6 Φ16			Endplate	722			187.1
1.79%	8 Φ16			Endplate	722			194.6
3.57%	16 Φ16			Endplate	717	Beam flange	62	208.8

Table B-14: Parametric test results on longitudinal axially unrestrained beam systems: endplate thickness

t_p (mm)	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)
	Support		Centre		Support connection		
	Comp.	w_i	Comp.	w_i	Component	w_i	
8	Rebar	271			Beam flange	1229	103.0
10	Rebar	264			Beam flange	664	128.4
12	Rebar	256			Beam flange	291	155.6
14			Lower bolt row	313	Beam flange	83	181.7
16			Lower bolt row	249	Beam flange	59	181.5
18			Lower bolt row	206	Beam flange	55	181.1
20			Lower bolt row	166	Beam flange	53	173.2

Table B-15: Parametric test results on longitudinal axially unrestrained beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %		Resistance of compressive components		Support connection failure		Centre connection failure		$P_{d,max}$ (kN)
	t_p mm)	R_d (kN)	%	Comp.	w_i	Comp.	w_i	
0.89%	10	741	80			Top bolt row	732	156.4
0.89%	10	926	100	Rebar	264			128.4
0.89%	10	1280	138	Rebar	264			128.4
0.89%	10	1852	200	Rebar	264			128.4
1.34%	10	741	80			Lower bolt row	728	170.2
1.34%	10	926	100			Lower bolt row	720	194.6
1.34%	10	1280	138	Rebar	414			198.3
1.34%	10	1852	200	Rebar	414			198.3

Table B-16: Parametric test results on longitudinal axially restrained beam systems: reinforcement ratio

ρ %	Rebar	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)	Main resistance action
		Support		Centre		Support connection			
		Comp.	w_i	Comp.	w_i	Component	w_i		
0%	-	Rebar	917			Beam flange	183	110.3	Compressive arching
0.45%	2 Φ16	Rebar	391			Beam flange	143	139.7	Compressive arching
0.89%	4 Φ16	Rebar	500			Beam flange	99	158.0	Compressive arching
1.34%	6 Φ16	Rebar	592			Beam flange	95	181.5	Transient catenary
1.79%	8 Φ16	Rebar	640			Beam flange	92	206.0	Tensile catenary
3.57%	16 Φ16			Lower bolt row	572	Beam flange	83	241.9	Tensile catenary

Table B-17: Parametric test results on longitudinal axially restrained beam systems: reinforcement ratio

t_p (mm)	Tensile components - failure				Compressive components - yielding		$P_{d,max}$ (kN)	Main resistance action
	Support		Centre		Support connection			
	Comp.	w_i	Comp.	w_i	Component	w_i		
8	Rebar	465			Beam flange	134	136.6	Compressive arching
10	Rebar	500			Beam flange	99	158.2	Compressive arching
12			Lower bolt row	496	Beam flange	77	181.0	Compressive arching
14			Lower bolt row	316	Beam flange	60	183.0	Compressive arching
16			Lower bolt row	253	Beam flange	55	184.7	Compressive arching
18			Lower bolt row	211	Beam flange	53	185.6	Compressive arching
20			Lower bolt row	172	Beam flange	51	179.0	Compressive arching

Table B-18: Parametric test results on longitudinal axially restrained beam systems: reinforcement of compressive beam flange for different connection configurations

ρ %		Resistance of compressive components		Support connection failure		Centre connection failure		$P_{d,max}$ (kN)	Main resistance action
	t_p (mm)	R_d (kN)	%	Comp.	w_i	Comp.	w_i		
0.89%	10	741	80	Rebar	535			180.0 [a]	Compressive arching
0.89%	10	1019	110	Rebar	500			174.6 [a]	Compressive arching
0.89%	10	1280	138	Rebar	420			158.2	Transient catenary
0.89%	10	1852	200	Rebar	344			147.5	Transient catenary
1.79%	10	741	80	Rebar	662			198.7	Tensile catenary
1.79%	10	926	100	Rebar	640			206.4	Tensile catenary
1.79%	10	1280	138	Rebar	595			219.4	Tensile catenary
1.79%	10	1852	200	Rebar	510			231.6	Transient catenary

*[a] Peak response achieved during the compressive arching phase

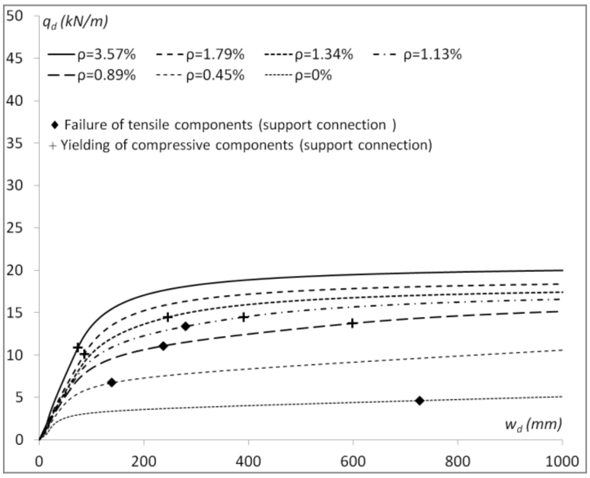

Figure B-1: q-w response of transverse cantilever beam systems for varying reinforcement ratios

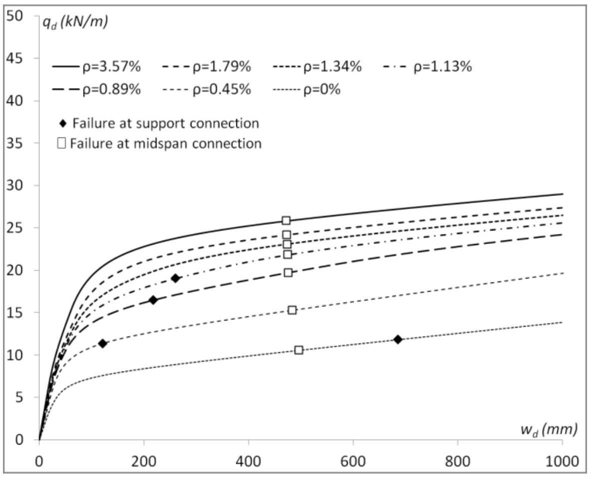

Figure B-4: q-w response of transverse unrestrained beam systems for varying reinforcement ratios

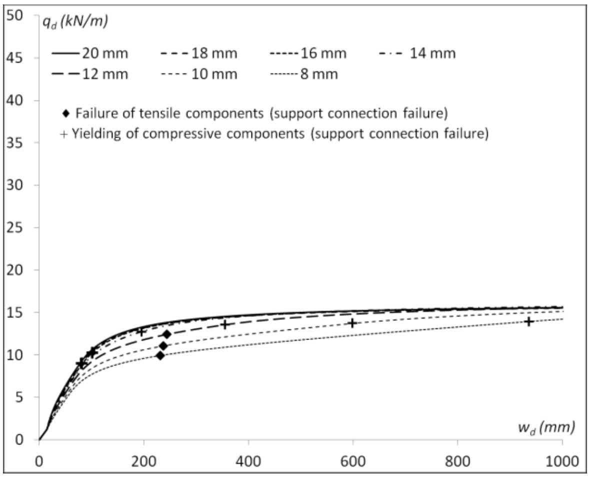

Figure B-2: q-w response of transverse cantilever system for varying connection endplate thickness t_p

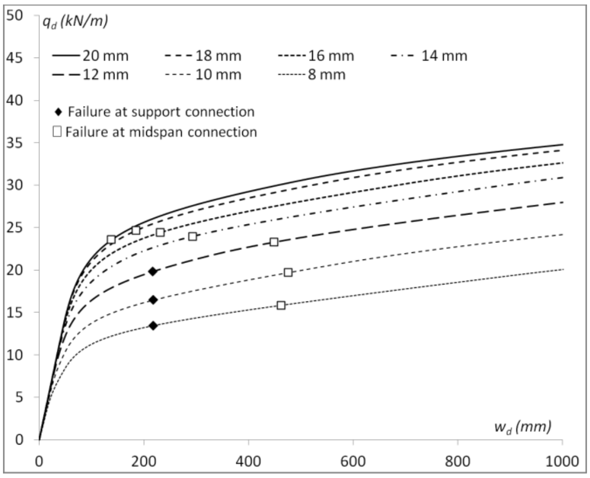

Figure B-5: q-w response of transverse unrestrained system for varying connection endplate thickness t_p

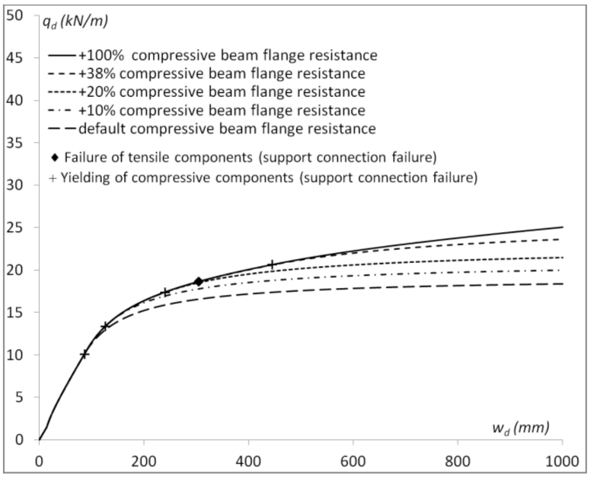

Figure B-3: q-w response of transverse cantilever systems for increased connection compressive components' resistance (ρ=1.79%, -10% to +100% variation in comp. resistance)

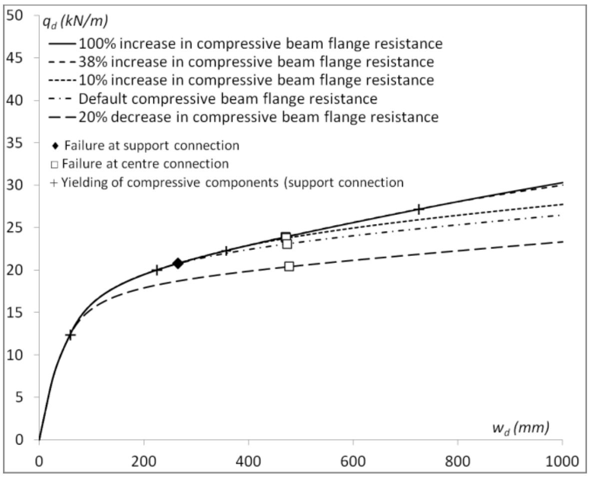

Figure B-6: q-w response of unrestrained beam systems for increased connection compressive components' resistance (ρ=1.34%, -10% to +100% variation in comp. resistance)

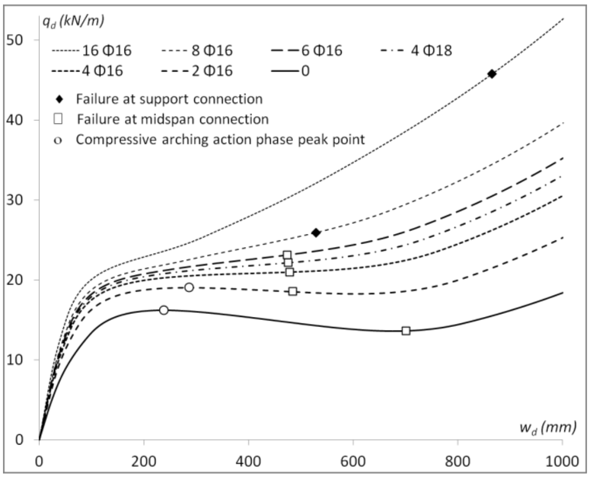

Figure B-7: q-w response of transverse unrestrained beam systems for varying reinforcement ratios

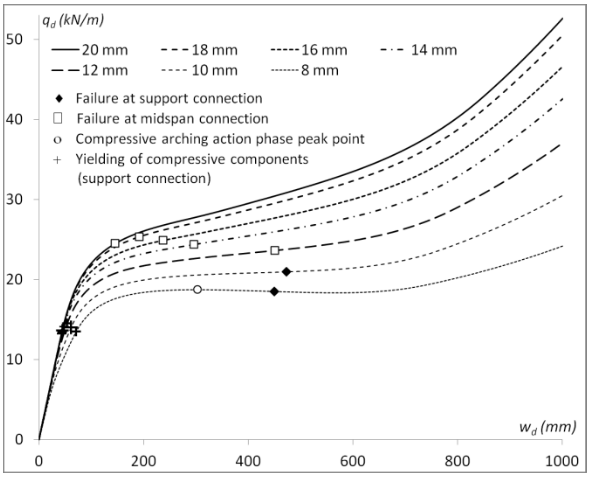

Figure B-8: q-w response of transverse unrestrained system for varying connection endplate thickness t_p

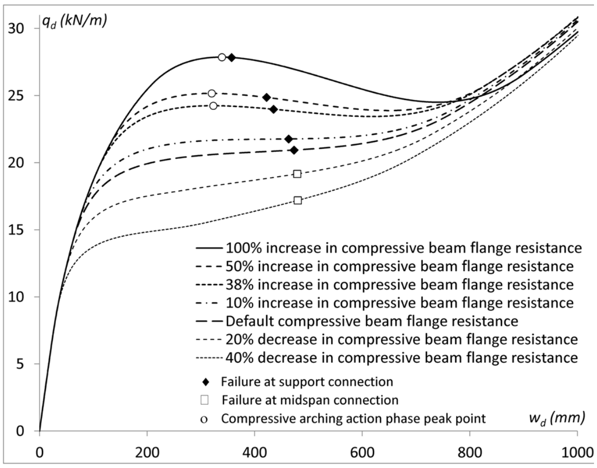

Figure B-9: q-w response of restrained beam systems for increased connection compressive components' resistance (ρ=1.79%, -40% to +100% variation in comp. resistance)

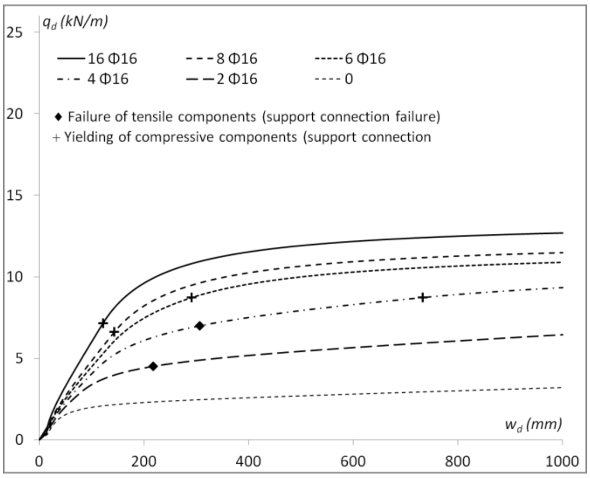

Figure B-10: q-w response of longitudinal cantilever beam systems for varying reinforcement ratios

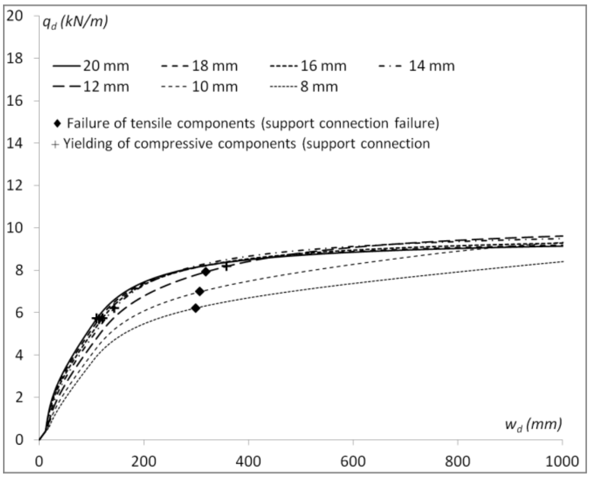

Figure B-11: q-w response of longitudinal cantilever system for varying connection endplate thickness t_p

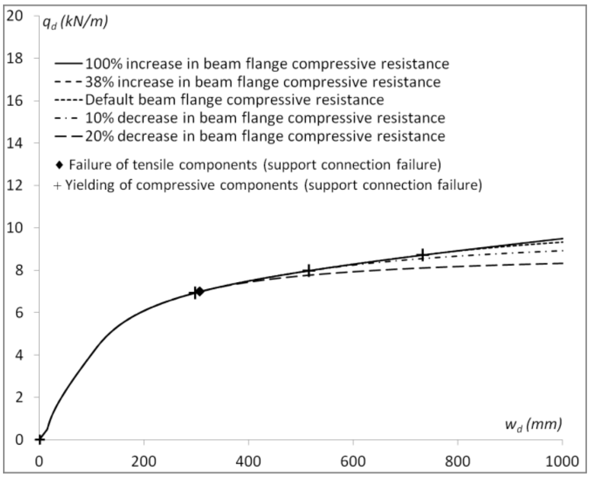

Figure B-12: q-w response of longitudinal cantilever systems for increased connection compressive components' resistance (ρ=1.79%, -20% to +100% variation in comp. resistance)

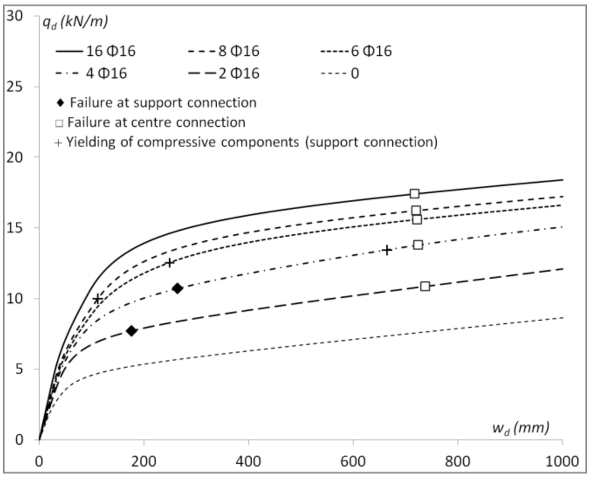

Figure B-13: q-w response of longitudinal unrestrained beam systems for varying reinforcement ratios

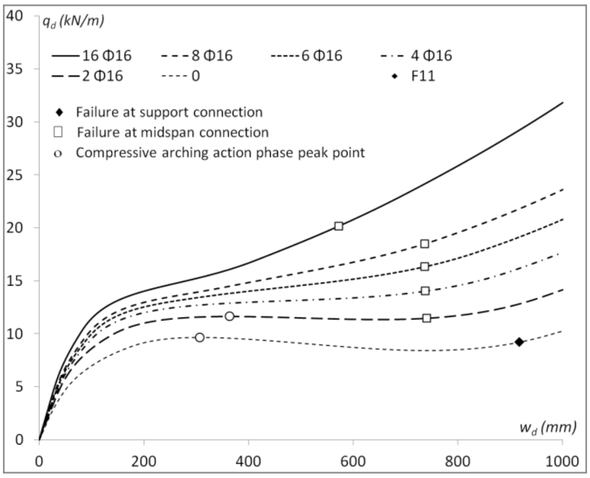

Figure B-16: q-w response of longitudinal restrained beam systems for varying reinforcement ratios

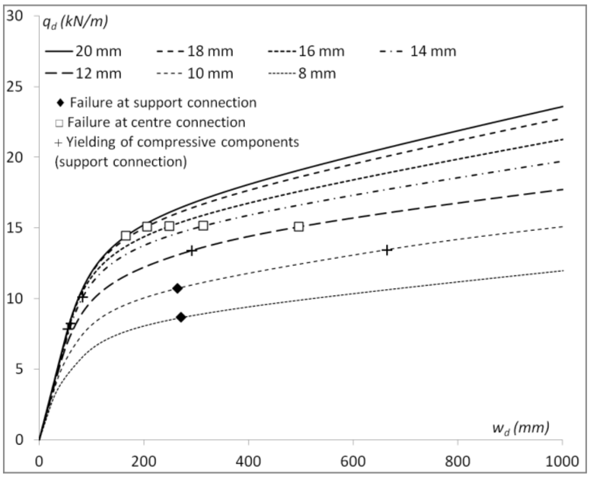

Figure B-14: q-w response of longitudinal unrestrained system for varying connection endplate thickness t_p

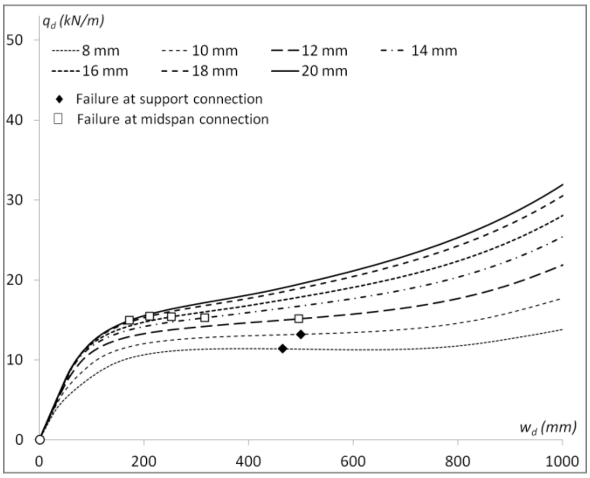

Figure B-17: q-w response of longitudinal restrained system for varying connection endplate thickness t_p

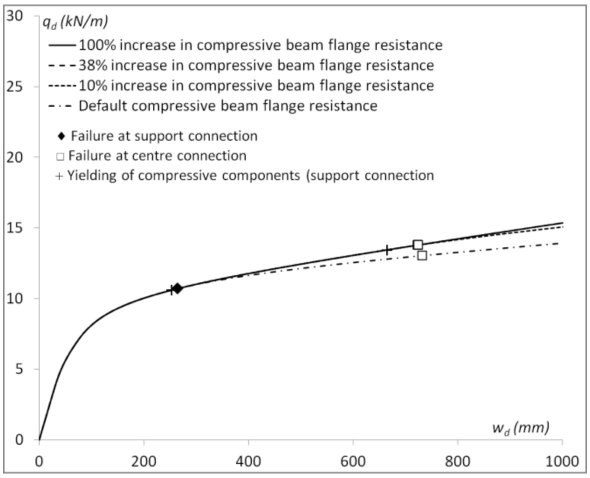

Figure B-15: q-w response of longitudinal unrestrained systems for increased connection compressive components' resistance (ρ=1.79%, -10% to +100% variation in comp. resistance)

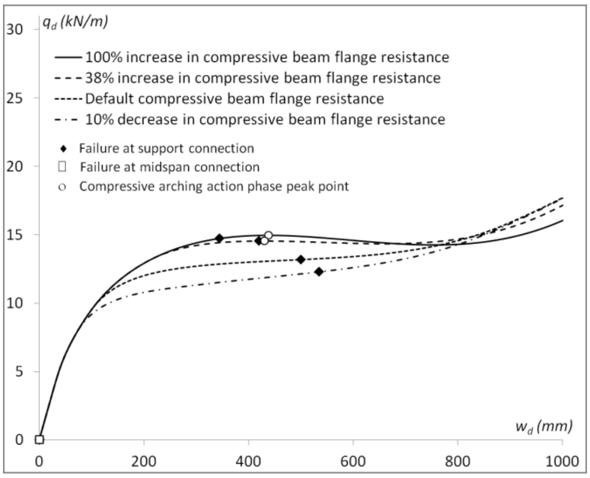

Figure B-18: q-w response of longitudinal restrained systems for increased connection compressive components' resistance (ρ=1.34%, -10% to +100% variation in comp. resistance)

Table B-19: Double parametric tests for the transverse (primary) axially unrestrained beam systems of the simplified Cardington composite frame

		Reinforcement					
		0	2Φ16	4Φ16	6Φ16	8Φ16	16Φ16
t_p (mm)	8			Support [a] Rebar [b] 218 mm [c] 162 kN [d]	Centre [a] Endplate [b] 266 mm [c] 212 kN [d]		
	10	mid-plate Endplate [b] 496 mm [c] 127 kN [d]	Support [a] Rebar [b] 121 mm [c] 136 kN [d]	Support [a] Rebar [b] 218 mm [c] 198 kN [d]	Centre [a] Endplate [b] 474 mm [c] 277 kN [d]	**Centre [a]** **Endplate [b]** **473** mm [c] **290** kN [d]	Centre [a] Endplate [b] 472 mm [c] 310 kN [d]
	12		Support [a] Rebar [b] 120 mm [c] 170 kN [d]	Support [a] Rebar [b] 217 mm [c] 238 kN [d]	**Centre [a]** **Bottom bolt row [b]** **291** mm [c] **304** kN [d]	**Centre [a]** **Bottom bolt row [b]** **446** mm [c] **314** kN [d]	Centre [a] Bottom bolt row [b] 444 mm [c] 335 kN [d]
	14	Centre [a] Bottom bolt row [b] 312 mm [c] 193 kN [d]	Support [a] Rebar [b] 120 mm [c] 199 kN [d]	Centre [a] Bottom bolt row [b] 293 mm [c] 288 kN [d]	Centre [a] Bottom bolt row [b] 291 mm [c] 304 kN [d]	Centre [a] Bottom bolt row [b] 290 mm [c] 314 kN [d]	Centre [a] Bottom bolt row [b] 287 mm [c] 335 kN [d]
	16	Centre [a] Bottom bolt row [b] 250 mm [c] 210 kN [d]	Support [a] Rebar [b] 120 mm [c] 219 kN [d]	Centre [a] Bottom bolt row [b] 231 mm [c] 293 kN [d]			
	18	Support [a] Top bolt row [b] 198 mm [c] 223 kN [d]	Support [a] Rebar [b] 120 mm [c] 240 kN [d]	**Centre [a]** **Bottom bolt row [b]** **185** mm [c] **296** kN [d]			
	20	sup-br1 Top bolt row [b] 127 mm [c] 206 kN [d]	Support [a] Rebar [b] 120 mm [c] 250 kN [d]	Centre [a] Bottom bolt row [b] 138 mm [c] 283 kN [d]			

[a] Critical connection position
[b] Critical connection component
[c] $w_{d,max}$ (maximum ductility)
[d] $P_{d,max}$ (maximum pseudostatic capacity)

* The bolded cells correspond to the candidate configurations satisfying the redesigning criteria (section 5.8.2.2)

Table B-20: Double parametric tests for the transverse (primary) axially restrained beam systems of the simplified Cardington composite frame

		Reinforcement					
		0	2Φ16	4Φ16	6Φ16	8Φ16	16Φ16
	8						
	10			Support[a] Rebar[b] 473 mm[c] 251 kN[d]	Centre[a] Endplate[b] 474 mm[c] 277 kN[d]	Centre[a] Endplate[b] 470 mm[c] 300 kN[d]	
	12		Support[a] Rebar[b] 409 mm[c] 257 kN[d]	Centre[a] Bottom bolt row[b] 451 mm[c] 284 kN[d]	**Centre[a]** **Bottom bolt row[b]** **446 mm[c]** **307 kN[d]**	**Centre[a]** **Bottom bolt row[b]** **442 mm[c]** **327 kN[d]**	
t_p (mm)	14	Support[a] Top bolt row[b] 634 mm[c] 215 kN[d]	Centre[a] Bottom bolt row[b] 303 mm[c] 277 kN[d]	**Centre[a]** **Bottom bolt row[b]** **296 mm[c]** **292 kN[d]**	**Centre[a]** **Bottom bolt row[b]** **291 mm[c]** **305 kN[d]**	Centre[a] Bottom bolt row[b] 288 mm[c] 315 kN[d]	
	16		Centre[a] Bottom bolt row[b] 241 mm[c] 285 kN[d]	Centre[a] Bottom bolt row[b] 237 mm[c] 299 kN[d]	Centre[a] Bottom bolt row[b] 233 mm[c] 309 kN[d]	Centre[a] Bottom bolt row[b] 230 mm[c] 317 kN[d]	
	18			Centre[a] Bottom bolt row[b] 192 mm[c] 304 kN[d]	Centre[a] Bottom bolt row[b] 188 mm[c] 313 kN[d]		
	20						

[a] Critical connection position
[b] Critical connection component
[c] $w_{d,max}$ (maximum ductility)
[d] $P_{d,max}$ (maximum pseudostatic capacity)

* The bolded cells correspond to the candidate configurations satisfying the redesigning criteria (section 5.8.2.2)

Table B-21: Double parametric tests for the longitudinal (secondary) axially unrestrained beam systems of the simplified Cardington composite frame

		Reinforcement					
		0	2Φ16	4Φ16	6Φ16	8Φ16	16Φ16
t_p (mm)	8						
	10			Support [a] Rebar [b] 264 mm [c] 129 kN [d]	**Centre [a]** **Endplate [b]** **722 mm [c]** **188 kN [d]**	**Centre [a]** **Endplate [b]** **720 mm [c]** **195 kN [d]**	
	12			Support [a] Rebar [b] 256 mm [c] 156 kN [d]	**Centre [a]** **Bottom bolt row [b]** **492 mm [c]** **193 kN [d]**	Centre [a] Bottom bolt row [b] 490 mm [c] 199 kN [d]	
	14			Centre [a] Bottom bolt row [b] 313 mm [c] 182 kN [d]	Centre [a] Bottom bolt row [b] 310 mm [c] 190 kN [d]	Centre [a] Bottom bolt row [b] 307 mm [c] 196 kN [d]	
	16			Centre [a] Bottom bolt row [b] 249 mm [c] 182 kN [d]			
	18						
	20						

[a] Critical connection position

[b] Critical connection component

[c] $w_{d,max}$ (maximum ductility)

[d] $P_{d,max}$ (maximum pseudostatic capacity)

* The bolded cells correspond to the candidate configurations satisfying the redesigning criteria (section 5.8.2.2)

Table B-22: Double parametric tests for the longitudinal (secondary) axially restrained beam systems of the simplified Cardington composite frame

		Reinforcement					
		0	2Φ16	4Φ16	6Φ16	8Φ16	16Φ16
t_p (mm)	8						
	10			Support [a] Rebar [b] 500 mm [c] 158 kN [d]	Support [a] Rebar [b] 592 mm [c] 182 kN [d]	Support [a] Rebar [b] **640 mm** [c] **207 kN** [d]	Centre [a] Endplate [b] 572 mm [c] 242 kN [d]
	12		Support [a] Rebar [b] 444 mm [c] 163 kN [d]	Centre [a] Bottom bolt row [b] 496 mm [c] 182 kN [d]	**Centre** [a] **Bottom bolt row** [b] **492 mm** [c] **196 kN** [d]	Centre [a] Bottom bolt row [b] 488 mm [c] 208 kN [d]	
	14		Centre [a] Bottom bolt row [b] 322 mm [c] 174 kN [d]	Centre [a] Bottom bolt row [b] 316 mm [c] 183 kN [d]	Centre [a] Bottom bolt row [b] 311 mm [c] 190 kN [d]	Centre [a] Bottom bolt row [b] 308 mm [c] 196 kN [d]	
	16			Centre [a] Bottom bolt row [b] 253 mm [c] 185 kN [d]	Centre [a] Bottom bolt row [b] 249 mm [c] 191 kN [d]	Centre [a] Bottom bolt row [b] 245 mm [c] 196 kN [d]	
	18						
	20						

[a] Critical connection position

[b] Critical connection component

[c] $w_{d,max}$ (maximum ductility)

[d] $P_{d,max}$ (maximum pseudostatic capacity)

* The bolded cells correspond to the candidate configurations satisfying the redesigning criteria (section 5.8.2.2)

Appendix C

Progressive collapse resistance of simple floors with shear tab connections at the interior of moment frames

A preliminary investigation - complementary to Chapter 6 - is reported for floor systems at the interior of the frame with shear tab connections. The data extracted from a series of experiments at the University of Alberta (S.A. Oosterhof and R.G. Driver, 2012, Oosterhof, 2013) for examining the behaviour of shear tab connections under large rotations was used as input to the Imperial College Method framework in order to approximate the pseudo-static response of gravity resisting beam systems. The results presented herein are based on the collaboration with Dr. Oosterhof during his stay as a visiting scholar at Imperial College London in 2013.

Using the ICL Method, an internal column loss scenario is considered for the NIST SDC-C moment resisting frame. Table C-1 summarises the differences in the beam system and the connection design between the arrangement used in laboratory tests (Figure C-1) and the arrangement modelled in the case study of this thesis.

Table C-1: Differences in beam system and connection design between the experimental and the NIST SDC-C interior frame arrangement

	SDC-C	Oosterhof & Driver	Anticipated effect
Beam section	W14x22	W12x96	*Insignificant: Behaviour independent of beam size for simple connections*
Column section	W18x119	W10x60	*Insignificant: No contribution from column components*
Beam length	9m (between centres of rotation)		-
Steel resistance	F_y = 344.8 MPa	F_y = 353 MPa	*Insignificant*
Plate dimensions	4/8 x 12 x 6 in $A_{SCD\text{-}C,p}$ = 465 cm^2 t_p = 12.7 mm	3/8 x 9 x 4.33 in $A_{O,p}$ = 251 cm^2 t_p = 9.5 mm	*The increased plate area is not expected to have an impact on connection ductility.* For the impact of the increased plate thickness see Table C-2.
Bolt steel grade	A490 steel	A325 steel	*Insignificant: Bolts are not critical components as they remain in the elastic phase throughout the response (Oosterhof, 2013)*

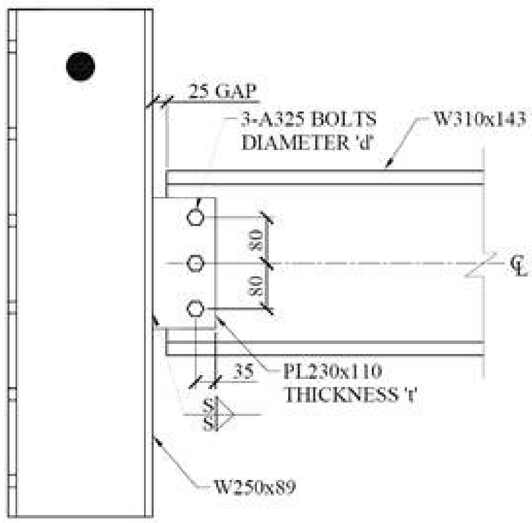

Figure C-1: Shear tab connection detail (Oosterhof, 2013)

The increase of plate thickness by 34% (from 9.5 mm to 12.7 mm) may have a different effect on the response depending on the failure criteria employed. Oosterhof identifies two approaches in calculating the ductility of a shear tab connection:

- A ductile (favourable) approach; the increase of thickness does not affect ductility, as the endplate is not a critical component.
- A conservative approach; the increased thickness of the plate increases the rotational capacity of the connection.

As the plate, which is a primary influencing component, is thicker in the case study than the experiments, a component model was constructed to estimate the increase in the ductility and the capacity of the connection. The results from the test arrangement closer to the NIST frame (Table C-1) were used with the Imperial College Simplified Method in order to model and calibrate the shear tab connection. Table C-2 shows the impact on connection ductility for the given increase in plate thickness according to both approaches.

The results in Figure C-2 show that even when using the ductile approach, the floor grillage still lacks 39% of the required capacity to resist progressive collapse.

Table C-2: Impact on connection ductility after increasing plate thickness in shear tab connections

Increase in t_p	Beam length	Approach	Connection capacity	Connection ductility
+34%	6m	Conservative	+12%	-20%
		Ductile	+43%	*No variation*
	9m	Conservative	+11%	-19%
		Ductile	+41%	*No variation*

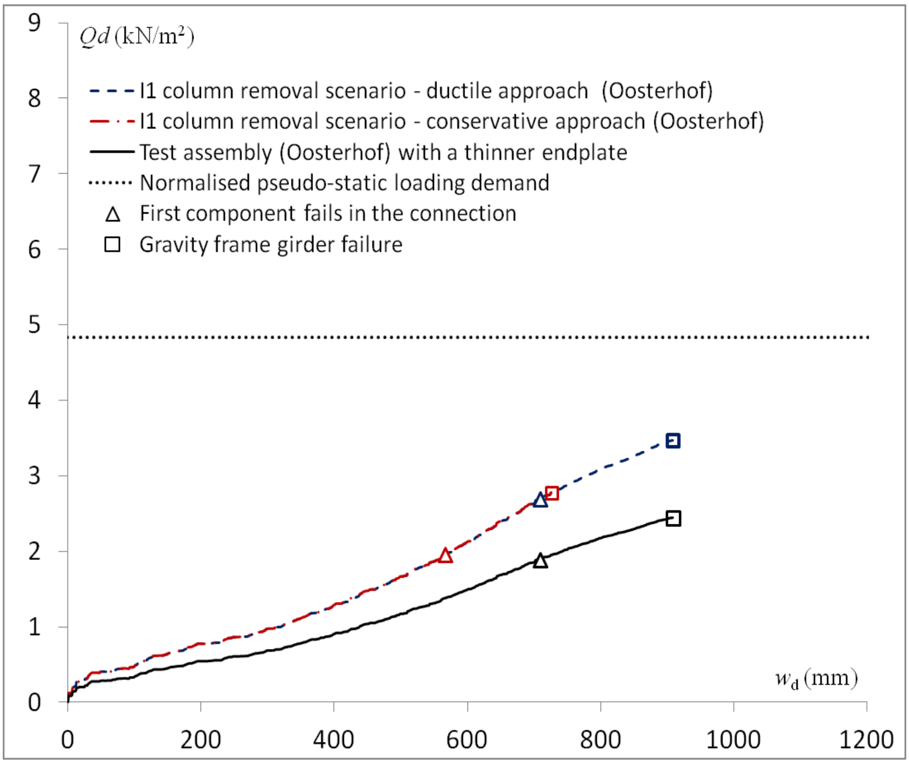

Figure C-2: Q_d / w floor pseudo-static response for column loss at the interior of the NIST IMF

Previous results by Dr. Oosterhof show that shear tab connections may display a partially rigid behaviour for very small rotations, during which they provide bending moment resistance and a peak in capacity thanks to compressive arching action. However, they quickly lose the ability to provide bending moment resistance and enter the tensile catenary phase, which dominates the response form, ductility and ultimate capacity. The connection fails due to the tear of the shear plate by the support connection upper bolt (bolt tear-out), which is loaded in bearing. After the crack initiates at this point, failure propagates at each successive bolt row very quickly. The main parameters that influence the ductility and maximum capacity of the connection are the endplate thickness, steel grade and the horizontal distance between the bolt line and the edge of the endplate.

Compared to the partially restrained connections examined in Chapter 5, the main difference lies in the very low capacity of the compressive components of a shear tab connection, which does not allow developing an effecting compressive arching or even transient catenary action. Although the very high ductility of the connection arrangement allows achieving significant capacity via catenary action, its low strength is expected to be insufficient for matching the pseudo-static capacity demand at the point of the removed column.

Appendix D

NIST beam system modelling sensitivity analysis results

Table D-1: Gain in system **ductility** for various percentile decreases / increases for double span beam systems in the **interior** of the moment frame perimeter (Chapter 6)

Beam system	Parameter	Percentile decrease or increase							
		-75%	-50%	-25%	-10%	+10%	+25%	+50%	+75%
G1[a]	Axial restraint	0	0	0	0	0	0	0	0
	$\varepsilon_{max,tension}$	-67	-44	-22	-9	8	*N/A*	*N/A*	*N/A*
	f_y & f_u	0	0	0	0	0	0	0	0
G2[b]	Axial restraint	+1	0	0	0	0	0	0	0
	$\varepsilon_{max,tension}$	-67	-42	-21	-8	9	21	42	62
	f_y & f_u	0	0	0	0	0	0	0	-16

[a] Original maximum deflection for G1: w = 1241 mm
[b] Original maximum deflection for G2: w = 822 mm

Table D-2: Gain in system **capacity** for various percentile decreases / increases for double span beam systems in the **interior** of the moment frame perimeter (Chapter 6)

Beam system	Parameter	Percentile decrease or increase							
		-75%	-50%	-25%	-10%	+10%	+25%	+50%	+75%
G1[a]	Axial restraint	-7	-4	-2	-1	1	1	2	3
	$\varepsilon_{max,tension}$	-38	-29	-19	-10	12	*N/A*	*N/A*	*N/A*
	f_y & f_u	-43	-29	-14	-6	6	14	28	42
G2[b]	Axial restraint	-1	-1	0	0	0	0	0	0
	$\varepsilon_{max,tension}$	-28	-19	-11	-5	6	19	55	113
	f_y & f_u	-50	-33	-16	-7	7	16	32	42

[a] Original maximum capacity for G1: P_d = 1102 kN
[b] Original maximum capacity for G2: P_d = 1237 kN

Table D-3: Gain in system **ductility** for various percentile decreases / increases for double span beam systems in the **edge (penultimate)** of the moment frame perimeter (Chapter 6)

Beam system	Parameter	Percentile decrease or increase							
		-75%	-50%	-25%	-10%	+10%	+25%	+50%	+75%
G3[a]	$K_{column,flex}$	*N/A*	4	1	0	0	-1	-1	*N/A*
	$\varepsilon_{max,tension}$	-71	-47	-24	-9	9	*N/A*	*N/A*	*N/A*
	f_y & f_u	-2	-1	-1	0	0	*N/A*	*N/A*	*N/A*
G4[b]	$K_{column,flex}$	14	6	2	1	-1	-1	-2	-3
	$\varepsilon_{max,tension}$	-66	-41	-19	-8	8	19	39	58
	f_y & f_u	-1	-1	0	0	0	0	1	1

[a] Original maximum deflection for G1: w = 1151 mm
[b] Original maximum deflection for G2: w = 880 mm

Table D-4: Gain in system **capacity** for various percentile decreases / increases for double span beam systems in the **edge (penultimate)** of the moment frame perimeter (Chapter 6)

Beam system	Parameter	Percentile decrease or increase							
		-75%	-50%	-25%	-10%	+10%	+25%	+50%	+75%
G3[a]	$K_{column,flex}$	*N/A*	-1	-1	0	0	1	1	*N/A*
	$\varepsilon_{max,tension}$	-27	-17	-9	-4	4	*N/A*	*N/A*	*N/A*
	f_y & f_u	-48	-32	-16	-6	6	*N/A*	*N/A*	*N/A*
G4[b]	$K_{column,flex}$	2	1	0	0	0	0	0	0
	$\varepsilon_{max,tension}$	-27	-18	-9	-4	4	12	26	46
	f_y & f_u	-50	-33	-17	-7	7	16	33	49

[a] Original maximum capacity for G1: P_d = 896 kN
[b] Original maximum capacity for G2: P_d = 1242 kN

Appendix E

Connection component characteristics

This appendix aims to provide specific details on the numerical description for a subset of relevant beam-to-column connections of the framed buildings analysed in Chapters 5 and 6. These include resistance and stiffness values for different nonlinear spring elements representing the main components of selected joints.

For the endplate bolted connections, the values for the default bare steel configuration (Figure 5-3) are presented in Table E-1 and the values for the default composite steel configuration (Figure 5-4) are presented in Table E-2. In both cases, the guidelines of Eurocode 3 (EN 1993-1-8, 2005) were used to derive the equivalent T-stub characteristics of the alternative connection configurations.

Table E-1: **Composite steel full depth endplate** for default support connection configuration

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Reinforcement bar in tension [a]	407.3	352.3	0.01	4.28
Bolt row 1 (top) in tension [b]	246.9	66.4	0.01	87.69
Bolt row 2 in tension [b]	138.6	36.6	0.01	87.69
Bolt row 3 in tension [b]	138.6	36.6	0.01	87.69
Bolt row 4 (bottom) in tension [b]	246.9	66.4	0.01	87.69
Beam flange in compression	2651.78 [c]	*1293.2*	0.01	-
Column flange in compression		1279.4	0.01	-
Column web in shear	1029.1	2015.7	0.01	-

[a] Calculated based on the solution provided by (Anderson et al, 2000)
[b] Calculated based on the equivalent T-stub in Eurocode (includes the endplate in bending)
[c] Calculated based on effective stiffness of the compressive components

Table E-2: **Bare steel full depth endplate** for default support connection configuration

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Bolt row 1 in tension [a]	385.7	103.0	0.01	46.74
Bolt row 2 in tension [a]	219.4	56.8	0.01	46.74
Bolt row 3 in tension [a]	219.4	56.8	0.01	46.74
Bolt row 4 in tension [a]	325.7	79.4	0.01	46.74
Beam flange in compression [b]	2651.78 [b]	*1439.4*	0.01	-
Column flange in compression [b]		1279.4	0.01	-
Column web in shear	1900.7	1291.9	0.01	-

[a] Calculated based on the equivalent T-stub in Eurocode (includes the endplate in bending)
[b] Calculated based on effective stiffness of the compressive components

For the fully welded connections, Table E-3 and E-4 present the component properties for the reduced beam section and welded unreinforced connection respectively (other components are considered to behave rigidly). In order to calculate the initial stiffness, the equivanelnt component is considered as an elastic body with a single degree of freedom; Chapter 3 explains how the dimensions of the equivalent components can be calculated for each connection type.

Table E-3: Bare steel fully welded reduced beam section connection RBS; W27x102 corresponding to NIST **SDC-D frame**; G1 girder to column connection

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Beam flange in tension [a]	1470.8	1234.0	0.01	86.70
Beam flange in compression	1470.8	1234.0	0.01	- [b]

[a] $K_{RBS} = [B_{eq}*T_{flange}*E_{young} / b_{beam}]$ (see Figure 3-4)

[b] Failure mode: inelastic local buckling initiated by critical bending moment loading in the middle of the equivalent width section

Table E-4: Bare steel fully welded unreinforced beam section connection WUF-B; W24x76 corresponding to **NIST SDC-C frame**; G3 girder to column connection

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Beam flange in tension [a]	758.5	1288.5	0.01	41.10
Beam flange in compression	758.8	1288.5	0.01	- [b]

[a] $K_{WUF\text{-}B} = 0.67*5.65*[(B_{beam}*T_{beam}) \wedge 0.5]$

[b] Failure mode: inelastic local buckling; failure criteria: critical moment loading

Table E-5: Bare steel fully welded reinforced with cover plates connection WCF-B; W36x150 corresponding to **SAC Los Angeles** frame; exterior girder to column connection

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Beam flange in tension [a]	12454	3190.2	0.01	71.65
Beam flange in compression	12454	3190.2	0.01	- [b]

[a] $F_{c,fb,Rd} = (W_{pl,eff,joint} * f_y) / (D - t_{fb})$

[b] Failure mode: inelastic local buckling initiated by critical bending moment loading in the middle of the equivalent width section

Table E-6: Bare steel fully welded reinforced with cover plates connection WCF-B; W27x114 corresponding to **SAC Seattle** frame; exterior girder to column connection

Component (equivalent spring)	Initial stiffness (kN/m)	Resistance (kN)	Strain hardening coefficient	Ultimate deformation (mm)
Beam flange in tension [a]	7983	4389.9	0.01	87.30
Beam flange in compression	7983	4389.9	0.01	- [b]

[a] $F_{c,fb,Rd} = (W_{pl,eff,joint} * f_y) / (D - t_{fb})$

[b] Failure mode: inelastic local buckling initiated by critical bending moment loading in the middle of the equivalent width section