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Robustness of Steel Framed Buildings

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Summary

Since the catastrophic failures of the twin towers at the World Trade Centre, avoidance of progressive collapse has become a major concern of designers of multi-storey buildings. Following the partial collapse of a residential apartment block in the UK in 1968, the Building Regulations were amended and required designers to build in measures to reduce the likelihood of damage to a small part of a building resulting in collapse of a disproportionately large part of the structure. All parts of a building are required to be tied together to ensure that they cannot be dislodged easily and, in the event of structural failure of a member, alternative load paths may be mobilised. In the years since 2001, UK practice has often been cited as good practice. Although the UK approach has appeared to work well, there has been little investigation into whether the design recommendations adequately protect a damaged structure from progressive or total collapse.

For this reason, a series of studies was carried out on a typical steel-framed building designed according to the guidelines given in the UK code for structural use of steel in building (BS5950). The analysis used LS-DYNA, a non-linear explicit/implicit finite element code capable of modelling the dynamic behaviour of structures. This investigation examined the structural performance of the buildings, such as the resisting mechanism or if collapse occurs, the failure mechanism during progressive collapse, when key structural members were removed. Most current guidelines for designing against disproportional collapse are based on a static analysis of a damaged structure or an assumed alternative load path which is in turn assured by compliance with design

rules for tying together of the structural members. The investigation concluded that the degree of deformation in a damaged structure is dependant on the time taken to remove a structural element. Thus, dynamic effects should be taken into account when studying/assessing building robustness. The study also examined the role of joint stiffness in resisting progressive collapse, the effect of the rate of loading on structural response, the magnitude of the force induced in the members adjacent to the damaged area and the ability of a range of joints to withstand the these forces and etc. As a result of the findings, a new design methodology (Hybrid design) to ensure robustness in steel framed buildings is proposed and discussed.

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I must give my appreciation to my parents, who have supported me both emotionally and financially at every stage of my progress throughout academia. My special thank go to my husband Phillip, for everything he ever done.

I would like to thank for my best friends-Enrique, and all the other friends/ colleagues I met in the UK, for your kindly help.

Declaration

This thesis is based upon research carried out at the Department of Civil and Structural Engineering, the University of Sheffield, UK. Except where specific reference has been made to the work of others, this thesis is the result of my own work. No part of this thesis has been submitted elsewhere for any other degree or qualification.

Ru Liu

Chapter 1

Introduction

1.1 Introduction

Progressive collapse is defined as the spread of an initial local failure from element to element that eventually results in the collapse of an entire structure or disproportionately large part of it. In the past few decades, research in this area has generally been in response to specific incidents, for example the partial collapse of Ronan Point, the Murrah Federal Building and, most notably, the total collapse of a number of buildings at the World Trade Centre. These incidents focused attention on civil design codes by raising the question of whether they provide adequate protection to progressive collapse; more research is needed to answer this question.

The UK was the first country to address structural progressive collapse and draft rules into its codes. The current UK code for structural steelwork, BS 5950 [BSI, 2000], clause 2.4.5 aims to prevent progressive collapse by tying the structure together to enable a damaged building to resist collapse by catenary action . However, there has been little investigation into whether the design recommendations adequately protect a damaged structure from progressive or total collapse.

In the UK, the design procedures implemented to avoid progressive collapse normally have three stages arranged in order of design complexity:

1 Tying members together against the collapse; if the 'tying' strategy is not adequate then -

2 'Localisation of damage' should be checked by notionally removing an element. The damaged area due to removal of the element is limited to 15% of the floor area or 70m^2, otherwise –

3 The element should be designed as a 'Key element' and be capable of resisting accidental loading as specified in BS6399 [BSI, 1996].

This research focused on stage 1 i.e. determining the magnitude of the tying forces generated in a damaged structure and comparing these to the prescribed design tie force.

Most design codes endorse the use of static analysis to safeguard buildings against progressive collapse. Recently it has been suggested [Marjanishvili, 2004] that progressive collapse should be considered a dynamic event because it '*involves vibrations of building elements and results in dynamic internal forces*'. This research aims to provide evidence to show that progressive collapse is a dynamic issue. But the main purpose is to examine the structural performance of a building during collapse, with particular attention directed to the resistance mechanisms which enable buildings to stand up and the magnitude of the forces induced in the remaining structure.

1.2 Objectives and Methodology

Progressive collapse is normally caused by accidental loads, which may arise from blast or impact, but these studies did not attempt to model the load that caused the damage. Rather, the main research objective of this research is to investigate the behaviour of steel frame structures *after* a key support is destroyed by an accidental load (i.e. blast). For instance, what are the magnitudes of the forces induced in the damaged frame, what is the resisting mechanism if the building stands up; otherwise what is the failure mechanism?

The studies were conducted using the non-linear Finite Element Package LS-DYNA [Reid, 1998; Halliquist, 1999; LSTC, 1999], which is specifically designed for the analysis of dynamic structural problems.

1.3 Thesis Layout

Chapter 2 Literature Review. This chapter reviews the historical lessons and considers the research that has already been done in progressive collapse. Design methods and design codes to prevent progressive collapse are reviewed

Chapter 3 Finite Element Method: Formulation and Initial Studies. Details of the FE package-LS-DYNA are presented and its use in this research justified.

Chapter 4 Finite Element Method: Modelling Strategy and Application. A small 3D steel frame was examined to establish a reliable modelling strategy and test the use of the analysis software.

Chapter 5 Modelling Structural Behaviour During Collapse. Two case studies are reported in this chapter: a pin-rigid 3D frame and a pin-pin 3D frame. The studies were designed to investigate the resisting mechanism for different forms of structure.

Chapter 6 Hybrid Design Method. A new alternative design method to improve the structural robustness and prevent progressive collapse is presented and examined in this chapter.

Chapter 7 Conclusions and Recommendations. Issues which have arisen from the analyses are discussed and observations are made. The main conclusions are drawn together and recommendations for future work are given.

Chapter 2

Literature Review

2.1 Introduction

Progressive collapse is a chain reaction of failures following damage to a relatively small portion of structure [Ellingwood, 1978]. In non-technical words it is referred to as a 'domino' effect. UK Building Regulations refer to disproportionate collapse [HMSO, 1970; HMSO, 1976; HMSO, 1991; HMSO, 1992; DETR, 1994; ODPM, 2004] and require 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". These are different failure scenarios, although both maybe 'disproportionate' to the initial failure. This study is focused on the issue of 'progressive collapse'. The UK was the first country to address progressive collapse following the in famous partial collapse of Ronan Point in 1968 [HMSO, 1968; The Structural Engineer, 1969; ISE, 1969;]. UK design rules to prevent progressive collapse have performed well and are often referred to as an example of good practice and widely cited by other countries [CEN, 2005; ASCE, 2002].

Back in 1999, doubts were raised[1] about current guidelines given by BS 5950 [BSI, 1990; BSI, 2000], namely that they are adequate to protect buildings against progressive collapse. By early 2001, a more detailed proposal for research funding had been proposed[2] but not submitted. Instead the proposal was used as the basis for a studentship, which the author accepted in August 2001 with a view to commencing the study in October 2001. The events of September 11 2001 sparked worldwide interest in this topic. However, recently published work [Corley, 2004; Marjanishvili, 2004] related to the collapse of the World Trade Centre buildings was conducted in parallel with the author's studies and, although relevant, became available too late to inform the direction of this study.

This chapter presents a brief overview of the information that contributes to an understanding of progressive collapse. The related main subject areas are:

- the importance of structural progressive collapse in history.

After all 'history is always repeated', and lessons must be learned from tragedies.

- previous research conducted in this area

[1] Dr J.B. Davison and Dr. A. Tyas, Structural Integrity of Steel Framed Structures, Research Proposal, June, 1999 (not submitted)

[2] Dr J.B. Davison and Dr. A. Tyas, Robustness of Steel Framed buildings, Research Proposal, April, 2000

- methods to design against progressive collapse
- design guidelines around the world

2.2 Historical lessons

Mechanisms that lead to progressive collapse may be investigated by firstly consulting the historical literature. The following structural disasters occurred within the last 50 years in two different countries. However, all three buildings suffered catastrophic failure due to progressive collapse, i.e. disproportionate structural failure following damage to a relatively small area.

2.2.1 Ronan Point Residential Apartment, London UK, 1968

A notable example of progressive collapse was the partial collapse of the Ronan Point apartment building in 1968 [HMSO, 1968; The Structural Engineer, 1969; ISE, 1969]. The collapse was caused by a gas explosion from a domestic cooker on the 18th floor which blew out the exterior wall panels causing a chain reaction of failure to follow that propagated horizontally and vertically.

The Ronan Point apartment building was constructed using pre-fabricated panels that were designed to withstand horizontal wind pressures. When the explosion occurred, the upper floor slabs failed at the outside edges because they were not supported by the exterior cladding. Therefore, continuity in the vertical load path was lost for the upper floor. Debris from floors 18 through to 22 fell onto floor 17, causing a massive overload. Floors 17 to 1 collapsed in succession as each floor became overloaded. This caused the entire corner of the building to collapse (Figure 2-1)

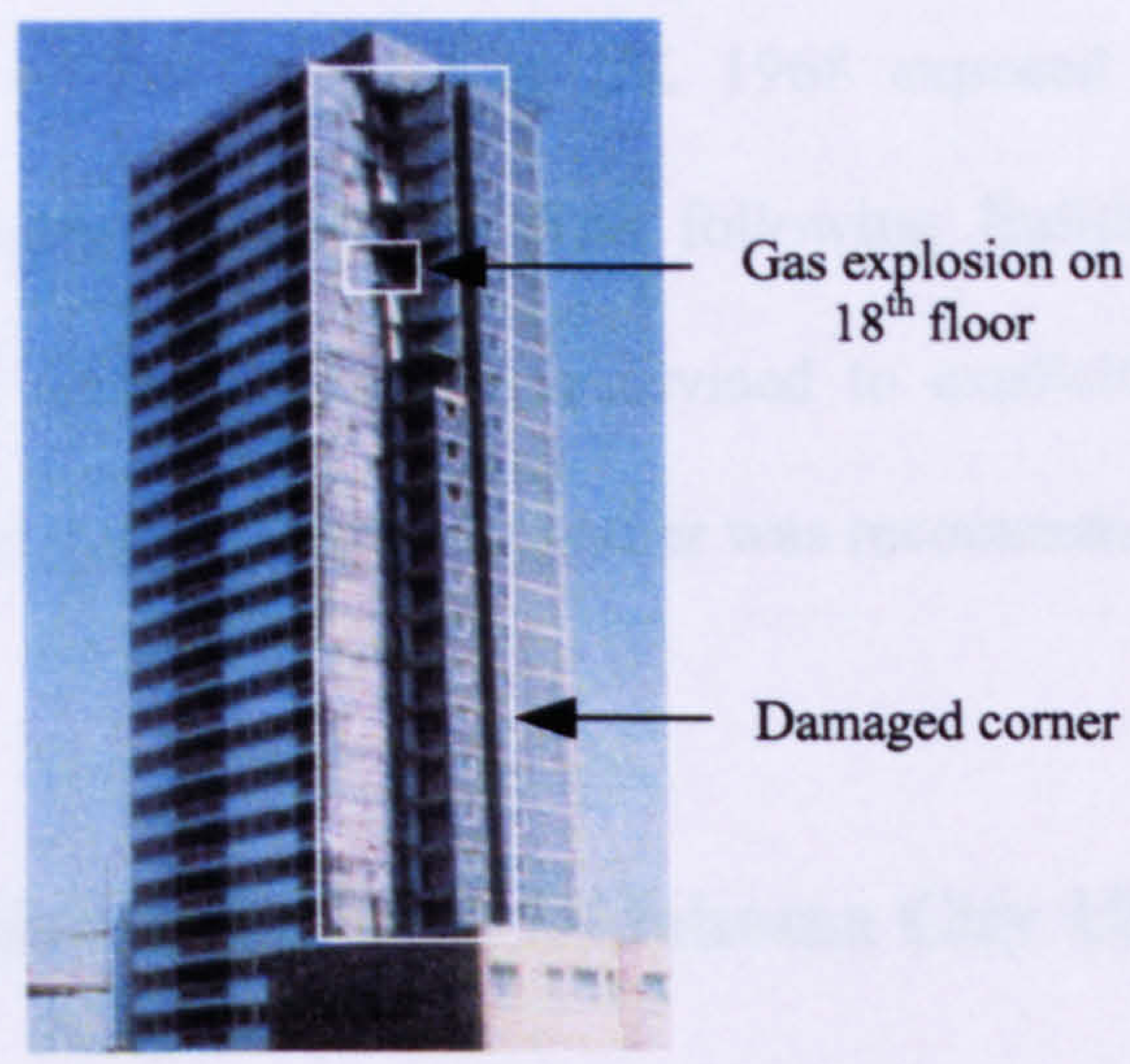

Figure 2-1Ronan Point after a gas explosion on the 18th floor

Following the collapse, much work was done by UK code of practice writers. This resulted in a number of recommendations to guard against disproportionate collapse. In 1975, UK Building Regulations adopted these recommendations, which cover horizontal and vertical continuity, horizontal loading and ductility. For structures greater than a certain number of storeys♣ [DETR, 1994; ODPM, 2004], where ties do not reach the minimum requirements, any single vertical structural member must be able to be removed without causing significant collapse. Where any vertical element cannot be removed, it and its connections must be able to withstand a specified overpressure

♣ 5 storeys required in Approved Document -1994 [HMSO,1994] ; 4 storeys in Approved Document -2004 [ODPM,2004] applied in any direction [HMSO, 1970; HMSO, 1976; HMSO, 1991; HMSO, 1992; DETR, 1994; ODPM, 2004].

The partial collapse of Ronan Point in UK 1968 exposed a significant gap in the understanding of progressive collapse. The following Building Regulations [HMSO, 1968; HMSO, 1970; HMSO, 1976] were revised to explicitly account for accidental loading and 'tying' structural members together was recommended.

2.2.2 Murrah Federal Building, Oklahoma City USA, 1995

A large vehicle bomb was detonated approximately 5m from the north face of the nine-storey Murrah Building in Oklahoma City (Figure 2-2).

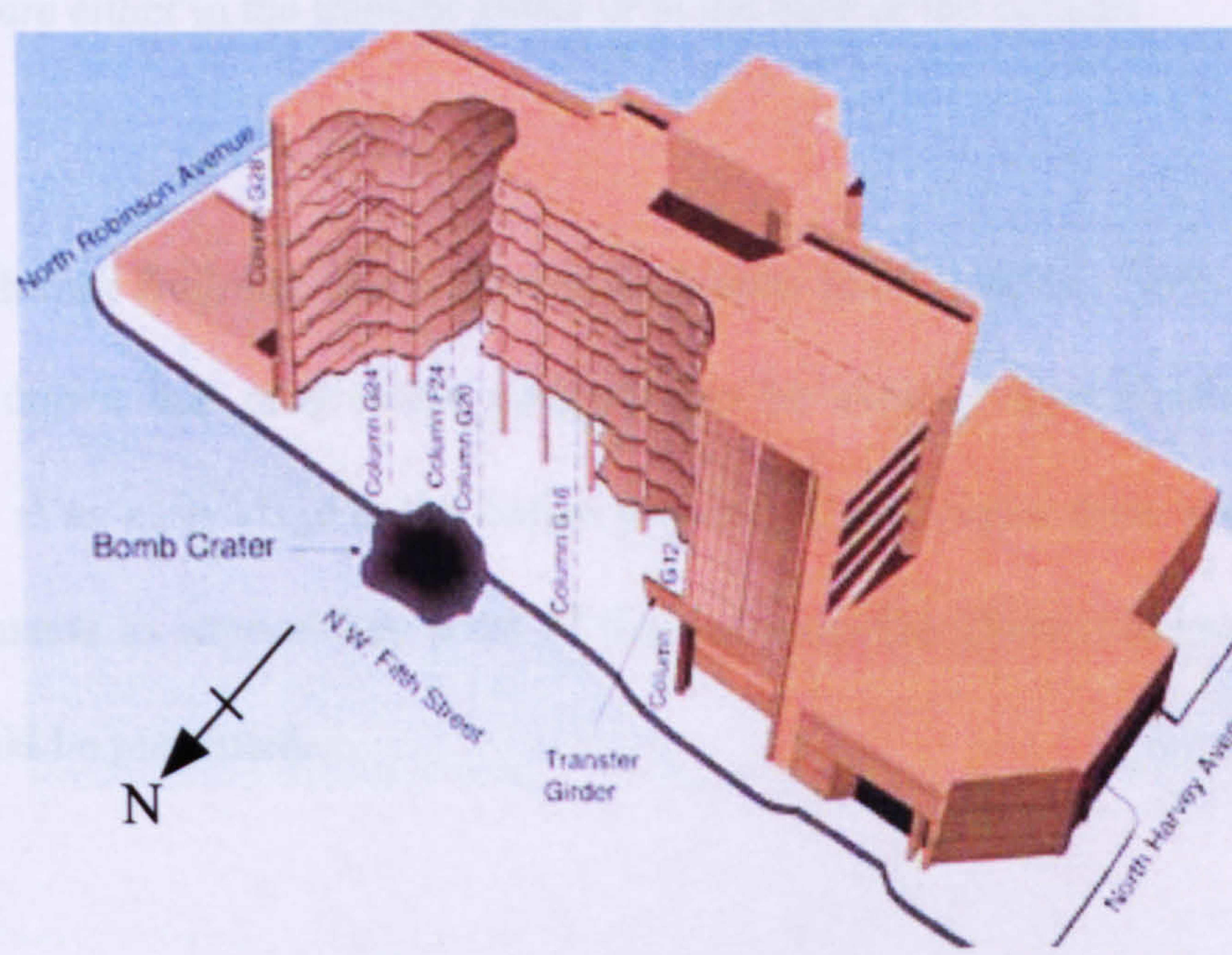

Figure 2-2 Failure Boundaries in Murrah Building [Corley, 1998]

The explosion and resulting collapse caused 168 fatalities [Corley *et al*, 1998; Corley, 2004]. The Murrah Building and other buildings nearby sustained substantial damage. The reinforced concrete slab and column construction was severely damaged at the north face. Column G20 was destroyed by the blast, causing other columns (i.e.G16 G24) to fail in shear, as a consequence. The transfer girder (see Figure 2-2) was then unsupported from the east wall to column G12. Calculations [Corley *et al*, 1998] indicated that the frame could not support itself with three columns missing from the same column line (G16, G20, G24). As a result, eight of the ten bays along the northern half of the building collapsed progressively, together with two bays on the south side. A very recent research paper [Corley, 2004] by Corley discussed three possible collapse mechanisms of the Murrah Building based on its original design and the data collected from site. Corley postulates that the root cause of the problem was a lack of continuity in the reinforcement in the structure either in the transfer girder or at the base of the column.

From the Murrah Building study, recommendations [Corley *et al*, 1998; Corley, 2004] have been drawn that progressive collapse can be avoided by considering structural redundancy at an early stage in the design process. If a designer does not rely solely on critical elements to support key parts of the structure, the chain reaction of successive failures could be prevented.

After the Oklahoma City bombing, Compartmentalised Construction[1], Special Moment Frames and Dual Systems[2] were recommended for designing federal buildings in the US. Those structural systems [Corley *et al*, 1998] would increase significantly the toughness of a structure when subject to catastrophic loading and provide additional mass and strength to help the building behave in a better way, by reducing the possibility of collapse.

2.2.3 World Trade Centre, New York USA, 2001

Two commercial airliners were hijacked and crashed into the two, 110 storey high, World Trade Centre towers on September 11, 2001. This was the worst building disaster in US history and resulted in massive loss of life. Of the 58,000 people estimated to be at the WTC Complex, over 3,000 lost their lives [FEMA, 2001].

The structural damage sustained by each tower from the impact, combined with the ensuing fires (Figure 2-3), resulted in the total collapse of each building. Corley [Corley, 2004] described the collapse:

> *'Once the collapse began, potential energy stored in the upper part of the structure during construction was rapidly converted into kinetic energy.*

[1] In Compartmentalised Construction, a large percentage of the building has structural walls that are reinforced to provide structural integrity in case the building is damaged.

[2] A detailed definition can be found in FEMA-302 [FEMA, 1997].

Collapsing floors above accelerated and impacted on the floors below, causing an immediate, progressive series of floor failures, each punching in turn onto the floor below. The collapse of the floors left tall, freestanding portions of the exterior wall. As the unsupported height of these freestanding exterior wall elements increased, they buckled at the bolted column splice connections and also collapsed. The process was essentially the same for both Tower1 and Tower2.'

The tragic events shocked people and caused general awareness of catastrophic collapse of structures. The World Trade Centre events have highlighted a lack of understanding of progressive collapse. According to a recent report from the Multihazard Mitigation Council (MMC), Americans concluded that progressive collapse is not well understood and defined, and more effort needs to be put into collecting existing research, identifying future efforts, and related areas. There is also a need to develop a National Standard for the prevention of progressive collapse [MMC, 2003].

Figure 2-3 Explosion of Tower2, when the second aircraft hit

2.2.4 Discussion

Progressive collapse is not a new research topic in the field of structural engineering. Early research on structural collapse can be found in last century [HMSO, 1968; The Structure Engineer, 1969; ISE, 1969; Allen and Schriever, 1972; Popoff, 1975]. Ronan Point partial collapse is a classic example and prompted the UK was to draft rules [HMSO, 1968; HMSO, 1970; BSI, 1972; HMSO, 1976; BSI, 1985; BSI, 1990; HMSO, 1991; HMSO, 1992] on preventing progressive collapse. The UK design procedures implemented to avoid progressive collapse [BSI 2000; DETR, 1994; ODPM, 2004] normally have three stages [BSI, 2000; Way, 2003; SCI 98/99] arranged in order of design complexity. Details about the design stages will be discussed in section 2.5.

The tying strategy is a direct design procedure in which a minimum tying force is specified. This minimum force is required to tie structural members in two horizontal directions and it is an accepted solution for design against progressive collapse; it has been adopted in many other countries' design codes [ASCE, 2002; BSI, 2005; CEN,2005]. However, there has been little investigation into whether the design recommendations adequately protect a damaged structure from progressive collapse; therefore it is necessary to conduct a study in this area.

The events of September 11th have also made engineers rethink whether structural performance to avoid a progressive collapse is well understood. Clearly, WTC is the worst structure failure in US, but it is not the first terrorist attack. Back in 1995, the Oklahoma City bomb caused 168 fatalities in the explosion and resulting collapse. The investigations of Murrah Building concluded [Corley *et al*, 1998] that 80% of the deaths were related to the progressive collapse rather than the blast. Subsequent research on the Oklahoma City bomb highlighted that it is important to have more than one load transfer path instead of relying on only a few key elements (the transfer girder). Following those studies, new design methods (such as Compartmentalised Construction, Special Moment Frame, Dual System) aimed to improve the structural redundancy have been recommended for construction of all federal buildings in the US. To a certain extent, those design approaches are useful to improve a structure's behaviour under some extreme threat, but for engineers it is more important to learn the lessons from those tragedies, and avoid collapse in future designs or at least limit the damage.

2.3 Review of Previous Research Work on Prevention of Progressive Collapse

Following the events of September 11, 2001 many reports have been published and much has been written about how the avoidance of progressive collapse may be best addressed. As much of this information became available after the research had commenced this section of the literature review has been divided into pre and post 11/9/01. The reader should remember that reports published in 2003 and later were too late to change the direction of the reported work. It is interesting to note that some of the recommendations in the post 9/11 work are addressed by this work.

2.3.1 Before WTC collapse

The partial collapse of Ronan Point apartment made engineers notice that progressive collapse is not understood and lots of related research work was conduced in the UK soon after the collapse [HMSO, 1968; The Structural Engineer, 1969; ISE, 1969; HMSO, 1970]. Ties, which provide structural integrity, were addressed in the UK building regulations to prevent progressive collapse. Since then, new areas of research into progressive collapse have opened.

Meanwhile in the US, parallel studies were carried out by Allen and Schriever [1972]. They summarised incidents involving progressive collapse/ abnormal load that happened in North American (US, Canada) between 1969 to1972. In 1978, Ellingwood [1978] discussed the design strategies that reduce the risk of progressive collapse by using the probabilistic method. In 1979, Ravindra and Galambos [1979] gave an illustration to develop the design criteria for steel buildings by applying the load and resistance factor method. In 1983, Gross [1983] presented studies of progressive collapse. In his 2D computer-based analytical model, he reported structural behaviour related to columns removed from different locations and he explained the alternative load path method which it is claimed can prevent progressive collapse. Later on in 1983, based on his previous study, Ellingwood [1983] discussed failure caused by abnormal load; the analysis examined structural vulnerability arising from unreinforced masonry walls facing a gas explosion.

In Sweden, Girhammar [1980] published his PhD thesis on 'Dynamic Fail-safe behaviour of steel skeleton structures having bolted connections'. In his thesis, the dynamic behaviour of steel skeleton structures due to primary damage was examined and some properties of different connections were taken into account.

In the UK, Pretlove [1991] reported his research into dynamic effects in progressive failure. He examined a loaded structure in which members break progressively. In his dynamic experiment, he included the transient overloads induced by the sudden fracture of a member and he showed that fracture failure of one member can cause other elements to fracture progressively before a new equilibrium state is reached.

In the UK, the tying strategy is the simplest way to provide the minimum robustness of structure to resist accidental loading, which means the connections must be capable to transferring the tying force. Therefore, in 1992, Owens and Moore [1992] presented a series of test data aimed to investigate the ability of simple steel connections to resist tying forces. This experimental series provided the background for the connection design approaches in the BCSA/SCI Green Book guide to simple connections [SCI /BCSA, 2002]

In 1996, Stefieck reported an interesting methodology to protect the exterior of a six-storey building, New York City Technology Center [Stefieck, 1996]. The building had been designed with a rigid frame but in order to increase its robustness the designer increased the size of the spandrels and columns, as well as the moment capacity and ductility of the beam-to column connections. In so doing, the frame had sufficient redundancy to enable it to withstand the removal of an exterior column.

After the Oklahoma City bomb, a number of researchers [Longinow, 1996; Yandzio, 1999] investigated the blast loading in detail. In 1998, Corley [Corley *et al*, 1998] reported on an investigation of the Oklahoma city bomb aiming to '*review the damage caused by the blast, to determine the failure mechanism for the building, and to review engineering strategies for reducing such damage to new and existing building in the future*'. As a result, the Compartmentalised Construction, Special Moment Frame and Dual System were recommended for all the new Federal buildings in order to improve structural redundancy.

In the UK, Beeby [1999] discussed the safety of structures, as he believed that robustness is not well understood. He devised a way to define robustness by using energy absorption either in a structure or in a member.

This section has reviewed some historical contributions in the research area of progressive collapse. Progressive collapse is not an isolated research area, instead it links many topics, i.e. structural dynamics [Clough, 1975; Smith and Hetherington, 1994], structural stability/ reliability [Lightfoot, 1961; Rubinstein, 1970; Melchers, 1987; Narayanan; 1989; Chen, 1991; Usami; 1998], material properties [Byfield, 1997], etc. It is difficult to cover all the related contributions, therefore, the thesis only covers what the writer considers to be the most relevant. The next section briefly reviews research conducted after the WTC event.

2.3.2 After WTC

The US authorities have expended a lot effort in addressing concerns about progressive collapse after 11/9/01. In May 2002, FEMA[1] in association with SEI/ASCE[2] published preliminary studies [FEMA, 2002] 9 months after the collapse. The report addressed that '*Structural framing systems need redundancy and or/ robustness, so that alterative*

[1] FEMA= Federal Emergency Management Agency, US

[2] Structural Engineering Institute of the American Society of Civil Engineers

paths or additional capacity are available for transmitting loads when building damage occurs' and additional studies were required.

At same time, in the UK the ISE[1] was working on 'Safety in Tall Buildings' [ISE, 2002] aiming to '*provide guidance and advice on the implications that follow that structural collapses and loss of lift at the World Trade Center*'. The recommendation for consideration in the section on the Vulnerability to progressive collapse is given as '*use structural elements with robust, ductile, and energy absorbing properties and tie them together with strong ductile connections*'.

In February 2003, the NYC department of Buildings established a Task Force '*to ensure that requirement, standards and practise in the design and construction of buildings provide safety for occupants of tall building*' and recommended that '*structural design guidelines for optional application to enhance robustness and resistance to progressive collapse*' be published.

Also in 2003, the MMC[2] [MMC, 2003] of NIBS[3] in association with GSA[4] held a Workshop on 'Prevention of Progressive Collapse' with the aim of '*collecting the*

[1] ISE= the Institution of Structural Engineer, UK

[2] MMC= Multihazard Mitigation Council, US

[3] NIBS= National Institute of Building Science, US

[4] GSA=General Services Administration, US

existing research and identifying future efforts to mitigate the impacts of progressive collapse'. The report concluded '*It was the consensus of the participants that there is a need for a coordinated national effort to develop engineering tools to assist in designing structure to resist progressive collapse and to develop methods to rehabilitate structure that are vulnerable to progressive collapse.*'

In June 2003, the GSA [GSA, 2003] published design guidelines on progressive collapse '*for minimizing the potential for progressive collapse in the design of new and upgrade building, and for assessing the potential for progressive collapse in the existing buildings*'

In March 2004, Hamburger [Hamburger, 2004] reported an analytical study of a one storey high 3D frame with the middle column removed using non-linear FE software SAP2000. According to his research, he concluded that catenary action was an alternative resisting mechanism for re-distribution of the load in a damaged frame.

At the same time in the UK, Byfield [Byfield, 2004] pointed out that since beams are often designed strong enough to resist twice the design load then the building is likely to be capable of surviving an extreme event. But he believed the strong beams would cause the connections to be the weak point in the building, and therefore result in a damaged structure which is non-ductile and potentially susceptible to progressive failure.

In 2004, Alexander [Alexander, 2004] suggested that instead of checking the structural behaviour of key element removal, there is a need to check all the columns removed in turn.

This section has briefly reviewed research/report on progressive collapse after the WTC event. It is difficult to include all the up-to-date research about progressive collapse as some research has not yet to be finished, therefore only the most relevant studies are given

2.4 Overview of Design Methods for Prevention of Progressive Collapse Caused by Accidental Loading

The probability of structural failure caused by abnormal load [Ellingwood and Leyendecker, 1978; Ravindra and Galambos, 1979; Ellingwood et al 1982; Gross and McGuire, 1983] can be stated as:

$$P\,(F) = P\,(F/A)\,P\,(A) \qquad (2.1)$$

in which

P (F) is the probability of failure;

P (F/A) is the probability of failure given that an abnormal load occurs;

P (A) is the probability of the occurrence of an abnormal load.

There are two ways to reduce the probability of failure, either reduce the probability of the occurrence of abnormal loading P (A) or reduce the probability that failure will be caused by abnormal loading P (F/A). Therefore, the design approaches for reducing the risk of a progressive collapse can be summarized as:

1. Event control – reduces P (A)
2. Indirect design - one way to reduce P (F/A)
3. Direct design - another way to reduce P (F/A), attempts to ensure that the structure can withstand abnormal loading

2.4.1 Event Control

Event control reduces the likelihood of the occurrence of an abnormal load P (A) and refers steps taken to avoid or protecting a building against incidents that might cause progressive collapse. This approach does not increase the inherent resistance of the structure and also depends on factors outside the designers' control therefore, in the past, it has not been a popular design method [Ravindra, 1978; Ellingwood 1978; Ellingwood 1983]. With the increase of terrorist attacks, it becomes clear that it is important in some cases to eliminate the possible threat and thereby reduce the risk of collapse. In this sense, event control becomes an important factor to be considered when protecting a building against progressive collapse.

Back in 1999, the SCI guideline [Yandzio and Gough, 1999] believed that '*preventive measures*' were '*the cheapest method of securing protection against the effects of blast*' and gave details such as external layout planning, access control, and etc. to minimize the effects of bombs. After 9/11/01, the GSA [GSA, 2003] guidelines have adopted a philosophy of event control and applied this to help to eliminate or at least reduce the potential terrorist threat thus protecting buildings.

2.4.2 Indirect Design

An indirect design approach is a way to reduce the probability of failure caused by abnormal loading (P (F/A)) by providing a minimum level of strength, continuity and ductility so that a structure has an inherent resistance to progressive collapse [Ellingwood 1978; Ellingwood 1983].

For example, the UK design code BS5950 [BSI, 2000] gives the requirement under section 2.4.5 of structural integrity that '*all buildings should be effectively tied together at each principal floor level*' and 'a *factored tensile force*' should be resisted by all horizontal members. When designed in accordance with these requirements, a minimum tying force arising from ties is provided thus ensuring that the building possess a degree of robustness that should prevent progressive (disproportionate) collapse in the event of damage to a small part of it.

2.4.3 Direct Design

There are two basic types of direct design, namely local resistance and alternate load path. The local resistance method provides sufficient strength to resist an abnormal load by ensuring all load-bearing elements remain in place. The alternate load path method permits local damage to occur but provides alternate load transfer paths around the damaged area. This enables the structure to sustain abnormal loads without total collapse.

2.4.3. Others

After the 11/09/01, a number of researchers [Hamburger, 2004; Marjanishvili, 2004; Corley, 2004; Shankar, 2004; Ellingwood, 2003; Burns, 2003; Choi *et al*, 2003; Krauthammer, 2003; Cagley, 2003] have expressed concern that ordinary building design is not adequate to safeguard against progressive collapse, and therefore have suggested a more sophisticated FEM analysis is necessary to assess the vulnerability of a building to collapses when a component, usually a ground floor column, is removed. This approach has been adopted in the GSA guidelines [GSA, 2003] on preventing progressive collapse. It has been also suggested that the application of earthquake-proof design methods might serve as a means of anti-progressive collapse design [MMC, 2003].

2.5 Review of Current Design Practices for the Prevention of Progressive Collapse

2.5.1 Introduction

Structural progressive collapse has resulted in loss of life and property throughout the world. Each country has its own building protection philosophy but not all countries recognise the need to mitigate against progressive collapse. The existence and potentially devastating consequences of abnormal loads have led to progressive collapse being acknowledged in most structural design standards. Most standards [BSI, 2000; ODPM, 2004; BSI, 2005; CEN, 2005; ASCE 2002] state that local damage to the structure shall not have catastrophic consequences, but the detailed provisions against progressive collapse vary from country to country.

The following section reviews the design requirements of European countries, and the US. Due to the early and important influence of the UK rules, the UK design codes are reviewed first.

2.5.1 UK design codes of practice

The UK Building Regulations are a legal statutory instrument, which refer to 'British Standard' Codes of Practice to refer. The latest UK building regulations [ODPM, 2004] categorise all buildings into one of four classes, that is class 1, class 2A, class 2B and class 3. All buildings, irrespective of the number of storeys, are required to have effective horizontal ties but buildings over 4 storeys are also required to have effective vertical ties [BSI, 2000; DETR, 1994; ODPM, 2004].

The investigation of the Ronan Point collapse indicated the importance of tying together structural elements. The Institution of Structural Engineers report [ISE,1969] and the subsequent Amendment (fifth) of the Building Regulations [HMSO, 1970], noted that '*the building should be so constructed that, in the event of an accident, the structure will not be damaged to an extent disproportionate to the extent of damage*' and recommended 'tying' structural members together. The tying can be developed either by supporting a load directly or by supplying an alternative load path. The possible 'post failure' conditions resisted by tying are illustrated in Figure 2-4 [SCI 98/99].

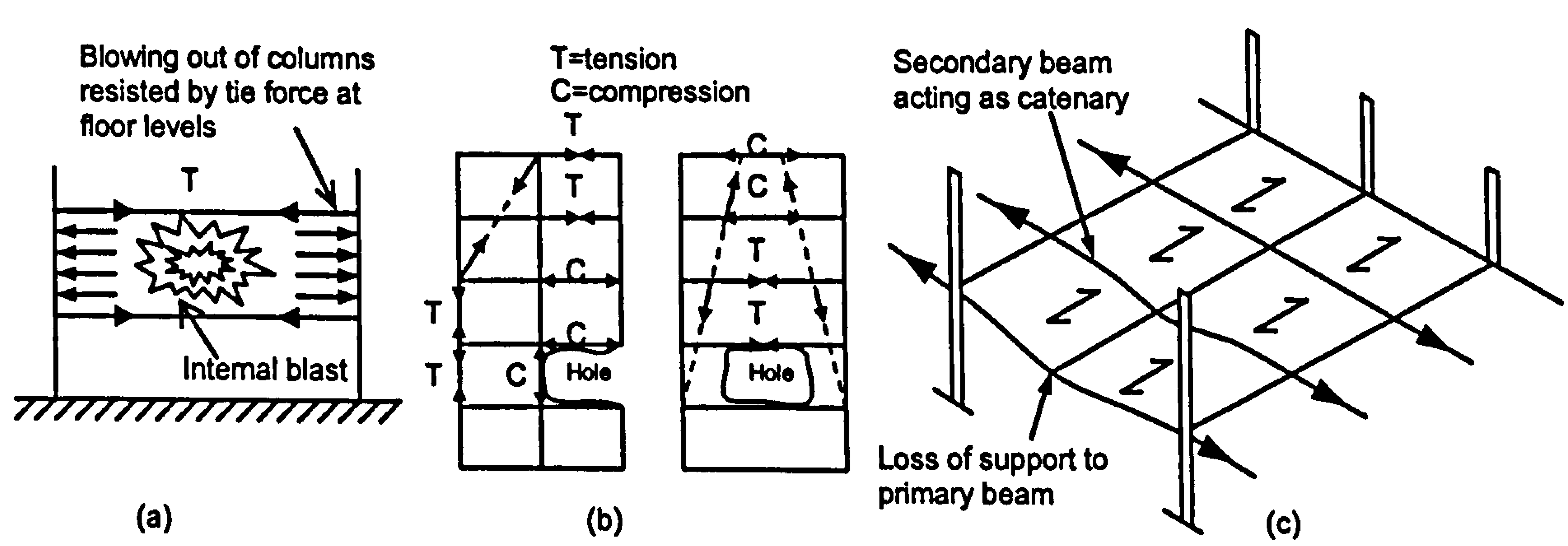

Figure 2-4 Simulations of post failure condition [SCI 98/99]

In the UK, the design procedures implemented to avoid progressive collapse normally have three stages [Way, 2004, SCI 98/99] arranged in order of design complexity:

1. Tying members together against the collapse; if the 'tying' strategy is not adequate then –
2. 'Localisation of damage' should be checked by notionally removing an element. The damaged area due to removal of the element is limited to 15% of the floor area or 70m^2, otherwise –
3. 'Key elements♣, defined as elements whose removal would result in a progressive collapse must be identified and designed out of the solution if possible. Where it is not possible to eliminate key elements, they should be designed to resist accidental loading as specified in BS6399 [BSI, 1996].

To determine the magnitude of the tying forces generated in a damaged structure, BS 5950 [BCI, 2000] requires steel members acting as horizontal ties to resist tensile forces of:

$$0.5(1.4g_k+1.6q_k)s_tL \quad \text{but not less than 75kN (internal ties)} \tag{2.2}$$

$$0.25(1.4g_k+1.6q_k)s_tL \quad \text{but not less than 75kN (edge ties)} \tag{2.3}$$

Where

♣ Key elements are defined as those structural elements at any one storey whose loss results in a collapse of the structure more than one storey above or below the element under consideration, or over a horizontal area in excess of that stipulated in the criterion.

g_k is the specified dead load per unit area of the floor or roof;

L is the span;

q_k is the specified imposed floor or roof per unit area;

s_t is the mean transverse spacing of the ties adjacent to that being checked

For the equations above, it has been stated [SCI 98/99] that they are based upon a beam with a span of twice the storey height deforming as shown in Figure 2-5. In the extreme condition, it is assumed that the beam rotates 45° at the supports. In order to satisfy equilibrium then the horizontal and vertical forces have to be equal. The 75kN limiting (minimum value) is simply based on good practice which would employ a minimum of 2 Mile 8.8 bolts in any structural connection, resulting in this capacity.

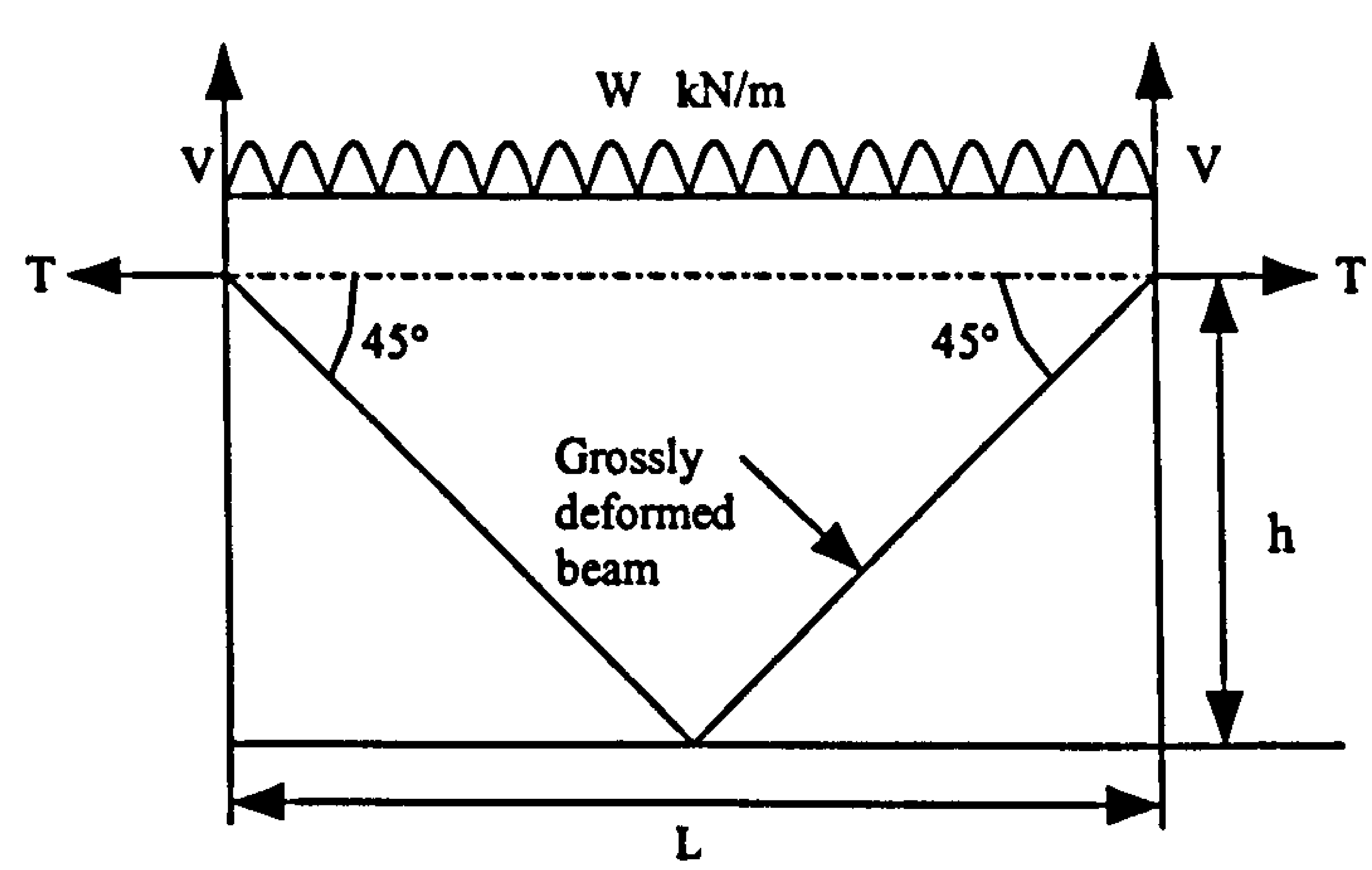

Figure 2-5 Derivation of catenary forces in BS 5950 [SCI, 98/99]

However, the accuracy and applicability of the guidance given in BS 5950 is questionable for a number of reasons:

1. It is assumed that the tie beams will be sufficiently ductile to allow a highly deformed catenary to develop.
2. The strength and stiffness of the structure adjacent to the damaged bay may affect the development of tie forces. This is not considered.
3. The interdependence of the response at different storey levels above the damaged bay is ignored.
4. Resistance to the mobilised tie force in the rest of the structure is not addressed.
5. Beam tie action is the only load resisting mechanism considered. Other load resisting mechanisms may exist.

2.5.2 Design Codes in Europe

The latest Eurocode1 part 7 'General Action-Accidental Actions' [CEN, 2005] provides '*strategies and rules for safeguarding buildings and other civil engineering works against identified and unidentified accidental actions*'. It categorises building as class 1, class 2- lower Risk Group, class 2- Upper Risk Group and class 3 which relates to the low, medium and high 'Consequence Classes' [Gerhard, 2000; Gulvanessian *et al*, 2002; Bertagnoli, 2003/2004; Moore, 2004; CEN, 2005]

EN 1991-1-7 covers the principles adopted in the previous draft code (i.e. ENV:1991-2-7), and it improves and adds some specific rules for safeguarding buildings, due to the consequences of local damage or failure because of an unexpected event. In detail, when designing limiting the extent of localised failure, mitigation can be achieved by using one or more of the following approaches [CEN, 2005]:

- *Designing key elements, on which the stability of the structure depends, to sustain the effects of a model of accidental action A_d*[♣]
- *Designing the structure so that in the event of a localised failure the stability of the whole structure or of a significant part of it would not be endangered;*
- *Applying prescriptive design/detailing rules that provide acceptable robustness for the structure tying for additional integrity, or minimum level of ductility of structural elements.*

In order to achieve structural robustness, EN 1991-1-7 suggests using horizontal and vertical ties. The tying force required in EN1991-1-7 for horizontal ties, is similar to that specified in BS5950, that is

$T_i=0.8(g_k+\psi q_k)sL$ or 75 kN, whichever is the greater (internal ties) (2.4)

$T_p=0.4(g_k+\psi q_k)sL$ or 75kN, whichever is the greater (perimeter ties) (2.5)

Where

s is the spacing of ties

L is the span;

♣ the recommended value of A_d is 34kN/m^2

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g_k is the permanent load

q_k is the variable load ;

ψ is 1.0

There are obvious similarities between the European Standard and British Standard on designing against accidental loading. They implement similar design philosophies, that is they require ties (horizontal as well as vertical), and the design of key elements to provide structural robustness.

2.5.3 Design Codes in US

Before the WTC collapse, steel designers in the US faced the problem of a lack of a unified national design code, and the 'Specification for the design, fabrication and erection for buildings' (hereafter referred to as the American Institute of Steel Construction specification) is normally referred to as a de facto national standard [Bertagnoli, 2003/2004]. The early edition of AISC [AISC, 2001] concerned the designing, fabrication and erection of steel framed building in a normal use. The new edition of 2002 also includes seismic provisions and provides the information to improve the 'design strength' in terms of enhancing the seismic safety. Structures should be capable of resisting the maximum considered earthquake at a near collapse or better performance level. Although the AISC specification does not provide clear guidance on the prevention of progressive collapse the redundancy and ductility required to resist seismic action is likely to ensure robust performance if damage from some other source occurred.

The loading combination [Ellingwood, 2003; GSA, 2003; Bloomberg, 2003] of a damaged structure in an accidental load event is as follow:

$$(0.9 \text{ or } 1.2)\, D + (0.5L \text{ or } 0.2S) + 0.2\, W \tag{2.6}$$

where

D, is dead load,

L, is live load,

S, is snow load,

W, is wind load,

For designing a key element or main load-bearing member to withstand the accidental effects then the loading combination is as follows:

$$(0.9 \text{ or } 1.2)\, D + A_k + (0.5L \text{ or } 0.2S) \tag{2.7}$$

Where,

A_k, is action due to abnormal load.

After the WTC attacks, the GSA (General Services Administration) Guidelines on progressive collapse were published [GSA, 2003]. Section 5 of this document gives requirements for a steel frame building (new construction or existing building) in a step-by-step analytical procedure for linear elastic, static analysis as follows:

Step1 Remove a vertical support from the location being considered and conduct a linear-static analysis of that structure. Load the model with 2 (D+0.25L). (The load factor 2 is a dynamic amplification factor to account for deceleration effects [Marjanishvili, 2004])

Step2 Determine which member and connections have Demand-Capacity Ratios (DCR[1]) values that exceed the acceptance criteria that was required

Step3 For a member or connection whose DCR exceeds the requirements, place a hinge at the member end or connection to release the moment.

Step4 At each inserted hinge, apply equal-but-opposite moment to the stub/offset and member end to each side of the hinge.

Step5 Re-run the analysis and repeat Step1 through 4. Continue this process until no DCR values are exceeded.

$DCR = Q_{UD}/Q_{CE}$,

where

Q_{UD} = Action force (demand) determined in component or connection/ joints (Moment, Axial force, shear, possible combined force)

Q_{CE} = Expected ultimate, un-factored capacity of the component and /or connection/joints (moment, axial force, shear and possible combined forces)

Most building codes in the US do not provide provisions that relate to general structural integrity [Ellingwood 2003, MMC, 2003; GSA, 2003, Shankar, 2004] and the WTC event has brought this issue to the fore. In order to achieve 'robustness' against progressive collapse, it is necessary to have structural integrity. Also the lack of a national design code in the US has been recognized as an urgent problem that should be addressed [MMC, 2003].

2.5.4 Discussion

In the previous section, the design requirements for preventing progressive collapse by improving the structural integrity in the UK, US and Eurocode have been outlined. The influence of the UK design rules on other code, especially to the Eurocode, is clear to see. After the tragedy of the WTC collapse, the US code writers faced pressure to produce a national building code that provides design guidelines for preventing progressive collapse. There is no doubt, that in the US, researchers have made rapid progress in the progressive collapse field [MMC, 2003; GSA, 2003; Hamburger, 2004; Marjanishvili; 2004; Corley, 2004; Shankar, 2004; Ellingwood, 2003; Burns, 2003; Choi *et al* , 2003; Krauthammer, 2003; Cagley, 2003] after the WTC. The lessons should be learned not only by US engineers, but all civil engineers should be aware of it and try to avoid this sort of tragedy happening again.

Historically, the concept of designing structures for protection against abnormal loading was brought about by the military engineer. Due to economical reasons, there is no extensive application of military design approaches for civilian structures. Structural failures have highlighted the inadequate protection provided by civil design codes. It is the task of civil engineers to find the best way to design and build structures that are resistant to extreme events without excessive expense.

2.6 Conclusions

The partial collapse of the Ronan Point alerted engineers to the importance of tying members together, therefore after this collapse the UK was the first country to specifically require that members should be tied together and minimum tying force values were established. As time passed, the minimum tying force has proven to be a very effective way to provide the structural integrity and prevent progressive collapse. Therefore, the tying strategy has been adopted in many countries design guidance for the prevention of progressive collapse. Clearly, as shown in section 2.5, the influence of the UK tying strategy can be found in Europe as well as the US. The contribution of a tying strategy as a good design practice in preventing the progressive collapse has been acknowledged world wide for nearly 30 years.

The 9/11 event sparked a full investigation of progressive collapse and questioned the understanding of progressive collapse. The research/report published after the event has raised the same doubt as to whether the tying strategy alone can provide enough robustness to prevent collapse. Furthermore, as many researchers consider that progressive collapse is a dynamic problem, the use of an essentially static approach (minimum tying forces) appears inappropriate. The research reported herein aims to give a better understanding of the forced generated in a steel framed building subjected to damage and compare these with the design value suggested in UK code.

Chapter 3

Finite Element Method: Formulation and Initial Studies

3.1 Introduction

The finite element method (FEM) is a powerful analytical tool for the study of the response of real structures. The studies presented in this chapter briefly review some applications of the FE method. Particularly, it investigates the applicability of the specific non-linear explicit/implicit package LS-DYNA [Hallquist, 1998; Reid, 1998; LSTC, 1999] for this study. The formulation of each basic structural component is briefly reviewed and its application to the problem at hand examined accordingly.

3.1.1 General Information about FE method

The finite element method (FEM) is '*a numerical procedure for analyzing structures and continua*' [Cook, 1989]. The basic concept behind the FEM is the subdivision of a region into sufficiently small regions so that the solution in each small region (element) can be represented by a simple function [Grandin, 1986].

3.1.2 Introduction of choosing FE package

The major task of this research focuses on the structural behaviour of a steel framed building during collapse, particularly the force induced. As discussed earlier (see Chapter 2), progressive collapse has been recognized [MMC, 2003; GSA, 2003; Corley *et al*, 2004; Marjanishvili; 2004; Liu et al, 2005] as a dynamic problem. It is well known that the dynamic behaviour of a structure is often difficult to predict, particularly when it has been damaged by an accidental load. Normally the solution to a structural dynamics problem is considerably more complicated than its static counterpart. The addition of inertia and damping (related to time) of a dynamic problem has to be taken into account [Clough, 1975]. Therefore, it is necessary to use the right finite element analysis code. In order to achieve the task, the finite element code in this study should be able to:

1. Model a complex dynamic event. The code should have the ability to combine static loads (e.g. self-weight) with rapidly applied dynamic loads resulting from a change in the load path due to the removal of a key structural element.

2. Model non-linear deflection, checked by the P-Delta (P-δ) effect [Chen, 1986, Gupta, 1999]. Structural response to a dynamic load may be expressed in terms of displacement. For a damaged structure, displacements are likely to be especially large when catenary action occurs. Therefore, the finite element code needs to be able to model non-linear geometric and material behaviour.

3.1.3 Explicit/Implicit analysis of LS-DYNA (LLNL-DYNA3D)

LS–DYNA (LLNL) [Hallquist, 1998; Reid, 1998; LSTC, 1999; Lin, 1999] is a general purpose finite element code for analysing the large deformation dynamic response of structures and its main solution methodology is based on explicit time integration [Hallquist, 1998]. When solving nonlinear transient problems, the advantage of the explicit method becomes more obvious, as the integrated time steps are used to update the solution by adding the increments for each time step. Therefore, there is no requirement for the inversion of the stiffness matrix and also no convergence is needed.

LS-DYNA also provides an optional solution based on implicit time integration. In detail, this implicit method is often used for solving static related problems and neglects the time steps during the calculation. The average acceleration and displacements are evaluated at time $t + \Delta t$, given by:

$$\{U_{t+\Delta t}\} = [K]^{-1} \{F_{t+\Delta t}\} \quad (3.1)$$

For linear problems, the solution of this equation 3.1 is unconditionally stable when the stiffness matrix is linear, and large time steps can be taken. When solving non-linear problems with the implicit method, the advantage is less obvious. It is difficult to solve the inversion of the stiffness matrix for non-linear problems, also the convergence is hard to achieve for highly nonlinear problems.

Accordingly, in order to solve a non-linear problem, explicit time integration algorithms are a better choice, as the explicit method is much less sensitive to machine precision than other finite element solution methods [LSTC, 1999]. Obviously, the explicit solution is not perfect, and it has limitations. The two major disadvantages of the explicit method are:

1 The time steps need to be very small in order to maintain the stability limit.

2 The calculation of internal forces♣ is computationally expensive.

Considering that the major task of this research is to determine the resisting mechanism during progressive collapse, non-linearities (geometry, material) [Chen, 1985] have to be included. Because of this, it was decided to choose the explicit approach. The details of the time integration loop used in LS-DYNA can be found in Figure 3-1.

♣ All the nonlinearities (including contact) are included in the internal force vector

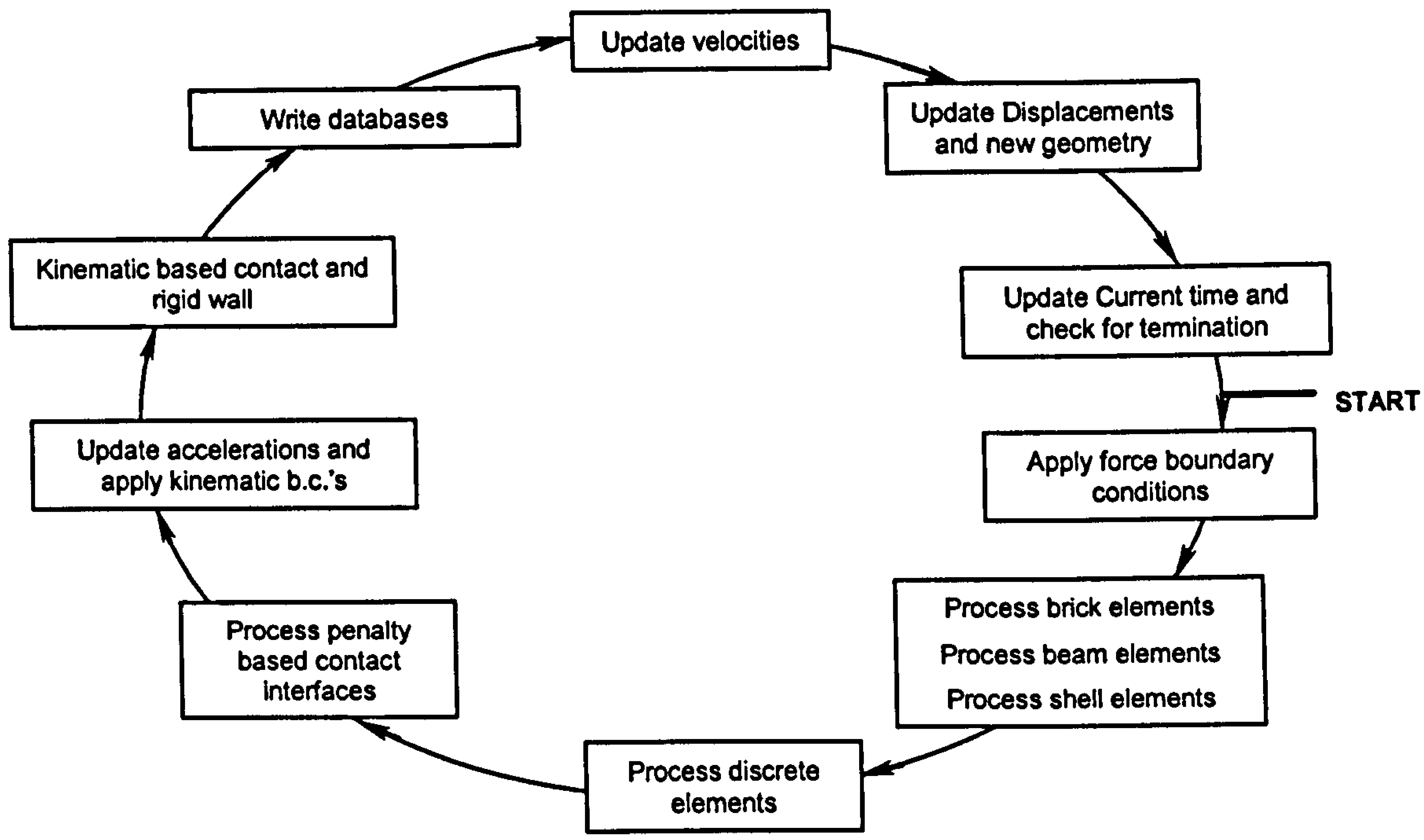


Figure 3-1 The time integration loop in LS-DYNA [Hallquist, 1998]

The original LS-DYNA public domain software DYNA3D, dates back to the mid-seventies, and was firstly developed by Lawrence Livermore National Laboratory (LLNL-DYNA). Since the first version of DYNA3D released in 1976, it has kept improving through the years and each new version of DYNA3D brings new features to the users, so that LS-DYNA seems to be the most appropriate FE software[Hallquist, 1998; Reid, 1998; LSTC, 1999; Lin, 1999] for this research.

3.2 Element Formulations

LS-DYNA3D is able to model different structural components as well as complete structures in three dimensions. A pre-processor Oasys-Primer [Oasys, 2002] is used to create all the geometry and Oasys-D3Plot [Oasys, 2002] post-processes all output data. The following section reviews three different numerical elements used in the research, namely *beam element*, *discrete beam element* and *shell element*.

3.2.1 Beam element

The steel beams and columns in a frame structure can be constructed in LS-DYNA3D using a *beam element* (see Figure 3-2). The coordinate of r, s, t is normally used to define the steel beam/column cross section in the local system, the reference node is n3, which determines the initial orientation of the cross section in the global system. Two different types of beam element formulation are currently implemented- Belytschko (BS) and Hughes-Liu (HL)

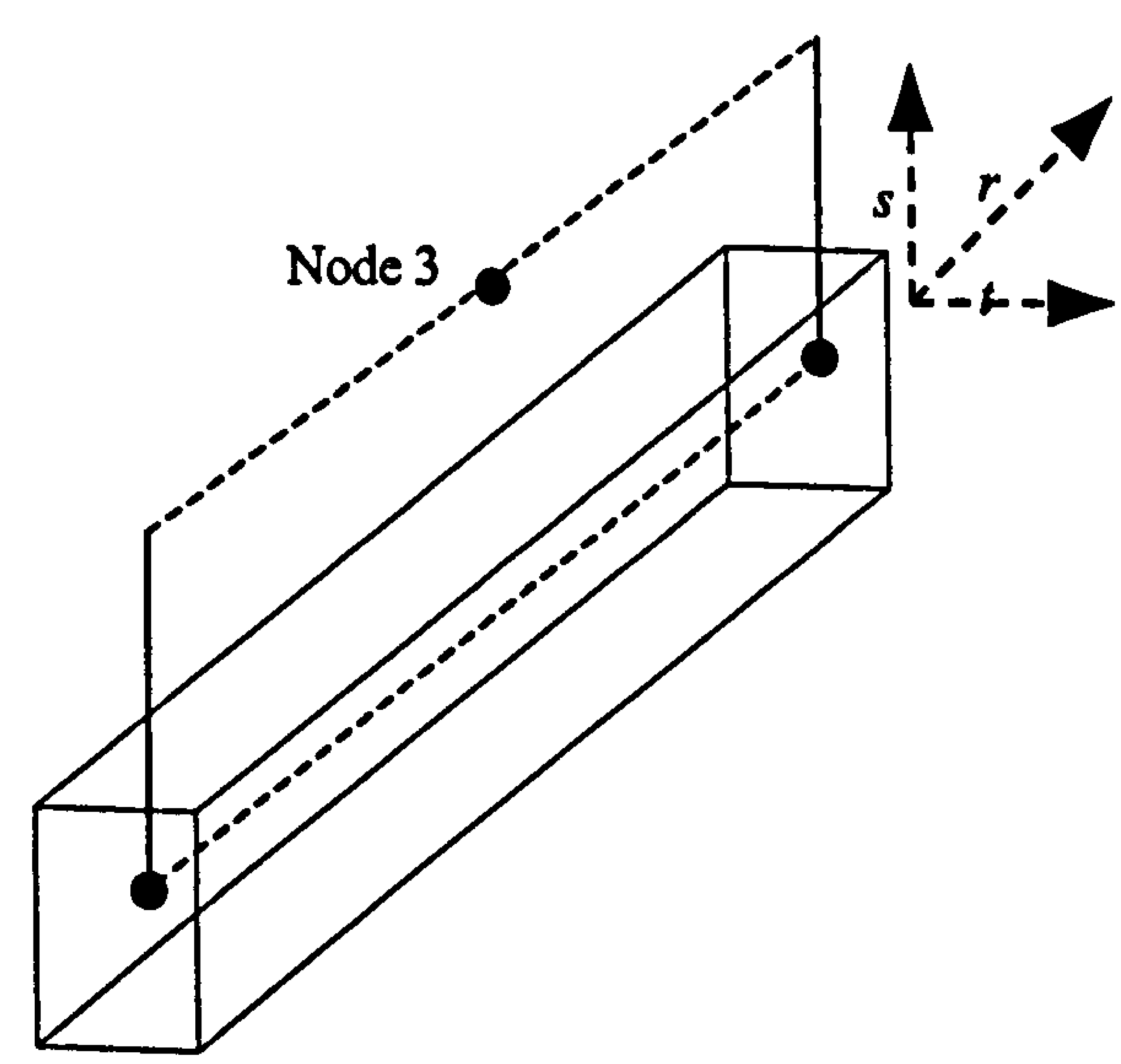


Figure 3-2 Detail beam element [LSTC, 1999]

3.2.1.1 Belytschko-Schwer beam element

The Belytschko-Schwer beam formulation employs a 'co-rotational technique' in the element to account for large rotation. This technique allows the BS beam to predict more accurate results compared to other beam elements. The co-rotational formulation of the BS beam uses two types of coordinate systems, one associated with each element (i.e. element coordinates which deform with the element), another is associated with each node (i.e. body coordinates embedded in the nodes) [Hallquist, 1998]. Details are shown in Figure 3-3.

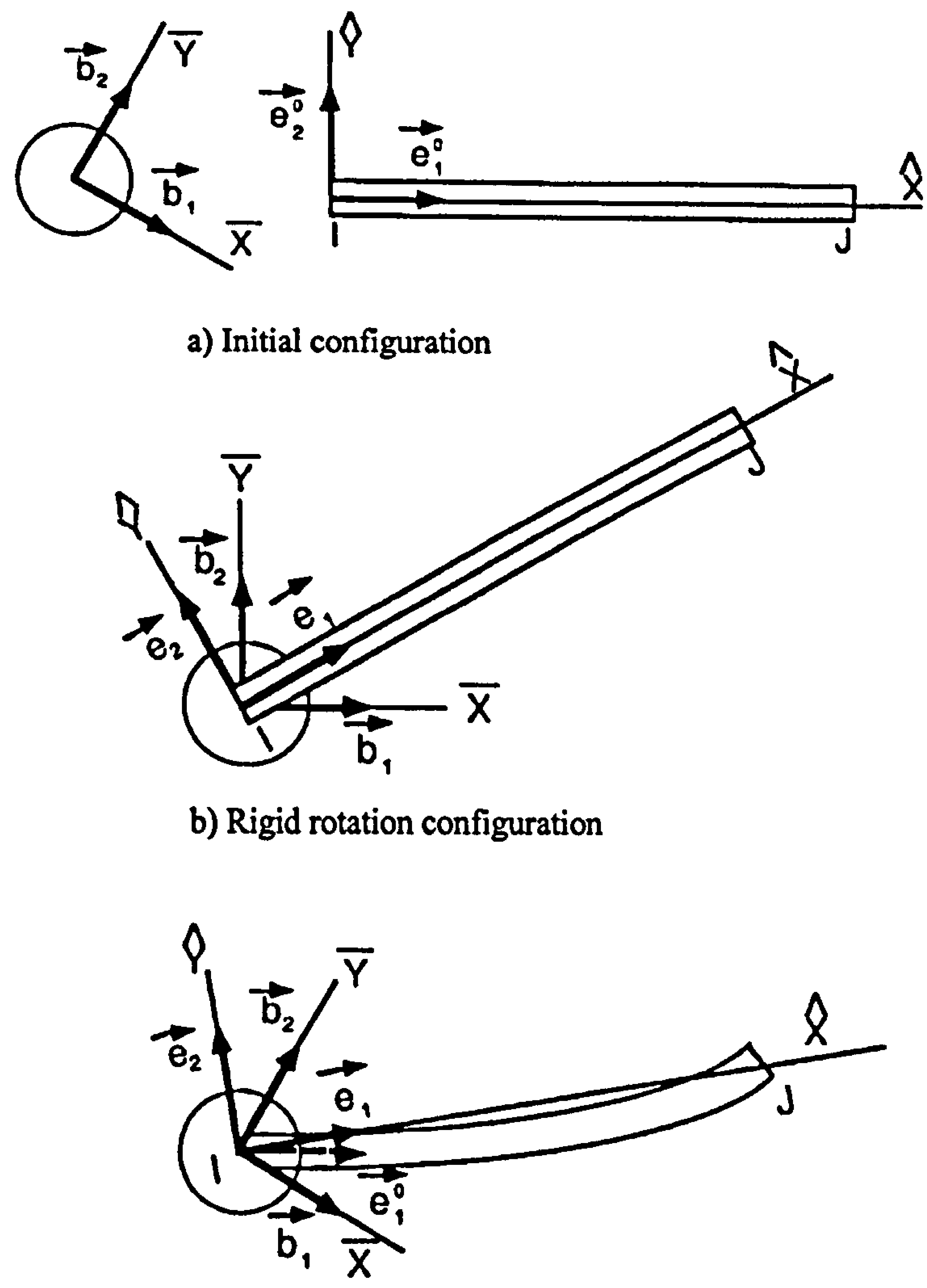


Figure 3-3 Co-rotational coordinate system of Belytschko-Schwer beam formulation [Hallquist, 1998]

When Belytschko-Schwer beam formulation is applied, the LS-DYNA user manual provides two ways for the user to define the beam properties. If the user defines the beam properties by second moment of area, I, then the resultant formulation is the BS by default. Or the user can model an arbitrary cross-section using a specified integration rule [LSTC, 2001; Oasys, 2001], one of which is the BS beam formulation.

A simple numerical test was conducted to discover the difference between the two applications of BS formulations (see Figure 3-4). It was decided to investigate the beam deflection after yielding, a uniform load of 250kN was applied quasi-statically (loading time 15second) to a S275 beam UB457 x 191 x 89.

The elastic-plastic material properties used were follows: Young's modulus of steel E=205,000N/mm^2, tangent modulus E_T=1000 N/mm^2 (E/200), Poisson's ratio of steel γ=0.3, density of steel ρ=7850 kg/m^3. The failure strain in the plastic stage was assumed to be 0.25.

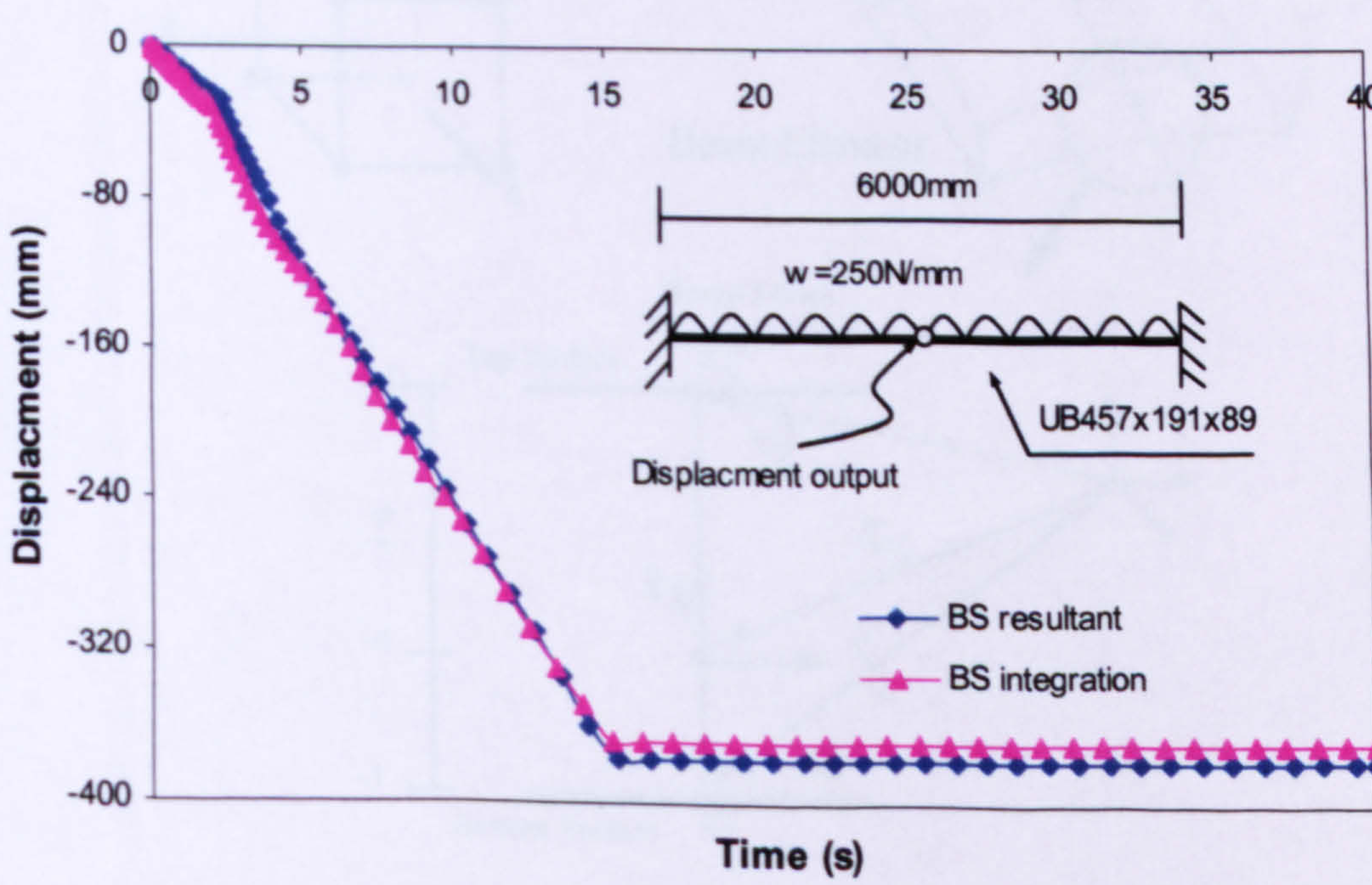


Figure 3-4 Comparison between different formulations of Belytschko-Schwer beam

Based on the analytical results, it was found that the two types of beam formulations predict similar results with a difference of 3%. It is likely that the integration formulation would predict a more accurate answer as it is based on a real cross section, but in terms of calculation time it is very expensive. The resultant formulation provides good results with less calculation time, so it was decided to use the resultant beam formulation for subsequent analyse.

3.2.1.2 Hughes-Liu beam element

This element has been formulated from a Hughes-Liu shell element [Hallquist, 1998]. Details are presented in Figure 3-5

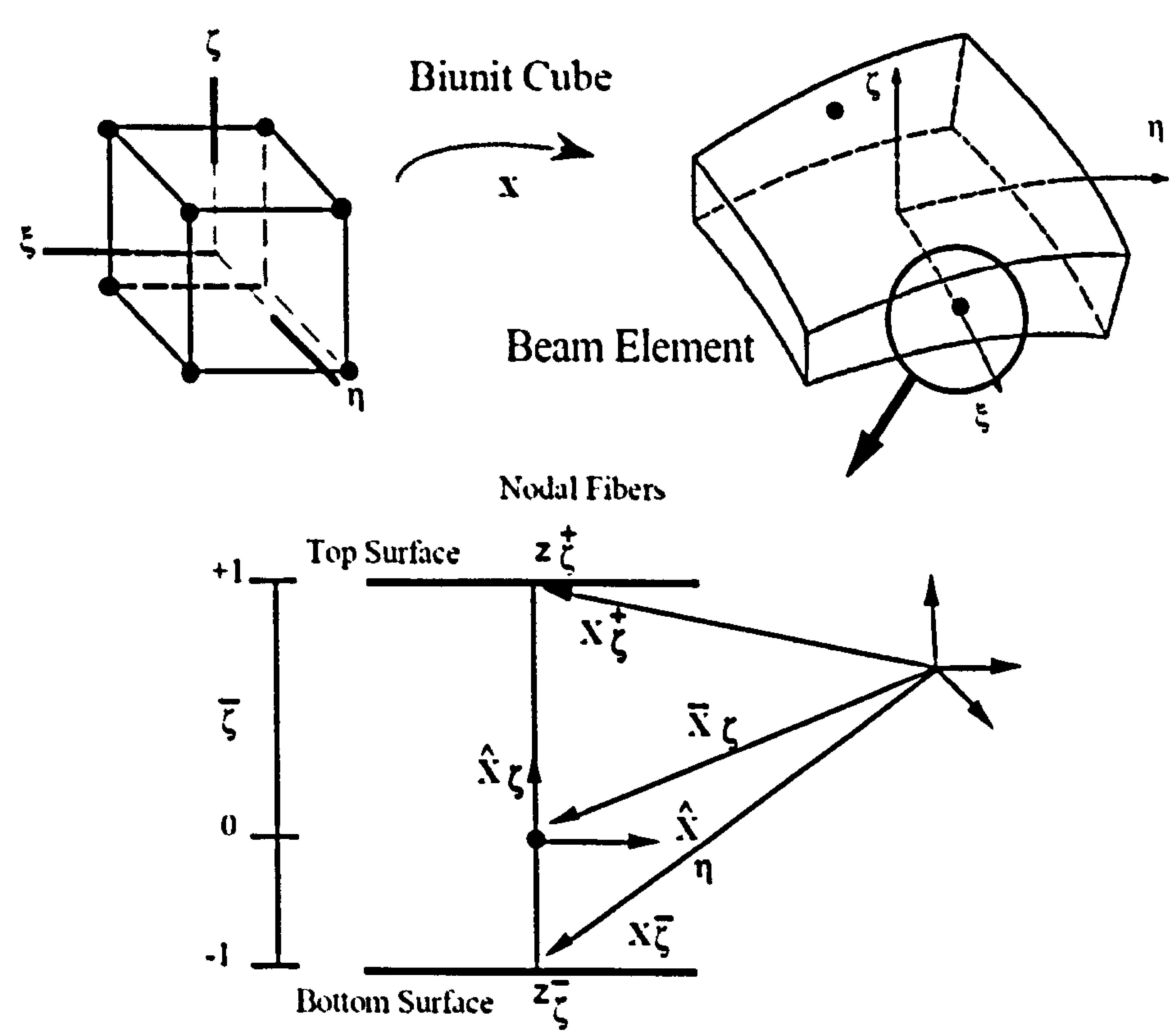


Figure 3-5 Details of Hughes-Liu beam element [Hallquist, 1998]

This HL integration beam is very efficient in terms of calculation, so this type of beam formulation was widely used for modelling the major load-bearing members (beams and columns) in the early stages of this research. However, it was later found that this formulation has difficulty predicting the large deflections associated with column buckling, as a result this type of formulation was restricted to the major load-bearing beams.

For example, consider the buckling resistance of a simply supported steel column UC305x305x118 (S275) using the Hughes-Liu beam formulation. A point load of 3300 kN is applied axially in 5 seconds to this perfectly straight column. (The material properties of the steel are the same as previously by stated.) The results of the analysis are presented in Figure 3-6.

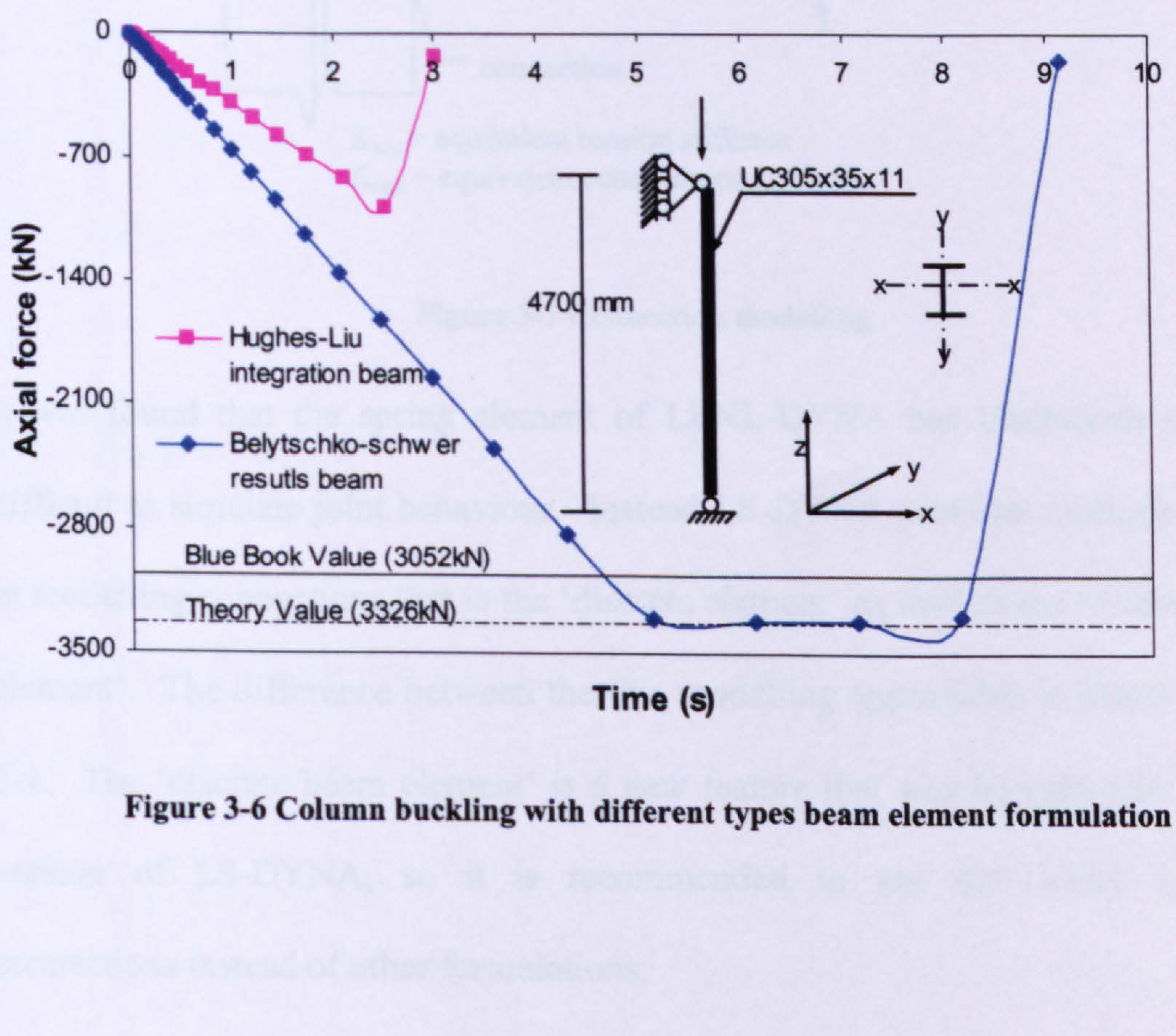


Figure 3-6 Column buckling with different types beam element formulation

Figure 3-6 shows clearly that the BS integration beams predict the buckling resistance more accurately, 3300kN compared to 1000kN for the HL integration beam which is more close to the theory value of 3326kN (see Appendix-A).

3.2.2 Discrete Element and Discrete Beam Element

At an earlier stage of the research, the connections between the beams and columns were modelled using spring elements in LLNL-DYNA [Lin, 1999] (see Figure 3-7).

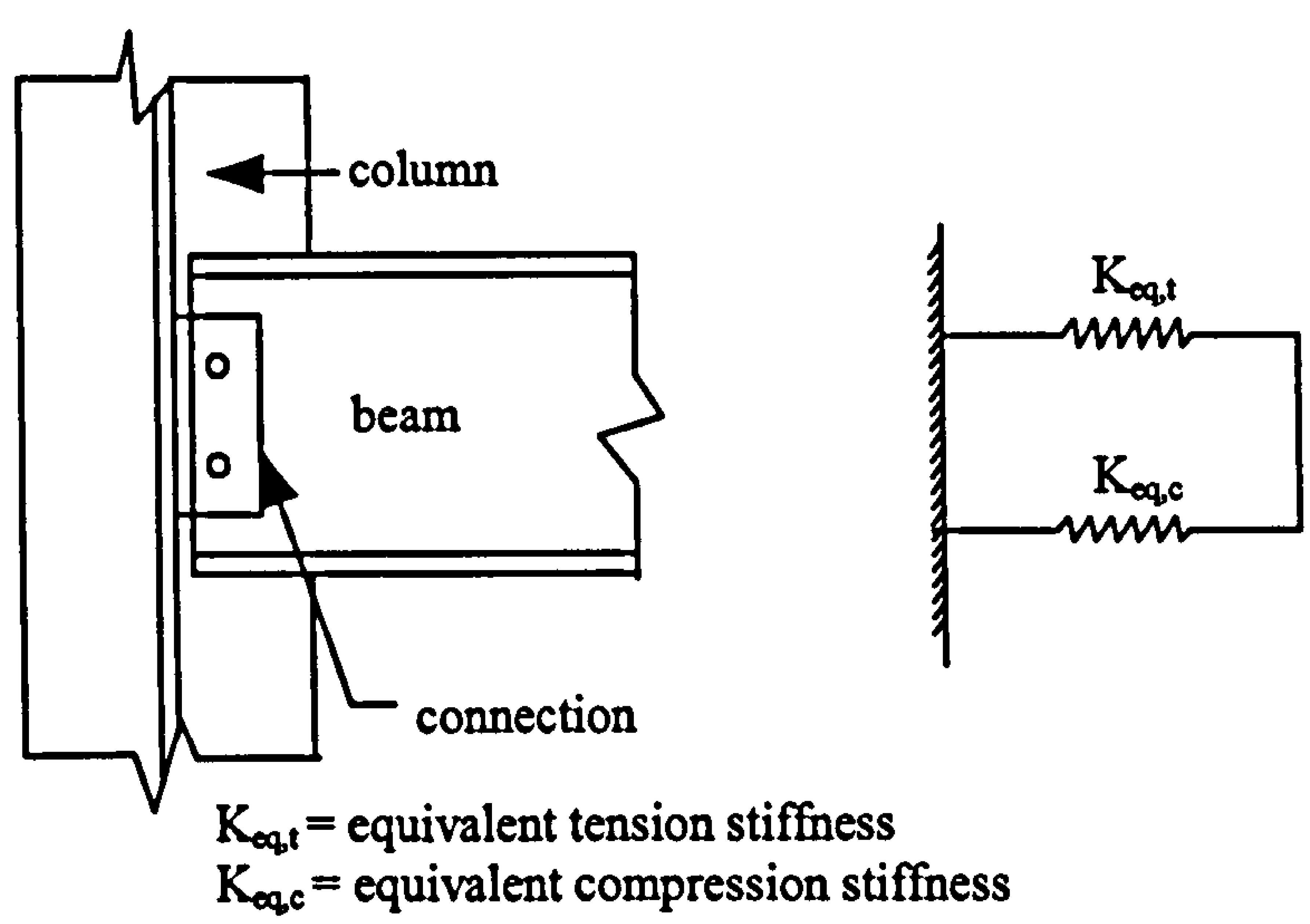


Figure 3-7 Connection modelling

It was found that the spring element of LLNL-DYNA has limitations and it is difficult to simulate joint behaviour. Instead LS-DYNA provides multiple choices in modelling connections that is the 'discrete element' as well as the 'discrete beam element'. The difference between the two modelling approaches is listed in Table 3-1. The 'discrete beam element' is a new feature that was brought into a recent version of LS-DYNA, so it is recommended to use this when modelling connections instead of other formulations.

Table 3-1Comparison of 'discrete element' and 'discrete beam element'

	Element type	Typical element	Typical Numerical Parameters	Time-step calculation
Discrete element	Discrete	Spring, damper	K[1]	Yes
Discrete beam element	Beam	Beam	TKR, TKS, TKT[2], RKR RKS, RKT	No

Clearly, the advantage of discrete beam elements is that they provide a wide choice of stiffness compared to a discrete element, and also they do not account for the time, which helps in reducing the period of computation. It was decided to use the discrete beam element to model the connection with different joint stiffness (pin, semi-rigid, rigid).

3.2.3 Shell element

The major task of this research is trying to identify the load path in a steel frame building without the extra redundancy provided by the composite slabs (see section 5.2). It is necessary to find a shell formulation that models the behaviour of a pre-

[1] Only one degree of freedom is connected

[2] Simulates the effects of linear elastic beam by using 6 springs and each acting about one of six local degree of freedom.

cast unit. Finally it was decided to use the Wilson 3 & 4 –node DSE quadrilateral shell [LSTC, 1999] to model the pre-cast units.

3.3 Numerical parameters

3.3.1 Mesh quality

In order to define a reasonable mesh range for a beam element, a set of numerical tests was conducted on a cantilever beam. Geometric details can be found in Figure 3-8.

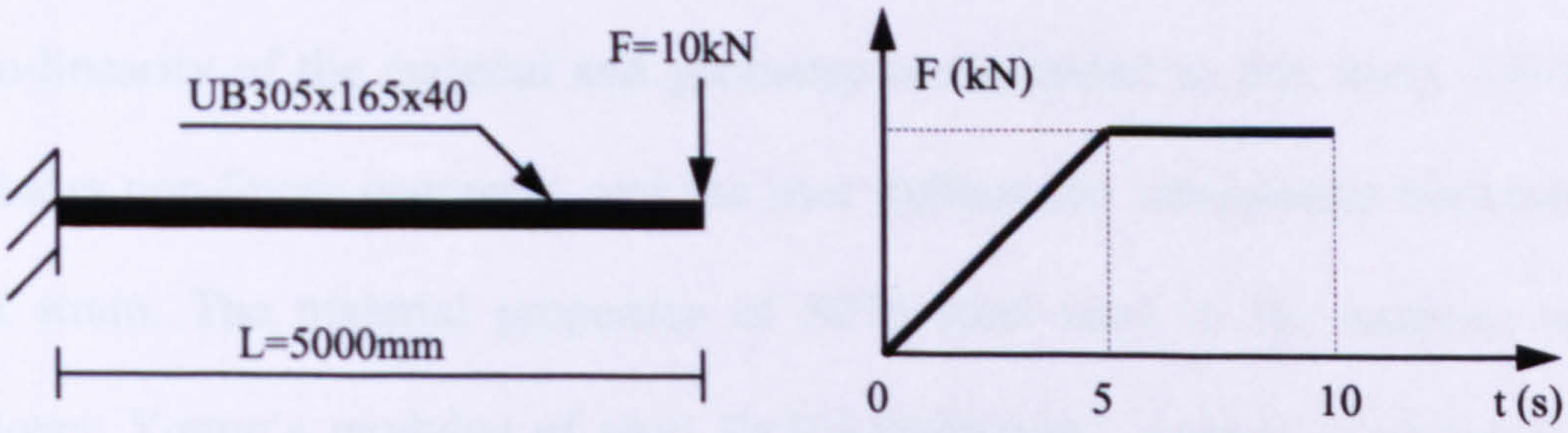


Figure 3-8 Numerical test of mesh quality

This beam has a quasi-static point load of 10kN, which is slowly applied (in 5 second) to the beam's major and minor axes. The following tests will examine the structural response (i.e. displacement) against the mesh numbers.

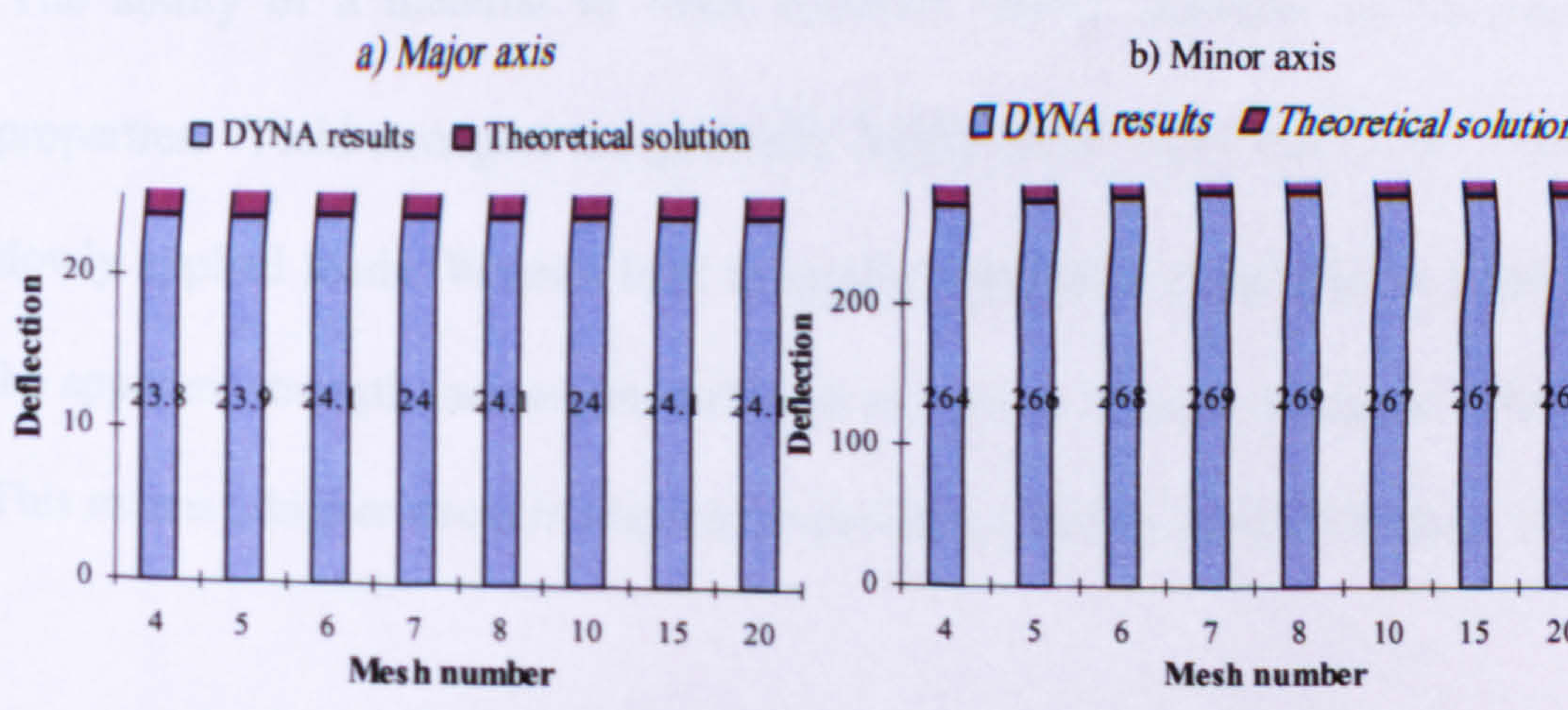


Figure 3-9 Results of mesh quality tests

Figure 3-9 shows that when the mesh number increases, the results are close to the theoretical solution. If the element has a fine mesh (that requires more computation time), then the results become more accurate. It is necessary to balance the relationship between accuracy and mesh density. It was found that when the mesh number was above certain number (i.e. 7), the results are similar. This phenomenon can be observed in both cases (major and minor). Finally, it was decided to use a mesh number of 10 for all the beam elements.

3.3.2 Material properties

Non-linearity of the material and geometry are included in this study. LS-DYNA includes non-linear geometry, and the user defines the relationship between stress and strain. The material properties of S275 steel used in the analysis were as follows: Young's modulus of steel E=205,000N/mm^2, tangent modulus E_T=1000 N/mm^2 (E/200), Poisson's ratio of steel γ=0.3, density of steel ρ=7850 kg/m^3. The failure strain in the plastic stage was assumed to be 0.25.

The ability of a material to resist dynamic failure depends on its mechanical properties. Yield strengths are generally higher under rapid stain rates than under slowly applied loads. When a load is rapidly applied to a material, a large part of the apparent strength increase is attributed to a lesser amount of plastic deformation. This means a higher stress is required to produce a failing strain [Yandzio, 1999].

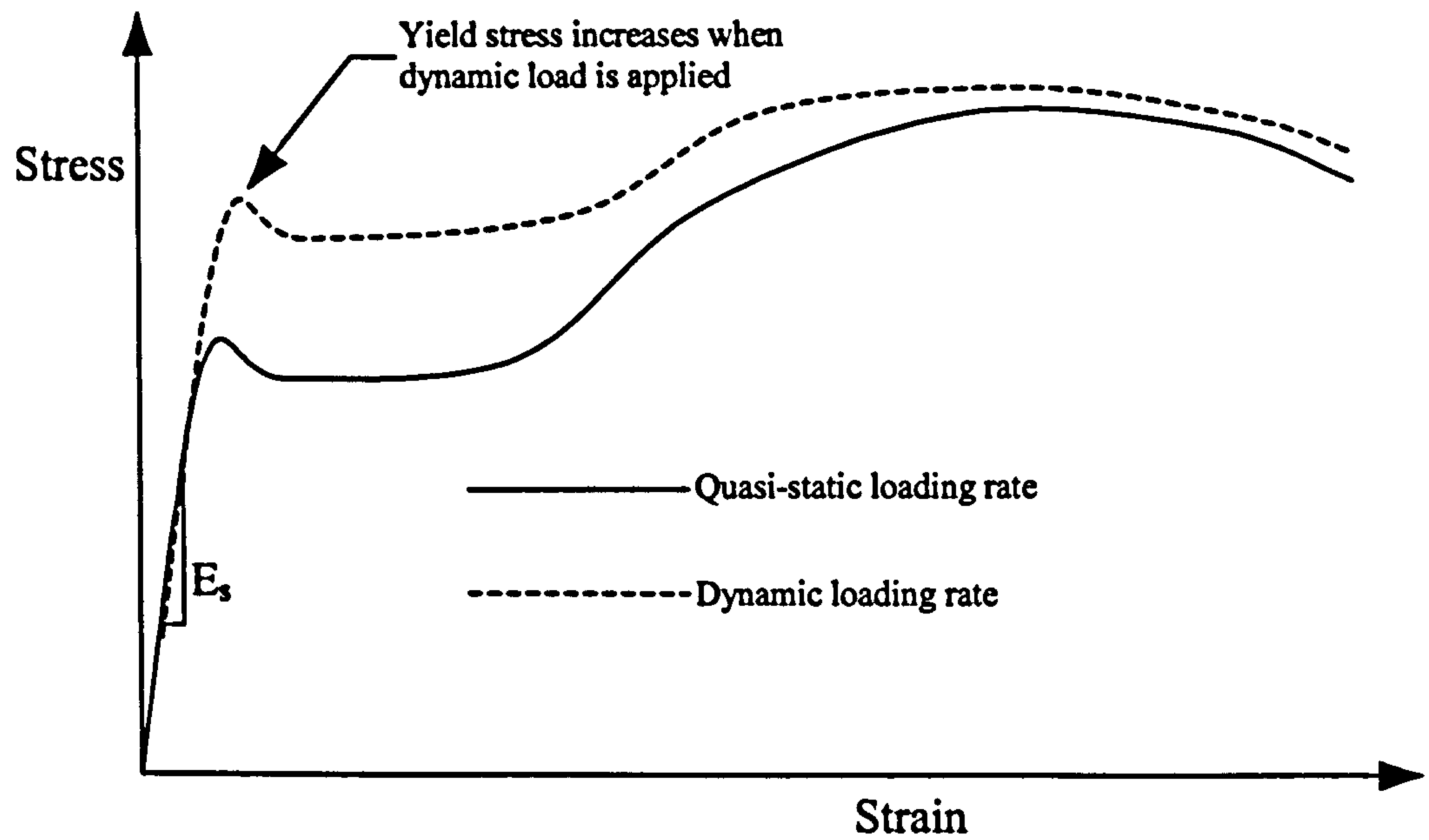


Figure 3-10 Typical stress-strain curves for structural steel [Yandzio,1999]

Figure 3-10 shows the mechanical properties of a commonly used low carbon structural steel (S275). It is apparent that the yield and ultimate stress is affected by the rate at which straining takes place.

The ratio of dynamic stress to static stress (yield or ultimate) is termed Dynamic Increase Factor (DIF).

$$DIF = \frac{\sigma_{dyn}}{\sigma_y} \tag{3.1}$$

Where, σ_{dyn} is the dynamic yield stress corresponding to a particular strain rate and σ_y is the yield stress under static load. If a dynamic load is applied over a period greater than 1 second, there will not be any increase in yield or ultimate stress, which means DIF equals 1; otherwise (e.g. 100ms, 10ms or 1 ms) a factor 1.05 of DIF should be applied to S275 and S355 steel [Yandzio, 1999]. It needs to be noticed that these factors are based on the time it takes to reach the yield stress. The material mill tests carried out at different rates of loading [Byfield, 1997] showed an average 4% difference, accordingly the dynamic enhancement of the material is not included in this study.

3.3.3 Translational and Rotational Stiffness

The connection is one of the key components in the structure, and usually the connections are assumed to act either as pins or as fully fixed, whilst in a real structure beam-to-column joints exhibit some flexibility and moment resistance. Therefore, the joints in reality are likely to behave as semi-rigid joints.

A discrete beam element has 6 numerical parameters [LSTC, 1999], which include three translation stiffnesses (TKR, TKS, and TKT), and three rotation stiffnesses (RKR, RKS, RKT). These represent the six degrees of freedom for a node. A pin joint should just be capable of transferring the force across the connection and therefore the joint should have large translation stiffness and very small rotation stiffness. Conversely a rigid joint has the ability to transfer the force as well as the moment across the connection, therefore the rigid joint stiffness of translation and rotation are relatively very large. Thus a semi-rigid joint should exhibit characteristics that lie somewhere between pin and rigid joints.

It is very important to define the numerical parameters correctly to get satisfactory results. For instance, it was found that the use of zero or numbers larger than 1×10^{14} should be avoided in defining the joint stiffness (translation /rotational) as this can cause erroneous results (i.e. numerical divergence). Therefore, it was decided to investigate the upper-limit of the translation stiffness TKR, TKS and TKT. This was done using a simple beam model as shown in Figure 3-11.

In order to find out the appropriate numerical range of the translation stiffness TKR, TKS and TKT, the rotation stiffness RKR, RKS and RKT can be taken as small values that themselves produce negligible effects on the overall structure. For this case, say values of 1 Nmm/rad. The studies were carried out using different cross-sections of beams. The results are shown in the Figure 3-11.

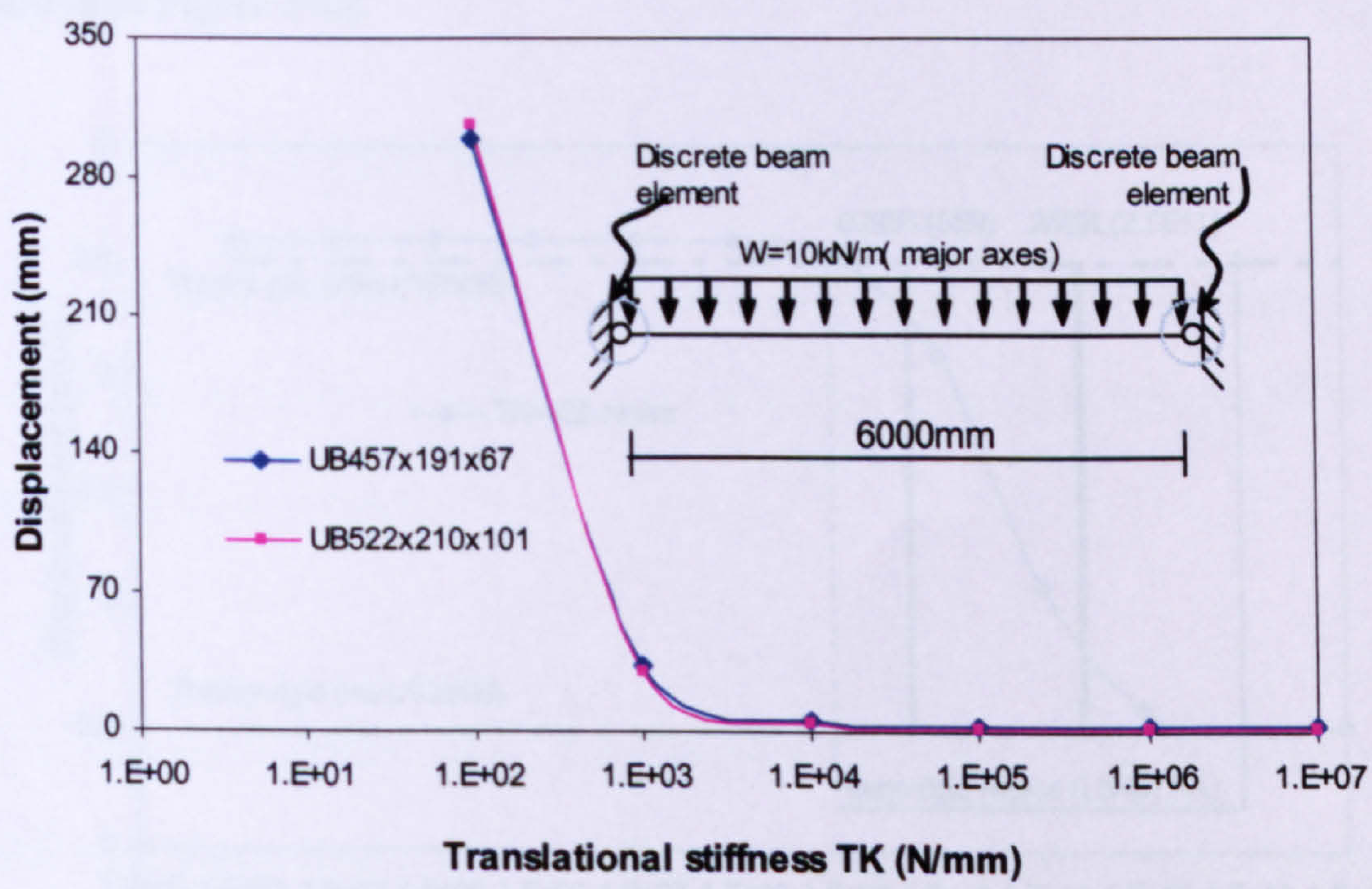


Figure 3-11 The effects of translational stiffness on maximum vertical displacement at loading level of 10kN/m

Although the study was carried out with two different cross-sections, they both exhibited similar behaviour. This common trend, was that as the translation stiffness increased the supports behaved more like a hinge, and when the translation decreased the supports behaved more like a roller. According to connection tests down by Owens [Owens and Moore, 1992], the maximum translational stiffness is about $4x10^4$ N/mm (web cleat). In conclusion a value of $1x10^5$N/mm for the upper-limit of the translation stiffness represents a really stiff joint behaviour.

In order to determine the most appropriate range of rotation stiffness values for use with semi-rigid analyses a universal beam was examined. A UB 457x171x67 was given the translation stiffness of $1x10^5$N/mm and the rotation stiffness was varied between $1x10^3$Nmm/rad and $1x10^{14}$Nmm/rad. The results of numerical tests were showed in Figure 3-12.

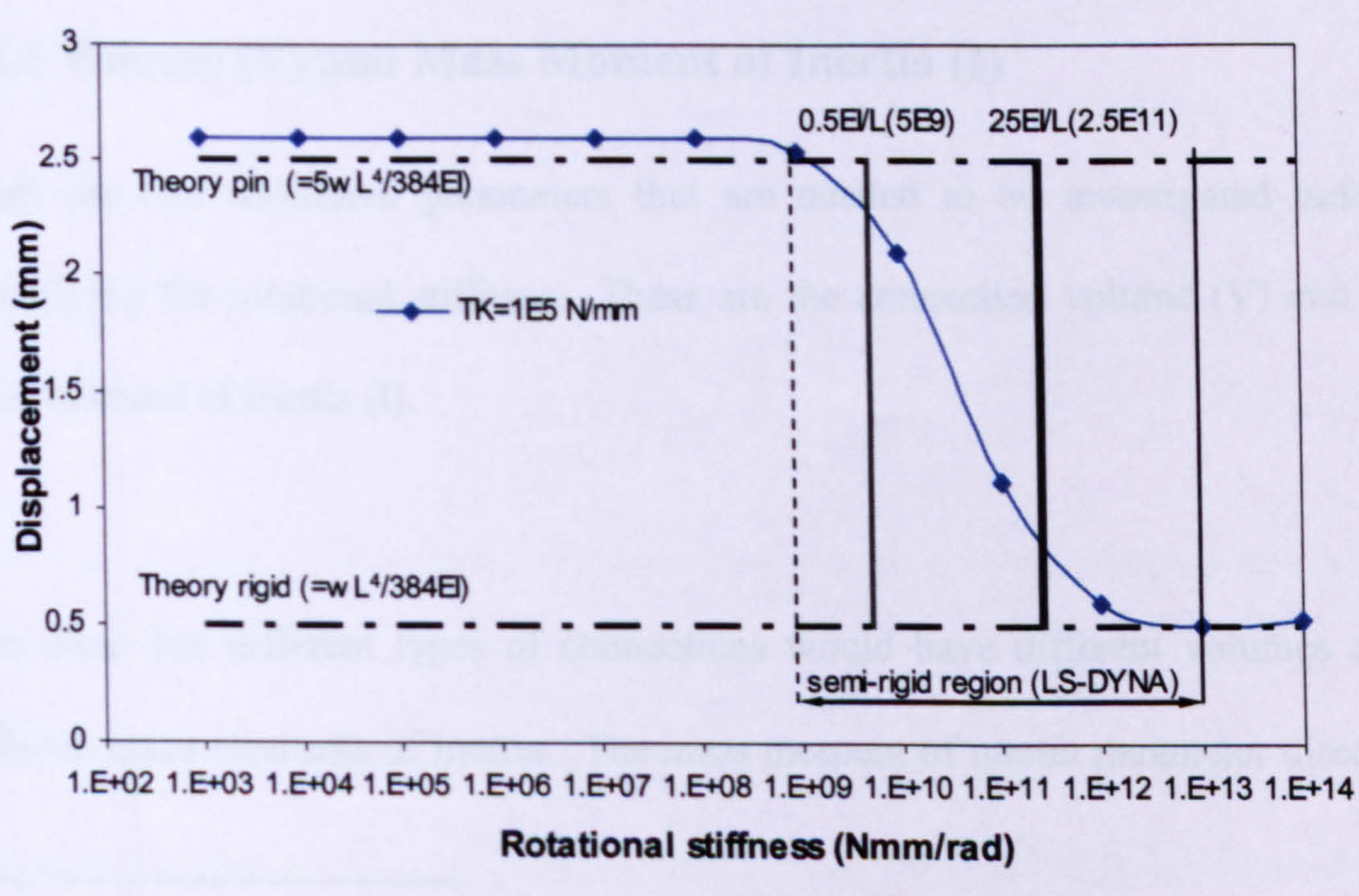


Figure 3-12 Illustration of Rotational stiffness of UB457x191x67 at loading ratio of 10kN/m

As shown in the figure above, the expected semi-rigid zone of rotation stiffness should lie in the gap between 0.5EI/L and 25EI/L♣ [BSI, 2005], while predictions from LS-DYNA were largely over this range.

It is observed that when a rotational stiffness of $1x10^9$Nmm/rad is applied, the joint exhibited the behaviour as a pin. It was also found that under this rotational stiffness (<$1x10^9$nmm/rad), a deflection of 2.6 mm was gained with an error rate of 4% compared to the theoretical value of 2.5mm. Therefore, it can concluded that $1x10^9$Nmm/rad is a critical value i.e. as rotation stiffness becomes less than $1x10^9$Nmm/rad the structure would behave like pin, and as rotation stiffness got bigger than $1x10^9$Nmm/rad the structure would start exhibiting semi-rigid characteristics.

3.3.4 Volume (V) and Mass Moment of Inertia (I)

There are two additional parameters that are needed to be investigated before quantifying the rotational stiffness. These are the connection volume (V) and its mass moment of inertia (I).

It is clear that different types of connections would have different volumes and different mass moments of inertia. The mass moment of inertia parameter directly

♣Pin < 0.5EI/L;
Rigid > 8EI/L for braced frame;
25EI/L for unbraced frame.

affects the rotation ability across the beam-to-column connection, therefore, it becomes a very important part of the connection.

This study followed the previous beam example (see Figure 3-11). The numerical tests were carried out with different combinations of V and I based upon a realistic value such as a pin connection ($V=2.5 \times 10^5 mm^3$, $I=10mm^4$) and a rigid connection ($V=2.5 \times 10^6$ mm^3, $I=100mm^4$), up to extreme values of $V=0.01mm^3$ and $I=1$ mm^4. In this example a specified translation stiffness of 1×10^5N/mm was used and a rotation stiffness 1×10^{10}Nmm/rad. Results are shown in Figure 3-13.

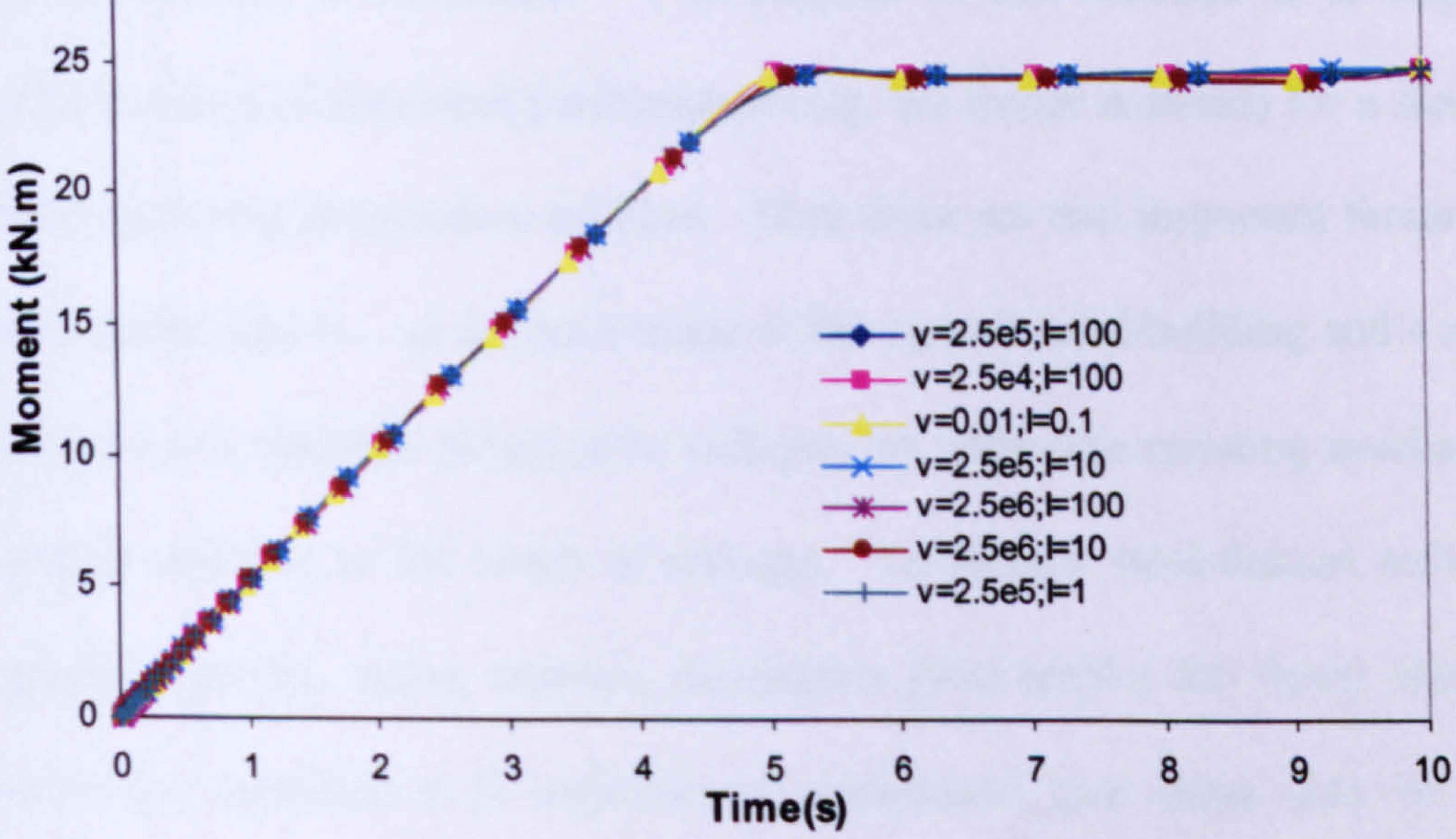


Figure 3-13 Comparison between the different combination of volume and mass moment inertia

The different combinations show the same results for each case, but the computation time varied significantly between 10 minutes and 120 minutes. It can be concluded that the two parameters (being volume and moment of inertia) just affect the computation time.

This study proves that when determining the parameters of volume and moment of inertia, it is possible to ignore their real physical values, and as such the one which has the most effective computing time can be chosen.

3.4 Conclusion

The FEM is a very useful and extremely powerful analytical tool for solving structural problems, but in order to use the FEM properly a good understanding of the formulation is necessary. The purpose of this research is to improve the understanding of structural performance (e.g. the forces induced) for a steel-framed building during progressive collapse. Thus there are two important factors need to be included that is – an accurate mode of the steel-framed building and a modelling procedure to simulate progressive collapse, or rather the resisting mechanism that prevent collapse in the event of damage. To form a steel-framed building in a computer model, beam, column, connection (and maybe the floor) elements are necessary, therefore it is important to understand how these may be modelled accurately. This chapter presented initial studies of these basic structural components and the analysis from the LS-DYNA presented a reasonable result close to the theoretical value for a simple column model (see 3.2.1.2).

Based on these initial studies of the structural components, a further study of how to model a damaged frame (i.e. progressive collapse) and a 3D steel-framed building will be presented in chapter 4.

Chapter 4

Finite Element Method: Modelling Strategy and Application

4.1 Introduction

This chapter describes a number of preliminary studies conducted to develop a modelling strategy. In chapter 3, studies were carried out with individual elements instead of a combination of elements forming a structural frame. In this chapter, a small-scale 3D steel building is presented, that consists of some basic structural member (i.e. beams or columns). An investigation into the dynamic response mechanisms developed in this building following removal of a column removal is initially presented. In general, the layout of the chapter is:

1) Introduction to the modelling strategy adapted to study a frame damaged by accidental loading.

2) Examination of the failure mechanism of a small steel framed building due to column removal.

4.2 Modelling the damaged structure

The dynamic response of a structure is always hard to predict [Clough, 1975; GSA, 2002; MMC, 2002]. However, intuitively, we expect two possible responses, a new equilibrium position or collapse. When a dynamic load (e.g. earthquake, explosion) is applied to a structure, the (possibly damaged) structure attempts to find new equilibrium positions. If the remaining structure cannot find new equilibrium positions, it will collapse.

This research focuses on the overall behaviour of a structure after an initial static equilibrium has been disrupted. In particular, the research aims to investigate whether during progressive collapse the damaged structure can maintain equilibrium (stand up) or not. If it stands up, what is the resisting mechanism; otherwise what is the failure mechanism.

As discussed earlier, most design rules [BSI, 1990; BSI, 2000; CEN 2002; ASCE, 2002] against progressive collapse are based on a static approach. Following the events at the WTC, it has been suggested [Corley, 2004; Marjanishvili, 2004] that since progressive collapse is a dynamic problem this approach is questionable. Progressive collapse is normally caused by accidental load, explosion, blast or impact. The current studies did not attempt to model the loads that cause the damage, instead it is more interested in what happens *after* the damage has taken place.

The modelling philosophy adopted for this study was thus to simulate the forces arising in a damaged structure by removal of a vertical support from a pre-loaded structure. Obviously, this modelling method is not as realistic as to simulate a pre-loaded structure with sudden column removal due to accidental load (i.e. blast). This method is too complicated to use and a sophisticated modelling technique (i.e. springback [Hallquist, 1999]) has to be used for this purpose. On the other hand, the benefit of modelling damage by applying a reduced force varying with time is easy to use, and also it provides the user opportunity to control time.

The modelling procedure used in this research involves two parts: i) analysis of the complete pre-loaded structure in order to determine the force in the column to be removed and ii) analysis of a structure with a support which represents the column removed over an increment of time (which was varied). A typical force-time history for the member removed is shown in Figure 4-1. Simulating the dynamic behaviour was simply achieved by varying the removal time (T), from 1ms up to 1 second.

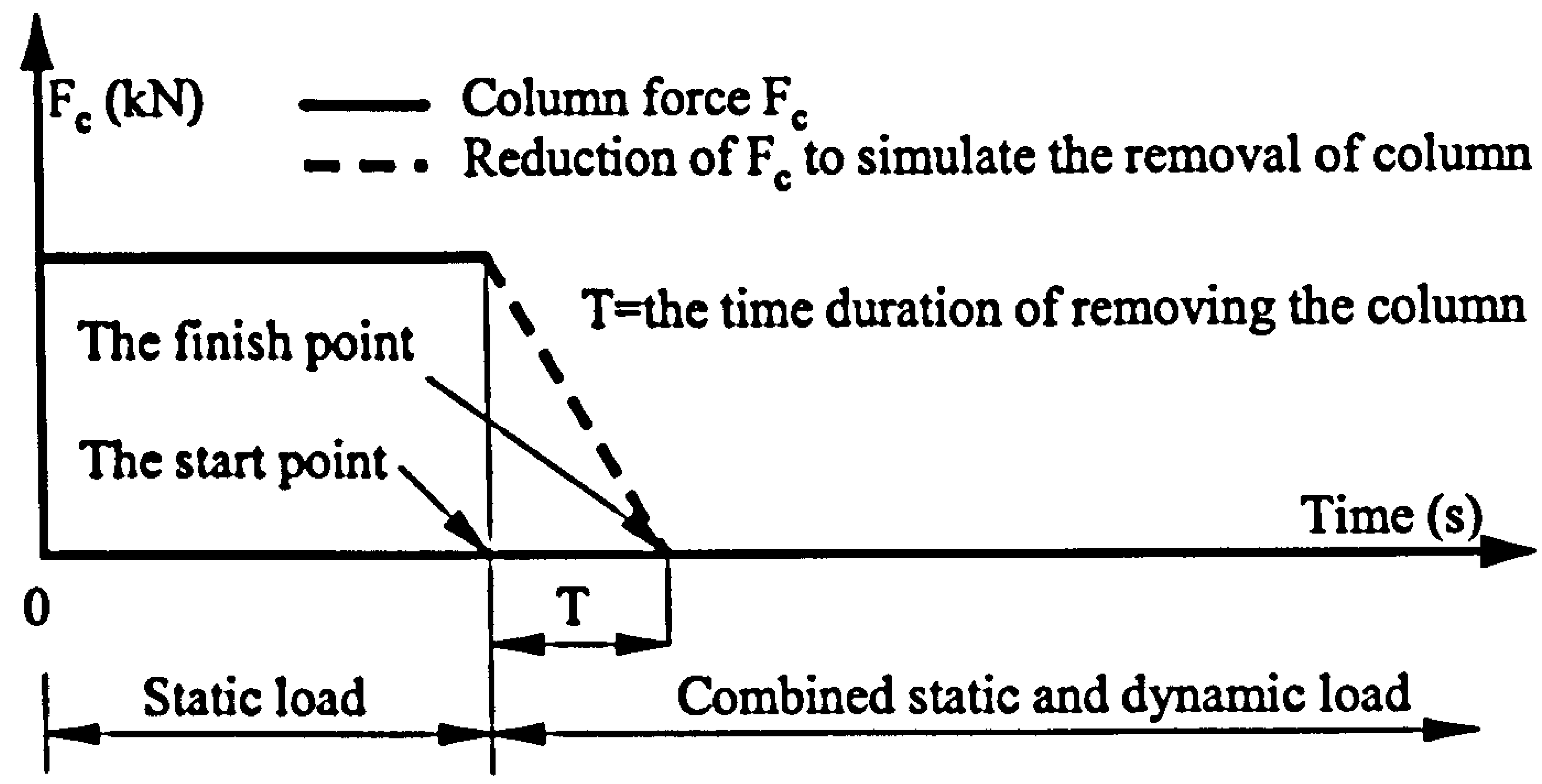


Figure 4-1Modelling procedure for progressive collapse

4.3 Modelling a Small 3D-Frame

4.3.1 Introduction

As an introduction to the study of a large frame (see chapter 5), a small 2 bay by 2 bay by 3-storey building was examined. The initial member sizes were chosen based on a static structural design in accordance with BS5950. A feature of this static analysis was that the chosen member sizes were inadequate to prevent the building from swaying and therefore they needed to be increased to enhance the lateral stiffness. It was intended that this small structure be braced (or have sufficient frame rigidity) without the need to model external horizontal restraints. In time it became clear that a small structure was too unrealistic to provide much of a detailed understating of real frame behaviour. Even so, it is worth considering the results of the isolated small structure as some useful results were found (see chapter 5).

The Hughes-Liu integrated beam element was used for modelling the steel beams and columns in the subsequent dynamic analysis. Rigid beam-column connections were used to provide the lateral stiffness and control the sway. It is acknowledged that the cross section of column is unrealistic in this small building, but it seems the heavy column is the only way to solve the sway problem without the bracing system. The member sizes (see Table 4-1) were chosen based on a structural design in accordance with BS5950:2000 [BSI, 2000]. The primary beams (B1, B2) are along the x direction, and tie beams (B3, B4) are located along the y axis. The cladding is not included in this study. The geometry details are shown in Figure 4-2

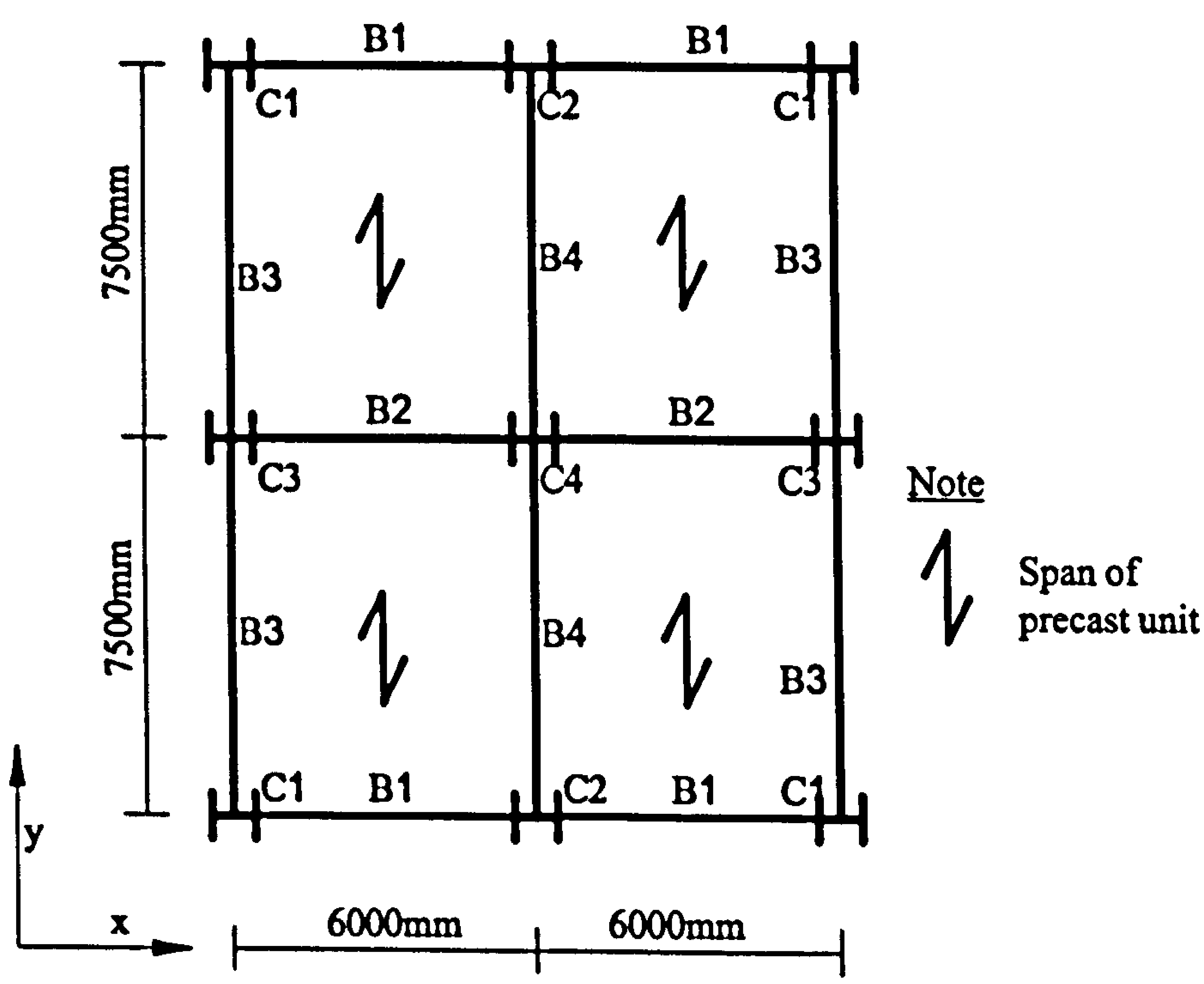


Figure 4-2 Structural plan of small building

Table 4-1 Member Sections for 3D small scale steel Frame

Beams		Columns	
B1	UB457x191x82	C1	UC305x305x240
B2	UB533x210x92	C2	Ditto
B3*	450x110	C3	Ditto
B4	Ditto	C4	UC305X305X137

Note Beams are non-composite, and the pre-cast units are not included in modelling.

This small scale 3D frame is a pre-study for the large scale building (described in chapter 5 and chapter 6). Therefore, it was decided to keep this 3D model as simple as possible in order to get a better understanding of the structural response during progressive collapse. It was also possible to compare these results with those from a linear elastic analysis programme.

* *B3/B4 is a user defined cross section in order to stop beam buckling laterally.*

	D(mm)	B(mm)	t_f(mm)	t_w(mm)	I_{yy}(cm^4)
*B3/B4	450	110	10	4	1330

4.3.2 Load Bearing Capacity of the Damaged Frame

The study in this section is an investigation of structural behaviour when one column, in this case column C3, was removed. Although, the design level loading of $1.4g_k+1.6q_k$ is called the ultimate loading capacity, it is unlikely that this loading level would cause the building to collapse [Byfield and Nethercot, 1998]. Therefore, it is necessary to find out the collapse loading level of the undamaged structure. Analyses showed that the collapse loading level of this building was 98kN/m ($=1.4g_k+1.75q_k$). Numerical tests were conducted based on removing column C3 in one second, and the LS-DYNA results are presented in Figure 4-3.

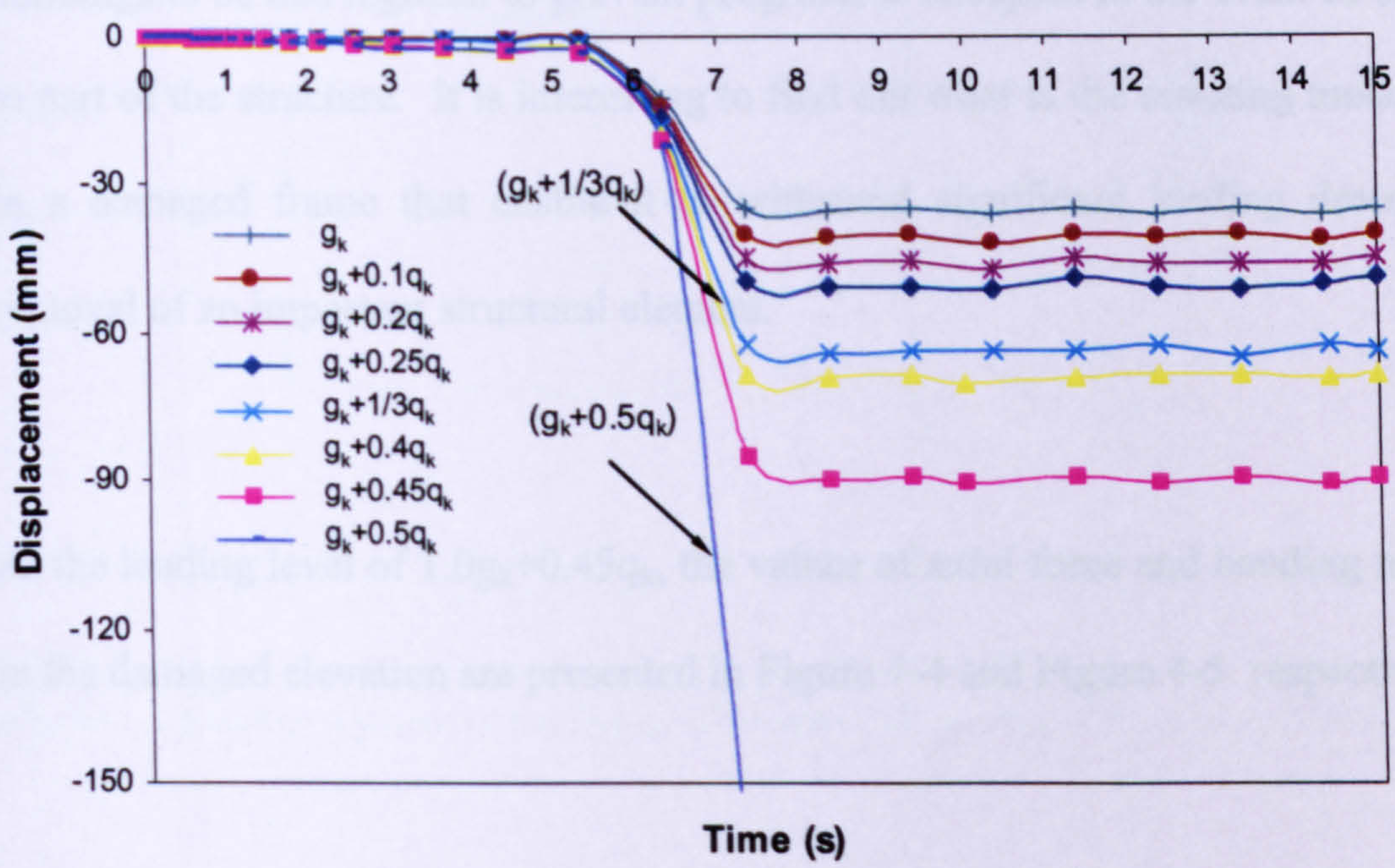


Figure 4-3Load bearing capacity of damaged frame when column C3 was removed in 1 second

Figure 4-3 shows that when one column (C3) was removed in one second, the damaged frame can take a loading level of $1.0g_k+0.45q_k$ without collapse. LS-DYNA predicts a maximum displacement of 90mm under this loading. When the loading level increased to $1.0g_k+0.5q_k$, the damaged frame collapsed. The current design guidelines [BSI, 2000] recommend a load level of $\gamma_f^{\clubsuit}$ $(1.0g_k+0.33q_k)$ when considering the response of frames to member notional removal. The final results from dynamic LS-DYNA analysis indicate that the damaged frame under consideration *can* take the loading level of γ_f $(1.0g_k+0.33q_k)$ when the column is removed over a time of 1s (But it should be remembered that the frame has same inherent overcapacity in that it is capable of resisting $1.4g_k+1.75q_k$ in the undamaged state).

The recent UK building regulations [HMSO, 2004] require certain categories of buildings to be tied together to prevent progressive collapses in the event of damage to part of the structure. It is interesting to find out what is the resisting mechanism in a damaged frame that enable it to withstand significant loading despite the removal of an important structural element.

At the loading level of $1.0g_k+0.45q_k$, the values of axial force and bending moment in the damaged elevation are presented in Figure 4-4 and Figure 4-5 respectively.

$^{\clubsuit}$ γ_f=1.05

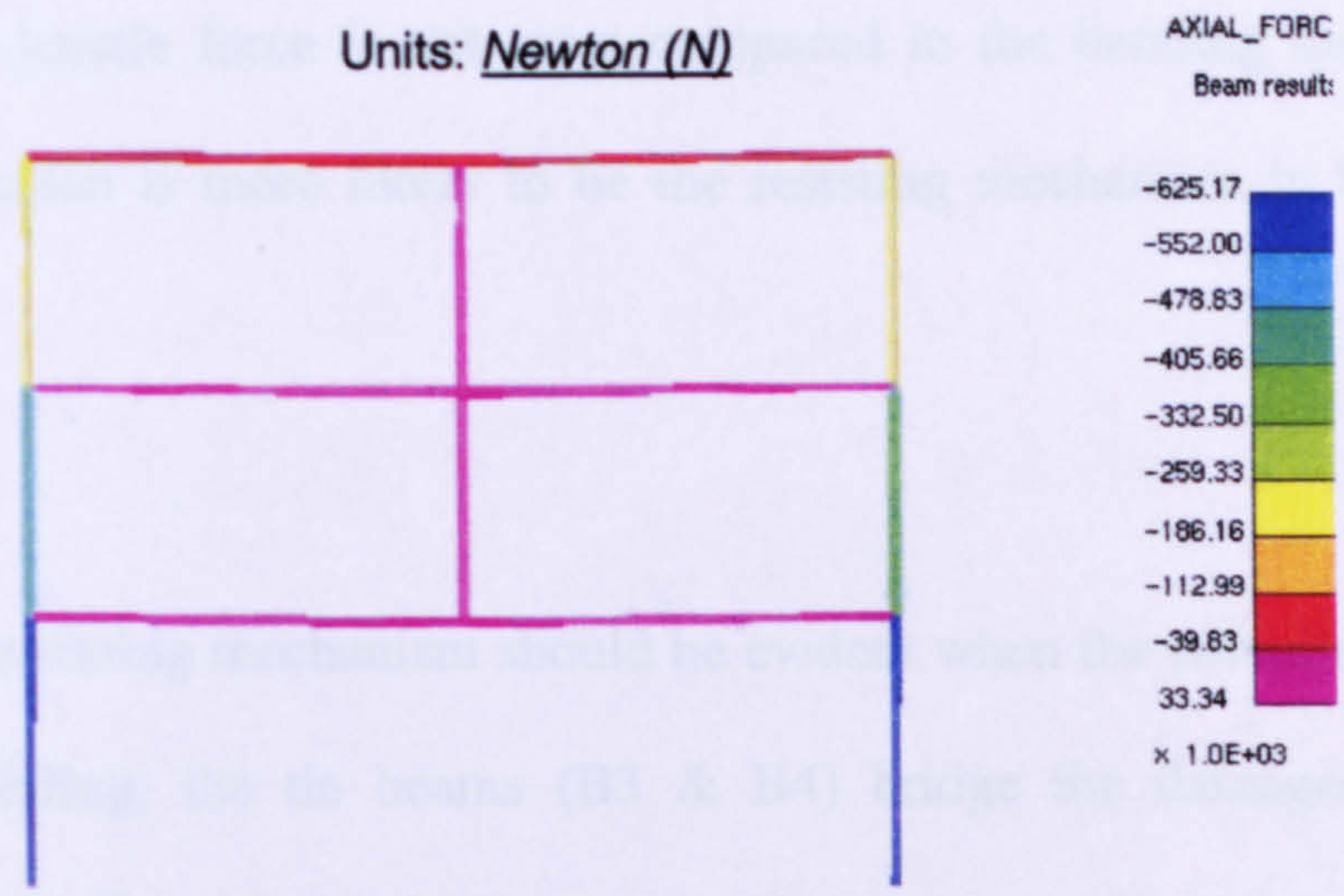


Figure 4-4Axial force in the damaged elevation when column C3 is removed in 1second

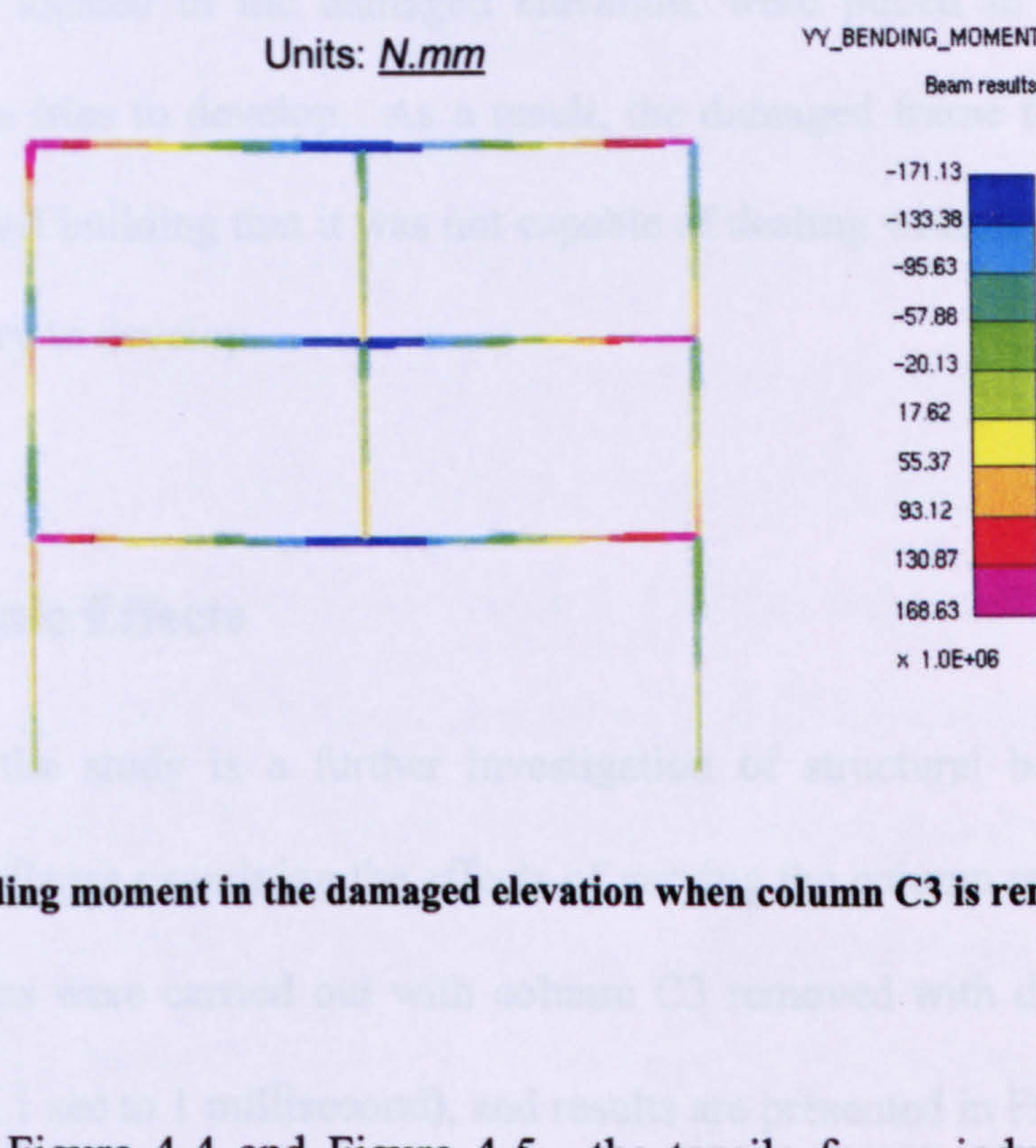


Figure 4-5 Bending moment in the damaged elevation when column C3 is removed in 1second.

According to Figure 4-4 and Figure 4-5 , the tensile forces induced appear less significant than the bending moment, suggesting that the bending resistance of the damaged elevation is more important than the tying action. Catenary action is suggested by BS5950 to be the resisting mechanism, but in this small 3D steel frame, all the members have rigid connections, which restrict large rotations, therefore, the tensile force is not great compared to the bending moment and so Virerendeel action is more likely to be the resisting mechanism in this particular case.

The possible resisting mechanism should be evident when the column was removed from this building; the tie beams (B3 & B4) bridge the damaged area to the undamaged parts through the rigid beam-to-column connections. In order to form a new equilibrium, it requires the damaged frame to either develop catenary action or Vierendeel action. In this case, as soon as the column was removed, the corner columns (C1) located in the damaged elevation, were pulled in by the ties, as catenary action tries to develop. As a result, the damaged frame failed because it was such a small building that it was not capable of dealing with the high horizontal forces which try to develop.

4.3.3 Dynamic Effects

This part of the study is a further investigation of structural behaviour during progressive collapse examining the effects of varying the column removal time (T). Numerical tests were carried out with column C3 removed with different removal times (T from 1 sec to 1 millisecond), and results are presented in Figure 4-6.

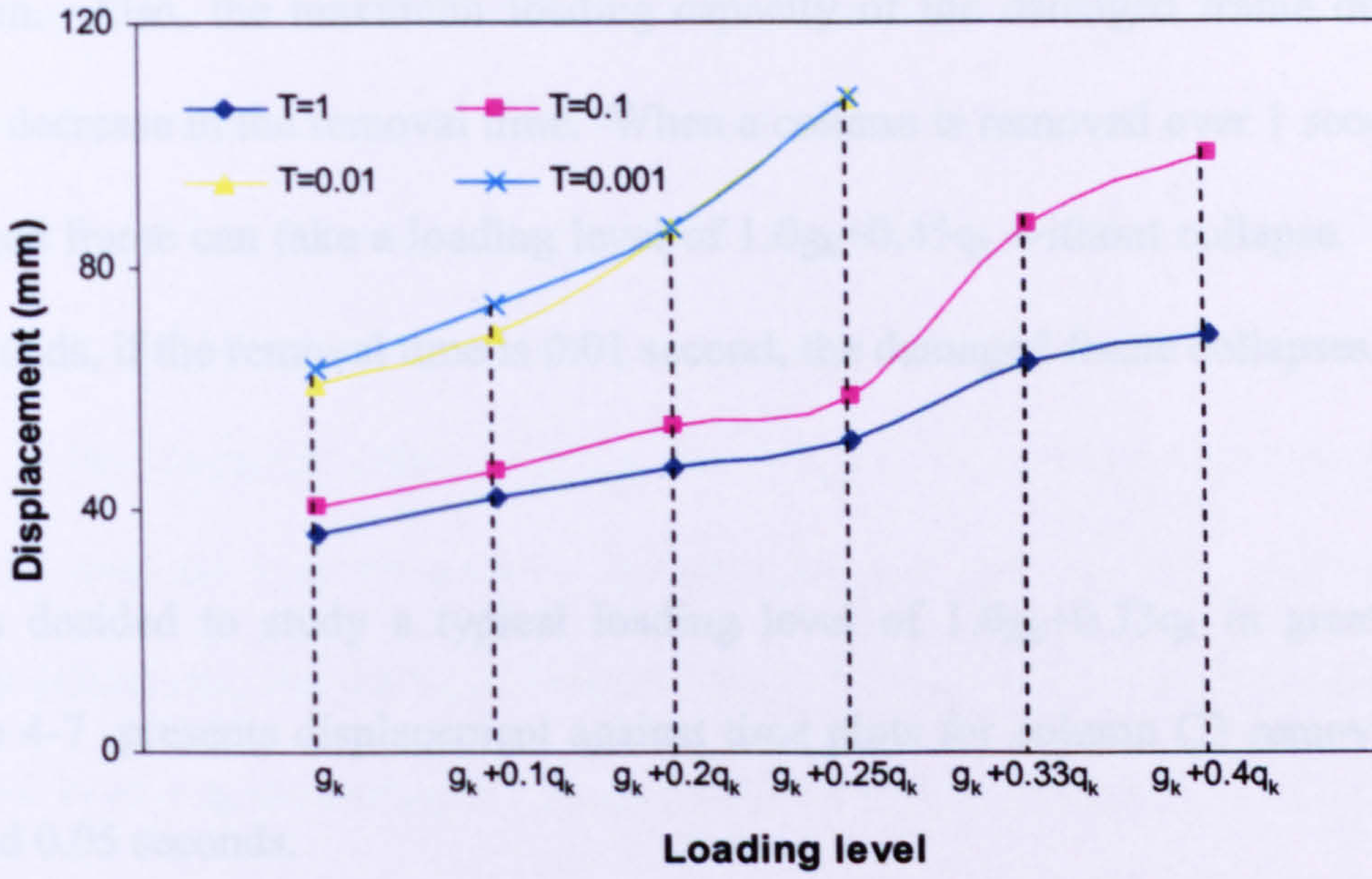


Figure 4-6Effects of loading level and column removal time on the vertical displacement at grid position C3

The results in Figure 4-6 show that the column removal time is an important factor which affects the dynamic structural behaviour assessed in terms of displacement. For instance, at a loading level of g_k, the maximum displacement is 50% greater when the column is removed in 1 millisecond than in 1 second. Also it was found that when the column removal time reached a certain stage, say 0.01 second, the results show only slight differences compared to that of 0.001 second. So there are two different regimes. One where the column is removed quite slowly and the structure behaves almost statically. The other is where the column is removed very quickly and the structure behaves very dynamically, which has a large dynamic overshoot of the quasi-static equilibrium position.

In general, the results suggest that the structural response during progressive collapse is related to the column removal time, emphasising dynamic nature of the problem. Also, the maximum loading capacity of the damaged frame decreases with a decrease in the removal time. When a column is removed over 1 second, the damaged frame can take a loading level of $1.0g_k+0.45q_k$ without collapse. For the same loads, if the removal time is 0.01 second, the damaged frame collapses.

It was decided to study a typical loading level of $1.0g_k+0.33q_k$ in great detail. Figure 4-7 presents displacement against time plots for column C3 removed in 1, 0.1 and 0.05 seconds.

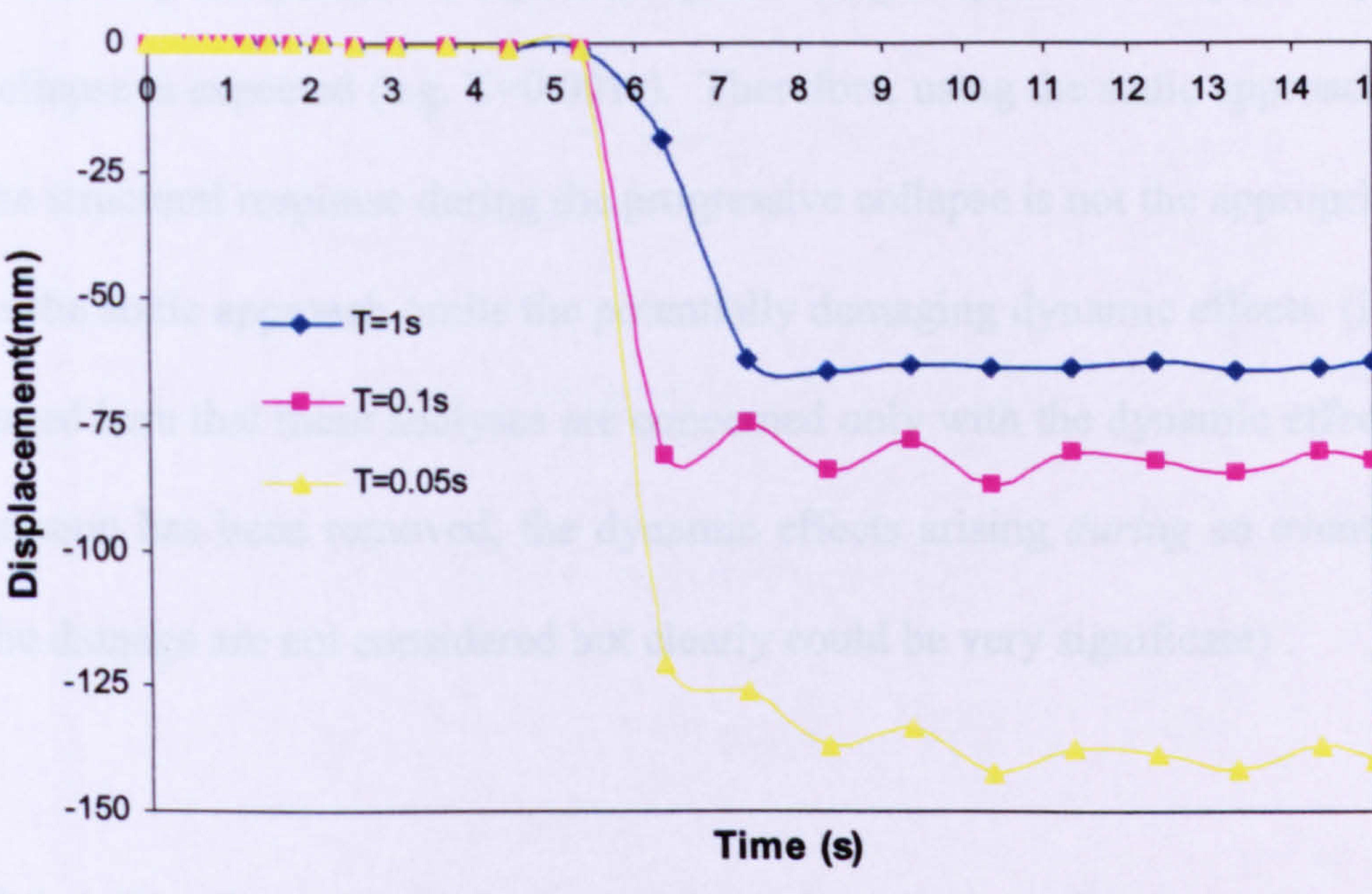


Figure 4-7 Vertical displacement V time plots for column C3 removed in different times (loading constant at 1.0 gk+0.33qk)

It was found that when the column was removed in 0.01 second, the damaged frame at this loading level ($1.0g_k+0.33q_k$) would collapse. It was also found that the damaged frame can survive when the column was removed in 0.05 second, albeit with gross vertical deflection.

From Figure 4-7 , it is not difficult to observe the influence of the removal time to the overall structural response. When the column was removed in 0.05 second, the damaged frame has the maximum displacement of 142 mm compared to 86.9mm of 0.1 second and 65mm of 1 second, that is 64% and 120% increase respectively. Thus, if the dynamic effects during the progressive collapse are ignored, the resulting analysis may be unsafe, and certainly not conservative. According to current guidelines, the loading level of $1.0g_k + 0.33q_k$ is the recommended loading level for checking the damaged frame. It follows that a static analysis of a damaged frame may not predict collapse, but performing a dynamic analysis may indicate a collapse is expected (e.g. T=0.001s). Therefore, using the static approach to predict the structural response during the progressive collapse is not the appropriate method, as the static approach omits the potentially damaging dynamic effects. (it should be noted here that these analyses are concerned only with the dynamic effects *after* the column has been removed, the dynamic effects arising *during* an event that cause the damage are not considered but clearly could be very significant) .

4.3.4 The Possible Resisting Mechanism

It is interesting to find out the resisting mechanism of a damaged frame when a column was removed in 0.05 second. LS-DYNA predicts the axial force and bending moment that are shown in Figure 4-8 and Figure 4-9 .

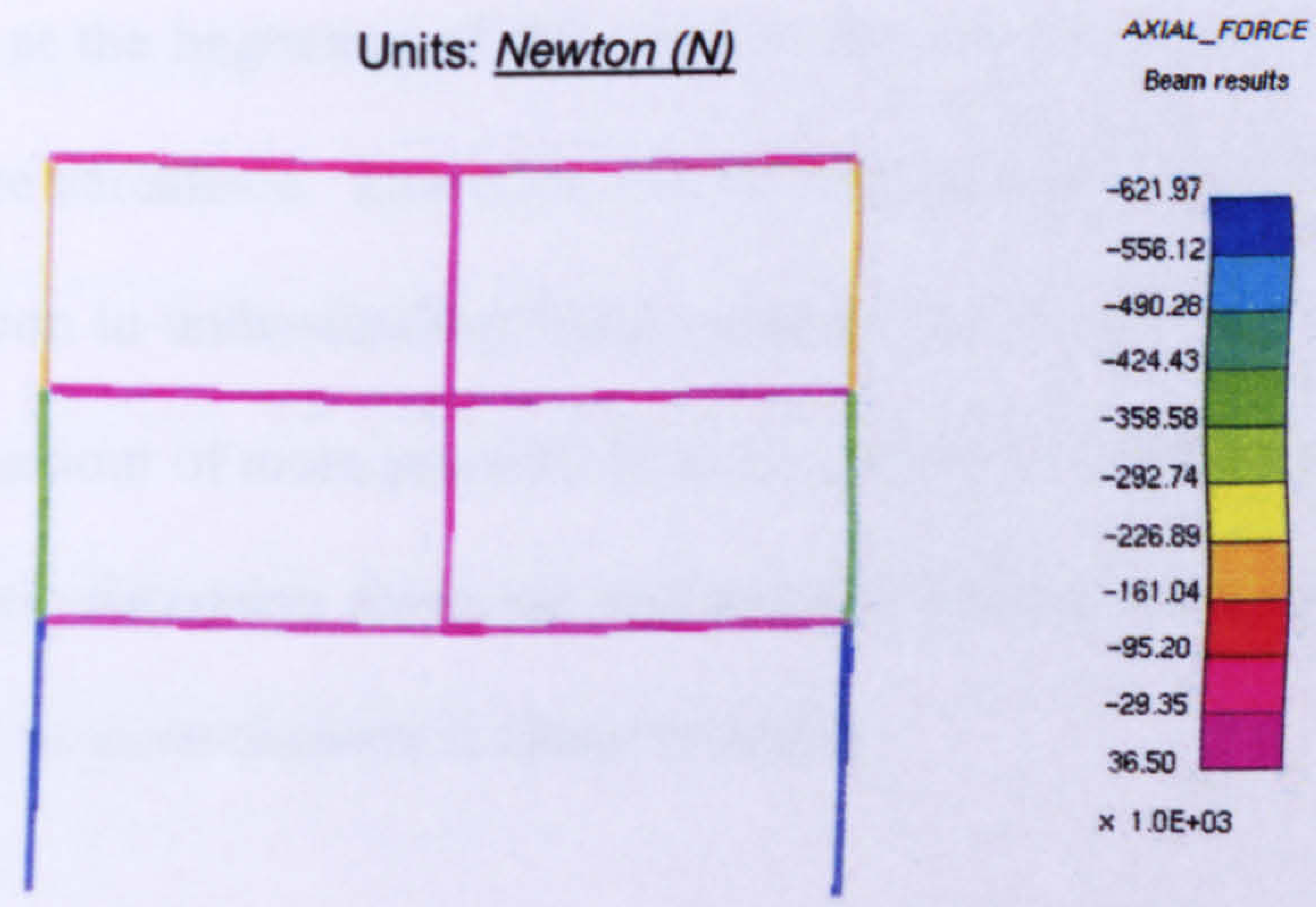


Figure 4-8Axial force in the damaged elevation at a loading level of $1.0g_k$+$0.33q_k$ when column C3 was removed in 0.05 second

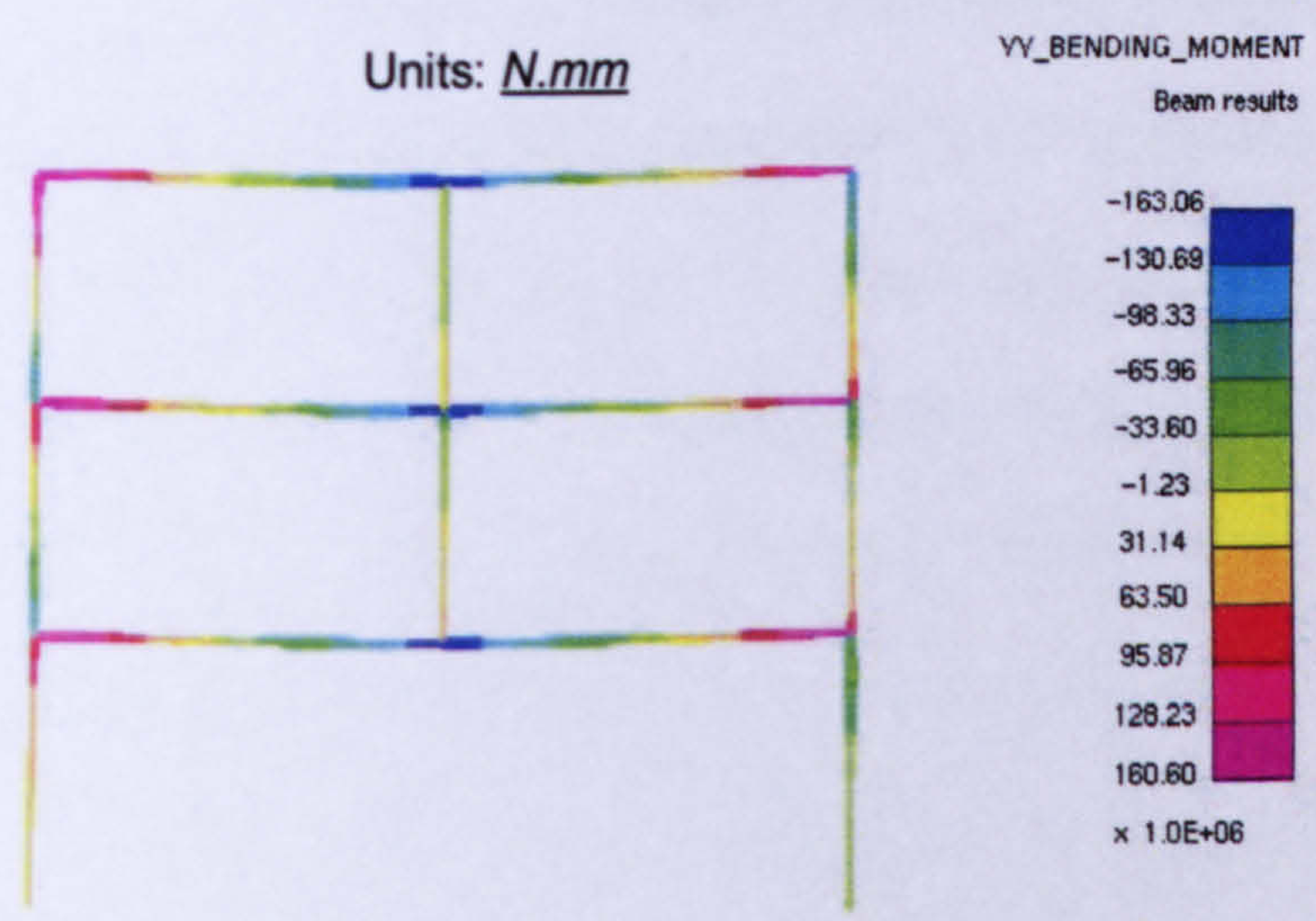


Figure 4-9 Bending moment in the damaged elevation at a loading level of $1.0g_k$+$0.33q_k$ when column C3 was removed in 0.05 second

Very similar behaviour was observed to that in the 1 second case, that is the tensile forces arising in the damaged frame is not great compared to the bending moments induced. This highlights that the resisting mechanism should be the same regardless of the column removal time. For the frame considered so far, this is a combination of catenary and Vierendeel action.

As discussed at the beginning of this section, the member sections of this small-scale frame are unrealistic. Therefore, the discussion about the resisting mechanism is a contribution to understanding frame response but it can not represent the real structural behaviour of more practical frames. Later on, in chapter 5 and chapter 6, a more detailed discussion about the resisting mechanism will be presented, as the frame studied in those chapters is closer to reality.

4.3.5 Discussion

This study was also intended to investigate joint stiffness effects. Instead of using the rigid connections, pin/semi-rigid connections were to be applied. However, if the rigid connection was replaced by a pin/semi-rigid connection, an alternative way of providing lateral stiffness is required. Bracing is the only way to provide lateral stiffness, because using external supports would attract loading and give unrealistic results. It was found to be very hard to brace this small frame without compromising its behaviour. The examination of the joint stiffness effects was difficult to achieve; therefore, an attempt was made to treat this small building as part of large building, but again on it was found that it is difficult to identify the stiffness of the adjacent building. These attempts to include joint stiffness are not included in detail here as they were inconclusive but the experience gained was useful in developing the work reported in chapter 6.

4.4 Conclusion

The studies presented in this chapter have given evidence that progressive collapse is a dynamic problem. An investigation of the resisting mechanism in a damaged frame was carried out, and it was found that Vierendeel action (bending moment) is the major supporting mechanism. In general, the overall resisting mechanism is likely to a combination of Vierendeel and catenary action.

Modelling a small structure cause a number of difficulties and it became clear that this small structure was too unrealistic to provide much of a detailed understanding of real frame behaviour. Therefore, it was decided to study a more realistic frame as reported in Chapter 5.

Chapter 5

Modelling Structural Behaviour During Collapse

5.1 Introduction

The non-linear finite element method has become a powerful analytical tool in the study of responses of real structures in different situations. Using FEM in this chapter investigates structural behaviour during progressive collapse. Important effects (such as *dynamic effects*, *column buckling* and *beam buckling*) that are present in real buildings have been included in the studies.

This chapter presents the results of studies of two common types of steel frames, those designed as *continuous design* and *simple design*. Continuous design assumes rigid joints and requires that the joints between beams and columns should have sufficient rotational strength and stiffness so that they are capable of resisting the moments and forces. On the other hand, the *simple design* which is widely used in

UK steelwork construction practice, assumes the joints between beams and columns do not to develop moments adversely affecting the members or the whole structure [BSI, 2000]. The distribution of forces around the structure may then be determined assuming all members are pin connected.

For convenience joints are usually assumed to act either as pins or as fully fixed, whilst in the real structure beam-to-column joints exhibit some flexibility and moment resistance. Therefore, the joints in reality are likely to behave as semi-rigid joints.

Composite construction, where the concrete acts compositely with a metal deck and the resulting slab acts compositely with the steel beams, is extremely popular in steel framed buildings. However, for this study it was the intention to identify the load paths within a steel frame, without the additional redundancy afforded by an in-situ composite slab. For this reason the frame was designed to carry precast units only, without the benefit of composite action between the slab and beams. In reality, this type of construction is commonly used, as shown in Figure 5-1 and Figure 5-2.

Figure 5-1 Construction Site a –Sheffield (photographed in 2004)

Figure 5-2 Construction Site b -Sheffield (photographed in 2004)

In steel frames, the current design guidance [BSI, 2000] requires that members should be tied together against progressive collapse. In a composite structure regardless of the continuity from the slab, it is assumed that tying is achieved through the primary and secondary beams (in two horizontal directions). Where pre-cast units are used, tying members are required in the direction that the units span (see Figure 5-3)

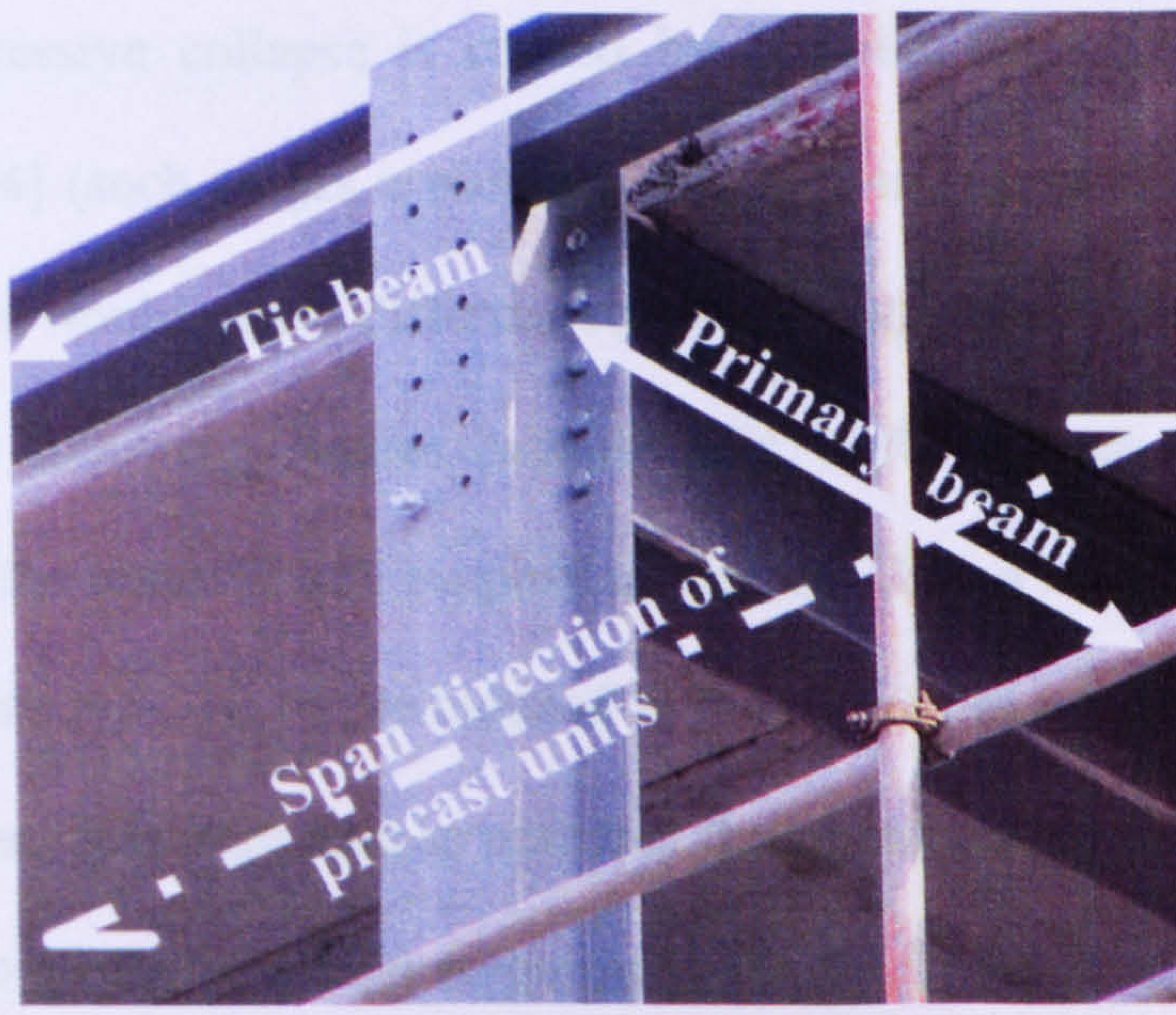


Figure 5-3 Illustration the Typical beam-to-column connection of a structure with precast units - photographed in 2004, Sheffield

In this study it was necessary to consider the ability of the frame *alone* to resist damage and for this reason a frame supporting pre-cast units (with the beams acting non-compositely) was designed and analysed rather than the more popular composite slab- composite beam system.

The latest UK building regulations [HMSO, 2004] now categorise all buildings into one of four classes. All office buildings, irrespective of the number of storeys, are required to be provided with effective horizontal ties but offices over 4 storeys are also required to have effective vertical ties [Moore, 2003]. In the UK the most popular steel framed structures are of relatively low level [Alexander, 2004; SCI, 1996] and therefore in the following section a 3-storey high building, designed in accordance with BS5950: Part1:2000 was studied.

Normally, progressive collapse is caused by low-risk high-consequence events [Alexander, 2004] (such as gas explosion, blast). It is important to recognise that different hazards would cause different structural responses, however, this is not included in the following studies because this work focuses on the structural behaviour *after* the removal of a supporting element (like a column). The primary concern of this study is the forces induced in the remaining members following the loss of a member. It is important to note that the research reported in the following section has ignored the structural response to the load that caused the removal of columns.

The modelling strategy was the same as discussed previously in chapter 4. A typical force-time history for the member removed is shown in Figure 5-4. By varying the removal time (T), from 1ms up to 1 second, the dynamic behaviour was simulated.

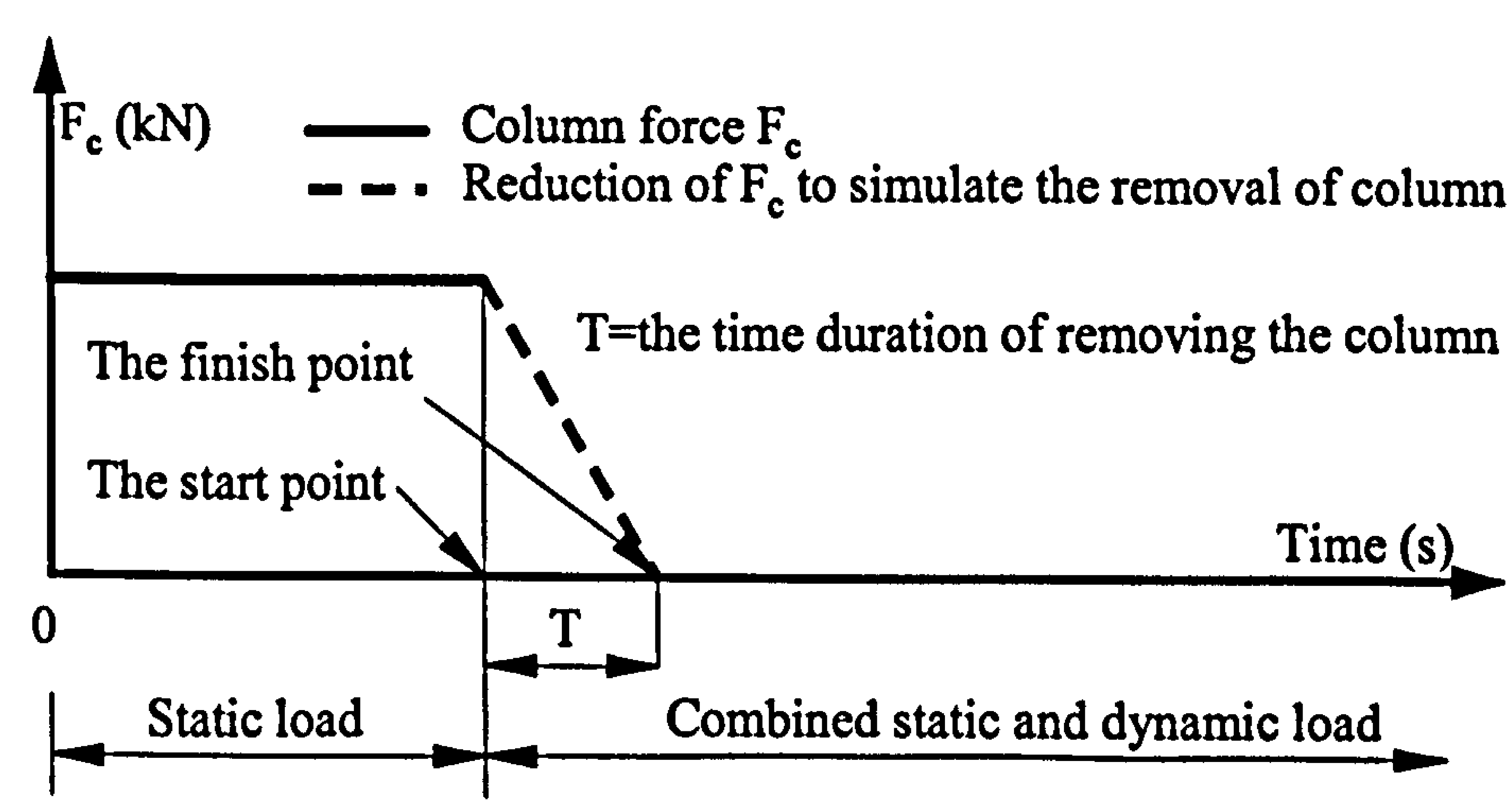


Figure 5-4 Modelling procedure for progressive collapse

5.2 New Modelling Feature

Although the previous studies of the small skeleton steel frame (section 4.2) gives confidence in the analytical tool, they only give a limited understanding of the structural behaviour during progressive collapse. As a consequence, it was decided to investigate a large scale frame.

A steel frame building can either be designed as a pinned frame in both directions with orthogonal bracing systems or designed as a pinned frame in one direction and a rigid frame in the other direction. In order to have a general understanding of steel framed buildings, it was decided to investigate two types of structure: i) a pin-rigid frame i.e. a rigid frame along the primary beam direction and a braced pinned frame in the other direction; ii) a pin-pin frame designed as a braced pinned frame in both directions.

As a starting point, the structural behaviour of a pin-rigid frame is examined. Initially it was decided not to include the floor (pre-cast units), as the rigid frames are the major loading bearing systems, but it was found the beams failed by lateral instability so the floor units had to be simulated to provide lateral restraint. The next section presents a way to model the floor units by shell elements and the contact between floor units and beams/columns without providing extra redundancy (i.e. composite action). A specified connection, named a pin-link, was used for this purpose.

5.2.1 Pin-link

The pin-link is an application of discrete beam elements (see chapter 3). Previous studies (chapter 3) have presented how to determine the stiffness (i.e. translational, rotational) for a normal beam-to-column connection. A pin-link encompasses most of the parameters associated with a normal pin connection, but a special parameter is defined for it.

The pre-cast floor unit is an application of a shell formulation. When considering insertion of the floor units to beams or columns, extra care is needed. The problem can be described as how to connect the two elements (shells, beams) without the contact between the floor and beams/columns becoming a critical part of the modelling of the large-scale building. Accordingly, a pin-link is used for connecting floors at the beam / column node. The application of a pin-link is based on the normal pin connection (restraining 3 translational degrees of freedom and releasing 3 rotational DOF), however, an additional translational DOF along the vertical direction (i.e. height of building) is released to avoid developing composite action between the floor and beams. Details of a pin-link application can be found in Figure 5-5.

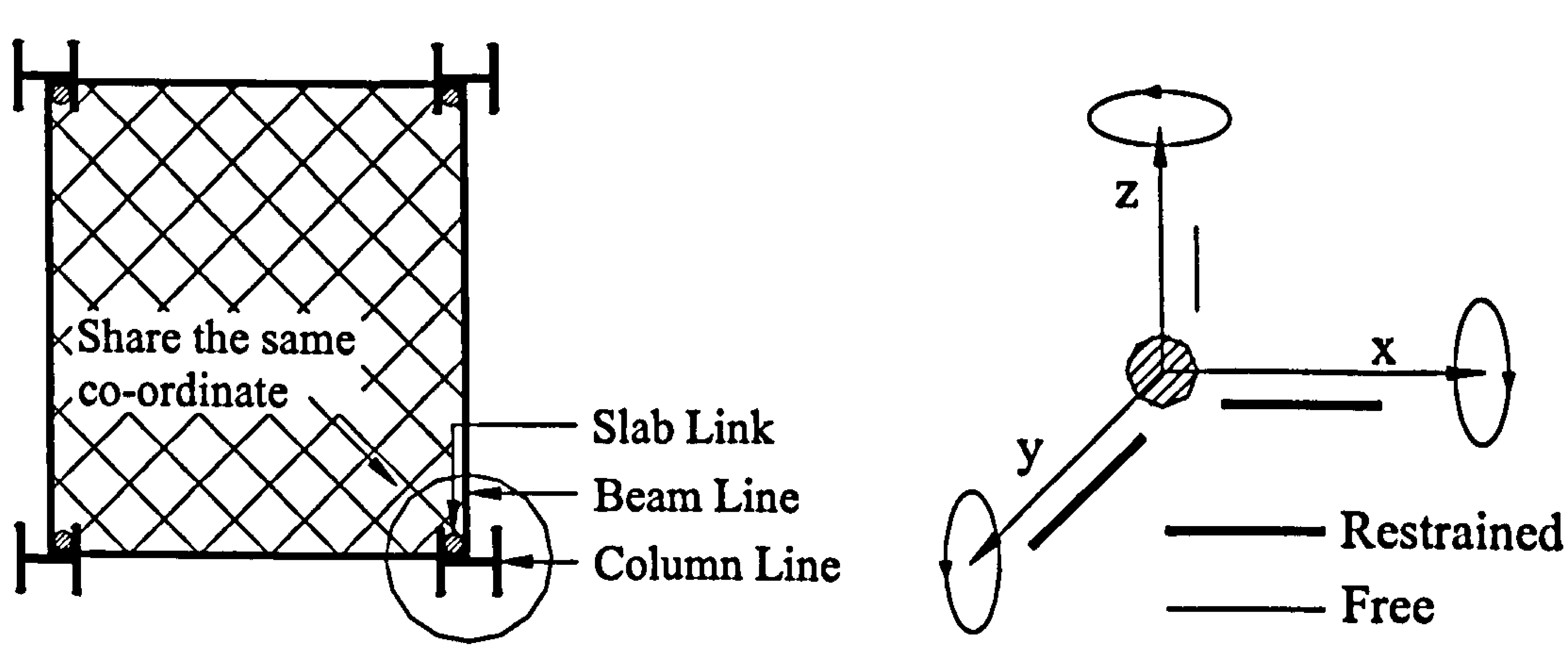


Figure 5-5 Illustration of Pin-link

The pin-link enables the structure to mobilise lateral stiffness (i.e. restrain the beam and act as a diaphragm) and also at the same time successfully avoids the inadvertent development of composite action. The pin-link was an important factor in the modelling of the structures that will be discussed in the following sections.

5.3 Modelling a pin-rigid frame

5.3.1 Introduction

A 3-storey building that has 6 bays along the x direction and 5 bays along the y direction was studied. The geometry details are shown in Figure 5-6 and Figure 5-7.

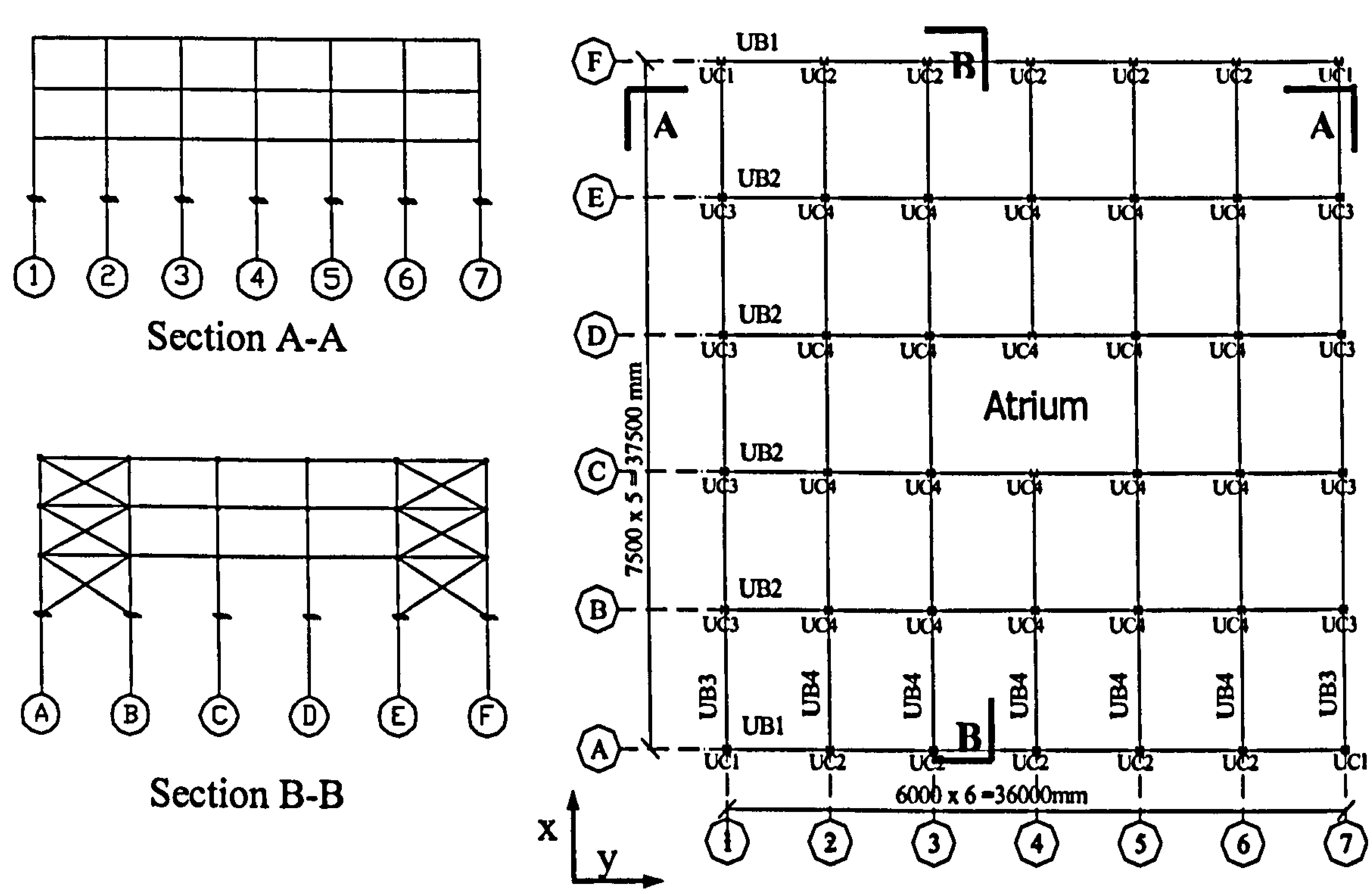


Figure 5-6 Geometric details of pin-rigid test frame

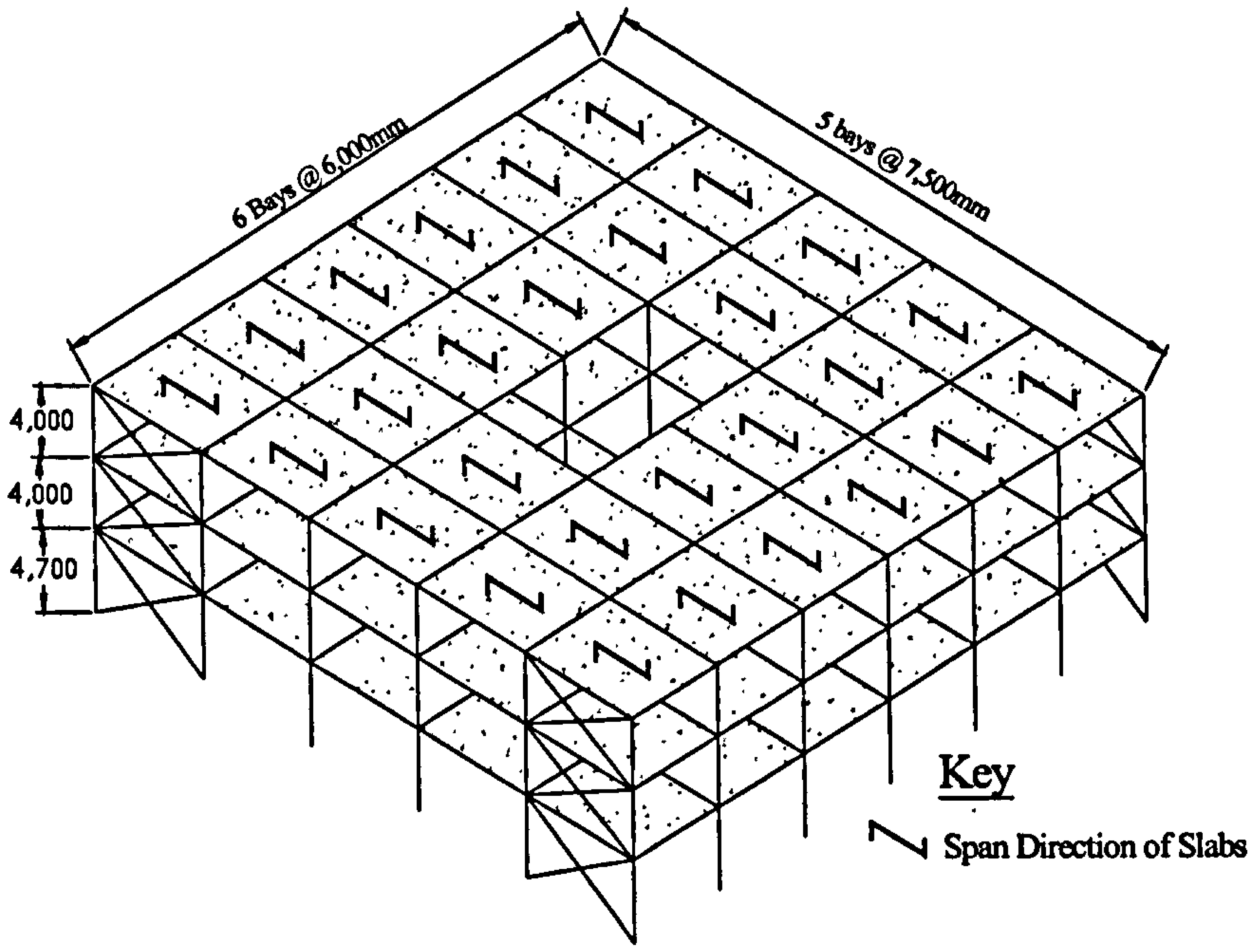


Figure 5-7 Arrangement of 3D pin-rigid test frame

This frame was designed according to BS5950-1:2000 [BSI, 2000]. The details of the design procedure can be found in Appendix-B. In this case the frame has rigid connections along elevation ①-⑦ section and pin connections along Ⓐ-Ⓕ elevation. Member sizes are presented in Table 5-1.

Table 5-1 Member Sections for 3-Storey Pin-Rigid Frame

	Beam		**Column**	
	Roof	Floor		
UB1*	UB 356x171x57	UB 457x191x74	UC1*	UC 356x406x287
UB2	UB 457x152x60	UB 457x191x89	UC2	Ditto
UB3	UB 305x127x42	UB 457x152x67	UC3	Ditto
UB4	ditto	ditto	UC4	Ditto

* For location of members see Figure 5-6.

The lateral stiffness of the frame along the x direction is provided by frame action and incorporates moment-resistant joints (rigid connections). The frame along the y axis is braced and non-sway and its lateral stiffness is provided by braced towers. In a continuous frame, the lateral stiffness is provided by each structural component which combine together to form a statically indeterminate system. It is not easy to analyse this structural system with hand calculations. Instead, a linear procedure elastic analysis software was used. (Oasys-GSA [Oasys, 2002]).

The current steel design code (BS5950) classifies frames as 'non-sway' when Pδ effects are negligible, otherwise they are 'sway-sensitive'. The second order effect is determined by λ_{cr} (elastic critical load factor), that is

$$\lambda_{cr} = \frac{h}{200\delta}$$

Where

h is the storey height

δ is the inter-storey sway cased by the application of notional horizontal forces only.

In terms of design, there are three ranges of λ_{cr} as shown in Table 5-1 [Way, 2003], each of which requires the adoption of a different design approach.

Table 5-2 Design action in relation to Pδ effects [Way, 2003]

Calculated λ_{cr}	**For clad structure where the stiffening effects of infill walls and cladding are ignored**		
	Second order effects	**Frame type**	**Design Approach**
≥ 10	Insignificant	Non-sway	Ignore second-order effect
<10 or ≥ 4	Significant	Sway sensitive	Amplify the Sway effects by k_{amp}
<4	Very significant		Perform a second-order elastic analysis on the frames

Ideally, the frame should be designed as a non-sway frame so that the P-δ effect can be ignored, but this also means the continuous frame would require very large member sections. It is not practical to increase the member sections to change a sway frame to a non-sway frame [Brown, 2002]. If the frame sways, and its λ_{cr} (elastic critical lead factor) lies between 4 and 10, based on current design guidelines, the frame is acceptable when its members are designed to resist amplified moments to approximate for the second order effects associated with the sway movement. In the pin-rigid frame under investigation, the rigid frames (those along the lettered grid lines in Figure 5-6) have been designed as sway frames.

Progressive collapse is a complicated structural phenomenon that is difficult to predict with simple analytical tools or design guidelines. In chapter 3 and chapter 4, the evidence gained showed that LS-DYNA (non-linear explicitly/implicit Finite

Element software) offers a way in which to gain a better understanding of real structural responses in different situations, and gives the opportunity to study structural behaviour during progressive collapse. The analytical model for this building employed 3 different types of elements - beam elements (beams and columns), discrete beam elements (connections) and shell elements (pre-cast floor units), and the details of the element formulations can be found in chapter 3.

5.3.2 Numerical Analysis

5.3.2.1 Loading Level Tests

According to current UK design guidance, a structure should be designed at the ultimate state to resist vertical loading factored at $1.4g_k+1.6q_k$ [Way, 2003]. However, the design load level is not usually sufficient to cause collapse, therefore it is important to investigate the real collapse level of the building in terms of robustness. If the collapse loading level of the building is greater than the factored design loads, then there exists a reserve of strength. Provided with this extra capacity, it is obvious that this structure would behave better when it meets any hazard. As the purpose of this study is not to quantify the overdesign in a frame but rather to find out how the loads re-distribute during progressive collapse, the spare capacity in the design should also be taken into account.

Analyses were conducted with 90kN/m ($\equiv 1.4g_k+1.6q_k$)♣. (For convenience, the loads in terms of kN/m used in the following sections all referred to UB1 unless

♣ For floor beam UB1, g_k is about 37kN/m which includes the cladding; q_k equals 24 kN/m noted otherwise.) If the building did not collapse under this load then an increased load level was applied and the analysis repeated. It was observed that the final collapse loading level is about 108kN/m ($=1.6g_k+2.0q_k$). Clearly, the overcapacity in the frame is about 20%. The axial force present in the members at this collapse loading level (108kN/m) is shown in Figure 5-8.

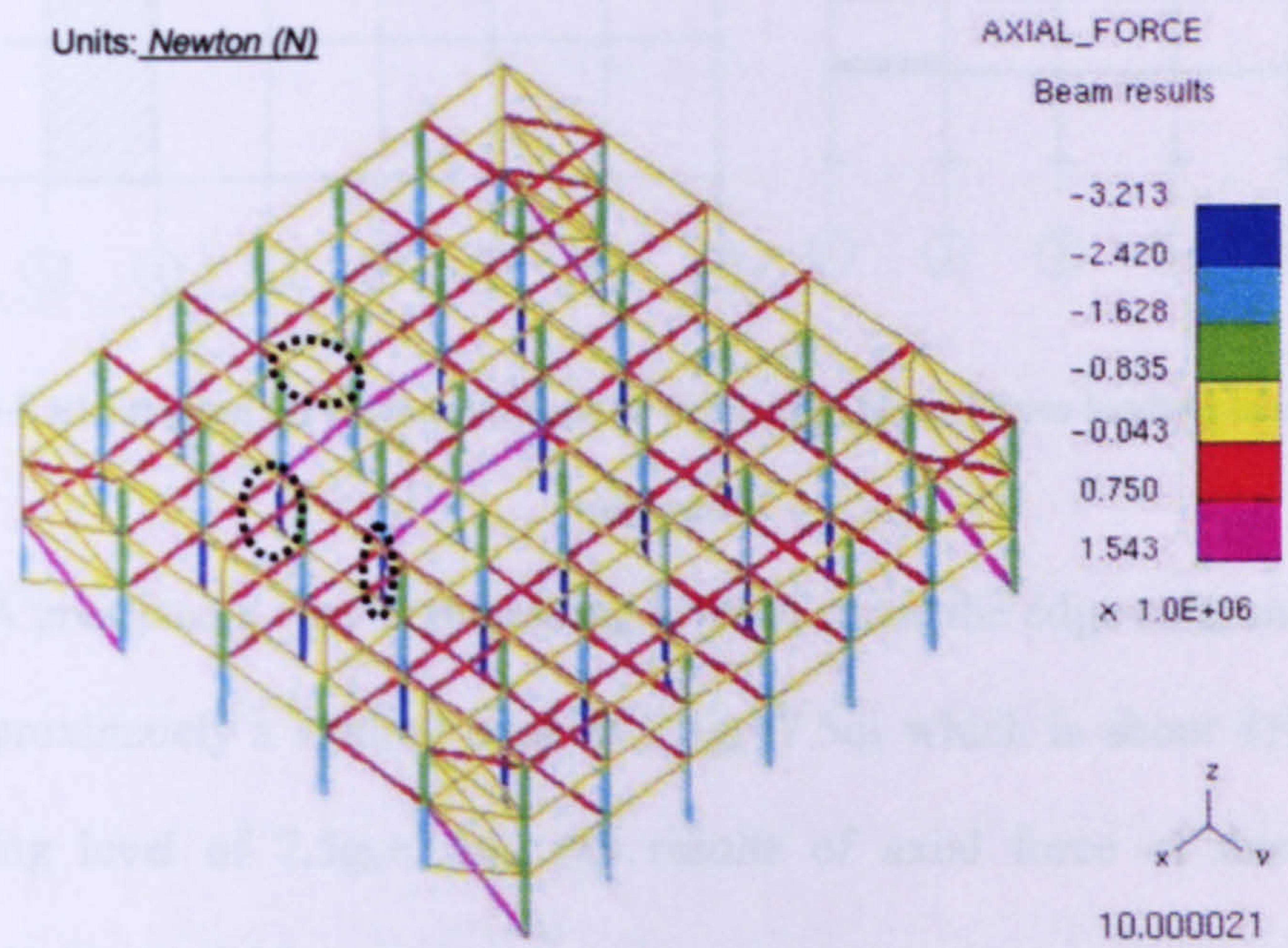


Figure 5-8 Pin-rigid frame –Axial force output (undamaged frame)

In Figure 5-8, some columns (especially UC4) were buckled. Although the frame did not collapse during the whole time period (that is 10 seconds), if the analysis ran for longer it would collapse. It is observed that at this collapse loading level of 108kN/m the middle columns were damaged (buckled), but not the edge columns. It is worth noticing that the edge columns of the building are more vulnerable in terms of malicious attack [Corley, 1998; GSA, 2003; MMC, 2003; Corley, 2004; Marjanishvili, 2004]. Therefore, it is necessary to find out the loading level that causes the edge columns to fail. A set of tests were then carried out for this purpose. This time, the loads were only applied to the edge space (along grid line ① and ②). Details of the tests can be found in Figure 5-9.

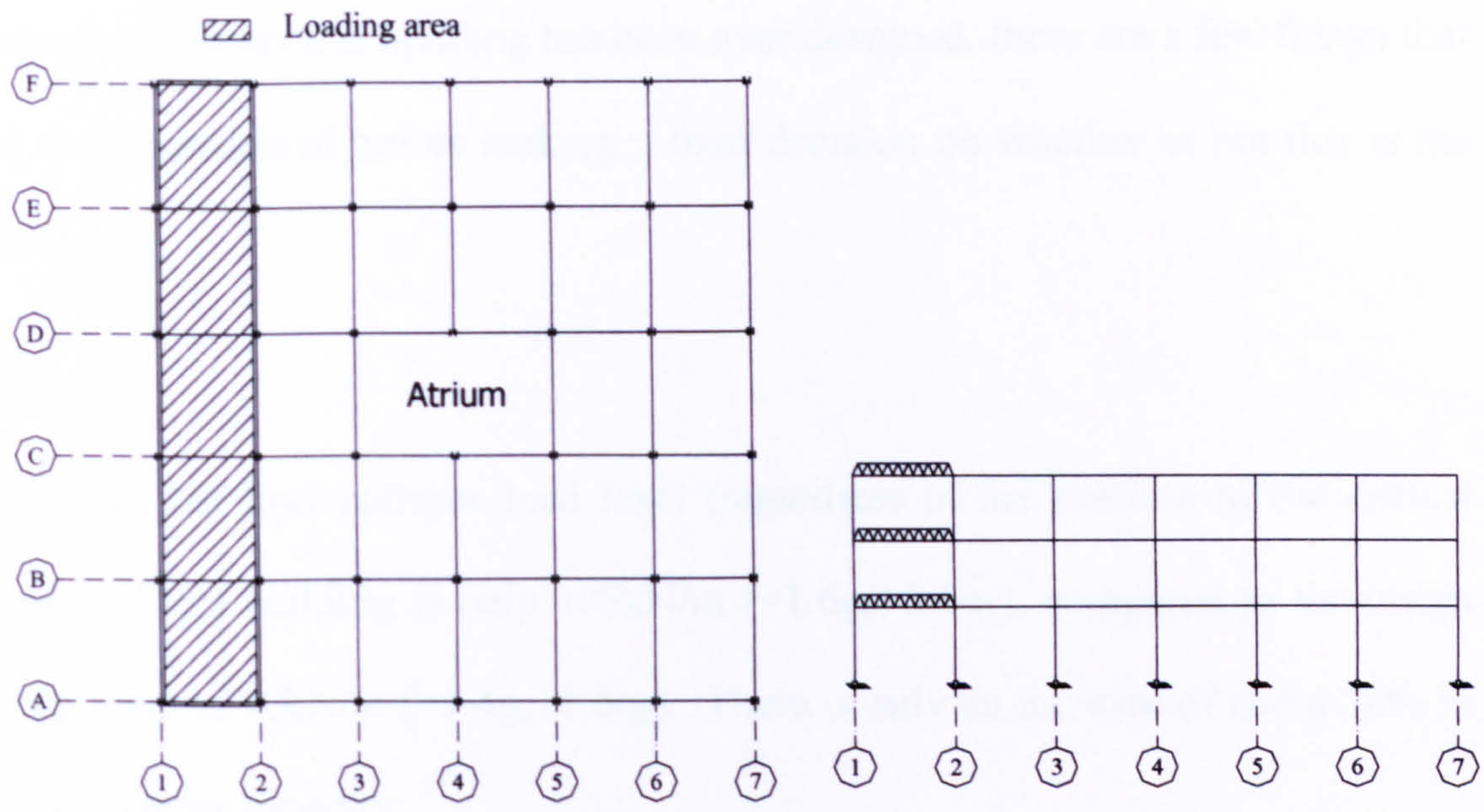


Figure 5-9 Illustration for numerical tests to determine the collapse loading level for edge columns

LS-DYNA predicted a very high loading level to cause the edge columns to buckle; it was approximately a loading level of $7.5g_k+7.5q_k$ which is about 459kN/m. At this loading level of $7.5g_k+7.5q_k$, the results of axial force of this frame are presented in Figure 5-10.

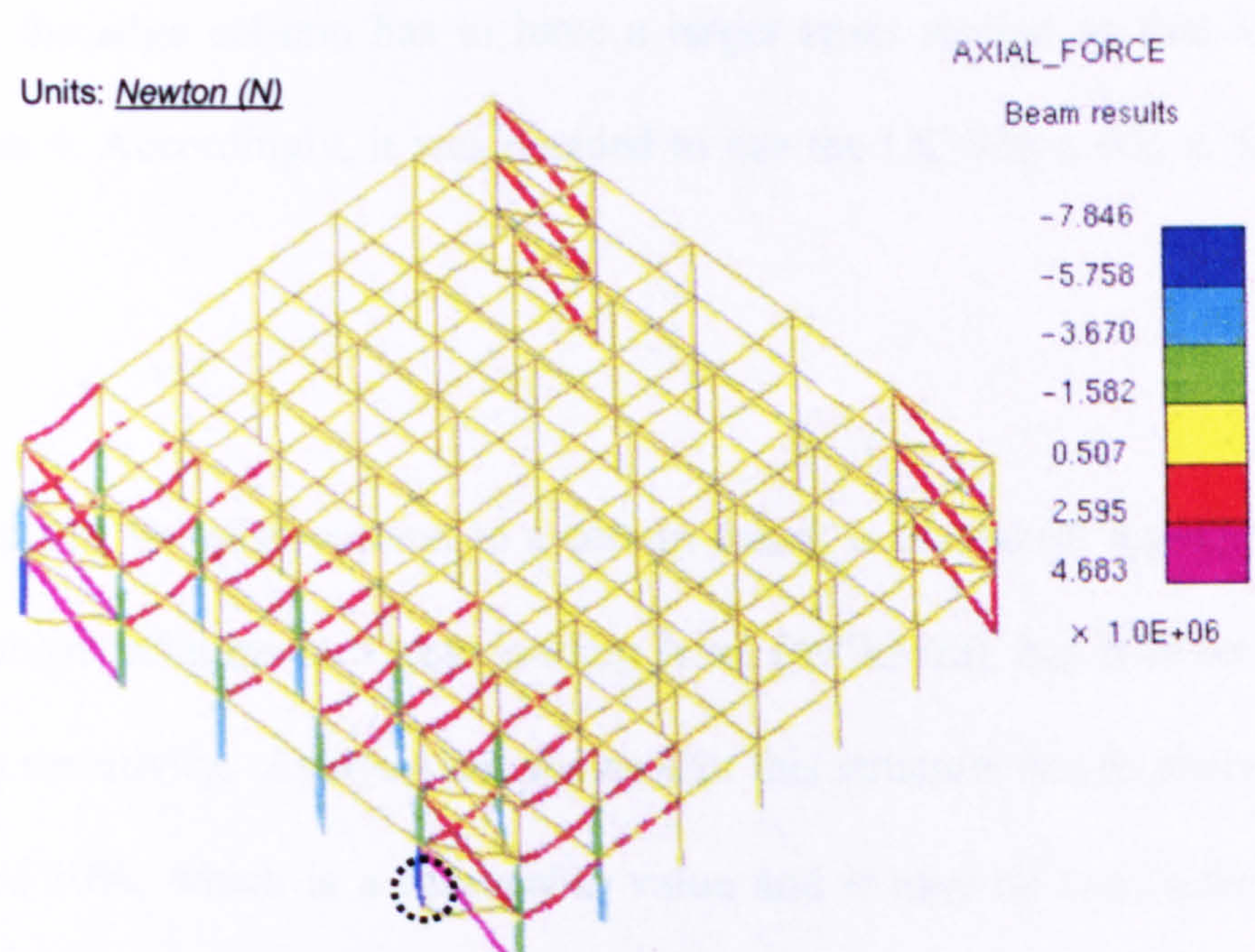


Figure 5-10 Axial force of the pin-rigid frame at loading level of $7.5g_k+7.5q_k$ to investigate the collapse loading level for edge columns

Although it appears this building has been over designed, there are a few things that need to be considered before making a final decision on whether or not this is the case.

First of all, the final collapse load level (regardless of the position of the critical column) of this building is only 108kN/m (=$1.6g_k+2.0q_k$), compared to its design loading level of 90kN/m (=$1.4g_k+1.6q_k$). There is only an increase of about 20% in the load bearing capacity.

Second, the frames along the lettered gridlines are sway frames. In order to use the amplified moment method, the λ_{cr} has to lie between 4 and 10. The results from Oasys-GSA have shown that if a small cross section for the edge column is chosen, the value of λ_{cr} is likely to be smaller than 4 (details can be found in Appendix-A). Therefore, the edge column has to have a larger cross section so that λ_{cr} can be greater than 4. Accordingly, it was decided to use the UC 356 x 406 x 287 for all columns.

It is agreed that the edge column to a certain extent is oversized, e.g. the buckling force for those columns is a high loading level (495kN/m), but it is necessary to limit sway sensitivity. Analysis has shown that this structure has an excess loading capacity of 20%, which is a reasonable value and it may be concluded that this building as a whole is therefore not grossly over designed.

5.3.2.2 Column ©① was removed in 1 second

The following numerical tests were considered with columns removed from the original frame. As discussed earlier, the edge (outside) of the building is more likely to be attacked or accidentally damaged, therefore, the following numerical studies considered the removal of column(s) along the outside face of the frame.

As a starting point, one column (column ©①) was considered to be removed in one second. Clearly, in a malicious or accidental loading case, an abnormally loaded column would require some finite time to fail entirely. The removal of the columns was modelled as follows (Figure 5-11).

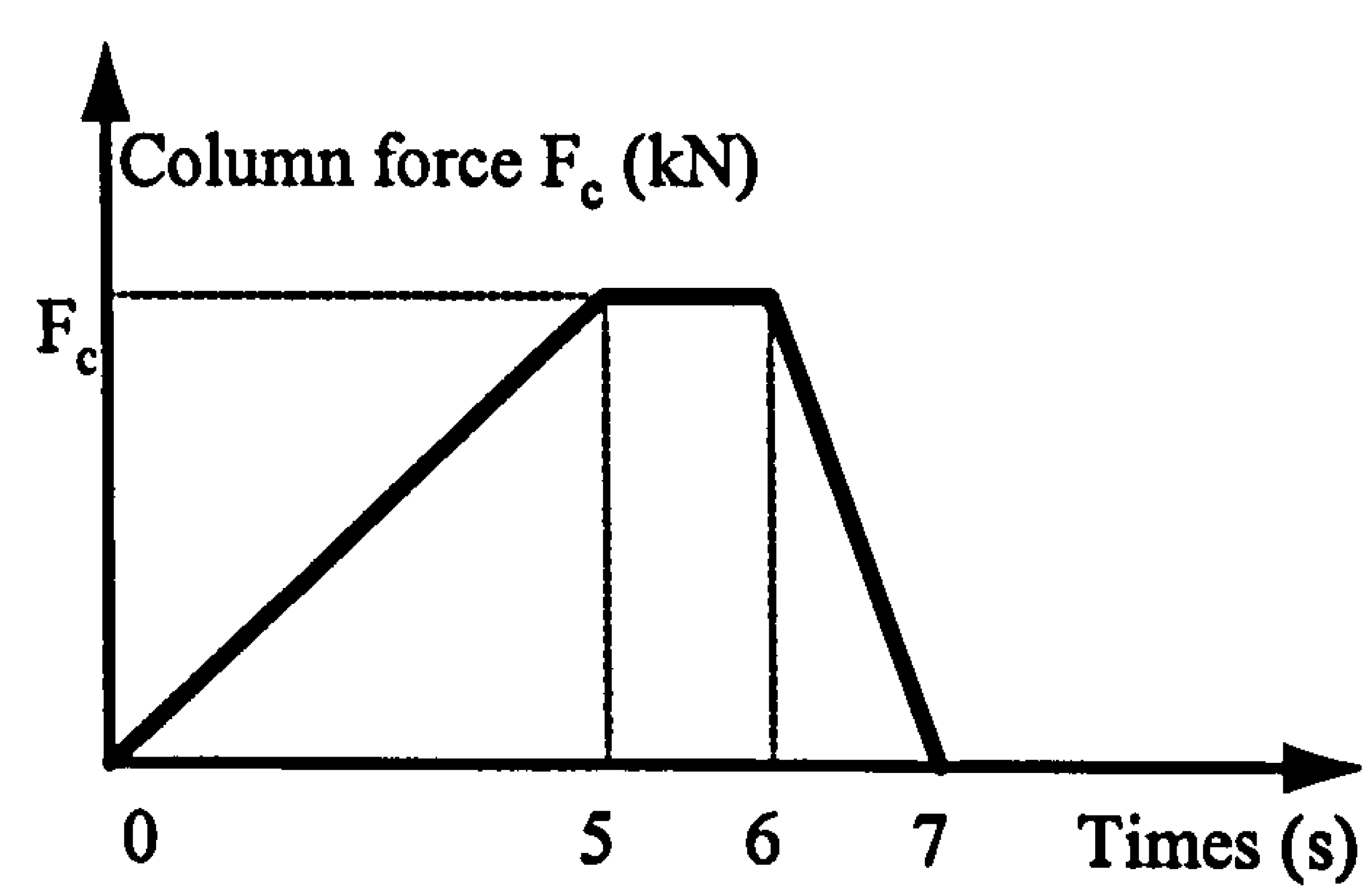


Figure 5-11 Illustration of pseudo-column force for analysis

The UK code [BSI, 2000; Way, 2003] permits the use of reduced loads and load factors ($1.0g_k+0.33q_k$) when considering the strength of a damaged structure. The following numerical tests were first conducted with this loading level. If the damaged building could stand, then increased loads were applied and the analyses repeated until the frame failed. The results are presented in Figure 5-12.

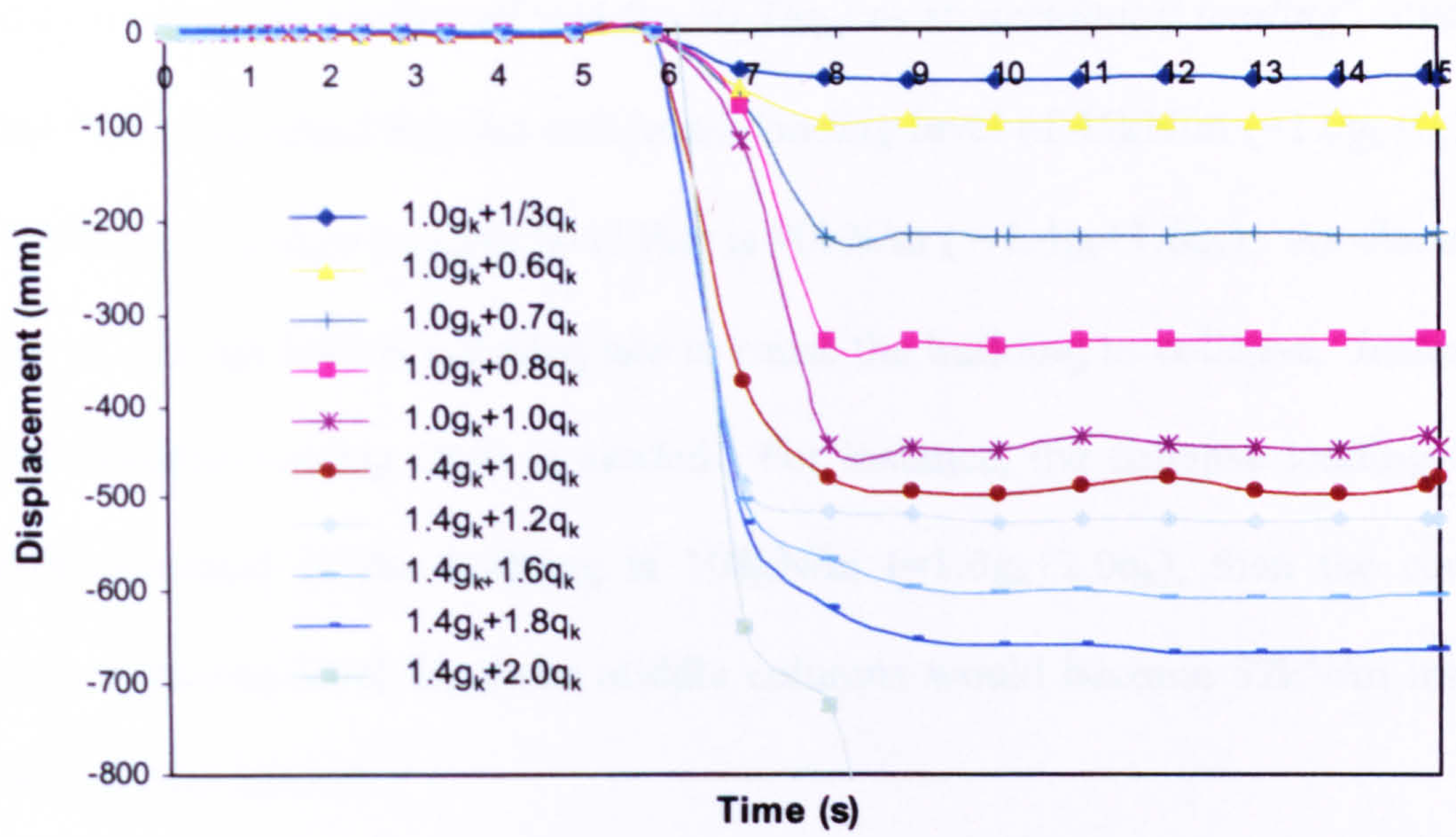


Figure 5-12 Illustration of vertical displacement at various loading levels when column ©① was removed in 1second from the pin-rigid frame

Figure 5-12 shows results from analyses carried out with different loading levels but the time taken to completely remove the load from ©① was held constant i.e. T (Figure 5-4) equals 1 second. LS-DYNA predicts that this frame can stand up with at loading level of 95kN/m (=$1.4g_k+1.8q_k$) and the structure collapsed when the loading level was increased to 100kN/m (=$1.4g_k+2.0q_k$). It is interesting to find out the reason that this damaged building can stand up with such a loading level, which is higher than the normal design loads of 90kN/m (=$1.4g_k+1.6q_k$).

5.3.2.3 Investigation of the load ratio λ ($\lambda_{d,}$, λ_r)

The loading level of γ_f [1] ($1.0g_k+0.33q_k$) is recommended by BS5950 when considering that the members are notionally removed. For this study, it was

[1] $\gamma_f = 1.05$

decided to name this loading of $\gamma_f\,(1.0g_k+0.33q_k)$ as an *accidental loading*♣ level. It needs to be emphasised that the *accidental loading* level of 45kN/m (=$1.0g_k+0.33q_k$) is related to the *design loading* level that is 90kN/m (= $1.4g_k+1.6q_k$). As discussed before, the design load is not adequate to cause the building to collapse. Instead, a revised collapse loading level is needed. For instance, the collapse loading level (middle columns) of the building is 108kN/m (=$1.6g_k+2.0q_k$), then the revised accidental loading level for those middle columns would become 52kN/m instead of 45kN/m (see below):

$$1.6g_k = 1.4G_{mid}^{`} \Rightarrow G_{mid}^{`} = 42kN/m; \quad 2.0q_k = 1.6Q_{mid}^{`} \Rightarrow Q_{mid}^{`} = 30kN/m$$

$$(g_k = 37kN/m;\ q_k = 24kN/m)$$

$$\gamma_f(1.0G_{mid}^{`} + 0.33Q_{mid}^{`}) = 52kN/m$$

Where:

g_k is designed dead load (kN/m)

q_k is designed imposed load (kN/m)

$G^{`}_{mid}$ is the revised dead load of middle column UC4 (kN/m)

$Q^{`}_{mid}$ is the revised imposed load for middle column UC4 (kN/m)

According to this simple calculation, it is noteworthy that the load ratio (λ) between accidental loading level and collapse level is around 50%, for the particular balance of dead and imposed used in this study.

♣ BS 6399 defines 34kN/m^2 for the accidental load.

$$\lambda_d = \frac{accidental}{collapse} = \frac{\gamma_f(1.0g_k + 0.33q_k)}{1.4g_k + 1.6q_k} = \frac{45}{90} = 50\%$$

$$\lambda_r = \frac{accidental}{collapse} = \frac{\gamma_f(1.0G_{mid}' + 0.33Q_{mid}')}{1.4G_{mid}' + 1.6Q_{mid}'} = \frac{52}{108} = 48\%$$

Where:

λ_d *is* a ratio for the designed loading level.

λ_r is a ratio for the revised loading level.

It was decided to investigate whether the ratio (λ) would be vastly changed by varying the balance between the dead and imposed load. The dead load is normally constant, for instance, in this case the g_k is about 5kN/m^2. On the other hand, the imposed load varies, which means it can be as high as 6kN/m^2 or it can be as low as 1 kN/m^2 [BS6399]. The results are reported in Table 3

Table 5-3 Effects on ratio (λ) when varying imposed and dead load

loading (kN/m^2)			loading level (kN/m^2)		Ratio λ (%)
	G	Q	γ_f(G+0.33Q)	1.4G+1.6Q	
varying imposed load	5	6	7.00	16.60	42%
	5	5	6.67	15.00	44%
	5	4	6.33	13.40	47%
	5	3	6.00	11.80	51%
	5	2	5.67	10.20	56%
	5	1	5.33	8.60	62%
varying dead load	6	6	8.00	18.00	44%
	7	6	9.00	19.40	46%
	8	6	10.00	20.80	48%
	9	6	11.00	22.20	50%
	10	6	12.00	23.60	51%
	11	6	13.00	25.00	52%
	12	6	14.00	26.40	53%
	4	6	6.00	15.20	39%
	3	6	5.00	13.80	36%

As can be seen in Table 5-3, it appears that the imposed load has more influence on the ratio compared to the dead load. If the same dead load is kept (e.g. 5kN/m^2) and the imposed load is varied, then it is found that the difference in the load ratios goes up to 48%, whilst there is a 20% difference when the dead load is varied and the imposed load is kept constant. The ratio (λ) listed in Table 5-3 shows that a range of 40-60% is reasonable. A ratio (λ) below 40% or above 60% is relatively rare for a pre-cast structure. Hence, the value λ_d (50%) and λ_r (48%) adopted for the 3D pin-rigid frame (middle columns) are acceptable.

It is also necessary to investigate whether the λ ratio would be in the same range when the edge column is removed. Although the dead load of the cladding is not included in Table 5-3, the external wall is equivalent to approximately 1kN/m^2 distributed across the floor. The previous results have shown that the edge columns failed at a loading level of 459kN/m (=7.5g_k+7.5q_k). This means that the revised accidental loading level should be 235kN/m, as follows:

$$7.5g_k = 1.4G_{edge}^{`} \Rightarrow G_{edge}^{`} \approx 198kN/m; \quad 7.5q_k = 1.6Q_{edge}^{`} \Rightarrow Q_{edge}^{`} \approx 112kN/m$$

$$(g_k = 37kN/m;\ q_k = 24kN/m)$$

$$\gamma_f(1.0G_{edge}^{`} + 0.33Q_{edge}^{`}) = 235kN/m$$

Where

$G^{`}_{edge}$ is the revised dead load of middle column UC3 (kN/m)

$Q^{`}_{edge}$ is the revised imposed load for middle column UC3 (kN/m)

The *revised accidental loading* level is 235kN/m, thus the λ_r in this case is around 51%. In fact, the damaged frame can only take a loading of 95kN/m. This demonstrates that the damaged frame only takes about 40% of the *revised accidental load*, which is about $0.5G'_{edge}$. In this sense, the damaged frame is less robust than the code suggests it should be. The current BS5950 has a recommended loading level of γ_f (1.0G+0.33Q), when considering the strength of a damaged building. In addition, it is found that the *ratio* $(\lambda = \frac{accidental}{collapse})$ should be normally around 50% for a 3D pin-rigid frame. The numerical results from LS-DYNA have shown evidence that the damaged frame cannot take such a high load ratio at the *accidental* limit state, instead the damaged frame can only take around 20% of the (true) collapse loading level.

5.3.2.4 The possible resisting mechanism

A major task of this study was to investigate the resistance mechanism which allows a damaged frame to remain standing. Current design guidance in BS5950:2000 Part1 requires members to be tied together against progressive collapse, and the tying strategy suggests that the remaining frame should develop catenary action. The details of the justification for this requirement have been found to be illogical [Brown *et al*, 2004]. For instance, in order to obtain the maximum rotation angle (assumed to be 45^0), the vertical deflection (h) has to be equal to half of the span (L/2), and normally the half span would be greater than the storey height, which makes it impossible to achieve the rotation of 45^0.

A study was carried out with a damaged frame at a loading level of 95kN/m ($=1.4g_k+1.8q_k=0.5G`_{edge}$), which is the maximum loading level that the damaged frame can bear. LS-DYNA predicts a maximum vertical displacement of 670mm (Figure 5-13) and axial force of 1000kN (Figure 5-14).

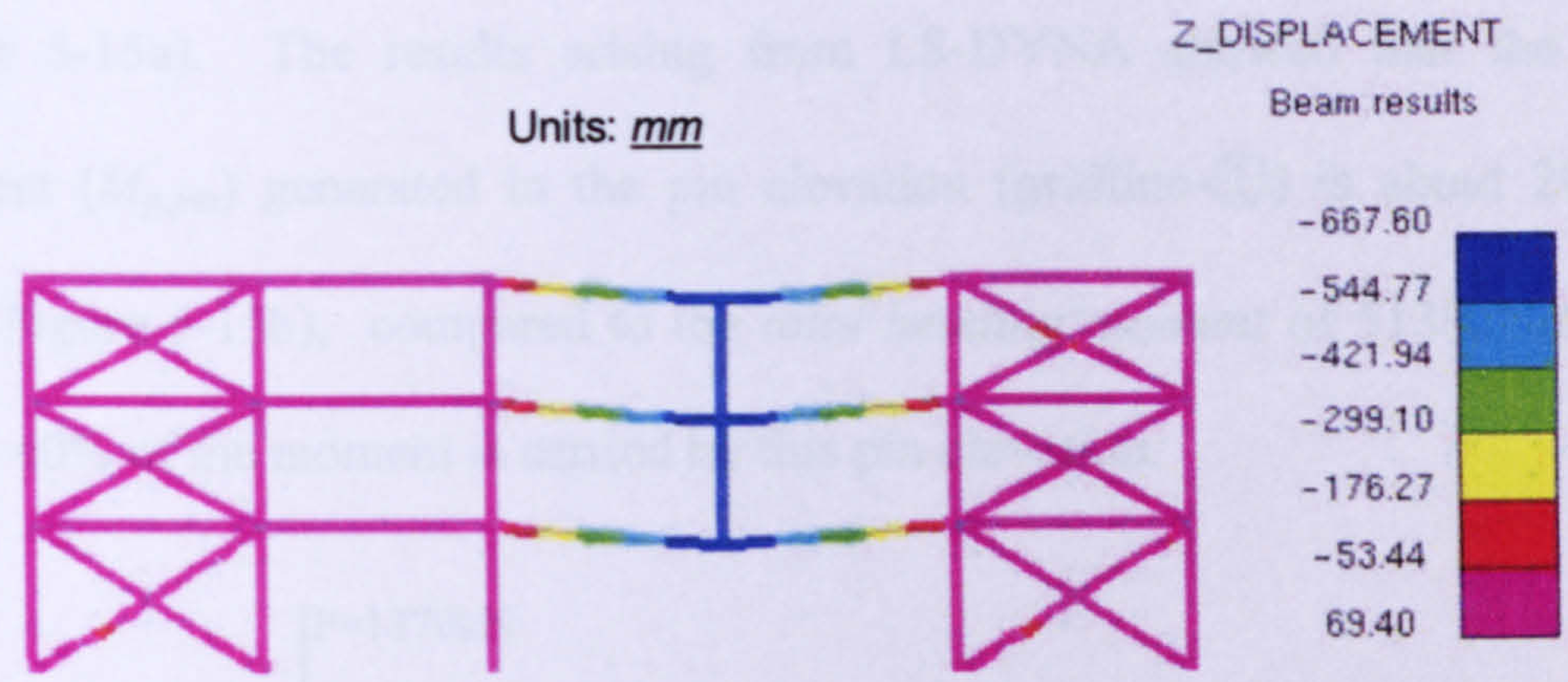


Figure 5-13 Displacement of the damaged elevation when column Ⓒ① was removed in second at loading level of $1.4g_k+1.8q_k$

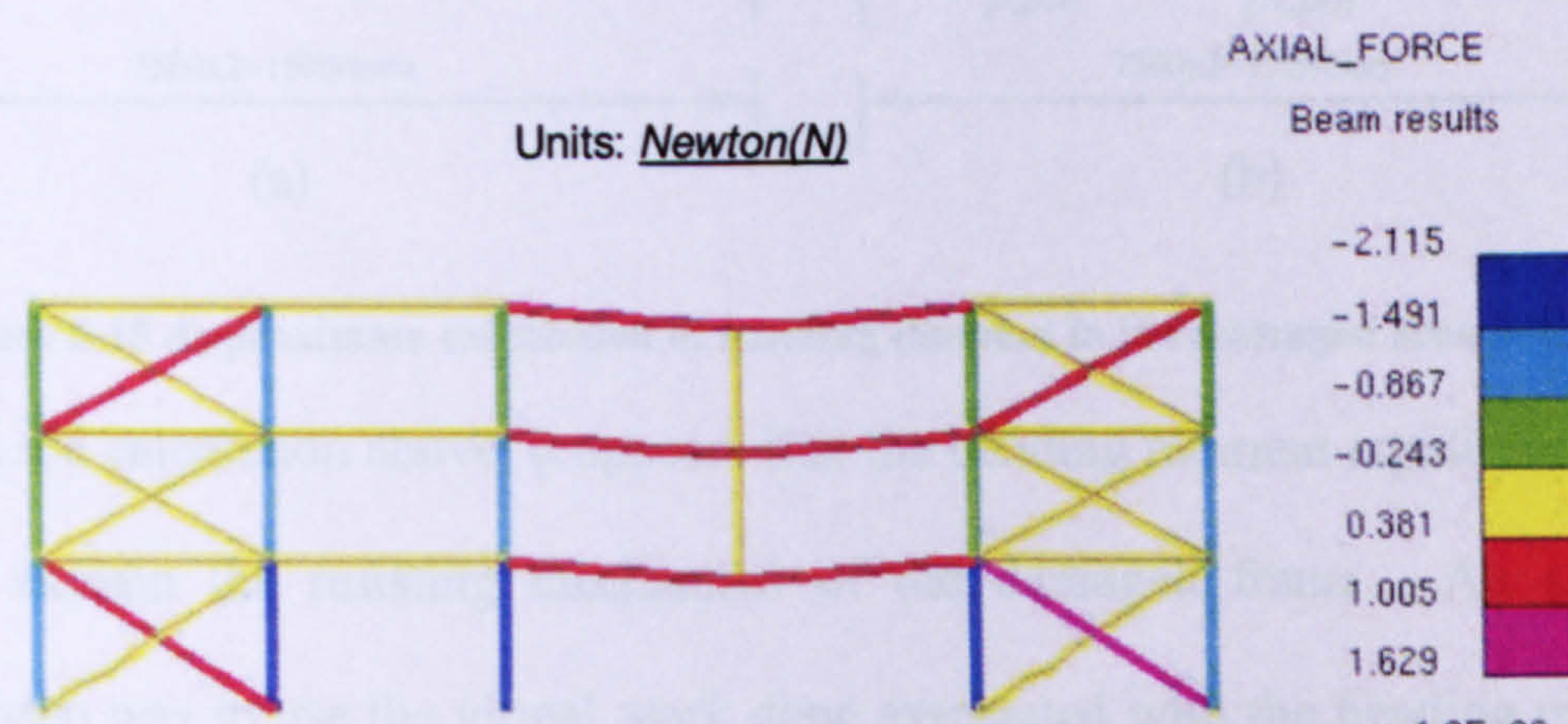


Figure 5-14 Axial force of the damaged elevation when column Ⓒ① was removed in second at loading level of $1.4g_k+1.8q_k$

Through simple trigonometry it is found that the maximum rotation angle for this damaged frame is about 5 degrees ($\tan\theta_{pin}=\dfrac{670}{7500}\Rightarrow\theta=5.1^0$), which suggests that catenary action is not the resisting mechanism. Whatever the resisting mechanism is, further study is needed.

Consider a point load of 1370kN, which represents the axial force of column ©① at loading level of 95kN/m, applied downward to the damaged frame along gridline-①. The *total bending moment ($M_{p,total}$)* of the damaged frame can be evaluated approximately by simple beam theory, that is about 5138kN.m (see Figure 5-15a). The results arising from LS-DYNA showed that the bending moment ($M_{p,pin}$) generated in the pin elevation (gridline-①) is about 2010kN.m (see Figure 5-15b), compared to the *total bending moment* of 5138kN.m, that is about 40% of the moment is carried by this pin elevation.

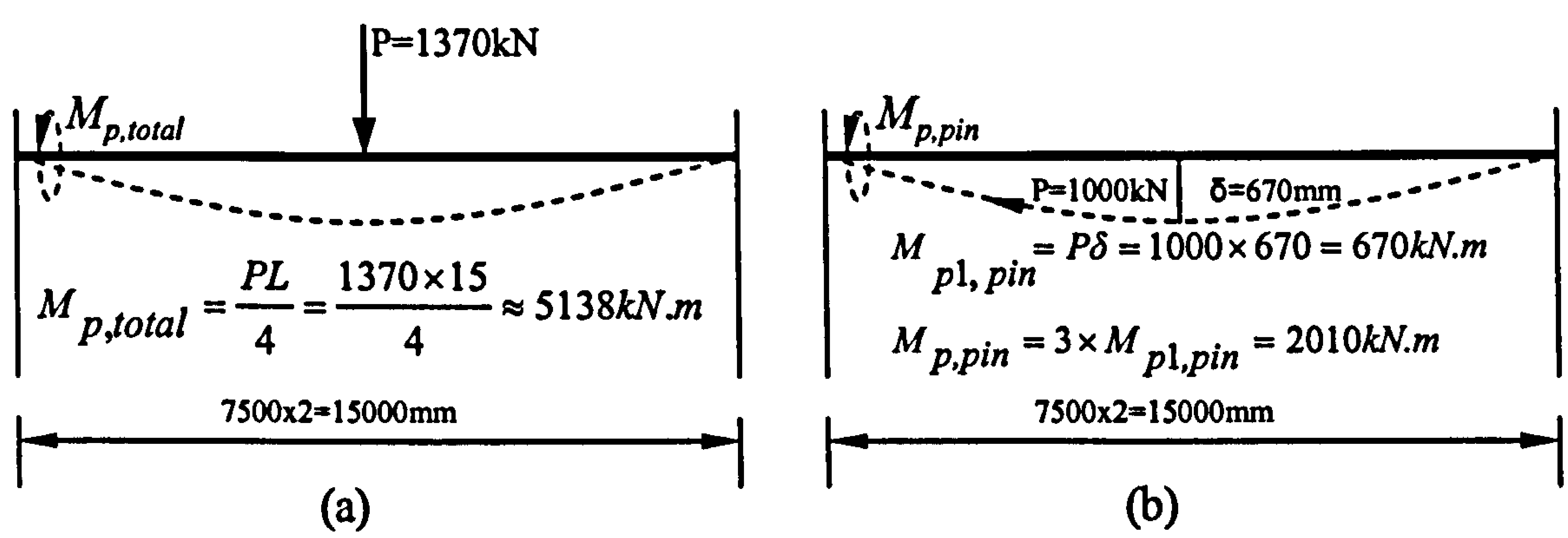


Figure 5-15 Approximate calculation of bending moment in the damaged area of the frame

From the calculation above, it appears that the bending moment equilibrium cannot well explain the resisting mechanism of the damaged frame. An alternative approach was to use the virtual work done associated with the bending moment to investigate the possible resisting mechanism of the damaged frame.

When applying a point load of 1370kN to the damaged frame, the input energy (W_{input}) associated with displacement (δ) caused by the point load should equal the force times the distance ($F\times\delta$), and the internal energy of this structural system can be expressed by the work done ($W_{internal}$) relating plastic hinges through rotations along both frames($W_{pin,\theta}$, $W_{rigid,\theta}$).

It needs to be noticed that when considering the rotation there is no virtual work done along the pin frame ($W_{pin,\theta}=0$). On the other hand, the virtual work ($W_{rigid,\theta}$) of the rigid frame should be a sum of 6 plastic moments (M_p= 3x479 +3x537=3048kNm). The bending moment in the rigid frame can be found in Figure 5-16.

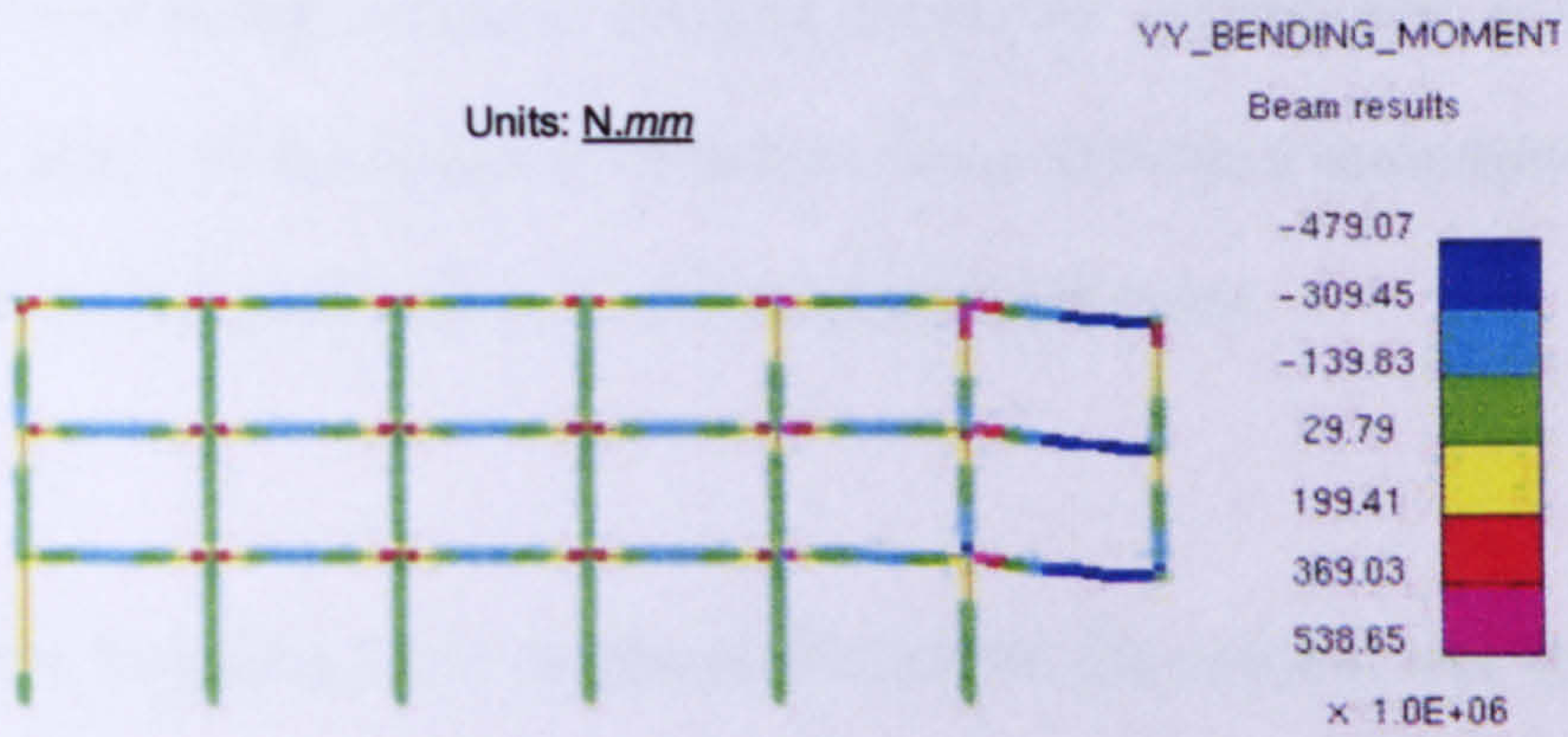


Figure 5-16 Bending moment of the damaged elevation when column ©① was removed in second at loading level of $1.4g_k+1.8q_k$

The input energy (W_{input}) should be equalled to the internal energy ($W_{internal}$), and calculation can be found as follow:

$$W_{input} = 3(F\delta) = 3\times1370\times0.67 \approx 2754\ kJ;$$

$$W_{internal} = 6M_p\times\theta = 6\times3048\times0.11 \approx 2012\ kJ;$$

Where

$\delta = 670mm$ (see Figure 5-13)

$\theta = 670/6000 \Rightarrow \theta = 0.11\ rad$

Based on the calculation above, it was found that the magnitude of the two types of energy (W_{input}, $W_{internal}$) due to a column removed are in a similar range. It is acknowledged that the strain energy stored either in bending or axial extension/compression is not included in the calculation. From an energy balance point of view, the internal energy (that caused by input energy) should be a sum of three parts: 1) work done at plastic hinges; 2) strain energy stored in elastic bending; 3) strain energy stored in axial extension or compression. Among these three parts, the majority of the energy is absorbed by the plastic hinges. The details associated with virtual work/energy stored in bending, extension/ compression are not a major task of this study, so the simple calculations above provide a reasonable indication of, rather than a full explanation of, the resisting mechanism.

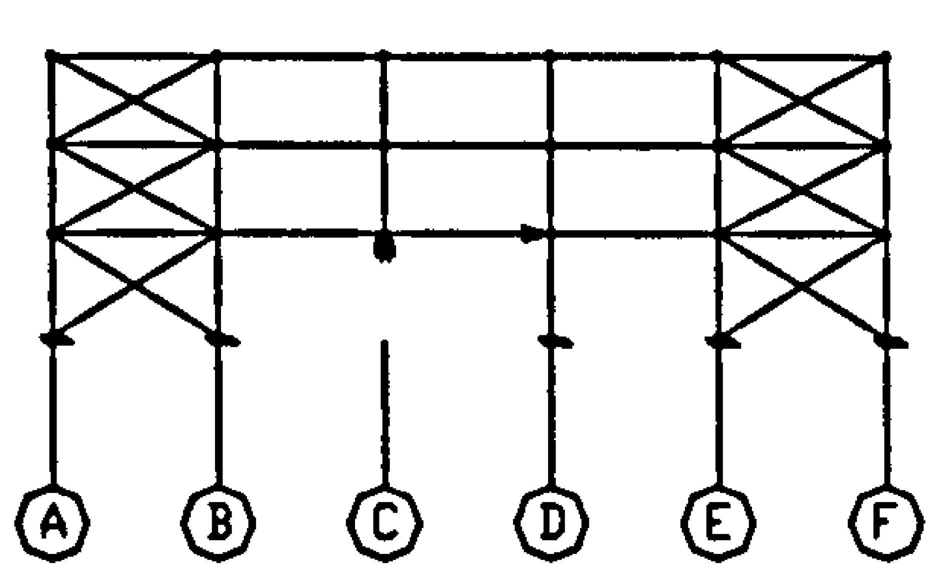
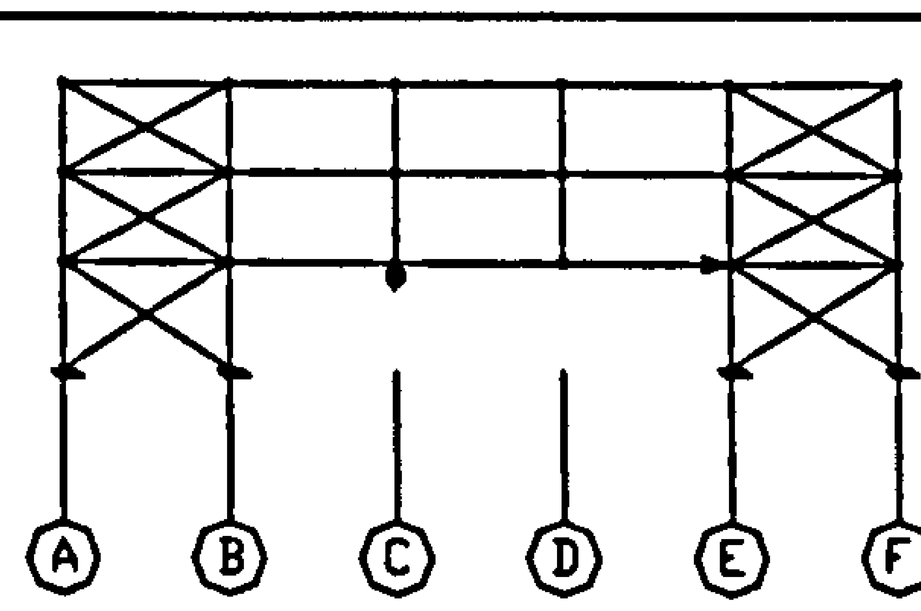
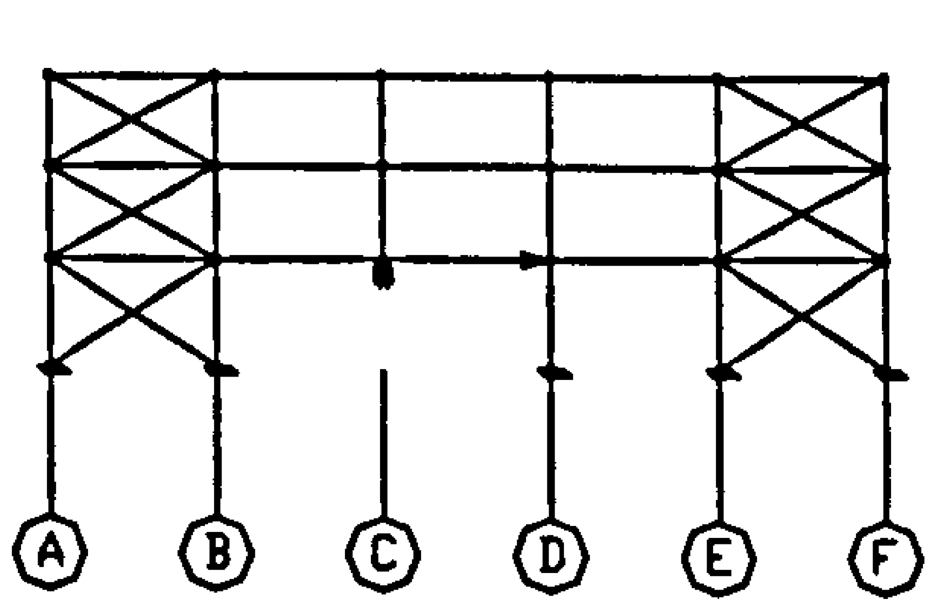
Based on the foregoing, it is suggested that when the column was removed, the damaged 3D structure was supported by the rigid frame (along gridline-Ⓒ) through frame action according to the continuity of the rigid connections. When hinges formed in the rigid frame, the catenary action takes place in the pin frame (along gridline-①) to support the damaged building. If deformation keeps developing, this damaged frame would collapse when the material failed. Therefore, it can be concluded that the resisting mechanism of this 3D pin-rigid frame is a combination of frame action and some catenary action.

5.3.2.5 Number of columns removed

If the resisting mechanism of the damaged frame is a combination of effects, then the location of the column to be removed is not important. As the 3D frame is a symmetric building, if only one column is removed at a time but at different locations the overall structural response should be similar. Probably the most dangerous place for column removal would be the four corner columns, but this study aims to examine the tying forces generated during progressive collapse and not find out the worst structural damage caused by column removal, so no further investigation about this point has been undertaken.

If two columns were removed from different locations, then the structural behaviour would be different. Numerical tests of two adjacent columns removed simultaneously were conducted at a loading level of 61kN/m ($\equiv 1.0g_k+1.0q_k$). Details of the tests can be found in Table 5-4.

Table 5-4 Summary of numerical tests conducted for numbers of columns to be removed

<table>
<tr><td>One column removed</td><td>Case1 one column UC3 (Ⓒ①) was removed</td><td>
</td><td rowspan="3">Note:
•
The point of Displacement output
►
The element of tying force output</td></tr>
<tr><td rowspan="2">Two columns removed</td><td>Case2.1 two columns (Ⓒ①& Ⓓ①) were removed</td><td>
</td></tr>
<tr><td>Case2.2 column UC3 (Ⓒ①) and UC4 (Ⓒ②) were removed</td><td>
</td></tr>
</table>

Theoretically, the results of case 2.1 compared with case 1 should not show much difference, because, as discussed earlier, the major loads were borne by the primary beams (UB1 and UB2). Those two columns are located along the pin elevation, so it should not affect the structural behaviour in the rigid gridlines. On the other hand, differences should be readily observed between the case 2.1 and case 2.2, as in the latter the columns were both located on the rigid gridlines, (i.e. into the plane of the page in Table 5-4) therefore the loading capacity would be reduced.

Among the three cases, LS-DYNA gives the worst structural response for case 2.2 predicting the average tying force for edge ties *(UB3) of 1150kN* and the maximum vertical displacement of 940mm (see Figure 5-17 and Figure 5-18).

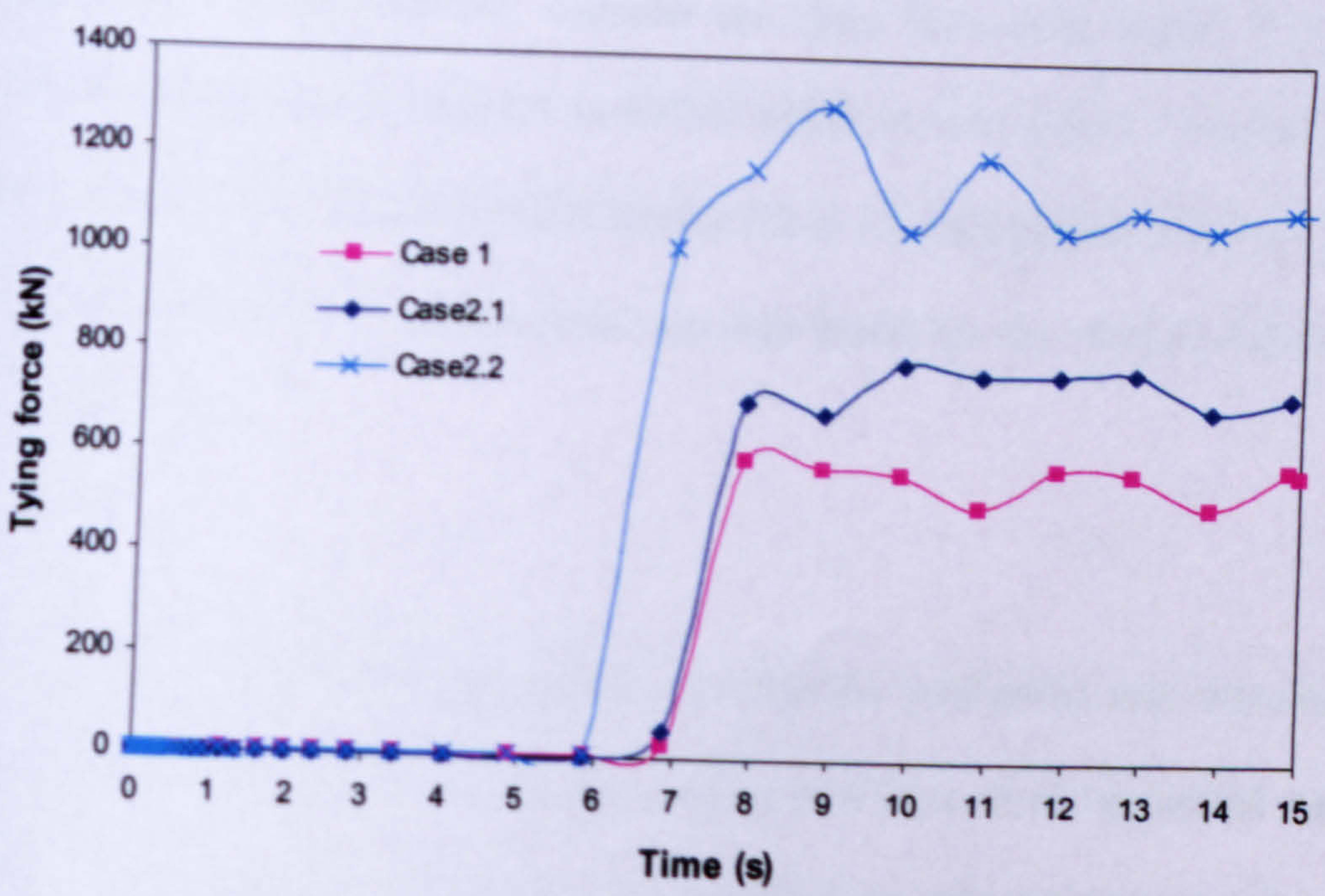


Figure 5-17 Illustration of tying force when column(s) removed in 1 second from the pin-rigid frame

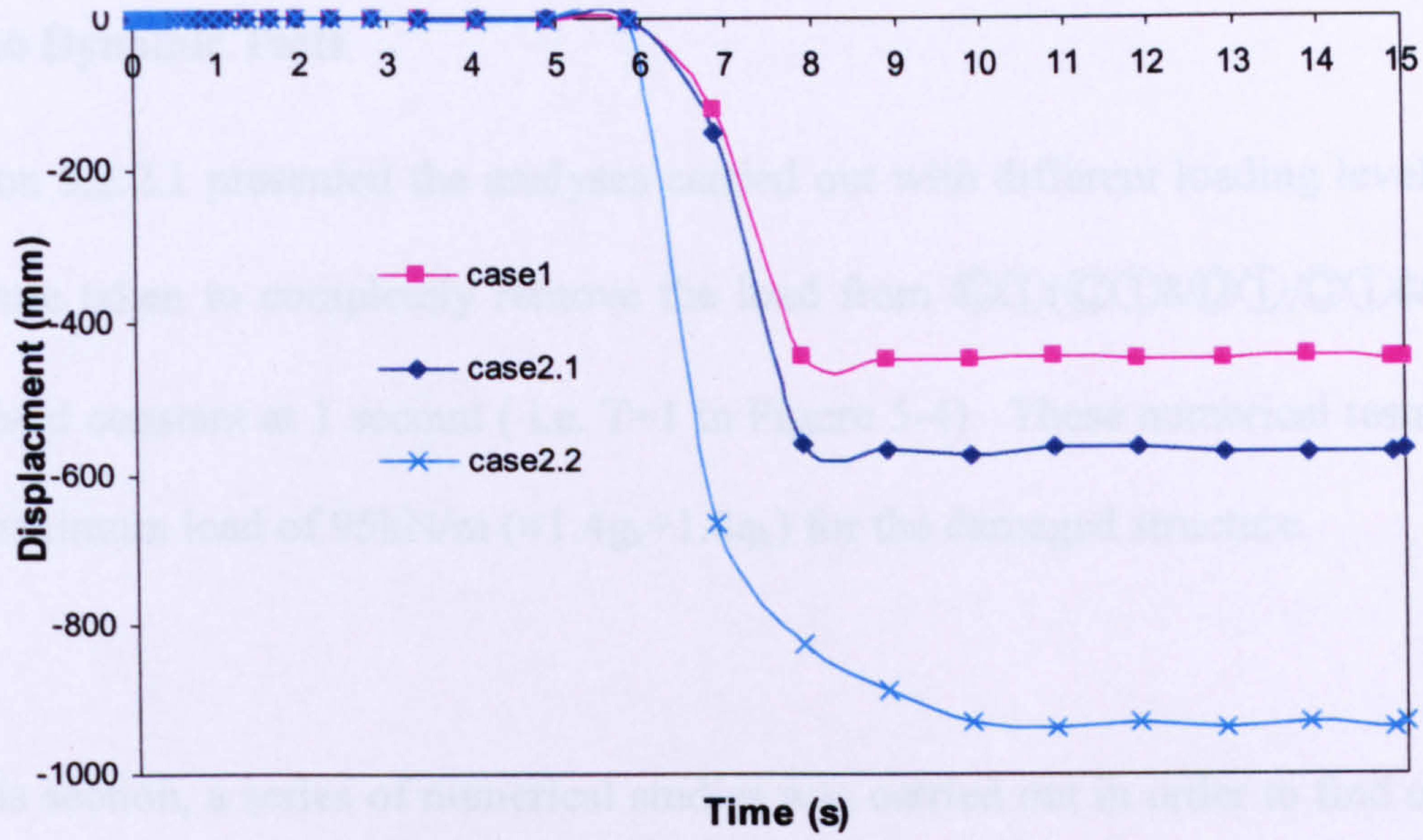


Figure 5-18 Illustration of displacement when column(s) removed in 1 second from pin-rigid frame

Clearly, the difference between case1 and case 2.1 is less obvious compared with the case 2.2. As an example, consider the tying force (see Figure 5-17). The maximum tying force in case 2.1 is 800kN which is 33% greater compared to the 600kN of the case 1, whilst 1300kN in case 2.2 is 1.6 time greater than case 2.1 and twice that of case 1. A similar trend can also found for the vertical displacement (see Figure 5-18).

This section has presented the results of when the column(s) was removed in 1 second. It is interesting to find out the influence of time on the structural response, therefore, the numerical tests in the following section were conducted with varying column removal time.

5.3.2.6 Dynamic Tests

Section 5.2.2.1 presented the analyses carried out with different loading levels and the time taken to completely remove the load from Ⓒ①(Ⓒ①&Ⓓ①,Ⓒ①&Ⓒ②) was held constant at 1 second (i.e. T=1 in Figure 5-4). These numerical tests gave the maximum load of 95kN/m ($\equiv 1.4g_k+1.8q_k$) for the damaged structure.

In this section, a series of numerical studies was carried out in order to find out the structural behaviour when the time to remove the column Ⓒ① (T=0.001, 0.01, 0.1, 1 as illustrated in Figure 5-4) was varied. The loading regime [SCI P-244, Blast and Ballistic Loading of structures] (Impulsive, Dynamic, Quasi-static) in terms of the natural period of the structure is not part of this study. This study focuses on the structural behaviour *after* the supporting element has been removed. Varying the column removal time allowed investigation of the dynamic effects on the remaining structure.

The following tests were carried out at the loading level of 56kN/m ($\equiv 1.0g_k+0.8q_k$), and the results are presented in Figure 5-19. The maximum displacement when the column was removed in 1 second was 320mm, and 460mm for removal in 1 millisecond.

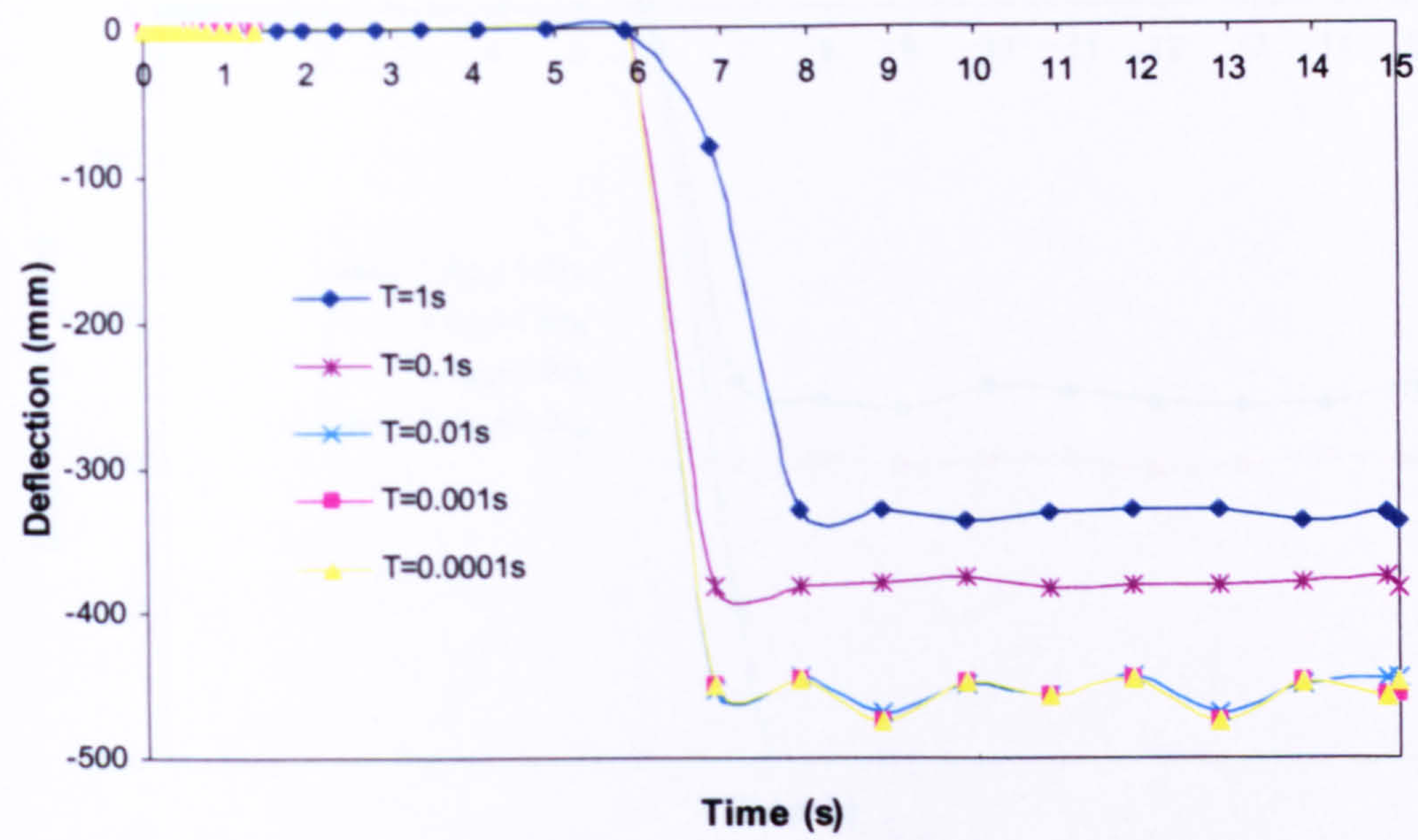


Figure 5-19 Displacement Vs time when column ©① was removed in different times from the pin-rigid frame at a loading level of $1.0g_k+0.8q_k$

The results presented in Figure 5-19 show that time is an important factor affecting the structural performance. When column ©① was removed faster than 1 millisecond, the results showed no difference. For instance, LS-DYNA predicted the same vertical displacement (average) of 460mm for the removal time equal to 0.01ms, 0.1ms and 1 ms. When the column was removed in 1 millisecond (or 0.1ms, 0.01ms) the structure had to develop about 44% more deformation to reach the equilibrium than was the case when the column was removed over 1 second. This shows that the structural response is a dynamic process.

To determine the maximum loading level of the damaged structure when the column was removed in 1 millisecond, a set of numerical tests were conducted with column ©① removed from frames at different loading levels, the results are presented in Figure 5-20.

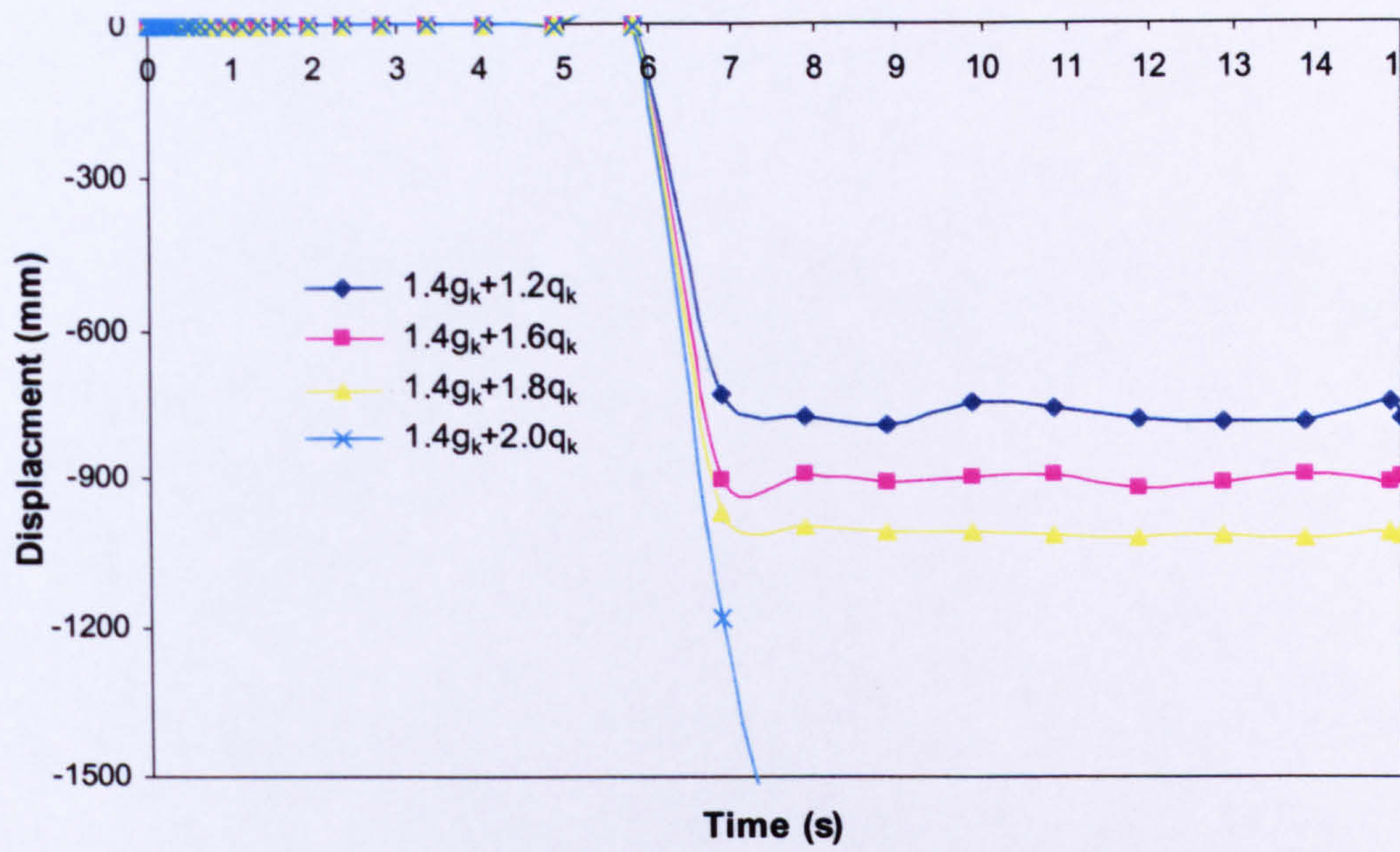


Figure 5-20 Displacement Vs time when column ©① was removed from the pin-rigid frame in 0.001 second with different loading levels

In Figure 5-20, LS-DYNA has shown that the maximum loading level that can be sustained when the column was removed in 1 millisecond compares well to when it was removed in 1 second, that is $1.4g_k+1.8q_k$ ($=95kN/m=0.5G'_{edge}$). However, the damaged structure behaves very differently with different removal times. When the column was removed in 1 millisecond, the damaged structure had a deflection of 1000mm compared to 670mm when column was removed in 1 second. This means that the damaged structure has to deform an extra 49% to reach equilibrium. The tying force is also affected by the column removal time (see Figure 5-21). When the column was removed in 1 millisecond, LS-DYNA predicted 1120kN for the maximum tying force and 1060kN for the average. This compares to the case of 1 second, where the increment is 22% and 27% respectively.

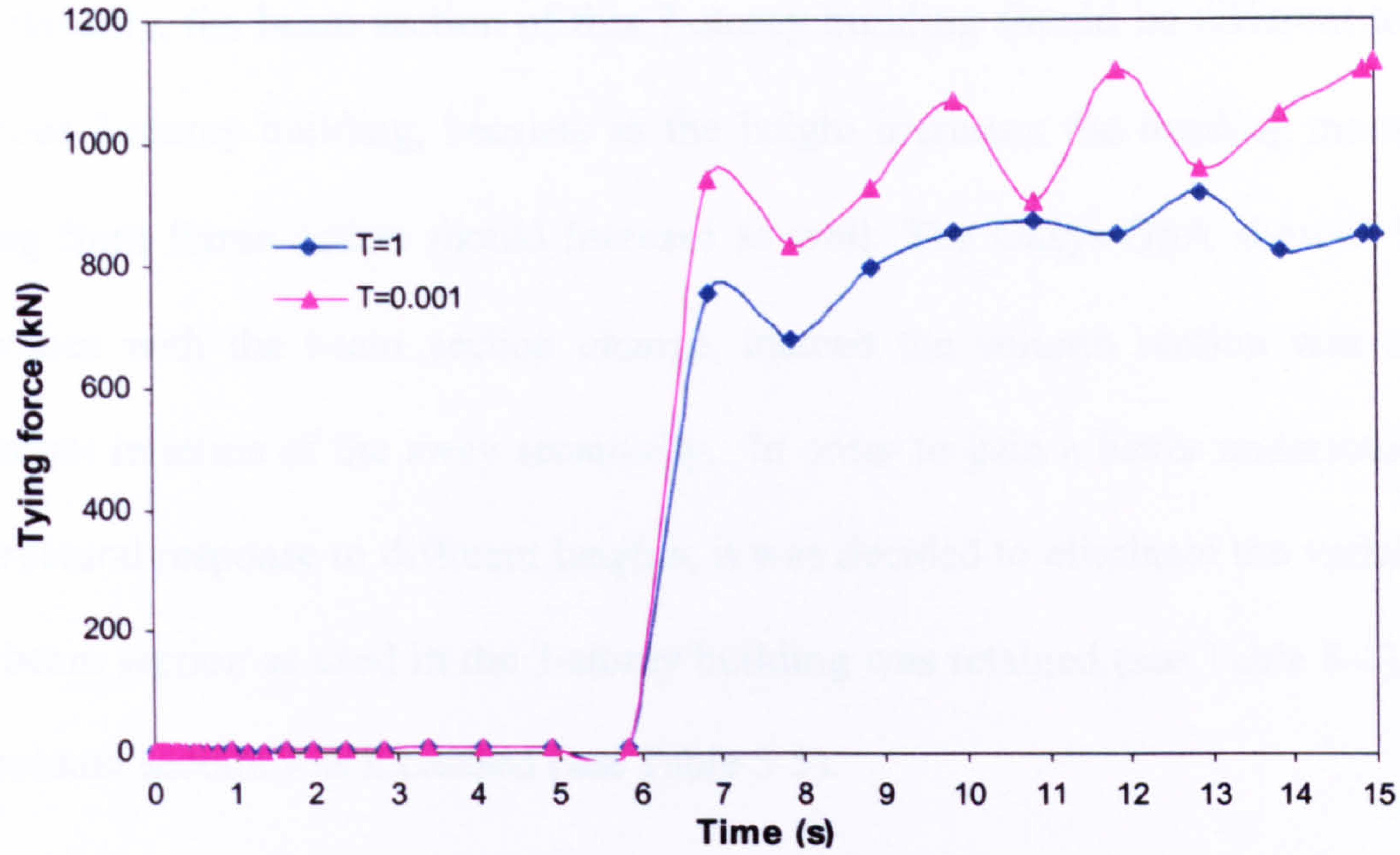


Figure 5-21 Comparison of tying force at loading level of $1.4g_k$+$1.8g_k$ with different column removal time

According to the above results, it was observed that the damaged structure has a different response (e.g. peak tying force depending on the time taken to remove the column load), which highlights that progressive collapse is a dynamic issue. Therefore, although it is possible to use static approaches to evaluate the structural behaviour this is not as accurate as a more rigorous dynamic analysis.

5.3.2.7 Height Effects

This part of the study was an investigation of building height effects on the structural behaviour during a progressive collapse. A 7-storey frame, sharing the same plan layout, structural form and design loading as previously (see Figure 5-22) was designed according to BS5950 [BSI, 2000].

Theoretically, the beam section of this 7-storey building should be different to the previous 3-storey building, because as the height increases the bending moments arising from frame action should increase as well. The Oasys-GSA showed little difference with the beam section change, instead the column section was more important in terms of the sway sensitivity. In order to gain a better understanding of structural response to different heights, it was decided to eliminate the variables. The beam section as used in the 3-storey building was retained (see Table 5-1), but the column section was increased (see Table 5-5).

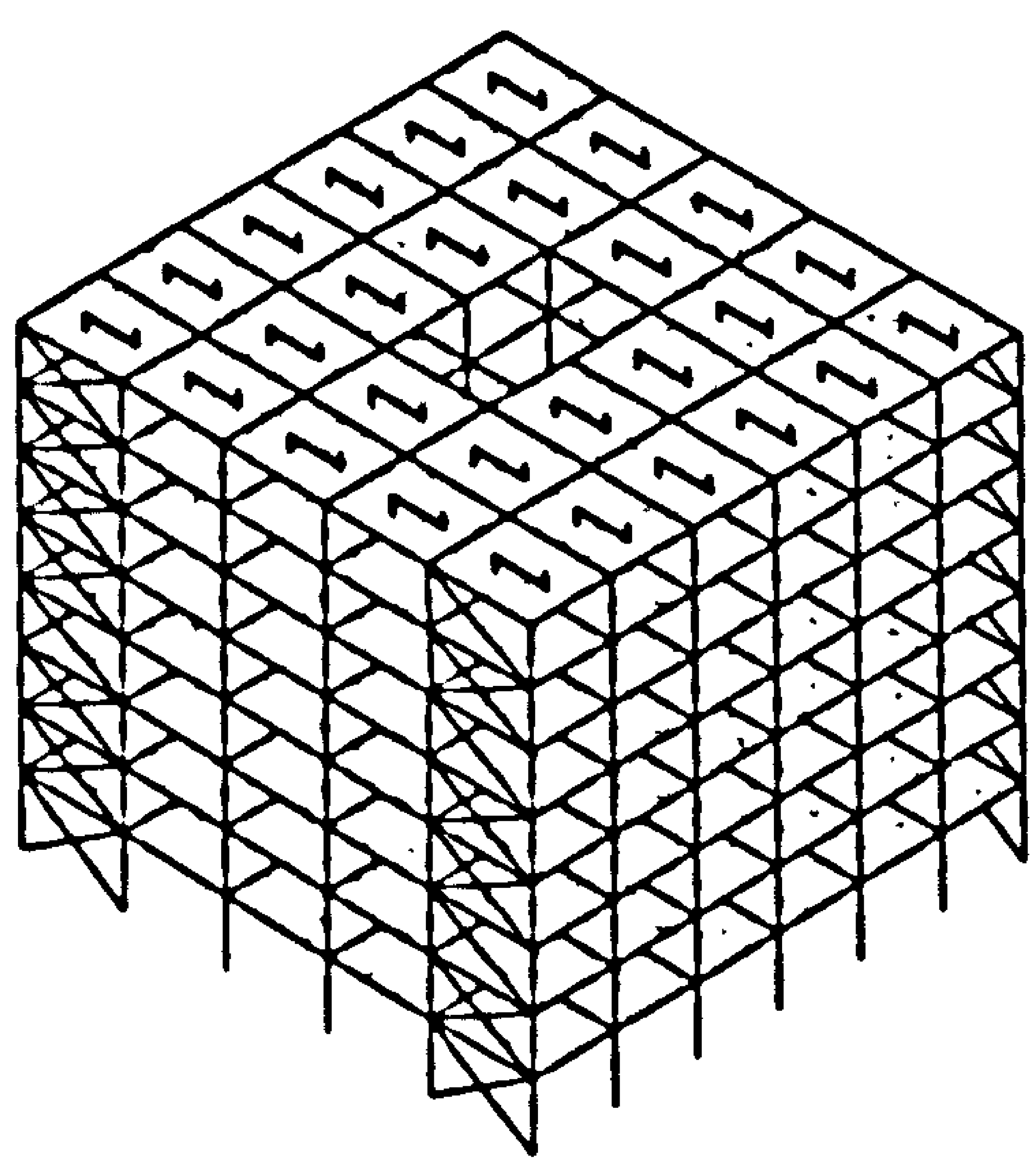


Table 5-5 Column size (pin-rigid)

UC1-UC4	356x406x393 1-3 356x368x177 4-6 305x305x118 7

Figure 5-22 3D Geometry of 7-Storey building

The purpose of the study of this 7-storey building (Figure 5-22) was to check whether the number of stories would affect the resisting mechanism of the damaged building. Therefore, the edge column ©① was removed and examined first. When column ©① was removed in 1 second, LS-DYNA predicted the tying forces shown in Figure 5-23.

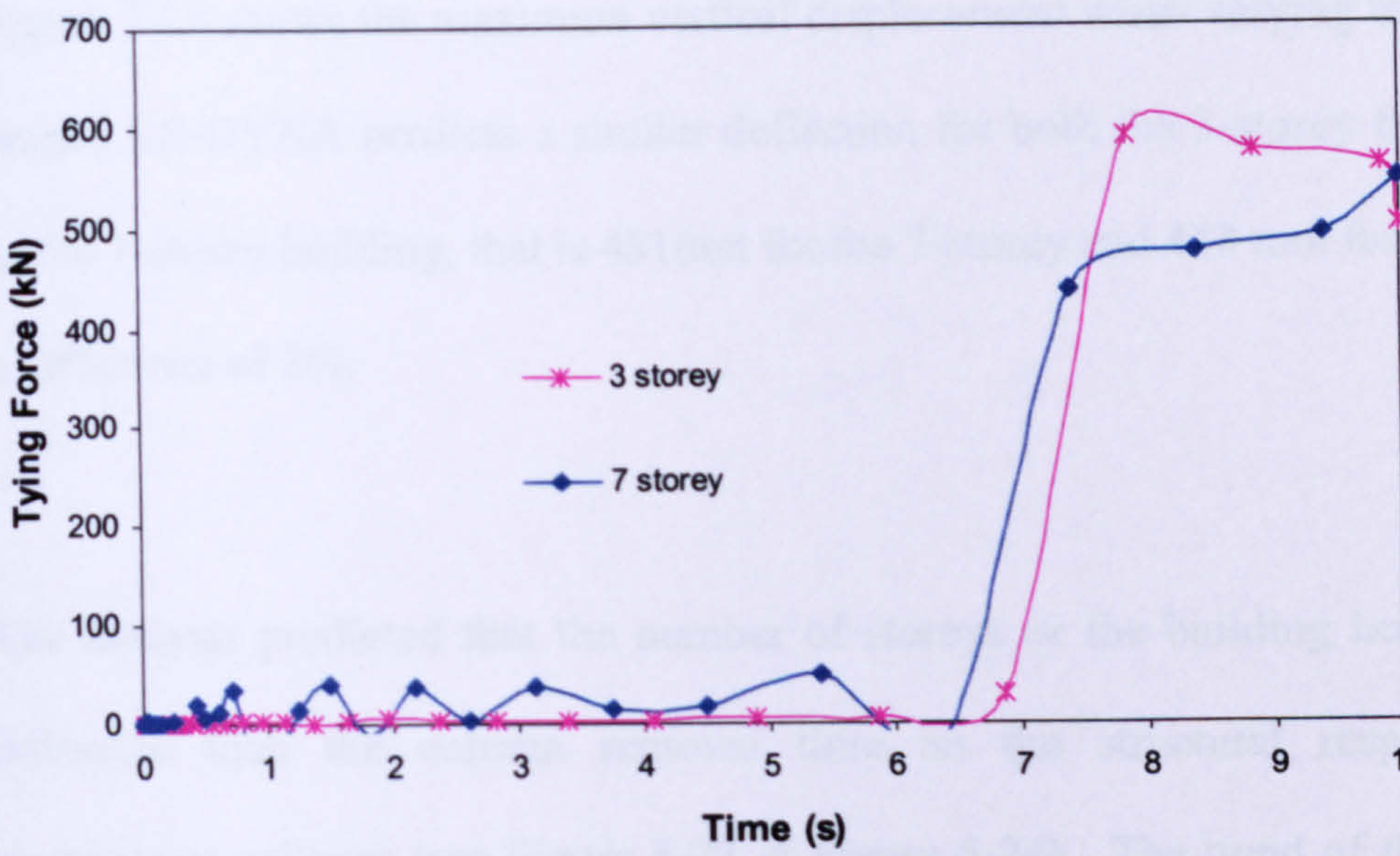


Figure 5-23 Comparison of the tying force between 3-storey and 7-storey pin-rigid buildings when the column was removed in 1 second at a loading level of 95kN/m (=1.4g_k+1.8q_k)

In Figure 5-23, the *peak* tying force for both buildings (3-storey and 7-storey) is similar, that is 610kN for the 3-storey and 550kN for the 7-storey building. Because of limitations of calculation time, the numerical tests were not allowed to have a very long analysis time, therefore the results only give an impression of the trends rather than a definitive answer.

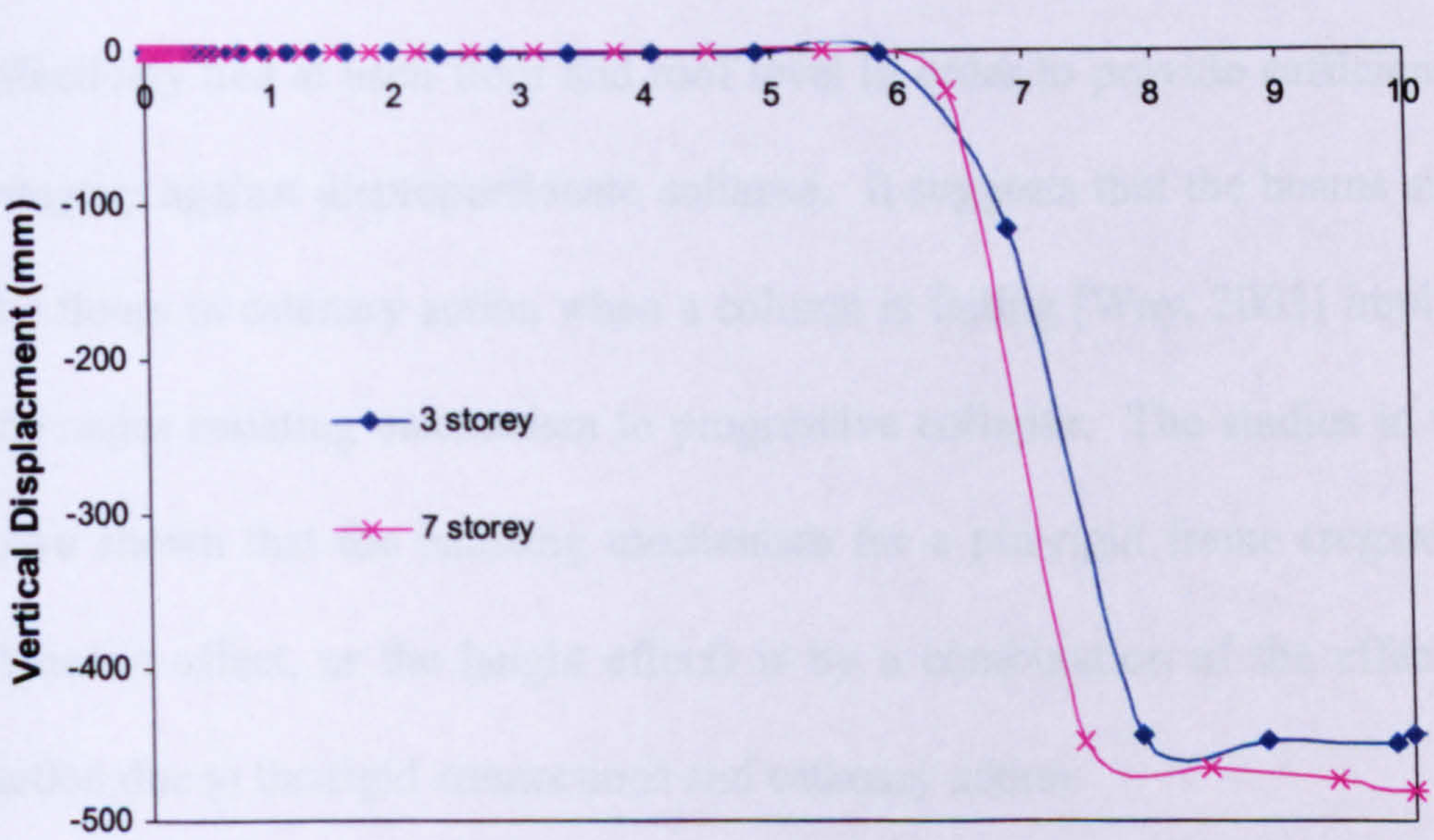


Figure 5-24 Comparison of the displacement between 3-storey and 7-storey buildings when the column was removed in 1 second at a loading level of 95kN/m (=1.4g_k+1.8q_k)

Figure 5-24 shows the maximum vertical displacement when varying the building's height. LS-DYNA predicts a similar deflection for both the 3-storey frame as well as the 7-storey building, that is 481mm for the 7-storey and 444 mm for the 3-storey; a difference of 8%.

The analysis predicted that the number of storeys or the building height has less influence than the column removal time on the structural response during progressive collapse (see Figure 5-23 & Figure 5-24). The trend of the maximum tying force shows that as the height increases the maximum tying force decreases, but the amount is not great (10%). A similar trend can be observed for the vertical displacement.

5.3.3 Discussion

The design guidelines in BS5950 recommend that structural members should be effectively tied at each floor and roof level in order to provide sufficient structural integrity against disproportionate collapse. It suggests that the beams should carry the floors in catenary action when a column is failing [Way, 2003] implying this is the major resisting mechanism to progressive collapse. The studies in this section have shown that the resisting mechanism for a pin-rigid frame (regardless of the dynamic effect, or the height effect) is by a combination of the effects of frame action due to the rigid connections and catenary action.

According to BS5950, the loading level of γ_f[1] $(1.0g_k+0.33q_k)$ is recommended when considering the columns notionally removed. The evidence has proven that this *accidental load* level is normally around 40%-60% of the collapse loading level of an undamaged frame. The numerical results from LS-DYNA have shown that the damaged frame can not stand up with such a high load ratio λ[2] (40-60%). Instead the damaged structure can only stand up with load ratio λ about 20%.

5.4 Modelling a pin-pin frame

5.4.1 Introduction

UK steelwork construction practice makes extensive use of *simple design*. This section introduces a 3-storey high building that was designed as a *simple* frame (in both directions) according to current UK steel design practice. Apart from that, this pin-pin frame shared the same outline and loads as the previous pin-rigid frame (see Figure 5-6). The geometry is shown in Figure 5-25 and Figure 5-26, and the member structural sizes are given in Table 5-6.

[1] $\gamma_f=1.05$

[2] load ratio $\lambda = \dfrac{accidental}{collapse}$

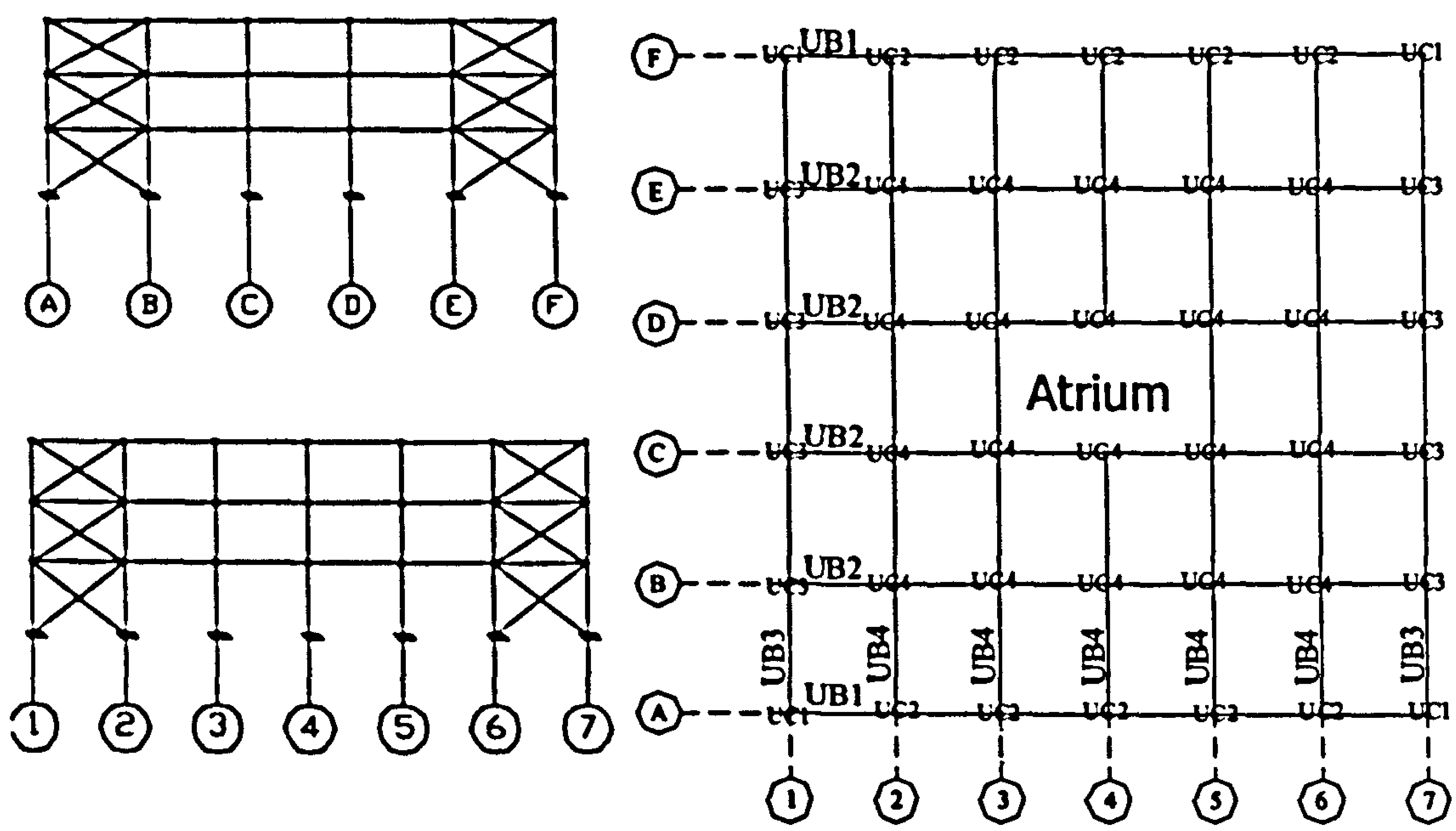


Figure 5-25 Outline for pin-pin frame

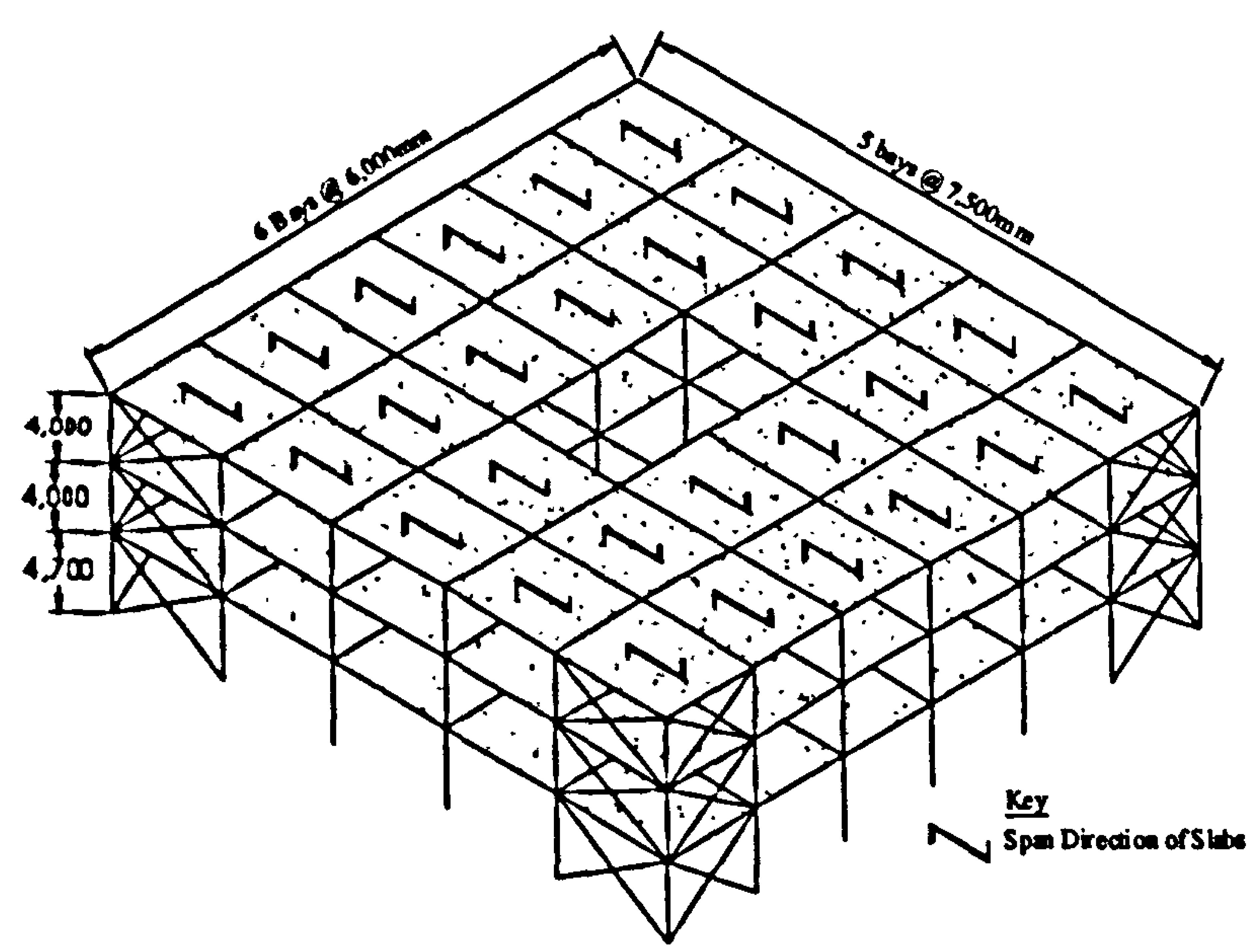


Figure 5-26 Arrangement of 3D Pin-pin test frame

Table 5-6 Member section sizes for the 3-storey pin-pin frame

	Beam		column	
	Roof	Floor		
UB1	356x171UB57	457x191UB74	UC1	256UC107
UB2	457x152UB60	457x191UB89	UC2	256UC107
UB3	305x127UB42	457x152UB67	UC3	256UC107
UB4	305x127UB42	457x152UB67	UC4	305UC118

This frame is braced in both directions; therefore it is a braced non-sway frame. The details of the design procedure can be found in Appendix-C.

For this pin-pin frame, it is difficult to model its behaviour without involving other modelling issues. In detail, the biggest challenge is to prevent primary beams buckling. This is not an issue in reality, as it can easily be solved by the lateral restraint provided by the pre-cast units on the top of the beams, but it is a problem for numerical simulation. The pin-link cannot stop the primary beams buckling. Other attempts to solve the problem such as a change to the beam's material properties so that the beam takes more compression cannot entirely solve this buckling problem without causing other numerical problems. Finally, it was decided to use pseudo beams for the primaries, in which the section was artificially increased about the minor axis second moment of area of the section but the correct cross-sectional area was retained.

Clearly, this pseudo beam section would also increase the bending resistance of the primary beams about the minor axis ($M_{y'y'}$) compared to the original design, but as discussed earlier, the major concern about this research is to find out the resisting mechanism in a damaged frame and also to check whether the tying strategy can safeguard the damaged building against progressive collapse. This pseudo beam approach is the best compromise to solve the buckling problem whilst not interrupting the load distribution route.

When applying the pseudo section to the primary beams, it was found that the magnitude of increment of the second moment of area of the cross section is important. If the second moment of area was increased more than 5 times greater than its original, then this affected the building's failure loading level. On the other hand, if it was increased by less than 2 times the original, then it could not prevent the primary beams buckling. Finally, it was decided to double the second moment of area for the 3-storey building and increased it by 5 times for the large 7-storey building.

The formulation of the Hughes-Liu integration beam is very efficient, in other words it saves a lot of CPU (calculation) time; therefore it was applied to model the beams as well as the columns of the previous 3D pin-rigid frame. However, the HL integration beam has difficulty in predicting a simple supported column that buckles about its minor axis (see chapter 3). Accordingly, a more sophisticated beam formulation (BS integration) was used to model the columns in pin-pin frame.

The difference between those two types of integration beams (BS integration and HL integration) is not great (see Figure 5-27). Figure 5-27 presents a comparison test with different integration beams when column ©① was removed from the 3D pin-rigid frame (see Figure 5-6 and Figure 5-7) in one second at a loading level of $1.4g_k + 1.6q_k$ (see also Figure 5-12).

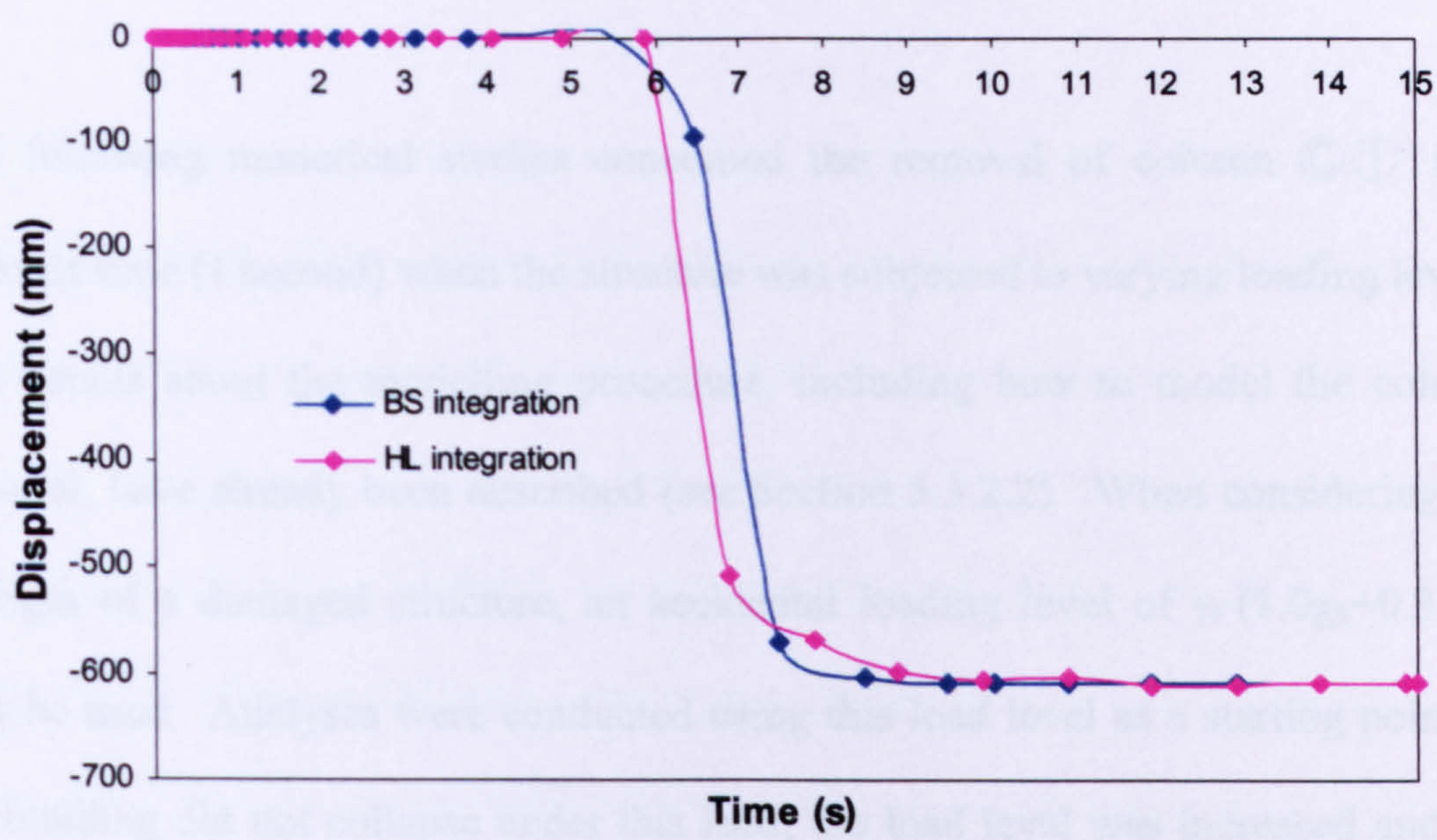


Figure 5-27 Comparison of displacement using BS and HL integration beam when column was removed in 1second at loading level of $1.4g_k$+$1.6q_k$ from the 3D pin-rigid frame

The results clearly show that the two different integrated beams predict the same final displacement of 610mm when the damaged building reaches equilibrium (t>=10s). Therefore, the previous results that were conducted using HL integration beams for columns in the 3D pin-rigid frame are still valid, and there appears to be no need for the further analyses to be repeated.

5.4.2 Numerical Analysis

5.4.2.1 Loading Level Tests

For the pin-pin frame, the undamaged building collapsed at a loading level of $1.4g_k+1.8q_k$ due to column failure, compared to the designed level of $1.4g_k+1.6q_k$, which appears reasonable.

The following numerical studies concerned the removal of column ©① in a constant time (1 second) when the structure was subjected to varying loading levels. The details about the modelling procedure, including how to model the column removal, have already been described (see Section 5.3.2.2). When considering the strength of a damaged structure, an accidental loading level of $\gamma_f\,(1.0g_k+0.33q_k)$ may be used. Analyses were conducted using this load level as a starting point. If the building did not collapse under this load, the load level was increased and the analysis repeated. The details are shown in Figure 5-28.

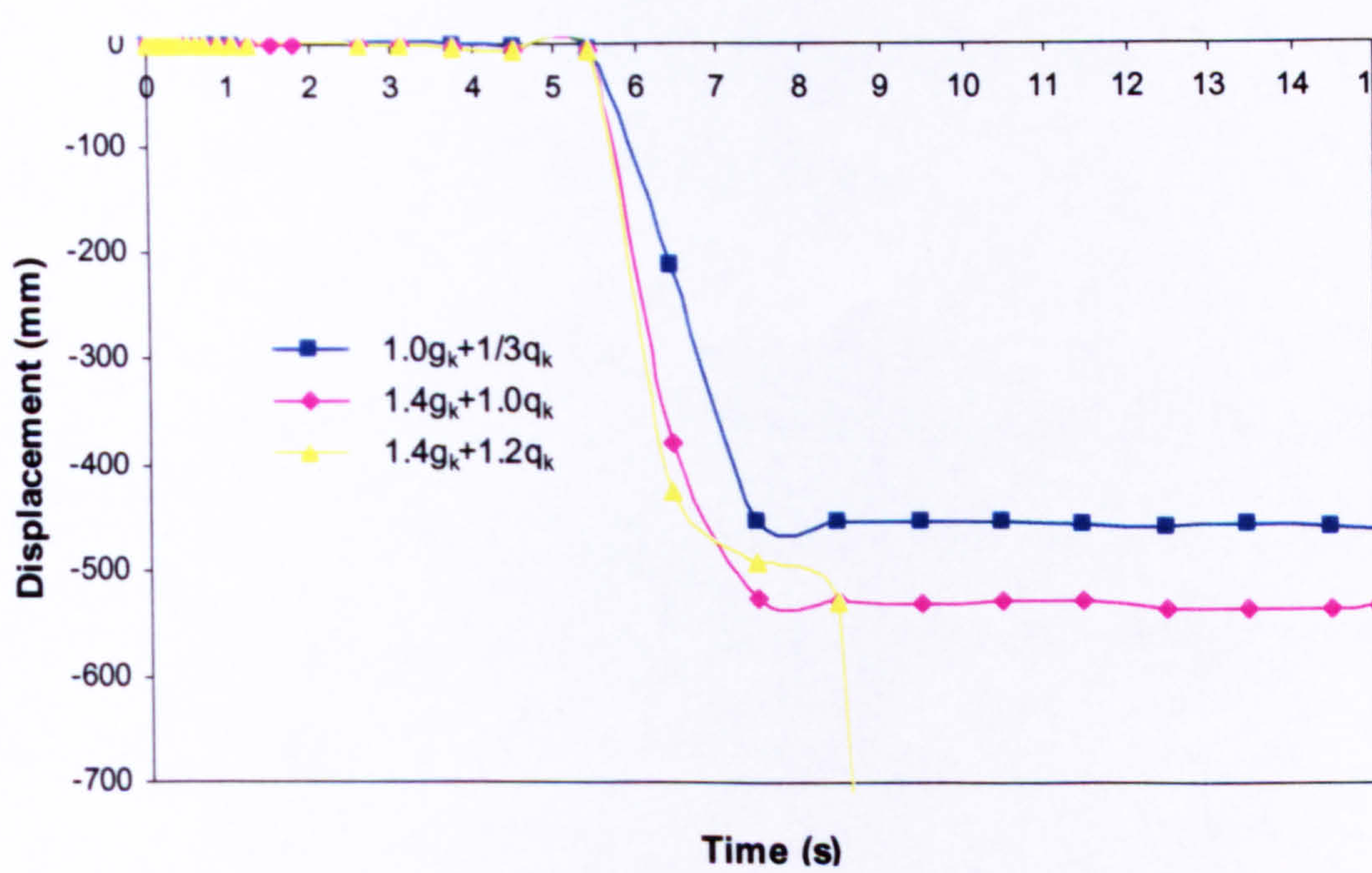


Figure 5-28 Vertical displacement when column ©① was removed in 1 second with different loading levels (3-storey pin-pin frame)

Figure 5-28 shows results from analyses carried out with different loading levels when column ©① was removed over 1 second (starting at time= 6s, see also Figure 5-11). These numerical tests gave the maximum loading level for the damaged structure as$1.4g_k+1.0q_k$. The structure collapsed when the loading level was increased to $1.4g_k+1.2q_k$.

The UK design procedures implemented to avoid progressive collapse normally have three stages [Way, 2003; SCI 98/99, Liu *et al*, 2005] arranged in order of design complexity. Tying members together against progressive collapse is a priority. This part of the research focused on stage 1 (details about the 3 design stages can be found in chapter 2) i.e. determining the magnitude of the tying forces generated in a damaged structure and comparing these to the prescribed design tie force. BS 5950 requires the steel members which act as horizontal ties must be able to resist tensile forces of:

$0.5(1.4g_k+1.6q_k)s_tL$ but not less than 75kN (internal ties) (5.1)

$0.25(1.4g_k+1.6q_k)s_tL$ but not less than 75kN (edge ties) (5.2)

According to this requirement the minimum tying force of the edge ties (UB3) in this study is about 222kN (for design details see Appendix-B). At the accidental loading level of $1.0g_k+0.33q_k$, the resulting tying force of this damaged frame is shown in Figure 5-29.

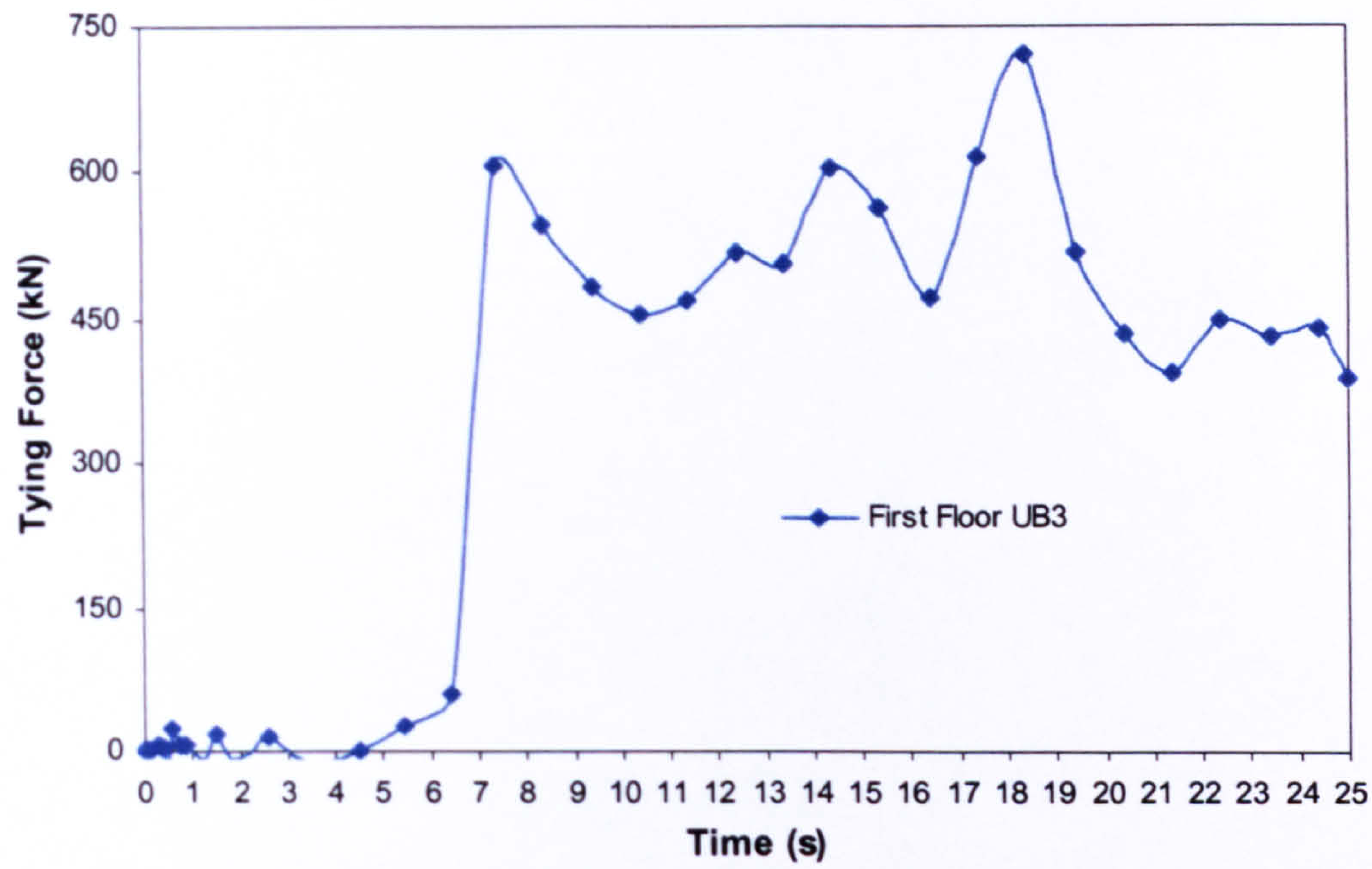


Figure 5-29 Tying force when column ©① was removed in 1 second from a 3-storey pin-pin frame

In Figure 5-29, the FE analysis predicts that the peak tying force is about 720kN (average about 480kN), which is about 2-3 times bigger than the design requirement of 222kN.

In this 3D pin-pin frame, clearly there is no cantilever behaviour (which was possible in the pin-rigid frame) as it was designed with pin connections in both elevations. According to current UK design guidelines, catenary action is the suggested resisting mechanism for this type of structure. It is difficult to conduct on investigation in to this full-scaled 3D pin-pin frame, because this 3D pin-pin frame is a very complicated numerical model that makes the major load re-distribution route less obvious. A small scale model was constructed to improve the understanding of the structural behaviour when column ©① was removed in 1 second. For this small-scale building, its geometry and material properties were kept the same as the original (full-scale) frame. The details of this small-scale model are presented in Figure 5-30.

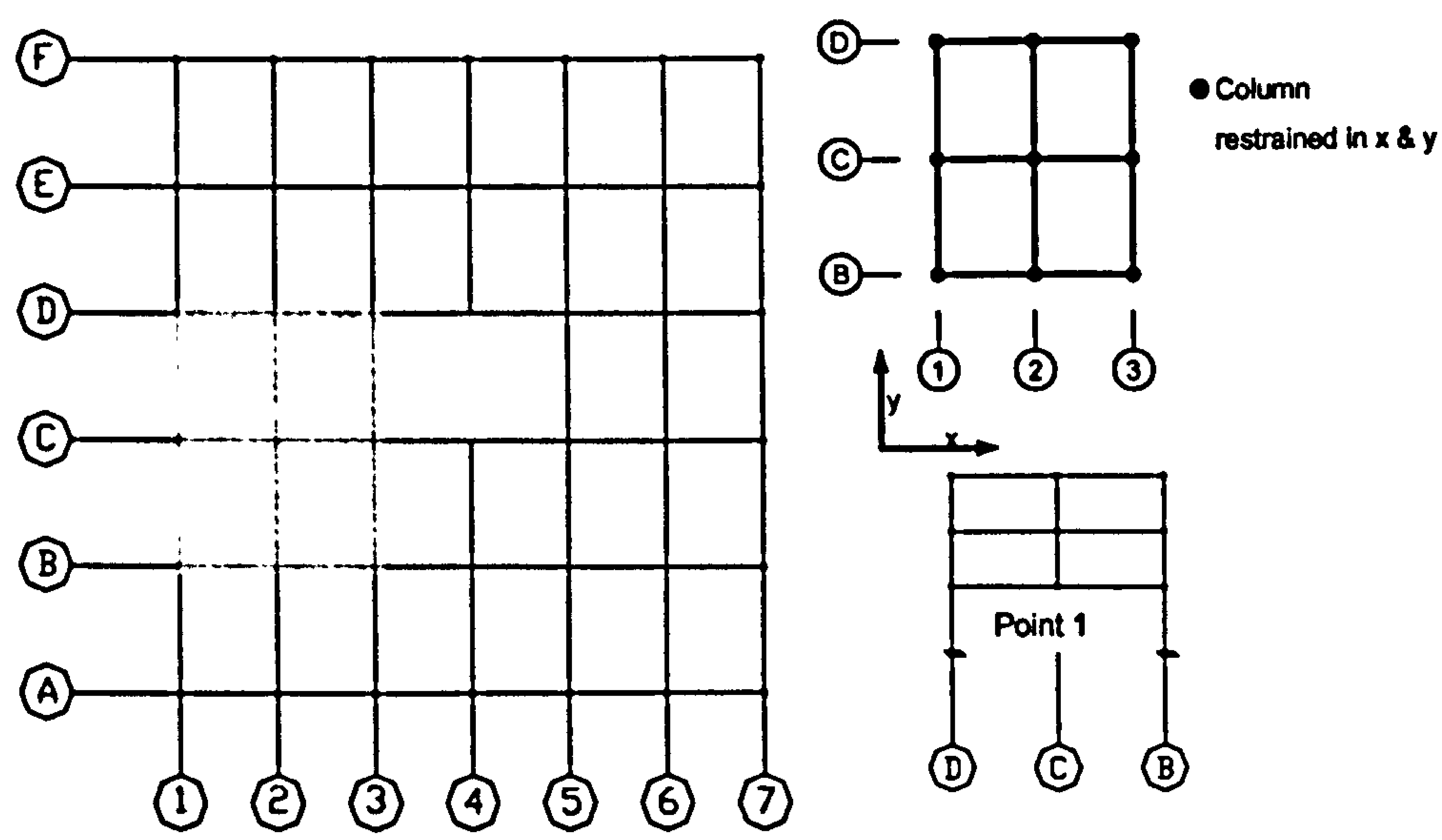


Figure 5-30 Substructure selected from the original 3-storey pin-pin frame

This substructure has limited numerical parameters, so it is relatively easy to follow the analysis in order to predict the resisting mechanism. The philosophy adopted here is if the small building can show some trend which is similar to the full-scaled building, then load re-distribution routes in both buildings should be the same. In the small building, **all columns** were restrained in plan (x and y) at each storey level, to eliminate the sway effects (lack of bracing system). The change of boundary conditions obviously affects the final results and the maximum displacement (at point 1) of this small-scaled building is about 320mm (see Figure 5-31) compared to the 480mm of the full-scale, which is about 33% less.

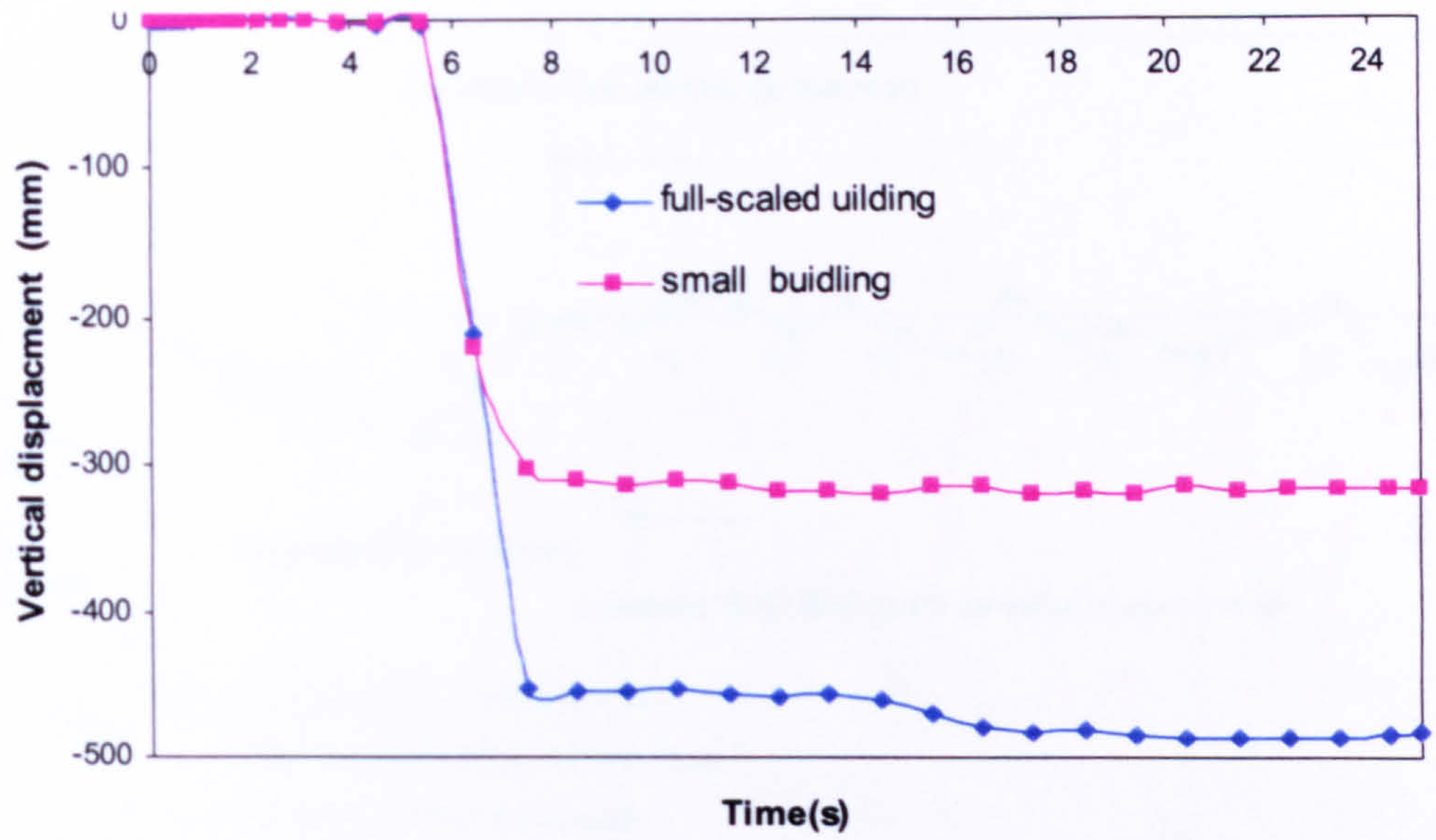


Figure 5-31 Illustration of displacement between substructure and original strucutreal when column ©① was removed in 1 second from the 3-storey pin-pin frame

Because there are so many members in the large frame, it is difficult to identify the load re-distribution route during progressive collapse. On the other hand, the loading re-distribution route in the small building is relatively easy to track as column Ⓑ① and Ⓓ① were restrained (in both x and y direction), which gives the damaged structure a direct load transfer path. The axial forces that were generated in the substructure when column ©① was removed in 1seond are presented in Figure 5-32.

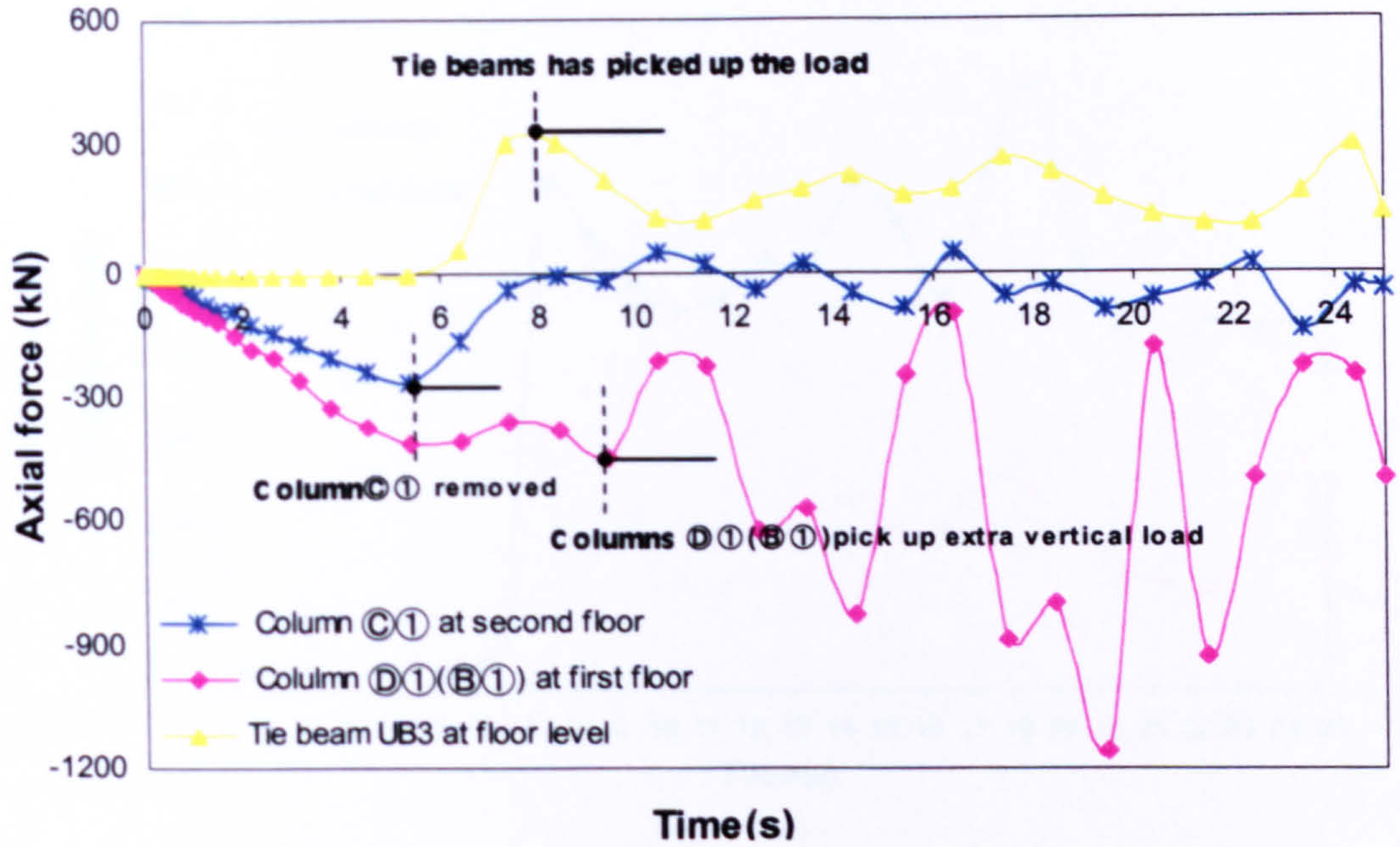


Figure 5-32 The axial force that generated in the damaged substructure

The results from LS-DYNA illustrate the direct load transfer path, i.e. as soon as column Ⓒ① was removed the tie beam UB3 picks up the force (300kN) from the column and bridges this force to the columns Ⓑ① and Ⓓ①.

Figure 5-33 compares the tying forces generated in the small building with those in the larger model notice that the small-scaled frame shows the same trend as the full-scale building (see Figure 5-33). Therefore, it is logical to assume they share the same loading re-distribution route, albeit that the stiffness and strength of the surrounding structure in the larger model is not as great as that provided by the boundaries in the small model.

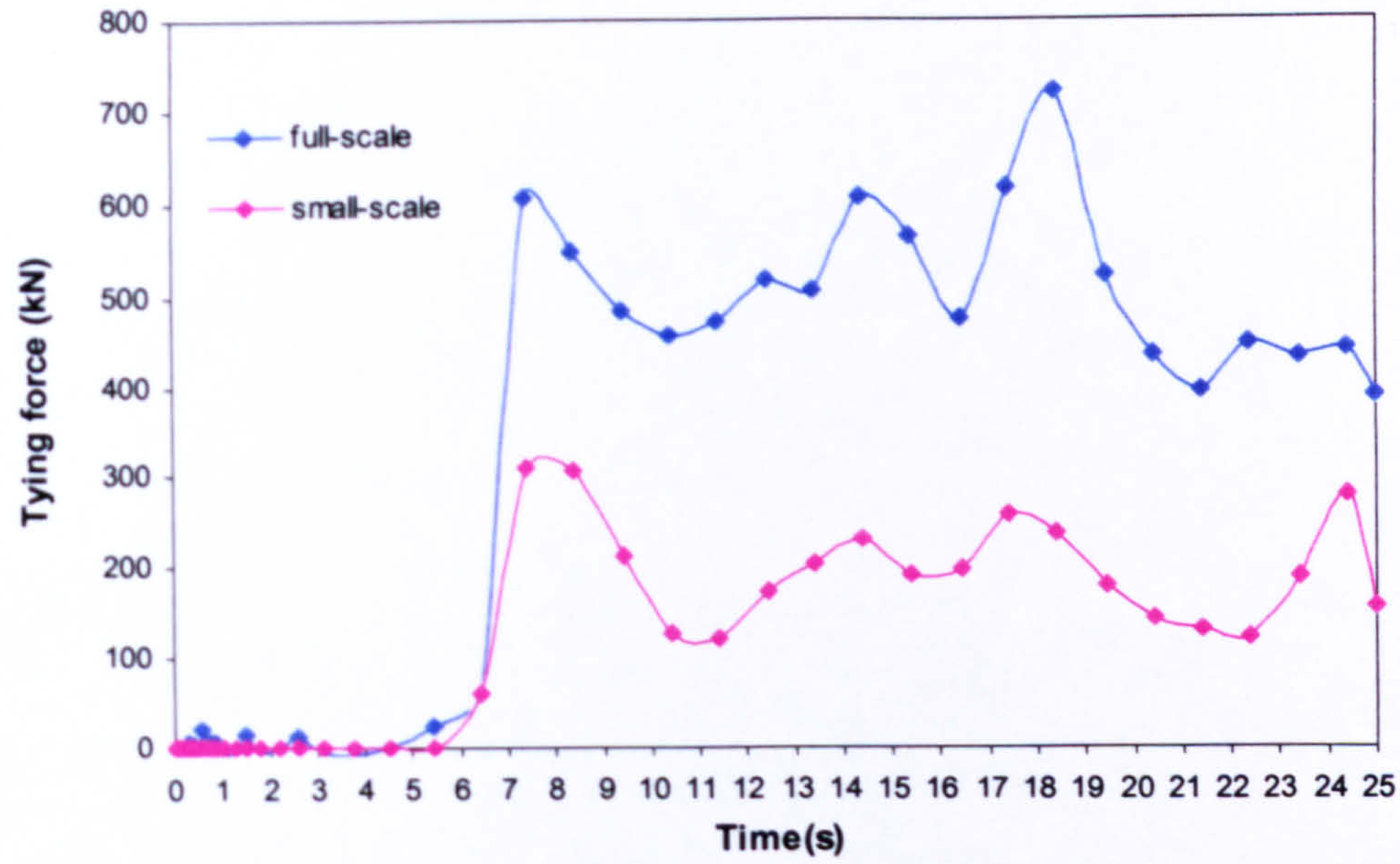


Figure 5-33Comparison of tying forces in the 3-storey full-scale and the small-scale building when the column was removed in 1 second

It is interesting to discover whether the structural performance would be different when the two columns are removed simultaneously from the original 3D building. The numerical tests were again conducted on the full-scale building with columns removed over 1 second (see Figure 5-34).

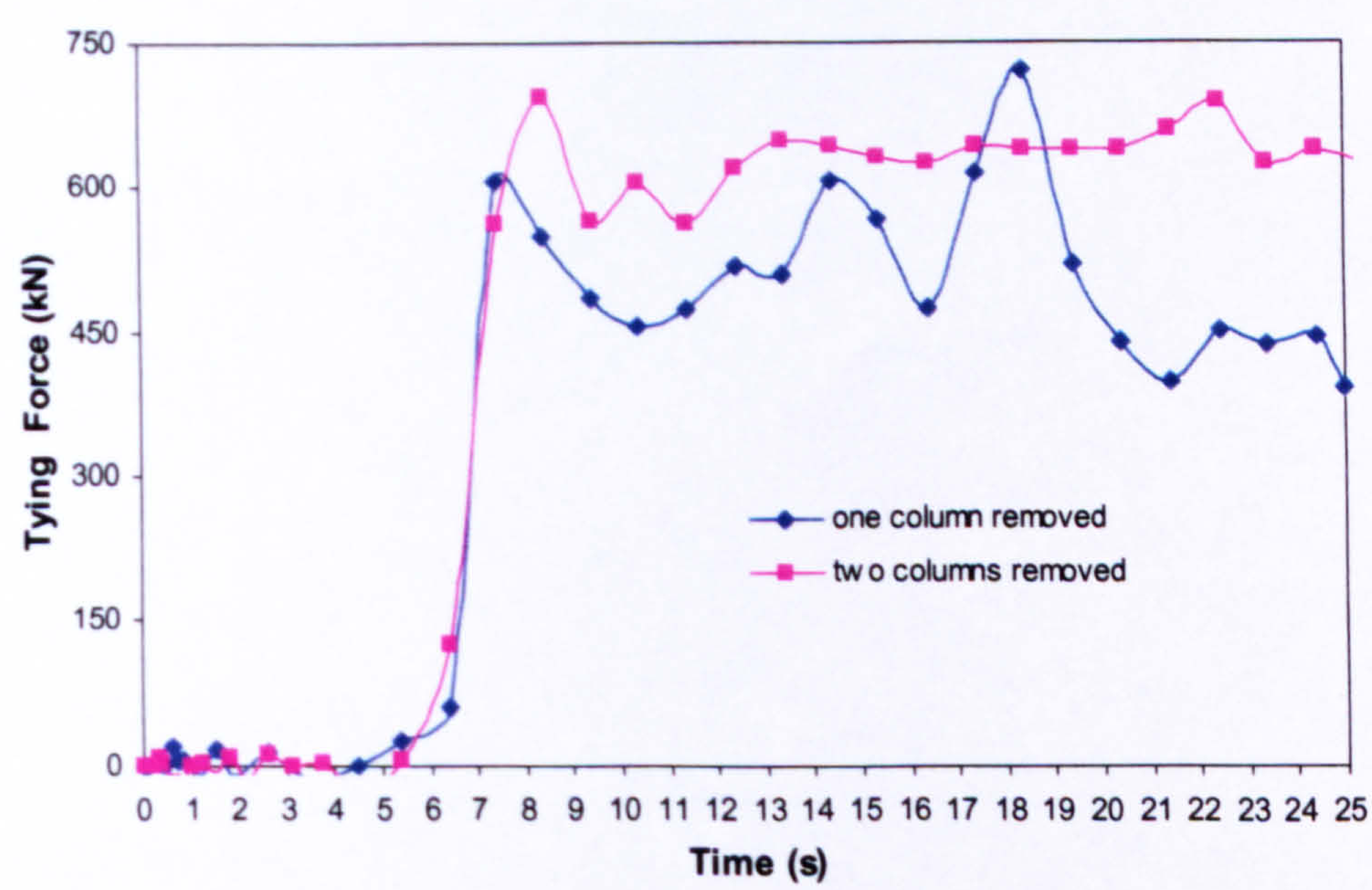


Figure 5-34 Tying forces generated in 3 storey pin-pin frame when one or two column(s) removed in 1 second

It appears that a similar peak tying force of 720kN is induced (albeit with a different time for the generation of the peak force - see Figure 5-34) but the average tying force for the two columns removal is about 650kN which is about 35% bigger compared to the case where just one column was removed.

The results presented in this section have a constant removal time of 1 second. The previous studies of the 3D pin-rigid frames have shown that the speed of column removal is an important factor that affects structural behaviour. The next section will present the results of numerical tests carried out to investigate the influence of column removal time on the tie forces induced in a pin-pin frame.

5.4.2.2 Dynamic tests

This part of the study was conducted to investigate the effect of the rate of column removal on overall structural behaviour. The test loading level is the accidental loading level of $1.0g_k+0.33q_k$. The column(s) removal time was varied between 1 second and 1ms. Figure 5-35 shows the results in terms of the maximum vertical displacement at grid position ©① when column ©① was removed.

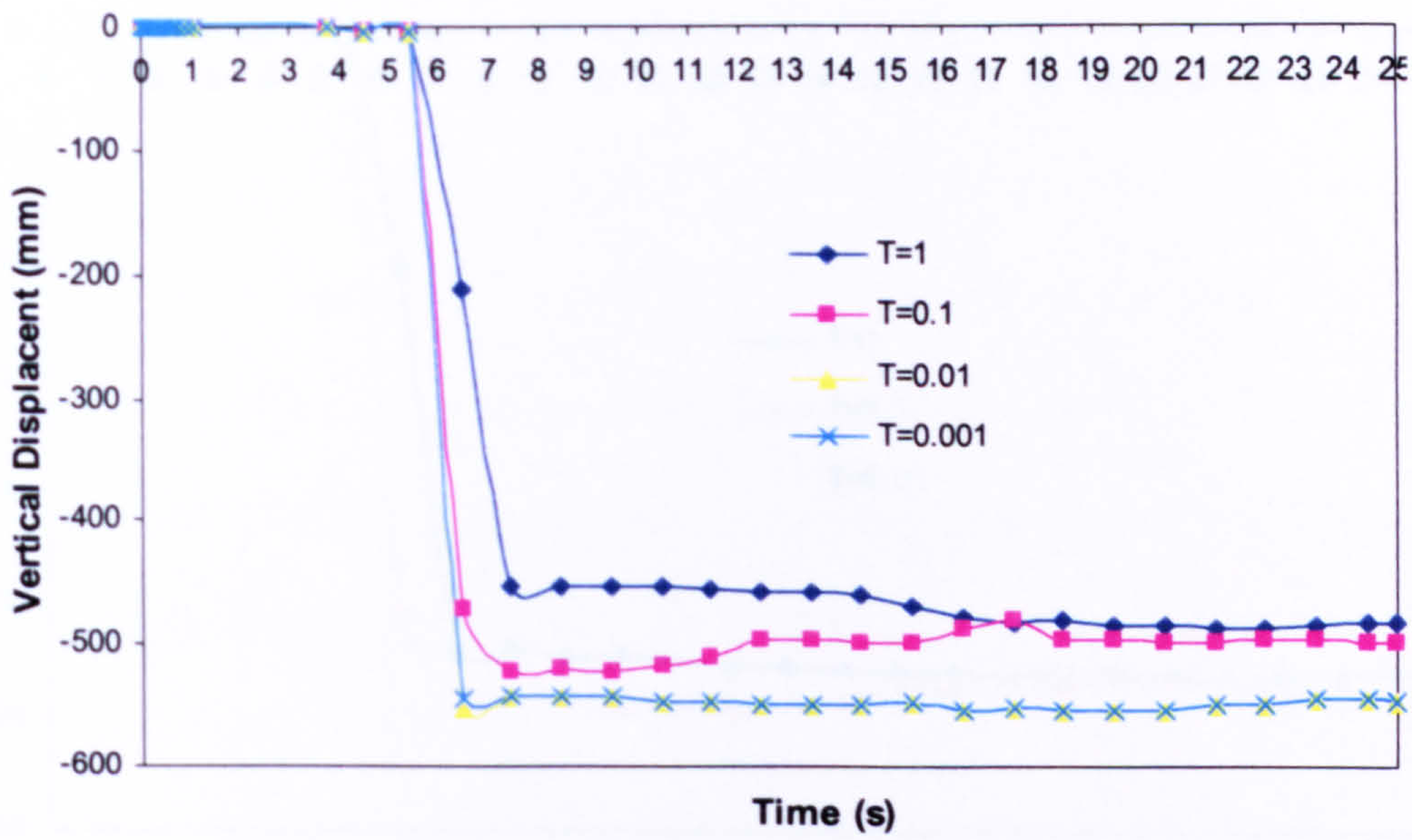


Figure 5-35 Influence of time of removal on the vertical displacement when column ©① was removed from the pin-pin frame at a loading level of $1.0g_k+0.33q_k$

The results presented in Figure 5-35 demonstrate that time is an important factor affecting the structural performance. LS-DYNA predicted 450mm vertical deflection when the column was removed in 1 second and 550 mm for the 1 millisecond case, which means the structure had to develop 60% more deformation to reach equilibrium than was the case when the column was removed over 1 second. A relationship can be observed, that is the peak deflection increased with the rate of removal of the column.

When two columns are removed from the building, obviously greater deflection occurs before the frame reaches equilibrium, but a similar trend with respect to the effect of the rate of column removal can be observed. Figure 5-36 shows results of maximum vertical displacement when both columns ©① and Ⓓ① were removed.

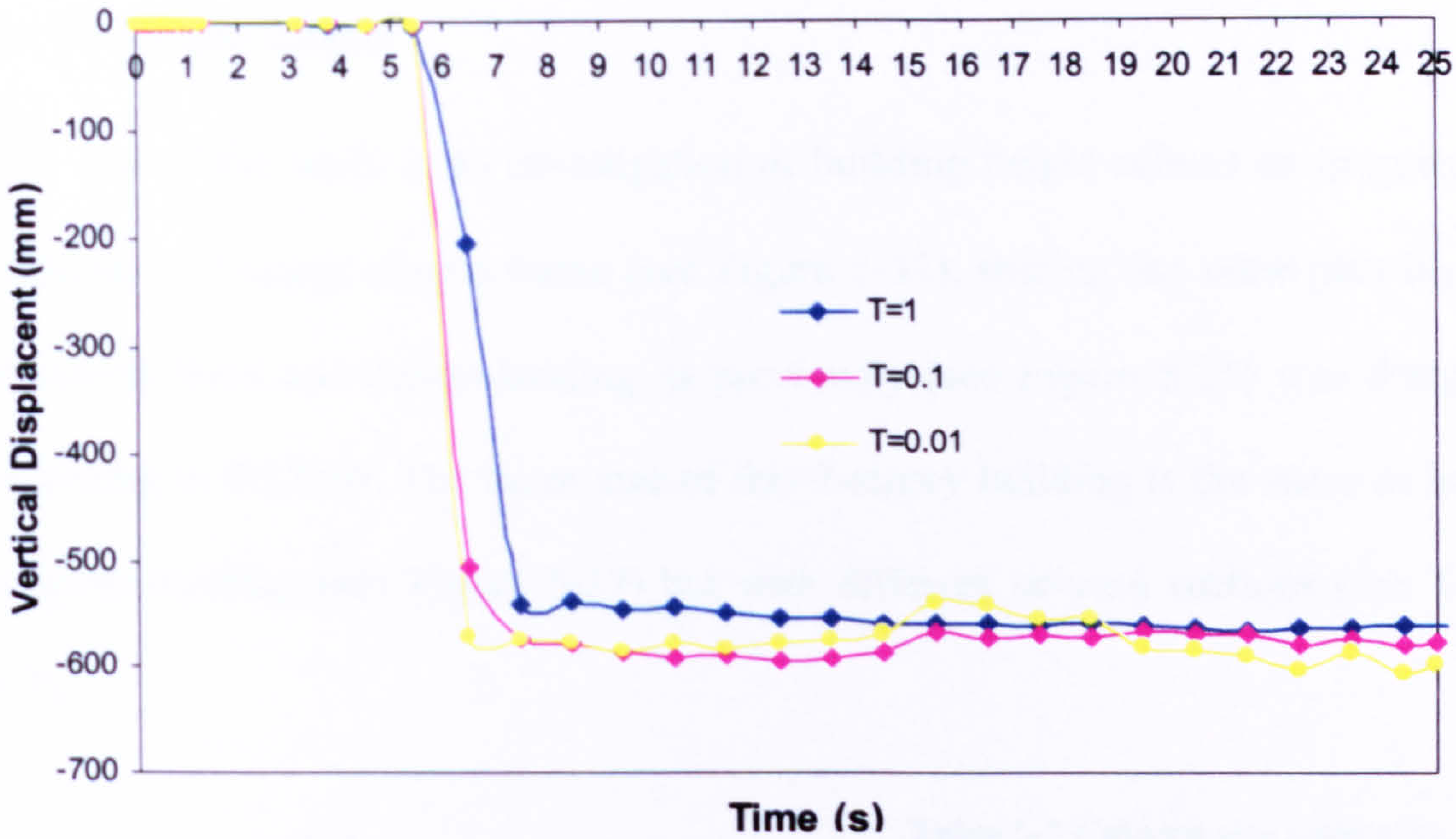


Figure 5-36 Vertical displacement when column ©①and ⓓ① are removed with different time from the 3 storey pin-pin frame at a loading level of $1.0g_k+0.33q_k$

Clearly, the time taken to completely remove the column affects the structural response. These results highlight the dynamic effects inherent in the frame response as a column is removed (even though the dynamic event that causes the removal of the element has not been modelled). The data also shows that when the column removal time reaches a certain rate, say 10 milliseconds, the final results do not differ much from the 1 millisecond case (see Figure 5-36).

This again shows that the structural response to progressive collapse is a dynamic event. Therefore, using static approaches to evaluate the structural behaviour during progressive collapse is not as accurate. Clearly for some loads, a static analysis would indicate that the structure would survive column removal whilst a dynamic analysis would predict collapse.

5.4.2.3 Height Effects

This part of the study is an investigation of building height effects on progressive collapse. A 7-storey simple frame (see Figure 5-37), sharing the same plan layout, structural form and design loading as previously (see Figure 5-25) was designed according to BS5950. The beam size of this 7-storey building is the same as in the 3-storey building (see Figure 5-37) but with different column sections (see Table 5-7).

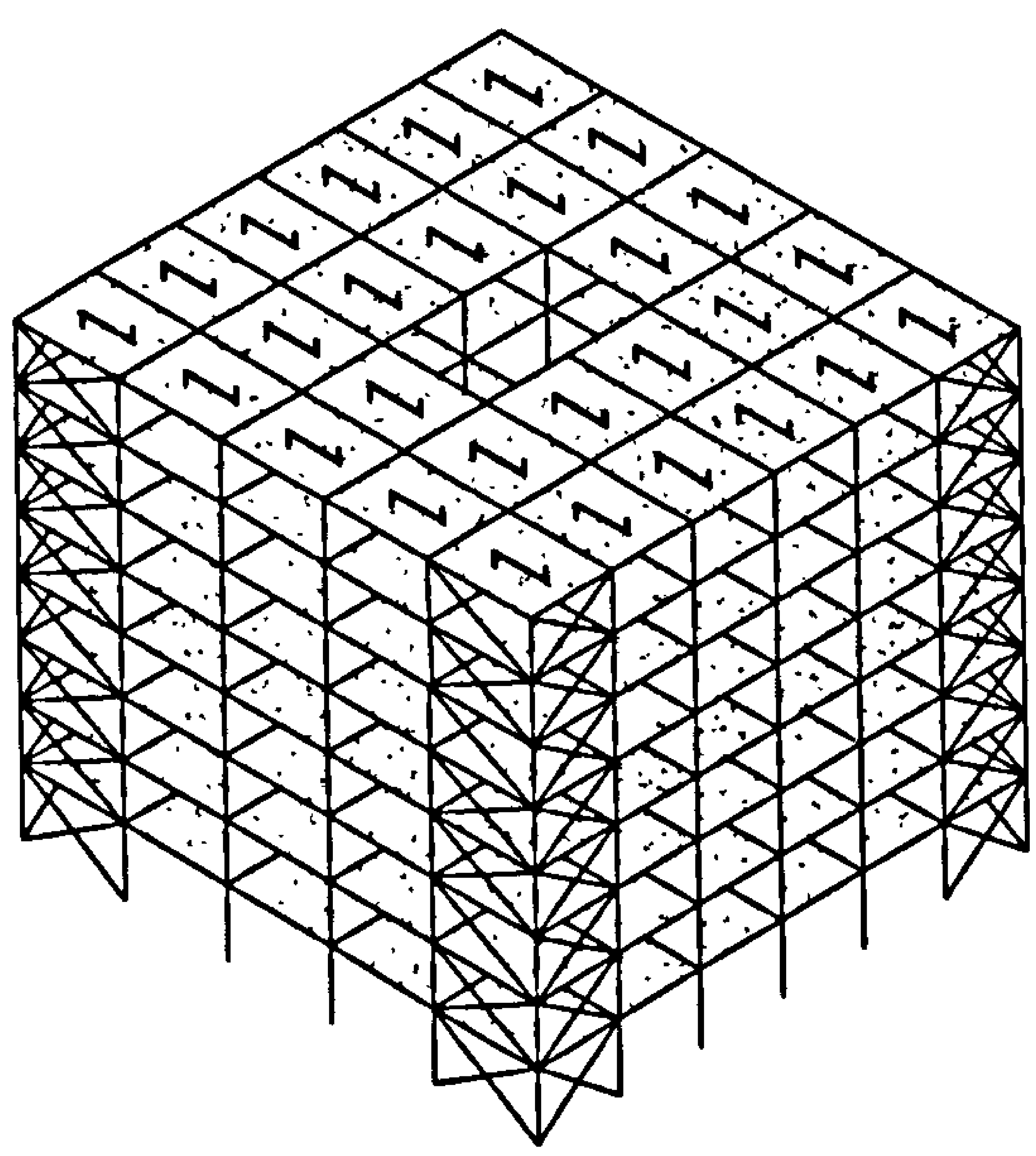


Figure 5-37 3D Geometry of 7-Storey building

Table 5-7 Column size (pin-pin)

UC1-UC3	305x305x158 1-3 254x254x89 4-6 203x203x71 7
UC4	305x305x198 1-3 305x305x118 4-6 203x203x71 7

According to the previous studies of the pin-rigid frame, the building height can affect the structural behaviour but this difference is not great. As for this pin-pin frame whether the same conclusion can be drawn is still unknown. The purpose of the following study is to investigate the structural response with different building heights. Again the accidental loading level of $1.0g_k+0.33q_k$ was selected as the test load. When the column ©① was removed in 1 second from this 7-storey building, LS-DYNA predicted the tying force shown in Figure 5-38.

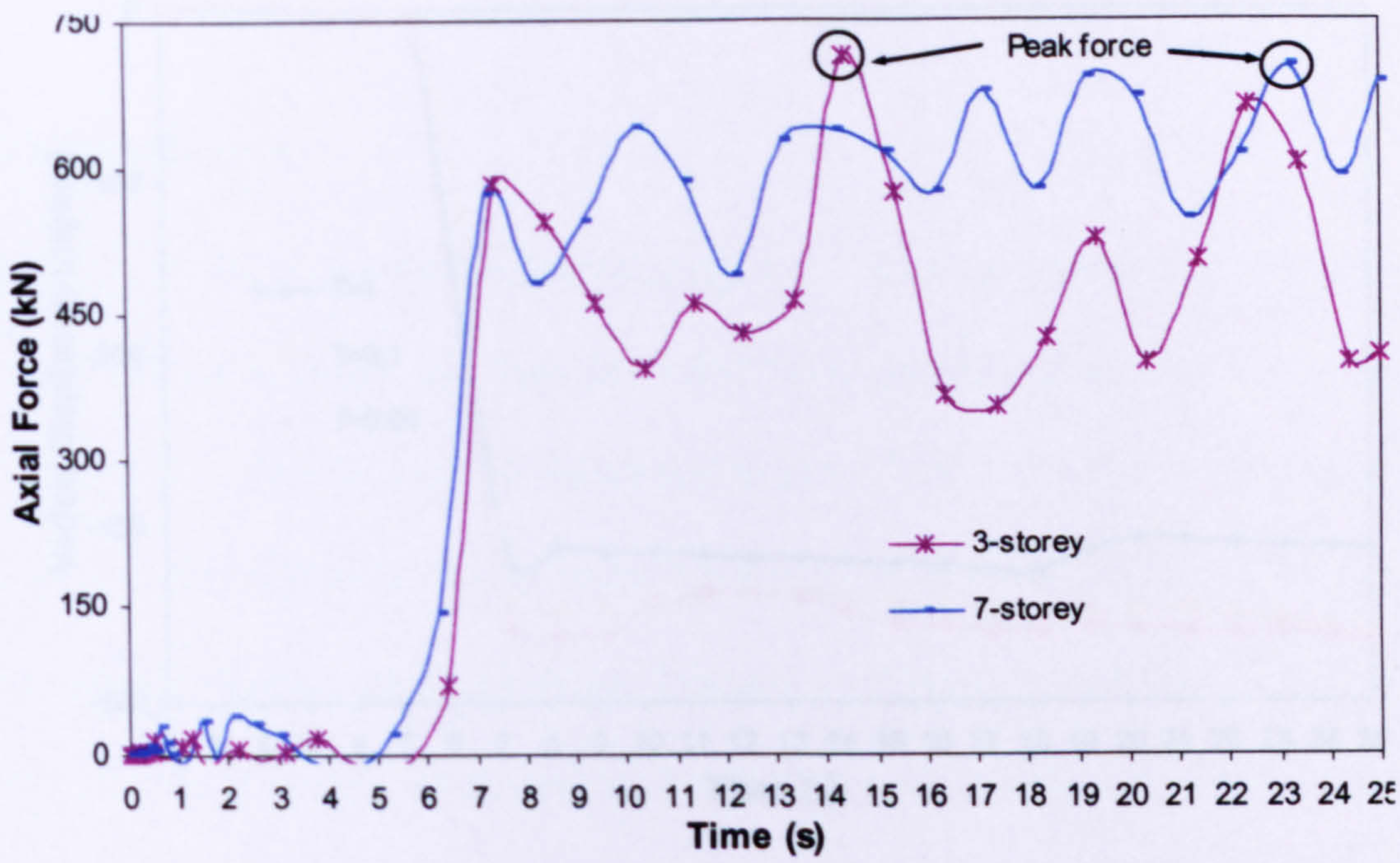


Figure 5-38 Comparison of the tying force between 3-storey and 7-storey when the column was removed in 1 second at loading level of $1.0g_k+0.33q_k$ from the pin-pin frame

Clearly, the *peak* tying force for both buildings (3-storey, 7-storey) is similar, that is 713kN for the 3-storey and 701kN for the 7-story. But the average tying force in the 7-storey is about 630kN compared to 480kN in the 3-storey, an increase of 31%.

It is interesting to investigate time influence of time of removal of the column on this 7-storey building. Thus, a set of numerical tests was conducted with varying column removal time from 1 second to 1 millisecond. LS-DYNA provided the results of the maximum displacement with varying column removal time for the 7-storey building, shown in Figure 5-39. Compared to the 3-storey (Figure 5-35), a similar trend can be observed, that is the peak deflection is related to the rate of removal of the column.

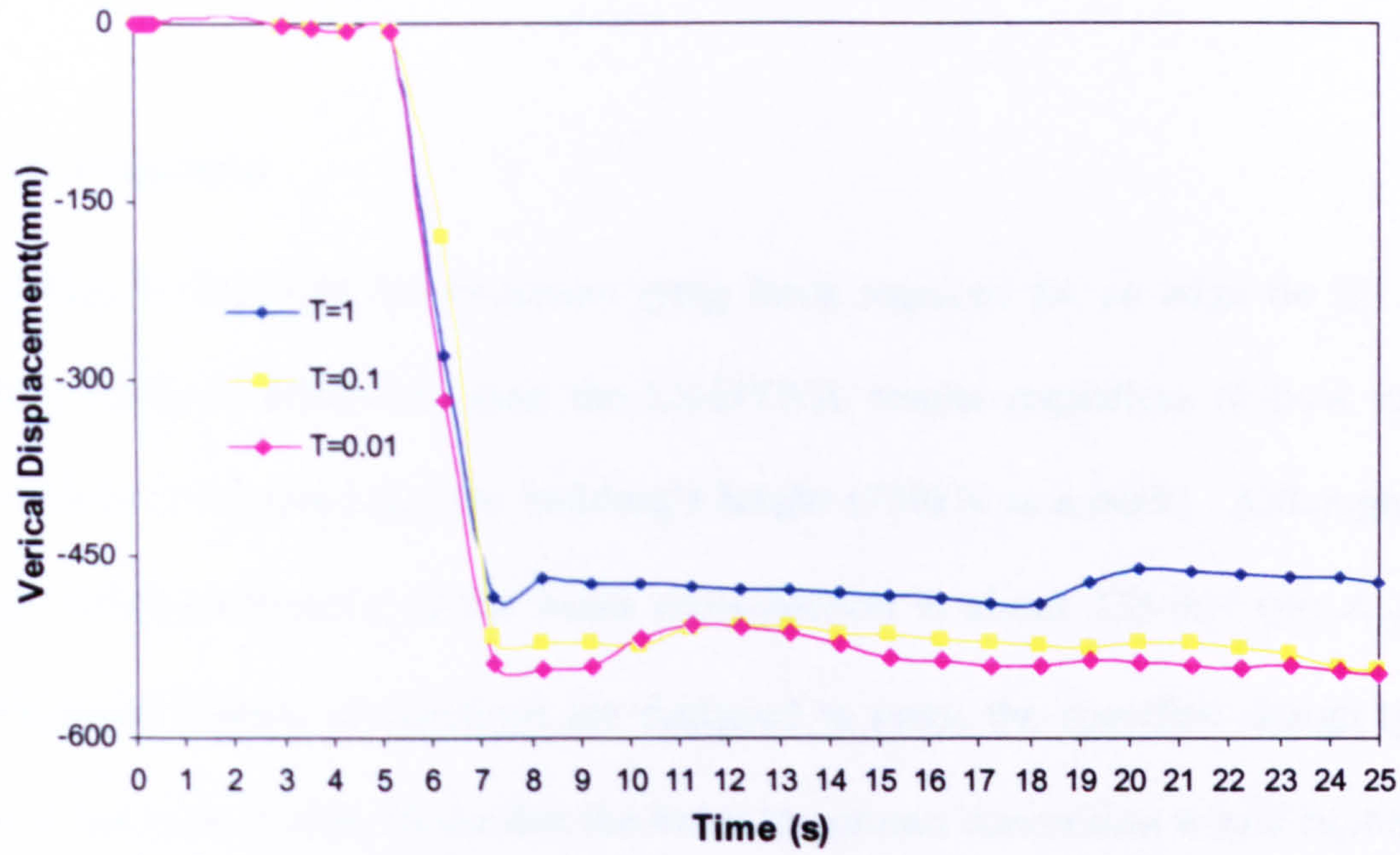


Figure 5-39 Comparison of the vertical displacement between 3-storey and 7-storey when a column was removed in 1 second at loading level of 1.0g$_k$+0.33q$_k$ from a pin-pin frame

In the 7-storey building, the displacement due to the removal of a column in 1 second and 1 millisecond is 485mm and 540mm respectively. The ratio between the maximum displacements measured with varying column removal times is about 11% for the 7-storey frame, compared with 22% in the 3-storey building, which suggest that the height of the building can improve the structural response in terms of dynamic effects. Increasing the structural height reduces the sensitivity of the structure to changes arising due to the length of time taken to remove the column, probably as a result of a greater number of redistribution routes. Clearly, in the 7-storey building there are more load transfer routes than compared to that in the 3-storey building. Therefore, the greater number of load redistribution routes can help the building behave better in terms of maximum vertical displacement as the frame resists the progressive collapse.

5.4.2.4 Discussion

According to BS 5950 the minimum tying force required for an edge tie B3 was 222kN, which is much less than the LS-DYNA results regardless of how many columns were removed and the building's height (730kN as a peak). Although the design tensile resistance of the beam cross-section is about 2354kN ($=\sigma_y A_g$), in conventional frames, connections are designed to resist the specified design tying force. Clearly, it is quite likely that the beam to column connection would rupture if its capacity was close to the target design value. However, the tie beams themselves (beams B3) would not fail. In UK practice, connections are designed to resist the ultimate limit state forces (usually bending and vertical shear) and checked that the tensile (tying) capacity exceeds the notional tying force. For this structure, if the connections were designed to satisfy the minimum requirements of the code, then the structure would likely collapse after removing column ©①, even though the design complies with the code recommendations. In this sense, the minimum tying force required by BS 5950-1:2000 is not adequate to safeguard against progressive collapse.

5.5 Conclusion

UK design rules give guidance on mitigating the effects of accidental damage to part of a structure by requiring beams to act in catenary action and tie a structure together. This study has investigated whether the design recommendations adequately protect buildings from progressive collapse.

It was found that when column ©① was removed from a pin-rigid frame, the damaged frame can only take a load ratio λ [3] of 20% instead of the 50%[2] (see Section 5.3.2.3). It was also found catenary action is not the resisting mechanism in a pin-rigid frame, instead a combination of frame action is more likely to be the resisting mechanism (see section 5.3.2.4).

On the other hand, in a pin-pin frame, when ©① was removed, it was found that the tying force generated in the damaged frame is much higher compared to the minimum tying force required for the connections (see Section 5.4.2.4), which suggested the connections would break before reaching the tying force. It needs to be acknowledged that in this numerical pin-pin frame model, a pin connection has (almost) zero rotational stiffness (see chapter 3 Section 3.3.3), whilst in a real pin frame, the real joints do have some, albeit modest, rotational stiffness (< 0.5EI/L). Therefore, in the *real* simple structure, the pin connections (e.g. partial endplates) have a reserve of moment capacity from its fabrication. Accordingly, in a *real* simple frame the resisting mechanism would likely be a combination action of catenary and Virenedeel action.

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[3] $\lambda = \dfrac{Accidental}{Collapse}$

[2] suggested by the reduced load and load factors in clause 2.4.5.3 of BS5950

The results from the two types of structure (pin-rigid frame, pin-pin frame) provide evidence that progressive collapse is a time dependent problem, i.e. a dynamic event. Removing a column over a short period of time led to larger forces and more deflection than was the case when the column was removed over one second. This effect was discernible irrespective of the type of frame, pin-pin or pin-rigid. The results of examining the height effects during the progressive collapse have shown that a greater number of load redistribution routes can assist the building to resist collapse (see Section 5.4.2.3).

In this chapter, the connections were assumed to function either fully pinned or rigid. In chapter 6, the influence of *real* joint stiffness on the performance of a frame designed using 'simple design' (clause 2.1.2.2 in BS5950) is examined. An alternative design approach to improve structural robustness is also presented.

Chapter 6

Hybrid Design Method

6.1 Introduction

This chapter briefly discuss the shortcomings of the UK rules in terms of their ability to adequately safeguard against progressive collapse. Based on the analytical results presented in the previous chapter (chapter 5), a hybrid design method that can improve structural performance during progressive collapse is proposed.

6.2 The current UK rules

The UK current design procedures implemented to avoid progressive collapse normally have three stages (see chapter 2) arranged in order of design complexity:

1. Tying members together against the collapse; if it is not adequate then –

2. 'Localisation of damage' needs to be checked. The damaged area due to removal of the element is limited to 15% of the floor area or $70m^2$, otherwise –

3. 'Key elements[♣] needed to resist accidental loading specified in BS6399 [BSI, 1996].

Tying members together against progressive collapse is a priority among those three procedures, but the scientific validity of the details about this requirement is acknowledged to be questionable [Brown *et al*, 2004]. The analytical results (in chapter 5) give further evidence that the current UK guidelines do not provide an adequate safety margin in terms of resisting total collapse when a single structural member is removed. The uncertainty arises because of the following two points:

1 Design procedures – static or dynamic

Previous results in chapter 5 showed that progressive collapse is a dynamic event. The recognition of progressive collapse as 'dynamic' concept is not new [MMC, 2003; GSA, 2003; Marjanishvili, 2004; Shankar, 2004; Corley, 2004]. The numerical studies presented in chapters 4 and 5 illustrate that progressive collapse is a dynamic event because the force generated depends upon the speed at which the column(s) is (are) removed. It was observed that the faster the column was removed, the worse the damage (e.g. displacement, tying force) the structure suffers. If the dynamic nature of the problem were to be included in design the current

[♣] Key elements are defined as those structural elements at any one storey whose loss results in a collapse of the structure more than one storey above or below the element under consideration, or over a horizontal area in excess of that stipulated in the criterion.

design procedures would need to be revised for this effect, possibly by some 'dynamic enhancement factor'.

2 Resisting (failure) mechanism- is it catenary action or not?

The UK design rules [BSI, 1990; BSI, 2000] require that structures be tied in order to resist progressive collapse. This is highlighted by the statement 'in *the event of column failure, the beams can carry the floors in catenary action to prevent collapse of the structure'* [Way, 2004]. Catenary action is assumed to be the resisting (failure) mechanism adopted by UK codes [BSI, 2000; Way, 2004] or at least it is the mechanism used to explain the rationale for the tying force. The resisting mechanism of the pin-rigid frame is a combination of Vierendeel action (see section 5.3.2.4). On the other side, when a column was removed from the pin-pin frame, the tying forces predicted by LS-DYNA were much bigger (about 2-3 times) than the minimum tying forces suggested by BS5950 (see section 5.4.2.1). This suggests that if the connections were designed according to this minimum tying force, they are likely to break and cause the building to fail, even if the applied load at the time is reduced to an accidental loading level.

These uncertainties, especially about the catenary action, bring into question the ability of the tying strategy to prevent a progressive collapse. Therefore, there is a need to explore possible ways to improve structural robustness still further. The available design approaches (UK, US) against progressive collapse, in general, can be summarised as [Shankar, 2004]:

- Redundancy or alternative load path;

This is a commonly used application according to its simplicity and directness. It requires that '*the structure is designed such that if any one component fails, alternative paths are available for the load in that component and a general collapse does not occur*' [Shankar, 2004]

- Local resistance

This approach requires the reducing of the risk of progressive collapse by providing the critical component with sufficient resistance against possible attack.

- Interconnection or continuity

This approach can be achieved either by means of adequate redundancy or local resistance or both.

Recent research by Hamburger [Hamburger, 2004] reported that catenary action was the alternative resisting mechanism for re-distribution of the load and supported a damaged frame. Meanwhile in the UK, Byfield [Byfield, 2004] pointed out that a possible way to improve the structural redundancy of typical frames, which have weak relatively brittle connections connecting strong ductile beams and columns, is by using stronger connections. Arising from the analytical studies reported in this thesis, another possible design approach is suggested to improve the structural performance and avoid a progressive collapse.

The author thinks that the connection is a more important component during progressive collapse, as it is the weakest part of the structure. Therefore, it is thought that the most effective way to improve structural robustness during progressive collapse is to improve the robustness arising from the connection. In the following section, a new alternative design method, a Hybrid Design, that aims to improve structural robustness is presented and discussed.

6.3 Hybrid design approach

6.3.1 Introduction

The underlying premise of the tying approach is that a damaged structure might redistribute loads by catenary action. Since the results outlined in an earlier chapter (chapter 5) bring into question the applicability of the tying force approach as the induced forces are so large, it is important to investigate whether there is an alternative design methodology which could be used to create additional robustness. If catenary action is insufficient, the most likely alternative load carrying mechanism is Vierendeel action arising from reserves in moment resistance and rotational capacity in the beam-to-column connections.

A hybrid design method has therefore been investigated, loosely based on the Wind-Moment design method in which the frame is made statically determinate by treating the connections as pinned under vertical loads yet rigid under horizontal loads [Nethercot, 1985; Brown *et al*, 1999; Brown *et al*, 2004; Salter, 1999; Hensman *et al*, 2000; Hensman, 2001; Bailey, 2003]. The main advantage of the wind-moment method is its simplicity, although at first it appears illogical. In 1996, Stefieck reported an interesting methodology to protect the exterior of a six-storey building, New York City Technology Center [Stefieck, 1996]. The building had been designed as a rigid frame but in order to increase its robustness the designer increased the size of the spandrels and columns, as well as the moment capacity and ductility of the beam-to column connections. In so doing, the frame had sufficient redundancy to enable it to withstand the removal of an exterior column. A combination of the wind-moment method and Stefieck's approach forms the basis of a hybrid design method.

The proposed hybrid design method retains the simplicity and practicality of the simple design method which is prevalent in the UK. A simple frame is designed assuming the members to be pin connected. This results in slightly oversized beams, because the real joint behaviour is not pinned but has some stiffness and strength, albeit small if simple connections (web cleats, tab plates or partial depth endplates) are used. By substitution of more substantial connections at the construction stage, for example a flush end plate or an extended endplate, a more robust frame can be achieved. In an extreme event, the reserve capacity inherent in both the beam sections and the connections will permit the frame to span over a damaged section utilising Vierendeel action as the alternative load carrying mechanism.

The philosophy adopted in the hybrid design method is thus to prevent progressive collapse by building in a reserve of strength and stiffness by the substitution of more substantial beam-to-column connections in a frame whose member sizes have been determined assuming the joints to be pinned. Research conducted into the effects of semi-rigid steel beam-to-column connections on the behaviour of frames has shown that the additional strength and stiffness is not detrimental to column capacity as the beneficial effects of joint stiffness outweigh the detrimental effects of moments transmitted to the columns [Gibbons, *et al*, 1993; Braham, 2004].

6.3.2 Analytical models

To examine the idea of the hybrid design method, a set of tests were conducted on the 3D building presented in Figure 6-1. This analytical model has the same geometry and member sections (Table 6-1) that were presented in the previous 3D pin-pin frame discussed in chapter 5.

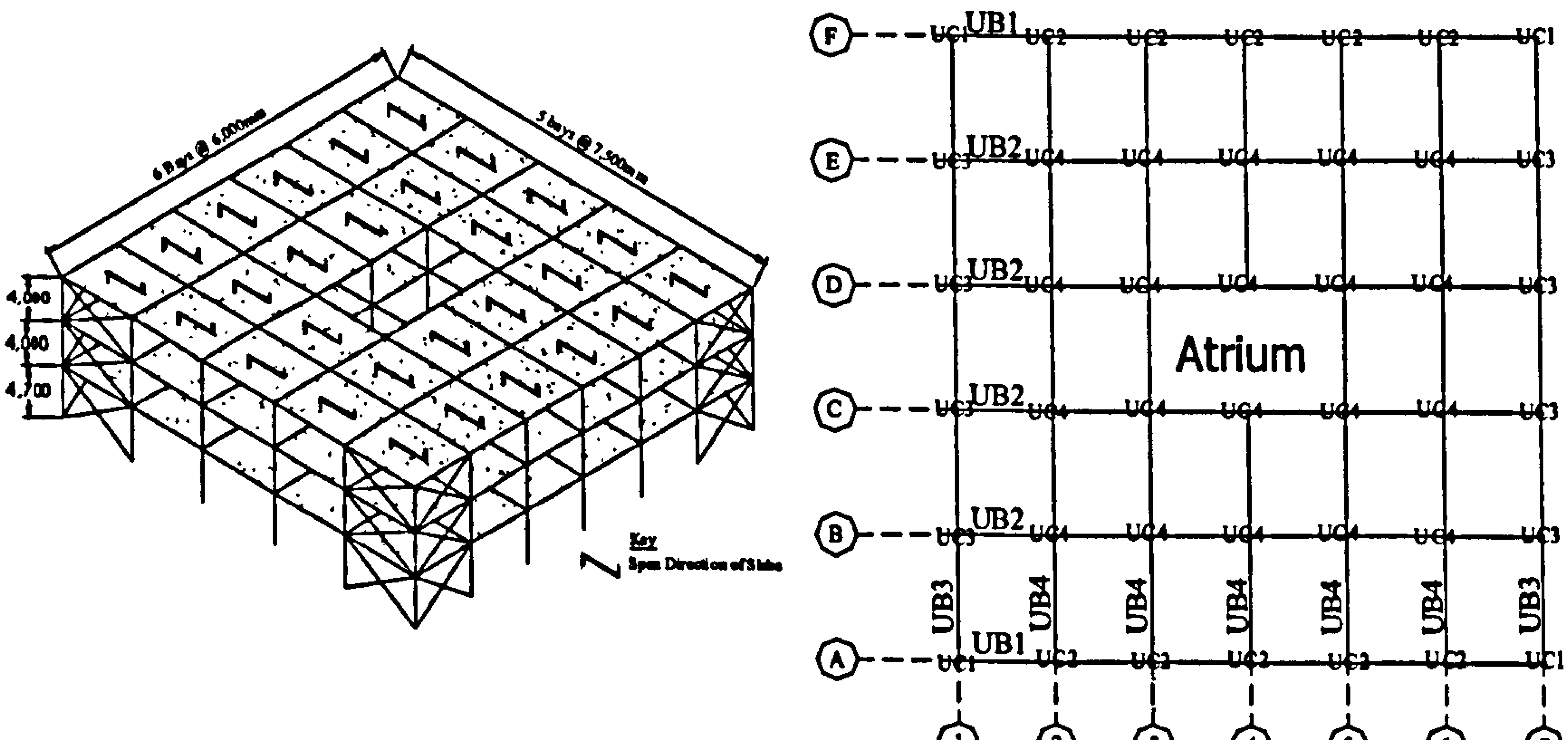

Figure 6-1 Details of the analytical model used to test the hybrid design method

Table 6-1 Member sections in the hybrid design test model

	Beam		**Column**	
	Roof	Floor		
UB1	356x171UB57	457x191UB74	UC1	256UC107
UB2	457x152UB60	457x191UB89	UC2	256UC107
UB3	305x127UB42	457x152UB67	UC3	256UC107
UB4	305x127UB42	457x152UB67	UC4	305UC118

As discussed before, the exterior facade of a building is the most vulnerable to damage; therefore the hybrid design method was only applied to the four exterior faces of the building, the interior beams, columns and connections were pinned (see Figure 6-2).

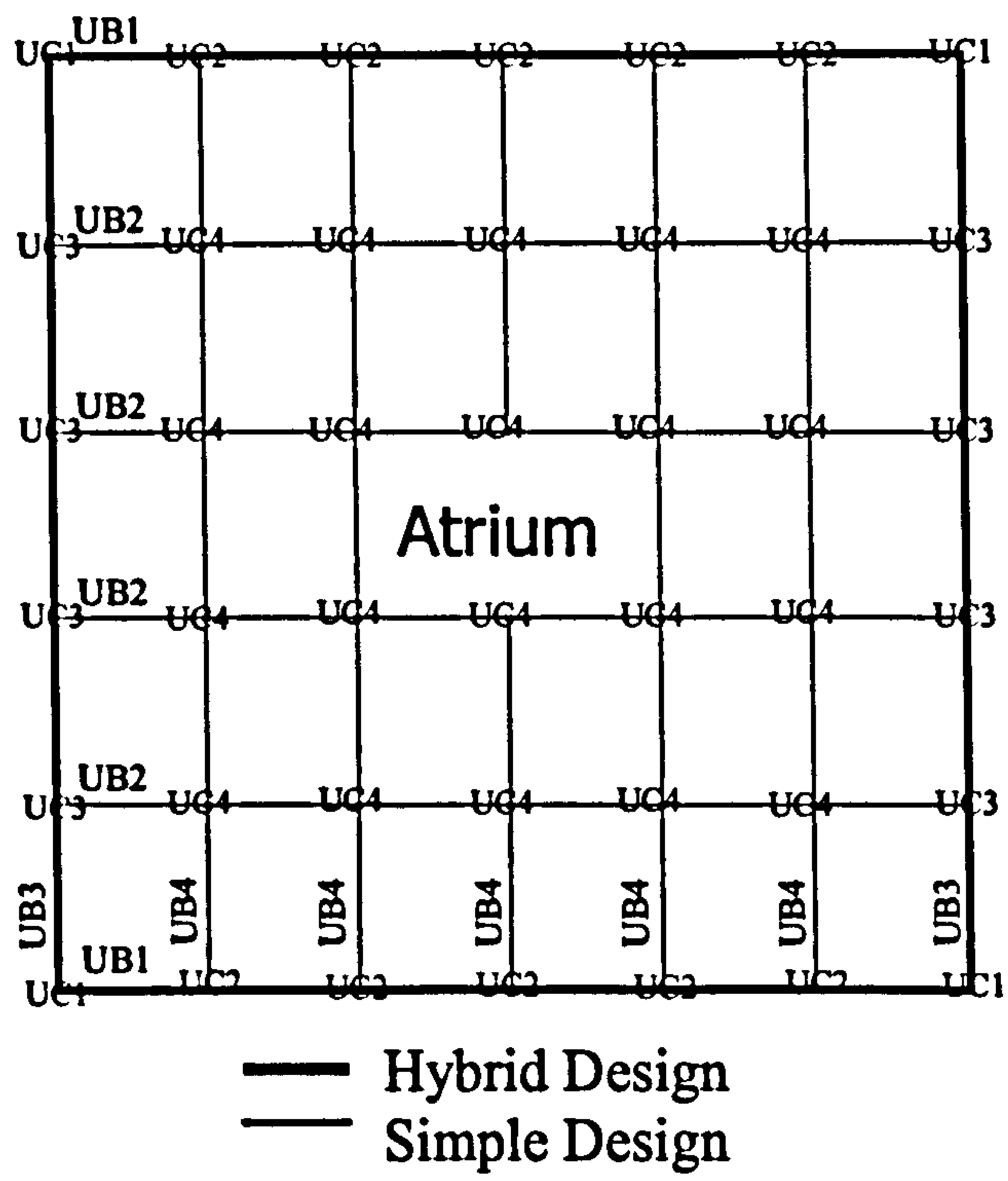

Figure 6-2 Details about hybrid design

It was decided to carry out a set of numerical tests to investigate the collapse loading level when the Hybrid Design Method (HDM) is applied. The previous pin-pin frame had a collapse loading level of $1.4g_k+1.8q_k$, so it is interesting to investigate the difference between the HDM frame and a normal pin-pin frame.

In order to apply the HDM, it is necessary to define a rotational stiffness for semi-rigid region connections. According to Eurocode3 [BSI, 2005] the semi-rigid rotational stiffnesses are defined as those between 0.5EI/L (pin) and 8EI/L♣ (rigid), that is $3.2x10^9$Nmm/rad and $6.4x10^{10}$Nmm/rad for the beam sizes and spans in use

♣ 8EI/L for braced frame

25EI/L for embraced frame

in the model frame. For this reason, it is decided to use rotational stiffness of 3 $\times 10^9$Nmm/rad and 5×10^9Nmm/rad for study. In 1992, the tests done by Owens and Moore [Owens and Moore, 1992] provide a wide range of real translational stiffness of the connections; in this study it was decided to use a translational stiffness of 4×10^4 N/mm. .

When rotational stiffness of 3 $\times 10^9$Nmm/rad is applied, the collapse loading level is about $1.4g_k+1.85q_k$. When 5 $\times 10^9$Nmm/rad is applied, $1.4g_k+1.95q_k$ is the collapse loading. Clearly, the trend is when the joint stiffness increases, the building can take more load although the magnitude is not great. The most important fact for the HDM is what happens if column is removed from the building, and whether HDM can improve the structural performance.

The numerical tests were then carried out with column ©① removed in 1 second at a loading level of $1.0g_k+0.33q_k$ in order to investigate the effects of the joint stiffness of HDM.

A set of analyses were conducted with rotational stiffnesses at the joints covering a wide range between 1×10^1Nmm/rad and 1×10^{13}Nmm/rad. LS-DYNA results (chapter4) showed little difference when the joint stiffness lay between 1×10^1Nmm/rad and 1×10^7 Nmm/rad, the connections behaving effectively as a pin joint. At the opposite extreme, a joint stiffness greater than 1×10^{11}Nmm/rad was effectively like a rigid joint.

Figure 6-3 shows results for tying forces against time for a range of connection stiffnesses between 5×10^8 Nmm/rad and 5×10^{10} Nmm/rad as well as the pinned and rigid cases

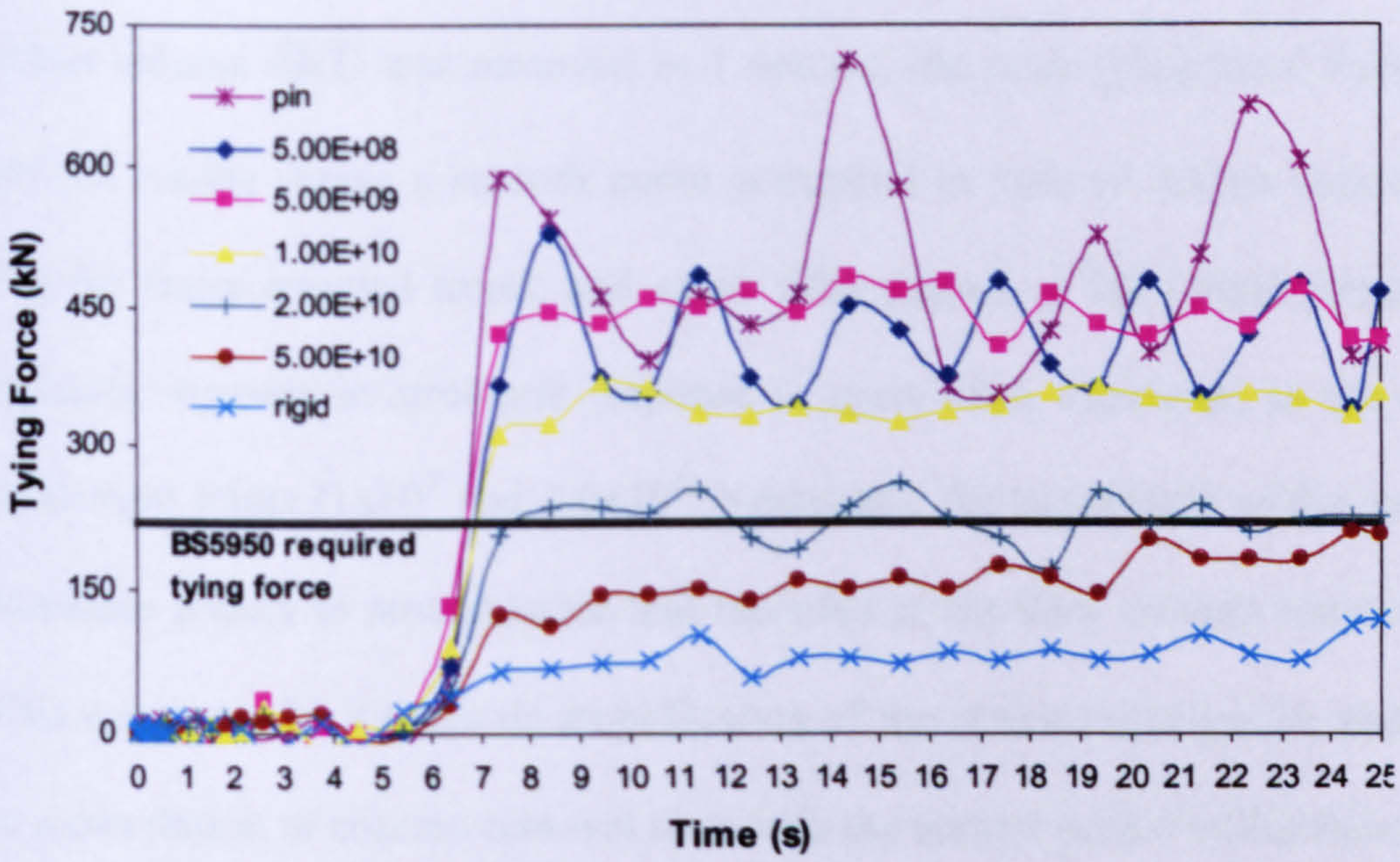

Figure 6-3 Illustration of tying Force of for a 3-storey HDM frame when column was removed in 1 second at loading level of $1.0g_k+0.33q_k$.

It is clear to see that the peak tying force is reduced as the joint stiffness increases. As the joint stiffness gradually increased, the resisting mechanism of this building becomes a combination of catenary action and Vierendeel action, so the tying force in the remaining structure is reduced as the bending moment plays an increasingly influential role.

As discussed before, the numerical tests have shown that the rate of column removal is an important factor that affects the structural response. A set of tests was therefore conducted with a different rate of column removal, and the results of these tests are shown in Figure 6-4.

When column ©① was removed in 1 second, the peak tying force from the LS-DYNA results shows a smooth curve compared to 1ms or 100ms removal times. For the faster removal times, and either fully pinned or fully rigid joints, a small dynamic increase in structural response is again seen. However, in the region of semi-rigid joints ($1x10^7$ and $1.0x10^{11}$ Nmm/rad), the magnitude of the tying force increases greatly to around twice that recorded in the slow column removal model. This appears to be a dynamic amplification of the structural response, possibly due to a correlation of column removal time with the natural period of the frame.

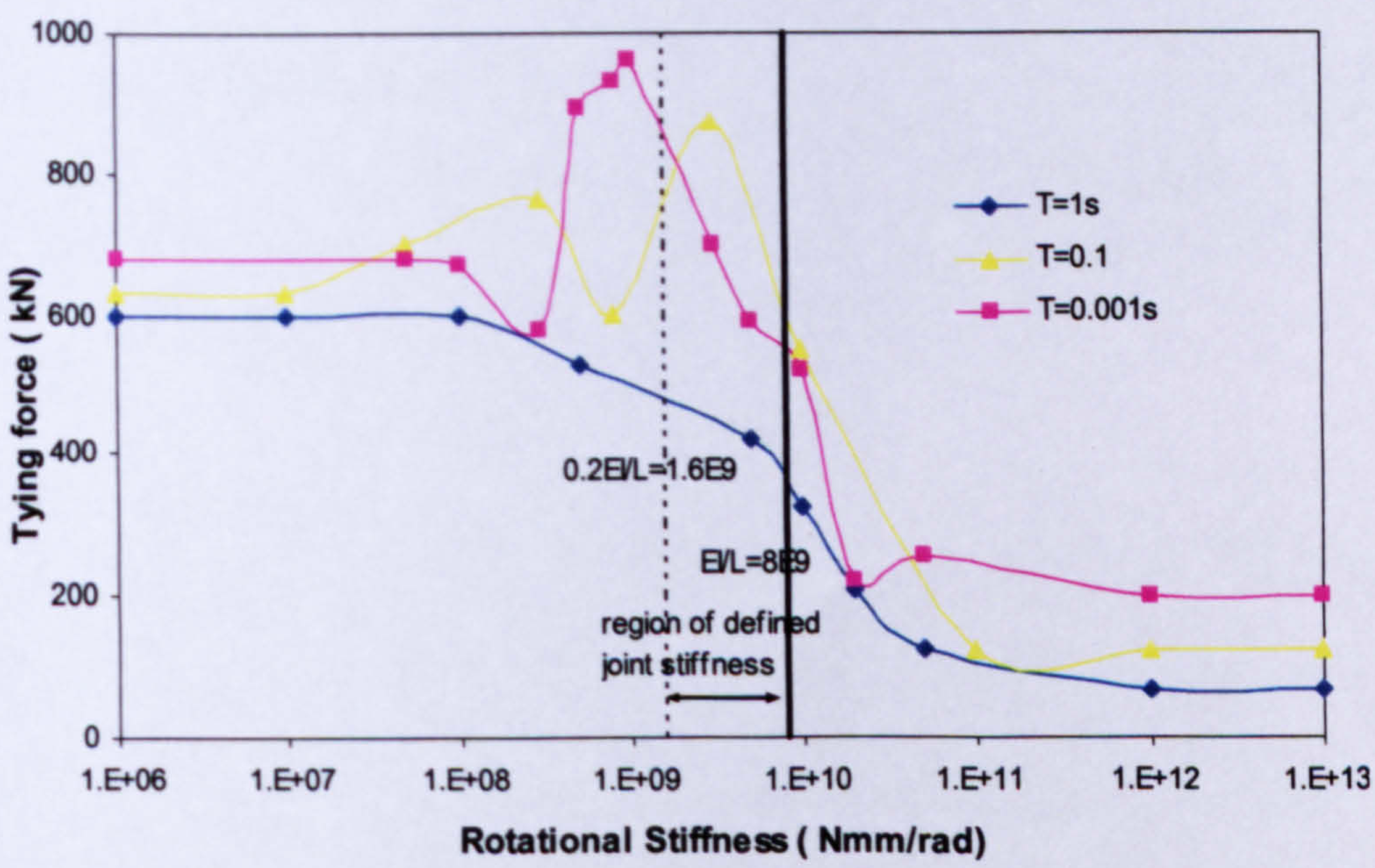

Figure 6-4 Illustration of peak tying force when column removed with different time from a HDM frame at loading level of $1.0g_k+0.33q_k$

In the hybrid design approach the frame members are selected assuming the beam-to-column connections act as pins (as in the UK simple design approach) but the frame is constructed with more substantial rigid or semi-rigid joints, for example extended or flush endplates. The analyses conducted to date show evidence of a dynamic amplification effect on structural response for particular combinations of rates of removal of columns and joint stiffnesses. This effect requires further investigation.

6.4 Conclusion

In this chapter, studies have reported the influence of joint stiffness on a simple frame. The analysis results have shown that joint stiffness would be an important factor to affect the structural performance during the collapse, and arising from this a possible way to improve the structural robustness without involving the additional design has been proposed, the hybrid design method (HDM).

It is well known that simple construction is widely used in the UK construction industry (see section 5.1). Previous studies (see section 5.4) have shown evidence that catenary action alone cannot resist collapse, as the connections would break before reaching the necessary tying forces. The application of HDM uses the fact that the connections in a real building do have some inherent (rotational) stiffness. The resisting mechanism of a real frame is a combination of Vierendeel and catenary action. The reserve of rotational stiffness can be achieved by inserting semi-rigid connections in the construction, and replacing the theoretically pin connections assumed in the initial design. For instance, a partial depth endplate connection can be easily replaced by a flush endplate or even extended endplate during the construction.

Providing extra robustness via semi-rigid connection around the perimeters of a building, the tying forces required can be reduced significantly. The resisting mechanism of the damaged frame has changed from relying only on tying force (catenary action) to a combination of catenary action and Vierendeel action. The numerical results from LS-DYNA have provided the evidence to support this view.

The biggest advantage of HDM is it improves structural robustness without an extra design stage. The HDM allows extra safety (redundancy) by not considering the joint stiffness during design. Instead the reserved redundancy is left for in service, emergency use.

Chapter 7

Conclusions and Recommendations

7.1 Introduction

The work described in this thesis arose from doubts surrounding whether the UK design code can safeguard buildings against progressive collapse. The tying strategy against progressive collapse has been in the UK design codes for more than 30 years, and it has been cited as good practice world-wide. However, there has been little research conducted into the UK design procedure to prove or otherwise whether the minimum tying force can provide adequate structural integrity against collapse. For this reason, research was carried out on a steel framed building designed according to current British Standard BS5950: Part 1-2000, and the structural behaviour investigated (e.g. load re-distribution, resisting mechanism) after a column was removed from ground level. The next section presents a discussion and overall conclusion.

7.2 Discussion and Conclusion

The worldwide rise in terrorism has made the security of important buildings a major concern. To date the 9/11 events remain the worst structural failure and engineers are keen to learn lessons to avoid this sort of tragedy happening again. It was found that the damaged WTC towers were suffering the progressive collapse that is the same type of failure that Murrah building suffered back in 1996. As the first design guidelines on preventing progressive collapse have been available in the UK since 1970 following the partial collapse of the Ronan Point Apartment, it is clear that the need to design buildings that are robust is still a major concern.

The UK design code uses the tying strategy as a primary choice when designing against progressive collapse. In fact, most countries including the US adopt this tying strategy into their design guidelines. The WTC event certainly raised awareness in engineers around the world and caused them to question the understanding of progressive collapse. Doubts about the current UK design code were raised in 1999 (see chapter 2), on whether they are adequate to protect building against progressive collapse, and this is the reason that this research was carried out.

The UK design procedures implement three stages in order of design complexity to avoid progressive collapse. Among those three design procedures, the tying strategy is the primary, and this study is therefore focused on checking the minimum tying force for tying members against the collapse [Way, 2003]. In detail, this research investigated tying forces generated in the damaged frame and compared them to the minimum tying force that designs require, and by doing this comparison provided evidence on whether this requirement is adequate or not.

This research was undertaken using Finite element analysis. The nonlinearities of geometry and material have to be included in order to get a realistic answer of the structural behaviour. Additionally, a finite element code able to solve dynamic problem was needed, as progressive collapse is a dynamic problem. To satisfy these requirements, it was decided to use LS-DYNA, a non-linear explicit/implicit finite element code capable of modelling the dynamic behaviour of structures for this study.

Chapter 3 reports a number of initial studies using the LS-DYNA, and concluded that this FE package is adequate for this study. Based on the results from chapter 3, Chapter 4 presented a further investigation on this FE package. A small-scale 3D skeleton steel frame was examined. A set of numerical tests was conducted on this steel frame when a column was removed from the ground floor. It was found that this frame was too small and unrealistic to provide understanding of real frame behaviour, a more realistic frame was needed. Although this 3D small-scaled framed is unrealistic, there is an interesting finding, that is the quicker the column was removed, the worse the structural response (e.g. larger deflection). Whether this finding is representative or not, required a more realistic frame to be modelled and tested.

The studies of chapter 3 and chapter 4 provided confidence in the analytical tool: LS-DYNA. Chapter 5 reported studies on two frames: a pin-rigid frame and a pin-pin frame both designed according to current design guidelines in BS5950. Chapter 6 examined the effects of *real* joint stiffness and proposed an alternative design approach to improve structural robustness. The main conclusions from these studies can be summarised as follows:

7.2.1 Resisting Mechanism

It has been found that the resisting mechanism for a 3D frame (see Figure 7-1) has to be a combination of actions, i.e. catenary action and Vierendeel action.

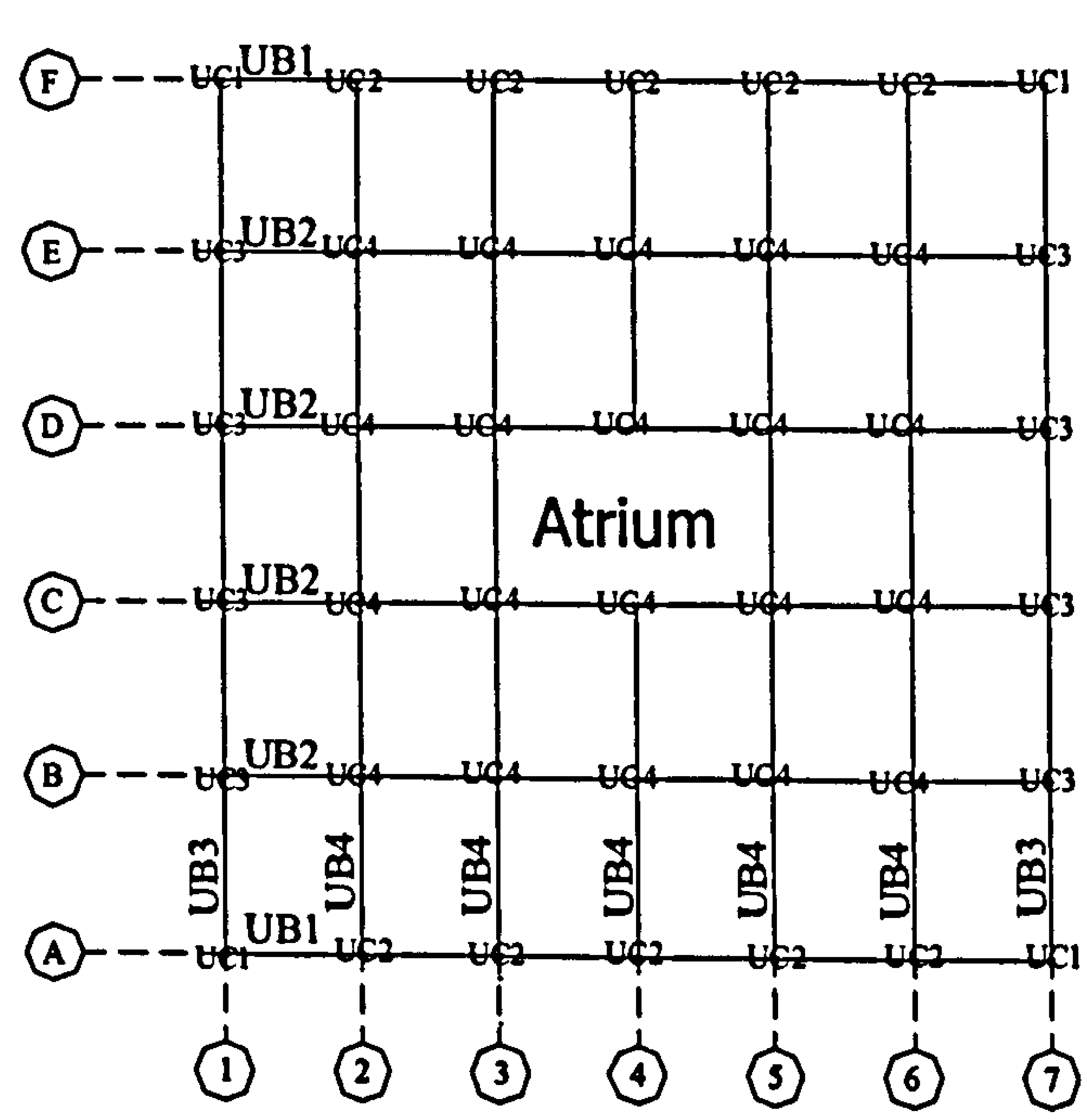

Figure 7-1 A typical outline for a 3D frame

It is easier to start the explanation by considering the resisting mechanism of a 3D pin-rigid frame. A pin-rigid frame (see Figure 7-1) has rigid frames along lettered gridlines and pin frames along numbered gridlines. Due to the continuity from the rigid connections (lettered gridlines) the Vierendeel action would support the rigid frames when damage occurs. On the other hand, pin frames (numbered gridlines) can only develop catenary action when damaged. The resisting mechanism of this 3D pin-rigid building is a combination of actions (details see section 5.3.2.4).

The resisting mechanism is the catenary action for a 3D pin-pin frame, as there is no other form of support (i.e. no bending moments can arise at beam-to-column joints). The numerical results demonstrated that a damaged pin-pin frame can stand up but with a tying force that is 2-3 times that of the design value, which the connection may be unable to take. It is also acknowledged that the (almost) zero rotational stiffnesses that are applied for the pin connections are unrealistic. In real pin frames, the joint do have some rotational stiffness from the fabricated connection, which suggests that in a real simple frame the resisting mechanism would likely be a combination of catenary and Vierendeel action (details see section 5.4.2.4 and 5.5).

7.2.2 Dynamic Effects

The results of varying the speed of column removal in either a 3D pin-rigid frame or a pin-pin frame proved that the progressive collapse is a time dependent problem. When a column was removed in less than one second, the damaged frame generate up to twice the tying force or displacement compared with when the column was removed over one second (details see section 5.3.2.6 and 5.4.2.2).

7.2.3 Height Effects

Studies examined the effect of the height of a 3D pin-rigid frame and a pin-pin frame on its ability to withstand damage. It was found that when the building height increased, the number of load re-distribution routes increased as well. Therefore, the number of storeys can assist the building to resist collapse. (Details see section 5.3.2.7 and 5.2.3.3)

7.2.4. Hybrid Design Method

It was found that the continuity of a 3D frame appears to be an important factor to provide redundancy. Providing continuity (rigidity), at least in one direction of a 3D frame, means the resisting mechanism of a damaged frame would be a combination of frame action rather than catenary action only.

A Hybrid Design Method is proposed which adopts this philosophy. HDM achieves a reserve of redundancy by replacing pin connections by semi-rigid connections on the perimeters of a 3D pin-pin frame. The results have shown that when a normal 3D pin-pin frame uses the HDM (around the outside frames), the damaged frame can behave in a better way, e.g. significantly reducing the tying force (see section 6.3).

In general, the findings from the studies can be summarised as:

1) The design value of the minimum tie force required by BS5950 to prevent progressive collapse of steel framed buildings is significantly smaller than forces generated in a damaged frame;

2) The effects of joint stiffness can affect structural performance during a collapse. By increasing the rotational stiffness of a connection the tying force arising in the damaged building can be reduced, and therefore assist the damaged structure to resist collapse;

3) The dynamic effects should be considered during the design;

4) Catenary action can provide a resisting mechanism in a theoretical pin frame but in reality the resisting mechanism of a damaged frame would be a combination of actions i.e. catenary action and Vierendeel action.

7.3 Recommendation for Future Work

The research work can be extended and modified and particular suggestions are follows:

1) A real or existing steel frame could be studied. The steel framed building used in this study, although designed according to the BS5950, is rather an academic case

study instead of a real design. In this sense, a building that has already been designed (by a design office) would be a good starting point to check the findings from the research applied to an existing building.

2) A steel frame with composite slabs. The numerical studies only investigated the structural performance of steel framed buildings with pre-cast units. Up-to-date, the research did not include study of composite action between the slabs and beams. This is of particular importance in buildings in fire and may also be of benefit to damaged buildings when fire is not present. As discussed earlier, the minimum tying force required by BS5950 is not adequate to protect the steel frame with precast units from progressive collapse. It would be of interest to make a comparative study of two different types of floor unit pre-cast units or composite slabs, from which the effects of composite action could be identified and the extra redundancy offered by slabs quantified when damage happens to the building.

3) Failure of the connections. This research excluded connection failure, so the real value of the tying force that breaks a connection is unknown. If numerical analysis can include a failure of the connection, this force can be identified. Providing this information would assist understanding of the connection behavior during the collapse, and possibly help a designer to choose a connection that can behave better when damage occurs.

4) Dynamic effects in the HDM. It is acknowledged that an additional investigation of joint stiffness is needed. When a column was removed faster than in 1 second, a high tying force appears in some semi-rigid regions. This may be a dynamic problem i.e. resonance, as some semi-rigid joint stiffnesses (K_{joint}) may cause the stiffness of a frame system (K_{whole}) to respond to the damage in a region at close to the natural period frequency (ω). A further study is needed to investigate this problem.

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Appendix -A

Inelastic Buckling of Column with Residual Stresses

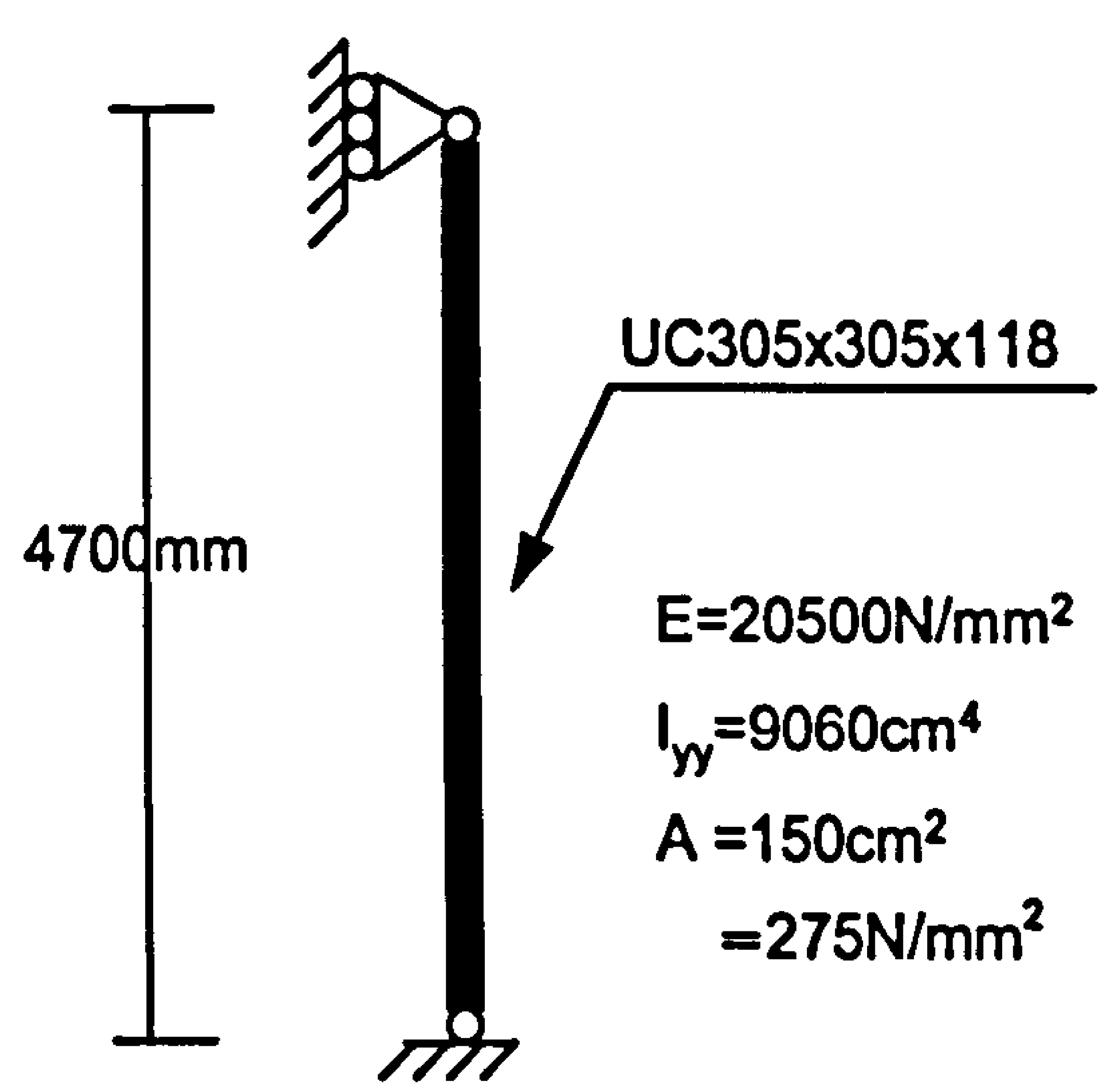
A.1 Equations

From Trahair [Trahair and Bradford, 1995]

$$\frac{P_t}{P_Y} = 1 - \frac{1P_Y}{4P_{oc}} \tag{A.1}$$

Calculations of inelastic buckling force P_t can be found below:

Note that E should be 205,000 N/mm^2



Elastic buckling force P_{co} is

$$P_{oc} = \frac{\pi^2 EI_{yy}}{L^2} = \frac{\pi^2 \times 20500 \times 9060 \times 10^4}{4700^2}$$
$$\approx 8300kN$$

The squash load P_Y is

$$P_Y = \sigma \times A = 275 \times 150 \times 100 = 3750kN$$

So, the inelastic buckling force P_t

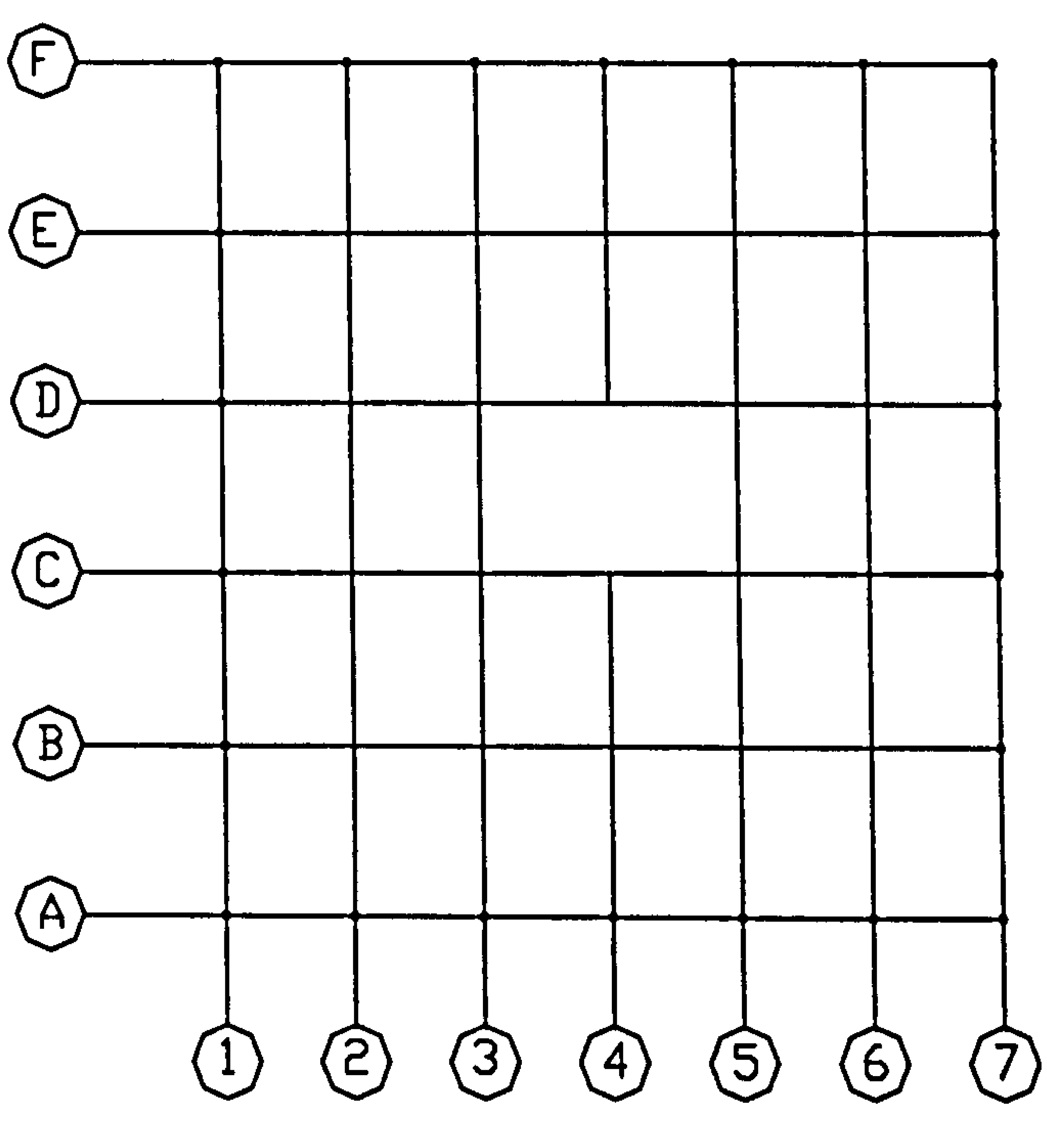
$$P_t = P_Y(1 - \frac{P_Y}{4P_{oc}}) \approx 3326kN$$

Appendix -B

Design Details of the pin-rigid frame

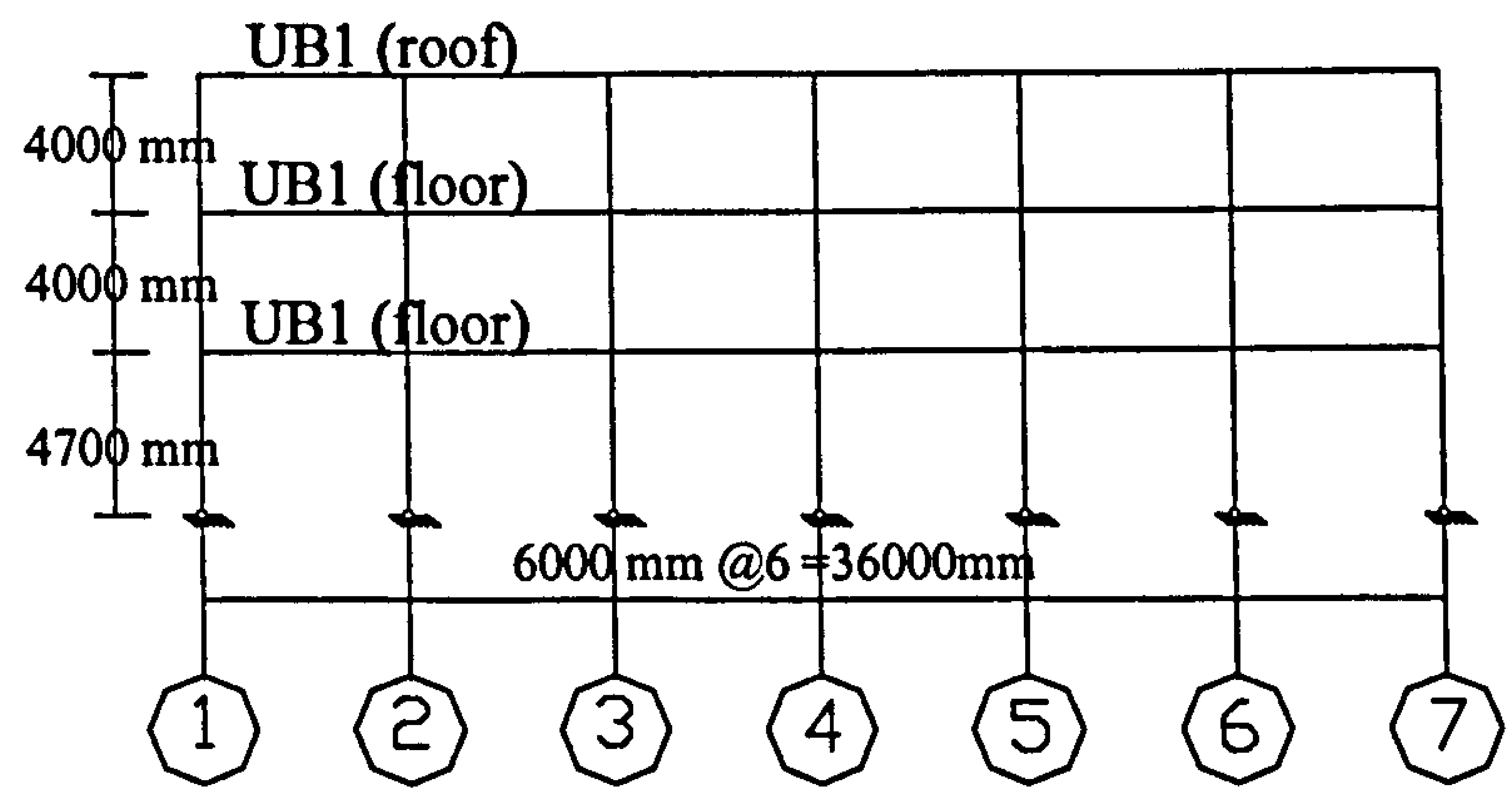
B.1 Sway check by Oasys GSA [Oasys, 2001]

The member section of the rigid frames is the primary concern for this 3D pin-rigid frame (see FigureB-1), as they are the load bearing frames. Therefore the extra considerations were given to those rigid frames along lettered gridlines.



FigureB-1 Outline for pin-rigid frame

The frames along lettered gridlines-Ⓐ/Ⓕ were studied first. The geometry detail of this rigid elevation can be found in FigureB-2.



FigureB-2 Geometry Details about rigid frame along gridline Ⓐ/Ⓕ

The member section for this rigid frame is initially selected as listed in TableB-1

TableB-1 Member section of rigid frame along gridline Ⓐ/Ⓕ

Beam B1		Column	
Roof	UB 356x171x57	C1	UC 356x406x287
Floor	UB 457x191x74	C2	ditto

An analysis from GSA [Oasys, 2001] showed that this rigid elevation is a sway frame (λ_{cr} >4). It is acknowledged that the member sections, especially the columns are oversized, so it was decided use the amplification factor to check for sway sensitivity. The results of applying the amplification factor are presented in TableB-2.

TableB-2 The Results of using implication factor to check the member size for rigid frame along gridline Ⓐ/Ⓕ

Floor

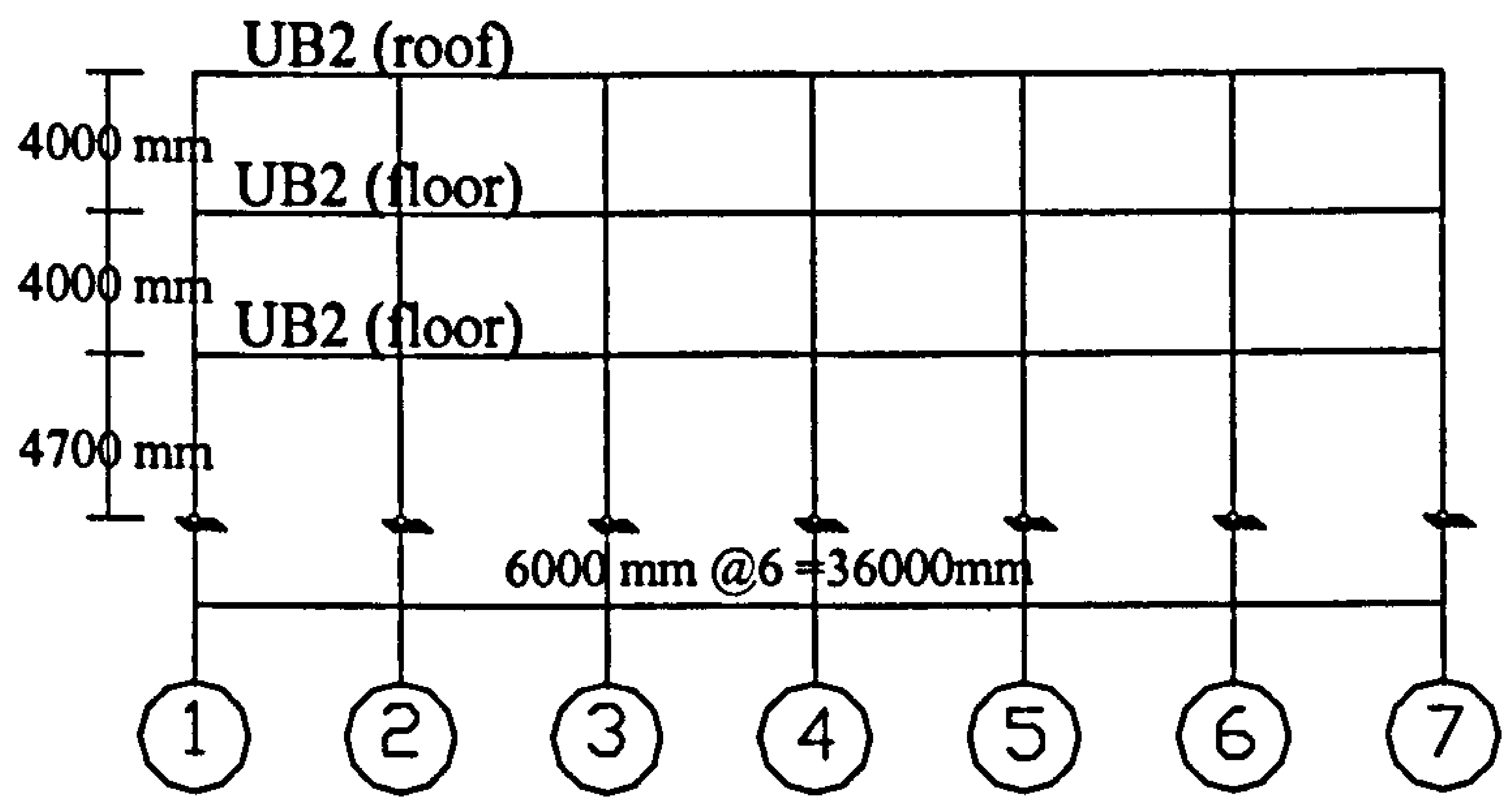
load case	NHF	λ_{cr}	k_{amp}	Bending (kN.m)			
				normal	restrain	sway	new effects
1.4D+1.6I	0.5%(1.4D+1.6I)	4.35	1.24	294	299	-5	300
1.2D+1.2I+1.2W	0.5%(1.2D+1.2I)	5.30	1.15	272	241	31	236
1.4D+1.4W	0.5%(1.4D)	10.3	1.04	214	173	41	256
1.D+1.4WI	0.5%(1.0DI)	14.7	n/a	167	123	44	211

Roof

load case	NHF	λ_{cr}	k_{amp}	Bending			
				normal	restrain	sway effect	new effects
1.4D+1.6I	0.5%(1.4D+1.6I)	4.4	1.24	148	171	-23	177
1.2D+1.2I+1.2W	0.5%(1.2D+1.2I)	5.3	1.15	139	138	1	138
1.4D+1.4W	0.5%(1.4D)	7.99	1.04	108	94	14	123
1.D+1.4WI	0.5%(1.0DI)	11.2		86	66	20	

The results have shown that when the amplification factor k_{amp} is applied to this sway frame, the members are adequate to resist the sway effects. Therefore, the member sections of this rigid frame do not need to be increased to be a non-sway frame.

A similar test was conducted to the rigid frames along gridlines Ⓑ–Ⓔ, and the geometry details are presented in FigureB-3



FigureB-3 Geometry Details about rigid frame along gridline Ⓑ-Ⓔ

The member section of this rigid frame along letter gridline-Ⓑ-Ⓔ is listed in Table B-3.

Table B-3 Member section about rigid frame along gridline Ⓑ-Ⓔ

Beam B2		Column	
Roof	UB 406x178x54	C3	UC 356x406x287
Floor	UB 457x191x82	C4	ditto

This rigid frame was also a sway frame, therefore similar test used the amplification factor was conducted, and results were presented in TableB-4.

TableB-4 The Results of using implication factor to check the member size for rigid frame along gridline Ⓑ-Ⓔ

Floor

load case	NHF	λ_{cr}	k_{amp}	Bending (kN.m)			
				normal	restrain	sway	new effects
1.4D+1.6I	0.5%(1.4D+1.6I)	4.59	1.21	411	417	-6	418
1.2D+1.2I+1.2W	0.5%(1.2D+1.2I)	5.64	1.13	359	331	28	390
1.4D+1.4W	0.5%(1.4D)	8.40	1.03	238	184	54	294
1.D+1.4WI	0.5%(1.0DI)	12.00	n/a	175	131	44	219

Roof

load case	NHF	λcr	kamp	Bending			
				normal	restrain	sway effect	new effects
1.4D+1.6I	0.5%(1.4D+1.6I)	4.59	1.21	208	238	-30	244
1.2D+1.2I+1.2W	0.5%(1.2D+1.2I)	5.64	1.13	184	189	-5	190
1.4D+1.4W	0.5%(1.4D)	8.40	1.03	112	94	18	131
1.D+1.4WI	0.5%(1.0DI)	12.00		89	99	-10	89

The tie beam UB3 and UB4 are not the major load bearing member, the detailed calculation can be found in Appendix C. The member section for this pin-rigid frame can be found in TableB-5

TableB-5 Member section for 3 storey pin-rigid frame

Beam			Column	
	Roof	Floor		
B1	UB 305x165x54	UB 457x191x74	C1	UC 356x406x287
B2	UB 406x178x54	UB 457x191x82	C2	ditto
B3	UB 305x127x42	UB 457x152x67	C3	ditto
B4	ditto	ditto	C4	ditto

Appendix -C

Design Details of the pin-pin frame

C.1 Design of the beams –roof and floor

Roof Beam

Loading kN/m^2

150 PC slab	2.33	kN/m^2
Finishes	1.8	kN/m^2
40 screed	1.2	kN/m^2
Total Dead load	5.33	kN/m^2
Impose	1.5	kN/m^2
Un-factored	7.0	kN/m^2
Factored	10.	kN/m^2

Brick Cladding (weight $2100 kg/m^3$)

Thickness(mm)	103x2=206	
Height(mm)	1000	
SW (kN/m) 2100x9.8x0.21x1		Un-factored 4.3 kN/m Factored 6 kN/m

For RB1 (roof beam)

Length L (m)		6
Loading width (m)	=7.5/2	4
Design loading w_i (kN/m)	=10x4+6	46
Bending moment M_x (kN.m) $= 1/8(w_i L^2) = 1/8\ (46 \times 36)$		207
Shear Force F_x (kN)	½ $(w_i L)$ =1/2(46 x 6)	138

Preliminarily choice 356 x 171 x 57

I_{xx} (cm^4)	16000	B.M. (kN.m)	$M_{cx}=\sigma S_x = 278$	M_x/M_{cx}-0.74
S_x(cm^3)	1010			
t (mm)	8.1	S.F. (kN)	$F_v=0.6\sigma tD=479$	$F_x/F_v=0.29$
D (mm)	358			

Section is satisfactory

Check the deflection δ

RB1 is the edge beam so check <u>*Dead + Impose*</u>

Un-factored loading (kN/m)	=4.3+6.0 x 4	29
Force W (kN)	=29 x 6	174
Limit of Deflection (mm)	=L / 360	16.7
δ (mm)=$5WL^3 / 384\ EI_{xx}$		15.3

Section is ok

Reaction(un-factored)

R_d (kN) =½ x (5.33 x 4 + 4.3) x 6 = 77

R_i (kN) = ½ x 1.5 x 4 x 6 = 18

<u>For RB2</u> <u>(roof beam)</u>

Length L(m)		6
Loading width (m)		7.5
Design loading w_i (kN/m)	=10 x 7.5	75
Bending moment M_x (kN.m) = 1/8($w_i L^2$) = 1/8 (75 x 36)		338
Shear Force F_x (kN)	½(w_iL)=1/2(75 x 6)	225

Preliminarily choice <u>**457 x 152 x 60**</u>

I_{xx} (cm^4)	25500	B.M. (kN.m)	$M_{cx}=\sigma\ S_x$ =355	M_x/M_{cx}=0.95
S_x(cm^3)	1290			
t (mm)	8.1	S.F. (kN)	$F_v=0.6\sigma tD$=608	F_x/F_v=0.37
D (mm)	454.6			

Section is satisfactory

Check the deflection δ

RB2 is middle beam therefore check the <u>Impose</u> only

Un-factored loading (kN/m)	=1.5 x 7.5	12
Force (kN)	=12 x 6	72
Limit of Deflection (mm)	=L / 360	16.7
δ (mm)=$5WL^3 / 384\ E\ I_{xx}$		4

Section is ok

Reaction(un-factored)

R_d (kN) = ½ x (5.33 x 7.5) x 6 = 120

R_i (kN) =½ x 1.5 x 7.5 x 6 = 34

<u>For RB3</u> <u>(roof beam)</u>

Length L(m)	7.5

Loading width (m)		0.5
Design loading w_i (kN/m)	= 10 x 0.5 + 6	11
Bending moment M_x (kN.m) = 1/8 ($w_i L^2$) = 1/8 (11 x 56)		77
Shear Force F_x (kN)	½(w_i L) = 1/2(11x7.5)	42

Preliminarily choice <u>305 x 127 x 42</u>

I_{xx} (cm^4)	8200	B.M. (kN.m)	M_{cx}=σ S_x =169	M_x/M_{cx}=0.46
S_x(cm^3)	614			
t (mm)	8.0	S.F. (kN)	F_v=0.6σtD=406	F_x/F_v=0.10
D (mm)	307.2			
A_g(cm^2)	53.4	Axial force (kN) F_t=σA_g=1469kN		>122kN

[Clause 2.4.5.3 RB3 (edge tie) =0.25 x (10 x 7.5 + 6) x 6=122kN]

Check the deflection δ

RB3 is edge beam therefore needs check the <u>Dead+Impose</u>

Un-factored loading (kN/m)	=4.3+6.0 x 0.5	7.3
Force (kN)	=7.3 x 7.5	55
Limit of Deflection (mm)	=L / 360	20.8
δ (mm)=5WL^3/ 384 E I_{xx}		18.4

Section is ok

Reaction(un-factored)

R_d (kN) = ½ x (5.33 x 0.5+4.3) x 7.5 = 27

R_i (kN) = ½ x 1. 5 x 0.5 x 7.5 = 3

<u>For RB4</u> <u>(roof beam)</u>

Length L(m)		7.5
Loading width (m)		1
Design loading w_i (kN/m	=10 x 0.5	5
Bending moment M_x (kN.m) = 1/8 ($w_i L^2$)=1/8 (5 x 56)		35
Shear Force F_x (kN)	1/2w_i L=1/2 (5 x 7.5)	20

Preliminarily choice as the RB3 <u>305 x 127 x 42</u>

I_{xx} (cm^4)	8200	B.M. (kN.m)	M_{cx}=σ S_x =169	M_x/M_{cx}-0.21
S_x(cm^3)	614			
t (mm)	8.0	S.F. (kN)	F_v=0.6σtD=406	F_x/F_v=0.05
D (mm)	307.2			

$A_g(cm^2)$	53.4	Axial force (kN) $F_t = \sigma A_g = 1469kN$	>225kN

[Clause 2.4.5.3 RB4 middle tie =0.5 x (10 x 7.5) x 6=225kN]

Section is satisfactory

Check the deflection δ

RB3 is edge beam therefore needs check the Impose only

Un-factored loading (kN/m)	=1.5 x 1	1.5
Force (kN)	=1.5 x 7.5	12
Limit of Deflection (mm)	=L / 360	20.8
δ (mm)=$5WL^3/384\ E\ I_{xx}$		5

Section is ok

Reaction(un-factored)

R_d (kN) = ½ x (5.33 x 1) x 7.5 = 20

R_i (kN) =½ x 1.5 x 1 x 7.5 = 6

Floor Beam

Loading kN/m²

150 PC slab	2.33 kN/m²
Finishes	1.0 kNm²
40 screed	1.2 kN/m²
Total Dead load	4.53 kN/m²
Impose (5+1)	6.0 kN/m²
Un-factored	11 kN/m²
Factored	16 kN/m²

Brick Cladding (weight 2100kg/m³)

Thickness(mm) 103x2=206

Height(mm) 4700

SW (kN/m)	2100x9.8x0.21x4.7	Un-factored 20 (kN/m) Factored 28 (kN/m)

For FB1 (Floor beam)

Length L (m)		6
Loading width (m)	=7.5 / 2	4
Design loading w_i (kN/m)	=16 x 4+28	92
Bending moment M_x (kN.m) $=1/8(w_iL^2)=1/8(92 \times 36)$		414

Shear Force F_x (kN)	½(w_iL)=1/2(92 x 6)	276

Preliminarily choice <u>457x191x74</u>

I_{xx} (cm^4)	33300	B.M. (kN.m)	$M_{cx}=\sigma S_x$ =454	M_x/M_{cx}=0.91
S_x(cm^3)	1650			
t (mm)	9.0	S.F. (kN)	$F_v=0.6\sigma tD$=679	F_x/F_v=0.41
D (mm)	457			

Section is satisfactory

Check the deflection δ

FB1 is the edge beam therefore needs check <u>Dead +Impose</u>

Un-factored loading (kN/m)	=20+11x 4	64
Force W (kN)	=64 x 6	384
Limit of Deflection (mm)	=L / 360	16.7
δ (mm)=$5WL^3$ / 384 E I_{xx}		16.2

Section is ok

Reaction (un-factored)

R_d (kN) = ½ x (4.53 x 4+ 20) x 6 = 115
R_i (kN) = ½ x 6 x 4x 6 = 72

<u>***For FB2*** (Floor beam)</u>

Length L(m)		6
Loading width (m)		7.5
Design loading w_i (kN/m)	=16 x 7.5	120
Bending moment M_x (kN.m) =1/8(w_iL^2) =1/8 (120 x 36)		540
Shear Force F_x (kN)	1/2w_iL=1/2 (120 x 6)	360

Preliminarily choice <u>457x191x89</u>

I_{xx} (cm^4)	41000	B.M. (kN.m)	$M_{cx}=\sigma S_x$ =553	M_x/M_{cx}=0.98
S_x(cm^3)	2010			
t (mm)	10.5	S.F. (kN)	$F_v=0.6\sigma tD$=803	F_x/F_v=0.45
D (mm)	463.4			

Section is satisfactory

Check the deflection δ

FB2 is middle beam therefore need check <u>Impose</u> only

Un-factored loading (kN/m)	=6 x 7.5	45
Force (kN)	=45x 6	270
Limit of Deflection (mm)	=L / 360	16.7

δ (mm)=$5WL^3/384\ E\ I_{xx}$	9.3

Section is ok

Reaction (un-factored)

R_d (kN) = ½ x (4.53 x 7.5) x 6 = 102
R_i (kN) = ½ x 6 x 7.5 x 6 = 135

For FB3 (Floor beam)

Length L(m)		7.5
Loading width (m)		0.5
Design loading w_i (kN/m)	=16 x 0.5+28	36
Bending moment M_x (kN.m) =$1/8(w_iL^2)$=1/8 (36 x 56)		252
Shear Force F_x (kN)	$1/2w_iL$=1/2(36 x 7.5)	135

Preliminarily choice **457x152x67**

I_{xx} (cm^4)	28900	B.M. (kN.m)	$M_{cx}=\sigma\ S_x$ =399	M_x/M_{cx}=0.63
S_x(cm^3)	1450			
t (mm)	9.0	S.F. (kN)	$F_v=0.6\sigma tD$=680	F_x/F_v=0.20
D (mm)	458			
A_g(cm^2)	85.6	Axial force (kN) Ft=σ Ag=2354kN		>222kN
[Clause 2.4.5.3		RB3 (edge tie) =0.25 x (16 x 7.5 +28) x 6 = 222kN]		

Check the deflection δ

FB3 is the edge beam therefore needs check Dead +Impose

Un-factored loading (kN/m)	=20+11x0.5	26
Force W (kN)	=26x7.5	195
Limit of Deflection (mm)	=L/360	20.8
δ (mm)=$5WL^3/384\ E\ I_{xx}$		18.5

Reaction(un-factored)

R_d (kN) = ½ x (4.53 x 0.5+20) x 7.5 = 86
R_i (kN) =½ x 6 x 0.5 x 7.5 = 12

For FB4 (floor beam)

Length L(m)	7.5
Loading width (m)	1

Design loading w_i (kN/m)	=16 x 0.5	8
Bending moment M_x (kN.m) = $1/8(w_iL^2)$ =1/8(8 x 56)		56
Shear Force F_x (kN)	$\frac{1}{2}(w_i L)$=1/2(8 x7.5)	30

Preliminarily choice <u>457 x 152 x 67</u>

I_{xx} (cm^4)	28900	B.M. (kN.m)	$M_{cx}=\sigma S_x$ =399	M_x/M_{cx}=0.14
S_x(cm^3)	1450			
t (mm)	9.0	S.F. (kN)	$F_v=0.6\sigma tD$=680	F_x/F_v=0.04
D (mm)	458			
A_g(cm^2)	85.6	Axial force (kN) Ft=σ Ag=2354kN		>383kN

[Clause 2.4.5.3 RB4 middle tie =0.5 x (17 x 7.5) x 6 = 383kN]

Check the deflection δ

FB1 is the edge beam therefore needs check <u>Impose</u> only

Un-factored loading (kN/m)	=6 x 1	6
Force W (kN)	=6 x 7.5	45
Limit of Deflection (mm)	=L / 360	20.8
δ (mm)=$5WL^3/384\ E\ I_{xx}$		4.3

Section is ok

Reaction(un-factored)

R_d (kN) = ½ x (4.53 x1) x 7.5= 17
R_i (kN) =½ x 6 x 1 x 7.5 = 23

C.2 Summary of design

Roof beam level

	Section size	Reaction		
		Dead(kN)		Impose (kN)
		½SW	½R_d	½R_i
RB1	UB 356x171x57	2.0	77	18
RB2	UB 457x152x60	2.0	120	34
RB3	UB 305x127x42	2.0	27	3
RB4	ditto	2.0	20	6

Floor beam level

	Section size	Reaction		
		Dead(kN)		Impose(kN)
		½SW	½R_d	½R_i
FB1	UB 457x191x74	3.0	115	72
FB2	UB 457x191x89	3.0	102	135
FB3	UB 457x152x67	3.0	86	12
FB4	ditto	3.0	17	23

C.3 Column Design -3-storey

Column length	Beams	Reaction		Own weight kN	Totals		Reduction (%)	Reduced design		Column size
		R_d (kN)	R_i (kN)		W_d (kN)	W_i (kN)		W_i (kN)	Load F (kN)	
Column1										
3-R	RB1	77	18	2						*256UC107*
	RB3	27	3	2	108	21	0	21	**185**	
2-3	FB1	115	72	3						
	FB2	86	12	3	315	105	10	95	**609**	
1-2	ditto	201	84	6	522	189	20	151	**973**	
G-1	ditto	201	84	6	729	273	40	164	**1283**	
Column2										
3-R	RB1	77	18	2						*256UC107*
	RB1	77	18	2						
	RB4	20	6	2	180	42	0	42	**319**	
2-3	FB1	115	72	3						
	FB1	115	72	3						
	FB4	17	23	3	436	209	10	188	**911**	
1-2	ditto	247	167	9	692	376	20	301	**1450**	
G-1	ditto	247	167	9	948	543	40	326	**1848**	
Column3										
3-R	RB2	120	34	2						*256UC107*
	RB3	27	3	2						
	RB3	27	3	2	180	40	0	40	**316**	
2-3	FB2	102	135	3						
	FB3	86	12	3						
	FB3	86	12	3	463	199	10	179.1	**935**	
1-2	ditto	274	159	9	746	358	20	286.4	**1503**	
G-1	ditto	274	159	9	1029	517	40	310.2	**1937**	
Column4										
3-R	RB2	120	34	2						*305UC118*
	RB2	120	34	2						
	RB4	20	6	2						
	RB4	20	6	2	288	80	0	72	**518**	
2-3	FB2	102	135	3						
	FB2	102	135	3						
	FB4	17	23	3						
	FB4	17	23	3	538	396	10	396	**1387**	
1-2	ditto	238	316	12	788	712	20	569.6	**2015**	
G-1	ditto	238	316	12	1026	1028	40	719.6	**2588**	

C.4 Bracing Design

Wind load

CP Ch V part2 1972
Basic wind speed V_B=40m/s
S_1=S_3=1.0 (no topography/ statistical considerations)
S_2 increase with height
C_f=1.0
1) 7.5 m bay
Building width =45m
$Vs=S_1 S_2 S_3 V_B$ $q=0.613V_S^2$ (N/m^2)

Height(m)		S2	Vs	q(kN/m2)	A(m2)	Force P(kN)	Foce F(kN)
R	13.7	1.10	44.0	1.187	45	53	27
3	12.7	0.85	34.0	0.709	180	128	90
2	8.7	0.78	31.2	0.597	180	107	117
1	4.7	0.67	26.8	0.440	212	93	100
G							47

Σ=381

2) 6m bay
Building width =36m
$Vs=S_1 S_2 S_3 V_B$ $q=0.613V_S^2$ (N/m^2)

Height(m)		S2	Vs	q(kN/m2)	A(m2)	Force P(kN)	Foce F(kN)
R	13.7	1.10	44.0	1.187	36	43	21
3	12.7	0.85	34.0	0.709	144	102	72
2	8.7	0.78	31.2	0.597	144	86	94
1	4.7	0.67	26.8	0.440	169	74	80
G							37

Σ=305

Notional horizontal force (NHF)

F_{NHF}=0.5 %(1.4 D +1.6 I)
F_{NHF_roof}=0.0475kN/m^2
F_{NHF_Floor}=0.085kN/m^2
$A_{Floor (roof)}$ = 45 x 36 =1620m^2

Along x-axis

NHF per tower = 0.085x1620/6=23kN

3-Story High

Level	NHF (kN)
R	13
3	23
2	23
1	23
G	23
	Σ=105

Along y-axis

NHF per tower= 0.085x1620/7=20kN

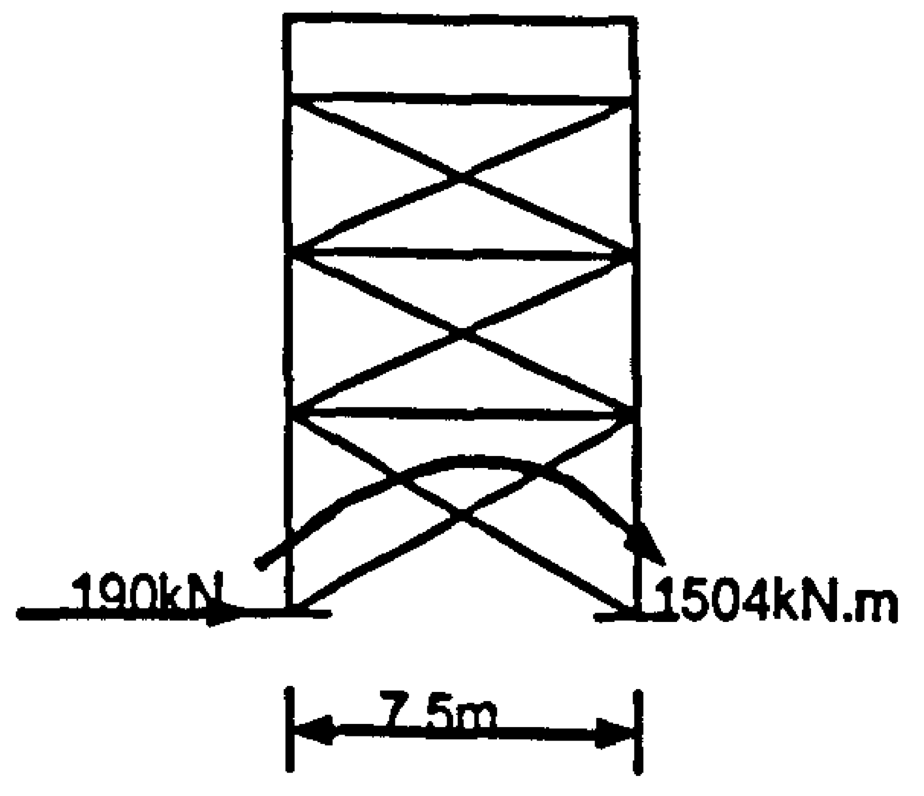
3-Story High

Level	NHF (kN)
R	11
3	20
2	20
1	20
G	20
	Σ=91

Bracing

3-story high

Height(m)		7.5m		6m	
		Foce F(kN)	Mmt(kN./m)	Foce F(kN)	Moment
R	13.7	27	366	21	293
3	12.7	90	1149	72	919
2	8.7	117	1022	94	818
1	4.7	100	471	80	377
G		47		37	
Σ		381	3008	305	2406
Σ/4		95	752	76	602



7.5m Bay

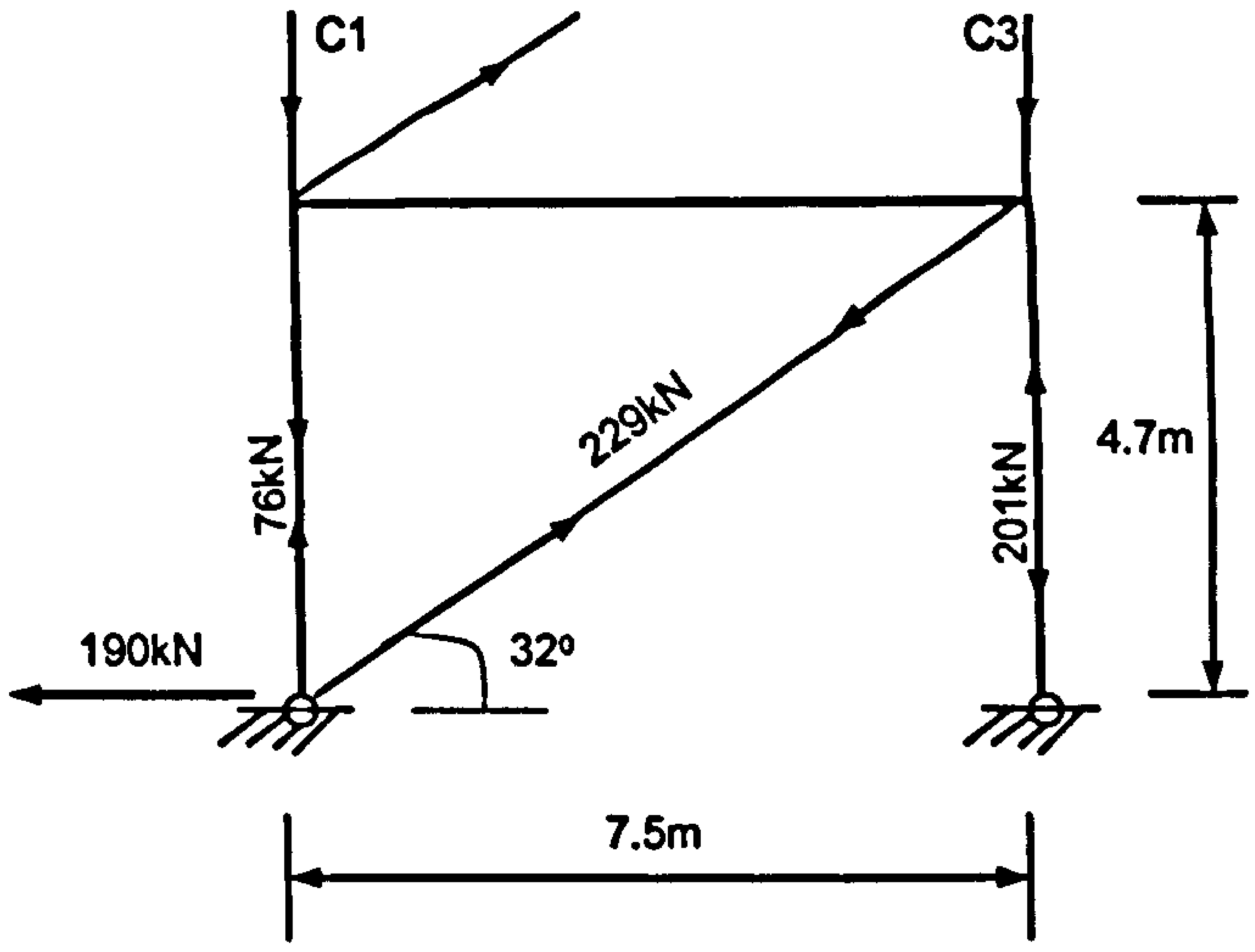
Force for one lateral wind bracing from table =95kN

Total force of each wind bracing

W_w=95x2 =190kN

Moment on each wind bracing

M_w=752 x 2=1504 kN.m



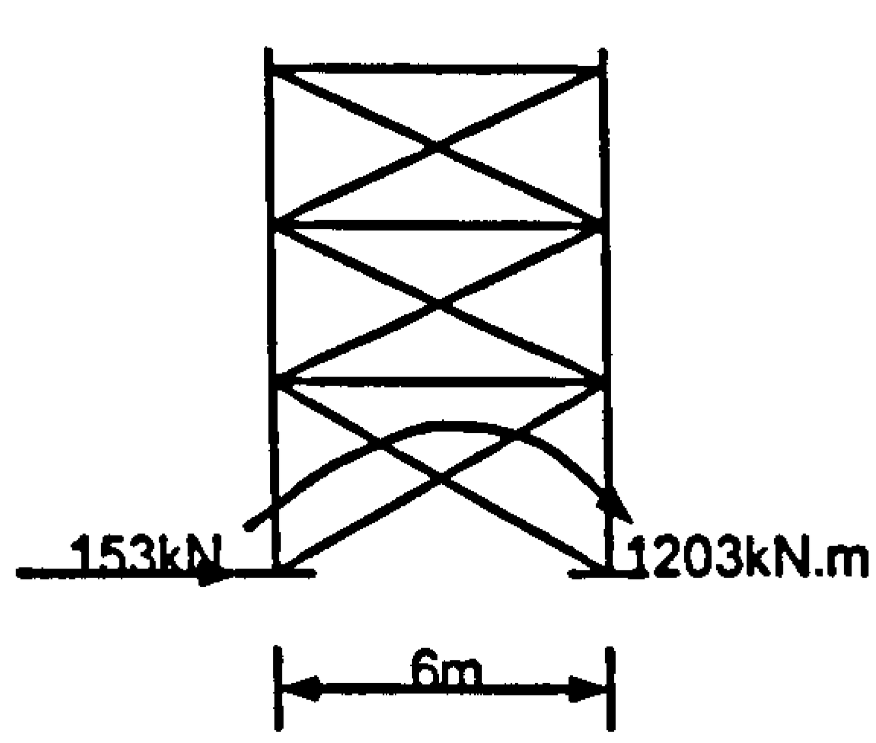
R_V=1504/7.5=201kN

R_H=190 kN

F_{C3}=201kN compression

$F_{diagonal}$ = 190/cos32° =229 kN tension

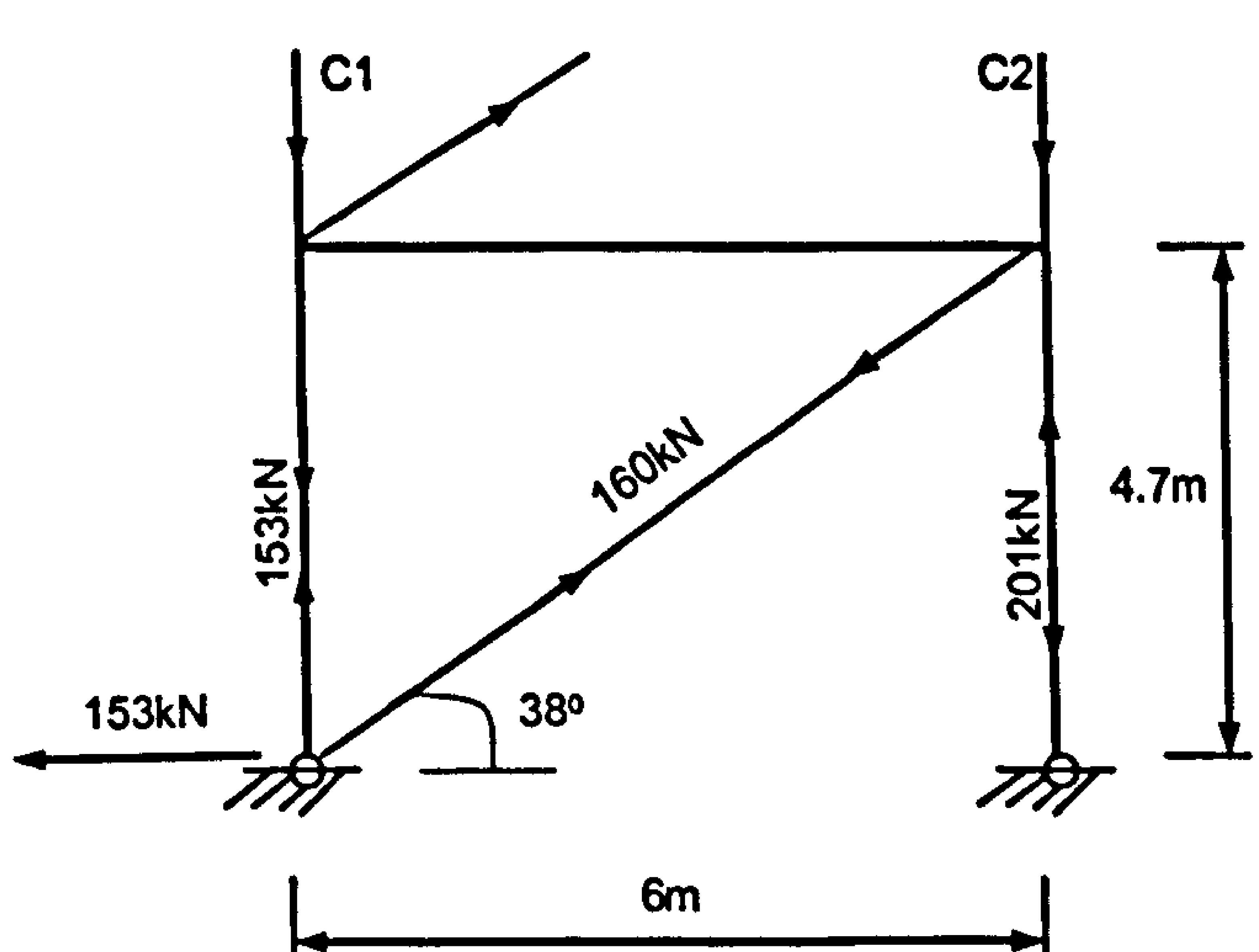
F_{C1}= 201-229sin32°= 76kN



6m Bay

Force for each lateral wind bracing
W_w=153kN

Moment on each wind bracing
M_w=2406/4 =1203 kN.m



R_V=1203/6=201kN

R_H=153kN

F_{C2}=201 kN compression

$F_{diagonal}$ = 153/cos38 ° =160 kN tension

F_{C1}= 210-153sin38°=153kN tension

7.5m /6 mbay

	Wd	Wi	Ww		Compression			Tension
			c	t	$1.4W_d+1.4W_w$	$1.2(W_d+W_i+W_w)$	$1.4W_d+1.6W_i$	$1.0W_d-1.4W_w$
C1	729	191	201	153	1302	1345	1326	515
C2	948	380	201	153	1609	1835	1935	734
C3	1029	362	201	76	1722	1910	2020	923
Maximum							♦	

The section was fine

Diagonal member

The force(maximum) due to wind only W_w=229kN tension
1.4 W_w=158kN tension/ compression

Choose a 90 x 120 plate

I_{xx}= 1.296x10^{7}mm^4 I_{yy} =7.29x 10^{6}cm^4 L=8.85m

$$P_{crx}=\frac{\pi^2 EI_{xx}}{L^2}=335\text{kN} >158$$

$$P_{cry}=\frac{\pi^2 EI_{yy}}{L^2}=188\text{kN}>158$$

As =10800mm^2

Pt =10800 x 275 = 2970kN>158