A Review of Structural Failures in Ireland

Trinity College Dublin

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A Thesis submitted for the degree of Master of Science

by

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Abstract

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Ian James Duckenfield

Many technical papers have been written and presented to the structural engineering community in Ireland giving details of major projects successfully completed. These projects are on time and under budget.

Far more interesting to practising engineers are the cases where things go wrong, and in some cases very badly wrong. Unfortunately the lessons learnt in such cases are hidden from view. No engineer wants to admit that he made an error which now, with the benefit of hindsight, is obvious. The risks of structural failures are extremely low, especially when compared to activities such as car driving and smoking. Nevertheless lives have been lost and life-altering injuries sustained by structural collapses in Ireland.

The author has over 40 years of structural design experience and has been collecting examples of failures and near misses. This includes cases where the author acted as an expert witness relating to failures. As part of the research work for the thesis, a number of senior structural engineers have been interviewed and their own experiences of failures collected. The methodology of the collection of the data is given in the thesis noting in particular the sensitive nature of the subject material.

This thesis presents 32 of these cases. They involve all the major structural materials and the consequences range from incidents involving multiple fatalities to purely financial loss. The 32 were selected from a longer list on the basis of the extent of the technical details and the interesting lessons learnt.

This thesis analyses these cases and outlines the structural issues involved. Further analysis of these cases involves splitting the results into the material used, the cause of the designer's error or the cause of the contractor's error. A further breakdown of the results considered the severity of the error. These analyses were undertaken to study if there were any trends.

The conclusions of the study are that there are a very wide range of errors made by both the designer and contractor. No cases were found where a new type of failure has taken place – all the causes are explicable by contemporary structural theory.

Although there is no dominant striking pattern to be seen in the cases studied, certain trends can be seen. They are:

- Contractor error is more likely to be the cause of the failure than a structural designer error. The split is around 60% contractor to 40% designer. This is seen as a significant result and should be viewed as significant to those assuming the Assigned Certifier role under the Building Control Amendment Regulations.
- The three potentially most serious failures were all designer errors. The potential outcomes of these errors were multiple fatalities. The three structures were a large bridge which is curved on plan, a set of precast stairs in a school and a building built on concrete piles in ground with heavy chemical contamination. The three cases have significantly different underlying causes.
- All three failures relating to composite decks were contractor errors. Again, the underlying causes are significantly different. This is a small sample but perhaps more careful site supervision of composite decks should take place.
- It is well known that masonry is constructed all over Ireland with little or no structural engineering input. Cases involving multiple fatal incidents with horizontal forces acting on walls are outlined. Contractor error is largely blamed in these. Site supervisors should be aware of the weakness of masonry walls subjected to horizontal pressures from such sources as wind, or crowd loads.

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1 Introduction

1.1 Historical Background

The oldest known structure in the world is a 23,000 year old wall at the entrance to what is known as the Theopetra Cave in Greece. Compared to this, Ireland's oldest structure – Newgrange – is a relatively modest 5,000 years. Newgrange is, however, older than the Pyramids in Egypt (Smyth, 2008 and Handwerk, 2016).

Rules for building are also ancient. King Hammurabi of Babylon in 1780 BCE produced a set of laws governing a number of things including construction. These rules were along the lines of "an eye for an eye". Law number 229 states that "If a builder build a house for someone, and does not construct it properly, and the house which he built fall in and kill its owner, then that builder shall be put to death." These laws have no reference to religion and God and have been called the first laws with separation of Church and State (Landau, 1997).

By the Renaissance period, from the 14th to the 17th century, failures were regarded as the price of progress and considered as Acts of God.

The Great Fire of London of 1666 destroyed 80% of the city. This led to the production of the first comprehensive London Building Act of 1667, the first to provide for surveyors to enforce its regulations. It laid down that all houses were to be built in brick or stone. The number of storeys and width of walls were carefully specified. Streets were required to be wide enough to act as a fire break (Manco, 2009).

The evolution of Building Codes was often the product of structural failures involving injury and loss of life. It has been said:

"A good judgment is usually the result of experience.
And experience is frequently the result of bad judgment."
(Petroski, 1985)

1.2 What are the Risks of Structural Failure?

This section was written a month after the balcony collapse in Berkeley, California which took the lives of 6 young people, 5 of them Irish (Mayton, 2015). The preliminary indications suggest dry rot in the timber (refer to Fig 1.2.1).

Significantly, Berkeley City Council have introduced new legally enforceable regulations which require all existing balconies, decks and stairs to be inspected within 6 months of the enactment of the legislation and subsequently every 3 years. Other regulations tighten up regarding materials used and structural design aspects. These regulations do not apply outside the boundary of the City of Berkeley. Perhaps other jurisdictions will amend their regulations following further loss of life.

Here regulations can be seen as reacting to a tragic event. This is discussed further in Chapter 2 of this thesis.

Fig 1.2.1 Fatal balcony collapse at Berkeley (TheJournal.ie, 2015)

What are the risks of structural failure compared to other hazards? Refer to Table 1.1.

Activity	Risk of Death x 10^{-8} hour (FAR- Fatal Accident Rate)
Rock climbing , while on rock face	4,000
Travel by helicopter	500
Travel by motorcycle and moped	300
Construction, high rise erectors	70
Smoking	40
Travel by air	15
Travel by car	15
Construction, average	5
Travel by bus	1
Accident at home, able bodied	1
Buildings falling down	0.002

Table 1.1 Risk of death on standardised basis for certain activities. (Tietz, 1999)

The Central Statistical Office in the UK – Annual Abstract 1974 (CSO, 1974) quotes:

Deaths attributable to structural failure

- *During construction* *8 average per annum*
- *Completed structures* *6 average per annum*

These statistics are over 40 years old and it is reasonable to believe that the figure for “during construction” has reduced significantly with the increased emphasis given to safety on site in the last number of years.

The statistics for deaths on construction sites is discussed further in Section 2.2.5 of this thesis.

Society's attitude to risk is not consistent. Smoking kills some 5,500 people every year in the Republic, whereas road deaths in 2015 were 162 – around only 3% of the smoking toll (HSE, 2017 and An Garda Siochana, 2016). What would society's response be if deaths from structural failures were at the rate of 5,500 per annum? It could be stated that the response would be massive and yet smoking kills this number with something of a shrug of the shoulders response.

Although society may not have a major concern regarding the risks of structural failure, practitioners most certainly do.

1.3 Objectives of this Research

There is no shortage of technical papers which analyse projects that are a total success – on time and under budget. Experience indicates, however, that things do not always go according to plan.

Engineers talk and there is a flow of conversations regarding problems, errors, fixes, etc. The author has been collecting anecdotes and lessons learnt for almost all his 40 year's structural engineering experience.

These failures and lessons learnt would include failures which occurred overseas which appear in the technical press. Local Irish failures tend to be less dramatic than the failures from abroad but the technical causes behind them are no less worthy of a wider audience.

Structures in Ireland tend to be smaller than those in other countries and there is less innovation regarding new techniques and materials. The Irish construction industry is smaller than equivalent industries in other jurisdictions.

This thesis will present a number of Irish failures. Their causes will be considered. Are there any patterns to be noted? Are the number of cases studied statistically relevant? The cases will be listed according to the material of construction, cause of failure and consequences.

A comparison will be made to establish if any patterns can be observed and if any overall lessons learnt can be extracted.

Therefore, the detailed objectives of this research are:

1. To present a series of case studies of structural failures that occurred in Ireland, but where the technical details have not been published.

2. In each case study, to identify and report the principal and secondary causes of each failure.

3. To analyse the multiple case studies to identify any common causes or features which are shared by many failures.

4. To identify whether structural failures are more frequently caused at the design stage or at the construction stage.

5. To critique the methodology used to collect the information the case studies with a view to suggesting improvements that would maximise the willingness of practising engineers to

participate in further studies and would minimise the risk of a researcher's bias skewing the results.

These objectives are discussed further in Chapter 4 and Section 5.3.

1.4 Methodology

1.4.1 General

The methodology used by

- CROSS
- Elskes
- Scheer

will be discussed in the following sections.
The methodology used in this thesis is outlined in Section 1.4.5.

1.4.2 Methodology used by CROSS

CROSS is a body and its name stands for Confidential Reporting on Structural Safety. It is promoted by the Institution of Structural Engineers, the Institution of Civil Engineers and the Health and Safety Executive (UK). CROSS is keen to receive reports from individuals and organisations on matters in relation to structural safety. CROSS publishes a Newsletter every quarter (CROSS, 2016). This is regarded as an important contribution to the construction industry and in particular to practising structural engineers.

CROSS has indicated that it has no standard format for receiving reports of structural failures, near misses and similar structural engineering matters (Soan, 2016 personal communication). CROSS publishes a concise summary of the report. This is deliberately a short version of the report saying that a long, detailed version would lose the interest of the typical reader. If CROSS has extensive details of a particular matter, then an additional report is linked on the CROSS website to a pdf. Therefore a reader with a particular interest in a specific matter can study a more detailed version.

In the CROSS reports, an account is given of what went wrong, and offers an opinion as to how similar occurrences might be avoided in the future. CROSS tries not to apportion blame.

1.4.3 Methodology used by Elskes

Edouard Elskes (1859 – 1947) was a prominent Swiss bridge engineer. He worked in England, France and Italy and became the Engineer in Chief of the Swiss Federal Railway. It is believed that he produced the oldest summarising documentation of building accidents (Scheer, 2010). His paper "Rupture des ponts metalliques: etude historique et statistique" or "Failure of metal bridges: a historic and statistical study" was published in the French language in 1894 (Scheer, 2010).

Elskes described the collapse of 42 iron bridges between 1852 and 1893 and listed them in a table and attributed the numbers of collapses as below

• Failure of foundation	8
• Failure due to unusual effects, e.g. impact	4
• Collapse during construction or dismantling	10
• Failure during load testing	9
• Insufficient load-bearing capacity without other recognizable causes	11
Total	42

There is no indication in his paper of a formal structure to how these cases were made known to him, how they were investigated or how they were classified.

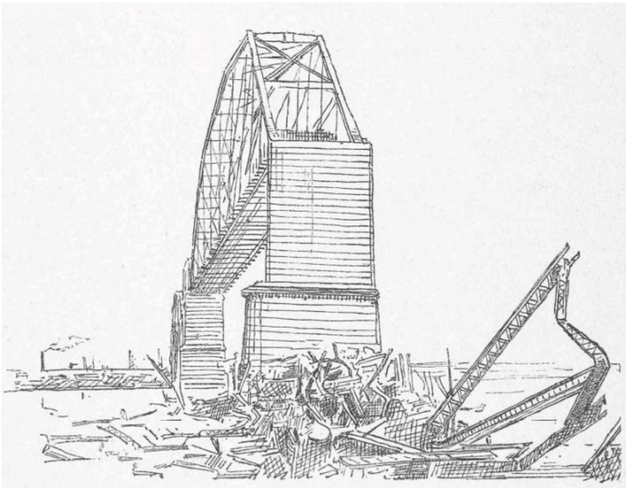


Fig 1.4.1 Collapse of Louisville Bridge (Elskes, 1894)

The analysis of each case is extremely perfunctory. For example, the collapse at Louisville gets a few lines in the text and only one word in the summary table – ouraga (or hurricane in English).

1.4.4 Methodology used by Scheer

Joachim Scheer is a German University Professor who has specialised in the study of structural failure and has worked on German code development committees (Scheer, 2010).

His most important publication is “Failed Bridges: Case Studies, Causes and Consequences”. The first edition had 446 case studies and the 2010 edition had 536 case studies. Scheer breaks these down into cases for which he has detailed information regarding the case and those for which he does not (refer to Table 1.4.2).

Ref	Content	Number of cases with detailed information	Number of cases without detailed information
1	Failure during construction	105	20
2	Failure in service without external action	107	35
3	Failure due to impact of ship collision	59	5
4	Failure due to impact from traffic under the bridge	19	0
5	Failure due to impact from traffic on the bridge	21	5
6	Failure due to flooding, ice floes, floating timber and hurricane	41	13
7	Failure due to fire or explosion	22	4
8	Failure due to seismic activity	6	0
9	Failure of falsework	60	14
Total		440	96

Table 1.4.2 Table of Failure Categories and the Cases Covered (Scheer, 2010)

It is clear from the Table 1.4.2, that the categories are arbitrary. For example, flooding, ice floes, floating timber and hurricane are listed as one category.

Interestingly, Scheer notes that, as the years pass, it is increasingly difficult to get information from the relevant parties as considerations of the legal implications of issues increasingly come to the fore.

Sheer deliberately refrains from repeating the well documented investigations regarding the cases. He does, however, refer to the investigations when he reaches alternative conclusions to those in the published work.

Sheer stresses that, by the nature of the study, the findings are subjective and that detailed statistical analysis is not possible. The subjective nature of the allocation of cause is a feature of the 32 cases given in this thesis. It is considered to be impossible to remove all elements of bias in giving causes of failures.

Although Scheer states that detailed analysis of the causes is not possible, he does however state that he can give general trends.

One of the striking conclusions of Scheer's work is his statement: “I find that there is hardly any case which could have been prevented by more detailed calculation”.

He goes on to say that “The basic cause of most catastrophes was either that possibilities of failure were never even considered, conditions were not thoroughly investigated or that in some way rashness or even foolishness was predominant during design or construction. Also on some occasions, successful structures have been the cause of failure in later structures when seemingly unimportant changes, such as in size or slenderness, turned secondary factors into major influences”.

Work by two other engineers studying structural failures have arrived at the same conclusions (Smith, 1976 and Hahn, 1970).

1.4.5 Methodology used in this Thesis

CROSS publishes short reports on the cases reported to it and makes no apportionment of blame. Elskes published his work in 1894. He gives no indication as to how the cases were investigated or classified. Scheer gives little information regarding the methodology of his work but states that many of the cases in his study have been thoroughly investigated in official reports.

A number of senior engineers in different offices were asked for their experiences regarding structural failures in Ireland. The engineers' roles were as designers, site supervisors, expert witnesses, etc.

The methodology was principally based around a series of one-to-one face-to-face interviews with experienced engineers known to the author. A formal checklist was not used and note taking was limited. This was to ensure that the informal nature of the interview was maintained. This approach was considered more likely to establish the full details of the case which may reflect poorly on the interviewee.

A striking feature of this phase of the work was how willing individuals were to share their experiences. There was almost a confessional angle to some of the interviews. Designers who had made an obvious error were embarrassed as to how basic their error was. It should have been spotted by a young graduate. They were, however, very frequently happy to share the lessons learnt.

The main thrust of the questions concentrated on the engineering aspects rather than the liability issues. No formal questionnaire was used. It was considered that an informal discussion would be more likely to produce a more frank outlining of the problems encountered. Frequently interviews were conducted in the engineer's home. This tended to increase the informal nature of the interview. The sensitive nature of these interviews should be understood. The engineers questioned were frequently making frank admissions regarding their incompetence.

Thirty two cases are outlined in this thesis. They were selected from a longer list of reported failures. The cases were selected on the basis that:

- most of the technical details of the failure were known. Frequently original drawings were available and
- it was possible to extract from the data available the lessons learnt in the case.

There can be no definitive way of proportioning liability for the error between the categories chosen. By its nature there has to be a judgement call in this matter. This is discussed more in Section 1.4.4 where Sheer indicates the subjective nature of the allocation of cause.

The author is an experienced structural designer and has some expertise as a designer. However, this expertise may have a bias associated with it. The author is aware of this fact and has taken all possible steps to minimise such bias in matters such as the apportionment of blame.

The objective of the study was to learn from the failures. The author's personal contacts within the industry gave a tremendous advantage in getting engineers to admit their mistakes. A procedure that would have appeared too formal would have undermined the nature of the interaction.

The case studies are organised by material type to make any trends associated with that material easier to identify.

2 Literature Review

2.1 General

With one minor exception, no details of the 32 cases studied in Chapter 3 of this thesis have, to the best knowledge of the author, been published. By their nature developers, contractors and designers want to keep structural problems and indeed collapses away from the attention of the public. Practitioners working in the construction industry are extremely aware that they run the risk of exposure to crippling financial liabilities.

The three masonry wall failures given in the thesis have appeared in the Irish national press (Irish Times 24 July 1986, Irish Examiner 20 July 2007 and Irish Independent 9 November 2010). These reports, however, gave no technical information regarding wall dimensions and the like.

In the area of the study of failures one name stands out – Henry Petroski. He is professor of civil engineering (and also rather curiously, history) at Duke University, North Carolina. Petroski has had a distinguished career and has published a number of books. These include "To Engineer is Human: the Role of Failure in Successful Design" (Petroski 1985). In this book Petroski not only analyses classic structural failures such as Tacoma Narrows and Hyatt Regency but looks at the wider role of engineering design for items such as car fuel tanks and the development of aircraft frames. He cautions against over reliance on computer output. Case study 28 of this thesis gives a real life example of just such reliance.

One quote from his book is:

> "The factor of safety is a number that has often been referred to as a "factor of ignorance" for its function is to provide a margin of error that allows for a considerable number of corollaries to Murphy's Law to compound without threatening the success of an engineering endeavour. Factors of safety are intended to allow for the bridge built of the weakest imaginable batch of steel to stand up under the heaviest imaginable truck going over the largest imaginable pothole and bouncing across the roadway in a storm". (Petroski, 1985)

In his later book "To Forgive Design" (Petroski, 2012); he considers the inferior iron rivets on the Titanic and the cracks that grew from the corners of the square windows of the Comet aircraft of the 1950s.

Other quotes attributed to Petroski (Petroski, no date given) are:

> "We can't simply blame the engineers when things go wrong because, no matter how well we plan, things don't go according to plan."

and when referring to service life:

> "Typically, highway bridges have about 50 years. But over in England, they have iron bridges approaching 250 years. In France, there are Roman viaducts that are approaching 2,000 years old. So a bridge can last a very long time if it's built properly in the first place and then maintained properly."

For the more mundane structural failures, of the type more commonly seen in Ireland, CROSS is a major source of information. This UK Structural Safety organisation combines the functions of CROSS - Confidential Reporting on Structural Safety and SCOSS - The Standing Committee on Structural Safety (SCOSS). SCOSS is an independent body established in 1976 to maintain a continuing review of building and civil engineering matters affecting the safety of structures.

The Cross website outlines issues and problems from confidential reporters. Recent such problems include (CROSS, 2016 (A)):

- Steel corrosion even when concrete encasement is present
- Failure to check designs produced by software
- Failure of stainless steel bars in a marine environment
- Failure of a cladding panel in high wind

In the following sections of the thesis, a number of failures are outlined. These are both Irish (Section 2.2) and International (Section 2.3). Particular note is made of the way failures result in the development of codes of practice and statutory requirements.

Section 2.4 outlines some failures in other industries – railways, aviation and petroleum. Once again it can be seen that codes, protocols and the like result from failures involving loss of life and near misses.

It is of interest to compare the failures of other industries and those in other countries with the Irish structural failures.

Is it reasonable to come to the conclusion that significant code development only occurs following a failure? One exception would be the transition to limit state design from permissible stress codes. The principal topic of this thesis is the description and analysis of failures rather than the evolution of codes and standards.

2.2 Noteworthy Irish Cases and Development of Codes

2.2.1 General

This section covers four noteworthy Irish cases where codes of practice and indeed statutory requirements in the construction industry have been created from failures or near-misses. These are:

- Priory Hall and the Building Control (Amendment) Regulations
- Structural Bolt Failure and the HSA Code of Practice for Design and Installation of Anchors
- Malahide Viaduct Failure and the Railway Accident Investigation Unit revised procedures for inspection of structures near railways
- Construction site fatal accidents and the Safety, Health and Welfare (Construction) Regulations 2006

Unlike the Case Studies outlined in Chapter 3 of this thesis, names will be used in Sections 2.2, 2.3 and 2.4. This is because there are no confidentiality issues here. All the information presented in Section 2 is based on publicly available information.

It can be clearly seen that the evolution of design codes and other forms of regulation in the construction industry frequently result from structural failures and similar incidents. This pattern of new regulations coming into force following incidents up to and including multiple fatalities can be seen in other industries – refer to Section 2.4.

2.2.2 Priory Hall and the Building Control (Amendment) Regulations

Priory Hall is a development of 187 apartments at Donaghmede, North Dublin. After the completion of the construction work and after the apartments were occupied, a number of serious fire deficiencies were discovered (RTE, 2011). Reference should be made to Fig 2.2.2.a.

These deficiencies included:

- ineffective fire walls
- ineffective fire stopping at service penetrations through fire walls
- problems at smoke vents
- problems at ceilings

These problems were mainly covered up on completion of the construction work (Fig 2.2.2.b) and are therefore difficult and expensive to remediate.

The complex was condemned by the Fire Inspector and evacuated by court order in 2011 (RTE, 2011).

Further inspection work revealed that the properties were in a worse state than was known up to that time. Further problems unearthed included:

- problems with the gas installations
- problems with the electrical installations
- pyrite was found in the basement car park
- problems with the lifts
- problems with external wall panels

It transpired that around 80% of the owners were not covered by their insurance and that Local Authority inspections were grossly inadequate. This high profile case ended up in the Supreme Court. A deal was struck and ownership of the complex was transferred to the Local Authority (Dublin City Council) and residents no longer had mortgages for those properties. The human cost on the residents was appalling. Political pressure increased with the unfortunate suicide of a resident who was unable to handle his mounting debt together with living in a property described as unsafe. The Priory Hall story was seen as the worst example of Celtic Tiger construction with dreadful quality control (Irish Times, 2014).

Fig 2.2.2.a Priory Hall (completed 2007) (Northern Sound, 2013)

Fig 2.2.2.b Priory Hall (after investigation work) (TheJournal.ie, 2011)

The outcome of this debacle was not a revision to the Building Regulations (Government of Ireland, 2012). The Technical Guidance Documents are "*deemed to satisfy*" type documents and are broken down into Parts A to M and give the technical rules for all the different aspects of construction. However, a significant change to the way in which building activities were monitored and signed-off came about when the Building Control (Amendment) Regulations (BCAR) came into force in March 2014. These Regulations created a new self-certification regime. New roles came into being on a statutory basis – the Design Certifier and the Assigned Certifier (Government of Ireland S.I. 80, 2013).

A Code of Practice giving a detailed account of the roles and responsibilities has been published by the Department of the Environment (Government of Ireland, 2013). In clause 4.2.1

the Code gives an indication of who can sign as the Design Certifier and/or as the Assigned Certifier:

> *(a) Architects that are on the register maintained by the RIAI under Part 3 of the Building Control Act 2007; or*
>
> *(b) Building Surveyors that are on the register maintained by the SCSI under Part 5 of the Building Control Act 2007; or*
>
> *(c) Chartered Engineers on the register maintained by Engineers Ireland under section 7 of the Institution of Civil Engineers of Ireland (Charter Amendment) Act 1969.*
>
> *Similarly, the Design Certifier must be one of the above registered professionals and must be competent to carry out their design and to co-ordinate the design activities of others for the works concerned.*

Additionally, the Code sets out the role of individual designers within different design disciplines (e.g. structural, civil, electrical, architectural) and how they co-ordinate with the overall Design Certifier.

> Designers should:-
>
> a) design their respective elements of work in accordance with the applicable requirements of the Second Schedule of the Building Regulations;
>
> b) provide the Design Certifier with the necessary plans, specifications and documentation that is required for lodgement at commencement stage;
>
> c) arrange to provide sufficient information to the Assigned Certifier to enable them to fulfil their role;
>
> d) as agreed with the Assigned Certifier, carry out work inspections which are pertinent to their elements of the Design, and liaise with the Assigned Certification terms of this and the required ancillary certification;
>
> e) notify the Assigned Certifier of their proposed inspection regime for inclusion in the overall Inspection Plan;
>
> f) provide the Ancillary Certificates when required by the Assigned Certifier and Design Certifier
>
> g) maintain records of inspection.

Interestingly, only fourteen months after the BCAR Regulations came into force they were revised to exclude one-off houses and domestic extensions. This relaxation was not universally welcomed by the construction industry and questions were asked as to what was the driving force behind it. The implication of this is that a large section of the construction industry is now not subject to the regulations. Arguably it is the section which most requires tighter control.

The BCAR Regulations have revolutionised both the design process and the supervision requirements on site. It is considered unlikely that the new, tighter, regulations would exist if the Priory Hall debacle had not happened and in such a high profile way.

2.2.3 Structural Bolt Failure and the HSA Code of Practice for Design and Installation of Anchors

This was a serious failure involving a single fatality and this outcome could be regarded as lucky because the accident could easily have resulted in additional fatalities.

This failure is not included in Chapter 3 of this thesis because two engineers have given different accounts of the events. (Confidential Sources, 2015). A criminal trial (see later) followed the accident and a transcript is believed to exist. Efforts to find it have been unsuccessful.

An office building was under construction. The stairs cores were insitu concrete with precast stairs and half landings. The precast elements were supported on shelf angles bolted to the insitu walls.

The following facts are not in dispute:

- While the precast stairs were being installed, the bolts holding the shelf angles failed resulting in a collapse of the precast sections.
- The bolts used were not in accordance with the bolts specified by the design engineer.
- The bolt installer had both “long bolts” and “short bolts” in his pocket and would use whichever he pulled from his pocket.
- The bolt installer did not use a torque wrench as required by the instructions issued by the bolt manufacturer. In fact, he didn’t know what a torque wrench was.
- A warning “bang” was heard and it was noted that there was a problem with the bolts at one level. The stair core was evacuated for a time but then access was permitted without the problem being properly investigated.

What is not clear is:

- Were the bolts as specified by the designer adequate?
- What is the weight distribution between the bolts at a half landing? Due to the layout of the bolts, stairs flight and half landing, the bolts will not take equal loads. Clearly bolts closer to the stair flights will be more heavily loaded than bolts remote from the stair flights.
- When the stairs are being lowered into place, it is inevitable that the unit bears on one area of the shelf angle before other areas share the weight. What is the designer’s responsibility in this case?

- The designer is responsible for the structural stability in the final, as constructed, condition. The contractor is responsible for stability during construction. In this case, should the contractor have added additional bolts?
- Should temporary propping have been used – propping was used elsewhere on the site?
- Was the bolt specification unambiguous?

The structural designers – both the project engineer as an individual and the company – faced a criminal trial. They were found not guilty. In separate legal proceedings both the main contractor and the sub-contractor responsible for the bolt installation were fined.

The collapse took place in 2002. The main contractor and sub-contractor were fined in 2007. The structural designers were found not guilty in 2009.

As part of the defence in the criminal trial of the design company and the project engineer of the design company a full sized mock-up was constructed in a testing laboratory. This mock-up was load tested. The forces acting on the bolts were analysed in considerable detail (Confidential Sources, 2015).

In 2010, the Health and Safety Authority published a 77 page document titled Code of Practice for the Design and Installation of Anchors (HSA (A), 2010). This code does not have the weight of statutory obligation but it would be unwise to ignore its provisions. As the code itself states “A failure to observe any part of this code of practice will not of itself render a person liable to civil or criminal proceedings.” The introduction of this code makes no reference to the fatal accident which provided an impetus for the publication of the code.

The code gives duties and responsibilities to the designer, anchor manufacturer and anchor installer. The summary of the duties of a designer are summarised as:

- Decides an anchor is required.
- Completes a design risk assessment to determine if it is a Safety Critical Situation.
- Gathers all necessary information to design the anchor, using form FM-01 (Appendix A) or a drawing to communicate the requirements.
- If the anchor manufacturer/supplier is assisting in the design, the designer must approve the design.
- If designing the anchor, completes form FM-02 (Appendix A).

The Code gives a form providing information regarding loads, concrete properties, anchor plate, and anchor type. This Code has resulted in a major tightening up of procedures of anchor bolt installation. It directly follows a bolt failure which caused a fatality.

2.2.4 Malahide Viaduct Failure and the Railway Accident Investigation Unit revised procedures for Inspection of Structures near Railways

The Malahide viaduct is twin track and crosses a tidal estuary. The viaduct was built in 1965 and replaces an earlier lattice girder construction. The precast post-tensioned concrete beams sat on new precast concrete bed stones which in turn sat on existing stone piers. The tidal range and configuration of the estuary were such that water passing under the viaduct and between the piers did so with considerable speed (Railway Accident Investigation Unit, 2010).

Grouting work to the piers below water level was carried out in the late 1960s and early 1970s and the viaduct performed without incident until 2009 (Railway Accident Investigation Unit, 2010).

A member of the public, who was a regular canoeist in the estuary, noted that some of the “stones” at the base of one of the piers had been washed away. He reported his observations to the railway authorities.

The following day, an engineer went out to inspect the viaduct. This engineer had no previous knowledge of the structure. Figure 2.2.4a below shows one of the photos he took (Railway Accident Investigation Unit, 2010).

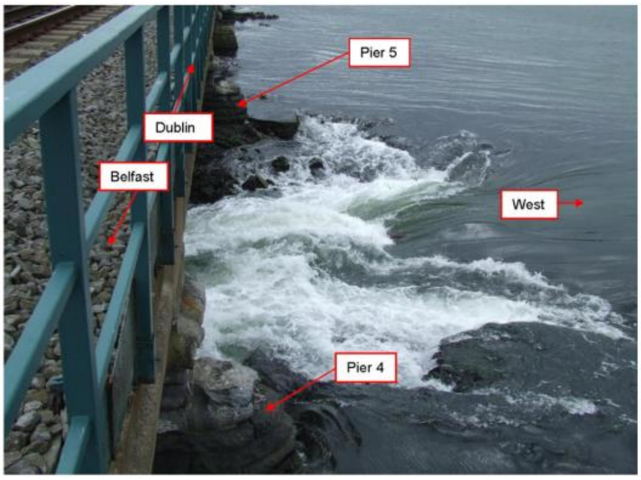


Fig 2.2.4.a Photograph taken of piers 4 and 5 taken 3 days before the collapse. (Railway Accident Investigation Unit, 2010)

Although this engineer noted some cracking in the stonework of the piers, he did not consider it to be a significant structural problem.

Early on the day of the collapse, a passenger on a train travelling over the viaduct noticed what he described as a whirlpool on one side of the viaduct. Returning over the viaduct later in the day and only 30 minutes before the collapse he again saw this whirlpool and photographed it. This photograph is reproduced in Figure 2.2.4.b.

Fig 2.2.4.b Photograph taken by a passenger only 30 minutes before the collapse. (Railway Accident Investigation Unit, 2010)

In the early evening of the day of the collapse, a passenger train passed over the viaduct without incident. This train was described as being packed with hundreds of passengers (Railway Accident Investigation Unit, 2010).

Only two minutes later, a train passed over the viaduct in the other direction. The driver noticed the viaduct beginning to collapse. The driver put his train into a “coast setting” and managed to reach the far side of the viaduct where he radioed the signalman who stopped all rail traffic. The collapsed section of the viaduct is shown on the two Figures below 2.2.4.c and d.

Fig 2.2.4.c The Viaduct after collapse (Institution of Structural Engineers 2012)

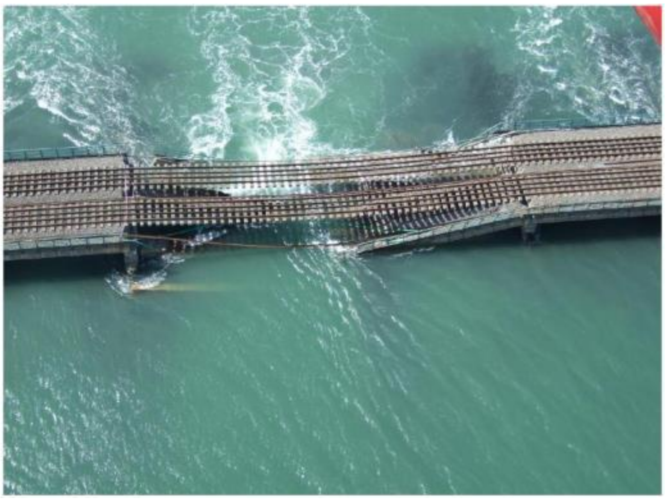


Fig 2.2.4.d Aerial view of the collapsed viaduct (Railway Accident Investigation Unit, 2010)

If the timing of the 2 trains had been slightly different and both trains had been on the viaduct at the same time, then the possible outcome could have been truly catastrophic.

The official Report (Railway Accident Investigation Unit, 2010) into the accident gave the immediate cause of the incident as undermining of the grout apron constructed in 1967 to 1972. This scour effect was not picked up in routine inspections.

The Report made fifteen recommendations including:

- track inspectors should not be required to carry out annual checks for scour. This work should be carried out by properly trained inspectors.
- the Railway Safety Commission (RSC) should only indicate that recommendations have been carried out when there is proper evidence to so do. Surprisingly, the RSC had marked as closed a requirement that they were to develop a flood and scour management system whereas they had not in fact done so!

2.2.5 Construction Site Fatal Accidents and the Safety, Health and Welfare (Construction) Regulations 2006

The safety record of the Construction Industry is not good – refer to Table 2.2.5.a and b.

Economic sector		2008	2009	2010	2011	2012	2013	2014	2015	2016
A-Agriculture,forestry and fishing	Crop and animal production,hunting and related service activities	20	10	22	22	20	16	30	18	1
	Forestry and logging	1	1	3	0	1	0	0	1	0
	Fishing and aquaculture	1	2	4	5	7	5	1	5	0
B-Mining and quarrying		1	2	0	1	1	2	0	2	0
C-Manufacturing		6	1	2	2	0	1	3	3	0
D-Electricity; gas, steam and air conditioning supply		0	0	0	0	0	1	1	0	0
E-Water supply, sewerage, waste management and remediation activities		2	0	2	3	4	1	0	3	0
F-Construction		15	10	6	6	8	11	8	11	0
G-Wholesale and retail trade; repair of motor vehicles and personal goods		3	2	4	2	3	3	4	2	0
H-Transportation and storage		3	6	3	7	1	4	4	4	0
I-Accommodation and food service activities		0	1	0	1	0	0	0	0	0
J-Information and communication		0	0	0	0	0	0	0	0	0
K-Financial and insurance activities		0	0	0	0	0	0	0	0	0
L-Real estate activities		0	0	0	0	0	0	0	0	0
M-Professional, scientific and technical activities		1	1	0	2	1	1	1	0	0
N-Administrative and support service activities		0	1	0	0	1	0	2	0	1
O-Public administration and defence; compulsory social security		1	2	0	1	0	0	0	4	0
P-Education		0	2	0	0	0	1	0	0	0
Q-Human health and social work activities		0	1	1	1	1	0	0	1	0
R-Arts, entertainment and recreation		1	0	1	1	0	0	1	1	0
S-Other service activities		2	1	0	0	0	1	0	1	0
Total		57	43	48	54	48	47	55	56	2

Table 2.2.5.a Fatalities per Industry 2008 – 2016 (HSA (B), 2016)

(note the statistics for 2016 relate to 15 February 2016)

The average over the period 2008-2015 is around ten construction deaths per annum.

Economic Sector	2002	2003	2004	2005	2006	2007
A - Agriculture, hunting and forestry	14	20	13	18	18	11
B - Fishing	3	0	4	2	2	12
C - Mining and quarrying	3	1	0	6	2	2
D - Manufacturing	7	7	2	7	4	4
E - Electricity / gas / water	1	2	0	0	0	0
F - Construction	21	20	16	23	13	18
G - Wholesale/retail trade; repair of goods	1	4	4	8	3	1
H - Hotels and restaurants	0	0	0	0	0	0
I - Transport, storage and communication	7	9	6	5	4	9
J - Financial intermediation	0	0	1	0	0	0
K - Real estate, renting, business	0	0	0	1	2	2
L - Public Admin / Defence	3	1	0	2	1	4
M - Education	0	0	1	0	0	0
N - Health / social work	0	0	1	0	1	0
O - Other community, social and personal services	1	4	2	2	1	4
Total	**61**	**68**	**50**	**74**	**51**	**67**

Table 2.2.5.b Fatalities per Industry 2002 – 2007 (HSA (B), 2016)

Over the preceding 6 year period the statistics were even worse, with a death rate approaching 20 per annum. The HSA changed their methodology in collecting statistics which is the reason the statistics are presented in 2 different formats.

A major initiative came into being with the passing of the Safety, Health and Welfare at Work (Construction) Regulations 2006 (Government of Ireland, 2006). These new regulations created new legal responsibilities for individuals involved in the construction process. They also created two new statutory roles:

- PSDP – Project Supervisor Design Phase
- PSCS – Project Supervisor Construction Stage

For the structural engineer involved in design, his new duties can be summarised as shown in Fig 2.2.5.a.

Designers must:

- Identify any hazards that your design may present during construction and subsequent maintenance
- Where possible, eliminate the hazards or reduce the risk e.g. can roof-mounted equipment be placed at ground level or can guard-rails be provided to protect workers from falling?
- Communicate necessary control measures, design assumptions or remaining risks to the PSDP so they can be dealt with in the Safety and Health Plan;
- Co-operate with other designers and the PSDP or PSCS;
- Take account of any existing safety and health plan or safety file;
- Comply with directions issued by the PSDP or PSCS;
- Where no PSDP has been appointed, inform the client that a PSDP must be appointed;
- The Safety Health and Welfare at Work Act 2005 requires designers to ensure that the project is capable of being constructed to be safe, can be maintained safely and complies with all relevant health and safety legislation.

Fig 2.2.5.a Statutory Requirements for Designers (Government of Ireland, 2006)

It is interesting to note that the fatal accident rate after 2007 is roughly half that for the preceding period. It is not known if the introduction of the new regulations in 2006 is responsible for the reduced accident statistics, but the author noted an increased awareness in safety in the design offices.

In Britain (HSE, 2015) it also appears that construction sites are becoming safer. There were 35 fatal injuries to construction workers in 2014/ 2015. This is around 20% lower than the 5 year average of 43 fatalities for the period 2010/11 to 2014/15. The same report states that almost half the fatalities were caused by falling from a height.

The approach taken by different design offices regarding how to comply with the regulations varies. One large design office has a standard form giving a prompt list to designers which, although it does not claim to address every possible hazard on site, does give a long list of risks to be addressed. The risks identified for a particular project are then fed into a document called the Design Risk Assessment and Risk Register. The designer must then see if is possible to revise his design to see if it is possible to eliminate or reduce the risk. This prompt list is shown in Fig 2.2.5.b below.

PM GROUP

Standard Form

Design Stage Safety Prompt List

(Source: 100.FC.11, Design Risk Assessment)

Directions

1. This Prompt List is used to identify items which are to be recorded on the Design Risk Assessment / Risk Register (refer to form 100.FR.04).
2. This list is not a comprehensive list of all possible design safety issues – additional items may also be identified.

Contents

Section		Section	
1. "Particular Risks" or Schedule 1 Hazards	(p 1 – 2)	5. Construction Issues	(p 7 – 10)
2. Design and Schedule Issues	(p 3 – 5)	6. Commissioning, Operation & Maintenance	(p 10 – 12)
3. Demolition / Enabling Works	(p 5)	7. Other Issues / General Comments	(p 12)
4. Temporary Works	(p 6 – 7)		

SECTION 1 – "Particular Risks" or Schedule 1 Hazards (Ref Construction Regulations 2013)

Only hazards which are of an exceptional or unusual nature are to be included in this section.

1.1 Work which puts persons at risk of falling from a height or burial under earth falls or engulfment in swampland that is aggravated by the nature of the work, process or environment

Consider:

- *Deep trenches and manholes*
- *Stability of adjacent structures*
- *Loose rock or debris, landslides*
- *Ground conditions – swamp, reclaimed land, karst, peat bogs*
- *Access for scaffolding and MEWPs*
- *Absence of edge protection*

Fig 2.2.5.b PM Group's Design Stage Safety Prompt List (PM Group, 2015)

2.3 Noteworthy International Cases and Development of Codes

2.3.1 General

Codes, procedures and statutory requirements in a wide range of industries most commonly evolve from accidents, some involving major loss of life. This is true in the building industry where the failures as outlined in this section have resulted in the creation and modification of design codes covering design and or inspection on site.

2.3.2 Tay Bridge

Construction commenced in 1871 on a major bridge structure in Scotland over the River Tay. The structure was a little over 3,000m in length with lattice girders spanning between brick piers.

The structure was single track with the train rails being at the level of the top of the trusses for most of the length of the bridge. There were however thirteen so-called "high girders" where the tracks ran at the bottom of truss level - to facilitate tall sailing ships passing beneath (Lewis 2007).

On the 28th December 1879 there was a violent storm in the area - said to be Beaufort Scale 10 to 11.

An express train travelling north to Aberdeen passed over the bridge and the bridge collapsed causing the train to fall into the water below. It was believed that there were around 74 or 75 people on the train. There were no survivors (HMSO, 1881). A Court of Inquiry was set up involving some of the leading bridge engineers of the day.

Fig 2.3.2.a Tay Bridge after collapse in 1879 (North British Railway Study Group 2016)

There were many theories and possibly many different factors connected with this catastrophe. These included:

- wind speed
- oscillation of the bridge
- speed of the train
- casting of the steel (particularly at lugs and variation of wall thickness)
- inspection of the bridge during instruction
- strength of the diagonal bracing
- bolt holes formed in a conical shape rather than cylindrical

One of the principal findings of the Court of Inquiry was that the designer of the bridge, Sir Thomas Bouch, made no special allowance for wind. This is a remarkable finding for modern engineers – wind loading is now a significant design consideration. (HMSO, 1880)

One of the results of the Court of Inquiry was the setting up of The Wind Pressure (Railway Structures) Commission. The speed of report writing and publication was impressive. The Court of Inquiry published its report in June 1880 – only 6 months after the disaster. The Wind Pressure (Railway Structures) Commission published its Report in May 1881 – less than a year after its establishment was recommended by the Court of Inquiry (HMSO 1881).

This Commission illustrates how consideration of wind loads was only in its infancy. The Commission found that some of the wind monitoring stations recorded only the "run in miles of the wind during each hour". This alone is a strange finding because this method of measurement gives no indication of the maximum gust speed. It finds the average speed over a period of an hour.

The Commission concluded, "... that for railway bridges and viaducts a maximum wind pressure of 56 lbs per square foot should be assumed for the purpose of calculation". The Commission goes on to recommend a Factor of Safety of 4 when calculating stress in the members but reduces this factor of safety to 2 for overturning where the restoring forces are gravity alone.

It is interesting to note that the cause of the Tay Bridge collapse is still a matter for academic study involving advanced computer analysis and forensic study. (Dow, 2007).

In this case the death of 75 people was the catalyst for the study of wind loadings on railway structures and the like.

2.3.3 Tacoma Narrows Bridge

When it was completed and opened to the public in July 1940, the Tacoma Narrows Bridge had the third longest main span in the world. This suspension bridge is in Washington State on the west coast of America. Previous bridge designs with these very long spans had an open lattice type truss beneath the road deck level. The Tacoma Narrows Bridge was the first one of its type to have deep plate girders under the deck level (Washington State Department of Transportation, 2005). The cross section is shown in Fig 2.3.3.a.

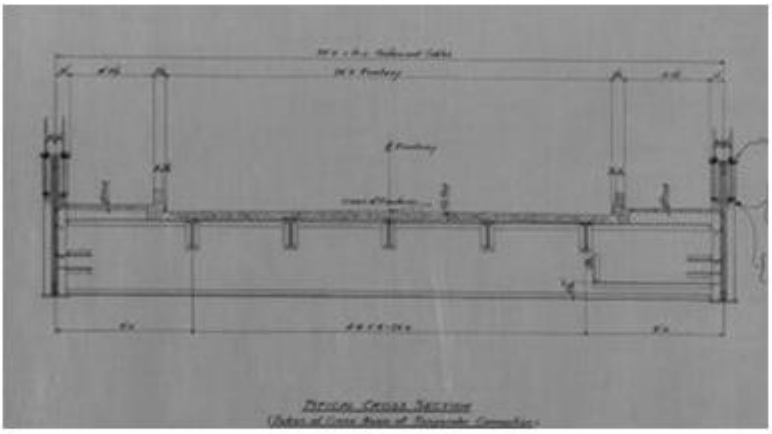


Fig 2.3.3.a Typical section through bridge deck (The Tacoma Narrows Bridge Blueprints, 2016) (note the deep plate girder on either side)

During construction problems were observed with winds causing significant oscillation of the bridge deck. Even as construction work was still underway, remedial work was recommended. This remedial work included the addition of cables and hydraulic buffers. Wind tunnel tests were undertaken and the results of these tests led to recommendations to form holes in the main girders. These recommendations were made only 5 days before the bridge collapsed.

Only 4 months after opening, and in winds described as being 64 kph (Beaufort Scale 8). The wind produced oscillation in the bridge and a spectacular collapse resulted. This was famously recorded in a black and white cine-film.

A report was urgently commissioned and published only 4 months after the bridge collapsed (Carmody, 1941). The principal findings of this Carmody Report were:

- The bridge deck had excessive flexibility
- The solid plate girder and deck acted like an aerofoil
- Aerodynamic forces were little understood and suspension bridge designs should be tested using a wind tunnel

Significantly the design of the bridge fully complied with the requirements of accepted structural theory at the time. Wind loading was not considered to be particularly important.

Modern accepted theory for the collapse of the bridge is the phenomenon called forced resonance - the wind providing an external periodic frequency which matched the natural structural frequency of the bridge. The vertical movement caused by vortex shedding then puts the deck into torsional flutter which causes large movements (Washington State Department of Transportation, 2005).

Although there were no fatalities or injuries in the collapse of the Tacoma Narrows Bridge, the financial consequences were severe and this failure precipitated the study of aerodynamic effects on structures.

Deer Island Bridge was a suspension bridge with a central span of 329 m. It was opened to the public in 1939 and had a cross sectional profile not dissimilar to that of the Tacoma Narrows

Bridge. Fairings were added to the bridge to provide a more acceptable aerodynamic cross section (Federal Highway Administration, 1996). Refer to Fig 2.3.3.b below.

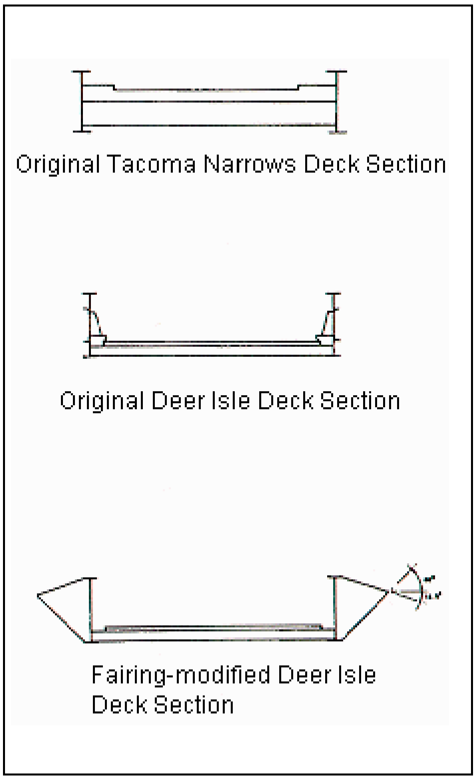


Fig 2.3.3.b Aerodynamic modifications to Deer Island Bridge (Federal Highway Administration, 1996)

Instability of bridges in wind was a well-known phenomenon. Over a hundred years before the construction of the Tacoma Narrows Bridge, the Menai Suspension Bridge was constructed in north Wales providing a transport link to Ireland. With a main span of 176 m, this was the longest span of any bridge at that time. The roadway was only 7.3 m wide and, without stiffening trusses, proved highly unstable in the wind (ASCE, 2016).

Similarly, the Millennium Bridge in London, which was completed in 2000, famously suffered from swaying from side to side when the footfall was at the same natural frequency as the bridge – so called “synchronous lateral excitation by pedestrians” (Strogatz, 2005). The source of the oscillations was different in the two cases – wind in the Tacoma Narrows case and footfall in the Millennium Bridge case, both bridges were loaded with forces close to the natural frequency of the bridge.

2.3.4 Ronan Point

Ronan Point was a 22 storey residential tower block in London. It was constructed as a so-called System Building with LPS - Large Panel System - precast concrete panels assembled on site. The building was handed over in March 1968 and only 2 months later a gas explosion caused the demolition of the south east corner of the building. The explosion took place on the 18th floor, i.e. near the top of the building (HMSO, 1968).

The explosion took place at around 05.45 and four people were killed. If the explosion had taken place a little later when a number of the kitchens in the corner of the building were occupied, then the death toll would probably have been higher. A view of the failed structure is shown in Fig 2.3.4.a below.

Fig 2.3.4.a Ronan Point (Historic England, 2016)

It is interesting to note that gas explosions were, at that time, far from uncommon. Chapman (Chapman, 2000) gives an average figure of gas explosions in the UK causing structural damage in the ten years prior to Ronan Point at 406 per annum.

Such system building was a common form of construction in this period. Frequently, joints between adjacent panels were formed using looped steel reinforcement and grout. However, Ronan Point did not have this jointing arrangement in key areas. The report (HMSO, 1968) was published just 5 months after the collapse. The report found that the gas instillation to the apartment in question was carried out by a plumber who was not registered. The report noted that the air pressures generated by the explosion were not exceptional. The key detail for an understanding of this failure was identified in the report as Detail D – refer to Fig 2.3.4.b below.

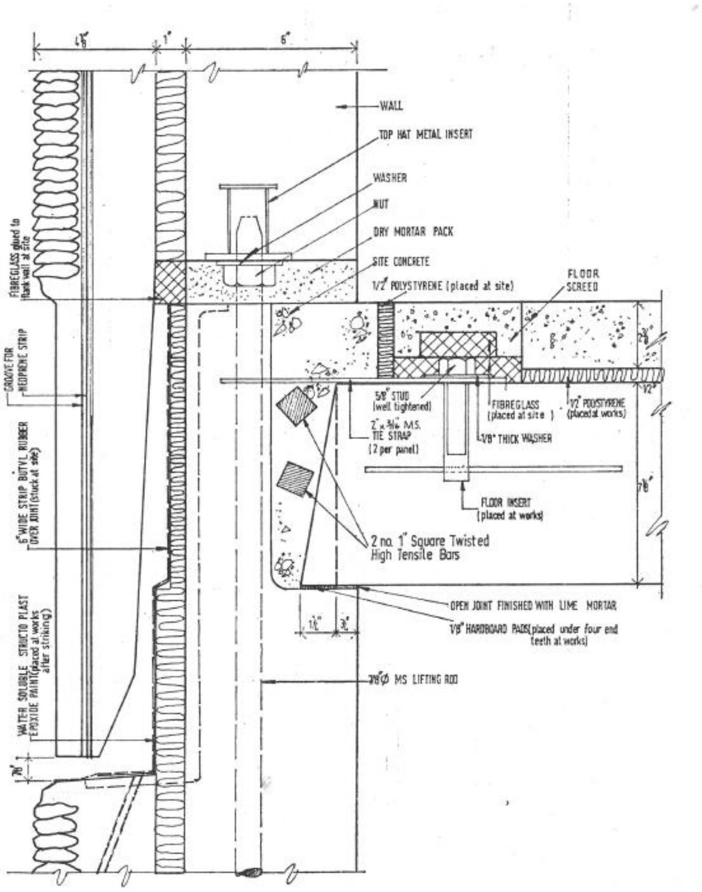


Fig 2.3.4.b Detail D showing the joint between the floor slab and the flank wall (HMSO, 1968).

It should be noted that there is only a 2 inch (50 mm) bearing of the concrete floor slab onto the load-bearing concrete element on which it sits. Although not clear in Detail D, the tie strap has an oval hole which has the advantage of easier construction but a potential flaw. The oval hole allows movement of the supporting wall before the restraining effect of the tie comes into play. The progressive collapse which completely demolished one corner of the building was made possible by what is now recognised as an inadequate tying arrangement.

It is interesting to note that calculations carried out for the report suggest that, had the explosion taken place at a lower level, the friction due to the added vertical load would have been sufficient for the joint to have been adequate.

The report notes that:

- Extensive voids and rubbish was to be found in the joints, which should have been fully grouted
- The erectors were paid on the basis of speed of erection
- The resident engineer was young and inexperienced
- “The possibility of progressive collapse is not covered in either the Building Regulations or Codes of Practice”
- “…it is significant that we have not been referred to any English Publication which has drawn attention to the need to think of tall System Buildings as civil engineering

structures requiring alternate paths to support the load in the event of the failure of a load-bearing member. It appears to us that there has been a blind spot among many of those concerned with this type of construction and it would be wrong to place the blame for the failure to appreciate the risk of progressive collapse upon the shoulders of this building alone."

The building was repaired but other structural faults became apparent later and the building was demolished only eighteen years after it was constructed.

The Ronan Point disaster generated an entirely new concept in structural engineering design. The Practical Guide to Structural Robustness and Disproportionate Collapse in Buildings (IStructE, 2010) has the following definitions and explanations:

> Disproportionate Collapse – A collapse after an event, which is greater than expected given the magnitude of the initiating event. The level of collapse expected for certain events may be given in regulations, but more often is related to public and professional perception. Even the most robust structure may suffer complete collapse if the event is severe enough, and this would not be considered disproportionate. IS EN 1991-1-7 recognises that total collapse may be sometimes acceptable:
>
> Progressive Collapse – The sequential spread of local damage from an initiating event, from element to element, resulting in the collapse of a number of elements. Whilst undesirable, a progressive collapse may not be disproportionate. Hence the term "progressive collapse" is not necessarily equivalent to "disproportionate collapse".

The current Irish Building Regulations (Government of Ireland, 2012) have only four clauses and one of them is devoted to Disproportionate Collapse. It is reproduced below as Fig 2.3.4.c.

Disproportionate Collapse.	A3	(1)	A building shall be designed and constructed, with due regard to the theory and practice of structural engineering, so as to ensure that in the event of an accident the structure will not be damaged to an extent disproportionate to the cause of the damage.
		(2)	For the purposes of sub-paragraph (1), where a building is rendered structurally discontinuous by a vertical joint, the building on each side of the joint may be treated as a separate building whether or not such joint passes through the substructure.

Fig 2.3.4.c The disproportionate collapse provisions of the Irish Building Regulations. (Government of Ireland, 2012).

2.3.5 High Alumina Cement

High Alumina Cement (HAC) was popular for precast and in particular precast, pre-stressed concrete in the UK in the 1950s to early 1970s. The rapid strength gain of this cement allowed quick turnaround of the precast moulds which was attractive to precast manufacturers (Report of the Working Party on HAC, 1976).

The properties of Ordinary Portland Cement (OPC) are based on calcium silicates whereas HAC is based on calcium aluminates.

A number of structural failures took place in the early 1970s. The two most prominent were

- Sir John Cass School – collapse of the swimming pool roof
- Camden School for Girls – collapse of the assembly hall

Very fortunately there was no loss of life or indeed injuries reported but clearly the potential for loss of life was severe.

These collapses triggered a major research effort which focused not only on what had gone wrong, but also what to do with existing structures which had or possibly had HAC. Very quickly HAC became an active topic to be discussed within the construction industry. The chemical problem was the conversion of HAC and the adverse factors included (Building Research Establishment, 2002):

- Free water cement ratio in excess of 0.4
- Higher air temperature at casting
- Aggregate containing minerals such as feldspar and mica.

Precast units using HAC had been permitted in the codes and standards in force at the time. Following these collapses and subsequent reports, the codes have been modified. The codes are now quite explicit with BS 5328-1:1997 stating “High Alumina Cement conforming to BS 915 should not normally be permitted for structural concrete (British Standards Institution, 1997).

2.3.6 Hyatt Regency Hotel

A hotel in Kansas City, Missouri had a multi-storey atrium between bedroom blocks. Elevated walkways connected the bedroom blocks and were hung down from the roof structure with steel rods. Crossbeams supported the walkway deck. These crossbeams were channels welded toe to toe. One of these walkways had the walkway at second floor level directly underneath and supported by the walkway at fourth floor level. The design drawings showed long rods hung from the roof structure passing the fourth floor and supporting the second floor in a single length of rod. The contractor proposed altering the support arrangement because he feared that the threads on the bar would be damaged when raising the fourth floor level.

The revised detail was submitted to the designer and the detail was approved (Brady, 2015).

The revised support detail can be seen in the Figure 2.3.6.a.

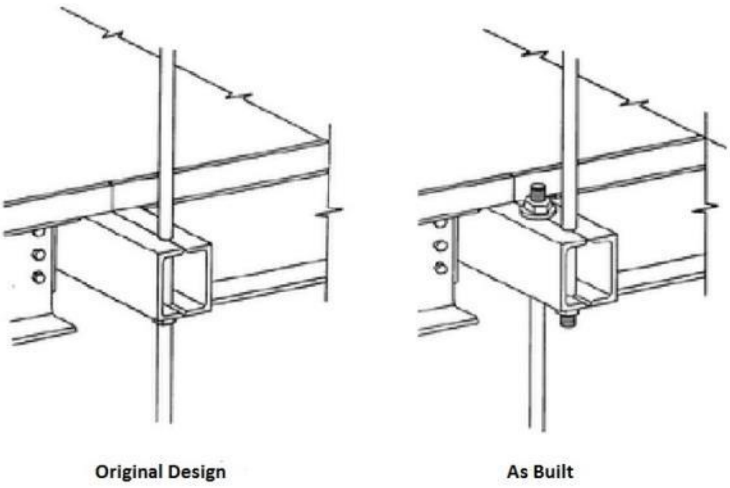


Fig 2.3.6.a The original design and the “as built” detail. (WordPress.com, 2011)

On the evening of 17 July 1981, a dance competition was being held on the ground floor of the atrium. Around 40 people were on the second floor walkway, with an additional 20 to 60 people on the 4th floor walkway. The catastrophic collapse killed 114 and injured a further 216. This was the deadliest structural collapse in US history until the collapse of the World Trade Centre in 2001. A detailed examination (Petroski, 1985) stated that the cause of the collapse was the change in the support detail, but the poor welding of the transverse channels was a contributory factor.

The designer and his project engineer were convicted of gross negligence, misconduct and unprofessional conduct and had their professional licences revoked in a number of States. The designer, J Gillum, has spoken at a number of conferences about the collapse, his role in it and the human factors that led to the failure. Fig 2.3.6.6 below shows a failed connection and Fig 2.3.6.c shows the aftermath of the collapse.

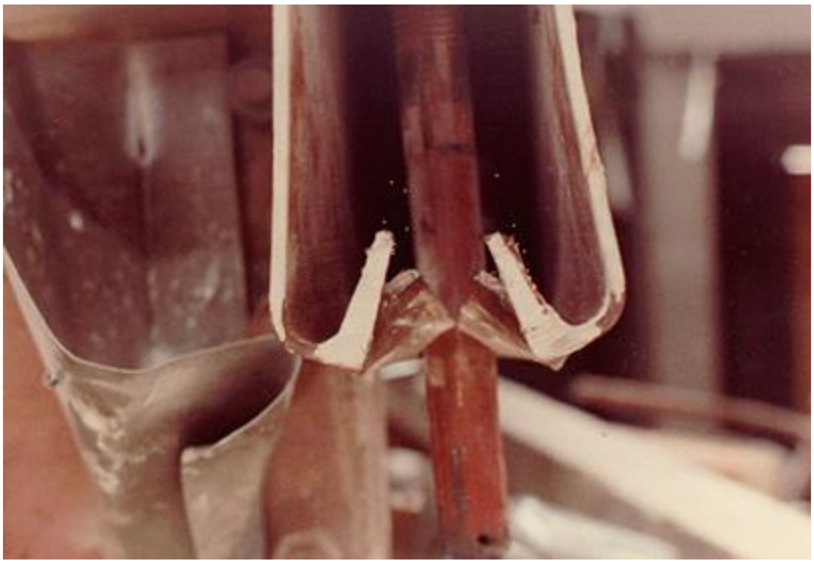


Fig 2.3.6.b Failed connection at transverse beam (Kansas City Star, 1981)

(note the weld connecting the toes of the channels has failed)

Fig 2.3.6.c Rescue efforts (Kansas City Star, 1981)

This failure did not produce a revision to the codes and standards but made the industry aware of the need to carefully check design changes suggested by the contractor.

2.3.7 Versailles Wedding Hall

A very popular form of construction in Israel in the 1980s was a proprietary system called Pal – Kal. Hundreds of thousands of square metres of this floor system are known to have been built.

The floor is cheap to construct. Essentially the method of construction is that of a flat slab but with an extensive number of voids in the slab. The bottom layer is only 40 mm thick with a layer of steel mesh in it. The upper layer is also formed of a layer of concrete only 40 mm thick with a layer of mesh in it. The voids are formed with steel sections of only 1 mm thickness.

In 1996, following a series of non - fatal failures, the Standards Institute of Israel set up a committee to investigate Pal – Kal. The ensuing report (Zailer Committee Report, 2002) concluded that the system failed to comply with Israel's concrete code.

The Versailles Wedding Hall was constructed using the Pal – Kal system. During a wedding reception in 2001 a catastrophic failure of the floor took place. 23 deaths were recorded. There were a number of factors in this collapse,

- The Pal – Kal system was known to be suspect
- The floor which collapsed was originally a roof with a low live load
- An additional storey was built on
- A few weeks before the collapse some partition walls below the floor in question, were removed. Although these were regarded as not load-bearing, they were in fact transferring load to the lower floor.

- After the partition removal, the floor in question was seen to have a noticeable sag. Strangely, a heavy concrete screed was added to the floor to give a flat upper surface.
- The spans of the floor were long. Such spans are sensitive to dynamic loads such as dancing.

Fig 2.7.3.a Versailles Wedding Hall (Zailer Committee Report, 2002)
(note the long spans and punching shear at column)

The Zailer Report investigated the circumstances of the failure and made a number of recommendations. These included that there should be a comprehensive and user friendly building code and that the number of government departments having an interest in building work should have their roles and responsibilities concisely defined.

In 2011, some ten years after the collapse, it is reported (Haaretz, 2011) that hundreds of buildings using the Kal – Pal flooring have yet to be examined and tested. Haaretz goes on to say that faulty identification and inaccessible documentation means that apartment buyers are not aware of the danger. Perhaps it will require another collapse to generate the political will to have all the buildings assessed and checked.

2.3.8 Piling Platform

Piled foundations are required in areas of ground with poor structural properties. Piling rigs can weigh up to 200 tons and have a high centre of gravity. Although piling rigs are track mounted to spread the weight of the rig onto the ground, there are numerous cases of piling rig collapses due to inadequate working platforms. The consequences of poor ground can be seen in Fig 2.3.8.a.

Fig 2.3.8.a Piling rig overturning (Construction Enquirer, 2015).

One potentially extremely serious incident occurred on a site immediately adjacent to an electrified rail line in the UK (refer to Fig 2.3.8.b). The ground conditions were very poor and a working platform of stone and a geotextile membrane had been installed. To remove an obstruction, a trench had been dug and the geotextile had been damaged. The trench was then poorly backfilled. When the rig crossed the trench, ground settlement took place and the rig collapsed across the railway line. The 25 kV overhead electric cables were brought down. A passenger train had travelled along the tracks just 2 minutes before the collapse. The subsequent inquiry (HSE, 2004) was told that the site managers were not aware of the critical importance of the backfilling.

Fig 2.3.8.b A continuous flight auger piling rig collapses across a railway line. (HSB Engineering Insurance, 2014).

The trade group representing the piling contractors, The Federation of Piling Specialists now have what is called a Working Platform Certificate. The main thrust of the certificate is to give "the principal contractor total responsibility for the provision of an adequate working platform. This responsibility includes regular inspections, modifications and repairs."

2.4 Other Industries

2.4.1 General

Major accidents have taken place in industries other than construction involving multiple loss of life. Outlined in this section are examples of accidents in other industries together with examples of how these industries responded. The response of other industries is important in that a comparison can be made to the response of the construction industry.

When considering Irish construction failures, international structural failures and accidents involving other industries, it is possible to conclude that codes, procedures and protocols usually only develop following accidents and their subsequent reports.

2.4.2 Irish Railways

In Section 2.2.4 of this thesis the collapse of the Malahide Viaduct is discussed. There have been a number of fatal accidents in Irish railway history which do not involve failures of structural engineering. These include in recent years:

- Gormanstown, 1974. Two fatalities when a runaway empty train collided with a second empty train which in turn struck a passenger train (Report of Inquiry, 1976).
- Buttevant, 1980. Eighteen fatalities when an express train was diverted into sidings when points were not interlocked (Report of the Investigation into the Accident at Buttevant, Co Cork on 1 August 1980, 1981).
- Cherryville, 1983. Seven fatalities when a train collided with the rear of a broken down train (Report of the Investigation into the Accident on the CIE Railway near Cherryville Junction, Co Kildare on 21 August 1983, 1984).

The Railway Safety Commission (RSC) has three main functions as set out in the 2005 Railway Safety Act (RSC, 2016). They could be summarised as:

- Conformity Assessment - Assessing Safety Management Systems (SMS) to ensure that they conform to all requirements
- Compliance Supervision and Enforcement - Auditing compliance with the procedures and standards prescribed in each approved SMS
- European and Legislative Harmonisation - Supporting the harmonisation of legislation with European Directives and Regulations

The Railway Investigation Accident Unit (RIAU) is stated to be an independent investigation unit within the RSC. One of the roles of the RSC is to track and close-out recommendations made by the RAIU. As discussed in Section 2.2.4 of this thesis relating to the Malahide Viaduct, the RSC was seen to have closed a recommendation relating to flood and scour when there was no evidence that they should have done so. Significantly, the report on the Malahide collapse

(Railway Accident Investigation Unit, 2010) showed that the RSC failed to comply with its own rules.

The Malahide Report refers to “loss of corporate memory”. This refers to the fact that when individuals retire or move to other jobs, their knowledge is not passed on to in-coming staff. It would appear that this is not the first time that loss of “corporate memory” has occurred in the railway industry.

2.4.3 Air Industry

In Ireland the Air Accident Investigating Unit (AAIU) is responsible for the investigation of air accidents, serious incidents and incidents that occurs within Ireland. The AAIU carries out investigations in accordance with the regulations laid down by the International Civil Aviation Organisation (AAIU, 2016).

An investigation carried out by the AAIU related to what is described as a serious incident at Dublin Airport in March 2015. (AAIU, 2015). The incident occurred during the hours of darkness.

The pilot of an outgoing aircraft was taxiing to the end of the runway. Air Traffic control (ATC) asked if the plane was ready for “an immediate rolling departure”. The reply from the plane was negative but the radio transmission was not clear and was understood to be affirmative. The taxiing aircraft was some distance from the end of the runway when ATC gave the instruction to the plane to line up and take off.

An arriving aircraft was cleared to land on the same runway as the departing aircraft. The two aircraft came within 807 m of each other on the main runway (refer to Fig 2.4.3.a).

The subsequent report (AAIU, 2015) does not use the phrases “Near miss” but uses the somewhat less alarming expression, “loss of separation”.

Fig 2.4.3.a Plan of runway (AAIU, 2015)

(Note the position of the two red stars representing the aircraft in their closest proximity.)

Following this incident, the Irish Aviation Authority installed additional safety equipment in the control tower. The TTT device – Time to Touchdown displays the time to touchdown of the next in-coming aircraft. New procedures for the controllers were implemented (Irish Times, 2015).

Again it can be seen that new safer practices come into force after a near miss incident.

2.4.4 Petroleum Industry

The MV Betelgeuse was a large crude oil carrier. It arrived at the Whiddy Island Terminal, Bantry, Co. Kerry in January 1979 carrying around 115,000 tonnes of crude oil. Early in the morning of January 8th, a massive fire accompanied by a number of explosions killed all the crew and a number of others – 50 in all – refer to Fig 2.4.4.a below.

The subsequent report (Tribunal of Inquiry Report, 1980) states:

> *"The initiating event of the disaster was the buckling of the ship's structure at or about deck – level and in way of the permanent ballast tanks / of the ship's manifold. This was immediately followed by explosions in the permanent ballast tanks and the breaking of the ship's back. These events were produced by the conjunction of 2 separate factors: a seriously weakened hull due to inadequate maintenance, and an excessive stress due to incorrect ballasting on the night of the disaster."*

The ship did not have a piece of equipment – a Loadicator. This piece of equipment was to be seen on virtually all large tankers at this time. The Loadicator monitors the loading and unloading of the vessel if the bending moments and shear forces in the hull of the vessel are critical. The Loadicator costs only a few thousand pounds and the report notes that no adequate explanation was offered as to why the Betelgeuse did not have one.

The report also notes that personnel from the oil company actively conspired to mislead the Tribunal. This suppression of the truth involved attempts to hide the fact that a safety officer was not in the control room at certain critical times when he should have been. Also, the report states that false entries were made in record log books.

The Tribunal made no less than 45 recommendations. These included revisions to Statutory Regulations covering the operation of oil facilities. The report was critical of the number of Government Agencies who have legal responsibilities regarding supervision and inspection of oil jetties. Specific recommendations in the report cover areas including the provision of Loadicator equipment, lifeboats, wave-height recorders, anemometers, logbooks, length of shifts, loudspeakers, fire mains, etc.

As can be seen in other industries, in this case, significant changes to safety regulations follow a disaster.

Fig 2.4.4.a The Whiddy Island oil terminal disaster. (Irish Times, 2014)

CHAPTER 3 – CASE STUDIES

3 Thesis Main Body – Case Studies

3.1 Outline

This chapter gives the case studies of 32 individual structural failures or potential failures.

The cases are grouped according to the materials of construction in the location of the primary cause of the failure as shown below. Strictly speaking, it could be said the foundations are not a material of construction. They are, however, an important source of structural failure and therefore are included in this list.

A. Steel
B. Insitu Concrete
C. Pre-Cast Concrete
D. Composite
E. Masonry
F. Timber
G. Foundations

The methodology for the collection of data is given in section 1.4.5 of this thesis. The methodologies used in similar studies are outlined in sections 1.4.2, 1.4.3 and 1.4.4.

3.2 Analysis of Results by Cause

The underlying structural cause will be identified for each case study. Frequently there will be more than one cause. Percentages are assigned under the following headings:

- Designer's lack of understanding of structural principles
- Designer's lack of understanding of loads
- Designer's lack of understanding of material properties
- Designer's poor communication to site
- Contractor's failure to comply with design

In each of the case studies, a graphical representation of the root causes is presented, based on an assessment of the percentage responsibility. This is shown and discussed in Chapter 4.

3.3 Analysis of Results by Potential Severity of Failure

The potential severity of the failure for each case is assessed. This is split between death, injury and financial cost. It is to be noted that these divisions are not based on what actually happened in the particular case but on what could be reasonably expected to be the outcome. This means that if there was a potentially extremely serious potential failure, and by a lucky set of circumstances such a failure did not take place, then this would be assessed as a very serious failure. The term "potential severity of failure" is used.

The "Consequence Class" referred to in Chapter 4:

A. Fatalities ≥ 10 persons
B. Fatalities < 10 persons
C. Injuries ≥ 10 persons
D. Injuries < 10 persons
E. Financial cost ≥ €10m
F. Financial cost < €10m

This classification is somewhat arbitrary and has been chosen by the Author to give a method of ranking the potential severity of failure. The allocation of a particular case to a particular severity category is a matter of judgement. Valid arguments could readily be made for certain cases being in other categories. A working design engineer makes judgement calls regularly as part of his work.

3.4 Financial Cost of Injury and Death

In the analysis of severity of failure, no effort has been made to put a financial cost on injury and death. Different figures for death in particular are published. These are used in analyses for justifications for road schemes and similar projects. (Goodbody, 2004).

There are parallels with the infamous Ford Pinto case in the 1970s. The Pinto car had a design defect which made the fuel tank likely to catch fire in a collision. Ford carried out an analysis which compared the cost of recalling and fixing the cars with the likely legal costs associated with the additional deaths likely to occur because of the defect. (NHTSA - National Highway Traffic Safety Administration, 1978).

3.5 Case Studies

The 32 case studies are presented in the following pages:

CHAPTER 3 – CASE STUDIES

3.5.1 Case Study 1 - Arched Portal Frame

Background

An industrial type single span portal framed building was designed with a curved rafter. The span was 38 metres with a height to ridge of 10 metres. A castellated rafter was selected for aesthetic reasons and also for service runs. Refer to Fig 3.5.1.a.

Fig 3.5.1.a. Photograph taken during steel erection showing the roller shutter door ope. (Confidential Source, 2015).

A large roller shutter door was required on one side.

What Happened

When the steel frame was erected, but before the cladding and roofing was installed, a distinct sag was noted in the rafter, which lined up with the centre of the roller shutter door (refer to Fig 3.5.1.b).

Fig 3.5.1.b. Photograph showing the sag in the castellated rafter. (Confidential Source, 2015).

Causes

The structural designer designed 2 frames – one was the normal portal frame and the other was the frame whose column was interrupted by the door. The different structural arrangement was recognised in the design. The standard rafter was 1200 x 267 / 267 x 196.8 CUB and the rafter in line with the door was 1200 x 305 / 305 x 289.1 CUB. The non-standard rafter was nearly 50% heavier.

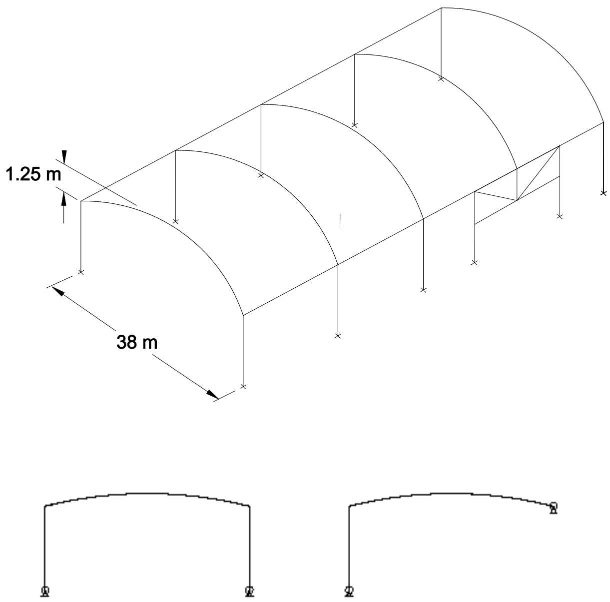


Fig 3.5.1.c. Structural frame showing standard frame and frame at door.

The 2 frames were both designed as having pinned bases as shown. The error is that the non – standard frame does not have a pinned base at the upper level. This support should have been modelled with a horizontally sliding joint with a spring.

The fix involved:

- jacking up the sagging rafter
- adding steel to the member over the door to increase its stiffness in the horizontal direction
- adding steel to form roof plan bracing
- adding diagonal steel from the rafter to the posts on either side of the door ope. This provides moment connections to the adjacent columns and thereby reduces the moment in the rafter span.

Discussion

The designer could probably consider himself as unlucky. The error is not obvious unless it was pointed out in which case, it becomes clear. The error was not picked up during the checking process which was described as a thorough check.

When the cold rolled purlins are added together with the steel roof decking a rigid flat plate is formed. This would have the effect of transferring load from the non-standard frame to adjacent frames. Again this would have the effect of reducing the moment in the rafter span of the non-standard span.

Underlying Structural Cause

The underlying cause of this case is assessed as given as the designer's lack of understanding of structural principles – 100%.

Potential Severity of Failure

Although there is a clear serviceability issue, it is hard to imagine that the error would cause a collapse. The potential severity of failure is assessed as a financial loss of less than €10 million.

3.5.2 Case Study 2 - Curved Bridge Bearing

Background

A large cable stay bridge has a significant curve in plan. This is a high profile structure and further details might identify it. The plan view and bearing details as designed are shown below – Figs 3.5.2. a, b, and c.

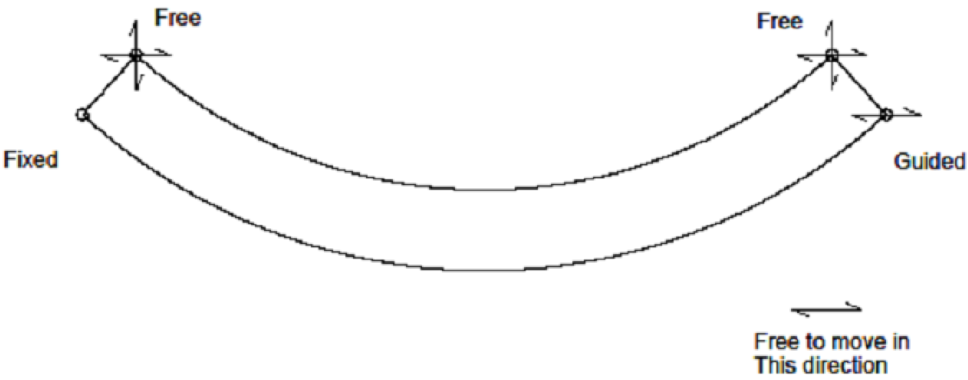


Fig 3.5.2.a Plan showing the arrangement of bearings.

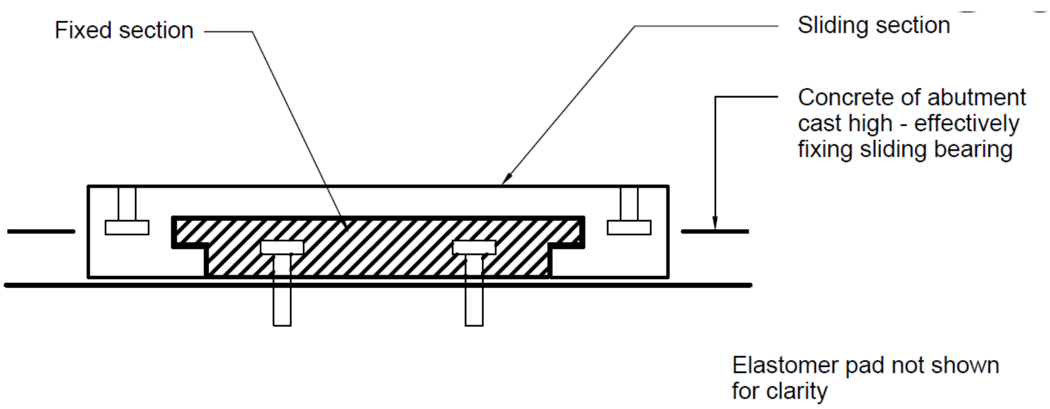
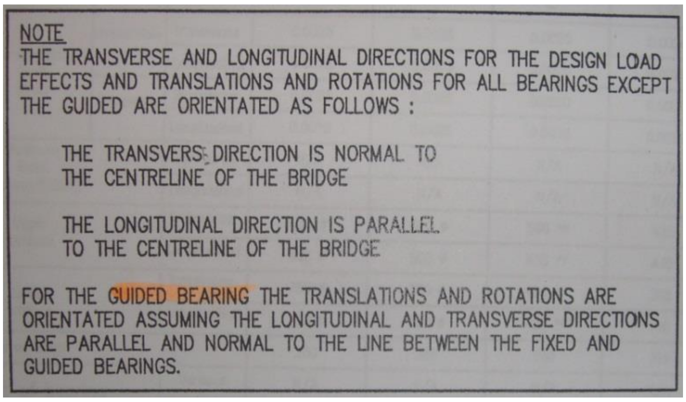


Fig 3.5.2.b. Diagrammatic section through guided bearing.

The design drawing for the bridge had the following note – Fig 3.5.2.c.



NOTE
THE TRANSVERSE AND LONGITUDINAL DIRECTIONS FOR THE DESIGN LOAD EFFECTS AND TRANSLATIONS AND ROTATIONS FOR ALL BEARINGS EXCEPT THE GUIDED ARE ORIENTATED AS FOLLOWS :

THE TRANSVERSE DIRECTION IS NORMAL TO THE CENTRELINE OF THE BRIDGE

THE LONGITUDINAL DIRECTION IS PARALLEL TO THE CENTRELINE OF THE BRIDGE

FOR THE GUIDED BEARING THE TRANSLATIONS AND ROTATIONS ARE ORIENTATED ASSUMING THE LONGITUDINAL AND TRANSVERSE DIRECTIONS ARE PARALLEL AND NORMAL TO THE LINE BETWEEN THE FIXED AND GUIDED BEARINGS.

Fig 3.5.2.c. Note on designer’s drawing (Confidential Source, 2015).

What Happened

A simple diagram could have shown the design intent of the designer regarding the direction of movement of the guided bearing. However, the information was given to the contractor in text form which was far from clear.

There was, however, a second and unrelated matter regarding the bridge. Measurements were taken of the tension in the cable stays. An analysis of the forces in the cables and the dead weight of the structure revealed a problem - the numbers failed to balance. On-site investigations revealed that the concrete of the abutment was cast higher than indicated on the design drawings. This had the effect of making the guided bearing into a fixed bearing. The bridge had a fixed bearing at both ends of the bridge. Refer to 3.5.2.e for an image of the bearing as constructed.

When the restraining force of the support which was previously modelled as a sliding bearing was taken into account, the cable calculations now made sense. There was a significant force into the abutment. There was cracking at the top of the abutment – see Fig 3.5.2.e below.

Fig 3.5.2.d. View of sliding bearing (Confidential Source, 2015).

Fig 3.5.2.e. Alternative view of sliding bearing (Confidential Source, 2015).

Causes

The designer's instructions to the contactor were unclear.

The contractor was at fault by casting the concrete too high.

Discussion

It was very fortunate that there was an error in casting the concrete high. The error in installing the guided bearing was potentially disastrous – the bridge could easily have come off its bearing.

Site supervisory staff were present at all relevant times. They failed to flag both problems – the incorrect installation of the bearing and the casting of concrete too high. Furthermore they failed to raise an issue regarding the unclear instruction regarding the bearing installation.

Underlying Structural Cause

The instructions to the site on the engineer's drawings were far from clear. Even after repeated reading of the note on the drawing, the engineer's intent is unclear. That said, is it reasonable to believe that the contractor should have asked for clarification of the note? Is it fair to allocate some of the blame to the contractor?

The underlying structural cause is assessed as the designer's poor communication 80% and the contractor's failure to get structural engineering input as 20%.

Potential Severity of Failure

A major catastrophic failure was possible in this case. The bridge could have slid off its bearings. The bridge is a major piece of infrastructure carrying thousands of people every day. It would be described as a near miss. The potential severity of failure is assessed as less than ten fatalities.

3.5.3 Case Study 3 - Silo

Background

A large silo, around 30 m high and with a diameter of 10 m had the capacity of around 3,000 tonnes of clinker. The weight of the silo and contents is carried by a series of major bolted connections at around 10 m over ground level.

The structure above the 10 m level serves to support the silo horizontally only – only nominal vertical loads are transferred here. It can be seen in the photograph below that the column below the 10 m level is a very heavy section whereas the column section above this level is significantly lighter.

What Happened

A truck was being filled and was under the silo. Without warning a sudden failure of the main bolted connections took place. The bolts failed in shear. The weight of the contents at the time of collapse is not known.

Remarkably, the silo collapsed vertically and was almost perfectly vertical when it came to rest – refer to Fig 3.5.3.a below.

The truck was completely flattened and fortunately there were no injuries.

Fig 3.5.3.a. View of silo after failure (Confidential Source, 2015).

Note the large plate girder A should be connected to the top of the heavy column section at B. Note also that the top of the silo C should be near the top of the framework at D. The aftermath of the failure can be seen in the 2 photographs below – Figs 3.5.3.b and c.

Fig 3.5.3.b. The remains of the truck under the silo (Confidential Source, 2015).

Fig 3.5.3.c. Failed bolted connection (Confidential Source, 2015).

Causes

It has not been easy to obtain information on this failure.

That stated, it has been suggested that the cause was due to the properties of the stored material. Clinker can arch when material is discharged from the bottom of the cone during normal discharging of the material. If this happens, then the sudden falling of the arched material together with all the material above collapses onto the cone.

This dynamic impact produces forces considerably higher than the static force. This theory is considered to be the most likely cause of the failure.

Discussion

A brief report to a third party has come to light relating to this failure. It states that this was a design flaw which has been "replicated world-wide". It was not possible to establish the capacity of the connections or grade of bolt. Efforts to obtain information from the owner of the silo were unsuccessful.

Underlying Structural Cause

The arching material theory will be considered as the cause of the failure for the purpose of the data analysis in this thesis. The designer should be aware of the possibility of the dynamic effects caused by the stored material.

The underlying cause of this case is assessed as the designer's lack of understanding of loads - 100%.

Potential Severity of Failure

The truck driver survived this failure but a fatal incident is clearly a real possibility. An indication of the potential severity of the failure is considered to be less than ten fatalities.

3.5.4 Case Study 4 - Change of Bolt Specification

Background

A 2 storey steel frame had a standard braced bay with cross bracing. The structural drawings showed 4 x 30 mm dia grade 8.8 bolts as holding down bolts at the base of the column. Although the column in question was part of a braced bay, the uplift forces were only nominal.

The column was 305 UC and was of 6 m height. The bolts were set into the concrete and there was a standard angle frame (120 x 120 x 8 RSA) at the bottom of the bolts. The bolts were shown as having the bolt head factory formed.

What Happened

The foundations were cast with the bolts and angle frame in place.

The steel erector was erecting the steel and had the column in position. Two of the bolts were lightly tightened.

There was a loud bang and the column fell headlong, narrowly missing a workman.

Causes

One of the bolts had failed suddenly and without warning. The other bolt remained in position, albeit severely bent. Analysis of the bolt showed that the specified 8.8 bolt to BS EN 20898 had not been used.

The bolt installed was in fact a direct alloy hardening steel – EN 24 T. The properties of this type of steel are given in BS 970 Part 2. Reference should be made to Table 3.5.4.a below.

The head of the bolt was not factory made. A nut had had its threads removed, been slipped over the bar and a weld formed on one side of the nut.

The failure occurred at a time without a significant wind load.

Discussion

The EN 24 T steel is not a regular structural grade. It is not suitable for welding, due to the formation of martensite. The fracture occurred in the heat-affected zone of the weld.

Vickers Hardness tests were undertaken on the bolt. A typical value of 314 Hv was found in areas remote from the heat affected zone. A reading very close to the fracture plane was as high as 680 Hv – ie significantly harder (Confidential Report, circa 1990).

The bolt failed as a brittle failure. There was no necking of the bolt adjacent to the plane of fracture, which would have been expected in a non-brittle failure.

Test	Bolt as tested	BS 970 – EN 24 T
Tensile strength MPa	993	850 - 1000
0.2% Proof Stress	866	635 min
Elongation %	18	13 min

Element	Bolt as Tested %	BS 970 – EN 24 T
Carbon	0.41	0.36 – 0.44
Silicon	0.31	0.10 – 0.35
Manganese	0.64	0.45 – 0.70
Phosphorus	0.024	0.040 max
Sulfur	0.007	0.050 max
Chromium	1.08	1.00 – 1.40
Molybdenum	0.21	0.20 – 0.35
Nickel	1.34	1.30 – 1.70

Table 3.5.4.a. Physical and Chemical Properties of the Failed Bolt compared to BS 970

The falling column, weighing over half a tonne was an obvious risk to the erection team.

However, if the braced bay of the completed structure was taking wind load and the bolts failed, the stability of the entire structure could have been compromised.

Underlying Structural Cause

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc – 100%.

The contractor changed the steel specification and formed the head with an improvised detail.

Potential Severity of Failure

Given the location of the bolt, a failure is only possible during construction. The column falling is a potentially fatal incident. As stated previously, the tension forces in the completed structure were only nominal. The design uplift in the column due to the wind and the cross bracing does not exceed the dead load in the column in the completed structure.

The potential severity of failure is assessed as less than ten fatalities.

3.5.5 Case Study 5 - Sports Stadium Roof

Background

A large sports stadium was designed with a long cantilever roof. The cantilever roof was a steel truss and had a span of around 34 m. The steel truss was supported on a precast concrete frame. Refer to Fig 3.5.5.a below.

The back support of the truss was a structural pin joint with a horizontal bolt. The front support was adjustable so that during erection all the trusses could be lined and levelled. The precast concrete had 32 mm dia reinforcement.

The drawings showed the front support for shims and a jack as being set back from the front face of the concrete. Refer to Fig 3.5.5.b below.

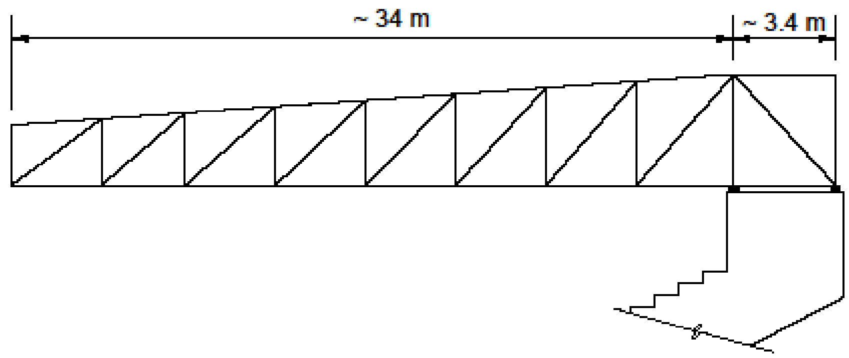


Fig 3.5.5.a. Section through the cantilever truss.

What Happened

The contractor was erecting these long span cantilever beams and a steel erector was at the end of the cantilever truss. The pin was installed at the rear, and the shims and jacks were installed at the front. The shims however were installed at the front of the concrete section as shown in Fig 3.5.5.b below.

Suddenly and without warning, the front top section of concrete sheared off. The steel truss at this location dropped around 100 mm and the tip of the cantilever dropped around 1 m.

The steel erector dropped with the truss, but held on to the steel. The erector was not injured. It is not known if the erector was wearing a harness.

Causes

The shims were not placed in the location as indicated on the design drawings.

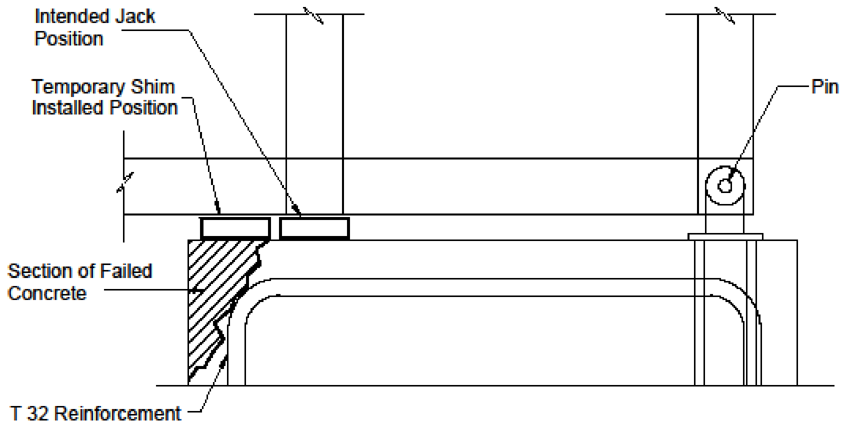


Fig 3.5.5.b. Detail at the support end of the cantilever truss.

The 32 mm reinforcement has an internal radius of 112 mm. This leaves the top corner of the concrete unreinforced.

Discussion

The designer stated that the area where the front shims should have been located was shown on the drawings although the safety aspects of not doing this were not stressed.

He indicated also that future similar details not only stress the importance of correct shim location but also uses a larger number of smaller diameter reinforcing bars. These smaller bars have a tighter radius at bends.

Although it is very unusual for designers to mark certain elements on drawings as a critical safety issue, an important item such as the shim location in this instance is sometimes highlighted. This could be done by putting a note in a box with larger text. Such items should be indicated to the contractor in the risk assessment documentation.

There are obvious comparisons with Case Study 18 where a similar failure at a corbel is discussed.

Underlying Structural Cause

The designer indicated a location for the bearing pad. That stated, he didn't make it plain that this was a structural safety issue.

The underlying structural cause is assessed as the designer's poor communication to site – 20% and the contractor's failure to comply with design drawings etc – 80%.

Potential Severity of Failure

It was lucky that this incident did not have a fatal outcome. The potential severity of the failure is assessed as less than ten fatalities.

3.5.6 Case Study 6 - Shelf Angles

Background

The expansive properties of fired clay bricks and the contracting properties of concrete blockwork have been the cause of numerous serious defects.

A solution to the vertical movement of the bricks is to introduce a brick support angle and a soft joint. Typically this joint would be specified for every third storey. An exception to this would be a 4 storey block where no joint would be specified.

Different clay bricks have different expansive properties and the specific properties for the particular brick should be considered in the façade design.

In the design of a shelf angle, equilibrium of moments require the following equation to be satisfied:

(weight of brick) x (distance of brick centreline to face of wall) = (tension force in bolt) x (distance from bolt centreline to point of rotation of angle)

What Happened

Case Study 6A

The design drawings showed a rolled steel angle with a thickness of 12 mm. The contractor had difficulty in sourcing the angle and ordered a bent flat refer to Fig 3.5.6.a below.

The flat was bent with a large radius bend. As the bricklayer was laying the bricks, the bolt pulled out.

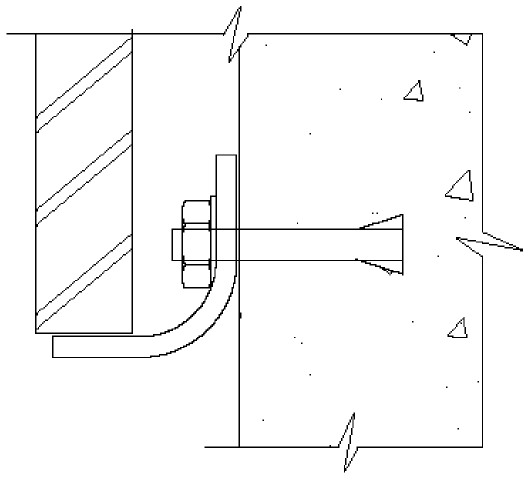


Fig 3.5.6.a. Shelf angle with bent flat rather than rolled steel angle.

(note the distance between the bolt centreline and the point of rotation of the shelf angle is approaching zero)

Case Study 6B

Construction tolerances should always be considered in the design process. This is particularly the case with Computer Aided Design because all dimensions are indicated with high accuracy.

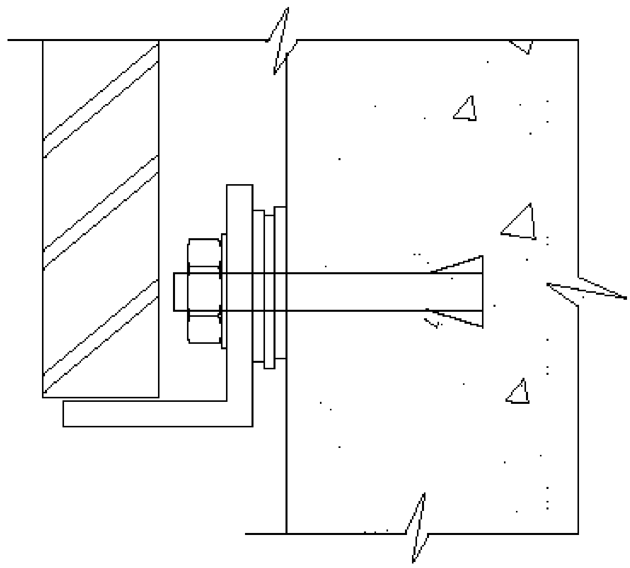


Fig 3.5.6.b. Shelf angle with washers inserted to correct for construction tolerances.

On this particular project, the shelf angle was to be fixed to the insitu concrete frame. The tolerances for the concrete exceeded the normal values. The contractor sought to correct this by adding what was described as “a handful of washers”.

The bolt was designed for pull out tension and shear. The addition of washers generated bending forces in the bolt for which it was not designed.

The shelf angle rotated as the wall was being constructed.

Causes

Case Study 6A

The lever arm for the bolt is effectively zero. Therefore the pull out force in the bolt is very large. The bolt behaved exactly as theory would suggest – it pulled out as the wall was being constructed. It is easy to see how a contractor would fail to appreciate the significance of the substitution of a rolled steel angle with a bent flat. It is not the fact that a bent flat was used, it was because the lever arm was approaching zero.

Case Study 6B

The bolt was significantly overstressed by the use of washers. The lever arm for the bolt was seriously reduced. The lever arm was reduced to half the diameter of the washers. If the lever arm was roughly halved, then the bolt tension is doubled. In addition, the bolt was put into bending for which it was not designed.

Discussion

Brick fixings will now be typically specified as stainless steel. A common specification for shelf angles and the like a couple of decades ago was galvanized steel. As rusting of these angles becomes apparent, it is confidently predicted that problems will manifest themselves in existing structures.

There are significant similarities in case A and case B. Although it could be argued that these two cases should be separately listed for analysis in Section 3.7, these 2 cases are indicated as one because the errors are comparable.

Underlying Structural Cause

The cause of the problem in case A was that the contractor failed to install an angle in accordance with the engineer's instructions.

Case B is a little more complex. Did the engineer's design take into account the dimensional tolerance which was inevitable on site? The answer to this question is unknown but it is quite possible that he did not.

The underlying cause of this case is assessed as. Designer's lack of understanding of structural principles – 20%, Contractor's lack of understanding of structural principles 20% and the Contractor's failure to comply with design drawings etc – 60%.

Potential Severity of Failure

The location of both cases here is not recorded. It is quite likely that they are both in an urban location and therefore the sudden failure of a large brickwork panel could result in multiple fatalities. The potential severity of failure is assessed as less than ten fatalities.

3.5.7 Case Study 7 - N Truss

Background

The Pratt truss is more commonly called the N-truss and is probably the most common form of truss in building structures. The diagonal members on one side of the truss centreline have a different orientation to those diagonals on the other side of the centreline. Typically the diagonals are in tension under gravity loads. This results in lighter members and is a more structurally efficient and cheaper form than the case where the diagonals are in compression.

What Happened

The structural engineer's drawings showed the arrangement as shown in Fig 3.5.7.a below:

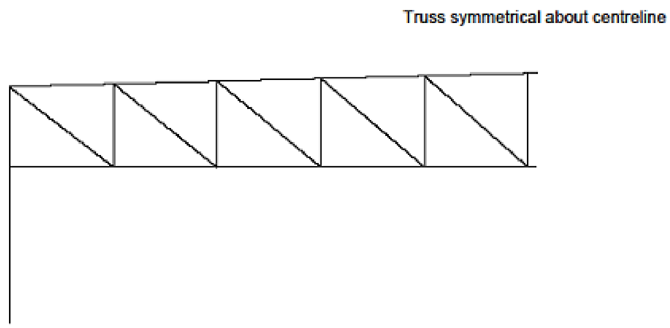


Fig 3.5.7.a. The truss arrangement as shown on the structural engineer's drawing.

The drawing was a manual drawing carried out before CAD drawings were available. With CAD drawing it would take only a few seconds to draw the entire truss whereas with manual drawing it would take the draughtsman longer.

Causes

The steel sub-contractor interpreted the designer's requirements as shown in Fig 3.5.7.b.

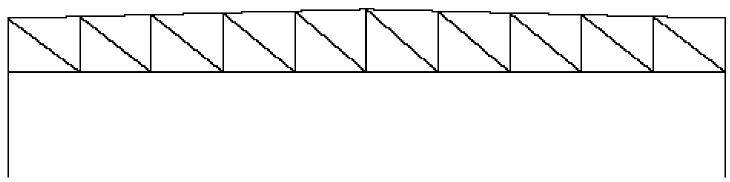


Fig 3.5.7.b. The truss arrangement as fabricated by the steel sub-contractor.

Clearly this was not the designer's intention. The diagonals on the left of the truss will be in tension whereas the diagonals on the right will be in compression.

Some of the diagonal members were overstressed when in compression.

Was the designer's drawing unambiguous? Does the word symmetrical mean mirror image? One dictionary definition for symmetry gives: *The quality of being made up of exactly similar parts facing each other or around an axis.* (Oxford Dictionary 2016). This definition probably only tends to confuse. Any designer's instruction which is not totally clear is an incorrect instruction.

In this actual case the error was spotted during construction and the trusses modified with both serious cost and programme implications.

Discussion

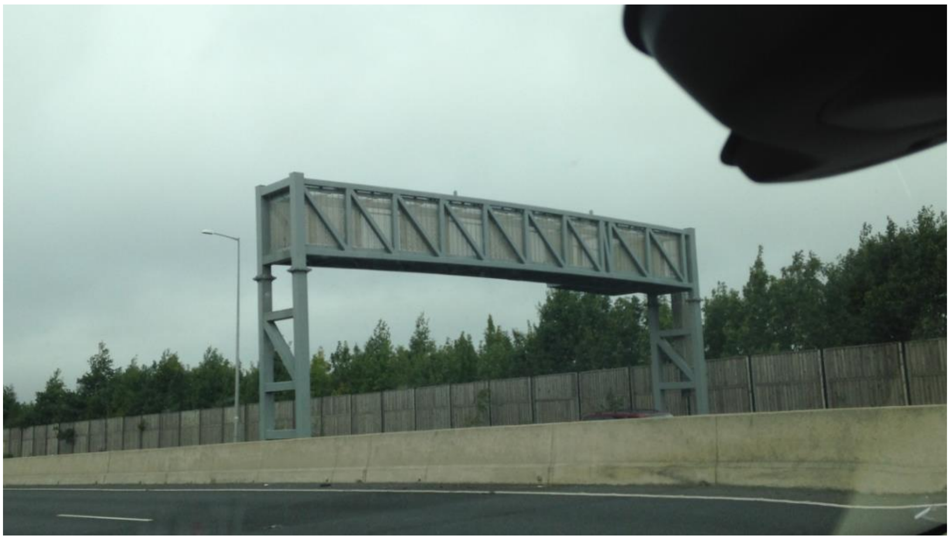


Fig 3.5.7.c. A sign gantry on the M50. (Ian Duckenfield, 2015)

A number of gantries on the M50 are similar in structural arrangement to the one shown above. Because the truss is acting as a portal frame in the direction along its axis, the diagonal members will be in both compression and tension depending on the direction of wind acting along the axis of the frame.

There are two more interesting features in the photograph:

- Why is there a very narrow panel with a near vertical diagonal? Is this an error?
- Why is there a panel at the far end with no diagonal member? This section is therefore a Vierendeel panel. The design of a Vierendeel girder is fundamentally different to the design of an N truss. The Vierendeel truss relies on bending moments in the frame. In contrast, the N–truss assumes a pin-jointed frame with diagonal members typically in tension under gravity loads.

Underlying Structural Cause

The structural engineer failed to show his intent by way of clear and unambiguous drawings.

The underlying cause of this case is assessed as totally the designer's poor communication to site.

Potential Severity of Failure

It is not easy to make an assessment regarding the probable result of this error. A sudden failure of a compression member could have fatal consequences. There is no information on where the defective truss was to be installed. Therefore a fatal incident at the lower end of the scale is considered the probable outcome if the defective truss had been installed. The potential severity of failure is assessed as less than ten fatalities.

3.5.8 Case Study 8 - Wind Damage to Roof of Sports Centre

Background

A major sports centre was constructed on a Design and Build contract basis. The roof construction was specified to be a proprietary system. This system had a British Board of Agrément Certificate. The outline of the build-up of the intended roof was a profiled single skin roof sheet spanning between halters. These halters have a thermal break and are fixed through the lower trapezoidal sheet to the purlin. The upper sheets are secured by folding the sheet profile over the top section of the halter. The form of construction can be seen in Figs 3.5.8.a, b and c.

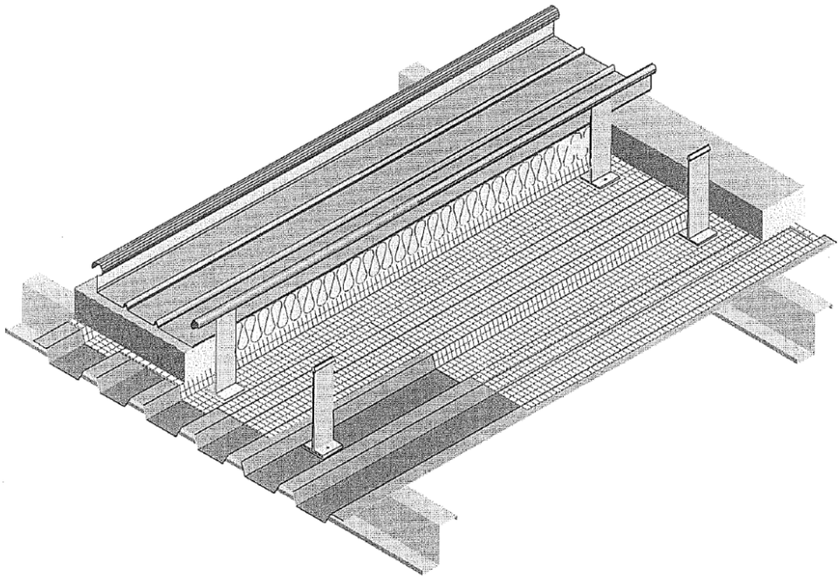


Fig 3.5.8.a. Proposed roof build-up. (Confidential Source, 2015).

(Note the halters are the vertical supports. The base of the halter is 60 x 60 mm.)

What was actually constructed on site was somewhat different.
The upper sheet was 300 mm wide which required the halters to also be at 300 mm centres. The problem arose because the wrong profiled sheet was used for the lower sheet. This lower profiled sheet had troughs at 225 mm centres. This meant that the halters could not be at 300 mm centres as dictated by the upper sheet.

To accommodate this difference, a site modification was made. Two top hat sections are incorporated in the space between the top and bottom sheets. These two top hat sections are at 90 degrees to each other. The upper top hat is continuous and the lower top hat is around 100 mm long.

The insulation and a vapour control layer (VCL) are non-structural elements and are not considered further.

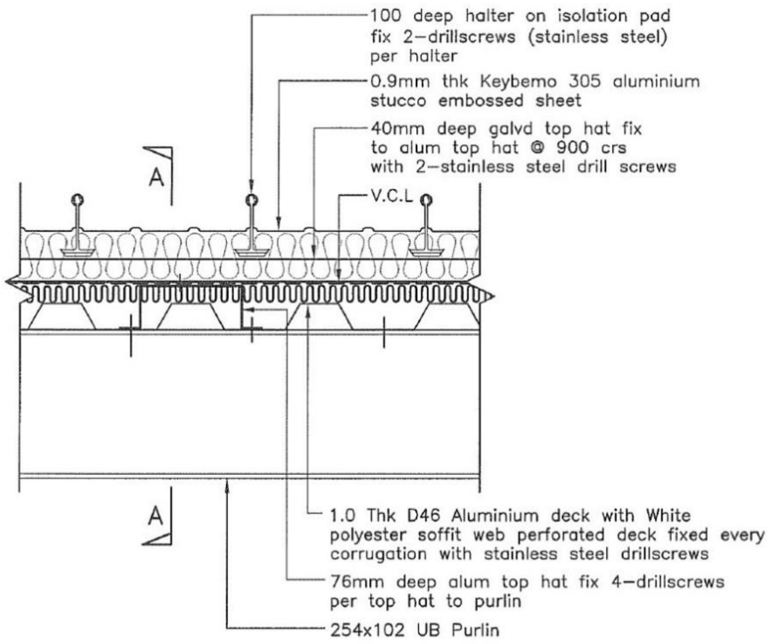


Fig 3.5.8.b. Typical as-built section through the roof (Confidential Source, 2015).

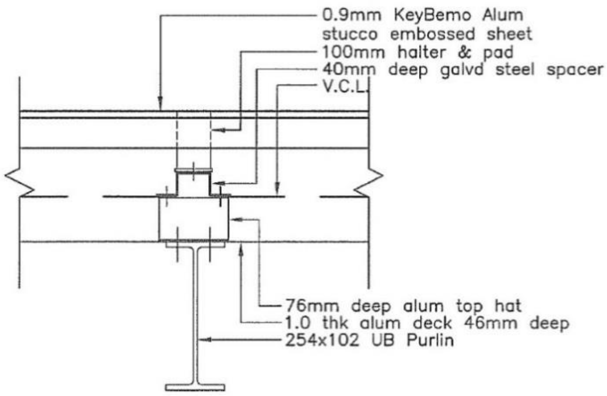


Fig 3.5.8.c. Section A-A through as-built roof (i.e. at 90 degrees to last section).

(Confidential Source, 2015).

The improvised lower top hat section is given as a “76 mm deep aluminium top hat fix 4 drillscrews per top hat to purlin” refer to Figs 3.5.8.b and 3.5.8.c above. The top surface of the top hat section is 130 mm in length, 2 mm thick and has a width (ie distance between folds in the plate) of 180 mm.

The fixings were stainless steel self-tapping screws of steel designation 304 – 1.4301.

What Happened

During a major storm, sections of the roof became detached. The loose debris from this roof was blown onto another roof where further damage resulted. Reference should be made to Figs 3.5.8.d,e and f below.

Meteorological data obtained from a station which is around 4 km away indicated that the wind speed at the time of the roof failure gave a 10 minute average wind speed of 24.2 m/s with a maximum 3 second wind speed of 37.5 m/s. This was indeed a major storm – approximating to Storm Force 12 on the Beaufort Scale. That stated, the roof should have been designed for wind speeds of this magnitude.

Analyses based on the wind speeds obtained from Met Eireann and using:

- CP3 Chapter V Part 2
- BS 6399 Part 2
- IS EN 1991-1-4

give similar figures for the wind uplift pressures – around 2.27 kPa.

Fig 3.5.8.d. Damage to roof (Confidential Source, 2015).

Fig 3.5.8.e. Damage to two top hat sections (Confidential Source, 2015).

(note the continuous upper top hat section and the lower top hat section as 130 mm long)

Fig 3.5.8.f. Damage to lower top hat section. (Confidential Source, 2015).

(note the pull-out value of the screw fixing will be significantly reduced by the buckling of the lower top-hat section)

Causes

The roof was not constructed in accordance with the manufacturer's instructions.

The lower top hat was undersized and a significant bending failure can be seen in the top surface of the section (refer to Fig 3.5.8.e). This bending will reduce the pull-out load that the self-tapping screws can take. The screw threads in the section of the top hat which has opened up, are no longer properly engaged in the metal of the top-hat. The load capacity of the screw is significantly reduced. The failure took place in these screws but the lower top hats remained in place albeit in a severely damaged form.

Discussion

The revised roof structure was under-designed. A significant question is what role the structural engineer should play in specifying wind uplift and reviewing the roofing contractor's drawings.

It is interesting to note that severe rusting had taken place at fixings. This is believed to be due to a lack of a separating membrane between dissimilar metals. Bimetallic corrosion is an electrochemical process involving two different metals. One metal corrodes preferentially to another when both metals are in electrical contact, in the presence of an electrolyte.

A cause for significant concern is that the photograph (Fig 3.5.8.g below) was taken only 2 years after the official opening of the building.

Fig 3.5.8.g. View of lower sheet. (Confidential Source, 2015).

Note perforated webs of deck and rusting at fixing.

Underlying Structural Cause

The contractor failed to construct the roof in accordance with the drawings.

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc.

Potential Severity of Failure

In a storm, a section of the roof deck could detach from the structure, causing fatalities - say less than ten.

3.5.9 Case Study 9 - Portal Rafter End Plate

Background

A portal frame is an extremely efficient form of structure when building warehouses and similar buildings. A large percentage of industrial shed-type structures are built with this form of construction. For the majority of frames of normal proportions, the critical design case is the dead plus live case. At the eaves connection, the frame is subjected to hogging moments. This means that the lower surface is in compression and the upper surface is in tension. The compression force is transferred by direct contact of the steel elements. The tension force, however is transferred by tension in the bolts. The bolts are therefore concentrated in the upper part of the connection.

In this case study, a steel portal frame structure was being erected. A problem arose because the holes in the rafter end plate would not match the holes in the stanchion.

What Happened

The end plate had been welded to the rafter upside down.

The steel erector decided to carry out an on-site modification and to drill new holes <u>in the column section</u>. The frame was erected with only a single pair of bolts at the top of the rafter – ie in the tension zone – refer to Fig 3.5.9.a below.

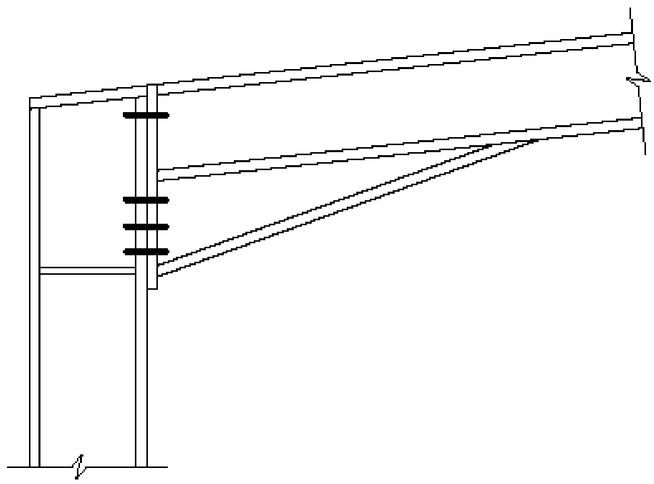


Fig 3.5.9.a. Detail of the portal haunch with the bolts in the incorrect configuration.

Causes

There was a simple error in the fabrication shop. The steel erector decided to modify the steel without reference to the fabricator's drawing office or to a structural engineer.

Discussion

It would be reasonable to assume that an experienced steel erector would know that the bolts at a portal haunch are concentrated at the top of the connection.

It is a fact that a large percentage of the structural steel fabricated in Ireland and Britain is in single storey portal industrial sheds. The design software for portals gives a lean, tightly designed frame with very little spare structural capacity in the frame. The same is the case with connection design.

Although this type of error is not something which a site inspector would be looking out for, this error was spotted.

It is considered likely that the top pair of bolts would have failed. It is also likely that load – shedding would have taken place to adjacent frames in the structure.

Underlying Structural Cause

This was a straightforward case of the contractor or rather the steel sub-contractor failing to manufacture the steel in accordance with the engineer’s details. However, when the lack of fit was noticed on site, the engineer’s advice was not sought.

The underlying cause of this case is assessed as the contractor’s lack of understanding of structural principles - 30%, the contractor’s failure to comply with design drawings etc – 40% and the contractor’s failure to get structural engineering input - 30%.

Potential Severity of Failure

A sudden dramatic failure of the bolts in the connection is considered likely. The failed frame would then shed load onto adjacent frames and a progressive collapse of the roof structure is likely to take place. Typical industrial sheds frequently have a low human occupancy. Also there are often high racking frames in the building and therefore although loss of life is quite likely, the anticipated fatality rate is considered to be low. The potential severity of failure is assessed as less than ten fatalities.

3.5.10 Case Study 10 - Steel Beam with Vibration

Background

Recent trends in building construction are for larger column-free areas which therefore require longer structural spans. This is particularly the case for office accommodation. With longer spans, vibration criteria become more important. Floor vibrations are generated by a number of sources but the most frequent are people walking, jumping and dancing (Steel Construction Institute 1989).

CHAPTER 3 – CASE STUDIES

Human perception of vibration is a complex subject but acceptable limits have been established for different building uses. Different limits exist for offices, hospital wards etc. Additional criteria should be considered in places where particularly sensitive equipment is used such as hospital operating theatres. The problem arises when the natural frequency of the activity e.g. walking, coincides with the natural frequency of the structure.

The main interest for practising structural engineers regarding vibration relates to long span composite floors. This case study, however, relates to a timber floor supported on steel beams.

Human perception of vibration is usually related to acceleration rather than displacement. Historically designers have used the natural frequency as the sole measure of acceptable performance. There are now commercially available software packages which offer sophisticated solutions.

In this case study, the suspended floor of a room which was used as a place of public assembly had timber floors supported by steel beams which spanned between masonry walls. The steel beams were spanning around 10 to 12 m but the sizes of the beams are unknown.

What Happened

Reports were received from people dancing of what was stated to be “an odd sensation”. The diagnosis was quickly made. The dancing rhythms were at the same frequency as the natural frequency of the steel beams.

Additional steel plates were welded to the underside of the bottom flange of the beam. This fix was successful and no further problems were reported.

Causes

The odd sensation was related to the vibration of the floor at certain dance exciting frequencies. The lighter the structural floor, the more likely that vibration issues would become apparent. A timber floor spanning onto steel beams is clearly a light weight system.

Discussion

SCI 076 states that the perception of vibration does not equate to lack of structural safety (Steel Construction Institute, 1989). However the more recent SCI P354 indicates that “a crowd of people jumping rhythmically can generate large loads and this may be of concern for both safety and serviceability evaluations” (Steel Construction Institute, 2009).

SCI Publication 076 gives lower limits for the natural frequency of a floor in normal use to be 3 Hz and for dancing and similar activities of 5 Hz. The more recently published document SCI P354 suggests lower frequency limits of 5 Hz for normal activities and 8.4 Hz for rhythmic activities.

As stated above, the size of the steel beams is unknown, but for illustrative purposes the following structural system could be used - timber joists spanning onto beams 406x178x67 UB at 4 m centres.

The natural frequency of the simply supported steel beam is calculated using the formula (Steel Construction Institute, 1989):

$f_o = 18 \times (y_o)^{-0.5}$

where

f_o = natural frequency in Hz

y_o = maximum short term deflection in mm with load = dead load + superimposed dead load (SDL) + 10% live load

For illustrative purposes take the loads as given in Fig 3.5.10.a below:

	G_k kPa	Q_k kPa	SDL kPa
Live load	-	2.5	
15mm floor boards	0.12		
Joists 225 x 44 at 350 c/c	0.22		
15 mm plaster	0.33		
Steel beam 60 kg/m at 4 m c/c	0.15		
Services			0
Total	**0.82**	**2.50**	**0**

Table 3.5.10.a Loading on deck.

Therefore, dead load + superimposed dead load + 10% live load

$= 0.82 + 0 + (0.1 \times 2.5) = 1.07$ kPa

Deflection $y_0 = (5/384)(w L^4 / EI)$ where:

w = load udl (1.07 x 4 = 4.28 kN/m)

L = span (11 m)

E = Young's Modulus (steel 210,000 N/mm^2)

I = second moment of area (2.43 x 10^8 mm^4 – steel section tables)

Deflection y_o = 16.0 mm and therefore

$f_o = 18 \times (y_o)^{-0.5} = 18\,(16.0)^{-0.5} = 4.50$ Hz

The natural frequency of the steel is calculated to be 4.50 Hz, which is less than the recommended value of 5 Hz.

The beam 406x178x67 UB used in the above example has a flange width of 178 mm and flange thickness of 14.5 mm. Consider then welding an additional plate 160 x 15 to the bottom flange. This additional plate is slightly narrower to allow longitudinal welding of the plate to the beam.

The second moment of area of the beam is 2.43 x 10^8 mm^4. The neutral axis of the composite beam drops to 252 mm and the second moment of area increases to 3.23 x 10^8 mm^4. This represents an increase of 33% in the second moment of area.

The deflection y_0 of the composite beam reduces to 16.0 x 2.43/3.23 = 12.0 mm

The natural frequency of the composite section becomes

$$f_o = 18 \times (y_o)^{-0.5} = 18\ (12.0)^{-0.5} = 5.20 \text{ Hz}$$

The natural frequency is increased to 5.20 Hz, which is above the recommended value of 5 Hz. It is therefore a relatively easy matter to correct this vibration problem.

Underlying Structural Cause

The structural problem of vibrating floors with dancing and other rhythmic activities (walking footfall, aerobics) has been extensively covered in structural design literature, most of which has been published in the last twenty years.

The age of the installation of the floor and steel beams is not known, but is believed to be several decades ago when the state of knowledge would be less than that available today.

The underlying cause of this case is assessed as the designer’s lack of understanding of structural principles – 100%.

Potential Severity of Failure

This is a classic serviceability issue. The risk of catastrophic collapse is very low. The potential severity of failure is assessed as being financial only with a cost of say less than ten million euro.

3.5.11 Case Study 11 - Transfer Beam

Background

An 8-storey office building was designed over a 2 level basement for car parking. A diagrammatic section is shown in Fig 3.5.11.a below. The site was a constricted inner city site with a small footprint. The structural frame consisted of an insitu concrete frame. The regular layout of columns at the upper office levels would not facilitate the circulation of cars in the two basement levels. One main central column was supported by a transfer beam at ground level. Due to of headroom limitations the transfer beam had both a downstand element and an upstand. The upstand was above the general slab level of the ground floor and was within the raised access floor depth. This upstand had been cored extensively for service runs. It is not known if approval had been given by the structural engineer for this coring work.

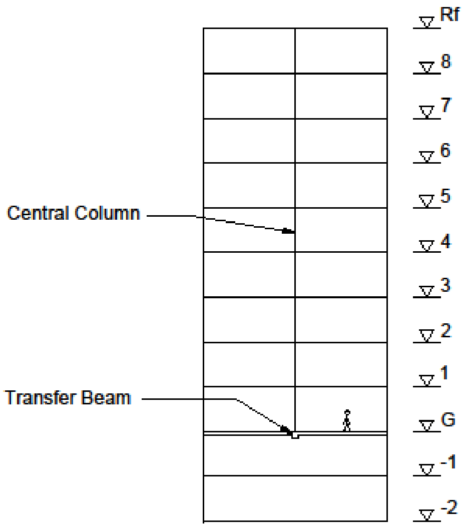


Fig 3.5.11.a. Diagrammatic section through building.

What Happened

During the later stages of construction significant cracking of the transfer beam was noticed.

The remedial work was complex and expensive and is best shown in diagrams. The following diagrams (3.5.11.b to f below) give an indication of the sequence and nature of the work undertaken.

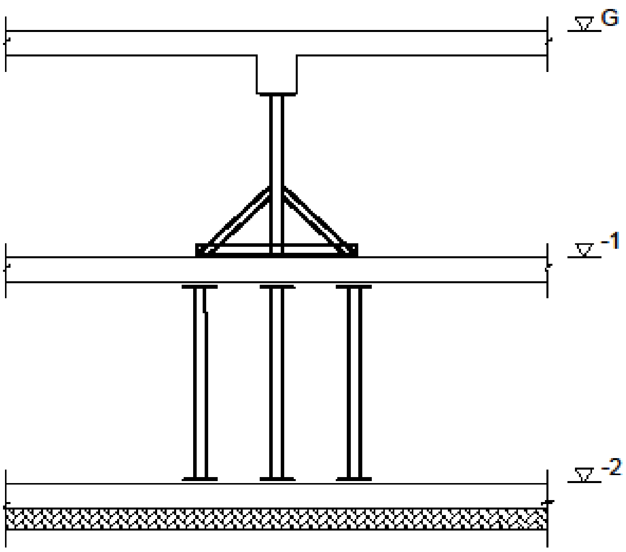


Fig 3.5.11.b. Emergency propping through 2 floors to spread the load at lower basement.

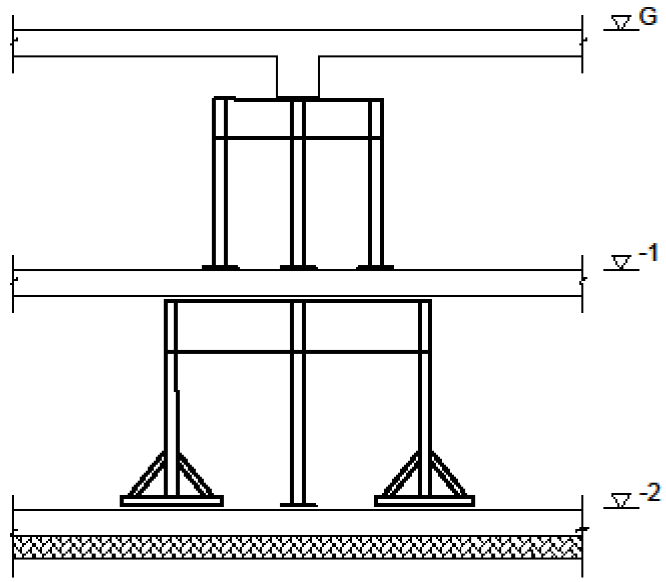


Fig 3.5.11.c. Additional props added to allow new foundation to be formed.

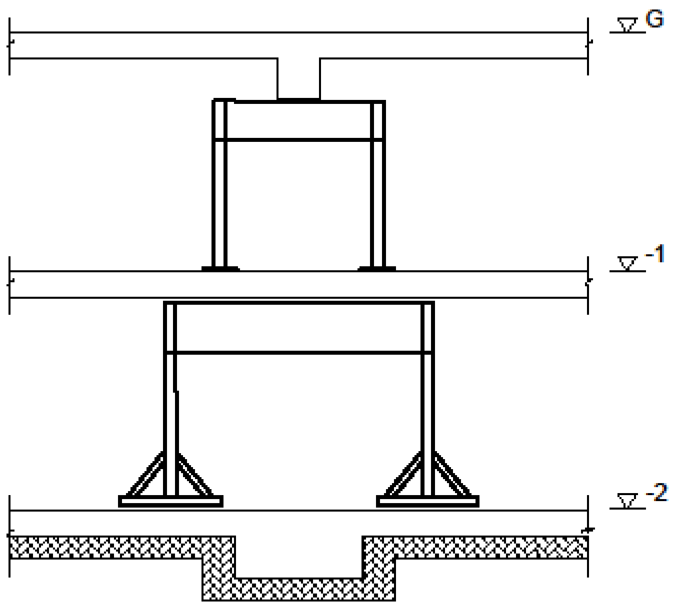


Fig 3.5.11.d. Temporary propping and installation of thickened lower basement slab.

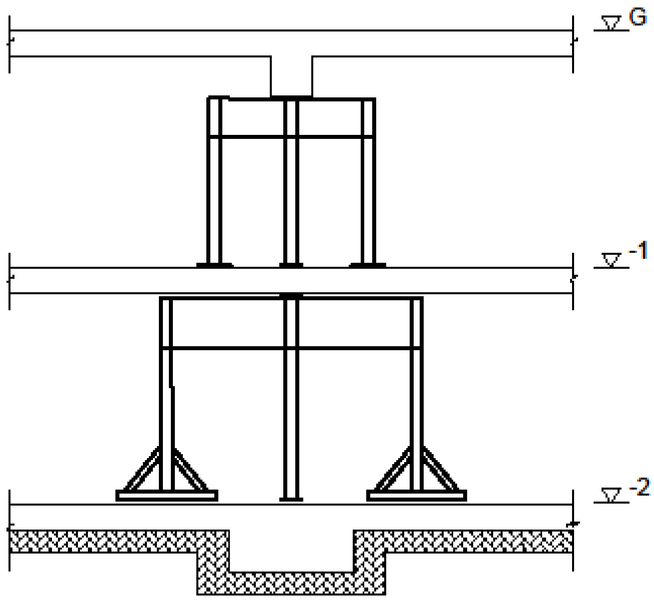


Fig 3.5.11.e. Permanent propping installed.

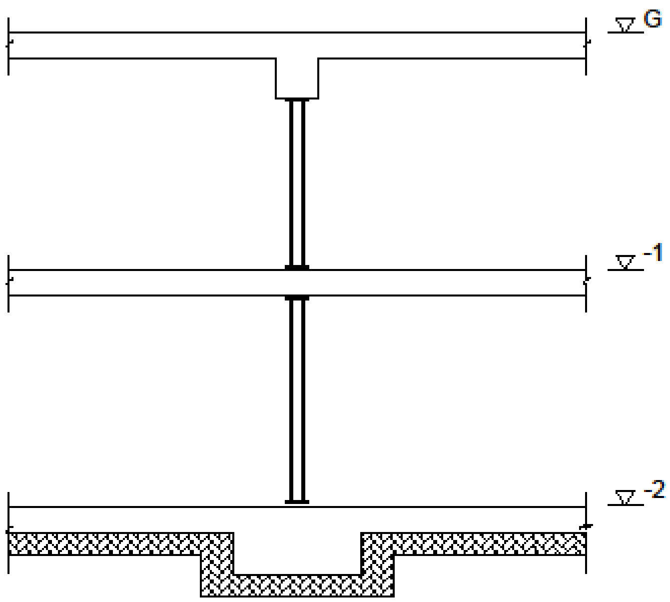


Fig 3.5.11.f. Temporary propping removed leaving permanent propping.

Photograghs were taken during the remedial work and are shown in Figs 3.5.11.g to j below.

Fig 3.5.11.g. View at upper basement level showing emergency prop and temporary props (Confidential Source, 2015).

(Note transfer beam and the depth of the transfer beam.)

Fig 3.5.11.h. View at lower basement showing temporary propping (Confidential Source, 2015).

Fig 3.5.11.i. Jacks ready to take load with deflection gauge installed

(Confidential Source, 2015).

The transfer beam was to be jacked up by 2.5 mm.

Fig 3.5.11.j. Slab in lower basement with slab thickening prepared (Confidential Source, 2015).

Causes

The structural designer made a serious error in the design of the transfer beam. Although contact was made with the designer, he confirmed his involvement but was not prepared to discuss the matter. He indicated that the matter was subject to legal actions and that the design company was no longer trading.

Discussion

Efforts to establish what exactly went wrong in the design office have not been successful. It is not known if the design aspects were the entire issue or if issues on site were factors. The information given in this case study was provided by an engineer who was not involved with the original construction.

A structural solution was proposed but this involved adding a support off the lower basement slab.

The lower basement slab was only designed for car loads and external water pressure. The lower basement slab was 300 mm thick and was supported by undisturbed boulder clay. Clearly adding a column carrying significant loads would give a punching shear problem in this slab. The solution was to create a slab thickening – no easy task in a building essentially structurally complete.

Underlying Structural Cause

The underlying cause of this case is assessed as given below. The designer significantly underestimated the gravity loads in this case. The underlying structural cause of the problem is judged to be the designer’s lack of understanding of material properties at 100%.

Potential Severity of Failure

The transfer beam was exposed to view and therefore significant distress would be noted in the beam if a bending failure was commencing. A shear failure is likely to be more sudden, although signs of significant distress would be noted before a collapse would take place. The potential severity of the failure is assessed as purely a financial cost of less than ten million euro.

3.5.12 Case Study 12 - Reinforced Concrete Column Joint

Background

A large reinforced concrete column was at an expansion joint giving two structurally separate columns – refer to Fig 3.5.12.c below. The dimensions and reinforcement of the column are as shown on the detail below. The two columns were separated by a 50mm flexible joint.

What Happened

The contractor chose to pour the 2 columns separately. The flexible material was simply wedged between the 2 reinforcing cages. The flexible material had no inherent strength. This was obviously an extremely unwise form of construction. The flexible board kicked over resulting in an ugly visible joint and a significant crack in the fair faced concrete.

A sawn joint was made as shown in the Fig 3.5.12.a and b below, but this was not a solution. How the column was left on site is not known.

Fig 3.5.12.a. Photograph showing dislodged soft board and crack in concrete. (Confidential Source, 2015). (Sawn joint highlighted)

Fig 3.5.12.b. Photograph showing base of column and sawn cut. (Confidential Source, 2015).

Causes

As outlined above, the designer's intent is shown in Fig 3.5.12.c below. The designer gives no indication as to how the joints should be constructed. It would be normally be assumed that the contractor would form the flat section of the column using normal shuttering.

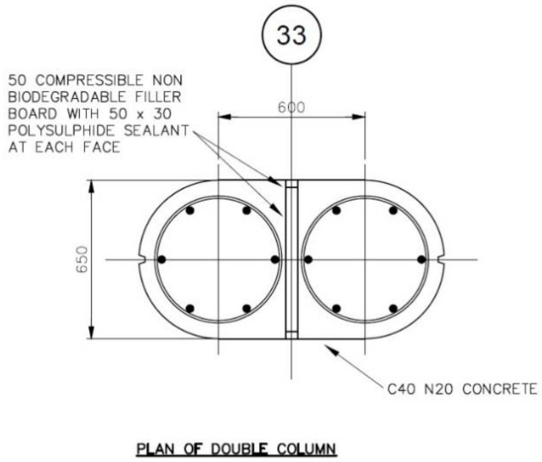


Fig 3.5.12.c. Detail taken from structural drawing showing joint. (Confidential Source, 2015).

Discussion

When the crack in the concrete first became visible, one of the theories put forward was that this was a reinforcement detailing problem. Given the geometry of the column, there was a triangle of concrete without reinforcement. It seems intuitive that the section of unreinforced concrete should not be monolithic with a reinforced section. However, reference to the usual reference documents such as the IStructE Detailing Manual indicated that there were no such limitations. There were various limitations regarding additional reinforcement for surface reinforcement.

Closer inspection on site however indicated that the true nature of the problem was the dislodged soft board.

Underlying Structural Cause

The contractor was manifestly wrong to think that the soft board had sufficient strength to withstand the hydrostatic pressure of a head of concrete.

The underlying cause of this case is assessed as the contractor's lack of understanding of structural principles – 70% and the contractor's failure to get structural engineering input – 30%.

Potential Severity of Failure

This problem is assessed as a financial issue only and at the lower end of the financial scale – say less than ten million euro.

3.5.13 Case Study 13 - Beam in Flat Slab

Background

A new office building was designed and constructed with an insitu reinforced concrete frame. The concrete of the ground floor level was of flat slab construction with a step of 300mm to facilitate a section of the floor with a raised access floor.

At the change in level a beam was formed with top steel, bottom steel and link reinforcement. It was not, however, a beam in the normal sense of the word – i.e. a beam spanning between supports.

What Happened

Cracks in the slab in the area of the beam were noted. Temporary props were installed. It was fortunate that the area under the ground floor was a basement and unoccupied. The cracks were injected with epoxy resin. One wonders if this injection was done with a spirit of "fingers crossed" rather than with the conviction that a permanent solution would be achieved.

The cracks were monitored over a period of years. New cracks were noted and the existing crack widths were seen to increase.

The solution was to install permanent structural props to the beam located in the basement area.

Causes

It became obvious, at an early stage of the investigation, that there was a serious design error here.

A study quickly showed that this beam was not spanning between supports. It was described as "a beam from nowhere to nowhere". The slab was spanning in a perpendicular direction to the "beam" at the change in slab level. A cross section is shown in Fig 3.5.13.a below.

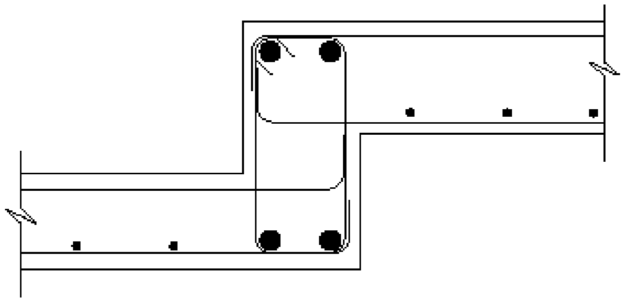


Fig 3.5.13.a. Section through beam at step in level. (Confidential Source, 2015).

Discussion

A number of complex analyses were undertaken, but none could justify the design as built.

This was a straightforward design error. If either:

- the beam spanned between columns
- the reinforcement was detailed in such a way that the moment could transfer across the beam

then there would not have been a problem.

It is to be noted that the beam was at 1/3rd points between columns – the location of minimum bending moment in the slab.

If link shape code 63 had been used rather than shape code 51, and the slab bars had long bobs, would sufficient moment have been transferred through the slab to beam to slab detail? The use of the shape code 63 would have meant that increased lap lengths would have existed and moment transfer increase. For the shape codes, refer to Fig 3.5.13.b below.

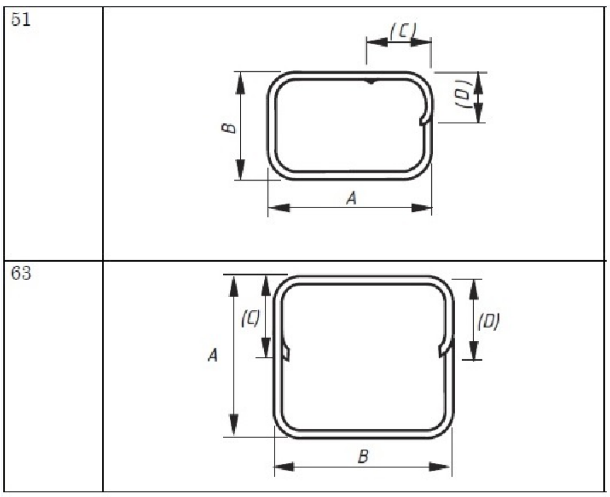


Fig 3.5.13.b. Link shape codes taken from BS 8666: 2005.

Underlying Structural Cause

The structural engineer had made straightforward simple errors.

The underlying cause of this case is assessed as being totally the designer's lack of understanding of structural loads.

Potential Severity of Failure

There was little risk of a catastrophic failure and the location of the problem was fortunate. The outcome was only likely to be financial and at the lower end of the scale. The potential severity of the failure is assessed as only financial with a cost of less than ten million euro.

3.5.14 Case study 14 - Precast Concrete Stairs

Background

The stairs in a school being constructed had a long span precast concrete stairs sitting on an insitu concrete frame. The stairs had two flights with a half landing. There was a standard halving joint detail where the precast met the insitu concrete.

The stairs were designed and cast by a specialist precast concrete supplier. A diagrammatic sections are shown in Fig 3.5.14.a and b below.

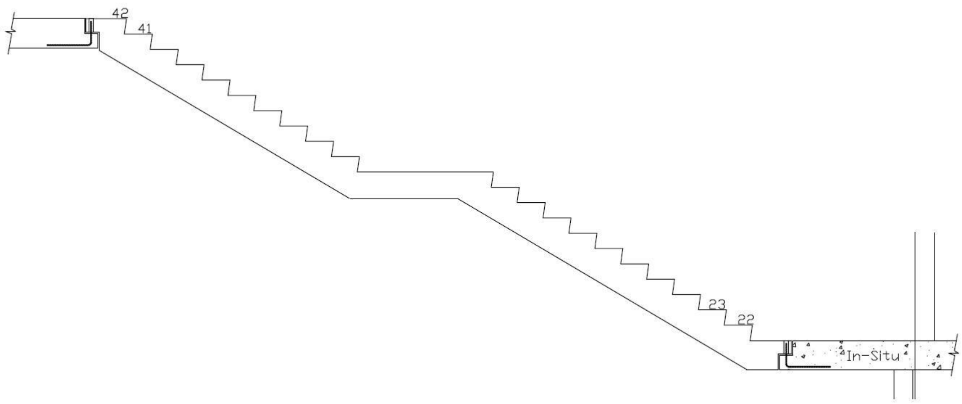


Fig 3.5.14.a. Section through stairs showing general arrangement and halving joints.

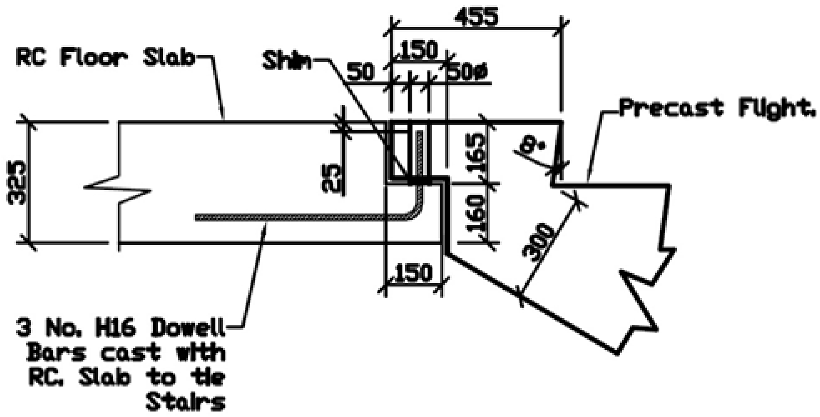


Fig 3.5.14.b. Detail at halving joint. (Confidential Source, 2015).

(Note: this the actual detail from the precast sub-contractor's drawing)

What Happened

Two workers were drilling holes in the stair flight for the handrail standards and they had just stepped off the stairs when a sudden and catastrophic collapse took place. There were no injuries.

The upper stairs failed, collapsed onto a lower, similar flight which in turn collapsed onto the ground floor. Photographs of the collapse are shown in Figs 3.5.14 c and d below.

Fig 3.5.14.c. Collapsed stairs. (Confidential Source, 2015).

Fig 3.5.14.d. Detail at top halving joint (note: dowel hole). (Confidential Source, 2015).

Causes

The primary cause of the collapse was the incorrect detailing of the reinforcement at the junction of the lower flight and the half landing – see Fig 3.5.14.e. The bottom steel is in tension and tries to straighten out. The correct detailing is shown below – see Fig 3.5.14.f.

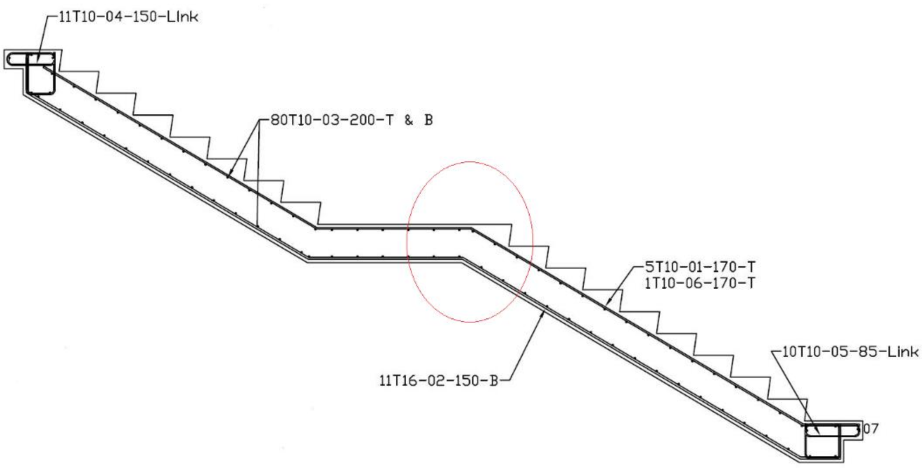


Fig 3.5.14.e. Reinforced concrete details.

(Note: this the actual detail from the precast sub-contractor. The area with the error in detailing is highlighted))

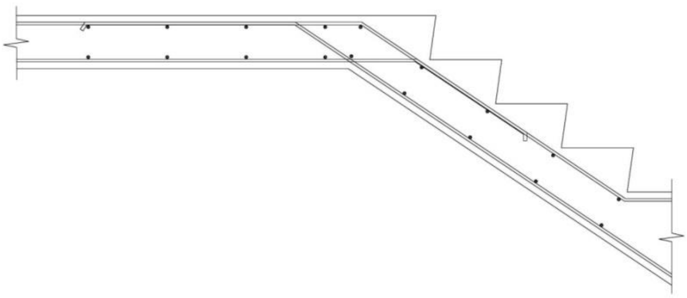


Fig 3.5.14.f. Correct detailing with proper anchorage of bottom steel.

There was also an error regarding the dowel bars at the upper halving joint. The main contractor had fixed the dowel bars incorrectly – putting them at the correct centres but measured starting from the incorrect wall. He had to cut the dowels off and was to grout new dowels in. This re-installation of the dowels did not take place.

The drilling work for the handrail standards provided the loading which initiated the collapse. There was essentially very little live load on the stairs at the time of the collapse and immediately before the collapse.

Another issue relates to the construction joint. The stairs were poured in two pours. The lower flight of the stairs was first cast in the yard face down. The concrete used was a self-compacting mix ie with a high slump. As shown in Fig 3.5.14.g below, the construction joint followed the line of the soffit of the flight and therefore was not at right angles to the plane of the bending moment. The high slump concrete would leave a smooth finish – not a rough one to receive the next pour of concrete.

The compression block of the concrete therefore had an inclined slip plane across it. There were however, two rows of reinforcement crossing the plane.

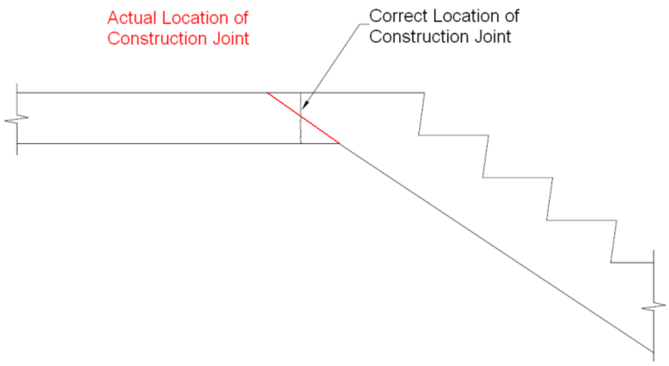


Fig 3.5.14.g. Location of construction joint.

Discussion

The detailing error is a serious error. All detailers would know that bars in this situation must cross and have a proper anchorage. This was the primary cause of the collapse.

That stated, if the dowels had been properly installed and grouted in, the stairs would probably have hung in a catenary type shape and a catastrophic collapse would have been avoided.

An interesting point raised in the aftermath of the incident relates to the fine imposed on the precast contractor. This contractor indicates that the fine relates not to the collapse itself but to the wording of their internal checking procedures. He states that if the procedure had mentioned that the checking work should be carried out “by a competent engineer”, then the fine would not have been imposed. He further argues that if a human error or oversight had taken place, then a successful prosecution would have been difficult.

The contribution of the construction joint line to the failure is not known but is considered to be a minor issue.

The stairs were installed some 7 months before the collapse and in that period would have carried normal construction personnel loading.

It is normal practice to insert steel shims at the halving joint and to grout the joint so that the stairs was bearing evenly across the joint. It was determined that shims were not placed in the joint. Again, it is not easy to determine what contribution, if any, this had to the collapse.

Underlying Structural Cause

The primary cause of this collapse was the incorrect detailing at the joint. Concrete is poor in tension and therefore steel has to be in the tension zone. The precast units are simply supported and therefore the bottom steel in the area of the half landing is in tension. At the bend in the reinforcement the steel bars try to straighten. There is nothing stopping this straightening and the bars burst the concrete cover off. The bars then lose their tension and a collapse follows.

The failure on the contractor’s part to install the dowel bars is a secondary fault.

Some of the blame is assigned to the contractor because of his omission of the dowel bars.

The underlying cause of this case is assessed as the designer’s lack of understanding of material properties – 50% and the contractor’s failure to comply with design drawings – 50%.

Potential Severity of Failure

The stairs were located in a school building under construction and the school was due to be completed and occupied a few weeks after the failure. It is easy to see that there could easily have been double digit fatalities of children if the stairs was carrying a full live load of children. The potential severity of the failure is assessed as fatalities involving more than ten persons.

3.5.15 Case Study 15 - Semi-Precast Floor

Background

The structural drawings show a semi-precast concrete floor spanning between load bearing concrete block walls. The precast section was only around 50 mm thick and contains the sagging bending steel reinforcement. The pre-cast section is not able to support the weight of the fresh concrete to be poured on top of it and must be temporarily propped during the concreting process. Figure 3.5.15.a below shows the significant details.

What Happened

It was the intention of both the contractor and structural engineer that the day works joint would be at the centreline of the block wall. However, the amount of concrete delivered on that day was a little short and a stop end was put in at about one metre from the wall.

The following day, a worker removed some of the temporary props to the slab. There was a sudden shear failure of the thin, pre-cast section of the slab. The slab fell onto the floor below. One man was killed and another received life altering injuries.

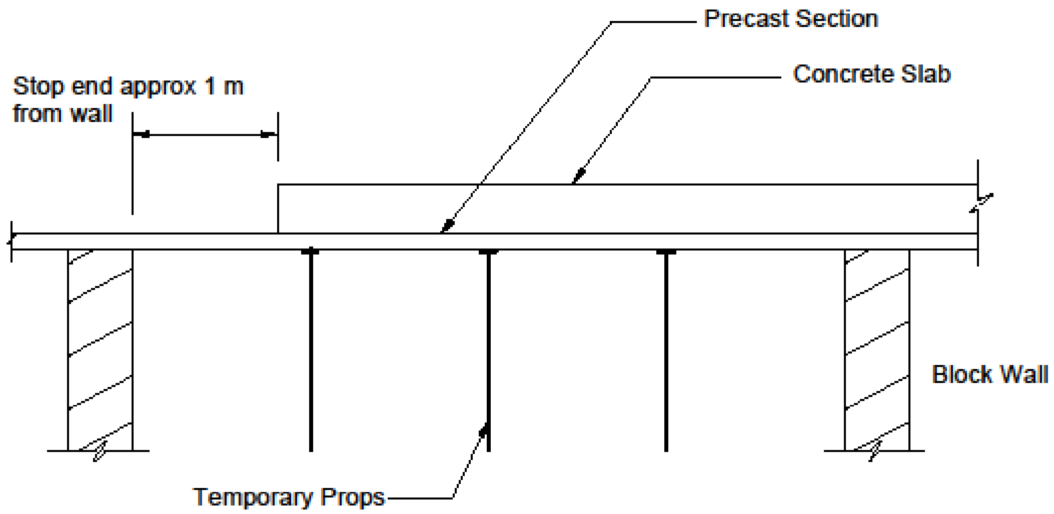


Fig 3.5.15.a. Section showing slab and temporary props.

Causes

The cause of this collapse was, following the removal of the props, the shear failure of the thin pre-cast concrete slab. The slab was subject to loads for which it was not designed. There was clearly poor communication on site.

Discussion

It is easy to see how this very serious collapse took place. When looking up at the underside of the slab, it is not possible to see that the insitu concrete has not extended up to the wall.

A significant aspect of this case is that the structural engineer was successfully sued in relation to this incident. Should the structural engineer have a clause in his concrete specification which specifically deals with this eventuality? Should there be a general clause stating that the contractor has total responsibility regarding the installation and removal of temporary supports? Most structural specifications prepared by structural engineers have explicit clauses stating that responsibility for structural stability during construction rests with the contractor. It is not known if, in this case, such clauses were part of the contract.

Underlying Structural Cause

The underlying cause of this case is assessed totally as the contractor's lack of understanding of structural principles.

Potential Severity of Failure

This was a fatal incident but it would be hard to see how fatalities in excess of ten could occur. The potential severity of the failure is assessed as fatalities involving less than ten persons.

3.5.16 Case Study 16 - Moment into Concrete Column

Background

A new office building had precast concrete columns and insitu concrete floor slabs. The columns were precast because they were partly exposed externally and a special finish could be obtained in the pre-casting yard.

What Happened

When the columns were cast and some of them were erected, it was discovered that there had been an error in the design of the columns. The nature of the error is not known. There are a number of possible causes including an underestimation of the loads, a misunderstanding regarding the mechanism for moment transfer to a precast column or even a mathematical mistake. The main steel was inadequate for the axial load with the moment induced from the insitu slabs.

The solution was to revise the joint detail at the slab to column connection. The original design was to have reinforcement projecting from the column and cast into the insitu slab. This would mean that the connection was monolithic and moments into the column would result and could be readily calculated.

The columns, as constructed, could carry the axial load but if the moment had been added, then the columns would have had inadequate main steel.

The solution was to design the connection as a pin so that the moment was reduced to zero. This was achieved by re-designing the connection with a large dowel pin which was on the centreline of the column as shown in Fig 3.5.16.a below.

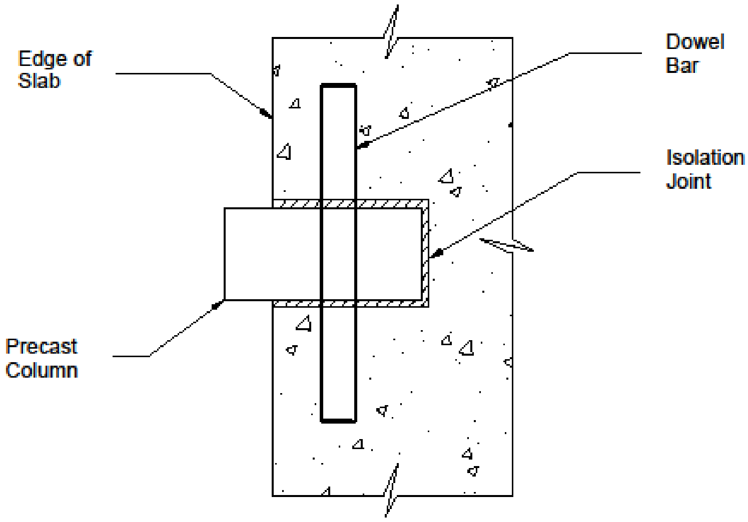


Fig 3.5.16.a. Diagrammatic plan of dowel connection. (Confidential Source, 2015).

Causes

The problem arose from a simple design error.

Discussion

A typical column design chart is shown in Fig 3.5.16.b below. It is Chart 38 in the Code. If the error had not been picked up, what would the consequences have been? The moment from the slab could have induced cracking in the outer surface of the column. This face was exposed to the elements and rusting and spalling would have been likely. It is considered likely that the evidence of structural distress would have indicated a problem and remedial work would have been undertaken.

The under-design was not major and it was lucky that a design change was possible which eliminated the problem.

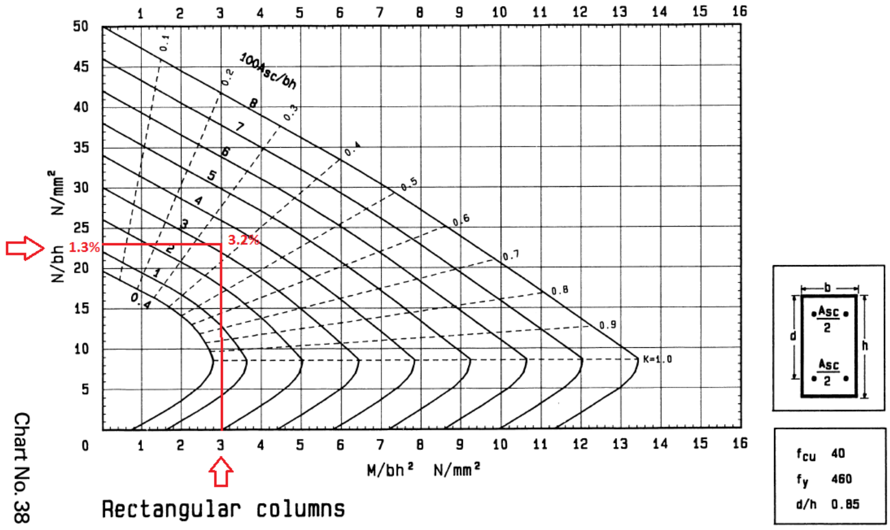


Fig 3.5.16.b. A typical column BS 8110 design chart.

(Note that the actual design values for the column in question are not known, but the diagram above illustrates the point. Using figures which would be typical, the area Asc is reduced from 3.2% to 1.3% by reducing the moment to a nominal value.)

Underlying Structural Cause

This was a straightforward error by the structural designer who underestimated the loads and moments into the column.

The underlying cause of this case is assessed as being totally the designer's lack of understanding of loads.

Potential Severity of Failure

The columns in this case were exposed and any distress in the column due to an underestimation of the axial forces and moments in the column would be seen. It is considered unlikely that a collapse failure would result. A major financial loss would be inevitable – say in excess of ten million euro.

3.5.17 Case Study 17 - Precast Concrete Stadium Steps

Background

A large sports stadium had a series of precast concrete planks acting as an inclined spectator viewing stand in a location exposed to the elements. The engineer's construction drawing showed plastic shims between the individual planks. These individual planks span onto beams at right angles to the planks. A diagrammatic section is shown in Fig 3.5.17.a below.

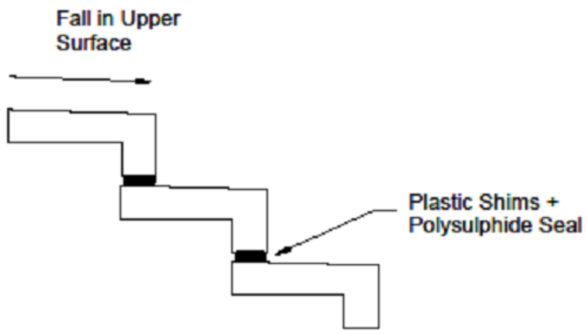


Fig 3.5.17.a Cross-section through precast concrete steps.

What Happened

During erection of the precast units the contractor used steel shims instead of the plastic shims specified. These shims were mild steel and not galvanized. The joint was sealed with a polysulfide type sealant but these are well known to break down after a few years.

The steps were exposed to rain and over a period of years, the shims rusted. This oxidation caused an expansion. The as-constructed gap between units was originally of the order of 6 mm, but some of the gaps after expansion were measured to be as large as 25 mm. This is shown diagrammatically in Fig 3.5.17.b below.

The units tipped back as shown below and considerable amount of rainwater was falling into the occupied areas below the stand.

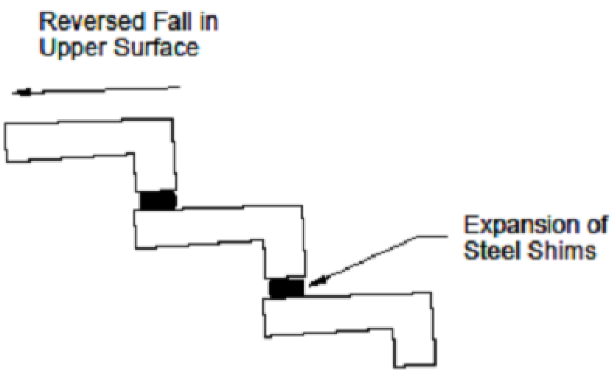


Fig 3.5.17.b Cross-section through precast concrete steps after expansion of steel shims.

Causes

The precast contractor used unprotected mild steel shims in an exposed location. The shims were ungalvanized and had no coating of any kind.

Discussion

The precast contractor accepted liability and had to incur considerable costs to correct the problem.

The engineer had specified a similar detail on a similar stadium at another location. Here, plastic shims had been used and no problems had been reported.

Underlying Structural Cause

This was a straightforward case. The designer had specified plastic shims but the Contractor had arbitrarily changed the specification to unprotected mild steel.

The underlying cause of this case is assessed as the contractor's lack of understanding of structural principles – 20% and the contractor's failure to comply with design drawings etc – 80%.

Potential Severity of Failure

This problem was only likely to be a financial matter and at the lower end of the scale – say less than ten million euro.

3.5.18 Case Study 18 - Precast Concrete Corbel

Background

An industrial shed type structure was constructed with a standard frame with a precast roof beam supported on the nib of the precast column.

The structure was later used as office type accommodation. The nibs were at around 6 metres over the ground floor slab.

What Happened

Suddenly, without warning a section of the concrete nib came crashing to the ground. The weight of the concrete section was around 20 kg. There were no injuries but workers were near and it could it could easily have been an incident with serious injuries or a fatality. An investigation showed differences between the as designed and as constructed condition – refer to Fig 3.5.18.a.

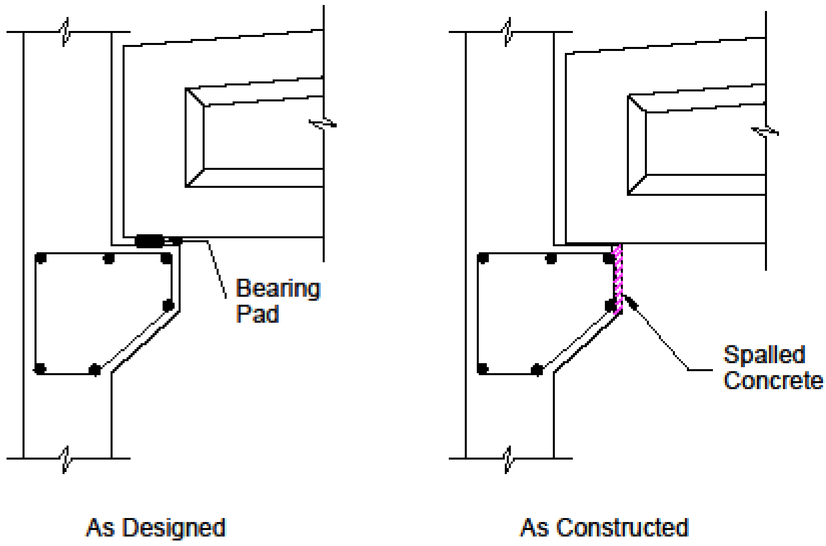


Fig 3.5.18.a. Design detail with the “as constructed” detail. (Confidential Source, 2015).

The section of concrete which fell is shown hatched.

Causes

The structural design showed a bearing pad between the roof beam and the nib. This had not been installed.

The weight of the beam was sitting on a concrete to concrete junction. Probably as some minor creep in the beam took place the load centreline moved to the outside of the nib. The unreinforced end section broke off and fell to the ground.

Discussion

This problem with corbels is well known. One of the functions of the bearing pad is to have an accurate location of the load. The precast erectors would be well aware of the issue. Given that the problem was not recorded elsewhere on the site, it is regarded as a “one-off”.

Problems have been recorded where the load is at the outer edge of the nib, and the moment into the nib is more than that assumed in the design.

Underlying Structural Cause

The contractor failed to install the beam as shown on the engineer’s drawings.

The underlying cause of this case is assessed as the contractor’s lack of understanding of structural principles – 30 % and contractor’s failure to comply with design drawings etc – 70%.

Potential Severity of Failure

A single death is considered to be a possible outcome of this failure.

3.5.19 Case Study 19 - Stud Welding

Background

A project had steel beams which were composite with the deck. The studs were to be welded to the deck directly – i.e. not through the deck. The studs were the standard 19 mm diameter type.

What Happened

The welding work was completed and the adequacy of the weld was to be tested with a bend test. A large percentage of the tests failed.

After the failed tests, the contractor re-welded the studs using a stick welding process. (Fig 3.5.19.a and b refers).

These re-welded studs also failed.

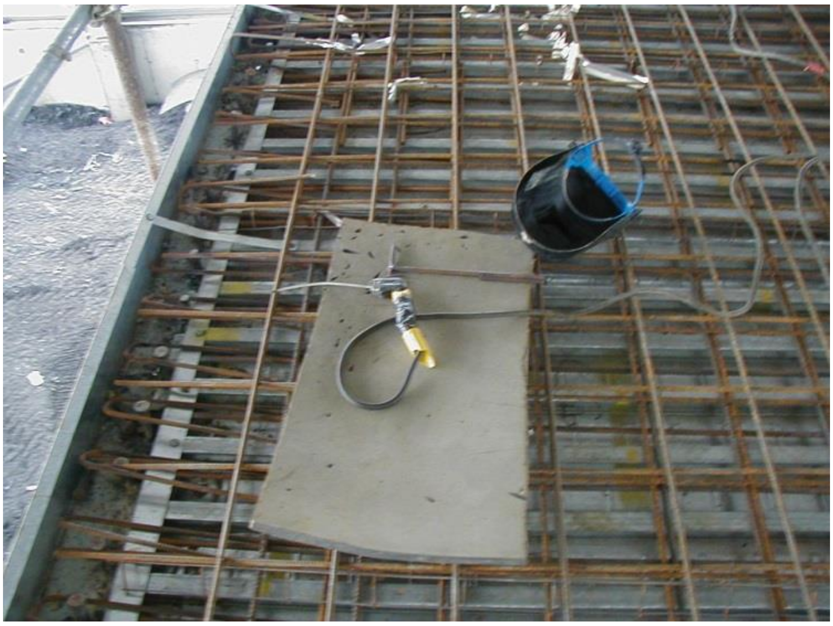


Fig 3.5.19.a. Photograph showing evidence of studs welded with sticks. The studs can be seen to the left of the photograph. (Confidential Source, 2015).

Fig 3.5.19.b. Photograph showing the stud welded to the steel beam using stick welding procedures. (Confidential Source, 2015).

Causes

An investigation into the cause of the first batch of failures identified 2 causes and the likelihood is that both played a part:

- inadequate welding current from the on-site generator
- rain - the welding took place on a rainy day and the area was not under cover.

The re-welded studs should not use normal stick welding. Shear studs must be stud welded to ensure that the full surface of the shear connector is welded to the base metal to provide the necessary shear resistance.

Discussion

Advice from the main supplier of steel studs indicates that the use of stick welding is likely to produce work hardening of the steel of the stud which is likely to produce brittle failure.

The stud testing using the bend over test picked up the inadequate welding on both separate occasions. The contractor was at fault in both instances.

Underlying Structural Cause

The procedures regarding stud welding are well understood in the industry. Generators with inadequate electrical output, procedures with water present and the use of stick welding techniques are all factors which an experienced contractor would understand.

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc – 60% and the contractor's failure to get structural engineering input – 40%.

Potential Severity of Failure

If the stud connections had failed, the composite action between the steel beam and concrete slab would be ineffective. The beam would deflect and sagging floors would most likely be spotted. A sudden catastrophic failure would be unlikely. Death or injury is considered to be improbable. A major financial loss is, however, very likely say in excess of ten million euro.

3.5.20 Case study 20 - Composite Deck

Background

The structural engineer had specified a section of roof as lightweight construction with steel profiled deck, insulation and a membrane.

After the collapse there was a dispute regarding what approvals were granted. What was agreed, however, was that there was a change, and what was actually built was a concrete slab with a steel deck. The deck was permanent shuttering and not composite.

What Happened

During the pouring of the concrete slab there was a sudden collapse of the deck – refer to Fig 3.5.20.a below. An operative working on the deck fell into the hole but managed to cling onto the mesh and was hauled up by a colleague. There was a 6 m drop to the next level. The operative was badly shaken but unhurt.

Fig 3.5.20.a. Deck failure during construction. (Confidential Source, 2015).

Causes

The deck failed in bending as can be seen in the photo.

Discussion

For most common deck types and spans, it is usual that the temporary condition during construction is the limiting case for determining the maximum span.

The clear span of the deck from masonry wall to steel beam was 3.08 m. The manufacturer of the steel deck produces tables giving advice regarding the maximum permitted span. The profiled deck was 60 mm deep and the depth of concrete over the slab 100 mm making a total slab thickness of 160 mm. The maximum clear span given by the manufacturer for a slab of this depth is 2.81 m.

The actual span over the recommended value is 3.08 / 2.81 = 1.10 ie 10%. The bending moment is proportional to the square of the span which indicates an overstress of 1.1 x 1.1 =1.21. This is less than the factor of safety which could be of the order of 1.5.

This suggests that additional loads were on the deck. It is considered likely that the concrete was heaped or that a generator was on the deck at the time of the collapse. A generator is used for the poker vibration of the fresh concrete. Heaping of concrete on decks of this type is specifically forbidden in the deck manufacturer's instructions.

As a separate issue, photos taken immediately after the collapse indicate that the construction at the external wall was incorrect as the cavity was not properly maintained (Fig 3.5.20.b).

It was lucky that no injury occurred. The risk to construction workers is clear.

Fig 3.5.20.b. Photograph showing absence of cavity at external wall. (Confidential Source, 2015).

Underlying Structural Cause

The contractor did not comply with the deck manufacturer's instructions.

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc – 70% and the contractor's failure to get structural engineering input – 30%.

Potential Severity of Failure

A failure of this type can only take place during construction when the fresh concrete is a dead load on the deck. It is usual for the span in the "during construction" phase to be smaller than the span in the "during service" condition in construction without temporary props.

The potential severity of failure is assessed as likely to cause injuries to less than ten persons.

3.5.21 Case Study 21 - Composite Beam with Semi Precast Slabs

Background

The design for a shopping centre showed a steel beam acting compositely with a semi-precast concrete slab. Steel studs were shown as being welded to the beam.

What Happened

The contractor invited prices for precast concrete. He received a cheaper price from a manufacturer of hollow-core units when compared with the semi-precast type.

Without reference to the structural engineer, the contractor appointed the sub-contractor offering the hollow-core option and installed the beam and slab as shown in Fig 3.5.21.a below.

What happened on site and the extent of the problem within the building is not known.

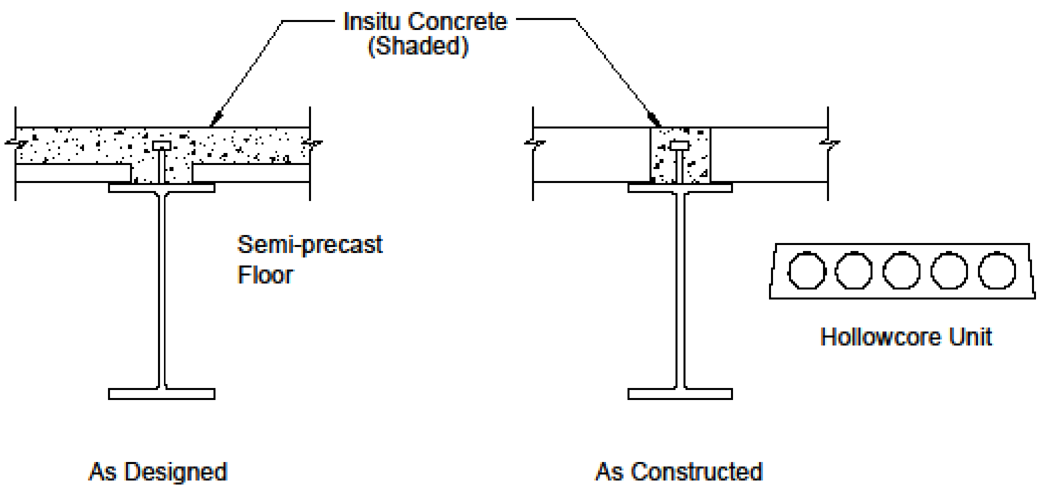


Fig 3.5.21.a. The structural engineer's design intent and the "as-constructed" beam detail.

Causes

The contractor altered the design without reference to the designer. He was apparently unaware of the significance of this revision.

Discussion

Perhaps the contractor thought that all precast concrete was the same and, if he could avoid the propping costs and insitu concrete work of the semi-precast work, so much the better.

The weight of the steel beam is reduced quite significantly if a design is composite rather than non-composite. What would have happened if the completed structure had a radically smaller compression block? It is not unreasonable to think that severe deflections would have taken place and possibly a failure.

Underlying Structural Cause

This is a clear case where the contractor made significant changes to the design with no reference to the structural engineer.

The underlying cause of this case is assessed as the contractor's lack of understanding of structural principles – 40%, the contractor's failure to comply with design drawings etc – 30% and the contractor's failure to get structural engineering input – 30%.

Potential Severity of Failure

Although significant deflections would have taken place, it is quite possible that a beam collapse and a fatality could result. The potential severity of the failure is assessed as fatalities involving less than ten persons.

3.5.22 Case study 22 - Masonry Wall in Warehouse

Background

A single bay steel portal frame building was constructed. The span was approximately 36 m and the length 85 m. The height to eaves is around 7.5 m and the height to the ridge is around 9 m. A full height wall was constructed at the time of the construction of the frame to divide the space into two areas. This wall was made of masonry blocks with steel wind posts.

Under a new occupier, the space was to be further divided so there were 3 separate areas. This new dividing wall was constructed with 215 mm wide concrete masonry units.

There are three roller shutter doors in the structure.

What Happened

On a windy day with one of the roller shutter doors partially open, a large section of the new wall collapsed (Fig 3.5.22.a and b). There were 2 fatalities.

Fig 3.5.22.a. Internal view after the collapse. (Confidential Source, 2015).

Fig 3.5.22.b. Alternative internal view. (Confidential Source, 2015).

Note the restraining wall at the section of wall which did not collapse

Causes

The newly constructed wall had no steel wind posts and no piers. The wall was on the line of one of the steel portal frames. There were however no ties between the blockwork and the steel, either at the column or at the rafter. The blockwork had been butted up to the steel frame with a mortar joint. The face of the steel frame is smooth.

There was no control joint in the 36 m length of the wall. The shrinkage of the blockwork over that length is considered very likely to have pulled the blockwork away from the steel frame. This would have had the effect of reducing a poor tie between wall and frame to a connection of very little or no value.

The collapse took place on a windy day. One of the roller shutter doors was half open. This door faced South West – the direction of the prevailing wind although the wind direction and indeed the wind speed are not known.

Steel portal frames are quite flexible. The wind forces on the day of the collapse are likely to have caused the rafter to have deflected upwards. This would have had the effect of reducing the already poor tie at rafter level to a connection of very little or no value. This would render the wall to be completely free-standing.

Photographs showing the post-collapse condition are shown in Fig 3.5.22.c and d below.

Fig 3.5.22.c. Portal column. (Confidential Source, 2015).

(note the absence of evidence of ties)

Fig 3.5.22.d. Portal ridge. (Confidential Source, 2015).

(again note the absence of ties)

Discussion

There were 2 sections of the wall which did not collapse. These were:

- a section of the full height wall. This was restrained by the wall to a store room which runs perpendicular to the wall which collapsed,
- the low section of the wall up to around 2.5 m over ground slab level. This section of wall is restrained by the office walls and by the ceiling joists to those offices.

The Eurocode gives rules for the design of internal masonry walls which are subject only to internal wind pressures. When the wall in this case study is analysed in accordance with the Eurocodes the wall is seen to be significantly overstressed. (Eurocode I.S. EN 1996: 2006).

The wall as built was around 7.5 m at the eaves and around 9 m at the ridge. If the wall was considered to be unrestrained at the top the maximum height allowed is around 3.2 m. If the wall was properly tied at the top, the maximum height allowed is around 6.5 m. These figures assume that there are no significant openings in the external walls of the building. The dimensions of the roller shutter doors are not known but if either roller shutter door is open in a storm even the 3.2 m and 6.5 m heights given above are too high.

Underlying Structural Cause

There was no design for the wall in question. However an experienced contractor should have been aware that a wall of these dimensions requires particular care with proper advice being taken.

The underlying cause of this case is assessed as totally the contractor's failure to get structural engineering input.

Potential Severity of Failure

The outcome of this collapse involved loss of life and it is arguably the case that more fatalities were quite possible. The potential severity of failure is assessed as fatalities involving less than ten persons.

3.5.23 Case Study 23 - Masonry Walls

Background

Free-standing masonry walls are notorious for failing and 3 examples are considered here.

What Happened

Case Study 23A

All that is known of these cases is what it reported in the press.

"Wall Collapses and Kills Boy.

A 5-year-old Dublin boy was killed yesterday afternoon when a concrete wall collapsed on him when he was playing in his back garden. David Rath, of St Patrick's Road, Drumcondra was in his back garden with his brother and another child when the accident occurred.

One of the children was on a swing attached to the wall and a tree, and it is believed to have caused the wall to collapse."

Irish Times 2 July 1986

Case Study 23B

“FAI to probe Bray Wall Collapse

The FAI and Bray Wanderers will today assess the situation that led to a wall collapsing at the Carlisle Grounds for the second time in two seasons.

…. The surge forward (of the crowd) led to 20 metres of the approximately 1.2 metre wall to fall forward and the crowd to spill onto the pitch. No one appeared to be injured.

A different part of the same wall succumbed to pressure …. In July 2009.”

Irish Independent 9 November 2010

Case Study 23C

“Redeveloping Croke Park.

Two years previously we also had a wall collapse on the toilet block on the Upper Hogan and there were a few people injured.”

Irish Examiner 20 July 2007

Causes

The construction details of the walls in question are not known.

Horizontal forces from crowd loading are well covered in the design codes.

However, the horizontal load from a child’s swing attached to a wall is a more difficult matter and the author knows of no design criteria for this.

Discussion

One wonders what percentage of boundary and garden walls in the country comply with the codes. Any engineer who has designed a free-standing wall in accordance with the codes, will see that a great number of the walls in Ireland fail to meet the requirements of the codes.

Underlying Structural Cause

How many boundary walls are designed by a structural engineer? The answer is very few. In the experience of the author, there is usually no structural engineer involved in the construction of housing estates.

The details of these cases are not known. The Croke Park construction may have had a formal design element but the other incidents probably didn't. The contractor in these latter cases didn't have a design but there are rules of thumb which were not complied with. Therefore the cause (somewhat arbitrarily) is given as shown below.

The underlying cause of this case is assessed as the designer's lack of understanding of structural principles – 20% and the contractor's failure to get structural engineering input – 80%.

Potential Severity of Failure

Case A was a fatal incident the death of a child. The potential severity of failure is assessed as fatalities involving less than ten persons.

3.5.24 Case Study 24 - Wind Damage to "Double Roof"

Background

An apartment block was constructed in a location with increased wind speeds due to the topography of the area. The roof had an unusual form of construction.

The roof consisted of a steel frame with some blockwork walls with timber joists spanning between the steel and masonry. Plywood sheets were nailed to the timber joists. Unusually, a second layer of timber joists were nailed to the lower timber layer and a second layer of plywood fixed to the upper joists.

The reason for this second layer of joist and plywood is not known. It may be that the second layer was added to provide falls in a required direction (refer to Fig 3.5.24.a).

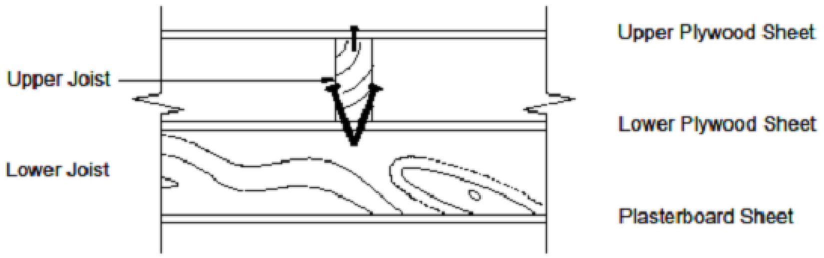


Fig 3.5.24.a. Cross-section through the roof

What Happened

On a windy day, a section of the roof blew off and landed in the car park (refer to Figs 3.5.24.b, c and d below).

There were no reports of injuries.

Fig 3.5.24.b. A section of roof landed in the car park. (Confidential Source, 2015).

(note sections of lower plywood attached to upper joists.)

Fig 3.5.24.c. View of damaged roof. (Confidential Source, 2015).

Fig 3.5.24.d. Additional view of damaged roof. (Confidential Source, 2015).

In this photograph note:

- near to the camera, the lower plywood is absent, whereas further away the lower plywood remains in place.
- the top flange of the joist hangers are not fixed to the blockwork.
- some of the lower joists have nails, whereas other joists have no nails visible.

Causes

The apparent intention of the contractor was that the upper level of joists were diagonally nailed to the lower joists. However, when this nailing work was carried out, the lower level of plywood was laid and it was not possible to see where the lower joists were located. Consequently most of the nails were only fixed to the plywood.

Discussion

Although the apartment block was in an exposed location, the wind speeds on the day of the failure were not severe. Meteorological records for the day when the roof damage occurred at 2 stations both around 15 km from the site indicate maximum gust speeds of 26.2 m/s and 23.7 m/s. These figures are well below the design wind speed for the area of 43.5 m/s.

The mechanism for the roof uplift damage is that the initial failure occurred at the poor connection of the upper joists to the lower structure. Once this section of roof lifted, the roof peeled back and at the downwind section the lower level of plywood was also removed. The section of damaged roof was largely in one piece on the ground. The number of nails which were fixed to the lower joists and the number of nails fixed to the plywood sheet are not known.

The plywood deck is 20 mm thick. The nails which are fixed to the plywood deck can therefore only have a maximum embedment of 20 mm. The pull-out force for the nails into plywood is therefore significantly reduced when compared to nails driven into joists.

Underlying Structural Cause

It is not known if this structure was built with the advice of a structural engineer. Clearly the contractor built the roof deck in a very unusual way with two separate levels of plywood deck.

The underlying cause of this case is assessed as given as the contractor's failure to comply with design drawings etc – 40% and the contractor's failure to get structural engineering input – 60%.

Potential Severity of Failure

As can be seen from the photographs above, the risk of significant injury is real. The assessment of the potential severity of failure is for injuries involving less than ten persons.

3.5.25 Case Study 25 - Sports Centre Roof

Background

The glulam timber roof to a sports centre with a swimming pool was designed by a specialist glulam contractor in Europe. The design of the roof had an unusually long cantilever at the eaves – some 6 metres. A diagrammatic cross section is shown as Fig 3.5.25.a below.

The roof sheeting was 25 mm plywood with membrane, which was supported by glulam purlins which in turn were supported by the main cantilever glulam roof beam.

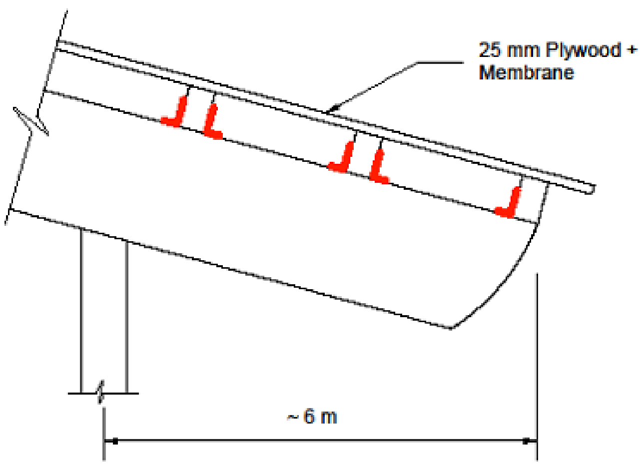


Fig 3.5.25.a. Section at cantilever eaves.

Although the design was carried out by the specialist design and build contractor, the design was to be reviewed by the local structural engineer.

What Happened

During a storm the end section of the plywood roof came loose and then there was what was described as a progressive peeling back of the plywood.

A section of the roof decking crashed through a glass skylight at the ridge of the roof. The broken glass and parts of the plywood roof came crashing down into a swimming pool. It was described that the glass fell between children and adults who were in the pool. Two adults and three children received slight injuries but there were no serious injuries.

Causes

Typically the purlins were connected to the main beam by two steel brackets. The end purlin however, only had a single bracket connecting it to the main beam. The reason for the omission is believed to have been that an “L” shaped bracket was unsuitable because the end purlin was flush with the end of the main beam. A straight bracket would be suitable in this location and no reason is known as to why a straight bracket was not specified at this location.

The wind load is a maximum at the leading edge.

The pressure coefficients for monopitch canopy roofs are given in Table 13 of BS 6399 – the wind code applicable at the time – which is shown below as Fig 3.5.25.b.

Table 13 — Net pressure coefficients C_p for free-standing monopitch canopy roofs

Pitch angle α	Load case	Overall coefficients	Local coefficients		
			A	B	C
0°	Maximum, all ζ	+0.2	+0.5	+1.8	+1.1
	Minimum $\zeta = 0$	–0.5	–0.6	–1.3	–1.4
	Minimum $\zeta = 1$	–1.2	–1.3	–1.8	–2.2
5°	Maximum, all ζ	+0.4	+0.8	+2.1	+1.3
	Minimum $\zeta = 0$	–0.7	–1.1	–1.7	–1.8
	Minimum $\zeta = 1$	–1.4 (– 1.2)	–1.4 (– 1.2)	–2.6	–2.6 (– 2.1)
10°	Maximum, all ζ	+0.5	+1.2	+2.4	+1.6
	Minimum $\zeta = 0$	–0.9	–1.5	–2.0	–2.1
	Minimum $\zeta = 1$	–1.4 (– 1.1)	–1.4 (– 1.1)	–2.6	–2.7 (– 1.8)
15°	Maximum, all ζ	+0.7	+1.4	+2.7	+1.8
	Minimum $\zeta = 0$	–1.1	–1.8	–2.4	–2.5
	Minimum $\zeta = 1$	–1.5 (– 1.0)	–1.5 (– 1.0)	–2.9	–2.8 (– 1.6)
20°	Maximum, all ζ	+0.8	+1.7	+2.9	+2.1
	Minimum $\zeta = 0$	–1.3	–2.2	–2.8	–2.9
	Minimum $\zeta = 1$	–1.5 (– 0.9)	–1.5 (– 0.9)	–2.9	–2.7 (– 1.5)
25°	Maximum, all ζ	+1.0	+2.0	+3.1	+2.3
	Minimum $\zeta = 0$	–1.6	–2.6	–3.2	–3.2
	Minimum $\zeta = 1$	–1.4 (– 0.8)	–1.4 (– 0.8)	–2.5	–2.5 (– 1.4)
30°	Maximum, all ζ	+1.2	+2.2	+3.2	+2.4
	Minimum $\zeta = 0$	–1.8	–3.0	–3.8	–3.6
	Minimum $\zeta = 1$	–1.4 (– 0.8)	–1.4 (– 0.8)	–2.0	–2.3 (– 1.2)

NOTE 1 Interpolation may be used for solidity ratio in the range $0 < \zeta < 1$ and for intermediate pitch angles.

NOTE 2 Where two values are given for $\zeta = 1$, the first value is for blockage to the low downwind eaves and the second value (in parentheses) is for blockage to the high downwind eaves.

NOTE 3 Load cases cover all possible wind directions. When using directional effective wind speeds, use:

a) these values of C_p with the largest value of V_e found; or

b) directional values of C_p from reference [6].

Fig 3.5.25.b. Extract from BS 6399-2: 1997 - Loading for Buildings - Code of Practice for Wind Loads.

For a nominally flat roof with a pitch of 5 to 10 degrees with a blockage ratio (Xi) = 1, the wind uplift coefficient C_p in the area of the leading edge of the cantilever is around 2.6 to 2.7. The depth of the canopy subjected to this load extends for 0.10 x the depth of the canopy which in this case is around 600 mm. Therefore the uplift force in this location can be severe.

The uplift force on the connection between purlin and main beam is unsymmetrical and therefore torsion is induced in the connection. The precise details regarding screw fixings are not known but the connection failed.

Discussion

Both the designer and the reviewer failed to identify a basic error. Although the load on the bottom purlin has half the contributory area of the other purlins, the pressure at the edge reaches a peak value. Additionally, the torsion effects caused by the eccentric connection will magnify the force on the connections considerably.

Underlying Structural Cause

The designer failed to consider the obvious implications of a connection with eccentric loads. There is little doubt that he would wonder how he could have missed such an obvious error.

The underlying cause of this case is assessed as the designer's lack of understanding of structural principles.

Potential Severity of Failure

Although it is considered possible that injury or even death is possible as a result of flying sections of deck, a financial loss is a more likely outcome. The assessment of the potential severity of failure is a financial cost of less than ten million euro.

3.5.26 Case Study 26 - Wind Damage to Plant Room

Background

The plant room on the roof of an apartment block was of standard construction. The blockwork walls supported a nominally flat roof consisting of a membrane which sat on plywood which in turn was supported by timber joists. The joists sat on a timber wall plate. The structural engineer's drawings showed steel straps fixing the timber wall plate to the block walls.

The span of the joists was around 3 m.

What Happened

On a day described as windy but not stormy, a section of the plant room roof was lifted up and sections fell into the car park. There were no reports of injury.

Causes

Flat roofs, particularly in exposed areas where wind uplift forces are increased, must be anchored down. A typical design recommendation is given below as Fig 3.5.26.a. The Standard states that these straps should be at centres not exceeding 2 m.

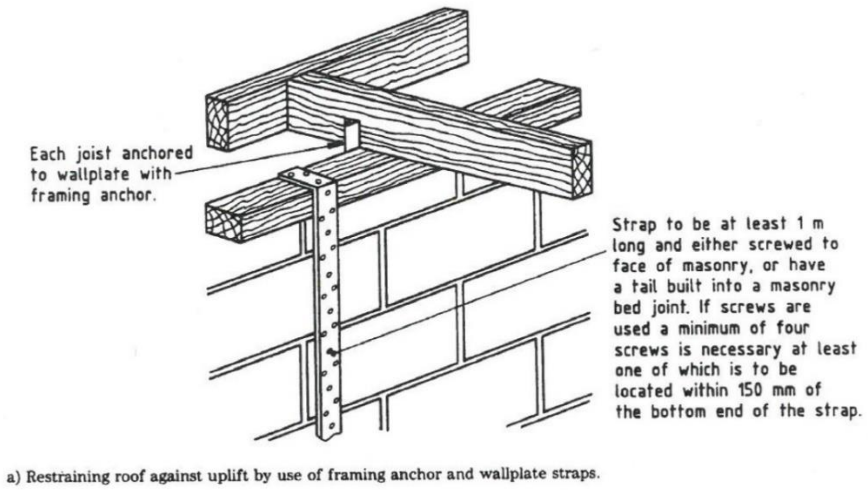


Fig 3.5.26.a. Wall plate strap detail taken from BS 8103 – Code of Practice for Low Rise Buildings.

A photograph showing a similar construction in the same development is shown below as Fig 3.5.26.b.

Fig 3.5.26.b: View of an undamaged roof in the same development similar to the failed roof.

(Confidential Source, 2015).

(Note the strap in place and that it would not be possible to fix straps behind the pipes. This is a case of the space is taken by whichever trade takes it first. Note also that the face of the wall plate is not in line with the inside face of the wall. This is significant – see below. It is reasonable to ask what kind of quality control took place on the site. Note the copper pipes are sometimes lagged and sometimes not.)

Fig 3.5.26.c below shows the damaged roof where the straps can be seen.

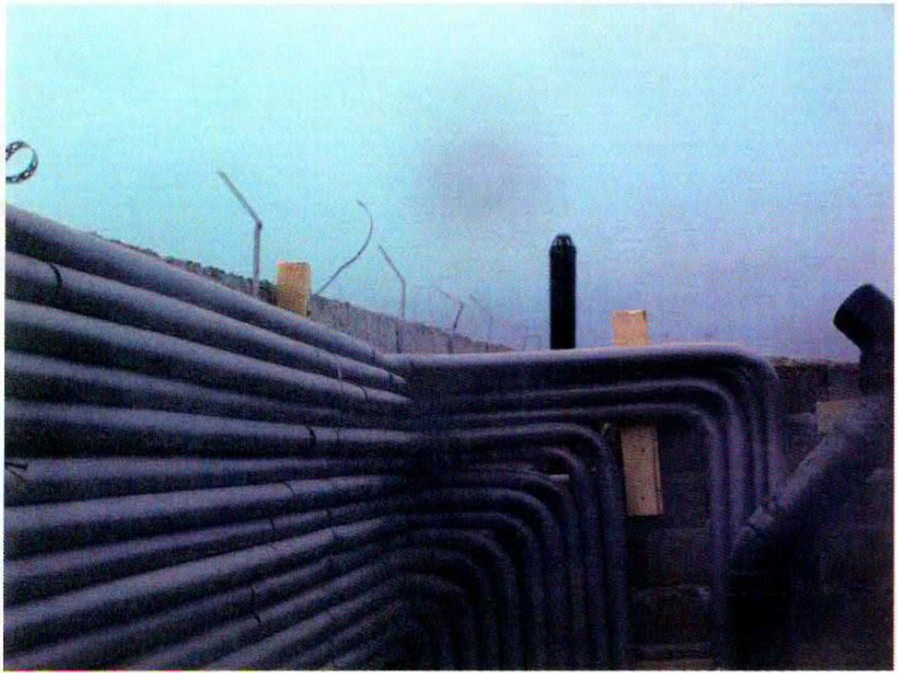


Fig 3.5.26.c. Roof structure ripped off. (Confidential Source, 2015).

(Note straps which appear to be closer than 2 m.)

Discussion

Copper plumbing pipes were installed before the roof was constructed. Therefore a section of roof did not have the wall plate strapped to the wall. It is believed that the section of unstrapped roof lifted up allowing wind to get under the rest of the lightweight roof and the roof peeled progressively off.

Additionally, in places the wall plate was seen to be central on the wall rather than having the inside face of the timber flush with the inside face of the blockwork. This as-built condition is not a good detail because, when the wall plate is subject to an uplift force the 90^0 bend in the strap can straighten out allowing the wall plate to lift off the wall and possibly allowing wind to get under the roof structure.

Underlying Structural Cause

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc.

Potential Severity of Failure

There are clear similarities with the Sports Centre roof failure (Case Study 25). Again, although it is considered possible that injury or even death is possible as a result of flying sections of deck, a financial loss of less than ten million euro is a more likely outcome.

3.5.27 Case study 27 - Sulfates in the Ground

Background

An inner city brownfield site was to be developed. The ground had poor physical properties and a driven piles solution was agreed. The precast concrete driven piles were installed.

The project was then put on hold before any pile caps were constructed.

10 years went by and the project was restarted. A main contractor was appointed and mobilised on site. When he excavated to expose the tops of the piles, all he could find was the reinforcing steel of the piles. The concrete of the pile had "simply disappeared". The concrete in the piles had reverted to the constituents of the concrete, namely aggregate and sand.

What Happened

Chemical tests of the ground revealed ground sulfate levels which were described as being in excess of any normal measure. Further investigations indicated that the site had, for a number of years, been used as a fertilizer plant. There was also a suggestion that the area had been used as a car dismantler's yard and that car batteries had been broken open to extract the lead plates. The acid in the batteries was allowed to seep into the ground.

Causes

No chemical tests of both the soil and ground water had been undertaken at the initial design phase.

Discussion

Sulfate attack on concrete is well documented. The standard reference for this is SD 1 Concrete in Aggressive Ground published by the Building Research Establishment. Part of this document is shown below as Fig 3.5.27.a.

There are two forms of sulfate attack. The mechanism for the more common form, the so-called conventional form is as follows. There must be present:

- a source of sulfates, generally from sulfates or sulphides in the ground
- mobile groundwater

- calcium hydroxide and calcium aluminate hydrate in the cement matrix

The sulfate ions penetrate the hardened concrete and react with the calcium aluminate hydrate to form ettringite. This mineral has a larger volume than the original chemicals and the needle shaped crystals cause internal expansive forces. When these forces exceed the tensile strength of the concrete, then cracking of the concrete takes place.

Assessing the aggressive chemical environment

Table C1 Aggressive Chemical Environment for Concrete (ACEC) classification for natural ground locations [a]

Sulfate				Groundwater		ACEC
Design Sulfate Class for location	2:1 water/soil extract [b]	Groundwater	Total potential sulfate [c]	Static water	Mobile water	Class for location
1	2 (SO_4 mg/l)	3 (SO_4 mg/l)	4 (SO_4 %)	5 (pH)	6 (pH)	7
DS-1	< 500	< 400	< 0.24	≥ 2.5		AC-1s
					> 5.5 [d]	AC-1 [d]
					2.5–5.5	AC-2z
DS-2	500–1500	400–1400	0.24–0.6	> 3.5		AC-1s
					> 5.5	AC-2
				2.5–3.5		AC-2s
					2.5–5.5	AC-3z
DS-3	1600–3000	1500–3000	0.7–1.2	> 3.5		AC-2s
					> 5.5	AC-3
				2.5–3.5		AC-3s
					2.5–5.5	AC-4
DS-4	3100–6000	3100–6000	1.3–2.4	> 3.5		AC-3s
					> 5.5	AC-4
				2.5–3.5		AC-4s
					2.5–5.5	AC-5
DS-5	> 6000	> 6000	> 2.4	> 3.5		AC-4s
				2.5–3.5	≥ 2.5	AC-5

Fig 3.5.27.a. Extract from SD1 - Concrete in Aggressive Ground (BRE, 2005).

This figure shows the design sulfate classes for different levels of sulfates in soil and groundwater. There are a number of factors to be considered when specifying concrete which is in contact with ground containing sulfates. These factors include ground water acidity and mobility, type of concrete element and concrete thickness.

It was extremely lucky that the construction work was stopped when it was. If, as would happen in almost all cases, the structure had been completed on foundation piles which were only going to be sound for a couple of years, the structural implications are obvious. A major foundation failure and catastrophic collapse could have taken place.

It could be said that the concrete in the piles, although it had reverted to the constituent aggregate and sand, would have had the effect of densifying the soil in the same way stone columns densify the ground.

Underlying Structural Cause

The engineer had failed to carry out a basic test of the chemicals in the ground on the site. This is particularly important on a brown-field site.

The underlying cause of this case is assessed as the designer’s lack of understanding of material properties.

Potential Severity of Failure

If the building had been constructed immediately after the piles were installed, as was the original intent, then the building would have been constructed on a piled foundation which eventually would have simply failed to exist. A catastrophic structural collapse would be reasonably foreseeable with multiple loss of life – say in excess of ten persons.

3.5.28 Case Study 28 - Sockets in Piles

Background

A bridge was to be constructed over a river. The plan shape is such that the spans vary with the largest span being around 16 m. The ground conditions are poor, with soft material including peat overlying rock. A geotechnical investigation was carried out and rock was found at 4.5 to 5.5 m below ground level. The rock was described as “strong to very strong, fresh to slightly weathered, thin to medium bedded, flat dipping (0 degrees to 10 degrees) dark grey argillaceous limestone”.

The piles were mini-piles – hollow steel of diameter 80 mm. Each pile had holes along its length. After the piles were formed to their final depth, grout was injected under pressure into the pile. This grout passed through the holes and improved the load bearing properties of the ground. Refer to Fig 3.5.28.a below.

The design required that the piles were rock socketed into the rock. The original plan was to carry out static load tests on the piles. The static load testing was deleted and dynamic tests were substituted. This dynamic testing used dynamic methods connected to a laptop. The exact nature of the dynamic testing is not known.

Fig 3.5.28.a. Mini-pile rig in use. (Confidential Source, 2015).

This photo in Fig 3.5.28.b below shows the precast beams being installed.

Fig 3.5.28.b. Lifting precast beams. (Confidential Source, 2015).

What Happened

A pile failure occurred during construction – refer to Fig 3.5.28.c below. Investigation showed that a number of piles had buckled and one side of the bridge had sunk by some 1.3 to 1.4 m.

Fig 3.5.28.c. View after pile failure. (Confidential Source, 2015).

Causes

The piles are slender and require a rock socket to form a moment connection at the base of the pile. The slenderness ratio of the pile is sensitive to the fixity condition at the base of the pile.

The rock sockets had all been tested using dynamic methods. All the tests had indicated that a good socket had been achieved, but investigations indicated that this was not the case. What had gone wrong?

It transpires that the laptop had been programmed for an 80 mm solid pile rather than hollow. This input error skewed the results to indicate that the piles had passed the test.

The connection at the bottom of the pile was assumed to be a moment connection, but had become a pin. It is not easy to give accurate effective length factors but, with some engineering judgement, it went from 0.85L to 2.0L. Refer to the effective length factors given in Fig 3.5.28.d.

Table 22 — Nominal effective length L_E for a compression member[a]

a) non-sway mode			
Restraint (in the plane under consideration) by other parts of the structure			L_E
Effectively held in position at both ends	Effectively restrained in direction at both ends		$0.7L$
	Partially restrained in direction at both ends		$0.85L$
	Restrained in direction at one end		$0.85L$
	Not restrained in direction at either end		$1.0L$
b) sway mode			
One end	Other end		L_E
Effectively held in position and restrained in direction	Not held in position	Effectively restrained in direction	$1.2L$
		Partially restrained in direction	$1.5L$
		Not restrained in direction	$2.0L$
[a] Excluding angle, channel or T-section struts designed in accordance with **4.7.10**.			

Fig 3.5.28.d. Extract from BS 5950 – 1: 2000 Structural Use of Steelwork in Building.

Discussion

The solution is best illustrated with the following diagrams – Fig 3.5.28.e to k. These diagrams are taken from the actual documentation relating to the corrective measures undertaken on site. The sequence of the work undertaken is shown in the diagrams.

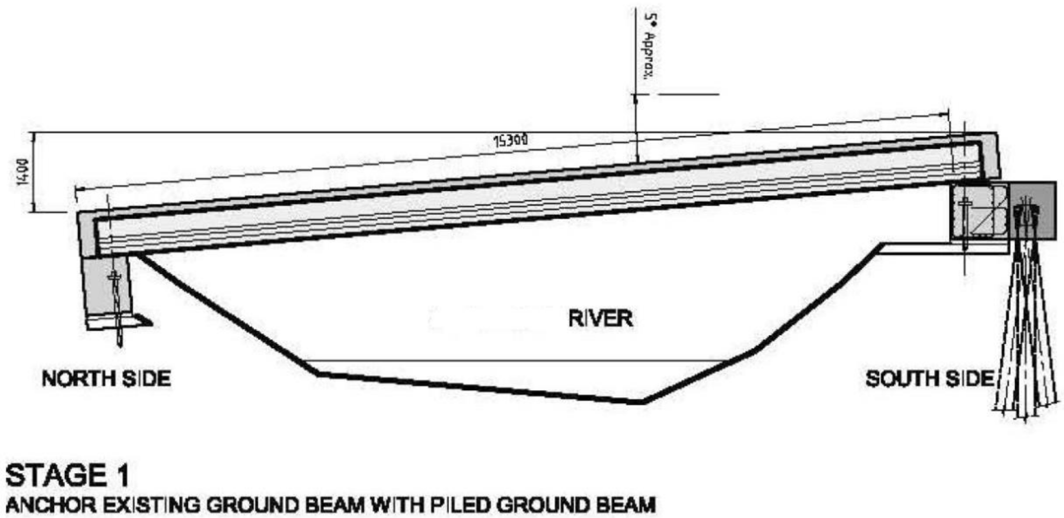
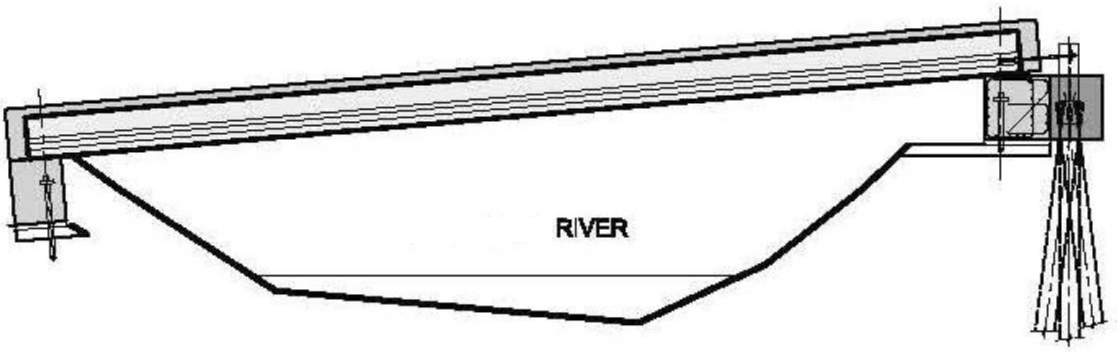
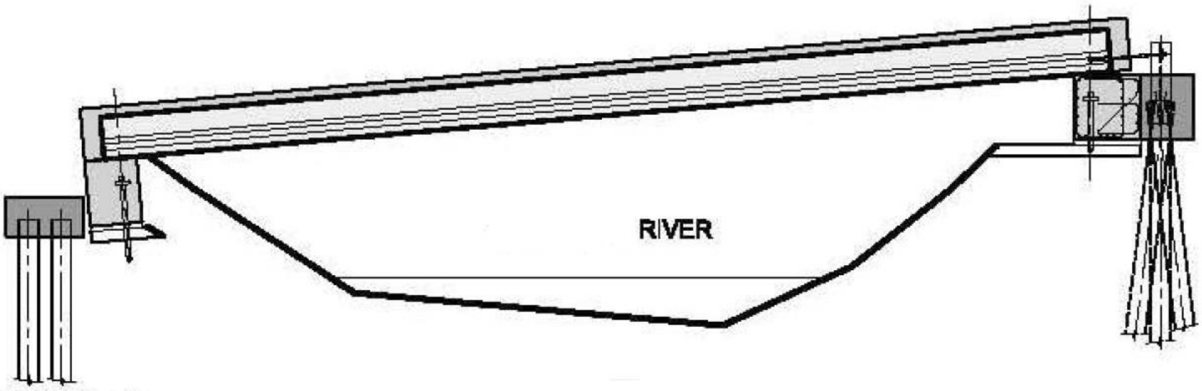


Fig 3.5.28.e. New raking piles are installed at the south abutment. (Confidential Source, 2015).



STAGE 2
INSTALL RESTRAINT STEELWORK
SEE DETAILS DRAWING No. BR0153

Fig 3.5.28.f.The restraint gives a moment connection to the top of the original piles on the South side. (Confidential Source, 2015).



STAGE 3
INSTALL PILED GROUND BEAM

Fig 3.5.28.g. New piles and ground beam at the North side. (Confidential Source, 2015).

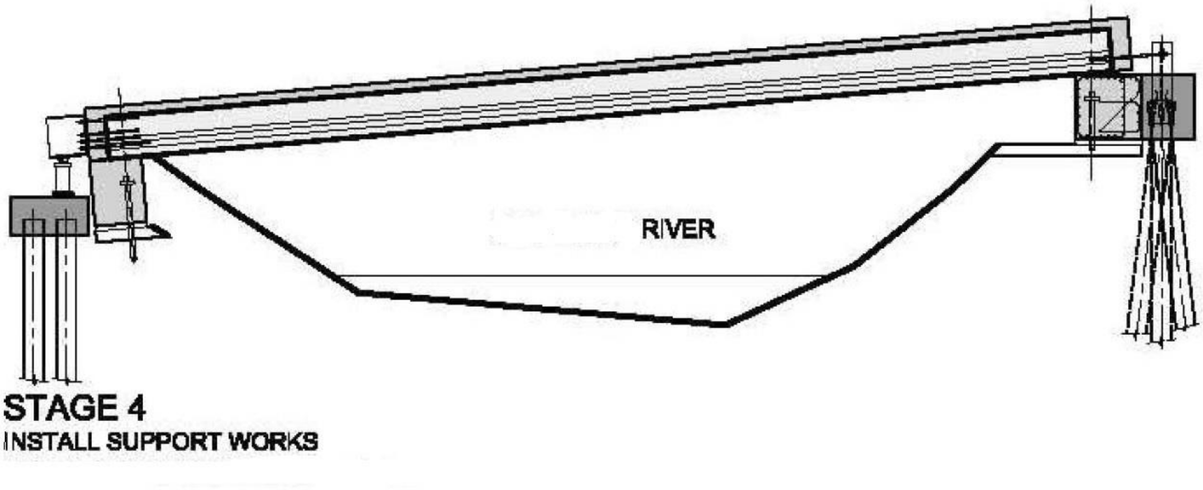


Fig 3.5.28.h. New bracket and jack at the north side. (Confidential Source, 2015).

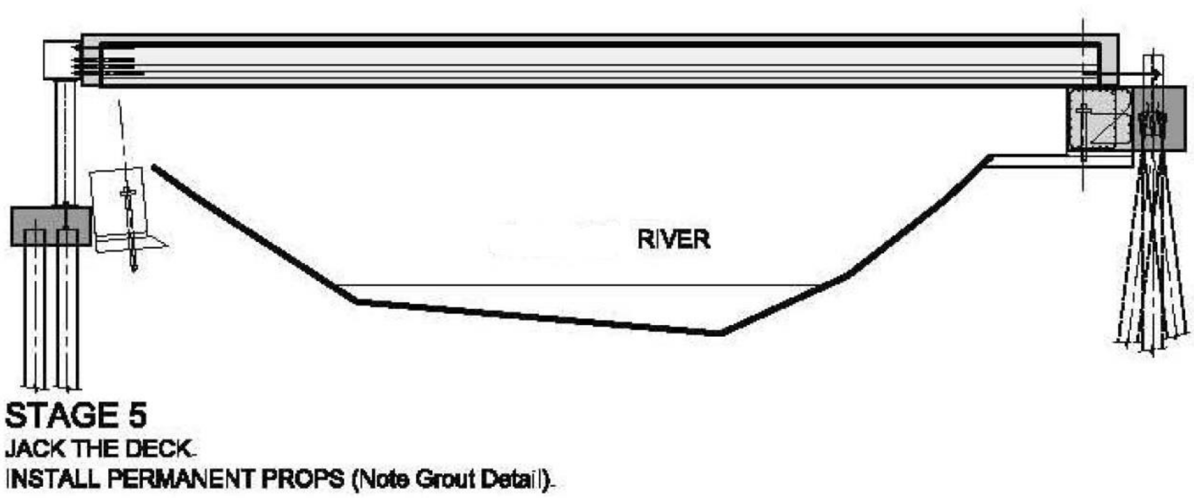


Fig 3.5.28.i. The deck jacked into a horizontal position. (Confidential Source, 2015).

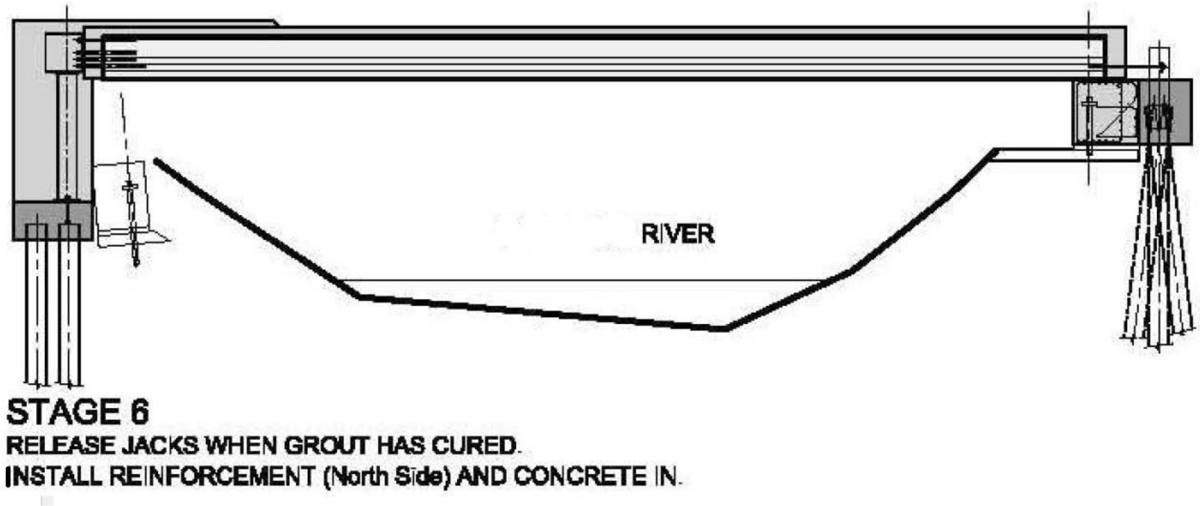


Fig 3.5.28.j Permanent supports installed on the north side. (Confidential Source, 2015).

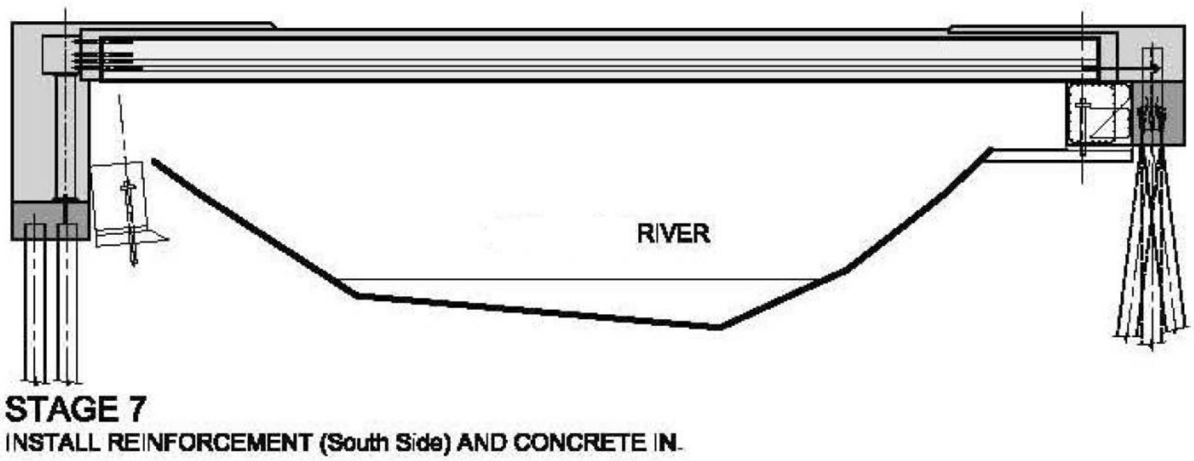


Fig 3.5.28.k. Concrete work completed at the south side. (Confidential Source, 2015).

Underlying Structural Cause

There was an error by the pile testing agency. This is not the first time where reliance on the output from a computer or black box has had disastrous results. The underlying cause of this case is assessed as the contractor’s reliance on computer output.

Potential Severity of Failure

A sudden buckling failure of bridge piles could easily have fatal results. The potential severity of failure is assessed as fatalities involving less than ten persons.

3.5.29 Case study 29 - Augered Pile and Electrical Cable

Background

A large extension to an existing plant required piled foundations and a continuous flight augured solution (CFA) was specified.

The site of the piled area had been declared as free of live services. The piling rig can be used very close to buildings – see Fig 3.5.29.a below.

What Happened

The CFA rig was piling close to the existing building and the auger engaged with a thick electrical cable. The cable was not live but it was still connected to heavy switchgear inside the building. The path of the cable was along a high level pipe rack which ran along a corridor at high level in a zig zag configuration. As the cable straightened severe damage was done to the services on the pipe rack. Some of these were high purity lines which had been fully commissioned and were part of the essential process lines for the facility.

Causes

The site was not cleared of services even though the site had been declared as free of live services.

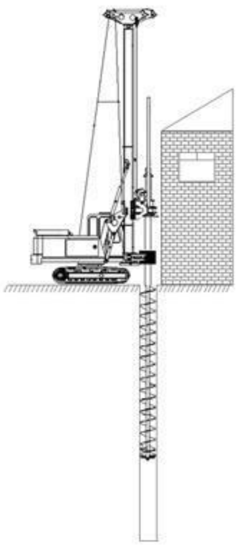


Fig 3.5.29.a. Diagrammatic representation showing rig close to the building.

Discussion

There was some 2 m of the cable outside the existing building. It had been cut by the main contractor to give the piling contractor a site free of services. However the cable was not cut close enough to the building. As can be seen in Fig 3.5.9.a, the rig can go very close to the building. There was sufficient cable for the auger to engage the cable.

There were no reported injuries.

However, the financial cost was very considerable. The major facility was put out of commission for a number of weeks.

Underlying Structural Cause

The contractor should have checked the site for all services – both live and dead.

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc.

Potential Severity of Failure

Although there were no injuries, there was a significant financial loss.

It is easy to see how injuries could easily have resulted from the incident. The assessment of the potential severity of failure is assessed as injuries to less than ten persons.

3.5.30 Case Study 30 - Foundation Piers and Frame Stability

Background

This was a project with a number of structural problems. The worst issue related to a design change in the foundations.

The steel framed building had 2 elevated floors and a grid of 9 m x 7.2 m. The original design had the columns springing off foundation pads which were below ground level. The pads sat on rock at around 2.5 m below ground level. The resident engineer proposed a different arrangement with concrete piers raising the level of the column baseplate to the underside of ground floor slab level. This would have made a level working platform for the vehicles carrying out the steel erecting work.

The modified foundation arrangement was designed in the structural engineer's office and the drawings issued showing the pier.

What Happened

During construction of the frame, it was noticed that the piers were showing signs of serious distress. There was cracking indicating a bursting failure of the pier.

There were around 60 foundations with this pier design.

Causes

The pier was around 500 mm x 500 mm in plan. There was a major error in the calculation of the vertical force in the pier. There was a serious under-design in the links around the vertical reinforcement in the pier.

Discussion

When the problem was properly analysed and the solution agreed, the structural frame was largely complete. The agreed remedial work was, for each column foundation in turn:

- jack up the structure with pre-determined load
- remove the hardcore around each pier
- remove the concrete of the pier
- re-cast the pier using heavier link steel and a permanent steel shuttering - see Fig 3.5.30.a.
- remove the jacks

A completely unrelated problem on the site was that the main building was designed without bracing and the frame had no moment connections!! Clearly this was a potentially more serious structural problem with the potential for a catastrophic failure. The informant for this case concentrated on the foundation problem rather than the potentially more serious issue relating to the lateral stability. This is because the foundation fix was more expensive and therefore more professionally embarrassing than the remedial work for the frame problem.

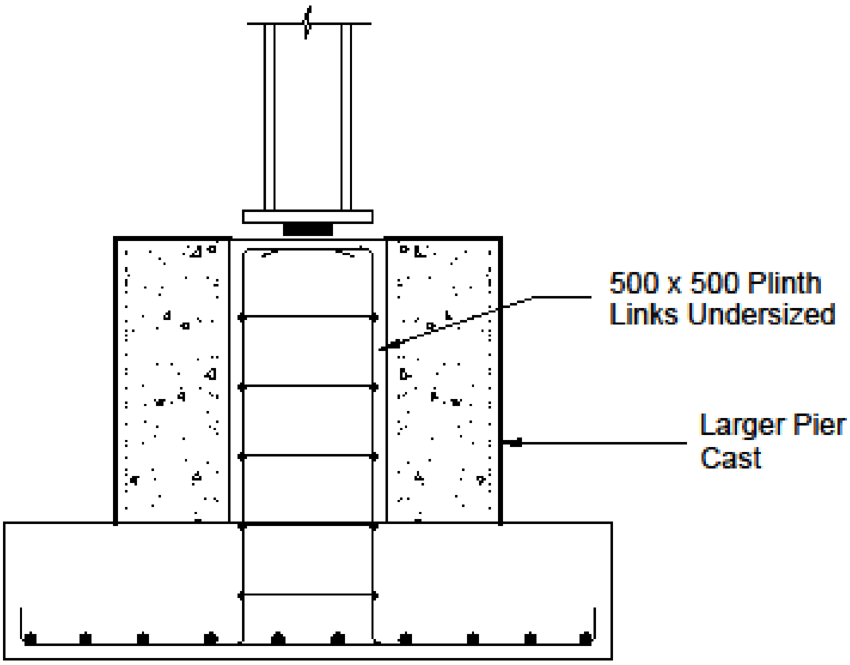


Fig 3.5.30.a. Section showing the foundation detail.

Underlying Structural Cause

Given that the piers were fully restrained on 4 sides by compacted hardcore fill, it is difficult to see a catastrophic collapse mechanism. It is, however, easy to imagine a severe serviceability issue with settling columns and cracking in finishes, etc.

The more serious of the problems – the frame stability – is also a designer error. To have two separate serious structural design errors on one project indicates that the checking of the design was very severely inadequate.

The underlying cause of this case is assessed as the designer's lack of understanding of structural principles.

Potential Severity of Failure

The foundation design error is likely to give rise to a major financial cost. However the issue regarding the lack of lateral stability could easily have resulted in multiple deaths. The assessment of the potential severity of failure is assessed as fatalities involving more than ten persons.

3.5.31 Case Study 31 - Karst Limestone – Sinkhole

Background

Excavation work was proceeding for the large base for a wind turbine. A sinkhole was discovered – refer to Fig 3.5.31.a and b below. No injury to the digger driver or damage to the digger was recorded.

What Happened

Excavation for a large wind turbine exposed limestone bedrock. This excavation revealed sinkholes in the bedrock.

Fig 3.5.31.a. Photograph showing sinkhole during excavation work (Confidential Source, 2015).

Fig 3.5.31.b. View of sinkhole – note water pump (Confidential Source, 2015).

Fig 3.5.31.c below gives an idea of the scale of the base.

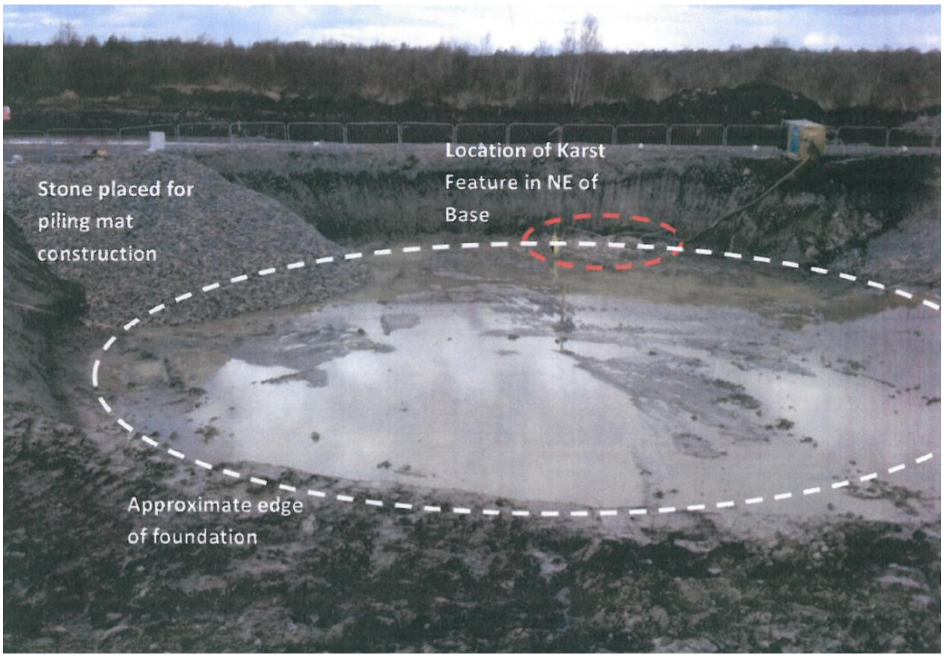


Fig 3.5.31.c. Outline of wind turbine base (Confidential Source, 2015).

Causes

The mechanism for the formation of karst features is well documented. (CIRIA, 2002). The carbonic acid is formed as rain passes through the atmosphere picking up carbon dioxide (CO_2), which dissolves in the water. Once the rain reaches the ground, it may pass through soil that can provide much more CO_2 to form a weak carbonic acid solution. This weak acid dissolves calcium carbonate which is present in certain rocks – most importantly limestone which is widespread in Ireland.

The name Karst derives from the Kras region on the border between Italy and Slovenia where the first research on karst topography was carried out.

Discussion

Approximately 40% of the island of Ireland, and 50% of the Republic, is underlain by limestone.

With very minor exceptions, Irish limestones belong to two periods of geological history, one known as the Carboniferous (around 300 – 340 million years ago) and the other known as the Cretaceous (70 – 120 million years ago). The Carboniferous limestones are normally hard and grey to black in colour, and are found in almost every part of Ireland (every county except Antrim and Wicklow); the Cretaceous limestone (chalk) is somewhat softer and normally white in colour and is found only in Ulster (Counties Antrim, Armagh, Down, Derry and Tyrone).

Given the extent of limestone bedrock, the number of reports of structural engineering problems associated with sinkholes is remarkably low.

Underlying Structural Cause

The contractual arrangement on site between the parties is not known. Given that limestone is present at shallow depths, it is reasonable to assume that the designer should have carried out a full survey before site selection.

The underlying cause of this case is assessed as the designer's lack of understanding of material properties.

Potential Severity of Failure

The foundation would arch over any small void. A large void could result in the failure of a single wind turbine. Given the remote location of wind turbines, a failure is considered unlikely to result in death or injury. The potential severity of failure is given as purely financial - less than ten million euro.

3.5.32 Case Study 32 - Lime Stabilized Ground

Background

A housing development was on raised ground. The raised section was around 5 m over the original ground level in the relevant area. The reason for this is not known but it is unusual and expensive (refer to Fig 3.5.32.a below).

Fig 3.5.32.a. View of retaining wall (note house in the distance). (Confidential Source, 2015).

What Happened

Although there were no issues in the rest of the estate, the end house was seen to have serious cracking problems – see Figs 3.5.32.b and c. The pattern of cracking suggested that the foundation closest to the retaining wall was suffering from significant movement.

Fig 3.5.32.b. Cracks in front and rear elevations. (Confidential Source, 2015).

Note cracks wider at top – hogging failure.

Fig 3.5.32.c. Front elevation – cracking over upper floor window. (Confidential Source, 2015).

Investigations relating to the compaction of the ground behind the retaining wall quickly identified the problem.

The SPT test is a simple site test – the essential details of which are now outlined. A thick-walled sample tube is driven into the ground by a slide hammer of 63.5 kg falling through a distance of 760 mm. The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm up to a depth of 450 mm is recorded. The sum of the number of blows required for the second and third 150 mm of penetration is termed the "standard penetration resistance" or the "N-value". The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae. (CIRIA 1995)

In one area a SPT test was carried out and the sample tube fell around 5 m under its own weight. The N value was therefore zero! This indicates ground which has no structural strength.

Fig 3.5.32.d below shows a cross section with the different underlying soils.

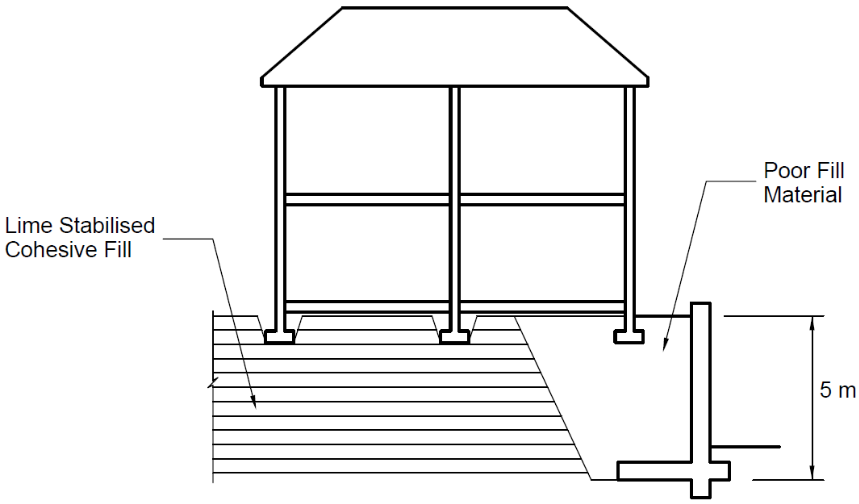


Fig 3.5.32.d. Section through house and retaining wall.

An aerial photo gives a clue regarding a repair to the road – see Fig 3.5.32.e below.

Fig 3.5.32.e. Aerial view. (Confidential Source, 2015).

(The retaining wall runs from the top right corner to bottom centre in a straight line. The house in the case study is closest to the retaining wall. Note the darker section of the road beside the retaining wall – this is the repaired section of road).

Causes

Further geotechnical work over the site showed that the fill material was cohesive material which had been lime stabilized. However, the material behind the retaining wall had not been compacted and lime stabilized.

Discussion

It is not known what happened to the house. The only solution appears to be demolition.

Underlying Structural Cause

Given that the structure was a house, it is very unlikely that a structural engineer was involved in the construction of the dwelling. It would be reasonable to believe that there was expert assistance in relation to the lime stabilized ground. However if the retaining wall was not in place when the lime stabilisation work was underway, then the end of the stabilized ground would be battered down to existing ground level.

The contractor failed to comply with normal practice – to check the load bearing properties of the ground when the foundations were being poured.

The underlying cause of this case is assessed as the contractor's failure to comply with design drawings etc – 30% and the contractor's failure to get structural engineering input – 70%.

Potential Severity of Failure

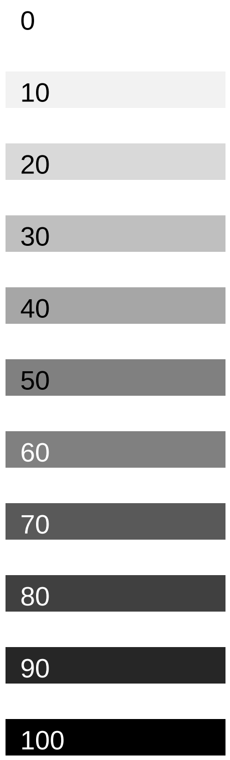
This was only likely to be a financial problem – and at the lower end of the scale. The assessment of the potential severity of failure is assessed as only financial cost - less than ten million euro.

4 Analysis of Results and Discussion

4.1 Analysis of Results

The case studies of 32 individual structural failures or potential failures are outlined in Section 3.5. These cases have specific lessons for the engineering community. This thesis addresses if there are any general lessons and any trends in this sample.

To make a graphical representation where trends could be seen, a series of grey shadings were used where 100% of the blame is indicated as black and 0% of the blame is indicated as white with grey graduations as shown:



This apportionment is now utilised to create four tables which show:

- Main structural material with designer and contractor errors split
- Severity of consequence with designer and contractor errors split
- Main structural material with designer and contractor errors combined
- Severity of consequence with designer and contractor errors combined

These tables are shown on the following pages as Tables 4.1.1, 4.1.2, 4.1.3 and 4.1.4.

The consequence class indicated in Tables 4.1.2 and 4.1.4 refer to the consequences as shown in Section 3.3.

Case No	Case	Designer - Lack of Understanding of Principles	Designer - Lack of Understanding of Loads	Designer - Lack of Understanding of Materials	Designer - Poor Communication to Site	Contractor - Lack of Understanding of Structural Principles	Contractor - Failure to Comply with Design Drawings etc	Contractor - Failure to get Proper Structural Input	Contractor - Other Errors eg Reliance on Computer Output
Steel									
1	Arched Portal Frame	100							
2	Curved Bridge Bearing				80			20	
3	Silo		100						
4	Change of Bolt Specification						100		
5	Sports Stadium Roof				20		80		
6	Shelf Angles	20				20	60		
7	N Truss				100				
8	Wind damage to Sports Centre						100		
9	Portal Rafter End Plate					30	40	30	
10	Steel Beam with Vibration	100							
Concrete Insitu									
11	Transfer Beam		100						
12	Reinforced Concrete Column Joint					70		30	
13	Beam in Flat Slab	100							
Precast									
14	Precast Concrete Stairs			50			50		
15	Semi-Precast Floor					100			
16	Moment into Column		100						
17	Precast Concrete Stadium Steps					20	80		
18	Precast Concrete Corbel					30	70		
Composite									
19	Stud Welding						60	40	
20	Composite Deck Failure						70	30	
21	Composite Beam with Semi Precast Slab					40	30	30	
Masonry									
22	Masonry Wall in Warehouse							100	
23	Masonry Walls	20						80	
Timber									
24	Wind Damage to Double Roof						40	60	
25	Sports Centre Roof	100							
26	Wind Damage to Plant Room						100		
Foundations									
27	Sulfates in the Ground			100					
28	Pile Failure - Bridge								100
29	Augered Pile and Electrical Cable						100		
30	Foundation Pier and Fame Stability	100							
31	Karst Limestone - Wind Turbine			100					
32	House - Lime Stabilized Ground						30	70	

Table 4.1.1 Summary of results - Main structural material with designer and contractor errors split.

Case No	Case	Consequence Class	Designer - Lack of Understanding of Principles	Designer - Lack of Understanding of Loads	Designer - Lack of Understanding of Materials	Designer - Poor Communication to Site	Contractor - Lack of Understanding of Structural Principles	Contractor - Failure to Comply with Design Drawings etc	Contractor - Failure to get Proper Structural Input	Contractor - Other Errors eg Reliance on Computer Output
2	Curved Bridge Bearing	A				80			20	
14	Precast Concrete Stairs	A			50			50		
30	Foundation Pier and Fame Stability	A	100							
6	Shelf Angles	A	20				20	60		
22	Masonry Wall in Warehouse	B							100	
28	Pile Failure - Bridge	B								100
27	Sulfates in the Ground	B			100					
15	Semi-Precast Floor	B					100			
3	Silo	B		100						
21	Composite Beam with Semi Precast Slab	B					40	30	30	
4	Change of Bolt Specification	B						100		
7	N Truss	B				100				
8	Wind damage to Sports Centre	B						100		
5	Sports Stadium Roof	B				20		80		
9	Portal Rafter End Plate	B					30	40	30	
23	Masonry Walls	B		20					80	
18	Precast Concrete Corbel	B					30	70		
20	Composite Deck Failure	D						70	30	
29	Augered Pile and Electrical Cable	D						100		
24	Wind Damage to Double Roof	D						40	60	
11	Transfer Beam	E			100					
16	Moment into Column	E			100					
19	Stud Welding	E						60	40	
13	Beam in Flat Slab	F	100							
31	Karst Limestone - Wind Turbine	F			100					
32	House - Lime Stabilized Ground	F						30	70	
17	Precast Concrete Stadium Steps	F					20	80		
26	Wind Damage to Plant Room	F						100		
1	Arched Portal Frame	F	100							
25	Sports Centre Roof	F	100							
12	Reinforced Concrete Column Joint	F					70		30	
10	Steel Beam with Vibration	F	100							

Table 4.1.2 Summary of results – Severity of consequence with designer and contractor errors split.

Case No	Case	Designer Error	Contractor Error
Steel			
1	Arched Portal Frame	100	
2	Curved Bridge Bearing	80	20
3	Silo	100	
4	Change of Bolt Specification		100
5	Sports Stadium Roof	20	80
6	Shelf Angles	20	80
7	N Truss	100	
8	Wind damage to Sports Centre		100
9	Portal Rafter End Plate		100
10	Steel Beam with Vibration	100	
Concrete Insitu			
11	Transfer Beam	100	
12	Reinforced Concrete Column Joint		100
13	Beam in Flat Slab	100	
Precast			
14	Precast Concrete Stairs	70	30
15	Semi-Precast Floor		100
16	Moment into Column	100	
17	Precast Concrete Stadium Steps		100
18	Precast Concrete Corbel		100
Composite			
19	Stud Welding		100
20	Composite Deck Failure		100
21	Composite Beam with Semi Precast Slab		100
Masonry			
22	Masonry Wall in Warehouse		100
23	Masonry Walls	20	80
Timber			
24	Wind Damage to Double Roof		100
25	Sports Centre Roof	100	
26	Wind Damage to Plant Room		100
Foundations			
27	Sulfates in the Ground	100	
28	Pile Failure - Bridge		100
29	Augered Pile and Electrical Cable		100
30	Foundation Pier and Fame Stability	100	
31	Karst Limestone - Wind Turbine	100	
32	House - Lime Stabilized Ground		100

Table 4.1.3 Summary of results - Main structural material with designer and contractor errors combined.

		Consequence Class	Designer Error	Contractor Error
2	Curved Bridge Bearing	A	80	20
14	Precast Concrete Stairs	A	70	30
30	Foundation Pier and Fame Stability	A	100	
6	Shelf Angles	A	20	80
22	Masonry Wall in Warehouse	B		100
28	Pile Failure - Bridge	B		100
27	Sulfates in the Ground	B	100	
15	Semi-Precast Floor	B		100
3	Silo	B	100	
21	Composite Beam with Semi Precast Slab	B		100
4	Change of Bolt Specification	B		100
7	N Truss	B	100	
8	Wind damage to Sports Centre	B		100
5	Sports Stadium Roof	B	20	80
9	Portal Rafter End Plate	B		100
23	Masonry Walls	B	20	80
18	Precast Concrete Corbel	B		100
20	Composite Deck Failure	D		100
29	Augered Pile and Electrical Cable	D		100
24	Wind Damage to Double Roof	D		100
11	Transfer Beam	E	100	
16	Moment into Column	E	100	
19	Stud Welding	E		100
13	Beam in Flat Slab	F	100	
31	Karst Limestone - Wind Turbine	F	100	
32	House - Lime Stabilized Ground	F		100
17	Precast Concrete Stadium Steps	F		100
26	Wind Damage to Plant Room	F		100
33	Two Adjacent Houses	F		100
1	Arched Portal Frame	F	100	
25	Sports Centre Roof	F	100	
12	Reinforced Concrete Column Joint	F		100
10	Steel Beam with Vibration	F	100	

Table 4.1.4 Summary of results – Severity of consequence with designer and contractor errors combined.

These four Tables above show a graphical representation from which trends might be identified.

4.2 Validity of Methodology

The author has spent his entire career which spans over 40 years as a designer. Although he has worked as a resident engineer and has been involved with site supervision, he has never worked for a contractor. He has worked in design offices mainly in Ireland but has worked for periods in design offices in the UK, the Far East, the Middle East and North America.

Does the author have an in-built bias? If the author is a designer, is he more likely to attribute a failure to a contactor rather than a fellow designer? The author has attempted to be fair in the apportionment of blame but does he have an in-built predisposition?

Similarly, most of the engineers who supplied the material used in the case studies were designers. Almost all of them would be senior structural engineers with many of them partners in large consultancies. Do they show a bias? Are they happier to report the errors of contractors rather than their own errors? In the event of a dispute on site or even if this dispute ends up in an arbitration or legal case, the events described by the consultant and the contractor are rarely the same.

The intimate, informal interview process ensured the maximum feedback but there is an unavoidable risk of bias. It would be an interesting parallel research project to have an experienced contractor undertake a similar study based on interviews with other contractors.

The methodology used in this thesis is outlined in Section 1.4. The methodology used by CROSS (Confidential Reporting on Structural Safety) is given in Section 1.4.2.

The Case Studies discussed in Section 3.5 of this thesis are 32 in number. They are taken from a longer list of cases given by the structural engineers interviewed by the author. The 32 were selected on the basis of having an interesting structural background with a “lessons learnt” which is worthwhile recording. This is discussed in Section 1.4.5.

Almost all of the structural engineers interviewed were well known to the author. It is reasonable to think that these structural engineers were much more likely to confide in someone they know and trust, rather than a person unknown to them. The author’s experience of discussing the case histories suggests that designers were indeed happy and willing to discuss their own shortcomings. There was almost a confessional tone to some of the discussions.

4.3 Discussion of Results

The 32 case studies in Section 3.5 cover all the main structural materials – steel, insitu concrete, precast concrete, composite steel and concrete, masonry and timber. Foundation problems are common and they are listed as a separate category. The underlying causes of the structural failures considered cover a wide range of factors. These causes include the designers’ lack of understanding of loads, lack of understanding of material properties, lack of understanding of structural principles and poor communication to site. Contractor errors were similarly broken down into categories including contractors’ lack of understanding of structural principles, contractor’s failure to comply with the design drawings and the contractors’ failure to get proper structural advice.

CHAPTER 4 – ANALYSIS OF RESULTS AND DISCUSSION

The four Tables 4.1.1 to 4.1.4 are diagrammatic representations of the case studies considered. The function of the shading is to establish if there are any identifiable trends or patterns in responsibility for the errors which took place. The graphical method is believed to have worked well to establish if there are any such trends.

An examination of the four Tables suggests the following:

1. There is no striking pattern in the results where a definite pattern of a failure type could be seen. There is a reasonably wide spread of the shaded boxes in the Tables. This is considered to be a result which is not surprising. The nature of the failures and structural problems are widely varying. If there was a striking pattern then it is likely that this would have been identified in the construction industry. No such dominant tendency is known in the industry.

   Although there is no dominant striking pattern to be seen in the Tables, certain trends can be seen. They are:

2. The errors outlined in the cases, primarily indicate that the error was entirely the designers or entirely the contractors. Contractor error is more likely to be the cause of the failure than a structural designer error. The split in the case studies considered is around to 40% designer to 60% contractor. This is calculated by considering Table 4.1.3. If the percentages for both the designer error column and the contractor error column are added, the totals are 1310 and 1890 respectively. This is equivalent to 40.9 to 59.1 or say 40% to 60%.

   This is regarded as something of a surprise as the author would have expected to see designer error to be more common based on his experience. This is seen as a significant result and should be viewed as significant to those carrying out the Assigned Certifier work under the Building Control Amendment Regulations. The recently created role of the Assigned Certifier puts considerable responsibility on him/her for supervision on site.

3. The three potentially most serious failures were all designer errors. The potential outcomes of these errors were multiple fatalities. The three structures were a large bridge which is curved on plan, a precast stairs in a school and the building built on concrete piles in ground with heavy chemical contamination. There is no pattern in these three cases – they have significantly different underlying causes.

   This is, however, not a large sample and little should possibly be read into this.

4. All three failures relating to composite decks were contractors' errors. These involved faulty installation of studs, overstressing the deck during construction and an unapproved substitution of hollowcore units for a semi-precast floor. Again, the underlying causes are significantly different. This is a small sample but perhaps more careful site supervision of composite decks should take place.

5. It is well known that masonry is constructed all over Ireland with little or no structural engineering input. This is true for the masonry walls given in case 22 and the three incidents outlined in case 23. Contractor error is largely blamed in these instances. These cases include fatal incidents and cases involving injury. All the walls in the cases were subject to

horizontal loads. The author is not aware of any structural failures of masonry walls subject to vertical loads. Site supervisors should be aware of the weakness of masonry walls subjected to horizontal pressures from such sources as wind, or crowd loads.

5 Conclusion

5.1 Summary of Work Done

For this research project a number of senior engineers have been interviewed and their experience of failures and near misses has been recorded. The wider list of case studies has been reduced to the 32 case studies which are discussed in this thesis. The basis for selecting the 32 cases from the larger set of incidents is given in the thesis.

The cases cover a wide range of materials – concrete, steel, timber, masonry etc. The causes of the failures or near misses are very varied. An assessment has been attempted in this thesis to see if there are any common factors in the cases studied.

Clearly there are lessons to be learnt from each of the cases but are there any overall lessons to be learnt from the overall list of failures? An analysis of the cases has been undertaken which seeks to apportion blame for the individual cases between the designer and contractor. There is a further analysis which splits the blame in accordance with this list:

- Designer – lack of understanding of structural principles
- Designer – lack of understanding of loads
- Designer – lack of understanding of materials
- Designer – poor communication to site
- Contractor – lack of understanding of structural principles
- Contractor – failure to comply with design drawings
- Contractor – failure to get proper structural input
- Contractor – other errors eg reliance on computer output

Tables are given which attempt to show in a graphical way the cause of the error on the basis of the material type and the severity of the consequence of the failure.

5.2 Main Conclusions

The causes of failures for the cases studied in different materials are many and varied. Given that the sample cases examined are only 32 in number, is it valid to try and draw conclusions from this sample? The answer to this is probably that conclusions can be reasonably drawn, but they should be treated with some caution.

The main points to be taken from the analysis could the summarised as:

1. No dominant pattern can be seen in the analysis of the cases studied. There were, however, certain trends in the data. These are listed below.
2. Contractor error is more likely to be the cause of the failure than a structural designer error. Although it is not possible to exclude any suggestion of bias, the ratio is assessed to be approximately 60% to 40%.

3. The three potentially most serious failures were all designer error. The potential outcomes of these errors would be multiple fatalities. There is, however, no common theme to these three cases.

4. The three failures relating to composite slabs were contractor error. The underlying cause in the three cases was different and the sample is small but it might be the case that closer site supervision of composite decks would be wise.

5. There are cases in the thesis related to masonry. All the cases involve failures due to horizontal forces rather than vertical loads. One of the cases resulted in two fatalities. All designers and site workers should be aware of the dangers of horizontal loads on masonry.

5.3 Were the Objectives Met?

The objectives of this thesis are given in Section 1.3 and are listed below.

1. To present a series of case studies of structural failures that occurred in Ireland, but where the technical details have not been published.

2. In each case study, to identify and report the principal and secondary causes of each failure.

3. To analyse the multiple case studies to identify any common causes or features which are shared by many failures.

4. To identify whether structural failures are more frequently caused at the design stage or at the construction stage.

5. To critique the methodology used to collect the information the case studies with a view to suggesting improvements that would maximise the willingness of practising engineers to participate in further studies and would minimise the risk of a researcher's bias skewing the results.

All the objectives were met.

A wide range of structural failures are discussed in this thesis. The failures are different with a wide range of causes. The issues involved in each case are analysed and discussed. The causes of the problems are given and blame apportioned between the designer and contractor. Graphical presentations of the results are given.

5.4 Recommendations for Future Work

In the event of a serious accident on site, or a failure of the structure having been completed, a full investigation will be carried out. This investigation will include input from the Health and Safety Authority. However there is no formal or indeed informal process relating to "near miss" incidents. In the UK the CROSS (Confidential Reporting on Structural Safety) is an invaluable tool aimed at the recording of such incidents. The reports from CROSS are widely read by structural engineers in Ireland.

None of the structural engineers interviewed as part of the research for this thesis have indicated that they have reported their case study to CROSS. There is thus a vast amount of knowledge lost to the profession.

Future research could continue the work in this thesis by interviewing a wider body of structural engineers.

A parallel study by an experienced contractor interviewing contractors would produce an interesting comparison with the results of this thesis.

Given that most of the engineers interviewed were from the Greater Dublin region, there exists a significant untapped body of knowledge from outside the capital.

Another suggested topic for future research could be the study of the evolution of building codes and standards. Is it the case that every revision to the building codes was preceded by a structural failure? The fatal collapse of Raglan House in Dublin in 1987 produced the Local Government (Multi-Storey Building Act) of 1988. This Act was only on the statute books for 2 years before the introduction of the Building Control Act of 1990. This new Act covered the disproportionate collapse provisions by reference to the British Structural Codes BS 8110 and BS 5950 (BSI, 1985 and BSI, 1985). In turn these British codes were developed following the Ronan Point collapse. Future Research could investigate the evolution of codes with particular reference to structural failures.

Another possible topic for future research could be an investigation into where practising engineers might foresee future problems. Engineers may have some hunch that something currently regarded as normal practice is likely to produce problems in the future. One possible area for this would be the design and construction of gangnail roof trusses. These are designed by computers and are designed with no additional factors of safety above the code requirements. The timber components are held by light gauge steel plates. These plates have teeth formed by punching the metal plate. These tooth plates have quite short teeth which are pressed into the timber to form structural joints. How effective are they over time? Can long-term changes in the timber properties adversely affect the structural strength?

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