Pipers Row Car Park, Wolverhampton

Quantitative Study of the Causes of the Partial Collapse on 20th March 1997

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Pipers Row Multi-Storey Car Park was built in 1965 and a 120 tonne section of the top floor collapsed during the night of 20th March 1997. Initial reports identified some of the factors which contributed to the initial punching shear failure which developed into a progressive collapse.

This report has quantified the wider range of factors which, in combination, led to the collapse. It covers more detailed investigation and testing of the construction, materials, degradation and repairs to the reinforced concrete Lift Slab structure. The sensitivity of the reactions, moments and shear stress distributions to construction tolerances, cracking and the repair procedures have been evaluated, as have the actual vehicle and thermal load effects. Comparisons of simplified design analysis with more rigorous analysis have been carried out, including ANSYS plate analysis of the whole slab and detailed DIANA non linear analysis of the stress concentrations around the Lift Slab shear head. The partial factors on load and strength have been compared to the variability of load effects and punching shear strength determined to BS8110 and on the basis of research studies. The reasons for the initial failure and its structural development into a progressive collapse have been considered.

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1 SUMMARY

A 120 tonne section of the top floor of Pipers Row Multi-Storey Car Park collapsed at about 3am on 20th March 1997. This report on the detailed quantitative investigation of the failure describes how aspects of design and construction reduced the factors of safety and inadequate inspection and partial repair of developing deterioration led to a punching shear failure at one column which then spread.

The lessons from this study have important implications for the safe design of flat slab structures and for the appraisal, inspection, maintenance and repair of concrete structures of all types. It has highlighted the need to identify in appraisal and inspection the risks from shortcomings of design and construction, and the effects of deterioration, before safety is compromised. It has identified some limitations in the current approach in British Standards to the design and appraisal of structures which are inherently brittle and susceptible to progressive collapse, which become more serious as deterioration develops.

1.1 DESIGN AND CONSTRUCTION

Pipers Row Multi-Storey Car Park was built in 1965 by British Lift Slab Ltd (BLS) using the Lift Slab system of construction, in which concrete floor slabs, cast at ground level, are lifted up precast columns and then supported on wedges engaging in welded angle shear collars cast into the slab. The configuration of the area which failed is shown on Figure 1.3 - 1 and Figure 1.4 - 1.

The design in 1964 was carried out to CP114 whose punching shear clauses give a poor, generally optimistic, prediction of strength. The basic calculations were reasonably carried out to CP114, but the holes adjacent to J2, tolerances on support wedges, and the effects of the detailing of the shear heads were ignored. The setting of the shear head support angle 22mm up into the slab reduced the strength of the slab and the 13mm mortar below provided uncertain fire protection even before premature corrosion spalled it off.

The concrete was highly variable, some with low cement contents, poor mixing and compaction, so that localised areas of concrete were substantially below the specified 20.5N/mm^2 strength. This reduced the shear strength and made the concrete susceptible to premature deterioration. The reinforcement in the slab was set low leading to premature corrosion on the soffit. The high cover to the top steel, somewhat offset by slight oversize on slab thickness, reduced the shear strength.

Concrete density was lower than the design value and surfacing much thinner, which reduced dead loading from the design 6.2kN/m^2 to 5.4kN/m^2. The tolerances on setting the four wedges supporting the slab at each column usually resulted in all the reaction being taken on one wedge. Typically wedges could be 5mm high or low relative to adjacent columns. Combined these misfits could have increased the effective shears around some columns by more than 40%.

1.2 MAINTENANCE AND INSPECTION, 1988 TO 1995

Severe local deterioration to concrete at the bottom of a ramp necessitated a substantial repair in 1987. After a punching shear progressive collapse during demolition of a similar Lift Slab car park at Coventry in 1988, a structural appraisal of Pipers Row for National Car Parks (NCP), the owner, concluded that it had a "*lesser margin of safety against a punching shear failure*" than a 1988 design and that it was important that this margin was "*not reduced by significant deterioration of the materials*". The recommended detailed survey and materials testing of the structure was not carried out.

Signs of concrete deterioration and corrosion induced spalling progressively developed over the decade prior to the collapse and some local patch repairs were carried out. Although further proposals for a detailed structural assessment and investigation of deterioration were made at intervals, no detailed survey and test programme was carried out prior to the collapse.

1.3 INSPECTION AND REPAIR, 1996 TO 1997

In 1996 parts of the deck slab which collapsed were patch repaired and re-waterproofed by Car Deck Maintenance (CDM) to stop leakage. There was an inspection by Harris & Sutherland (H&S), Structural Engineers for NCP, but there was no proper investigation of deterioration, assessment of the structural effects of the deterioration and repair or specification for a structural repair. The repairs at H2 and J2 were in the punching shear zones adjacent to the columns, see Figure 1.3.1. At both locations a substantial area of concrete had become friable to below the level of the top reinforcement which was corroding. If the drawings had been consulted, the sensitivity of the structure to punching shear with the degradation and with the repair to below the top reinforcement, should have been apparent. The repairs were not taken out to the limits of deterioration leaving substantial areas of friable concrete with corroding reinforcement unrepaired around H2. The area of deep degradation and reinforcement corrosion around I2 was not identified.

The repair material had properties well matched to the better concrete, but it was stiffer and stronger than the weak concrete in the areas repaired. It was not well compacted around the reinforcement or well bonded to the substrate concrete, which had not been cut out down to sound concrete. Although some props were used during the repairs it is unclear if they were tightened to carry full dead load while the repair fully hardened.

In January 1997 the slab was checked by CDM because of further leakage. A crack adjacent to J2 was found and considered potentially serious. In February NCP and H&S inspected the crack and local propping at J2 was considered appropriate. A full inspection with materials testing and structural assessment of the whole car park was recommended. The propping, detailed inspection and assessment had not started when the collapse occurred on 20th March, but the area of slab which collapsed and the slab below were coned off.

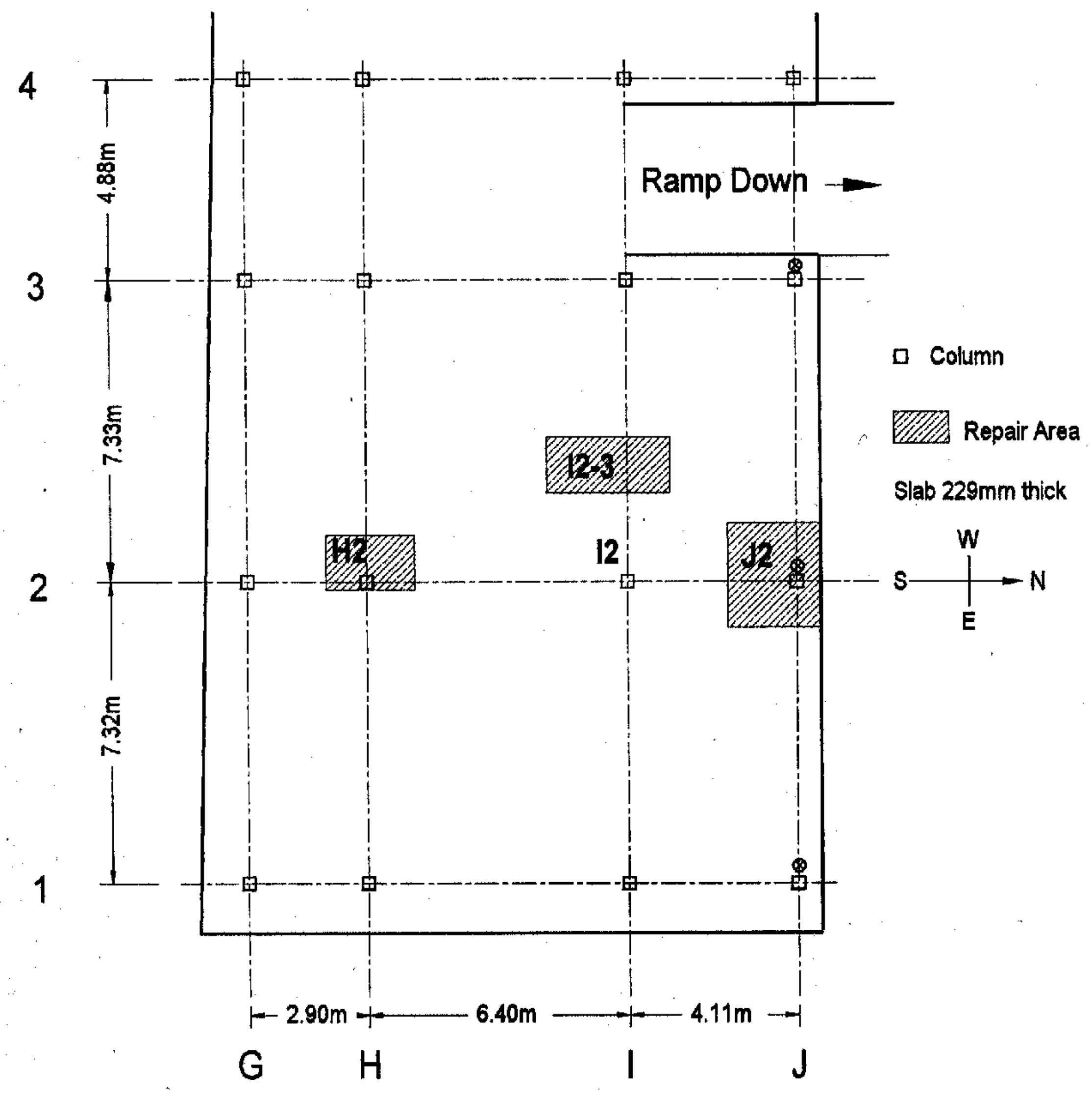

Figure 1.3 - 1 Plan of 4th floor slab at Pipers Row showing repaired areas

1.4 THE COLLAPSE

A 120 tonne, 15m by 15m section of the top floor collapsed at about 3am on 20th March 1997 when punching shear failure at one column led to a progressive collapse as similar failures followed at eight adjacent columns. Fortunately the car park was empty, so there were no injuries, but the closure and subsequent demolition of the whole 400 space car park caused substantial disruption to Wolverhampton.

The initial punching shear failure in the concrete around the shear head could have occurred at H2 or I2, as at both the deterioration and/or repair substantially reduced the anchorage to the top steel essential for shear strength. Because of sensitivity to support wedge construction tolerances, H2 and I2 had similar indeterminate wide ranges of possible effective shear force from self weight at the time of collapse. The Lift Slab shear head and column connections remained intact.

Which ever failed first, the other would have failed shortly after leaving the central area of slab sagging with failures in flexure and increased shears to the edge columns. The sagging slab, acting as a tension membrane, pulled the edges inwards and then split along Line 2. Fortunately the lack of continuity in the slab and the reversal of moments towards G3 and H3 lead to the bottom steel peeling down out of the slab. This, together with the strength of the ramp between I3 and J3, prevented the further horizontal spread as I3 and J3 punched. The edge columns then failed in punching at H1 and I1 and G2 and J2. G1 and J1 with bottom steel welded to the collar were pulled over, punching at J1 but not at G1. The pull of J1 towards I1 indicates that the central H2/I2 failure must have preceded the failure at J2. The folding down of the slab would have reduced the impact onto the undeteriorated 3rd Floor slab below, but it was probably fortuitous that this did not fail, setting off progressive failure down and across the whole structure.

1.5 THE DETERIORATION

The design provided good falls for drainage, but the construction tolerances, deflection and creep of the slab created areas of ponding on the surface.

Much of the surfacing and concrete surface layer of the failed slab was in good condition with minimal carbonation (1 to 2mm). In some areas, including around H2, I2 and J2, waterproofing leakage and ponding saturated the slab and frost damage developed to fissure and microcrack the concrete, particularly in local areas of high porosity and low strength. Salt crystallisation of urea, used for de-icing for the previous decade, possibly also contributed to this. Some indications of ASR and ettringite formation were noted in petrographic examination, but not at a level which would have significantly contributed to the deterioration of the concrete. The friable degraded material was up to about 100mm deep, well below the top reinforcement and a shallow, ~5mm, intermediate layer separated it from the undeteriorated concrete below.

Deep carbonation developed into the microcrack system and zones of high porosity, possibly aggravated by CO_2 released from the breakdown of urea. When microcracking reached the top reinforcement the carbonation, not chlorides, initiated corrosion. Around I2 and around the repaired area at H2 corrosion indicates that degradation, sufficient to destroy most of the bond from steel to concrete, had developed for most of the upper top layer (T1) of reinforcement and some of the lower (T2) layer.

Figure 1.4 - 1 Collapsed 4th floor slab at Pipers Row Car Park, Wolverhampton on 25/3/97

1.6 ANALYSIS OF LOAD EFFECTS

The reactions and moments and resultant effective shear stresses at column locations have been analysed using BS8110 sub-frame, overall grillage and ANSYS plate idealisations. The sub-frame analysis, with reduction in column reactions and moments from the moment redistribution permitted by BS8110, but not BS5400, gives shears 20% less than grillage and plate analysis for inner columns H2 and I2, with 20% more to edge columns like J2. This could lead to a significant underestimate of the maximum load effects in assessment, which can be important for a brittle punching failure mode.

The effective shear forces for a range of load cases, were determined using the elastic ANSYS plate analysis. Table 1.6 - 1 summarises the major cases for H2, I2 and J2, relative to the self weight 'Reference Dead Load' *RefDL* condition with perfect fit and uniform temperature.

Table 1.6 - 1
Load effects: effective shear V_{eff} around columns as % of self weight *RefDL*

	V_{eff} as % of self weight *RefDL*		
Loading, Self Weight +	H2	I2	J2
Self Weight only *RefDL*	100%	100%	100%
Design Live loading (2.40kN/m2)	143%	143%	145%.
Medium vehicles in marked bays and aisles	113%	110%	137%
Differential temperature			
For night of collapse 20/3/97.	102.5%	104.2%	98.6%.
March clear day	102.0%	106.2%	128.5%
4 wedge vertical misfit +5mm	124%	115%	116%
All Load on one wedge +5mm	153%	142%	143%

The partial factor γ_f on load covers, with a good margin, the dead and vehicle loading and temperature effects. However the effects of construction tolerances on setting the slabs on wedges with the Lift Slab system are too large to be safely covered by the load factor and need explicit consideration in design and assessment.

Creep, star cracking over inner columns and the deterioration would have altered the distribution of reactions and moments between the columns and slab tending to reduce the effective shears at I2 and H2 and increase those at J2 and other edge columns. With the limited propping during cutting out and repair there would have been a reduction in effective shear force at H2 and J2 and a small increase at I2 of about 5%. Overall these effects would have only marginally influenced the risk of failure and fall within the margins which are covered by the partial factor γ_f on load effects.

1.7 STRENGTH

Research on punching shear strength since the 1960s has shown the design method in CP114 to be over simplified leading to an overestimate of strength for most structural configurations. It is clearly inappropriate and inconsistent with IStructE recommendations Appraisal of Existing Strucures, to use CP114 rather than BS8110 for appraising older structures. BS8110:1985 (current prior to the collapse) and BS8110:1997, incorporated a substantially improved method for punching shear design in normal in-situ flat slab construction with regular spans, but give little guidance on structures departing from this form.

At Pipers Row:

- irregular spans from lines G to J,
- uneven spacing of top reinforcement near the column,
- flexibility of the long shear head angles at H2 and I2
- angles supporting the slab 22mm above the soffit,
- higher shear stresses arising on the weaker face of the shear perimeter
- stress concentration from the wedge supported shear head
- holes at J2

led to substantial ambiguities in interpreting BS8110 for assessment. This has been demonstrated by the wide scatter of estimates, generally overestimates, of strength at H2, I2 and J2 by engineers adopting different assumptions.

A review of the research literature and the detailed analyses carried out have led to specific recommendations for adapting BS8110 for the appraisal of irregular flat slab structures and those with Lift Slab or similar shear heads. A degree of uncertainty remains about the effect of the stress concentrations from the shear heads unevenly supported on wedges which can only be resolved by a series of tests.

These recommendations have been used to calculate the shear strengths of the as-designed and as-built slab at H2, I2 and J2 and to examine their sensitivity to low strength concrete, variation in the thickness of the slab and the depth of the top steel before deterioration.

The BS8110 partial factor on shear strength of γ_m = 1.25 is too low to cover the loss of shear strength from permitted variations in slab thickness and cover depth, other variability influencing strength and the risk of errors in estimates of strength even before deterioration develops.

As degradation of the surface of the slab develops there is no loss of shear strength until the bond anchoring the top steel around the shear perimeter is reduced. As the depth of the friable layer or the splitting of cover by reinforcement corrosion, relaxes the bond to reinforcement around the shear perimeter the shear strength drops. At Pipers Row the degradation and some corrosion had developed along much of the length of the top steel at H2 and I2. The cutting out for repair weakened the structure and with only limited propping of the slab and the repair poorly bonded to friable substrate the as-built strength would not have been restored.

The clearly evident progressive deepening of degradation was the dominant factor in weakening the structure to the point at which it collapsed under self weight with a marginal increase in shear from the nocturnal cycle of temperature differential.

The partial factors in BS8110 contain no allowance to cover the early stages of strength loss from deterioration. Once surface deterioration starts to reduce the effective anchorage of top reinforcement around a punching shear perimeter, a structure must be regarded as at risk. It must be assessed for its as-built and for its deteriorated strength. Repair procedures must also be rigorously reviewed including the damage from cutting out, the effectiveness of propping, the risks of repair debonding and the need for through bolting to enhance punching strength.

The development of progressive collapse from the brittle initial punching shear failure was a particularly worrying feature. It has parallels in a range of other flat slab progressive collapse failures, some of which have been catastrophic eg 500 killed when the Sampoong store collapsed. It is not clear why BS8110 does not insist on detailing of bottom steel through columns to ensure a ductile punching failure which will not spread. Without ductility a much increased factors of safety is needed. The 'generally applicable' safeguards against progressive collapse in BS8110 are not sufficient for flat slabs and can result in an increased risk of an initial punching failure dragging down more of a flat slab structure.

1.8 MAIN RECOMMENDATIONS

To reduce risks of unforeseen structural collapse, similar to that at Pipers Row, the following recommendations are made.

Inspection

The owner of a structure should ensure continuity of engineering responsibility for its safety, with formal transfer of responsibility and all drawings and records of construction, inspection and maintenance when changes in engineering personal are made.

A structure specific inspection regime should be developed based on a structural reassessment and full inspection including identifying:

- patterns of evident and potential deterioration, ponding, concentration of deicer, etc.,
- the sensitivity of structural elements to deterioration,
- specific procedures for checking easily inspectable and difficult to inspect elements
- areas needing specific surveys and testing.

An inspection and maintenance schedule should then define the long term frequency of and method of inspection for each type of element, which should be regularly reviewed on the basis of survey results.

Surveys and testing

Where inspection identifies indications of poor as-built quality or developing deterioration, surveys and testing must be specifically related to the assessment of strength loss and rate of future deterioration. The current practice of carrying out corrosion check surveys in isolation may miss localised weak areas

In determining the current concrete strength, sufficient core strength data to establish the variability of strength must be supplemented by specific coring into areas showing indications of poor quality construction and/or premature deterioration.

Some structural forms and details where deterioration effects are not superficially inspectable may require the development of special inspection procedures.

Where drawings are not available for potentially sensitive elements, sufficient investigation to establish reinforcement details must be undertaken.

BS8110 design

BS8110 design procedures for flat slab punching shear design require detailed review and revision to achieve probabilities of failure against punching shear and progressive collapse comparable to those for other structural elements.

The effective shear V_{eff} on the perimeter can be seriously underestimated by some of the wide range of permitted analysis methods, as they do not establish an upper bound appropriate to a brittle failure mode. The nominal shear stress on the perimeter is oversimplified and can similarly underestimate shear stresses.

The very simplified load combinations and partial factors in BS8110 on load and strength need rigorous review, including consideration of the allowance for actual construction tolerances and the effects of the initial stages of deterioration.

The generalised robustness requirements need review, as for some types of structure including flat slabs without adequate over-column bottom steel, they do not ensure that progressive collapse will not develop.

Assessment concrete structures

Assessment of strength cannot be undertaken without detailed as-built reinforcement drawings marked up with modifications and details of previous repairs.

When old structures are checked, significant changes in known risk identified in research and/or reflected in changes in standards since the time of design, as well as loss of strength from deteriration, must be considered. Current design codes provide a starting point for assessment of as-built strength, but where structural configurations or details fall outside their scope the implications and appropriate interpretation of the code must be derived from first principles. Better guidance on this is required.

Detailed information from testing and assessment may permit reductions in partial factors for some structural elements. For others which are more sensitive to construction tolerances, do not comply to current detailing requirements, where the effects of deterioration are uncertain or where consequences of failure are more serious than for the as-designed ductile in-situ construction assumed in BS8110, higher partial factors may be required.

Structural elements with brittle failure modes, or which can deteriorate so that failure becomes brittle, must be assessed for the upper bound load effects without the assumption of redistribution of reactions and moments allowed for ductile elements.

The risk of progressive collapse should be explicitly considered by establishing the effects of redistribution after the failure of individual elements as the generalised robustness clauses in BS8110 are not adequate for some structural forms.

For lift slab and other structures similarly sensitive to construction misfit, the construction tolerances on setting on wedges or supports must be explicitly considered in all load combinations.

There is very limited guidance or research data on the effect of various forms of deterioration on the strength of concrete structures. To safely and economically sustain an ageing infrastructure both research and the development of detailed guidance need to be given priority.

Repairs

Procedures for effective structural repair of concrete are available and are well summarised in European Standard Draft ENV 1504 Part 9 Principles.

Once deterioration, corrosion or cutting out exposes reinforcement, it must be considered as a potential structural problem requiring a check by a structural engineer to establish if a structural repair, as distinct from a corrosion control, infill or cosmetic repair, is required.

For structural repairs a detailed method statement with reinforcement drawings marked up with limits of cutting out and support requirements should be prepared by a structural engineer.

Punching shear anchorage zones are particularly difficult to reliably patch repair, so full depth recasting or through bolted repairs are likely to be required once effective anchorage has been damaged.

Other

The effects of carbonation and salt crystallisation from Urea deicer on the deterioration of low strength concretes should be researched.

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3. INTRODUCTION

3.1 BACKGROUND

Pipers Row Multi-Storey Car Park, Wolverhampton, was built using the Lift Slab system in 1965. At about 3am on March 20th 1997, a 15m by 15m section of the top floor slab, weighing about 120 tonnes, collapsed onto the slab below. The car park was fortunately empty, so there were no injuries. The closure and subsequent demolition of the whole car park caused substantial disruption in Wolverhampton.

Shortly after the collapse, Structural Studies & Design (SS&D) were instructed by the Health & Safety Executive (HSE) to inspect the structure with HSE engineers and to work with them in investigating the causes of the collapse. National Car Parks (NCP), the owners of the car park, instructed Harris & Sutherland (H&S) to investigate the collapse for them and to co-operate with the parallel HSE investigation. Sandbergs carried out a materials investigation and testing for NCP working with H&S.

A series of initial reports were prepared on the developing investigations by SS&D [H264 - 372] for HSE and by H&S [H2 - 9] [1] and Sandberg [H1] for NCP. The source data for these reports and their final versions have been utilised in drawing up this report. These reports identified a wide range of factors relating to the original design, construction, materials, degradation and repairs which might have contributed to a greater or lesser extent to the initiation of the failure and its development into a progressive collapse. However the relative importance of these factors and the location of the initiation of the failure was not clear. The punching failure could have initiated at:

- H2 an inner column at which the deteriorated slab had been partially repaired in 1996
- I2 an inner column where the slab had seriously deteriorated, but had not been repaired.

or J2 an edge column which had been repaired in 1996, had developed a crack and was about to be propped at the time of collapse.

Information from the initial investigations has been disseminated in an HSE press release [2] and covered in papers to a number of specialist conferences [3, 4, 5] on car park design and maintenance. The Standing Committee on Structural Safety (SCOSS) highlighted the risks from deterioration in car park structures both before [6, 7] and after [8] the Pipers Row collapse.

SCOSS and industry concerns about the wide range of problems developing from premature deterioration in car parks has led to:

- The Institution of Structural Engineers producing a 3rd edition of the 'Design recommendations for multi-storey and underground car parks', published June 2002 [9]

- The Institution of Civil Engineers producing 'Recommendations for the inspection, maintenance and management of car park structures' published 2002 [10]

- The Office of the Deputy Prime Minister producing 'Enhancing the Whole Life Structural Performance of Multi-Storey Car Parks' published September 2002 [12].

There was particular concern that Pipers Row Car Park, in which deterioration was clearly evident, should have failed locally and then developed into a progressive collapse, without the risks being apparent to those responsible for the structure. HSE commissioned the further studies covered by this report to fully identify and quantify those factors which contributed to the initial failure and to its spread. This should enable guidance to be developed for the improved design, construction, appraisal, inspection, maintenance and repair to enhance the safety of the concrete structures, particularly where deterioration is developing.

3.2 HSE Contracts for the Quantification Study

In 1999 HSE placed Contracts for the Quantitative Study of the Causes of the Partial Collapse Pipers Row Car Park, Wolverhampton as follows:

Contract A — with Structural Studies & Design Ltd (SS&D), HSE RSU Agreement 3976/S33.070, for overall contract coordination and the preparation of this report.

Contract B — (Drawings and code analysis and assessment) with Amey Vectra Ltd (AV) comprising:

Contract B1. — CAD drawings of the 'as- designed' structure for the area of the collapse.
Contract B2. — Design calculations and analysis to CP114, BS8110 using sub-frame analysis and BS8110/CIRIA 110 grillage analysis and comparisons with BS5400.
Contract B3. — CAD drawings of the 'as-built' and collapsed structure including recording retained slabs.
Contract B4. — Overall ANSYS plate analysis of 4^{th} floor slab.

Contract C — (Shear strength) with Building Research Establishment (BRE) comprising:

Local DIANA non-linear finite element analysis of H2/I2 column, shear head and slab stress distribution including effect of repair.
Research based estimates of punching shear strength 'as-designed'
Research based estimates of '1997' Strength with deterioration and repair.

Contract D1 — (Testing concrete samples) with Building Research Establishment (BRE) for:

Sectioning the retained slabs and columns and obtaining samples for testing
Strength and stiffness testing of concrete samples
Petrographic and chemical tests on concrete composition, deterioration and reinforcement corrosion.

Contract D2 — (Testing repair samples) with the Royal Military College of Science at Shrivenham (RMCS) which covered:

Specialist testing of repair materials to CIRIA TN141 with matched concrete samples

Contract E — (Differential temperature) with Building Research Establishment (BRE) for:

Measurement of temperature profiles in an exposed retained slab.
Collection of associated meteorological data.
Comparison of measurements with computer predictions.

This report brings together and summarises the information from these contracts and other sources to identify and quantify the causes of the initial failure and the development of progressive collapse.

3.3 Other Information Sources

HSE have obtained and collated a comprehensive set of documents relating to the design, construction and maintenance of Pipers Row Car Park. Some of these are confidential. Those documents which provide essential information to support this report are referenced using the HSE document numbers prefixed with H thus [Hnn].

The original British Lift Slab Drawings [H144 - 189] and calculations [H190 - 201] are available and these, with information from former British Lift Slab/RM Douglas staff summarised in Appendix 1, have provided information on the design, construction method, intended specification for construction and site practice.

HSE have obtained and collated NCP records on the maintenance and inspection of the car park and repair work tendered for and/or carried out from 1988 to 1995 together with the documents and some photos covering the re-waterproofing and repairs in the area where the partial collapse occurred in 1996 and the identification of the crack at J2 and subsequent actions in 1997.

Immediately after the collapse in March 1997, the failed area of structure was inspected in detail and photographed by H&S, HSE and SS&D. Much of this information is contained in the H&S Report of 22/5/97 to NCP [H2] and the HSE record sets of photographs [H10 - 119] and SS&D initial reports [H204 - 207] and photographs [H203]. Summary lists of all photographs are listed in Appendix 2. HSE, SS&D and H&S discussed and agreed on the sections of columns and slabs to be retained. Most of these were extracted prior to the structure being demolished, but some were too fragile for removal.

Following the collapse NCP instructed Sandbergs to carry out a materials investigation along lines agreed by HSE, H&S and SS&D. They sampled from the remaining structure and the retained slabs for analysis, petrography, and strength. This is reported in Sandbergs Final Report to H&S of 22.7.97 [H1]. Sandbergs retained the sampled material from their testing programme at their Clapham Laboratory, and it was moved to BRE Watford for detailed examination for this study. The full set of Sandberg photographs, see appendix 2 taken in 1997 were made available and they checked their detailed records to clarify particular points.

The retained slabs and columns were kept under cover in NCP Horse Fair car park in Birmingham from shortly after the collapse until October 1999 when they were measured up and then moved to BRE Cardington for further detailed examination and sampling for this report.

Extensive research literature on punching shear behaviour and reports on similar failures have been identified and reviewed by BRE and SS&D and particular assistance on this has been provided by Prof. Paul Regan. Research and literature reviews by SS&D on other structural and materials topics which arose during the investigation have been assisted by a number of specialists in the construction industry, research organisations and Universities.

4. DESIGN AND CONSTRUCTION OF CAR PARK, 1964 - 65

4.1 LIFT SLAB CONSTRUCTION

Pipers Row Car Park in Wolverhampton was designed and constructed by British Lift Slab Ltd (BLS). The floor slabs were lifted in May and June 1965, suggesting the casting of the floor slabs in March to April 1965.

The 'Lift Slab' system [13] was widely used in the UK in 1960s and since, for the rapid cost effective construction of a range of flat slab structures including many multi-storey car parks. It was originally developed in America and has been used in many countries. The detailed design has evolved over the years with changes in design requirements and refinements in the construction procedure.

The basic 'Lift Slab' system is based on:

- Constructing foundation blocks with sockets for precast columns,
- Preparing the site to the required profile of the soffit of the floor slabs to act as a bottom shutter,
- The erection of precast columns containing wedge seatings at floor slab levels,
- The casting of the large sections of floor slabs at ground level, one on top of another, with cast in sliding steel shear heads around the columns to support the slab,
- Lifting the slabs up the columns to the level of the seatings and setting them on four wedges engaging into the steel shear head,
- Infilling with mortar between the shear head and column to create a moment connection between column and slab,
- Casting infill sections between the large sections of floor slab to join up the whole structure.

Appendix 1, prepared following discussions with retired BLS personnel, more fully describes the details of the Pipers Row construction.

4.2 DRAWINGS

A full set of the original BLS drawings T130 [H144 – H189] for the whole structure are available. Amey Vectra extracted the data from the original drawings to produce CAD drawings of the geometry and reinforcement of the 4th floor slab for the area bounded by the edge of the structure adjacent to grid lines G and J and 1 and 4, encompassing the collapsed area. These as-designed drawings provide the reference for the analyses and as-designed appraisals of the structure by AV and BRE.

The particular features of the shear head design at H2, I2 and J2 and their relationship to the reinforcement are shown on Figures 4.2 - 1 to - 3. At H2 and I2 the long projecting angles of the shear head 'collar' resulted in the displacement of the top reinforcement T1 bars and T2 bars more widely, away from the column. At all locations the underside of the angle was ½" (12mm) above the soffit with the slab supported on the upper surface 22mm above the soffit. The ½" below the angle was filled with mortar to provide fire protection. The support wedges engaged into the short angle and can be seen in Figure 8.2.7 - 1 of the sectioned I2 column head. At J2 the shear head lacks the projecting angles and the irregular disposition of reinforcement and the holes for lifting and drainage complicated the shear behaviour.

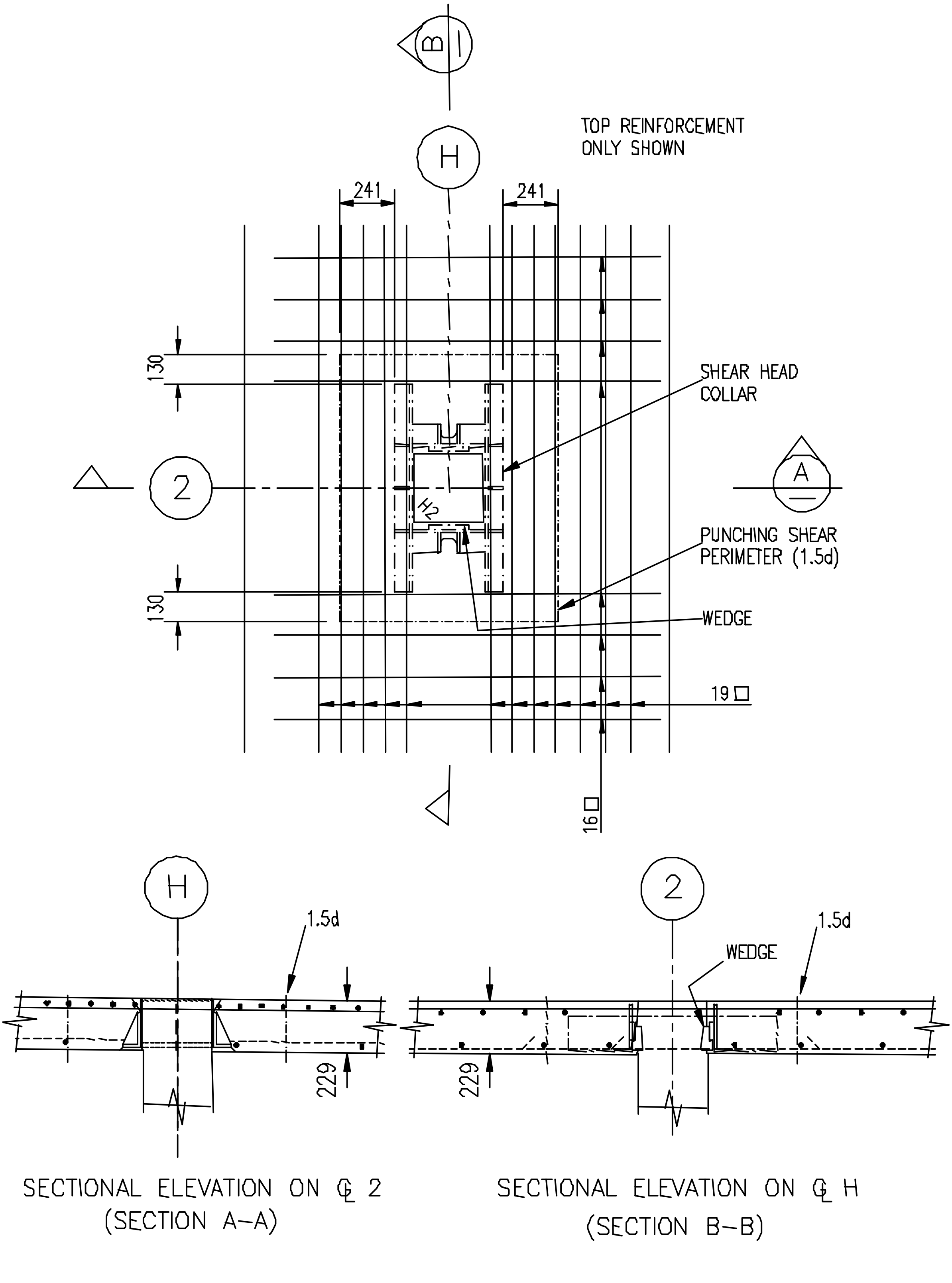


Figure 4.2 – 1 Reinforcement and shear head 'collar' at H2

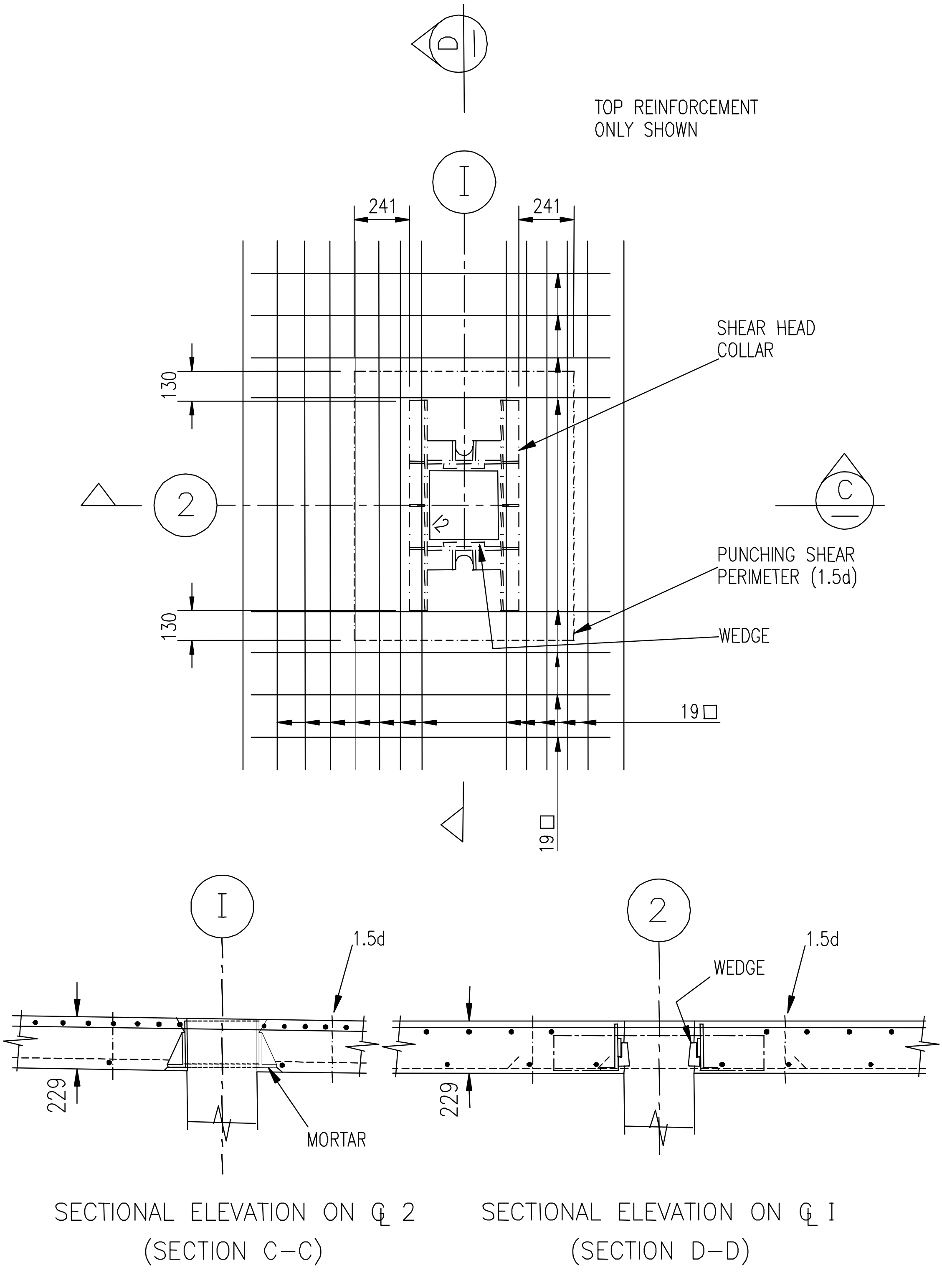


Figure 4.2 - 2 Reinforcement and shear head 'collar' at I2.

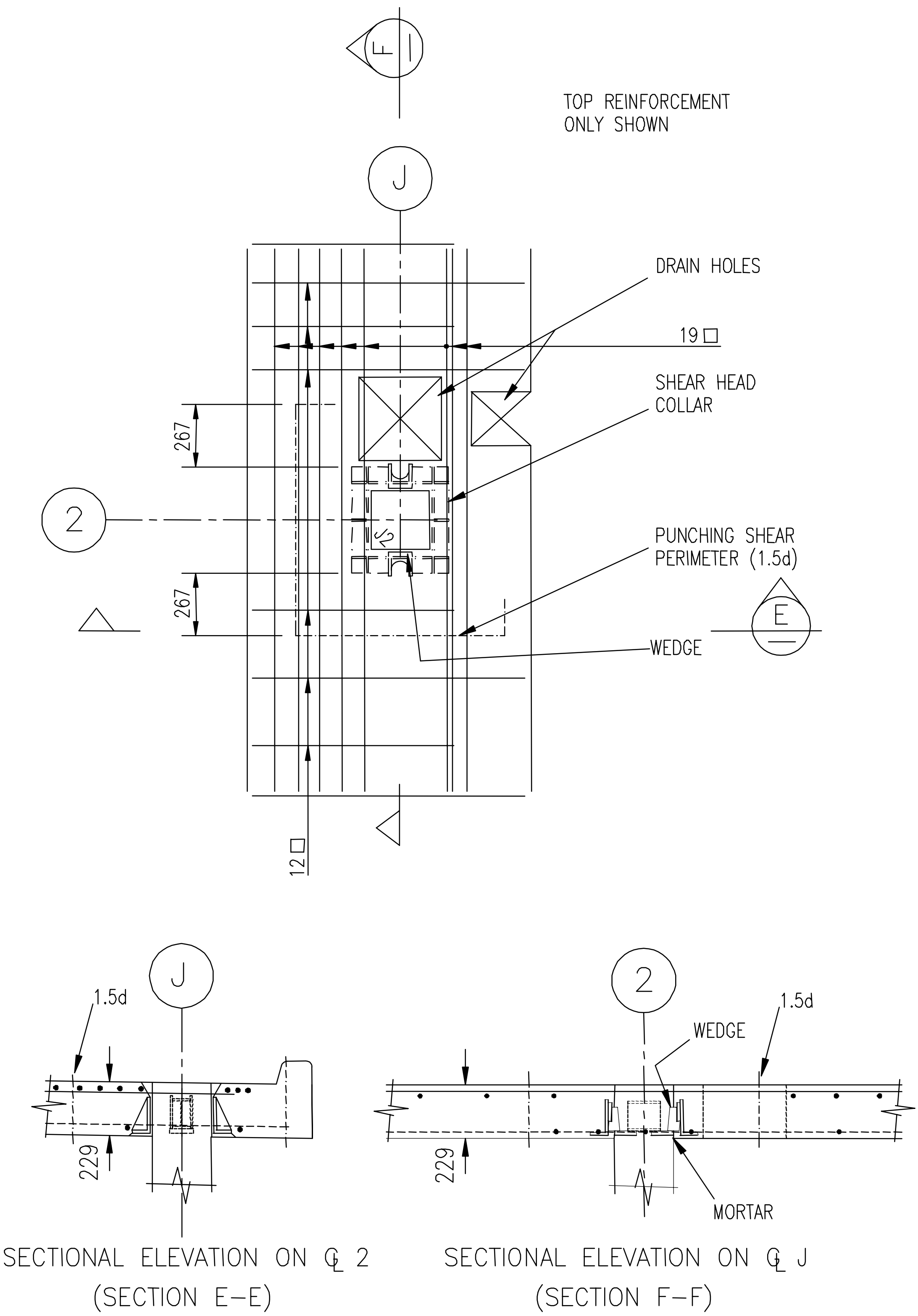


Figure 4.2 - 3 Reinforcement and shear head ‘collar’ at J2.

4.3 DESIGN

The original BLS design calculations are available [H190 – H201]. The basic design robustly followed the requirements of CP114 [14] and BS449 [15] with loading to CP3 [16]. In accordance with CP114 the punching shear stress was checked for the working load column reaction divided by a nominal shear perimeter area, with no consideration of the moment transferred to the column. The calculations show no explicit consideration of the effects of variation in support from the tolerances on setting wedge levels, the detailing of the shear head or of the holes near edge columns like J2.

The permissible shear stress was 100lb/in^2 (0.69N/mm^2) at working loads for a specified 1:2:4 mix 3000lb/in^2 (20.7N/mm^2) 28day works cube strength. The 'roof' top slab design loading for columns [H10] was:

9" slab	113lb/ft^2	(5.41kN/m^2)
'Roofing'	17lb/ft^2	(0.81kN/m^2)
Live load	50lb/ft^2	(2.40kN/m^2)
Total	180lb/ft^2	(8.62kN/m^2)

4.4 CONSTRUCTION MATERIALS

CP 114 durability recommendations for the basic 1:2:4 mix specified were based on a specified cover in Clause 307, which was ½" (13mm) for slab reinforcement but 1½" (38mm) for external work, unless protected by a coating. In Section 7 Maintenance CP 114 recommends *"A periodic check should be made (e.g. every 3 or 5 years) to detect any excessive cracking or other defect of the concrete."*

The specified concrete was a 3000 lb/in^2 mix (20.5 N/mm^2). The available reports, Appendix 1, indicate that this concrete was supplied by a ready mix contractor to meet the cube test requirements. There are reports of difficulties in obtaining cement at that time due to strikes and that this, combined with the cost advantages, led the ready mix concrete suppliers to minimise the cement content consistent with just achieving the cube requirements. The reinforcing steel was predominately imperial sizes of square twisted (SQT) high yield bars to BS1114, with a few plain round mild steel bars to BS785.

There were reports [H2, App. D] of some clay contaminated aggregate supplies, but the records suggest this material was not used in the construction. There are also reports, Appendix 1, that calcium chloride was used as an accelerator in some pours of concrete during the winter months. In the detailed testing of materials these reports have been checked to establish actual characteristics of the materials in the failed slab.

5. DEVELOPMENT OF DETERIORATION AND REPAIRS 1987 TO 1995

Since the mid 1980s there have been a number of reports to NCP on the condition of the structure when work was tendered for and/or carried out. Overall these reports show growing evidence of areas of substandard concrete and premature deterioration which were never properly investigated or remedied. NCP had no formal arrangements for the regular inspection of the structure for deterioration.

There were a range of types of deterioration reported to NCP. Some are of a form not directly related to the failure, eg star cracking of the slab around columns, flexural cracking of the soffit, leakage through the slab, spalling of the soffit concrete due to the corrosion of reinforcement and spalling of the mortar below corroding shear head angles.

There were also reports of local areas of water proofing breakdown and ponding on the top slab leading to potholing and larger areas of breakdown of the surface layer of concrete exposing the top reinforcement. This was clearly indicative of the developing deterioration in the slab around columns which could substantially reduce punching shear strength and which led to the failure in 1997.

5.1 1987 REPAIRS.

In 1987 NCP instructed Peel and Fowler, Consulting Engineers, to investigate and have necessary repairs carried out to an area of deterioration to the slab at the foot of the ramp at I5-6 between top deck levels 8 and 9, outside the area of collapse. Peel and Fowler's invoice dated 5.2.87 indicates that they:

inspected the damage,
assessed its effects, on the basis of the original drawings which were obtained,
specified the repair work,
guided the repair contractor (Tygon) *'in respect of the process of repair, on the requirements for temporary works'*
and made a number of site visits.

These items indicate that Peel and Fowler had a comprehensive brief and followed industry good practice in relation to the main items of work, although full details are not available. It was noted at the time of this repair that some of the original concrete was of exceptionally poor quality, but the wider implications of this for the other parts of the structure were not followed up.

5.2 REACTION TO COVENTRY LIFT SLAB CAR PARK COLLAPSE IN 1988

In 1988 an extensive progressive collapse developed unexpectedly during the demolition of a car park in Coventry of similar design. The rest of the demolition was achieved by initiating further progressive collapses. Following this Douglas Specialist Contractors (DSC), who were the successors to British Lift Slab (BLS), who built the car park, wrote to all owners of similar car parks about the sensitivity of this type of structure. However the emphasis of this letter was on hazards during demolition. It does not refer to the risk of brittle punching shear failure and progressive collapse or to the possible weakness of the 1960s CP114 column slab connection.

Wolverhampton BC wrote on 25.11.88 to National Car Parks enquiring about the safety of Pipers Row Car Park in the light of the Coventry collapse and took some photographs of the structure [H203]. DSC in a follow up letter on 6.12.88 reassuring NCP made clear that safety depended on *"due maintenance ... on a regular basis to both the structure and fabric of the building."*

After consultation with Douglas Specialist Contractors (DSC), NCP asked Ove Arup on 14.12.89 to inspect the car park and make a preliminary assessment of its safety. This followed DSC advice that Arups had carried out detailed studies on other similar car parks and the collapse at Coventry. Some specific testing on the punching shear strength of Lift Slab column slab connections were carried out at that time. It is not clear if the results of these tests by Arup for DSC were made known to NCP or more widely.

Arup's initial advice on 22.12.89 was that the structure did not need "*immediate emergency measures*", but its condition merited a more detailed study of the design, construction and condition. On the basis of a check on the drawings and initial calculations Arups expressed the opinion in 10.2.89 that Pipers Row had a "*lesser margin of safety against a punching shear failure*" than a 1988 design would have had, and that it was important that this margin was "*not reduced by significant deterioration of the materials*". They made detailed recommendations for an investigation of the condition and further assessment of strength.

Ove Arup were then further instructed by NCP on 17.2.89 to carry out the first stage of a more detailed assessment of strength and to carry out, using a sub-contractor, an investigation of the condition of the car park. It is not clear if any work was done on this after the letter of 10.2.89, but there is no record of Ove Arup submitting reports on or accounts for this.

In 1989 the top deck of the car park was resurfaced with a waterproof layer by Fibre-dec. It is not clear what checks on the condition of the top slab of the car park were carried out at that time. There are no details of any repairs which Fibre-dec might have carried out in preparing the surface for waterproofing.

Gallifords quotation of 31.3.89 for *"minimum required at the present time."* gives the areas of deterioration to be repaired as:

> *"up to 50mm deep 15m^2*
> *up to 50mm deep 100 patches up to 300mm x 300mm*
> *up to 50mm deep 86m to 150mm wide at cracks.*
> *up to 100mm deep 1m^2*
> *up to 25mm deep 210m^2 "*

They added that *"It is quite apparent that considerably more work could be carried out to improve the floor surfacing, ...*"

Ove Arup were instructed by NCP on 10.4.89 to check repairs quoted for by Galliford on 31.3.89. However NCP did not instruct Galliford to proceed with this work and Arup do not appear to have had any involvement with Pipers Row after that letter.

5.3 FURTHER QUOTATIONS AND REPAIRS 1990 TO 1995

There does not appear to be a detailed record kept by NCP of the location, extent of deterioration and method of repair of the waterproofing and the concrete for Pipers Row. There is no copy of any specific instructions to the NCP Regional Building Surveyor, or to the local operational management, to inspect Pipers Row regularly to identify signs of deterioration either generally or particularly in the slab around column heads, and to refer it for proper investigation and repair.

In May 1991 NCP received a quotation from Groundpatch for $51m^2$ of repairs to the slab top surfaces for £2,984 and a separate quotation for soffit and edge beam repairs for £7,418. NCP placed this order on 12.6.91 and the work was carried out. The exact locations and depths of this repair work are not clear. There is no indication of an investigation of the reasons for and the extent of the deterioration, of any structural consideration of these repairs or of any engineering input to the specification.

It appears that in 1991 NCP were considering a major extension and refurbishment of Pipers Row and instructed Mason Revis their QS to obtain the quotation dated 30.9.91 from Douglas Special Contracts. DSC clearly carefully checked the condition of the slabs in preparing this quotation, including some carbonation and chloride checks. The estimate for refurbishment amounted to £320,000. An item in this quotation is *"Providing concrete capitals at each column/slab connection."* This apparently referred to improved fire protection or replacement of spalled mortar under shear head angles. No action was taken on the DSC quotation.

6. INSPECTION AND REPAIR, 1996 TO 1997

6.1 REPAIRS AND RE-WATERPROOFING 1996

In January 1996 the local NCP management reported to the NCP Regional Building Surveyor, that damage had developed after a period of frost and that ponding was leading to leakage and lime damage to cars. The lime damage risk lead to the closure of some parking spaces below the top slab. An area of 350m^2 was recommended for resurfacing.

NCP requested CDM on 12.1.96 to visit the car park and advise on the problems of leakage and to quote for waterproofing and associated patch repair works. CDM, after inspecting the car park and discussing the work with NCP by phone, wrote to NCP on 24.1.96 stating that:

> *"The damage to the concrete is substantial in several parking bays with deterioration of up to 60mm in one bay.*
>
> *As requested we have given a price for both removal and waterproofing but we do envisage quite substantial repairs which will become apparent on excavation."*

The CDM quotation covered:

> *"<u>Removal of Surfacing and Spalling concrete from deck</u>*
>
> *Remove damaged surface and concrete by mechanical means and remove from site. Clean area to ascertain damage to steel and any potential structural weaknesses."*

and the waterproofing of repaired areas. The quote does not cover for any repairs. The cutting out is limited to 'damaged' concrete and does not seem include for any additional cutting out that would be appropriate for a structural repair.

It is not clear who was to evaluate *"the damage to the steel and any potential structural weakness"* once the area had been opened up for inspection. CDM were not Structural Engineers.

NCP confirmed their verbal instructions for cutting out with their order dated 29.1.96. The initial cutting out seems to have been done in January for the areas IJ-89, H2 and J2, as shown in CDM's photos [H120 - 128]. The full cutting out prior to repair at H2, J2, and I2-3 was done in March. CDM took some photographs in January 1996. Figures 1.6 - 1 and 2 of the exploratory cut outs at J2 [H126] and H2 [H128] are from that set.

After the preliminary cutting out CDM again wrote to NCP on 31.1.96 with their repair quotation. This refers to the repair, when the weather improves, of :

> *"50m^2 of previously investigated concrete deck*
> *10m^2 of deck not previously investigated*
> *15m of soffit spalling."*

No action was taken on this by NCP until 19th February when they phoned H&S to ask them *"to inspect and report on the repair works"* liaising directly with CDM, as confirmed in H&S letter of 20.2.96. NCP wrote to CDM on 21st February instructing them to commence the works as detailed.

No investigation of the extent and depth and nature of the deterioration was carried out. No drawings were available to any of those involved on site. No structural check on the effects of deterioration or on the effectiveness of the proposed repair was carried out. The procedure used for the repairs was that appropriate for non structural infill repair under waterproofing. No written specification and method statement for the repairs was prepared.

CDM started the repairs sometime in March 1996 and they contacted H&S. CDM used a repair mix based on the proprietary Flexcrete Admixture 850 [17] with added 'pea shingle', which was found to be a flaky crushed aggregate. There were about 25 Acrow props on site but where they were used, how much they were preloaded and how long they were kept in place while the repair gained strength is unclear.

An engineer from H&S visited the site on 20th March and inspected the work in progress on that day and took a set of photographs [H132] including Figure 6.1 - 3 and two others of cutting out prior to a repair. This seems to have been during the cutting out at I2-3 and the exposed reinforcement is consistent with this. The repairs at IJ8-9, H2 and J2 seem to have been completed, but unwaterproofed at that time. H&S briefly reported to NCP on 28th March and had indicated to CDM that the edge of the repairs should be square cut. The nature of H&S's brief from NCP was unclear.

CDM completed the waterproofing about two weeks later. H&S recommended a wider investigation of the structure in their letter of 28.3.96, but NCP took no action on this.

Figure 6.1 - 1 CDM photo [H126] of initial cutting out at J2, January 1996.

Figure 6.1 - 2 CDM photo [H128] of initial cutting out at H2, January 1996.

Figure 6.1 - 3 H&S photo [H132] of cutting out for repairs at I2-3, 20th March 1996.

6.2 FURTHER LEAKAGE AND DETERIORATION AND CRACK AT J2, 1997

In January 1997, following reports by local operational staff of further leakage problems in the area that collapsed, in particular at I-J and 1-2, NCP rang CDM and asked them to investigate these leakage problems. When CDM visited the site they noticed and photographed [H129 - 131] the severe crack, Figure 6.2 -1, on the edge of the slab at J2, adjacent to the 1996 Repair. They reported their concerns about this immediately by phone to NCP from the site and advised the coning off of the immediate area which was done. CDM confirmed this in a letter of 4.2.97 in which they also reported the deterioration of the slab from ongoing frost shattering and quoted for the substantial repairs and resurfacing required for the deck.

Figure 6.2 - 1 CDM photo [H131] of crack to slab edge at J2, January 1997

The next action by NCP was on about 10th February when they contacted H&S and arranged for a joint site visit on the 11.2.97 to inspect the crack. On site H&S gave verbal advice about the crack recommending coning off and propping, as briefly confirmed in H&S letter of 26.2.97. There was some misunderstanding between NCP and H&S about H&S's brief and the responsibilities for further action.

H&S were also asked on 11.2.97 to prepare for an overall evaluation of the deterioration to the structure and recommend repairs. H&S made a preliminary walk round inspection on 11.2.97, so they could plan this work. H&S started during March to obtain Drawings and to arrange for a corrosion investigation by Balvac for this overall refurbishment.

In mid March (~14th) NCP unambiguously instructed H&S to arrange the propping at J2. H&S had approached contractors, but no work had been done, at the time of the collapse on 20th March. Just after the collapse H&S wrote to NCP setting out the approach they would adopt in assessing the safety of other Lift Slab car parks. Although this was after the event, it is indicative of the detailed methodology H&S would probably have adopted if NCP had instructed them to assess Pipers Row more extensively in 1996 and which they would have followed after the investigation by Balvac initiated in 1997.

7 THE COLLAPSE, 20TH MARCH 1997

7.1 COLLAPSE AND INITIAL RECORDING.

At about 3.20 am on the morning of 20th March 1997 the top floor slab supported by columns G1, G2, H1, H2, I1, I2, J1 and J2, an area ~15m by ~15m weighing about 120 tonnes, collapsed onto the 3rd floor slab below. The view from above after the collapse is shown in Figure 7.1 - 1. There was no load other than self weight on the failed slab at that time.

The weather conditions from the nearest Meteorological site at Birmingham Airport, about 18 miles to the South East of Wolverhampton are on record [H202] and indicate a cool night with the air temperature having fallen from 10EC to 4EC at 3am with a slight wind of about 6knots. The wind on the open airport site would be greater than in the built up area around Pipers Row. There was a mostly clear sky (0/8 - 1/8 cloud) which would have led to a radiant heat loss from the top slab surface to below air temperature.

After the collapse National Car Parks, the owner and operator, instructed H&S to investigate the reasons for collapse. The Health & Safety Executive initiated its own investigation. HSE agreed with NCP that they would work with H&S to record the details of the structure and the damage caused by the collapse and to arrange to retain substantial samples of the material from the structure for further inspection and testing. These are shown in Figure 7.2 – 1. This joint recording of the damaged structure enabled the demolition of the whole car park to be carried out shortly after the collapse.

SS&D was instructed to assist HSE in the investigation and visited the site on 25th March. At that time none of the collapsed material had been moved. A joint inspection was carried out with HSE and H&S. Because of the uncertain safety of the 3rd Floor slab no direct access was permitted. A long reach mobile hoist was on site enabling the fractured slab and columns to be closely inspected and photographed without venturing onto the structure.

H&S have reported in detail to NCP on the collapse [H2 - 4]. Some aspects of the analysis in those reports have been further clarified by H&S [H5 - 9]. HSE Birmingham carried out some of the recording with H&S and took a comprehensive set of photographs, HSE P1 to P10 Series [H10 - 119], as listed in Appendix 2, and made dimensioned sketches of critical details [H133 – 143]. SS&D submitted an initial report to HSE [H205]. A press release was issued by HSE at the end of April 1997 [2] giving the ‘Interim Results of Pipers Row Investigation’.

Figure 7.1 - 1 Photo from above the collapsed slab [H17]

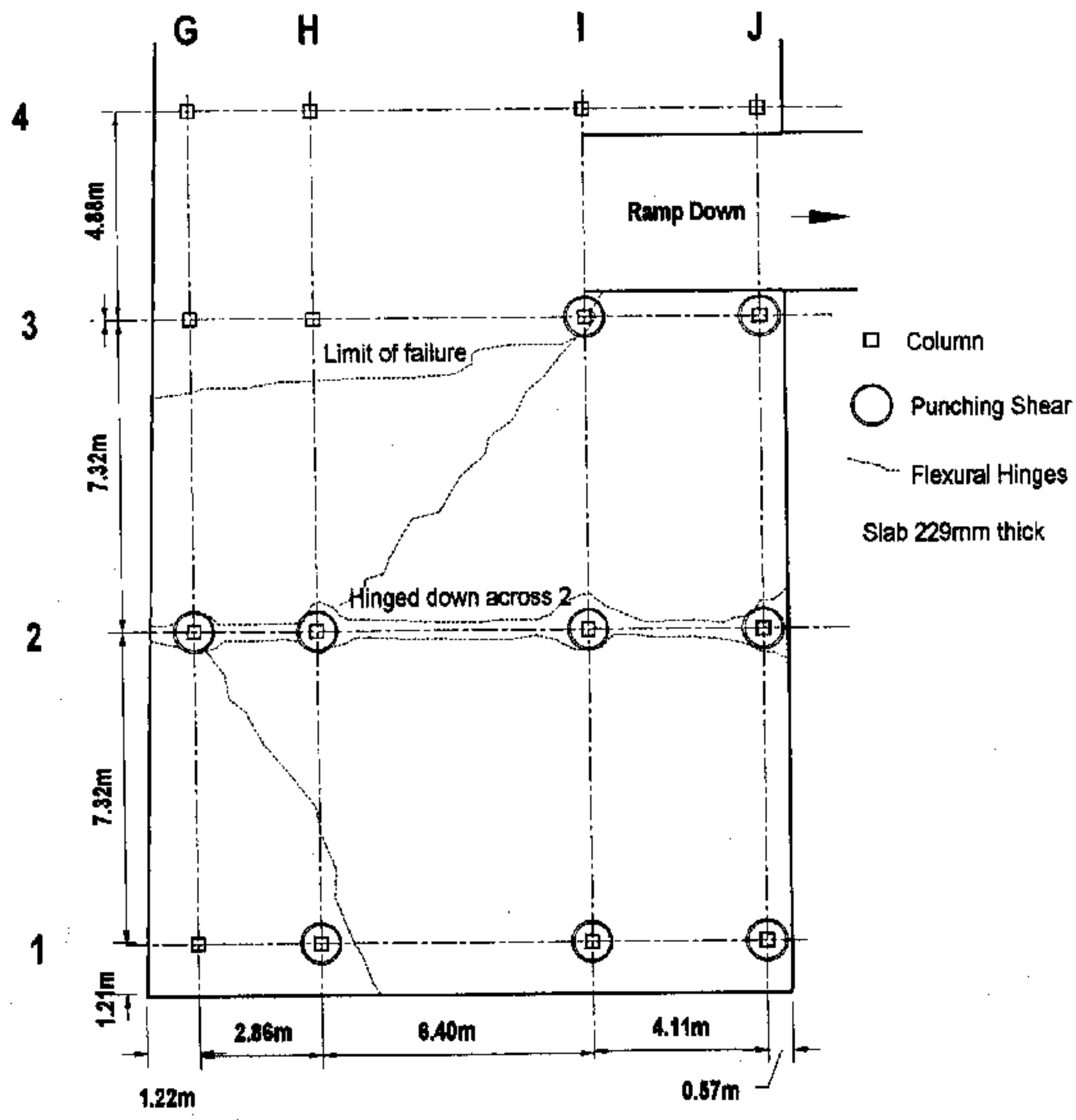


Figure 7.1 - 2 Layout of collapsed slab

7.2 LARGE RETAINED SAMPLES

On the basis of the joint inspection on 25th March HSE, H&S and SS&D identified various large sections of the 4th floor slab and column for careful removal prior to and during demolition. These are shown marked on the slab in Figure 7.2 - 1. While some sections were too fragmented to be cut out and lifted down during demolition, a good representative set, including samples of repair material and a section of the 3rd floor slab S20 were obtained and moved for storage under cover at the NCP car park at Horse Fair in Birmingham, Figure 7.2 - 2. These large retained samples are listed on Figure 7.2 - 3 with their locations shown on Figure 7.2 - 4 based on the HSE Sketch 1 [H133].

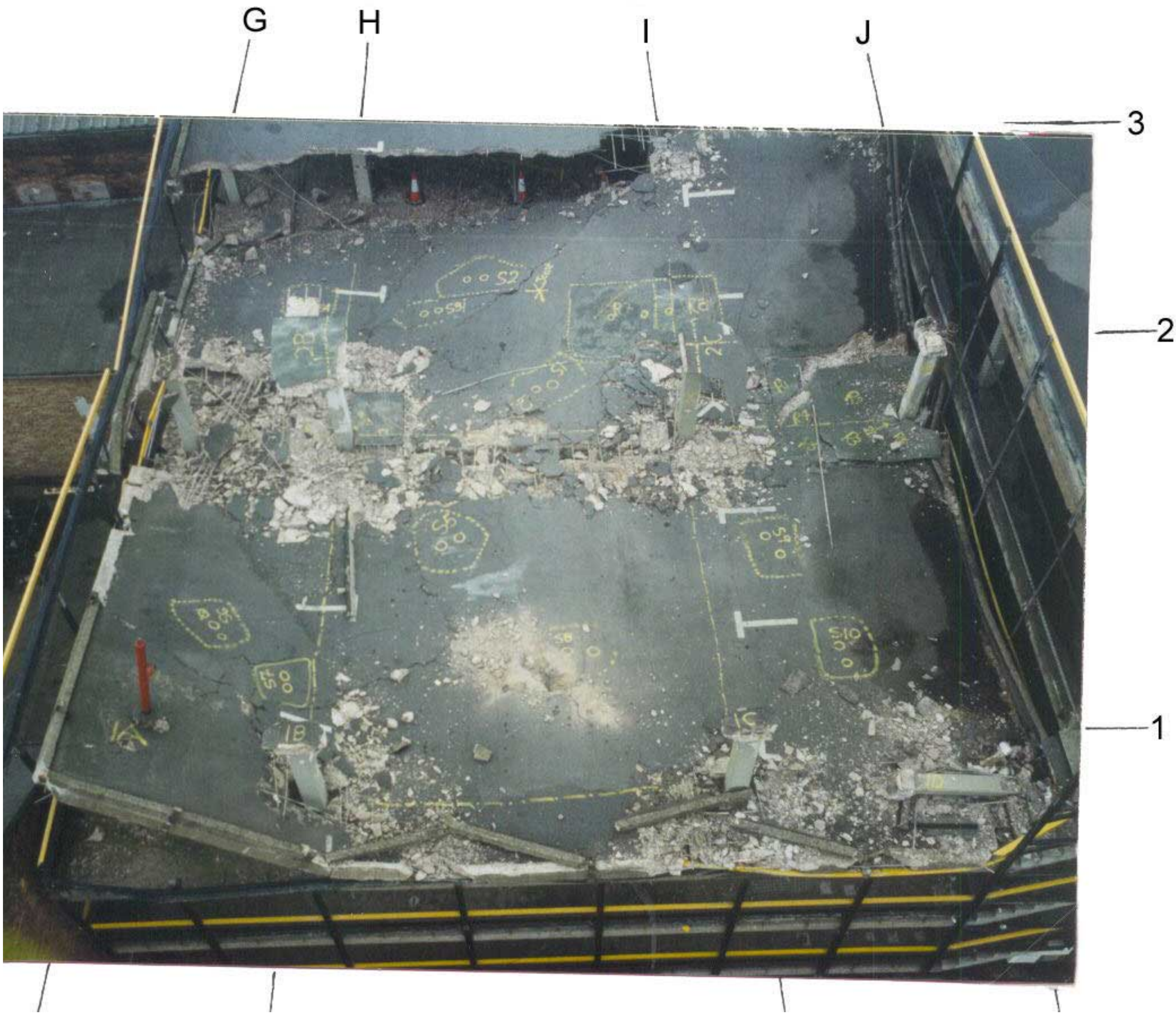


Figure 7.2 - 1 Marked locations for retained slab samples.

Figure 7.2 - 2 Retained samples in store Birmingham

Large Retained Samples Stored at Horse Fair Car Park Birmingham

H&S Store Sample	Location Type	Amey Vectra	Remarks	
Slabs				
S 2	S 2	S 2	Used for temperature recording at BRE	
S 3	S 3	S 3		
S 4	S 4	S 4	Bad Delamination, Poor Cores	
S 5	S 5	S 5	Bad Delamination, Poor Cores	
S 6	S 6	S 6		
S 7	S 7	S 7/1	Badly damaged surface, in three parts	
	S 7	S 7/2		
	S 7	S 7/3		
S 8	S 8	S 8		
S 9	S 9	S 9		
S 10	S 10	S 10		
2B3MF1	S20	S20	Third Floor slab	
Repairs				
2B	H2R	S2Ba	H2 Repair Large	
2B	H2R	S2Bb	H2 Repair Small	
R1	J2R	2Da	J2 Repair	
R2	J2R	2Db	J2 Repair	
R3	J2R	2Dc	J2 Repair	
Columns				
G2 [2A]	G2C	2A	G2 Column	
H2 [2B]	H2C	2B	H2 Column	
I2 [2C]	I2C	2C	I2 Column	Cut open to inspect wedges
J2 [2D]	J2C	2D	J2 Column	
[C1]		C1	?	

Figure 7.2 - 3 List of retained large samples

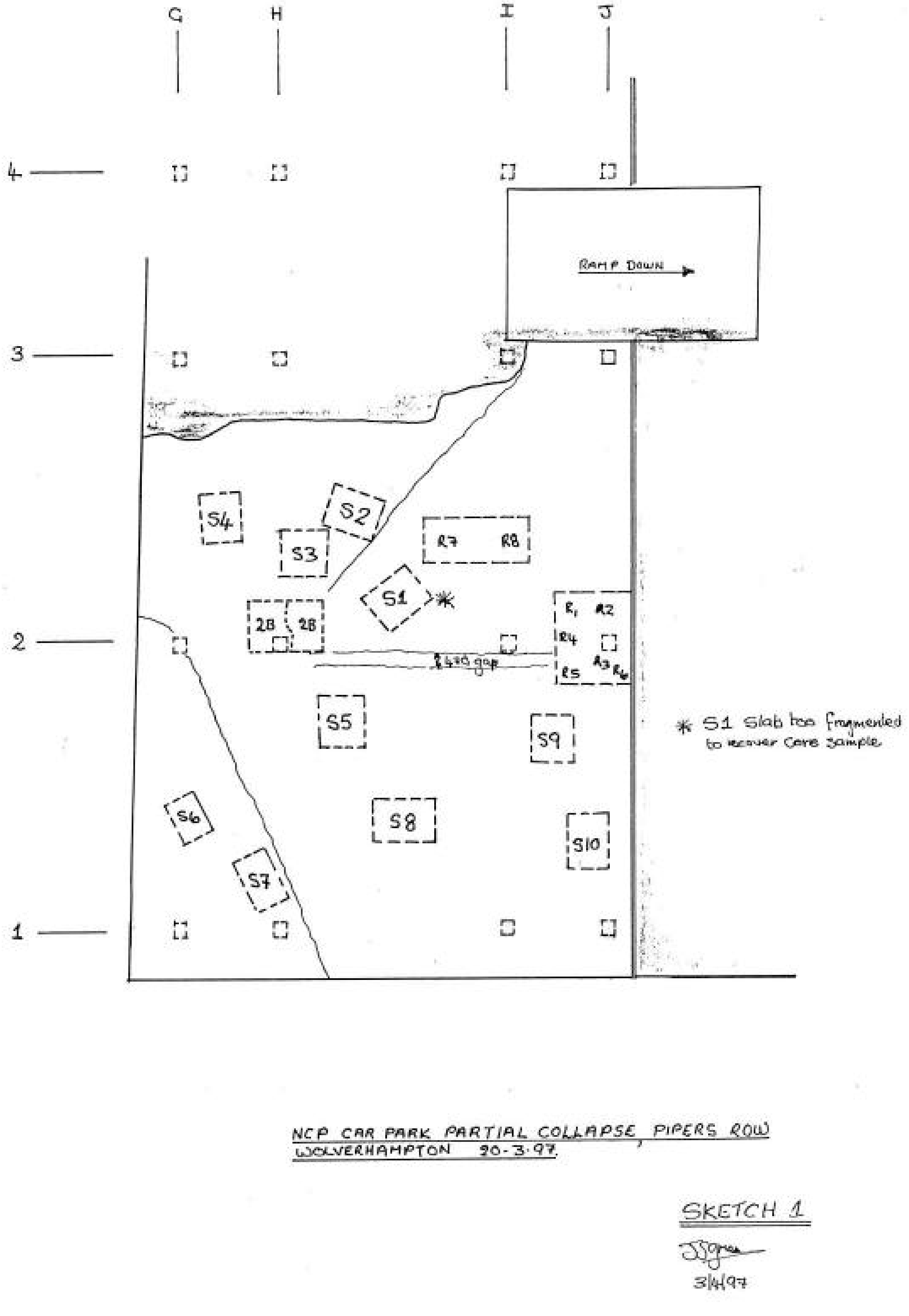


Figure 7.2 - 4 Location of retained column and slab samples.

7.3 SANDBERG MATERIALS TESTING

The objectives for an initial series of tests on materials from the retained samples were agreed by HSE, H&S and SS&D on the 25th March. Sandberg developed this into a coring and testing programme including some samples from other parts of the structure.

Sandbergs produced a series of drafts reports and samples for inspection as results became available. These were discussed with HSE, H&S and SS&D and some additional testing was agreed on. The final results were set out in Sandbergs Materials Report 15643/S to H&S on 22/7/97 [H1]. Sandbergs also took a comprehensive set of photographs of the structure, the large retained samples and their cores, as listed in Appendix 2.

The data from Sandbergs final report, plus further information made available by Sandbergs from their notes, their samples and photos, have been utilised in preparing this report. The review of the Sandberg report identified a number of issues which needed to be clarified by further testing. This testing has been carried out by BRE and RMCS under this contract with all sets of results analysed by SS&D and summarised in Sections 8.3 and 8.4.

7.4 PHOTOGRAPHS AFTER THE COLLAPSE

The photographs taken shortly after the collapse by H&S, HSE, SS&D and Sandbergs provide the main record of the condition of the structure, the extent and types of degradation and the modes of fracture in the initial punching shear failures and the development of the progressive collapse. The list of sets of photographs is in Appendix 2.

All the photographs have been reviewed for features and information on the condition of the car park and the extent of degradation and the failure modes at the column heads and in the progressive collapse. The information from the photographs, collated with the measurements taken immediately after the failure and the data from the retained samples, has been used to determine the condition of the structure when assessing the as-built and deteriorated strength in Section 10.

The following features of the structure, its condition and the mode of failure are of particular importance. The references of photographs which clearly show these features are given. Photographs from the HSE sets and SS&D sets have been used as the primary source with the photographs from the other sets to supplement when appropriate. Eight photographs, which show key features not included elsewhere have been reproduced as Figures 7.4 -1 to 8.

Overall View of Slab Failure

-	Punching and flexural failure along Line 2	P1-8 J1-10,11,12
-	Flexural failure with bottom steel peeling out stopping progressive collapse before Line 3.	H&S 5-17
-	Column J1 pulled towards I1 not J2, indicating failure did not start from J2.	J1-9

Column Head Shear Failures

-	H2 General View	J1-17
-	H2 Shear Failure Surface	P3-5 P2-12a
-	H2 Corrosion and degradation around failure	J1-17 J1-22
-	H2 Repair failure	P3-3

Column Head Shear Failures (cont.)

-	I2 General View	P3-13 J1-19 or 21
-	I2 Shear Failure Surface	P3-8 or 9
-	I2 Corrosion and degradation around failure	P3-11 or 13 J1-21 or 22
-	I2 Degradation of slab towards repair at I2/3	P1-5
-	J2 General View	P1-7 P3-10
-	J2 Repair failure	J1-16
-	I3 Punching shear failure surface, load transferred to ramp.	P2- 10a , J1-20
-	G1 Column pulled over by membrane tension, but did not punch through as bottom steel welded to angle.	P4-11a P1-12 or 13 J1-4

Repaired Areas

-	Overall view showing repaired areas Note fragmented surfacing where degradation had developed outside the repaired areas.	P1-9 P2-8a
-	Repair at H2	P1-6 J1-17
-	Repair at I2/3	P1-5
-	Repair at J2.	P1-7 J1-16

Features Elsewhere in the Structure

-	Areas of ponding	P1-9 S3-27
-	Pothole development.	SA-20
-	Typical star cracking round Column M9 Level 3	S3-19 S3-36
-	Soffit reinforcement spalling repaired.	S3-32
-	Leakage through roof slab	SA-30
-	Slab surface disintegration on a lower floor.	SA-18 S3-34
-	Mortar spall from corroding shear head angle	S3-15

Cutting out of Large Retained Samples

-	In-situ locations of large retained samples	P2-7a
-	Area around S7 above and below.	P2-6 P1-11
-	Cutting out S7, surfacing shattered to reveal degradation.	P2-6a
-	Cutting out S8	P2-5a

Figure 7.4 - 1 H2 general view

Figure 7.4 - 2 H2 Close up view showing repair, degradation and corrosion

Figure 7.4 - 3   I2 General view

Figure 7.4 - 4 Close up view of I2 showing degradation, corrosion and fracture surface

Figure 7.4 - 5 J2 General view showing delamination of repair

Figure 7.4 - 6

G1 column, with bottom steel welded to shear head, did not fail in punching shear

Figure 7.4 - 7 Punching shear fracture at I3, deformation limited by support from ramp.

Figure 7.4 - 8   Typical star cracking around shearhead.

8. FURTHER INVESTIGATION AND TESTING OF CONSTRUCTION AND MATERIALS, 1999 - 2001

8.1 INSPECTION AND TESTING PROGRAMME

Design factors of safety should provide a sufficient allowance for the normal variations arising from reasonable construction practice and tolerances. Actual construction often shows substantially greater variability than the tolerances in standards. In a failure analysis it is necessary to check the actual magnitude of these variations in concrete strength and stiffness, reinforcement location etc and the sensitivity of the structure to them. At Pipers Row the degradation of the surfacing and concrete had become severe in some areas and the extent, depth and nature of this has been evaluated, along with the characteristics and effectiveness of the repairs. This section brings together the data from all sources on the geometry and materials of the 'as-built' structure, the degradation and the repairs.

8.1.1 Initial inspections of retained columns and slabs by SS&D and HSE

Inspections of the retained columns and slabs in the store were carried out by SS&D and HSE at Birmingham and at Cardington at various stages before and during the contract. These inspections have guided the investigation, sectioning and sampling. Some detailed checks, additional to those reported by Amey Vectra and BRE have been made on specific aspects and are summarised in Section 8.2.

At the start of the contract work in September 1999 the retained large samples of slabs and columns (as shown in Figure 7.2. - 2 and listed in Figure 7.2- 3) had been in store inside a partitioned off section of Horse Fair car park in Birmingham since the sampling carried out by Sandberg in 1997. The conditions would have allowed some drying of the slabs. The drying would have increased the rate of carbonation from that in the site conditions. Carbonation would have been developing from the date of fracture on all the surfaces exposed at the time of the collapse, during the demolition shortly afterwards and during Sandbergs sampling.

8.1.2 Recording of retained material by Amey Vectra.

Amey Vectra measured up the slabs to produce a record set of CAD drawings of each slab which are included in their report [H214]. These drawings record:

- the overall thickness and dimensions of the slabs surfacing
- the type, size, depth and spacing of the reinforcement
- the locations size and depth of the cores sampled by Sandbergs in 1997

These drawings were then used by BRE [H219] to mark up their further sampling from the slabs. The data on the slab depths, location of and cover to reinforcement were used in the analysis of the 'as-built' tolerances of the construction.

8.1.3 Materials testing by BRE

Following the recording by Amey Vectra at Birmingham the slabs and columns were moved to BRE Cardington for more detailed inspection. After preliminary inspection they were sectioned by diamond sawing to enable the cut faces to be examined so that a suitable programme for coring and cutting sets of smaller samples for chemical, physical and petrographic tests could be agreed. The cut faces of the large slabs were also examined for features of mix and compaction variation and the depth of cover and carbonation depth and this was recorded by BRE. This and the results of testing on the smaller samples are recorded in BRE Contract Report D1 to HSE [H219]. The implications of their findings, along with data from other sources, have been summarised in Sections 8.3 and 8.4.

8.1.4 Comparative testing of repairs and concrete by RMCS

The potential dimensional incompatibility between the repair material used and the substrate concrete leading to additional interface stresses contributing to delamination had been identified at the time of the collapse and was emphasised in Sandbergs report [H1]. RMCS Shrivenham have made a substantial contribution to the research literature [18 - 20] on testing repair materials and analysing the effects of incompatibilities between concrete and repairs.

CIRIA [21] and the Eurocode on Concrete Repair [22, 23] make recommendations for testing repair materials. These are based on testing cast samples of repair materials or scaled down concrete test procedures.. Test procedures were selected and adapted by SS&D and RMCS to give comparative data on the range of physical properties of cut samples of Pipers Row repair material relative to the concrete.

The available sections of repair material and the concrete with which it was being compared, were sawn into prisms matching standard sizes where possible. However the limited thickness of the repair material and fragility of much of the concrete made it difficult to obtain intact samples of the standard size. BRE who cut the samples for testing at Shrivenham had to adjust the sample sizes to the available material and the test procedures were adapted accordingly. Inevitably these small samples led to a higher scatter of data and this, together with size effect on the test, has been considered in interpreting the results.

The testing, with matched sets of 'Poor' and 'Good' concrete and repair samples, covered :

- Dimensional changes from wetting and drying, based on testing to CIRIA TN141 [21],
- Creep at $10N/mm^2$,
- Young's Modulus E (Secant at $10N/mm^2$) on loading and unloading for creep test,
- Coefficient of thermal expansion, 5 to 40^oC,
- Modulus of rupture tensile strength, alias flexural strength,
- Compressive strength based on testing the ends of modulus of rupture tests.

RMCS Shrivenham Report on Contract D2 to HSE [H221] sets out details of the material tested, the test methods and results obtained. This data has been further analysed by SS&D. The results, with comparable data from the BRE and Sandberg tests, are summarised and their implications are reviewed in the following Sections.

Because the repairs separated from the concrete during the collapse, there were no samples on which pull off tests could be carried out.

8.2 Detailed inspection of retained columns and slabs by SS&D

8.2.1 Inspection of slabs.

The retained slabs have been inspected before and after the sectioning to clarify the overall picture of deterioration in the slab. The samples taken by Sandberg in 1997 and loaned to BRE Watford for the duration of this study were also examined to clarify various points. The BRE [H219] and Sandberg [H1] detailed petrographic examinations and tests have been related to their location in the retained slabs and the location of the retained slabs to the overall structure on the basis of the photographic record. The photographs by CDM [H120 - 131] and Harris & Sutherland [H132] of deterioration before and the sets taken immediately after the collapse, see Appendix 2, have been reviewed and related to this data.

In this review the main focus is on those features of construction and deterioration which may have contributed to the punching failure with only brief notes on items which did not directly contribute (eg soffit local spalling.). Particular attention has been paid to:

- variation of cover and location of reinforcement,
- variability in the mixing, compaction, apparent quality of concrete, defects and day joints
- corrosion
- variability in the depth of degradation relative to reinforcement depth.
- surfacing and underlying degradation
- the as-built detail of the connection of the slab via the shear head and wedges to the column.

Miscellaneous features of construction

The close examination of the sawn sections of the slabs at Cardington clarified a number of details of as-built construction.

The variation in mixing and compaction and in cover to reinforcement was evident on the cut faces and is reported in detail with photos in the BRE report D1/2001.

It was clear from slab S4 that the faces between pours of concrete were not always well prepared as described by the BLS engineer, Appendix 1, prior to casting against the flat vertical face of the previous pour (see BRE report D1 [H219] p54 Figure1). This detail was away from the area close to H2, I2 and J2, so did not influence the initiation of the collapse. No similar details were evident close to H2, I2 or J2 after the collapse or from subsequent examination of the photographs.

Sandberg drew attention to the loose bond of one bar cored through in sample S8 in Plate A6 of their report [H1]. This section of bar was checked. It was close to the end of the bar and appeared to have pulled to break the bond and loosen it during the collapse. A careful examination was made for signs of plastic settlement, an alternative explanation. This creates a slight cavity under a top bar as the concrete settles. This is known to cause serious reduction in bond strength. No significant signs of this were found on any of the top bars sectioned.

8.2.2 Prior detectability of reduced punching shear strength

Particular inspections were made to consider the extent to which those features which contributed to the collapse would have been identifiable by inspection or testing prior to the collapse.

Arup had identified in 1989, from a preliminary assessment based on the drawings, that the structure was vulnerable to deterioration, but this was not followed thorough with detailed inspection and testing. No structure specific inspection regime focused on the vulnerability to deterioration identified by Arups was established. Various indications of deterioration in the slabs were noted for at least a decade before the collapse.

The high risk of collapse at H2, I2 or J2 was not apparent to those involved with the inspection and repair of the slab at H2 and J2 in 1996. In 1997 the focus of attention for inspection and proposed propping was on J2, not H2 or I2 where failure initiated, see Section 10.5.6. Over that period no drawings showing the particular features of the shear head and slab reinforcement had been obtained from British Lift Slab by those inspecting the structure and no assessment of strength reserves and effect of deterioration and repair on it, had been made.

With the hindsight and the detailed analyses carried out after the collapse and for this report it is now clear that the factors of safety were eroded to the point of collapse by the combination of a wide range of factors. Some of these relate to the column reactions and moments and as designed strength which would only have been apparent on the basis of assessment. Other factors contributing to the loss of strength relate to the as-built structure and its deterioration and repair. Some would have been apparent

- during the inspection and repair work in 1996,
- if a thorough inspection and appraisal had been carried out prior to the repair,
- only after the collapse

It is important, for developing improved inspection and assessment procedures for deteriorating concrete structures, to identify the extent to which the weakness of Pipers Row could have been identified prior to collapse. In this section the prior detectability of features contributing to the failure have been considered

The question of the extent to which other areas of the car park were approaching collapse from deterioration has not been considered in this report. However there are reports of areas of deterioration throughout the car park, of the same type as those which triggered the collapse,.

8.2.3 Features apparent during 1996 repair work

The principal features which were clearly apparent during the inspection and repair work in 1996 were the areas of surfacing breakdown and the underlying friable surface layer to below the level of the T1 and T2 layers of top reinforcement with the associated corrosion of the steel. The major effects of this on bond strength and the punching shear strength are set out in Section 10.4.6. Deterioration to this depth was the largest factor in triggering the collapse. Deterioration around the reinforcement should have alerted a structural engineer to the need for a review of the drawings and a detailed appraisal of strength. The potential vulnerability of the flat slab to this form of deterioration should have been clear from a marking up of the depth and extent of deterioration on to the section of the column slab reinforcement as in Figure 8.2.3 - 1.

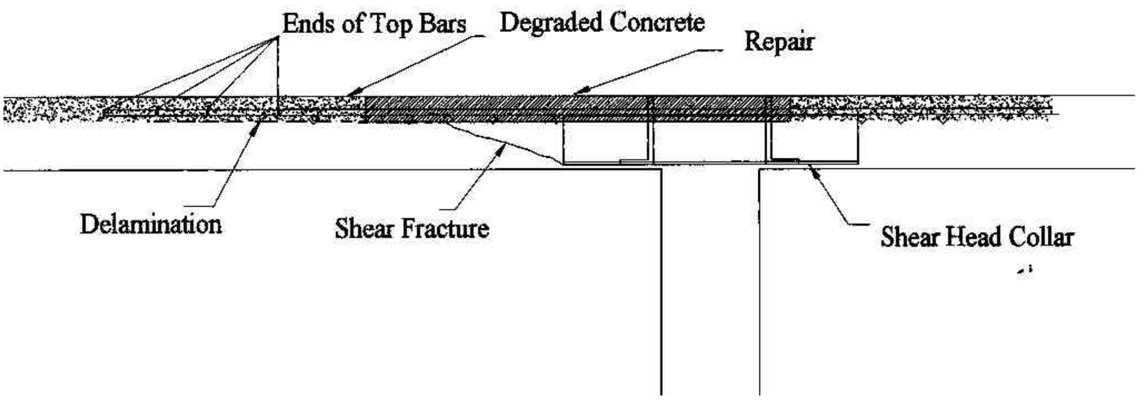


Figure 8.2.3 - 1 Marking of degradation depth and shear fracture plane on Section of H2

The examination after the collapse, and the photos taken then, show a severity and extent of deterioration around the repaired areas which would have been present, but only slightly less developed, in 1996.

- At H2 corroded reinforcement and signs of degraded concrete to below the T2 reinforcement were found all round the repaired area

- At I2 the signs of severe deterioration to below the T2 steel could have been found by simple investigation in 1996. The band of degradation shown by fragmentation during the collapse and corrosion extends from I2 to the areas of repair in the middle of the slab between I2 and I3 and towards the J2 repair.

- At J2 the signs of slab deterioration outside the repair can be seen towards J3 and towards I2.

If the normal specification for cutting out deteriorated concrete to sound material, clear of corrosion, below and at the boundaries of the repairs, had been followed, it would have revealed that the extent of the weakened slab was far wider than the areas which were repaired.

The retained slabs and the examination of the post collapse photos show a patchy pattern of deterioration. There are substantial areas with attached surfacing (eg Slab S2), without surface degradation or top reinforcement corrosion which broke into large sections during the collapse. There are other areas where the slab had surface degradation (eg Slab S5 and S7), reinforcement corrosion was developing and the surfacing and the surface layer fragmented during the collapse.

8.2.4 Features which could have been identified by thorough inspection and testing before or during the 1996 repairs

After a collapse the weakest elements and critical features of construction and deterioration become self evident. When inspecting and testing a structure with a normal budget, only a limited number of areas and tests can be carried out and it is, in part, a matter of chance if testing identifies the most vulnerable features. This review considers, on the basis of the examination of the collapsed structure and the retained slabs and the Sandberg and BRE data, with the benefit of hindsight, the extent to which features found after the collapse might have been identified at the time of the 1996 repair contract.

Various strategies can be adopted for selecting sample locations following an inspection of reinforcement drawings and an overall walk over survey to identify signs of apparent problems eg cracking, ponding, leakage, corrosion spalls, frost damage, traffic damage to surfacing, potholing etc..

A good understanding of the form of the structure and reinforcement and its sensitivity to various forms of deterioration enables the odds of finding potential problem areas to be substantially improved by using a structure specific inspection and testing procedure. For Pipers Row, with a known sensitivity to punching shear failure after the Coventry failure and Arups assessment, this would include specific checks for deterioration or corrosion induced delamination in the slab around columns.

Star cracking, which is often found in flat slab structures from dead load flexural stresses [24], is not necessarily a sign of punching shear distress. It was present at some locations in Pipers Row, as shown in photographs by Wolverhampton BC taken in 1988 [H203] and Sandberg after the failure, see Figure 7.4 - 8. It could have been associated with columns set high giving a higher reaction. A punching shear failure gives few warning signs of circumferential cracking until close to failure as described in Section 10.4.3 and 4. For the Pipers Row top slab any such signs would have been be hidden by degraded concrete and surfacing.

The 1996 repairs were triggered by leakage problems and the repair was primarily carried out to refill a cavity prior to re-waterproofing. The initial opening up in January 1996 exposed an area of deep degradation with corrosion developing on the top layers of reinforcement at IJ-89. Further inspection by CDM identified the three areas of surface deterioration in the failed slab, at H2, J2 and I23, which were repaired. It is not clear if CDM's investigation was based on areas were NCP wanted leakage stopped or on a wider check for areas of damaged surfacing or degradation.

Normally a hammer survey would be used to check a slab and would identify hollow sounding areas from corrosion delamination. The friable material at Pipers Row would have sounded dead relative to sound concrete, and the thin surfacing would have somewhat blurred the difference. So visual inspection combined with hammer checks for looseness in the surface material would have had to be the guide to identify the deteriorated areas. These signs would have become clear by the time the degradation had extended to a depth of 20mm or more, at which the bond and shear strength would have started to be seriously reduced. This could have been used to identify the areas of surface degradation throughout the car park.

Once the areas of surface deterioration were identified they could have been investigated by progressive opening up prior to repair to establish depth and extent of friable concrete and the extent of corrosion and damage to bond strength. Propping would be appropriate in punching shear areas including anchorage zones, when initial cutting out showed degradation had reached the top of the T1 reinforcement.

The normal corrosion survey investigation procedures for checking chlorides and carbonation depth relative to reinforcement cover would not have clarified the extent and severity of deterioration at Pipers Row because:

- corrosion at Pipers Row was not associated with significant chlorides
- carbonation of the top surface was minimal, 1 to 2mm, where the surfacing was intact.
- the carbonation developing through the microcracks in the degraded concrete to initiate corrosion was not easily detectable using phenolthalene spray.
- the high depth of top cover would be regarded as a sign of improved durability and the significance of this in reducing punching shear strength would seldom be noted in a corrosion survey report.
- Surfacing would have prevented the use of a simple ½ cell survey to detect corrosion.

Coring to check the compressive strength of concrete in the structure, if carried out over a wide area with a typical sample set of 6 to 8 cores, would probably have indicated a concrete with a characteristic strength of about 28N/mm^2 well over the 20.5 N/mm^2 specified. This might have been used to justify an increased strength. This can be seen from the sets of data after the collapse by Sandberg and BRE(see Section 8.3.1).

Only if areas of potentially poor concrete were selected for coring on the basis of signs of premature deterioration could the presence of local areas of low strength concrete have been identified prior to the collapse. Possible indicators would be areas with:

- frost damaged friable surface concrete, but this would not pick up weak concrete below areas with good waterproofing.
- deep carbonation of the soffit indicated by corrosion spalling to reinforcement with cover over 10mm,
- large creep deflections indicated by the development of ponding
- honeycombing as an indication of poor compaction.

Various non destructive tests are available for specialist inspection to identify, when calibrated, potential low strength areas for coring. These include:

- Pundit or similar UPV pulse echo tests, where a drop from 4.5km/s to <4.2km/s would indicate low strength,
- Surface pull off or pull out tests, like the internal fracture tests to BS1881 Part 207 used by Sandbergs,
- Schmidt hammer.

To avoid the complications arising from the surfacing and to establish the strength of the base concrete below the surface degradation, these checks could have been carried out on the slab soffit. When repairs were being carried out quality control pull off tests could have provided further data on the strength of the underlying concrete, as well as the effectiveness of surface preparation of substrate concrete.

All the data from inspection and testing needs to be used with available construction records and recollections of those who worked on construction, to establish an understanding of the degree of quality control. Problems can arise from occasional defective pours, short periods of alternative materials supply or general lack of quality control. All these were features found during the post mortem at Pipers Row and are consistent with the recollections of those who worked on the construction, as noted in Appendix 1.

The characteristics of variability of the data from testing on a structure need to be compared with the expected normal distribution in a simple statistical comparison. Samples which break during coring and results at the lower end of the distribution need particular attention and may indicate the need for additional sampling.

The simple surface examination of cores prior to testing can often give indications of the reasons for low strength, but thin section petrographic examination with chemical analysis provides the clearest picture when problems are identified. A high variability in the material is the most important indicator of poor quality control which would indicate a risk of local areas of low strength, possibly substantially worse than indicated from a small set of core strength tests as at Pipers Row.

8.2.5 Inspection of Repairs

The sectioning of the repair samples from H2 (Samples S2Ba and S2Bb) and J2 (Samples S2Da - c) showed the proprietary Flexcrete Admixture 850 with added flaky coarse fraction to ~5mm, (rather than CDMs reported 'pea shingle') to be of reasonable quality, but with a high content of entrapped air voids and some segregation of the aggregate. The tests carried out by RMCS showed it to be substantially stronger and stiffer, but with similar moisture and thermal movement characteristics, to the underlying concrete see Section 8.3. It should be noted that Flexcrete in their instructions for use [17] state that this material is not suitable for use with concretes weaker than 20N/mm^2 .

The main limitations of the repair were:

- the poor compaction around the T1 reinforcement at H2 with T2 reinforcement just exposed, but left in the substrate concrete,
- the poor bond achieved between the repair and the substrate concrete,
- the failure to cut out fully to sound material below and around the repair.

No bond coat or prior coating to reinforcement had been applied. While these coatings are recommended by materials suppliers, various published test comparisons have shown that satisfactory adhesion of the repair and corrosion protection of the reinforcement can be achieved without prior coating, provided surfaces are well prepared and a cementitious mix is well compacted.

The poor compaction of the repair around the T1 reinforcement at H2 did not adversely effect the bond between the strong repair and that reinforcement, as the weak link was at the repair to substrate concrete interface above the T2 bars. The H2 repairs remained intact in two pieces with the T1 reinforcement well embedded in them after the collapse. However the slab had delaminated along the repair to concrete interface which was largely at the level of the interface between T1 and T2 bars. Thus the T2 bars with their cover removed and a poorly adhering repair would have lost substantial bond strength, weakening the slab in punching shear even before the delamination developed.

The surface of slab S2b from H2 was examined in detail by BRE prior to a petrographic section being taken through the concrete repair interface. BRE [219] Sketch map S2bb pp 97 to 99, Annex 2 to Appendix 7] was marked up with areas of fracture of the underside of the repair classified as:

- Failure in repair near interface,
- Interface failure due to weak bond,
- Substrate concrete failure below the interface.

About a third of the fracture fell into each category.

The thin section (BRE P99/1546/10) through an area of repair interface which had fractured in the concrete showed that 70+% of the interface was entrapped air often filled with debris with the 27mm layer of underlying concrete attached being porous, microcracked and fully carbonated. This and examination of the cut faces through the repairs, are indicative of:

- the cutting out for the repair having not been taken down to undeteriorated concrete,
- poor cleaning of the surface prior to placing the repair,
- poor placing of the repair material so as to entrap air,
- the underlying concrete down to below the T2 bars being friable so the fracture occurred below the weak interface.

All these features would have shown up if coring and pull off tests, with examination of the fracture surfaces and cores to check the repair interface had been carried out, following good practice for such a structurally important repair. There is no way now of establishing if a delamination of the repair to concrete interface and/or in the friable concrete below, had started to develop prior to the collapse. With these relatively shallow repairs this could have been checked by a hammer survey at J2 and H2 when the discovery of the crack at J2 alerted NCP to potential problems.

8.2.6 H2 and I2, Location and Indent of Bottom Reinforcement Bars onto Angles.

The original drawings show three ½” (13mm) square twisted bottom reinforcing bars just resting on each side of the lower flange of the long angles at H2 and I2. However the tolerances on the length and placing of these bars made their seating length on the angles uncertain. Potentially they could have increased the shear capacity of the column head by dowel action.

The indent of the bars could be clearly seen where they projected to sit on the angle flange. However most of the bars stopped short of the angles with only one of the three resting on the angle to a depth of 47mm on the face of I2 towards J2 . On the face of I2 towards H2 there were no signs of indents of bars.

On H2 column head face towards I2 there were only signs of one bar seated on the angle to a depth of 25mm. There was no indication of bars being seated on the opposite face. The potential influence of these bars on strength is considered in Section 10.4.7.

8.2.7 I2 Column Head Fracture, Concrete Quality and Tolerances on Wedges.

The top of column head I2 (H&S Mark 2C) can be clearly seen in Figure 7.4 - 4 taken after the collapse, with a deteriorated top surface, corroded reinforcement and the punching shear fracture remaining on the side towards J2. The surfacing, cover concrete and reinforcement came away from the concrete attached to the column during removal. The impressions of the reinforcement are clear on the projecting concrete attached to the column. The column head was cut off about a metre below the slab level and was moved to BRE Cardington with the other column heads.

The discussions with British Lift Slab's engineer on the construction procedure, reported in Appendix 1, and the stated tolerances on the drawings, indicated the risk of lack of fit in the wedges which carry the reaction from shear head angles into the column. The Amey Vectra ANSYS analyses, see Section 9.2.7, have checked for the sensitivity of shear distributions for the cases with all the wedges 5mm high or low and with all the load on one wedge at each column in turn.

The I2 column with shear head and part of the concrete shear fracture surface was sectioned through the projecting fracture surface towards J2, so that petrographic checks by BRE on the concrete for comparison with other tested concrete could be made. The results showed that pre-collapse microcracked carbonated concrete, of higher porosity and water cement ratio than any of the other samples petrographically examined, extended down to the level of this shear fracture surface. This suggests a weak <20N/mm^2 concrete in the I2 shear fracture zone.

The steel shear head angles were cut through to expose the 2 pairs of wedges and their seatings, which carry vertical reaction. Figure 8.2.7 - 1 shows wedges A and B towards I1 resting on the column seating just after the cut angle was detached. Full sets of photos recording the condition were taken by SS&D and HSE. The indent and corrosion free areas of the wedge seating clearly showed that almost all the vertical reaction was being carried on wedge A (towards I1/H2) with a little on its neighbour B. (towards I1/J2) The two wedges, C and D (towards I3), on the other side of the column had heavy corrosion developed on the bearing surfaces indicating that they had not been carrying any load.

The mortar infill between the slab shear head and column was placed after the slab had been seated on the wedges. The mortar at I2 had clearly fully filled the gap on three sides. The fourth side had been in contact with the column face and no mortar had penetrated, but corrosion of the vertical angle face had developed as discussed below in 8.2.8. The mortar infill would have provided a full moment connection from slab to column from the time it was cast.

The soffit of the long angle at I2 showed a slight 2mm bow suggesting that it might have distorted under load, but this could also have been the result of fabrication distortion.

Figure 8.2.7 - 1 Sectioning of Column I2 shear head to reveal wedges

8.2.8 Corrosion, Leakage and Cracking at Column Heads.

The soffit face (underside) of the shear head angles at columns showed considerable corrosion. This is consistent with reports of the spalling of the ½" sprayed asbestos mortar infill below a proportion of the angles throughout the car park and remedial and proposed remedial work to this over the decade prior to the collapse. Spalling of this would have had no effect on punching failures or the overall collapse.

The objective of BLS in setting the angles up in the slab seems to have been to provide a fire protection to the angles, while maintaining a flat soffit for the slab on slab casting procedure. The substantial weakening effect of this reduction of 22mm in the effective punching shear depth of the slab is evaluated in Sections 10.3 and 4. The fire protection of this mortar as-built would have been uncertain in a fire, because of the risk of its premature spalling. The mortar rapidly carbonated leading to corrosion of the underside of the shear head angles which spalled the mortar off at a number of locations.

There were signs of water flow through I2 shear head leaving deposits on the column. When the side face of the angle was cut out it showed that the water flow had caused corrosion of the vertical face of the angle adjacent to the column on the one side where there was not sufficient clearance to permit grouting.

The long sides of the angles had clearly acted as crack inducers for the flexural 'star' cracking. For the upper half of the angle vertical face there were signs of corrosion as carbonation down the crack had initiated rusting. This corrosion would not have contributed to the failure. For the lower half of the angle face the contact pressure from the moment had prevented crack formation leading to carbonation and consequent corrosion. The interaction of flexural star cracking and top steel tension with punching shear is important and considered in Section. 10.5.4, where the DIANA analysis predicts cracking similar to the reported star cracking and that at I2.

8.3 DATA ON PROPERTIES AND DIMENSIONS OF CONCRETE AND REPAIRS

The principle material properties assumed in design, as established by testing materials from the 'as-built' structure and the values adopted for the assessment of the 'as-designed' and 'as-built' loads and strength have been summarised in Figure 8.3 - 1.

The full data sets from BRE, AV, RMCS and Sandberg have been collated and analysed by SS&D and the principle data is summarised below.

8.3.1 Compressive Strength.

Detailed testing programmes for concrete strength and stiffness were carried out as part of the BRE Contract D1 and RMCS Contract D2. The combined data from the Sandberg, BRE and RMCS compressive strength tests have been reviewed and summarised by SS&D. The skew resulting from not testing the material which was too weak for successful sampling, has to be taken into account. These tests were carried out on 'undeteriorated' material from below the surface layer of degradation. In interpreting this data the wide variability of mix characteristics reported in the detailed petrographic examinations carried out by Sandberg and by BRE need to be considered.

A 'normal' 3000lb/in^2 (20.5N/mm) concrete cast in 1964/65 would be expected to have had a mean cube strength of about 27N/mm^2 at 28 days. With cement strength gain from further hydration with time since 1965 this might have increased by about 30% to an expected mean strength of 35N/mm^2, with a characteristic of about 29N/mm^2.

The compressive test data from all the Sandberg, BRE and RMCS tests is plotted as equivalent cube strengths on Figure 8.3.1 - 1. The data has been separated into that from:

- 'Failed Slab' obtained from retained slabs S2 to S10
- 'Other Slabs' including S20 from the 3rd floor

The Sandberg data from 'other' areas (except for two outliers) are in line with the 35N/mm^2 expected mean. The samples from S20 3rd floor slab (the slab below that which collapsed) tested by BRE and RMCS gave a mean of 47N/mm^2 and a characteristic of 38N/mm^2. However other areas of this slab are likely to have varied from this particular pour, as with the variability found in the top slab.

This data suggests that a set of 6 to 8 random check cores from the car park as a whole would probably have a compressive strength range of 30 to 50N/mm^2, with about 28N/mm^2 characteristic strength. This is consistent with the originally specified 20.5N/mm^2 concrete with normal strength gain with age .

A similar set from the 'failed' slab would have given a range of 22 to 35N/mm^2 , with about 20N/mm^2 characteristic strength indicating concrete just complying with the design value of 20.5N/mm^2, but would have been less at 28 days before age gain.

Cores from the failed slab taken specifically from 'poor' concrete, indicated by the worst areas of apparent degradation at the time of repair and/or from areas of high carbonation depth and spalling on the soffit, would have had a range of 17 to 30N/mm^2, indicating a characteristic of about 15N/mm^2. Concrete in cores which fractured during coring may well have had still lower strengths. As the structure has about 380 column heads, 19 would be expected to have concrete strengths less than the characteristic.

Figure 8.3 - 1

Comparison of properties used in design and analysis with test data from the structure

		Design and appraisal assumptions				BRE DIANA		Test data from as-built structure			
		CP114	BS8110	BS8110	Amey	'As- built' appraisal		Failed slabs	Failed slabs	Other slabs	Repair
			Slab	Column	Vectra	Concrete	Repair	worst area	typical	Av.	
Saturated Density	kg/m3								2250	2337	
Density	kg/m3	2414				2300	2300				
E Long term	kN/mm2				20.5	20	30	9	18	24	31
E Short term	kN/mm2		24	28	10.25	10	15		8		17
Poisson's ratio					0.2	0.2	0.2				
Compressive strength											
Fcu Design	N/mm2	20.5	20.5	41	21	as tests char.					
Average								22	27	35	70
Characteristic								15	20	28	65
Tensile Strength char.	N/mm2	-	-	-	-	1	6	<1.3	1.7	2.5	5
Slab Thickness		229				235			235		
Slab weight	kN/m2	5.41				5.3					
Surfacing Thickness		36				5			5		3
Surfacing weight	kN/m2	0.81				0.1					
Coefficient of Thermal Expansion											
Average					12				11.3		11.5
upper									12.3		12.1
lower									10.8		10.3

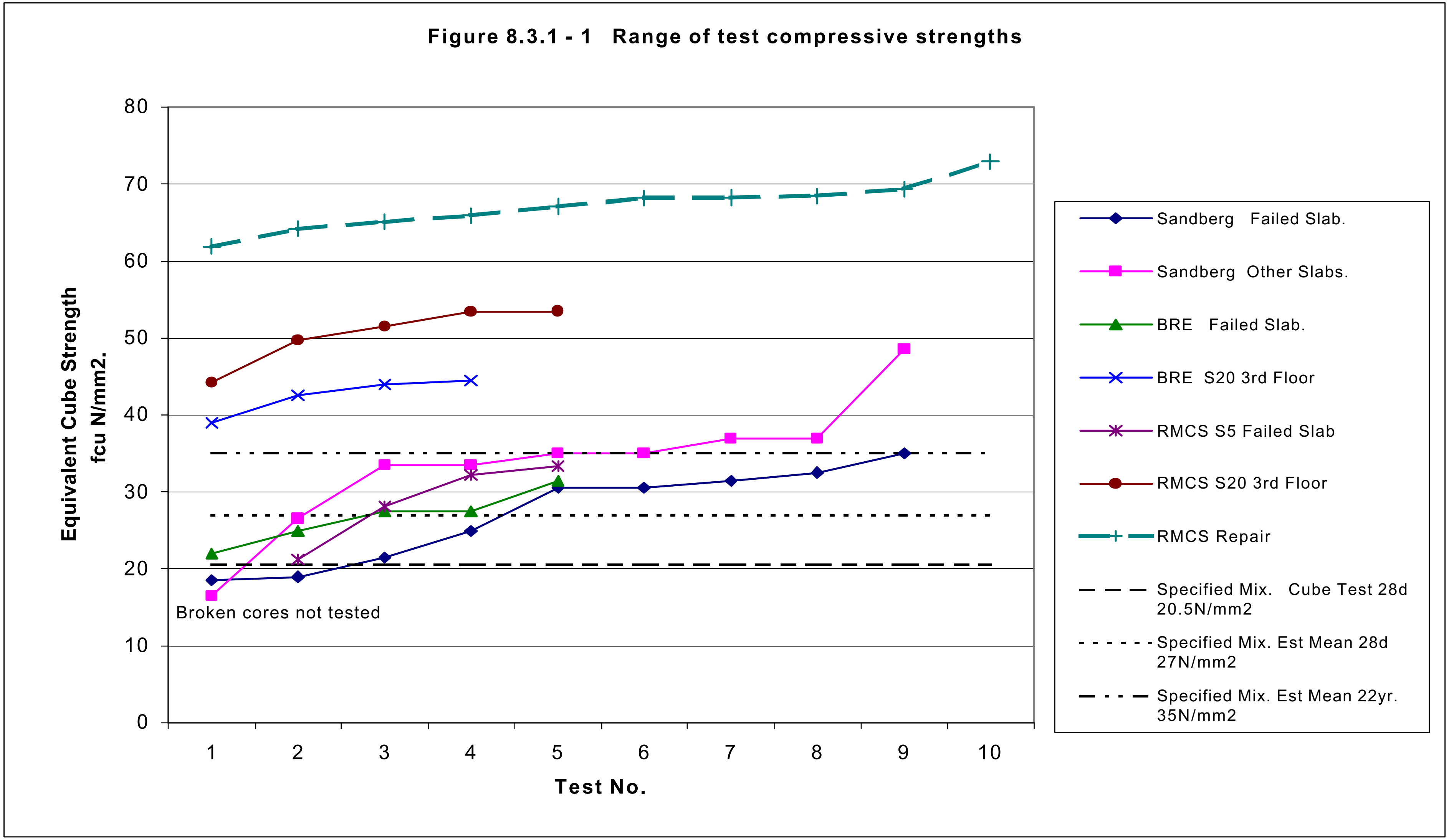


Figure 8.3.1 - 1 Range of test compressive strengths
Equivalent Cube Strength fcu N/mm2.
Test No.
0
10
20
30
40
50
60
70
80
1
2
3
4
5
6
7
8
9
10
Broken cores not tested
Sandberg Failed Slab.
Sandberg Other Slabs.
BRE Failed Slab.
BRE S20 3rd Floor
RMCS S5 Failed Slab
RMCS S20 3rd Floor
RMCS Repair
Specified Mix. Cube Test 28d 20.5N/mm2
Specified Mix. Est Mean 28d 27N/mm2
Specified Mix. Est Mean 22yr. 35N/mm2

This demonstrates the unreliability of the normal practice of taking 6 to 8 cores at random from a structure to establish an as built characteristic or 'least credible' strength for structural appraisal. This type of testing was adopted by H&S to check other car parks immediately after the Pipers Row collapse. Had an inspection and testing survey been carried out before the collapse, random sampling would have been unlikely to have picked up the localised areas of particularly low strength concrete which contributed to the failure at Pipers Row.

To have identified the areas of low strength concrete in an overall materials investigation of Pipers Row before the collapse it would have been necessary to:

- determine the standard of construction quality control from the variability of concrete in an initial set of core strength tests and from the examination of cores.

- supplemented the random sample set with additional cores from areas where signs of premature deterioration indicated sub-standard concrete.

Internal fracture tests

Sandbergs carried out a set of internal fracture tests to BS1881 Part 207, interpreted using BRE IP 22/80, on the retained slabs. On their data they report in Appendix F1 of their Report of 22/7/97 average compressive strengths of 25 to 64 N/mm^2 on all samples except S4 and S5. Tests into the degraded surface material on S4 gave 16N/mm^2 and on S5 <8N/mm^2. It is not clear the extent to which some of these were into the deteriorated surface material as distinct from the underlying un-degraded concrete. The test depth is nominally 17mm so surface condition could have been governing.

The interpretation of these internal fracture test results was based on the average of 6 tests but many sets show high 'K' coefficient of variation. Potentially internal fracture tests could be used for checking for low strength areas to be cored for definitive strength data. The use of the average value only in interpretation tends to mask the indications of local variability apparent in individual results which could provide a warning of the risk of areas of low strength.

Repair

The RMCS tests of repair mortar cut from the larger section of the repair at H2 gave a mean equivalent cube strength of 71N/mm^2 and a characteristic of 65N/mm^2.

8.3.2 Tensile Strength

The tensile strength data was obtained from BRE and Sandberg splitting and RMCS modulus of rupture (flexural strength) tests. In making assumptions on the strength of the slab and repair at the time of failure consideration must also be given to:

- the samples which fractured in preparation
- the petrographic reports on the degraded material in the upper part of the surface which was too weak to sample and test.
- the poor bond at the interface with repair

For concrete from the 'failed' slab with a compressive strength range of 20 to 35N/mm^2 splitting tensile strength values of 1.9 to 2.8N/mm^2 would be expected. Overall the tensile results show a comparable range of results to those expected from the compressive tests. The low values of between 1.65 and 1.9N/mm^2 recorded are consistent with the weaker material down to 15N/mm^2 found in compression tests. These results indicate that the normal relationship of compressive to tensile strength implicit in the methods of calculating punching shear strength are valid for this concrete.

The RMCS modulus of rupture tests of repair mortar, cut from the larger section of the repair at H2, gave an equivalent splitting strength (0.75 x test modulus of rupture, see Neville 'Properties of Concrete' p 597) in the range of 5 to 8N/mm^2 and the 'good' concrete in S20 from the 3rd floor gave values over 3N/mm^2. However, because of sample size constraints, the S20 data was from three point test which tends to give a somewhat higher strength than the four point test.

8.3.3 E Young's Modulus. Stiffness of Concrete and Repairs

The variability and poor quality of the concrete in the 'failed' slab area shows up in the low values E (Young's Modulus) and USPV data. The BRE data for the 'failed' slab samples gives values of E ranging from 9.1 to 21.1kN/mm^2 . The 3rd Floor slab S20 was stiffer with E ranging from 21.9 to 25.2kN/mm^2 .

The BRE loading rate was (10 seconds 0 to 5N/mm^2). To standardise the data the E has been based on the gradient of the stress strain curve between ~3 and ~5N/mm^2. To illustrate the range of stiffness the stress strain curves for a core from a 'poor' concrete slab S5, two cores from 'intermediate' concretes from slab S9 and a core from the 'good' concrete in slab S20 from the third floor have been plotted on Figure 8.3.3 -1.

RMCS results from the slower creep test loading (0 - 10N/mm^2) on 'failed' slab samples range from E = 17.6 to 20.5kN/mm^2 . This excludes one outlier sample of 'good' concrete from Slab 8 (S8/5/4) with an E of 34.7 kN/mm^2, which also showed substantially better creep and USPV than the rest.

The USPV values for the 'poor' concrete ranged down to 3,850km/s compared to 4500km/s for 'good' concrete and this provides a potential basis for identifying areas of substandard concrete in-situ.

Code guidance for 20N/mm^2 concrete suggests a short term E = 25kN/ mm^2 (range 21 to 29kN/mm^2). The analyses cases carried out by Amey Vectra and BRE used E = 20.5kN/mm^2 for the short term E and 10.25kN/mm^2 for the long term E and the deflection data can be pro rata adjusted to examine the sensitivity to the measured E values.

The low E in the failed slab would have had the following consequences for the structure:

- The elastic deflections when lifted would have been pro rata increased.
- The sensitivity of reactions to differentials in column levels would be reduced.
- The precast columns in 41N/mm2 (6000lb/in^2) concrete with a higher E than the slab, would attract more moment under surfacing and live load and as creep deformations of the slab developed applying rotations to column heads. This would increase the effective shears V_{eff}.

The greater deformations from the low E would have been a significant factor in the development of ponding, which led to slab degradation, despite the generous falls provided in the initial design.

The average E for the repair when tested by RMCS was 31N/mm^2 short term, similar to the best concrete from Pipers Row.

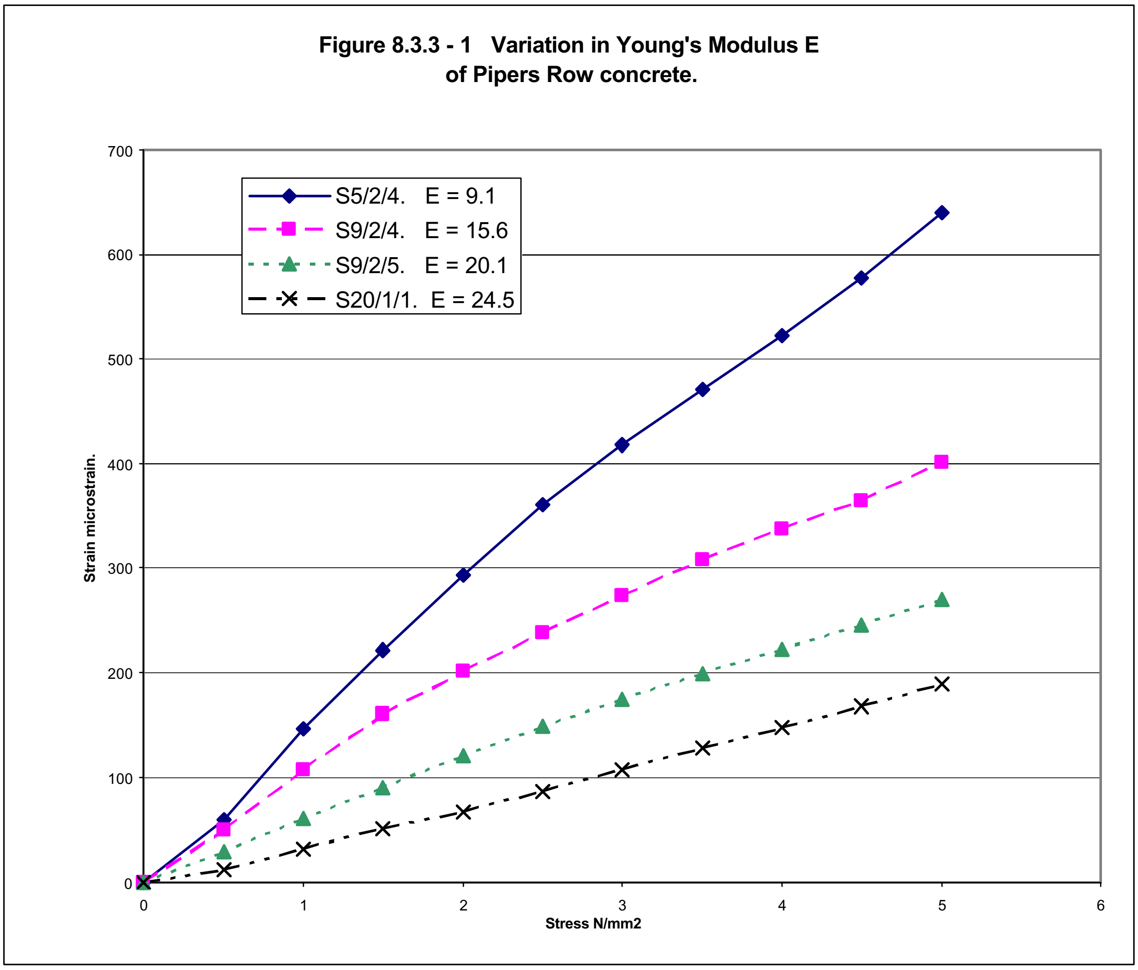


Figure 8.3.3 - 1 Variation in Young's Modulus E of Pipers Row concrete.

8.3.4 Creep

The creep and shrinkage of the concrete had a major influence on the development of deflections at Pipers Row. Because of the unequal spans, these deflections led to the redistribution of moments from the slab into the columns increasing the effective shear stress and to the development of areas of ponding where deterioration of the slab was accelerated.

In design to CP114 a very simplified approach to deflection control was adopted and BS 8110 is similar in specifying span depth ratio rather than requiring a detailed calculation of deflection based on a long term stiffness to represent creep and shrinkage. In Part 2 Section 7 of BS8110 a more detailed treatment of creep is set out which indicates that creep deflections would have been increased if the lifting was carried out at an early age..

RMCS carried out creep tests on 40 x 40 x 160mm samples cut from concrete of a range of apparent qualities from the failed slabs (5 samples + 1 unstressed control) and from repair material (3 samples + 1 unstressed control). A steel bar of approximately the same dimensions as the cementitious samples also acted as a control. Testing was conducted to 'BS 6319: 1984. Testing of resin compositions for use in construction. Part 11: Method for determination of creep in compression and in tension'.

A 100mm Demec gauge was used to monitor strain on all four faces. The creep test was conducted over a period of 5 weeks under a compressive load of 10N/mm^2. Moisture content was maintained by clingfilm wrapping. Measurements were taken once a day during the first week, then once a week for a period of five weeks. The samples were then unloaded and the final readings taken. Humidity and temperature were monitored. Full details are in the RMCS Report [H221] and fuller analysis of the data has been carried out by SS&D.

The creep strains are plotted in Figures 8.3.4 - 1 for the failed slab concrete samples. They show that sample S8/5/4 4 - 7 is similar to that expected for a reasonable quality >20N/mm^2 concrete with a good E Young's Modulus of 35kN/mm^2 and a maximum strain after 5 weeks of 500 microstrain The other four concrete samples were of low quality with lower E and total strains at 5 weeks in the range 1100 to 1300 microstrain, which would have substantially increased long term deflections in the structure. The creep factors (long term strain/short term strain) were 1.75 for 'reasonable' concrete and 2.25 for 'low quality' concrete.

Figures 8.3.4 - 2 shows the creep strains for the repair samples which are similar to the S8/5/4 4 - 7 concrete with total strains at 5 weeks in the range 540 to 620 microstrain well matched to good concrete.

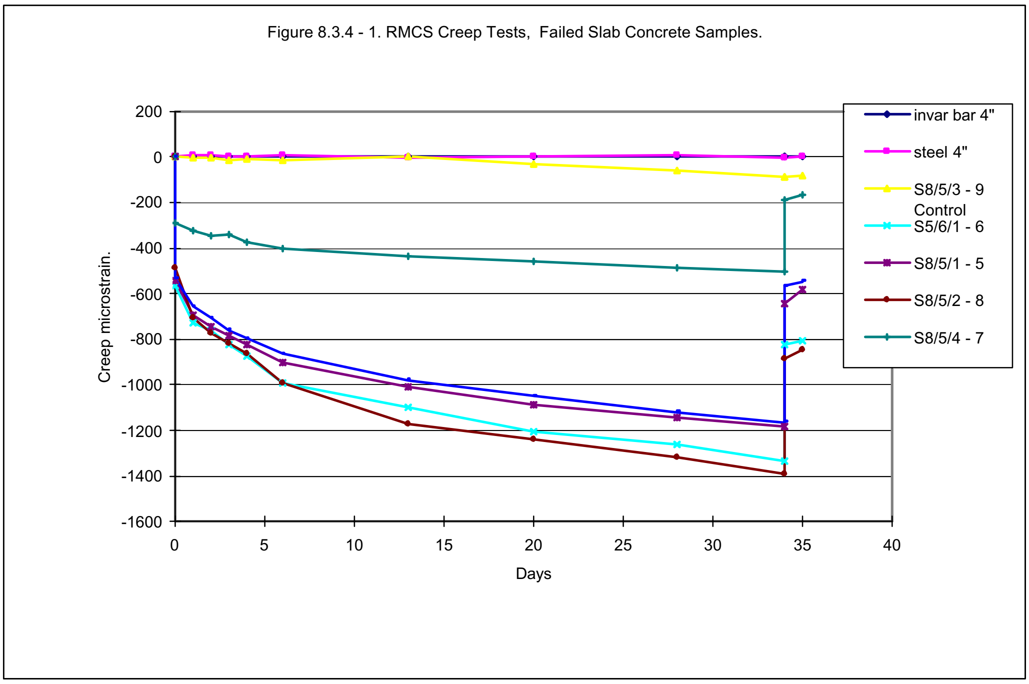


Figure 8.3.4 - 1. RMCS Creep Tests, Failed Slab Concrete Samples

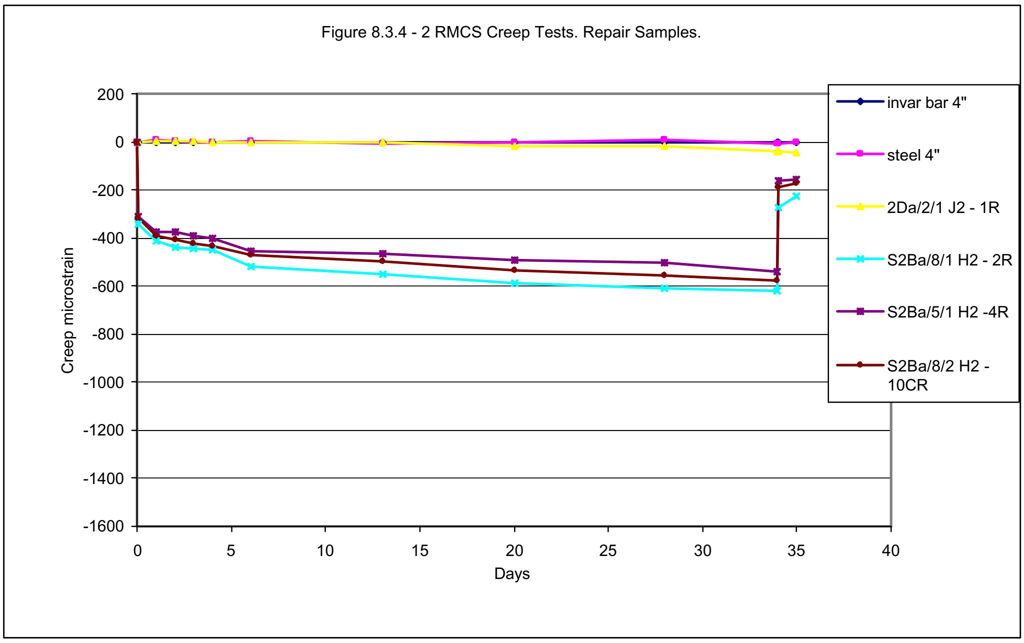


Figure 8.3.4 - 2. RMCS Creep Tests, Repair Samples

8.3.5 Shrinkage and Thermal strains

Samples of concrete and the repair material were tested by RMCS following the principles in CIRIA TN141 'Standard tests for repairs and coatings for concrete , Part 3 : Stability, substrate compatibility and shrinkage tests', [21] adapted for the particular materials available. The concrete samples where divided into 'Poor concrete'(Samples 7 to 9) from S5 and 'Better concrete' (Samples 10 to 12) from S9 and S20 from the third floor. The repair material (Samples 1 to 6) was cut from H2 Column head repair sample S2Ba. Because of the fragility of S5 it was only possible to obtain 40 x 40 x160mm cut prisms from it, all the other samples were 50 x 50 x 200mm prisms.

The strains on the four faces of each prism were recorded by Demec gauge and their weights determined as the prisms were:

1. wetted for 7 days in a water tank at 20°C,
2. cooled to 5°C then heated to 40°C and allowed to cool to 20°C, but there were erratic results due to temperature variation,
3. allowed to dry on a laboratory bench at 20°C (~50%RH) for 34 days,
4. wrapped to prevent moisture loss while they were again cooled to 5°C, then heated to 40°C. This was a longer cycle than 2 and enabled temperatures to properly stabilise.
5. Dried in a ventilated oven at 40°C for 7 days (~10%RH check)
6. Dried in a ventilated oven at 105°C for 3 days

The magnitude of these strains needs to be related to the tensile strain (150 to 250 microstrain) at which concrete cracks. The strains also need to be considered relative to the differentials in moisture and temperature in the slab and the repair which develop with changes in temperature and as it wets and dries.

Full details are in the RMCS Report. A fuller analysis of the data, including the relationship of weight change to strain, has been carried out by SS&D. The strains through this sequence are set out on Figure 8.3.5 - 1 which plots the average values of strain, adjusted to 20 ^{o}C, for each group of samples Repair, 'Poor' and 'Better' Concrete. This shows that there is a broad similarity in moisture movement for all the material types. The more rapid shrinkage of the 'Poor' concrete is largely a result of the smaller sample size necessary because of the fragmentation of the material during cutting.

Thermal expansion

The coefficient of thermal expansion for the 12 samples were very similar. Results were based on the average of Stage 4 cooling and heating only, as the Stage 2 results had variations in temperature and so were not used.

All the samples of concrete had coefficients in the range 10.8 to 12.3, average 11.3microstrain per °C The repair material had coefficients in the range 10.3 to 12.1, average 11.5microstrain per °C These are close to the standard value of 12microstrain per °C used in analysis and there is no differential between the repair and original concrete.

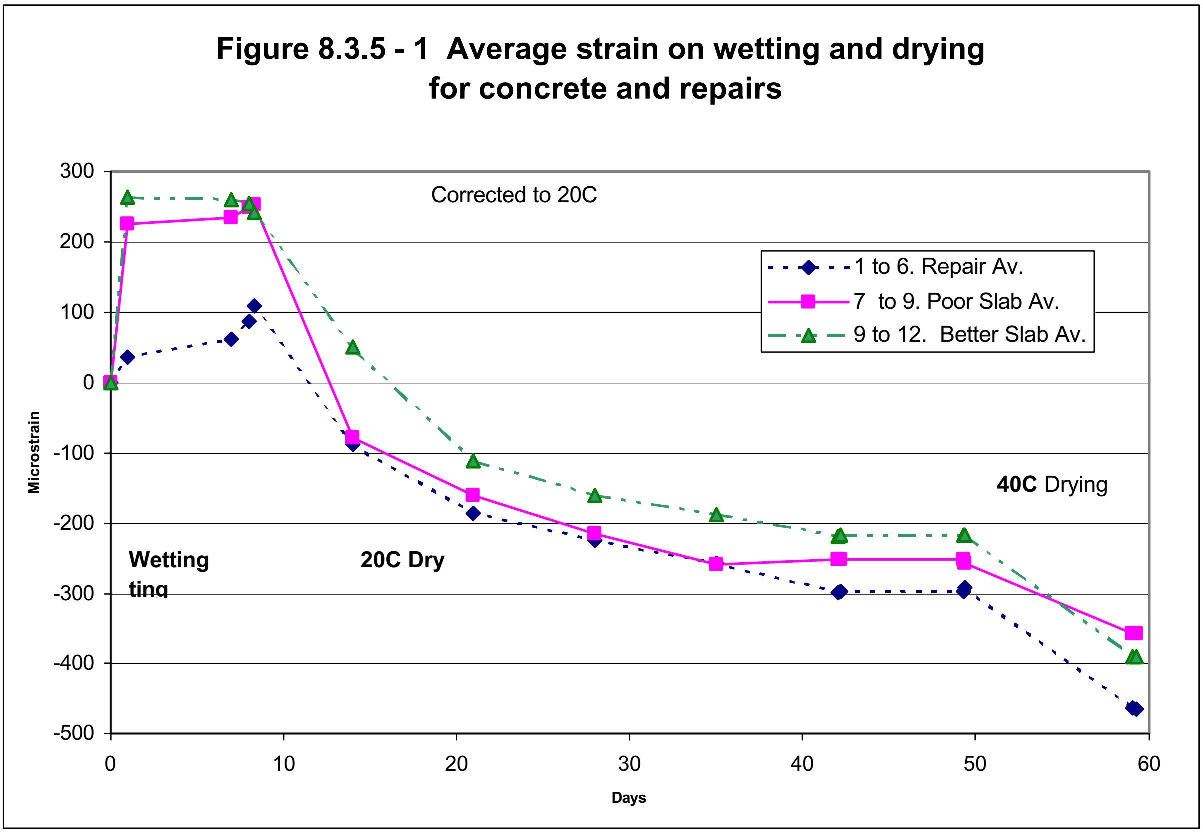


Figure 8.3.5 - 1 Average strain on wetting and drying for concrete and repairs

8.3.6 Slab density

The Sandberg data shows the slab concrete of the 'failed' area has an average saturated density of 2270kg/m^3 (range 2170 to 2330 kg/m^3) compared to 'other' areas with 2337kg/m^3 (range 2240 to 2410 kg/m^3). The BRE and RMCS density data give similar results on these concretes. RMCS data shows that the repair material average density was 2320kg/m^3 (range 2275 to 2388 kg/m^3), similar to that for concrete from 'other' areas.

8.3.7 Slab thickness and self weight.

The concrete slab thickness was checked by Amey Vectra and BRE on the retained slabs and some measurements were taken on site by H&S and HSE immediately after the collapse. The average measured slab thickness was 235mm with a range from 225mm to 250mm, compared to the 9" (229mm) specified.

The saturated density of cores recorded by Sandberg was 2270 kg/m^3 for the failed slab area, with 2337 kg/m^3 for those from other areas. Allowing for reinforcement and a 3% reduction of density from drying of the waterproofed areas of slab a reasonable density for calculating loading at the time of failure would be 2300kg/m^3 compared to the design 2414kg/m^3 for the 9" (229mm) slab. Taking this with 235mm thickness gives a dead load from the slab of 5.30kN/m^2 compared to the 113lb/ft^2 (5.41kN/m^2) of the original design.

8.3.8 Surfacing Thickness and Self Weight.

The surfacing on the top floor slabs retained after the collapse ranged from 3 to 6mm thickness. The surfacing over the repairs was slightly less at 1 to 3mm. A reasonable figure for 'as constructed' surfacing weight is 5mm depth at a density of 2000kg/m^3. This gives 10kg/m^2 (0.1kN/m^2), significantly less than the 83kg/m^2, (0.81kN/m^2 or 17lb/ft^2), which was allowed for in design. The lower floors were unsurfaced.

8.3.9 Location and Cover to Reinforcement.

The punching shear strength is sensitive to the effective depth *d* of the top reinforcement above the level of the support for the slab and the distribution of reinforcement adjacent to the column.

Immediately after the collapse HSE made measurements of the lateral spacing of bars at H2 and I2. The as-designed and as-built lateral spacing of reinforcing bars relative to columns H2 and I2 is shown in Figure 8.3.9 - 1. It was not possible to take similar measurements at J2.

The variation in the thickness of the slab (see 8.3.7), actual size of the reinforcement bars and the depth of cover as placed, all contribute to the variation of the as-built effective depth from the as-designed value.

The design was based on ¾" round bars, but square twisted bars (SQT) were used with a greater depth (23.9mm compared to 19mm) which reduces the effective depth for T2 bars by 2.5mm and the T2 bars by 7.5mm.

The as-built cover to the top bars in the slab was found to be variable and almost always low in the slab reducing the effective depth. Without shear steel there was little support for the bars during the pouring of the slab. HSE took measurements [H134, 135] immediately after the collapse giving 40mm as the representative cover depth at H2 and 29mm at I2. These values have been used as the basis for as-built strength checks with the mean slab depth of 235mm.

The cover depth to reinforcement in the slabs S2 to S10 and S20 were checked by Amey Vectra (at edges) and BRE (on cut faces) as set out in their reports.

Only slabs S3, S4, S7, S10 and S20 had top reinforcement with covers noted as follows:.

S3	SQT	53, 44, 27, 30mm
S4	Round	10, 28, 23, 12, 22mm
S7	SQT	31, 34 mm
S10	SQT	49mm
S20	SQT	43, 49, 49, 58, 64mm

This is not sufficient data to permit statistical analysis, but confirms the high variability of cover depth generally greater than the ¾" (19mm) specified.

The variation in cover to the bottom reinforcement and the variable depth of carbonation, reported in detail by BRE [H219], contributed to the premature development of corrosion of the soffit reinforcement with associated spalling. The bottom reinforcement did not influence the punching shear failure initiation.

H2 T1 bars 3/4" SQT

	As-designed	As-built
	mm	mm
G2 side		
11	1509	
10	1328	
9	1147	1054
8	966	934
7	823	814
6	721	714
5	619	614
4	517	514
3	412	379
2	313	299
1	211	154
Column edge	153	153
Column Cl	0	0
Column edge	153	153
1	201	228
2	286	353
3	370	478
4	456	708
5	541	867
6	695	
7	917	
8	1200	
9	1542	
I2 side		

H2 T2 bars 5/8" SQT

	As Designed	As-built	
	mm	mm	
H3 side			
6	1648		
5	1286	1232	
4	1025	933	
3	863	813	
2	701	673	
1	539	478	
Column edge	153	153	
Column Cl	0	0	
Column edge	153	153	
1	539	453	
2	701	593	
3	863	723	
4	1025	930	Est.
5	1286		
6	1648		
H1 side			

I2 T1 bars 3/4" SQT

	As Designed	As-built
	mm	mm
H2 side		
11	1509	
10	1328	
9	1147	1283
8	966	1053
7	823	883
6	721	753
5	619	663
4	517	548
3	412	373
2	313	253
1	211	218
Column edge	153	153
Column Cl	0	0
Column edge	153	153
1	201	298
2	286	328
3	370	428
4	456	533
5	541	673
6	695	723
7	917	
8	1200	
9	1542	
J2 side		

I2 T2 bars 3/4" SQT

	As Designed	As Built
	mm	mm
I3 side		
6	1648	
5	1286	
4	1025	783
3	863	683
2	701	573
1	539	503
Column edge	153	153
Column Cl	0	0
Column edge	153	153
1	539	583
2	701	683
3	863	768
4	1025	943
5	1286	
6	1648	
I1 side		

As-built bar locations . H&S 1997 Report [App. 2 JSG Sk 2 I2 and SK3 for H2]

As-designed bar locations from original BLS drawings

Figure 8.3.9 - 1
Comparison of as-designed top reinforcement bar spacing with as-built locations

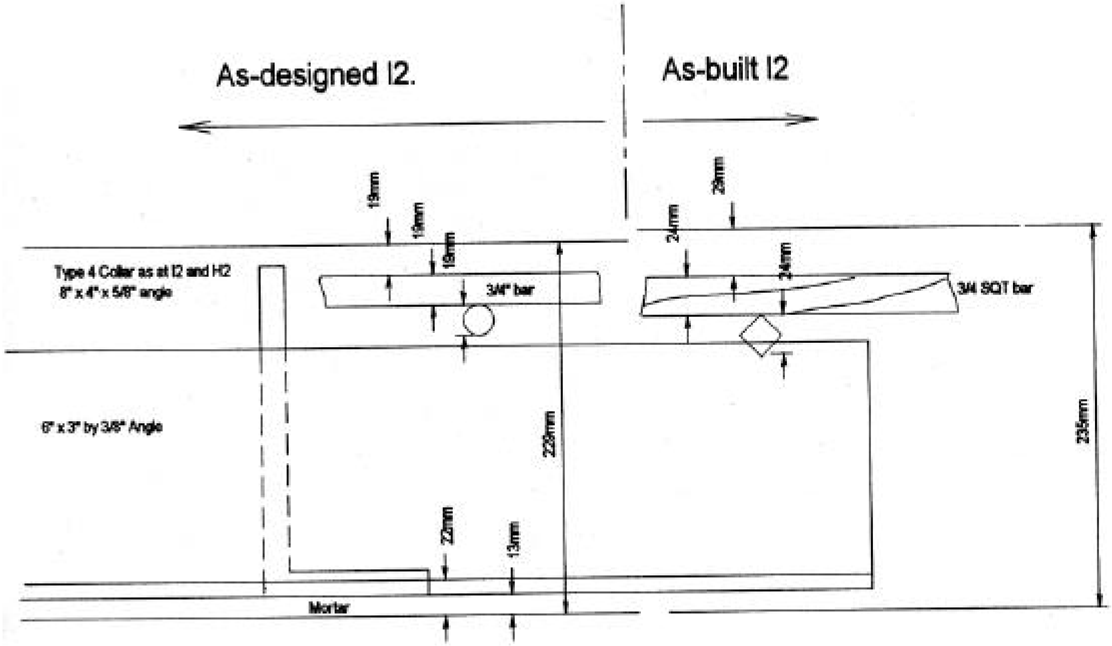


Figure 8.3.9 - 2 As-designed and as-built cover to top reinforcement

8.3.10 Repair dimensions.

The size and location of the repairs at H2 , J2 and IJ-23 were recorded by HSE after the collapse [H137] and are shown on Figure 8.3.10 - 1. Thickness of the retained samples of repair material were measured by Amey Vectra and are recorded on the drawings in their report [H214] as follows:

Table 8.3.10 -1 Repair sample dimensions.

Ref.	From	Size mm	Thickness Range	Thickness Average
S2Ba	H2 Repair Large	1320 x 850	50 to 90	73
S2Bb	H2 Repair Small	1300 x 750	50 to 70	55
2Da	J2 Repair	650 x 400	70 to 90	79
2Db	J2 Repair	460 x 420	50 to 85	66
2Dc	J2 Repair	460 x 320	45 to 65	58

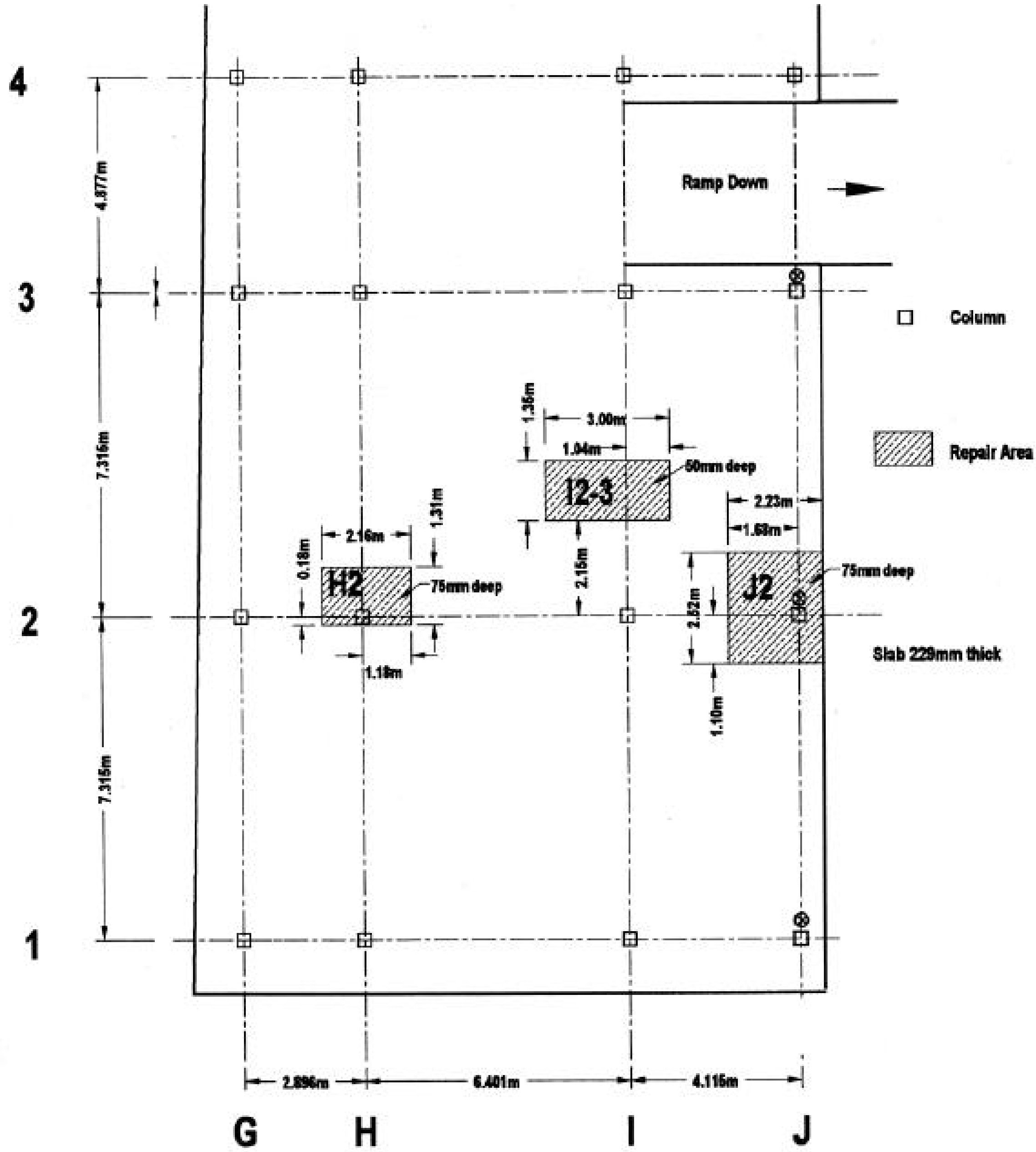


Figure 8.3.10 - 1 Location of repairs

8.4 QUALITY AND CONDITION OF CONCRETE AND REPAIR

8.4.1 Degradation of the Slab Surface.

Sandbergs [H1] and BRE [H219] have carried out and reported on detailed petrographic examinations of samples from the concrete slabs recovered from the collapsed area and slab S20 from the 3rd floor below the collapsed area. The Sandbergs examinations identified the layer of surface degradation and the contribution of frost to its development. The BRE petrographic examination was planned to resolve the uncertainties identified by Sandbergs and to check for other features of the concrete and degradation which could have contributed to the loss of strength. This section reviews the implications of their findings on the degradation of the concrete and the development of the progressive collapse.

The mixing of the concrete, its composition and its compaction was found to be highly variable. This is the primary reason for the high variability in stiffness and strength for the concrete below the degraded surface layer which was apparent from the materials testing.

The degradation of the concrete was variable in extent and depth. There were substantial areas where the surfacing waterproofing layer remained adhering to the concrete slabs. In these areas there was minimal surface carbonation (~1 mm) indicating that the surfacing was applied early in the life of the structure and had remained intact. There was no frost damage in these areas and the strength characteristics of the concrete and its bond to the reinforcement would not have degraded with time.

There were large areas where the surface waterproofing layer had broken down and water had penetrated the porous low strength concrete. This lead to the progressive development of frost degradation. This initially superficial concrete degradation spread as the surrounding area of waterproofing progressively broke down. Water collecting in the slight depression would have saturated the concrete making it more vulnerable to frost and deepened the deterioration. The cycles of drying and wetting would have concentrated salts and then leached them out.

Where this degradation had developed to the level of the reinforcement, corrosion would have initiated. The further degradation with the corrosion would have progressively destroyed the strength and bond of the concrete to the reinforcement. This degraded concrete was partially carbonated in the porosity and fissures for its depth. There was a relatively thin transition layer from degraded concrete to the un-degraded concrete of the lower part of the slab.

Urea prills were used in place of rock salt for deicing at Pipers Row for over 10 years before the collapse. The sagging of the structure lead to ponding and potholes formed where concrete degraded. There would have been evaporative concentration of urea in these depressions. When urea breaks down it releases carbon dioxide. This breakdown, which occurs over a period of weeks, may well have contributed to the depth and severity of carbonation in the degraded areas which lead to corrosion.

As urea is concentrated by evaporation, it can recrystallise in the surface porosity of the concrete degrading it in a similar way to frost deterioration. This may have contributed to the degradation at Piper Row. The research [25, 26]on the use of urea notes the risk of crystal growth degradation, but indicates that this is not a problem where concretes are of good quality >40N/mm^2 and where rain will wash the urea off the structure. These conditions apply to most bridges where Urea is widely used for deicing, but not at Pipers Row where ponding developed due to the creep of the slab and waterproofing deteriorated to expose the concrete to Urea concentration.

The effect of the degradation of the surface layers of the concrete on the punching strength around columns was critically determined by the relationship of the depth of degradation to the depth of the two layers of top reinforcement. Depth of reinforcement was variable with the centroids of the T1 and T2 bars being typically in the range 30 to 50mm and 50 to 70mm respectively. BRE and Sandbergs have shown the nature and variability of depth of degradation in the sampled areas. However the critical areas were the anchorage zones around the shear perimeters at H2, I2 and J2 where samples were not recoverable.

The BRE studies on carbonation and corrosion indicate that it was the fissuring of the degraded concrete which enabled carbonation to develop to initiate corrosion. Corrosion of reinforcement indicates degradation and fissuring at least to the depth of the corrosion. The locations of corrosion on top steel apparent in the photos taken after the collapse therefore give the best indications of the degradation depths and areas where bond strength would have been severely reduced. Figures 7.4 - 2 and 8.4.1 -1 of reinforcement from around H2 and Figure 8.4.1 - 2 of I2 clearly show that this corrosion had developed round the T1 and onto the lower T2 top reinforcement.

Figure 8.4.1 - 1 Corrosion indicative of fissured and degraded slab at H2

Figure 8.4.1 - 2 Corrosion indicative of fissured and degraded slab at I2

The structural significance of this for punching shear strength is considered in detail in Section 10.4.6.

Some indications of ASR and ettringite formation were noted in the BRE petrographic examination, but not at a level which would have significantly contributed to the deterioration of the concrete. Checks did not indicate any other potential causes of the degradation.

The petrographic examinations did not show dusty aggregates to have caused particular problems but cement aggregate bond was not good. Sandbergs noted that the high silt fraction may have increased the water demand leading to greater frost susceptibility.

8.4.2 Overall features of corrosion

Corrosion of reinforcement in concrete is initiated when:

Carbonation of surface layers reduces the alkalinity so that the protective passivation of steel surface is lost

or chlorides from calcium chloride or from the ingress of chloride in road salts destroys the passivation.

Once corrosion starts there is a loss of area of the reinforcement and as generally the volume of rust is greater than that of the steel corroded, expansion can spall off the surface concrete destroying the bond essential for structural effectiveness. At Pipers Row the depth of corrosion found on the top reinforcement bars was never sufficient to reduce their tensile strength by loss of area. In some locations, eg around H2 and I2, the corrosion was clearly acting with the frost degradation to crack and delaminate the reinforcement from the slab. However the indications are that the loss of bond was primarily the result of the surface breakdown of the concrete which permitted secondary corrosion, as distinct from the more usual corrosion initiated spalling followed by a breakdown of the concrete.

Most of the top reinforcement exposed by the collapse and in the examination of the retained samples was free of signs of corrosion. However, areas of corrosion of the top reinforcement are clearly apparent in photographs (eg HSE P3-7 of H2 and P3-11, 16, 17 of I2) taken immediately after the collapse adjacent to H2 and I2. The sets of photos also show corrosion of top reinforcement at some other locations. Similarly the repair at I2-3 adjacent to I2 showed deep degradation which must have extended into the area around it. The visible corrosion and fragmentation apparent on photos taken after the collapse show that this deterioration extended into the area around I2.

Some superficial corrosion of reinforcement would have developed over the two years between the collapse and the BRE study. During this period the slabs were stored in a car park in Birmingham. The more severe areas of corrosion which predated the collapse have been identified on the bars recovered. These have been subject to detailed examination by BRE.

8.4.3 BRE detailed examination of corrosion of top reinforcement

The Sandbergs and the BRE testing detected no significant chlorides in the concrete around the top reinforcement of the slab or elsewhere in the slab. There were no signs of chlorides arising from Calcium Chloride which reports indicate may have been used in some of the original concrete.

However, BRE's detailed analysis identified chlorides in significant quantities locally in the corrosion deposits. Checks on the use of deicing salt in the car park show that the use of rock salt (NaCl) was discontinued in the mid 1980s, since when Urea Prills have been used. It is possible that the chlorides in the corrosion deposits remain from the early period of NaCl use with the chloride having been leached from the surrounding degraded carbonated concrete around the steel over the intervening years. The research literature [26] on the use of urea as an alternative to NaCl for deicing bridge structures states that urea does not itself lead to corrosion initiation in the reinforcement.

8.4.4 Detailed examination of corrosion and degradation of the slab soffit

Various reports, since about 1988, on the condition of the structure and repairs indicate that some areas of the soffit of the slab was locally deteriorating with spalling along the lines of the bottom reinforcement. Sandbergs inspection immediately after the collapse also reported this.

The Sandberg and BRE data shows soffit carbonation depths typically in the range 10 - 20mm, which in the damp of the car park induced corrosion in the bottom reinforcement which had covers of typically varying from 10 to 30mm.

The patch repairs of these spalls seem to have been superficial and would be unlikely to have re-established fully effective bond of the bottom reinforcement. Where this spalling and de-bonding was extensive, the flexural behaviour of the slab could have changed with redistribution of moments to hogging zones over supports. The extent of soffit corrosion in the area of collapsed slab would not have had a significant effect on slab moments or have influenced the initial failure.

8.4.5 Repair Material and Interface

The proprietary repair material used by CDM was reported by them to be based on Flexcrete Admixture 850 with 'pea shingle' added in place of the 6mm granite recommended by Flexcrete. The as-cast material is described in the Sandberg [H1] and BRE [H219] reports. It was mixed from a pre-pack of cementitious materials and admixtures with crushed flaky coarse aggregate to 6mm. This material was reasonably well mixed, but with a high content of entrapped air and with some segregation giving a varying aggregate/cement ratio through the depth.

The detailed examination of the repair to substrate concrete interface is reported by BRE and by Sandberg and show that the adhesion of the repairs to the original concrete was not of a standard required for a structural repair. The condition of the available samples did not permit the use of pull off tests to check interface strength.

The implications of the repair characteristics and the features of the interface identified by petrographic examination have been discussed in Section 8.2.5 and the structural effects are considered in Section 10. 4.6, 10.5.4 and 10.5.6.

SECTION 9. LOADING AND ANALYSIS

9.1 INTRODUCTION

In this section the basis for the CP114 and BS8110 design rules for reinforced structures are briefly reviewed. Those features of the Pipers Row top floor slab around columns H2, I2 and J2 which influenced the risk of failure, either by increasing load effects, Section 9.3, or by reducing strength, Section 10, are identified and quantified. The magnitude of these influences have been subject to detailed analysis under the Amey Vectra and BRE contracts and are more fully covered in their reports [H209 - 215] [H217, 218]. The results and interpretation by SS&D of these detailed analyses are summarised below.

9.1.1 Conventions of Design and Appraisal

Reinforced concrete design has evolved on the basis of convenient, robust simplifications. These were embodied in CP114: 1957 [27] , which was used by British Lift Slab to design Pipers Row Car Park [H190 - 194]. With the growing body of experimental data on reinforced concrete behaviour and drawing on the lessons from the partial collapse of Ronan Point and other structures, a more detailed, comprehensive and complex code of practice CP110 [28] was issued in 1972. This was further developed and refined into BS8110: 1985 [29], with amendments to No 4 in September 1993 and was reissued as BS8110: 1997 [30] , with no significant change relevant to flat slabs but renumbered.

CIRIA 110 [24] reviewed the developments in BS8110 in relation to flat slab design. BS5400 [31]for bridge structures has been developed following the same approach as BS8110, but with some more specific provisions on flat slabs.

The IStructE report 'Appraisal of Existing Structures'[32] sets out the principles to be followed when evaluating the strength of old structures where design does not comply with current standards and/or deterioration has developed. There are Highways Agency recommendations [33] for assessing the strength of old bridge structures, which enable the particular features of the structure to be taken more fully into consideration. SCOSS has recommended [6] that bridge engineering procedures and explicit consideration of progressive collapse, are appropriate for appraising car park structures.

The six main aspects of the design process which influenced the reserves of strength at Pipers Row were:-

- The loading used for design
- The factors of safety applied to the load γ_f and strength γ_m in the calculations.
- The way in which the analysis of the overall structure apportions shears, moments and reactions to the elements of the structure.
- The design of the structure for the flexural bending moments in the slab and columns.
- The design of the slab around the columns for shear.
- The requirements for robustness and ductile behaviour against progressive collapse.

The failure at Pipers Row initiated from punching shear around column H2, I2 or J2, so the analyses in this report focuses on these locations. Flexural analysis of the slab, which dominates the design process, has been comprehensively carried out by H&S [H2 - 9]and by Amey Vectra [H210 - 213] and is dealt with in detail in their reports. The influence of this flexural behaviour, and local anomalies in flexural design identified by H&S and AV, on the distribution of column reactions and moments and punching strength has been considered in this report. The influence of flexural behaviour on the initial spread of the collapse and then in limiting it has also been considered, but other aspects of flexural behaviour are not discussed in detail.

9.1.2 Factors of Safety

In CP114 design was based on nominal loads and nominal permissible stresses which had been evolved to provide an overall factor of safety against ultimate failure of over 1.7. In BS8110, as discussed in its forward, and BS5400 this has been replaced by explicit factors of safety on both load γ_f and material strength γ_m. The magnitudes these factors reflect the variability of the load or strength under consideration. The fundamental basis for this was set out in CIRIA Report 63 [34] aimed at rationalising the factors to give a consistent level of the risk appropriate for the function of the structure. In practice the BS8110 factors have only marginally changed the actual reserves of strength in structures, as the new partial factors were largely calibrated to produce designs similar to those to CP114.

At Pipers Row at the time of collapse the ultimate limit state condition was reached. It is necessary to consider if the factors of safety in design and appraisal are sufficient for this brittle mode of failure. This has been done by comparing design assumptions with the loads at the time of failure, the as-built strength and the extent to which it had been eroded by deterioration.

Load factor γ_f

BS8110 Clause 2.4.1.3 states that γ_f is to *"take account of*

> *unconsidered possible increases in load,*
> *inaccurate assessments of load effects,*
> *unforeseen stress redistribution,*
> *variations in dimensional accuracy and*
> *the importance of the limit state being considered".*

BS8110 Table 2.1 uses $\gamma_f = 1.4$ for dead loads and $\gamma_f = 1.6$ for imposed live loads for load combination 1 with reductions in combination with wind. Temperature effects, misfit or foundation differential settlement are not explicitly considered. The γ_f is deemed to cover these effects and the range of normal tolerances and seems to have been calibrated on the basis of in-situ concrete construction with ductile details to enable local overstress to be safely distributed.

For Pipers Row with:

- the possibility of misfit at supports,
- BS8110 simplified analysis which can underestimate the most adverse reactions
- brittle punching shear details
- and a progressive failure limit state

the use of the standardised γ_f results in inherently greater risks.

BS5400 for bridges adopts a more comprehensive approach to partial factors with temperature and misfit being explicitly considered in load combinations.

Materials factor γ_m

The materials factor γ_m in BS8110: 1997 Clause 2.4.2.2 takes into account:

> *"differences between actual and laboratory values,*
> *local weaknesses,*
> *inaccuracies in assessment of the resistance of sections,*
> *the limit state being considered".*

Strangely in BS8110:1997 Table 2.2 γ_m is:

1.05 for steel reinforcement, a well controlled material with a ductile failure mode. It was 1.15 until Amendment No. 6 incorporated in 1997.

1.5 for concrete in flexure and axial load,

but is only

1.25 for shear strength without shear reinforcement, the mode of failure at Pipers Row which was sudden and brittle.

The factors do not include any explicit allowance for loss of strength from deterioration, as the general perception with concrete was that strength gain with time would enhance reserves of strength. In the following sections the magnitude of each of these elements underlying γ_f and γ_m are evaluated.

9.2 LOAD EFFECTS ON COLUMN REACTIONS, MOMENTS AND EFFECTIVE SHEARS

9.2.1 Introduction

Pipers Row car park was designed to CP114 for the self weight of the slab, an allowance for surfacing and an equivalent uniformly distributed load representing parked vehicles and other loads arising from its use. The secondary influences on the distribution of loads and moments in the structure from temperature effects, differential settlement of foundations and tolerances in construction were generally considered to be covered by the overall factor of safety when CP114 was used for in-situ construction.

In BS8110 the treatment of loading is very similar. However Clause 2.4.1.4 draws attention to the need to consider the effects of construction. BS5400 for bridges, requires a wider range of load combinations including thermal effects to be considered explicitly, with varying factors of safety applied. For appraisal the IStructE 'Appraisal of Existing Structures' [32] and the Highways Agency appraisal rules [33] allow a more detailed evaluation of actual loads and where appropriate, reductions in load factors.

Figure 9.2.1 - 1 sets out as a table the loadings and load factors for the ultimate limit state which apply to the top floor slab of Pipers Row under CP114, BS8110, BS5400 and those secondary factors which have a load effect which are not explicitly considered in those codes.

										Actual
			BS8110 Design			BS5400		BS5400		Loading
	CP114 Design		Combination			Combination 1. HA		Comb. 3 Temperature		Pipers Row
	lb/ft2	kN/m2	kN/m2		yf	kN/m2	yf	kN/m2	yf	20/03/1997
Dead load: structure	113	5.41	5.41	Gk	1.4	5.41	1.15	5.41	1.15	5.3
Dead load: surfacing	17	0.81	0.81	Gk	1.4	0.81	1.75	0.81	1.75	0.1
Live load		2.4	2.5	Qk	1.6	2.5	1.50	2.5	1.25	none
Differential temperature in top slab.		nc	in yf						1.3	yes
Temperature restraint		nc	in yf						1.0	yes
Erection misfit height		nc	in yf				ea		ea	yes
Erection misfit one wedge high		nc	in yf				ea		ea	yes
Foundation settlement			in yf				ea		ea	unlikely

nc not considered ea Engineer to assess. Cl.5.5

Figure 9.2.1 - 1. Load Combinations for ULS, excluding wind and snow.

Reference dead load, *RefDL,* ***for comparison of analyses.***

Immediately prior to the collapse the slab was unoccupied, so the reactions resulted from the slab self weight (dead load) only. A 'Reference Dead Load (*RefDL*)' load case has been used for comparisons of load effects and analytical approaches. *RefDL* is based on measured properties for self weight, perfect fit, uniform temperature with column reactions and moments, taken as the average of the Amey Vectra (AV) ANSYS pinned (AV LC1) lifting case and the column head fixed to slab case (AV LC5), representing the applied deformations resulting from the creep in the slab.

For *RefDL* the BS8110 effective shear force V_{eff}, has been determined from the unfactored values of V, M_{tx} and M_{ty} from which the V_{effx} and V_{effy} have been derived at the column locations. This ANSYS plate analysis has been used for the reference against which other methods of analysis are compared, as it's idealisation and elements are closer to the real structural form than the sub-frame and grillage alternatives.

In calculating $V_{eff} = V_t(1 + 1.5M_t/(V_t\ x))$ a reference set of values of x, the width of perimeter face, have been used, based on the full angle dimension plus 1.5d on each side, with as-designed slab dimensions (229mm depth, 19mm cover, 19mm T1 and T2 bars and seating 22mm above soffit, giving effective depths d of 178mm and 162mm). For the comparisons of relative load effect at J2 comparisons are based on the calculated V_{eff} as well as the $1.25V_t$ simplified value. The sensitivity of V_{eff} to the range of assumptions influencing the perimeter dimensions are further discussed in Section 10.3.

9.2.2 Dead Load

The dead load (self weight) G_k of the slab has been checked using the actual thickness and density of the slab and surfacing (see 8.3.7 and 8.3.8) . The dead load of the concrete slab was 5.30kN/m^2 compared to the (5.41kN/m^2, 113lb/ft^2) assumed in the original design. The surfacing was thinner than that originally considered giving 10kg/m^2 (0.1kN/m^2) compared to the 83kg/m^2, (0.81kN/m^2 , 17lb/ft^2) allowed for in the design. This gives a total slab plus surfacing dead load of 5.40kN/m^2 compared to the 6.22kN/m^2 which was allowed for in design. The lower floors were unsurfaced at 5.30kN/m^2. The weights of the edging kerb, drains and barriers were small and have not been included in the analyses.

At the time of collapse, there were no vehicles present, so the loading on the top slab was only 87% of the design dead load. The factors influencing the distribution of dead load reactions and moments between columns from the building tolerances, creep and thermal effects are considered below.

For the 'Reference Dead Load' (*RefDL*) case 5.40kN/m^2 has been applied to the Amey Vectra ANSYS plate analysis of the slab. The details of this analysis are set out in the Amey Vectra Contractors Report section 1300-218-C501 [H215].

The slab was lifted into place with simple support on the wedges at the columns, which were then 'fixed' with a mortar infill to create a moment connection. The column head rotations from the surfacing and the creep of the asymmetric spans over time would have transferred moments into the columns, redistributing the shears around the shear perimeter. The *Ref DL* case has been based on the average of the lifting case (LC1)and the case with the column fixed to the slab (LC5), adjusted to 5.40kN/m^2. The *RefDL* stresses are the best estimate of those in the top slab of Pipers Row Car Park at the time of collapse, if there had been perfect fit during construction, no temperature differentials and the design material properties had been achieved in an uncracked and undeteriorated slab.

9.2.3 Live Loads

The original design live loading Q_k for the car park was 50lb/ft^2 (2.4kN/m^2), compared to the 2.5kN/m^2 in the current BS6399:Part 1, which is valid for vehicles of up to 2500kg. This blanket loading is applied in BS8110 as a uniformly distributed load over the whole floor area with checks on loading on alternate spans and for unloaded cantilevers where the structure departs from the criteria on uniform layout. This covers loadings from moving and parked vehicles with some allowance for loadings which can arise from crowd loading, contractors equipment and materials during construction and maintenance, etc..

Actual vehicles

To determine the sensitivity of Pipers Row to the normal range of vehicle loadings a small sample survey of manufacturer's data on vehicle weights and dimensions was carried out by SS&D. This showed that vehicles, including a reasonable allowance for their contents, could be simplified as:

	'small'	at 8.8kN, (typically 1100cc)
	'medium'	at 12.8kN, saloons and estates (typically 2000cc)
and	'large'	at 21.6kN, including large sports utility vehicles, Range Rover, Transit, Jeep

The loading from the 'medium' vehicles was applied as 3.2kN wheel loads in the Amey Vectra ANSYS plate analysis as indicated in Figure 9.2.2 -1. It was split into

V1, for vehicles in the 8 marked bays from lines 1 to 2,
V2, for the 4 marked bays between lines 2 and 3,
V3, for 2 waiting vehicles between H and I.

For 'small' vehicles the loading can be factored by 0.69. For 'heavy' vehicles a factor of 1.69 would be required.

The effect of vehicles on the column perimeter shears depend on the direction of the column head moments relative to the other load effects. In summarising the most adverse load effects the V_{eff} in both x and y directions have been checked.

Vehicle Load Cases for Plate Analysis.

V1: 8 Parked cars between Lines1 & 2.
V2: 4 Parked cars between Lines 2 & 3.
V3: 2 waiting cars between Lines H & I.
3.2 kN per wheel for 'Medium' Vehicles

Factor by 0.69 for 'Light' cars.
Factor by 1.69 for 'Heavy' Cars

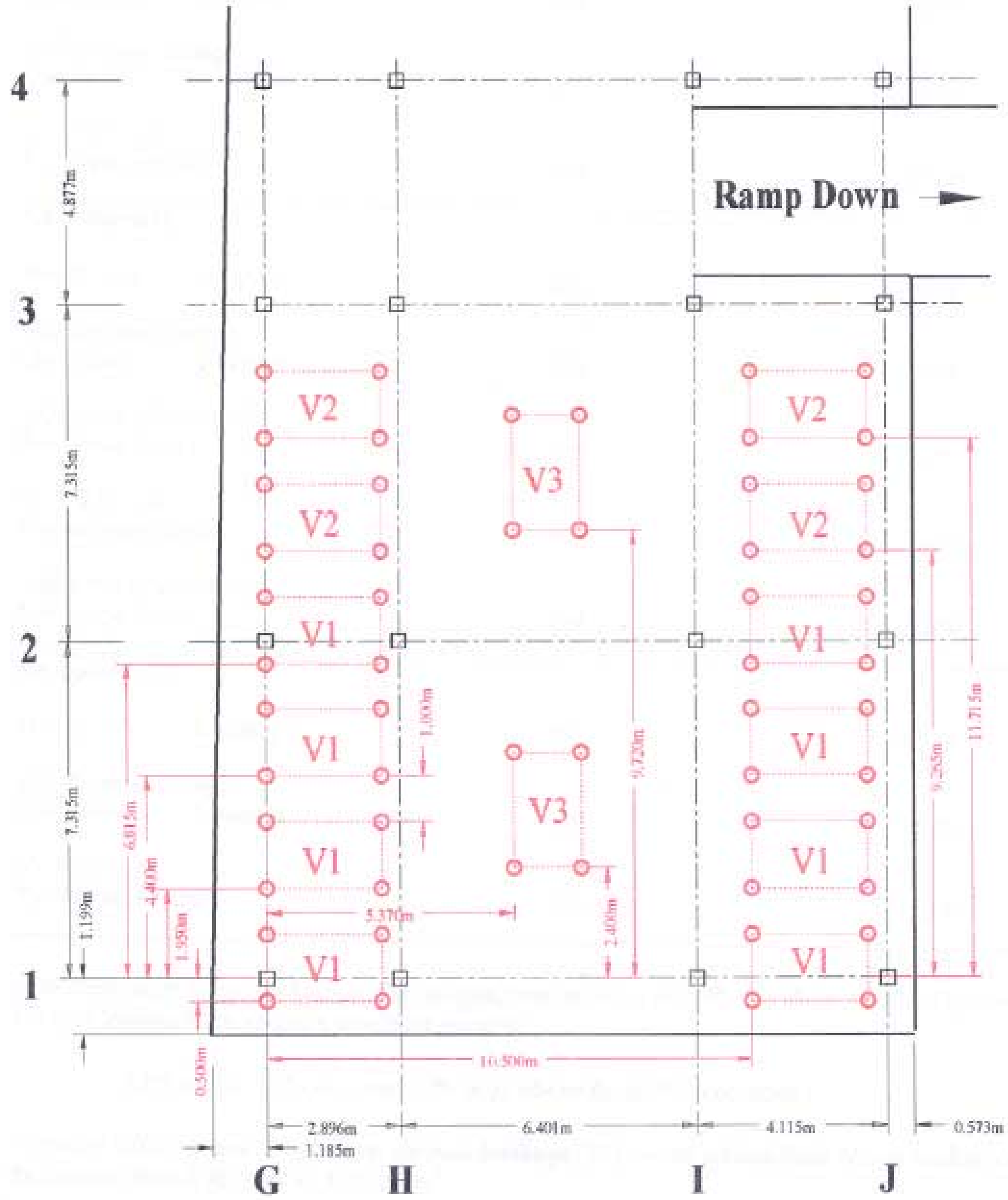


Figure 9.2.2 -1. Vehicle loading in bays on slab.

Table 9.2.2 - 1
Increase in V_{eff} and % change relative to the *RefDL* from vehicle loading

Condition	Unfactored (γ_f =1.0) *RefDL* + Vehicles V_{eff} kN.	Most Adverse V_{eff} % of *RefDL* %
For Column H2		
RefDL only 5.4kN/m^2	244	100%
LC7 Original Design Live Load 2.4kN/m^2	350	143%
V1 + V2 + V3 Full Medium Cars	276	113%
for Column I2		
RefDL only 5.4kN/m^2	275	100%
LC7 Original Design Live Load 2.4kN/m^2	393	143%
0.69 x (V1 + V2 + V3) Full Small Cars	295	107%
V1 + V2 + V3 Full Medium Cars	304	110%
1.69 x (V1 + V2 + V3) Full Large Cars	324	118%
for Column J2		
RefDL only 5.4kN/m^2	94	100%
LC7 Original Design Live Load 2.4kN/m^2	136	145%
V1 + V2 + V3 Full Medium Cars	129	137%

In normal usage in the weeks up to the collapse, until it was coned off, it is likely that the V_{eff} from the cycle of loading from vehicles would range up to:

13% at H2, 10% at I2 and 37% at J2 above the *RefDL* condition.

A recent ASCE review of American car park loadings [35] concluded that their design loading could be reduced from 2.4kN/m^2 to 1.91 kN/m^2.

9.2.4 Thermal Effects.

The collapse of the car park slab in the middle of the night, without vehicle loading on it, suggested that the thermal conditions over the period leading up to the collapse may have redistributed the reactions and moments on the columns sufficiently to trigger the collapse.

Figure 9.2.4 - 1 shows the air temperatures over the month of March 1997 up to the day of collapse.

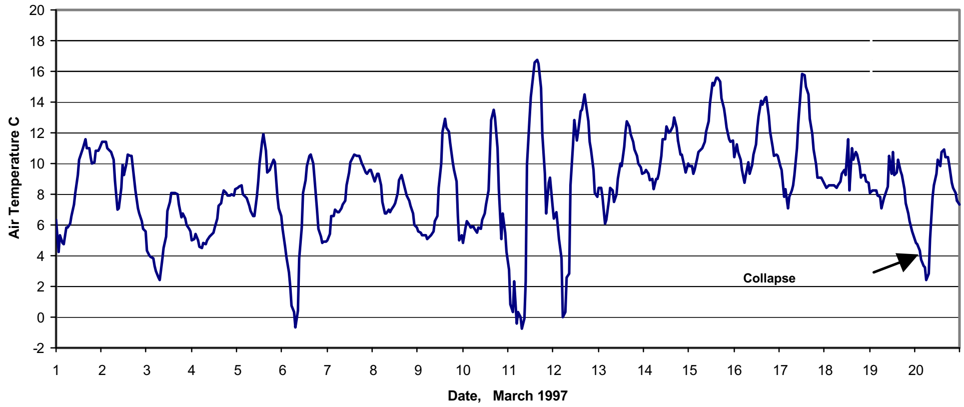


Figure 9.2.4 - 1. Air temperature March 1997, Birmingham Airport

Figure 9.2.4 - 2 shows how, during the night 19th - 20th March 1997, the cloud cleared, the wind fell to 3m/s and the air temperature dropped to 4°C at 3am, the time of the collapse. This data is based on the data from Elmdon, Birmingham Airport, the nearest Meteorological Office station. Conditions at Wolverhampton, 18miles NW of Elmdon would have been slightly different. Meteorological Office figures give the wind at 10m above an open surface and a standard dry bulb temperature in a screen.

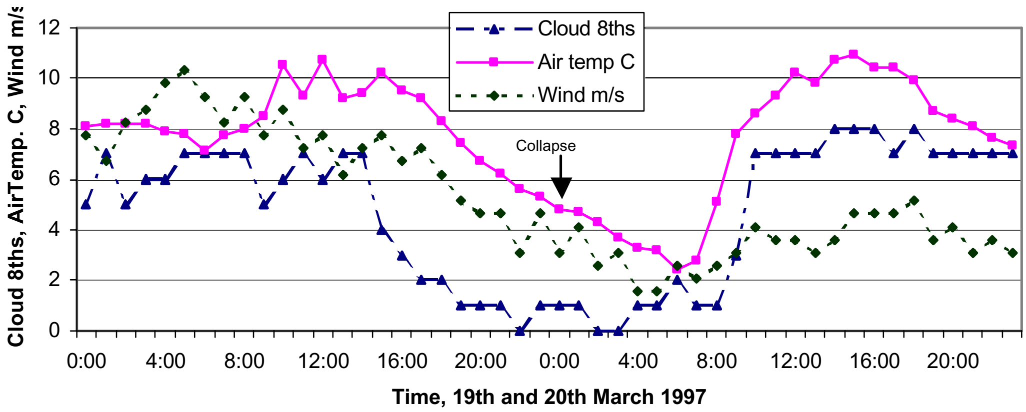


Figure 9.2.4 - 2 Meteorological Record for 19th - 20th March 1997,

The wind at Pipers Row slab level would have been substantially slower than at the 10m height. With a clear sky and a low wind the radiant heat loss from the top of the top slab would have cooled it to below air temperature. The soffit and the slab below it would have been sheltered from radiant heat loss and would have cooled more slowly lagging behind air temperature.

The thermal conditions have been evaluated for the period leading up to the collapse and for the differential temperature conditions which might be considered in design or appraisal. In doing this the following scenarios have been considered:

a) Frost action of water in saturated microcracks in the concrete or concrete/repair interface triggered failure.

b) The thermal redistribution of slab shears V_{eff}, increased stress to initiate punching cracking at a weak location, which then spread.

c) Thermal cycles of shear stress at close to the limit 'fatigued' the concrete to initiate failure when punching shear cracks reached a critical size.

d) The strains due to the thermal differentials between the repair and the concrete, acting with structural strains, progressively delaminated the repair from the concrete at H2 and/or J2.

The frost damage to the concrete has been clearly identified by BRE and Sandberg as a major factor in the development of the degradation of the concrete over the years before the collapse. However the air temperature in March 1997 only briefly dipped below freezing and, although radiant heat loss may have lead to some surface frost, it would not have penetrated more than a few mm. The week before the collapse was mild with temperatures above 6°C. Scenario a) of a frost triggered collapse can therefore be discarded.

To evaluate the effects of scenarios b), c) and d) during the period up to the collapse four approaches were compared:

i) For building structures to BS8110 temperature conditions are not normally explicitly considered in design. BS5400 for bridges contains differential and overall temperature requirements which relate to the extreme conditions considered in design, rather than the particular conditions leading up to failure at Pipers Row. They were included in the Contract B appraisals carried out by Amey Vectra.

ii) A simple robust analytical model was developed at TRRL in the 1970s and calibrated by comparison with site measurements on test slabs at TRRL and on bridges. This was used to derive BS5400 rules. This has been applied to the Pipers Row slab geometry for some particular weather conditions.

iii) More detailed thermal analysis by BRE using the advanced BRE APACHE software.

iv) Monitoring the actual temperature gradients in an exposed section of slab from Pipers Row with associated meteorological data.

BRE have reported in detail [H220] on ii) iii) and iv). The data and its implications for Pipers Row are summarised below.

Temperature Analysis

The analytical approaches ii) and iii) calculate the temperature differentials through the slab using models of the thermal behaviour of the slab subject to radiant heat loss to the night sky and solar gain during the day, combined with changes in air temperature and the effect of wind. These give a good basis for calculating the extreme cases, which arise with a dry slab, clear skies and no wind.

The influence of varying cloud cover and wind is more difficult to model, partially due to the limited availability of meteorological data on actual radiation variation during the day and night for various cloud conditions and on the relationship of surface wind to the standard meteorological record 10m value.

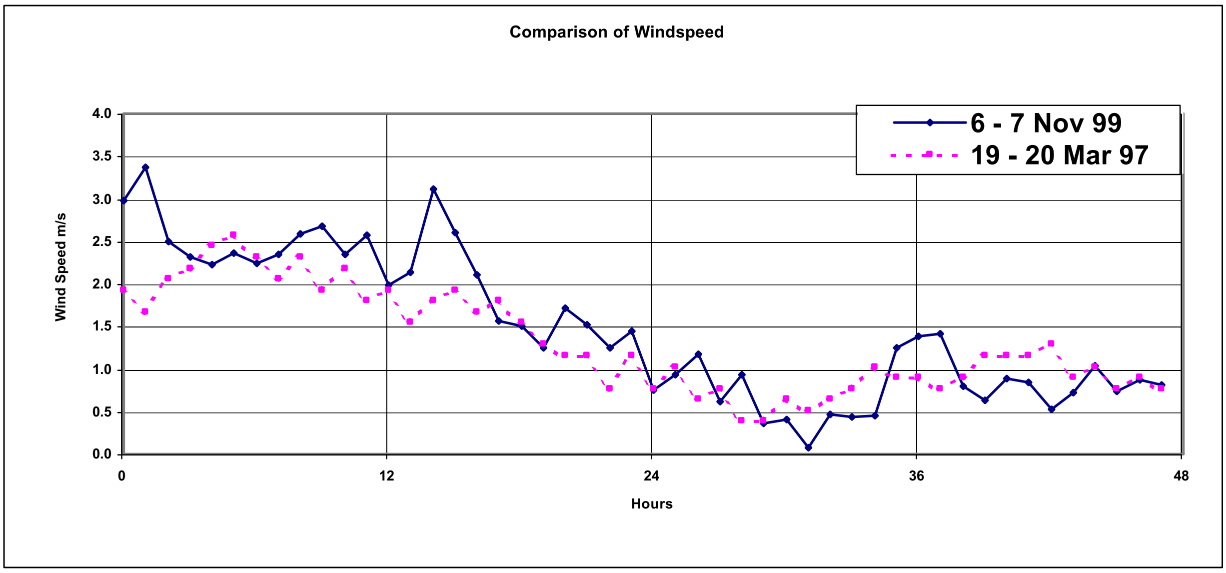
It has become apparent during this study that the major limitation of these analytical approaches arises from the thermal influence of rain and the condensation, evaporation or freezing of surface moisture layers. These processes are very sensitive to the wind speed over the surface and are not covered in currently available analytical models, nor is the input data of actual meteorological conditions available. For the period leading up to the Pipers Row failure they would have had a major influence.

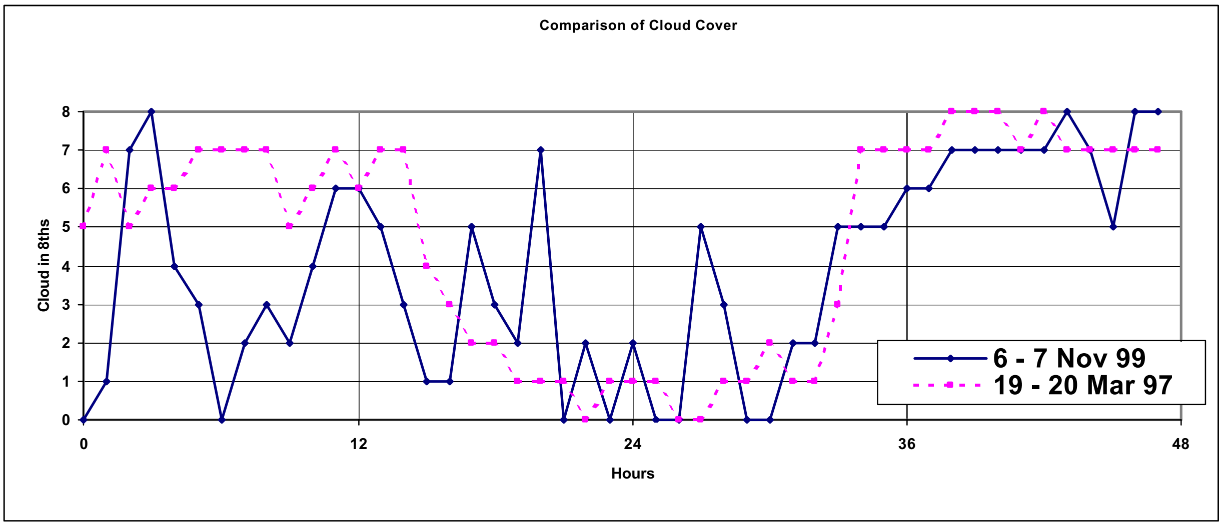
A measure of the magnitude of these effects can be judged from the following comparisons.

Change	Relative Heat Energy
Heating 229mm thick slab by 18C	100
Evaporating 1mm layer of surface water	475
Condensing 1mm layer of surface dew.	- 475
Freezing 1mm of dew or rain to ice.	- 65
Heat Loss in 1 hour to clear night sky	- 65
Heat Gain over noon hour in March	365

In March 1997 the weather conditions were in the range where these moisture effects would have substantially modified the thermal gradients. The photographs taken immediately after the collapse show water ponded on the adjacent top slabs. The moisture effects were a major factor in deciding that the measured profiles from the exposed slab S2 at BRE were the most reliable indication of the temperature conditions in the slab preceding the failure. While the existing analytical models could have been developed to cover the moisture factors it would have been outside the scope of this study on Pipers Row, particularly as very good data was becoming available from the instrumented slab.

At the start of the study it was considered that it would be necessary to use the monitored temperature profiles in the slab and associated meteorological conditions, to calibrate the APACHE model. This could then be run for the night time conditions of 19th - 20th March leading up to the collapse. By fortuitous chance the records for the night of 6th - 7th November 1999 showed an almost identical pattern of temperature, wind and cloud as those reported for the night of the failure. Figure 9.2.4 - 3 compares the meteorological records. While day time solar radiation gain varies substantially through the year the radiation to the clear night sky is a constant throughout the year so March and November overnight conditions are comparable.





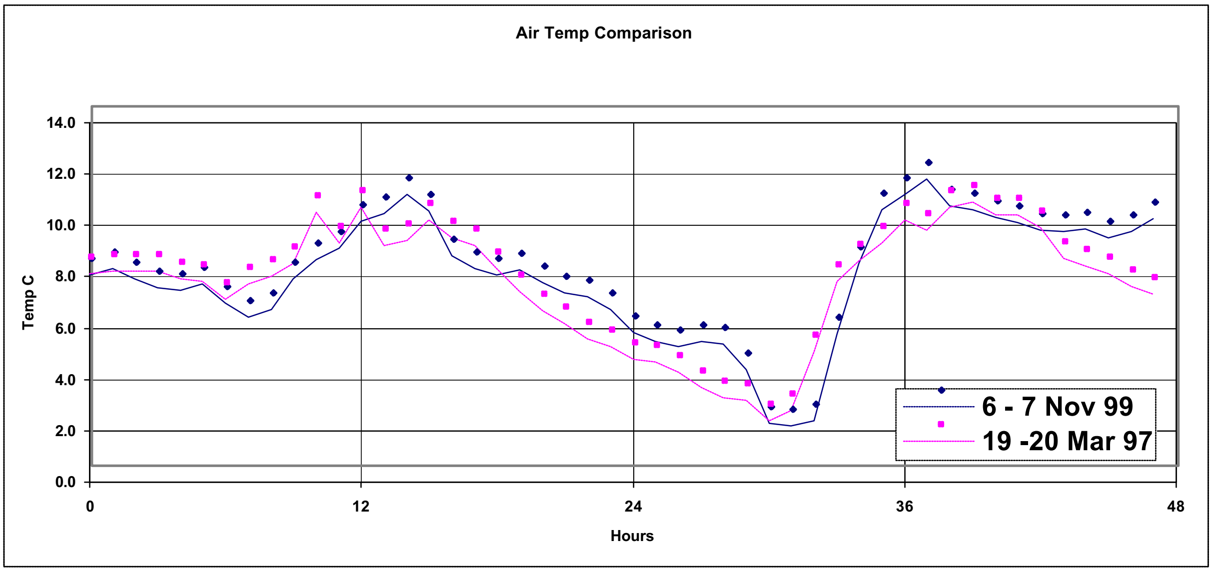


Figure 9.2.4 - 3

Meteorological records 19th - 20th March 1997 and 6th - 7th November 1999

The comparisons of the temperature profiles, for the recorded weather conditions at BRE for the 2 days 18th and 19th November 1999 calculated using APACHE analysis [H220] with known thermal properties of the slab and the site recorded temperatures showed substantial discrepancies. Although an improved comparison was obtained by using properties empirically adjusted to improve the fit, this required assumptions at variance from well established physical properties.

Because of the availability of the recorded slab data and the relatively low cost of extending the slab recording period to cover a wide range of conditions, no further development of the APACHE approach was undertaken.

BS5400

The principles of overall and differential temperature effects are well set out in BS5400 and have been considered in the Amey Vectra calculations [H213]. They give an overall air temperature 50 year range of 338C to -188C with a deck temperature range of 348C to -108C with a summer positive differential of 208C through the deck and 48C negative. However, the particular conditions of load effects from restraint from the temperature differential between the top and underlying slab are not explicitly covered in BS5400.

TRRL analysis and tests.

The research carried out at TRRL in the 1970s [36, 37] has been reviewed, including the data on the measured temperature gradients in slabs at TRRL and on bridges. These studies formed the basis of the BS5400 simplified rules. Mary Emerson (who carried out the earlier research and who retains the software) carried out for BRE under Contract E, runs for the extreme midsummer clear sky daytime conditions and the winter clear sky nightime conditions, for a slab similar to that at Pipers Row. The results for three of the time steps, including the most adverse, are shown in Figures 9.2.4 - 4 and 9.2.4 - 5. The idealised concrete slab was 245mm thick plus the top 5mm as ‘surfacing’, whose thermal properties are similar to concrete but with an adsorption characteristic for the darker surfacing. This compares with the 229mm nominal, 235mm average, concrete slab with ~5mm surfacing at Pipers Row.

The extreme differential temperature gradients from a fitted line to the curve of the profile are:

	Slab Average Temperature C 8	Differential Top to Bottom of Slab, C 8
Summer Max	26.4	21.2
Winter Min	-11.8	-4.4

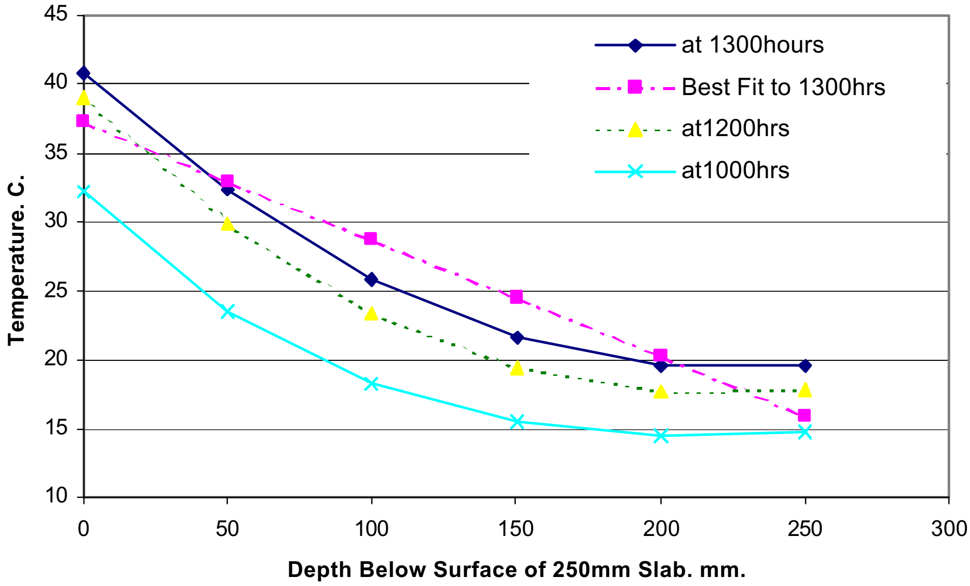


Figure 9.2.4 - 4 Calculated extreme day slab temperature profiles

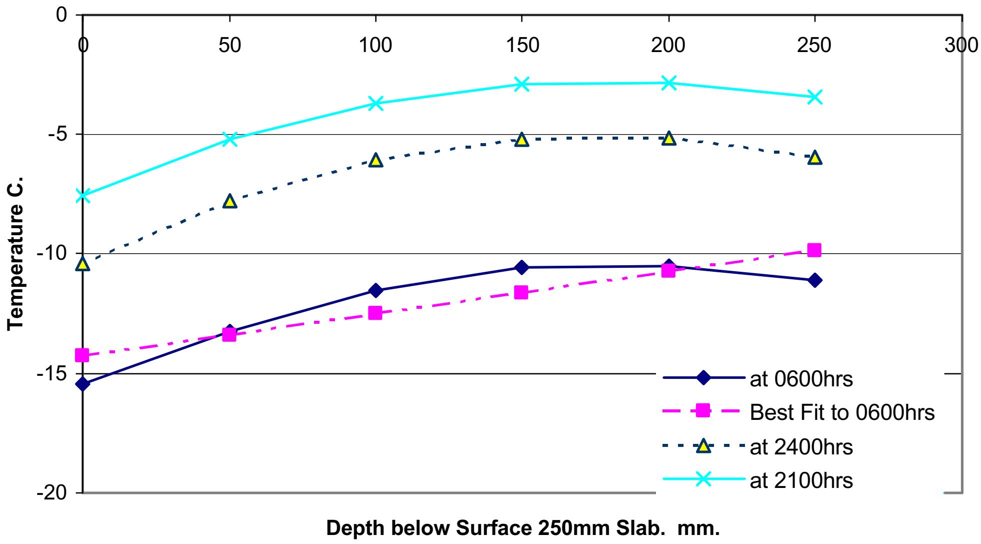


Figure 9.2.4 - 5 Extreme night slab temperature profiles

Instrumented Pipers Row Slab S2.

The instrumented slab S2 from Pipers Row was used to record temperature profiles, in conditions similar to those for the top slab at Pipers Row, as fully described in the BRE Report on Contract E [H220]. The slab S2 was one of the larger and less damaged sections with intact surfacing retained after the collapse and had been cored and tested by Sandbergs. It was mounted on blocks 680mm above a concrete ground slab in an open area on the BRE site. Figures 9.2.4 - 6 and 9.2.4 - 7 show the site and instrumentation. The core holes were refilled and the surfacing replaced over them. There was a slight fall to help drainage off the slab, but some water was retained by the roughness of the surfacing.

Figure 9.2.4 - 6 Slab S2 exposed at BRE for temperature recording

Figure 9.2.4 - 7 Surface of Slab S2 at BRE

Temperatures were recorded on the slab surfacing, on the concrete just under the ~5mm of surfacing and on the slab soffit. Within the 229mm thick slab temperature gauges were inserted and grouted in drilled holes at 25mm, 75mm, 112mm, 150mm, and 200mm below the top of the concrete, with further gauges at 100mm below the soffit and on the surface of the concrete ground slab 680mm below. The cloud data came from the nearest Meteorological Office site at Northolt. The BRE solar radiation measuring equipment was on the roof of an adjacent building and its output was combined with that from a small dedicated met station recording dry bulb, humidity and wind speed at slab level adjacent to the slab. A surface wetness gauge was fitted just above the slab, but it was noted that while it 'wetted' at the same time as the slab, it dried more quickly, as water was retained to a greater depth by the roughness of the surfacing.

These instruments fed a data logger. It was decided to run this continuously and sift the record for relevant periods, rather than recording only when conditions suggested a period of potential interest. A set of readings were logged every 10 minutes and down loaded weekly from 20th October 1999 to 31st March 2000. These were combined with the Met office and radiation data and then Emailed from BRE to SS&D were they were analysed. Figure 9.2.4 - 8 shows a typical hourly extract of the data set. For initial selection of periods of interest, the difference in temperature between the gauges 25mm below the surface and 25mm above the soffit were compared. From this sift, periods were selected, which were similar to the night of the collapse or which gave the greatest positive and negative differential gradients, which were then evaluated in detail.

The initial analysis of the raw data showed that the gradients through the slab were irregular and that during uniform stable temperature conditions there were unexpected variations between temperature readings. This suggested that there could be errors in the individual gauge calibrations and possibly some drift with time. Because the study was focussing on small differences in temperature, these errors were significant. BRE carried out a re-calibration exercise on the accessible gauges and duplicated some gauges.

To re-zero all the gauges the slab was sheltered and wrapped and the temperatures were monitored during a period with overcast steady temperature conditions to achieve uniform temperature for all gauges. These corrections have only been applied to the small set of data for the periods of greatest interest. To avoid problems from gauge datum drift the calibration was checked for a period of uniform temperature in stable cloudy conditions at night close to the time of each analysed subset.

Typical plots of the data have been prepared.

Figure 9.2.4 - 9	shows the two day cycle of temperatures on the full set of gauges and the cloud cover for the 6th and 7th November 1999, which closely matches the conditions for the 19th and 20th March 1997 leading up to the collapse.
Figures 9.2.4 - 10	shows the differential temperature gradient through the slab for 1am, 2am and 3am on 7th November 1999, similar to the conditions just before the collapse at 3am on 20th March 1997.
Figures 9.2.4 - 11	shows typical daily cycles of differential temperature in the slab for the period 13th March to 31st March 2000 for: the slab depth differential between 25mm below the top surface and 25mm above the soffit and the surface repair differential between 25mm and 75mm deep.

The records show that the 229mm slab with 5mm surfacing, exposed to the normal range of air temperature above and below the slab responds relatively quickly to changes in ambient temperature conditions. This limits the normal clear night negative differential temperature in the slab to about -1°C. The nocturnal differential temperature effects on an open car park roof slab are much less severe than in a building roof slab where the soffit is kept warm.

However the TRRL tests [37] in 24th Jan 1976 on 200mm thick slabs with 0mm, 54 mm and 104mm surfacing, with snow on it so it was kept cold above while being warmed by rising air temperature below, showed a 3°C negative differential temperature. This condition did not arise during the BRE monitoring or at Pipers Row in the month prior to the collapse.

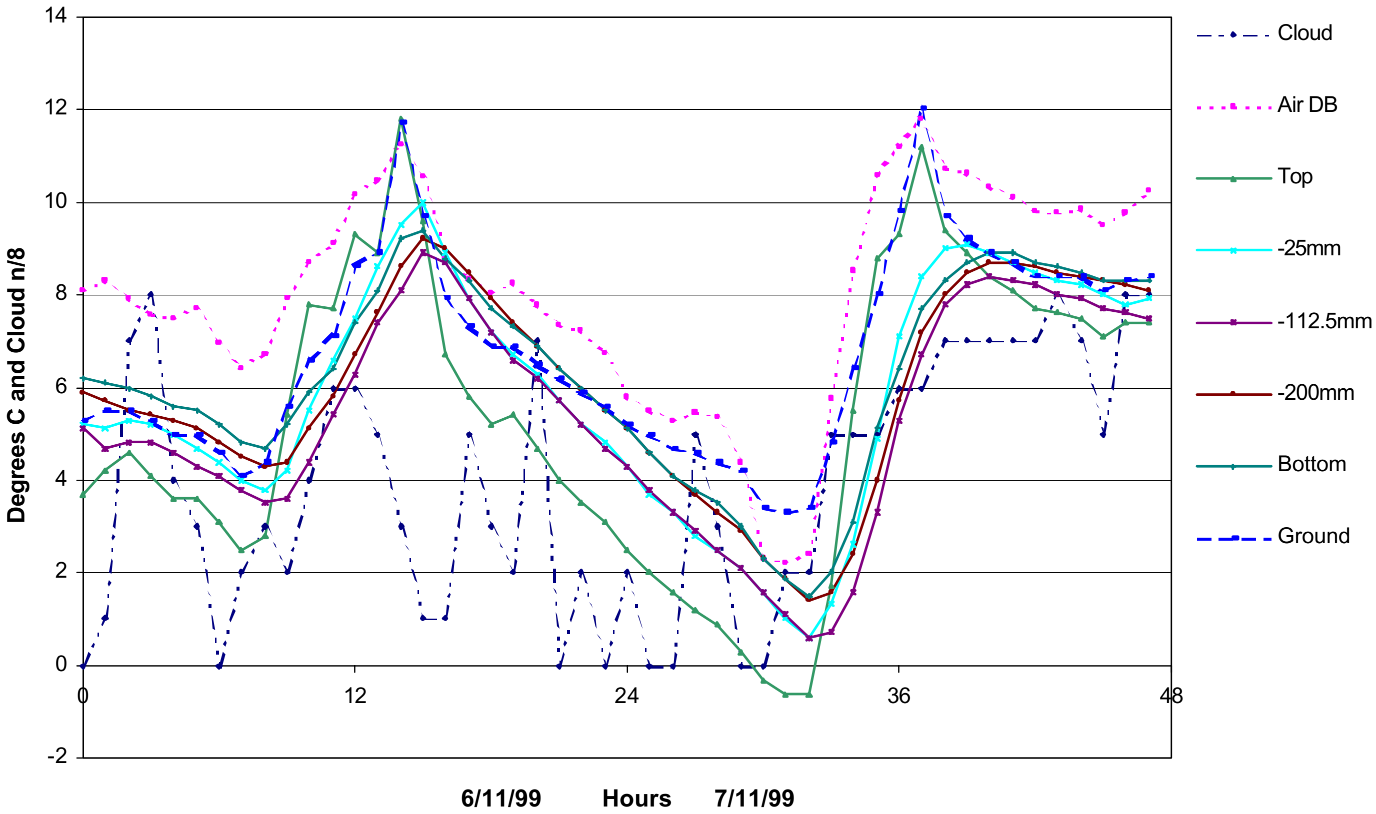
Two shock loading thermal conditions can arise, which being short term, have a mainly surface effect which is of particularly importance for repairs with a weak bond. The chilling effect when salt (NaCl) is added to ice can reduce surface temperatures to - 22°C, there is a similar effect with urea. This would not have been a factor at Pipers Row in the frost free period prior to the failure in March 1997. It could have adversely effected the repairs over the previous year. The other extreme negative differential temperature condition which can arise and is the basis of one of the tests for repairs in the Eurocode for concrete repair [22], is the hot summer day with a thunder shower of rain or hail.

Figure 9.2.4 - 8.

Contract E : BRE Meteorological and Temperature Data from Slab S2.

Time			<<<<<<<<<<<<<<<<<<Meteorological Data >>>>>>>>>>>>>>>>>							Slab Temperatures (DegC)										25mm below surface relative to 25mm above soffit
			Dry Bulb Ambient Shade Temp (DegC)	*Relative Humidity (%RH)*	*Wind speed*	*Solar Global on horiz Composite*	*Solar Diffuse on horiz Composite*	*Surface Wetness*	*Cloud Cover*	*Top of surfacing*	*Interface slab /surfacing*	*25mm down*	*75mm down*	*112.5mm down (mid-depth)*	*150mm down*	*200mm down*	*Slab soffit*	*100mm below soffit*	*Ground surface (680mm below slab*	Top Hot +
		Abbr.	DB0	RH1	WS	SR1	SR2	SW	CC	T1	T2	T3	T4	T5	T6	T7	T8	T9	T10	T3 -T7
GMT		Units	DegC	%RH	m/s	Watts/sqm	Watts/sqm		octals	DegC	DegC	DegC	DegC	DegC	DegC	DegC	DegC	DegC	DegC	DegC
		Level mm								-5	0	25	75	112.5	150	200	225	325	905	
Date Hr Min	Mins	Hours																		
01/11/1999 07:00	420	7.00	14.08	88	2.3	5	1	Wet	8	12.7	11.6	12.1	12.2	11.7	12.5	12.5	12.8	12.8	13.3	**-0.39**
01/11/1999 07:10	430	7.17	14.02	89	2.0	5	1	Wet		12.6	11.6	12.2	12.2	11.8	12.6	12.6	12.9	12.9	13.2	**-0.38**
01/11/1999 07:20	440	7.33	14.02	90	2.3	5	2	Wet		12.6	11.7	12.2	12.2	11.8	12.6	12.6	12.9	12.9	13.2	**-0.38**
01/11/1999 07:30	450	7.50	14.07	89	2.2	5	0	Wet		12.8	11.7	12.3	12.3	11.8	12.6	12.7	12.9	12.9	13.3	**-0.40**
01/11/1999 07:40	460	7.67	14.19	87	2.3	8	6	Wet		12.9	11.7	12.3	12.3	11.9	12.7	12.7	12.9	12.9	13.3	**-0.40**
01/11/1999 07:50	470	7.83	14.21	86	2.0	10	6	Wet		13.0	11.8	12.3	12.4	11.9	12.7	12.7	13.0	13.0	13.4	**-0.40**
01/11/1999 08:00	480	8.00	14.18	87	2.3	10	8	Wet	7	13.0	11.9	12.3	12.4	11.9	12.7	12.7	13.0	13.0	13.3	**-0.37**
01/11/1999 08:10	490	8.17	14.14	87	1.6	9	6	Wet		13.1	12.0	12.4	12.4	12.0	12.8	12.7	13.0	13.0	13.4	**-0.33**
01/11/1999 08:20	500	8.33	14.07	85	1.7	25	23	Wet		13.1	12.0	12.5	12.5	12.0	12.8	12.7	13.0	13.0	13.3	**-0.29**
01/11/1999 08:30	510	8.50	14.08	80	1.9	22	22	Wet		13.2	12.1	12.5	12.5	12.1	12.8	12.8	13.0	13.0	13.4	**-0.24**
01/11/1999 08:40	520	8.67	14.15	79	2.6	24	25	Damp		13.4	12.3	12.6	12.6	12.1	12.8	12.8	13.0	13.0	13.5	**-0.20**
01/11/1999 08:50	530	8.83	14.16	79	1.9	92	93	Damp		13.4	12.4	12.7	12.6	12.1	12.9	12.8	13.1	13.1	13.5	**-0.14**
01/11/1999 09:00	540	9.00	14.13	78	2.1	76	70	Damp	7	13.4	12.5	12.8	12.7	12.2	12.9	12.8	13.1	13.1	13.5	**-0.08**
01/11/1999 09:10	550	9.17	14.13	76	2.0	60	56	Damp		13.4	12.6	12.8	12.8	12.3	13.0	12.9	13.1	13.1	13.6	**-0.02**
01/11/1999 09:20	560	9.33	14.13	76	2.2	83	80	Wet		13.6	12.7	12.9	12.8	12.3	13.0	12.9	13.2	13.2	13.6	**0.01**
01/11/1999 09:30	570	9.50	14.20	76	2.3	92	89	Damp		13.9	13.0	13.0	12.9	12.3	13.0	12.9	13.2	13.2	13.7	**0.05**
01/11/1999 09:40	580	9.67	14.32	77	2.0	122	118	Damp		14.1	13.3	13.1	12.9	12.4	13.1	13.0	13.3	13.3	13.8	**0.10**
01/11/1999 09:50	590	9.83	14.41	77	2.0	124	122	Damp		14.3	13.5	13.2	13.0	12.5	13.1	13.1	13.3	13.3	13.9	**0.17**
01/11/1999 10:00	600	10.00	14.31	79	1.8	124	125	Damp	8	12.9	13.0	13.3	13.1	12.6	13.2	13.1	13.4	13.4	13.5	**0.22**
01/11/1999 10:10	610	10.17	14.02	82	2.2	87	82	Wet		12.5	12.6	13.4	13.2	12.6	13.3	13.2	13.4	13.4	13.3	**0.21**
01/11/1999 10:20	620	10.33	13.66	85	1.7	69	68	Wet		11.9	12.3	13.3	13.2	12.7	13.4	13.2	13.3	13.3	13.1	**0.16**
01/11/1999 10:30	630	10.50	13.63	84	1.6	42	41	Wet		12.1	12.4	13.3	13.2	12.7	13.4	13.2	13.4	13.4	13.5	**0.08**
01/11/1999 10:40	640	10.67	13.92	83	1.9	105	105	Wet		12.4	12.7	13.3	13.2	12.7	13.4	13.2	13.5	13.5	13.7	**0.02**
01/11/1999 10:50	650	10.83	14.18	83	1.8	112	111	Wet		12.9	13.0	13.3	13.2	12.7	13.4	13.3	13.6	13.6	14.0	**0.01**
01/11/1999 11:00	660	11.00	14.36	82	1.4	156	155	Wet	7	13.6	13.3	13.4	13.3	12.7	13.4	13.3	13.7	13.7	14.2	**0.02**
01/11/1999 11:10	670	11.17	14.64	82	2.0	189	184	Wet		14.1	13.4	13.5	13.3	12.8	13.5	13.4	13.8	13.8	14.5	**0.05**

Figure 9.2.4 - 9 6th and 7th November 1999
Slab Temperatures and Cloud



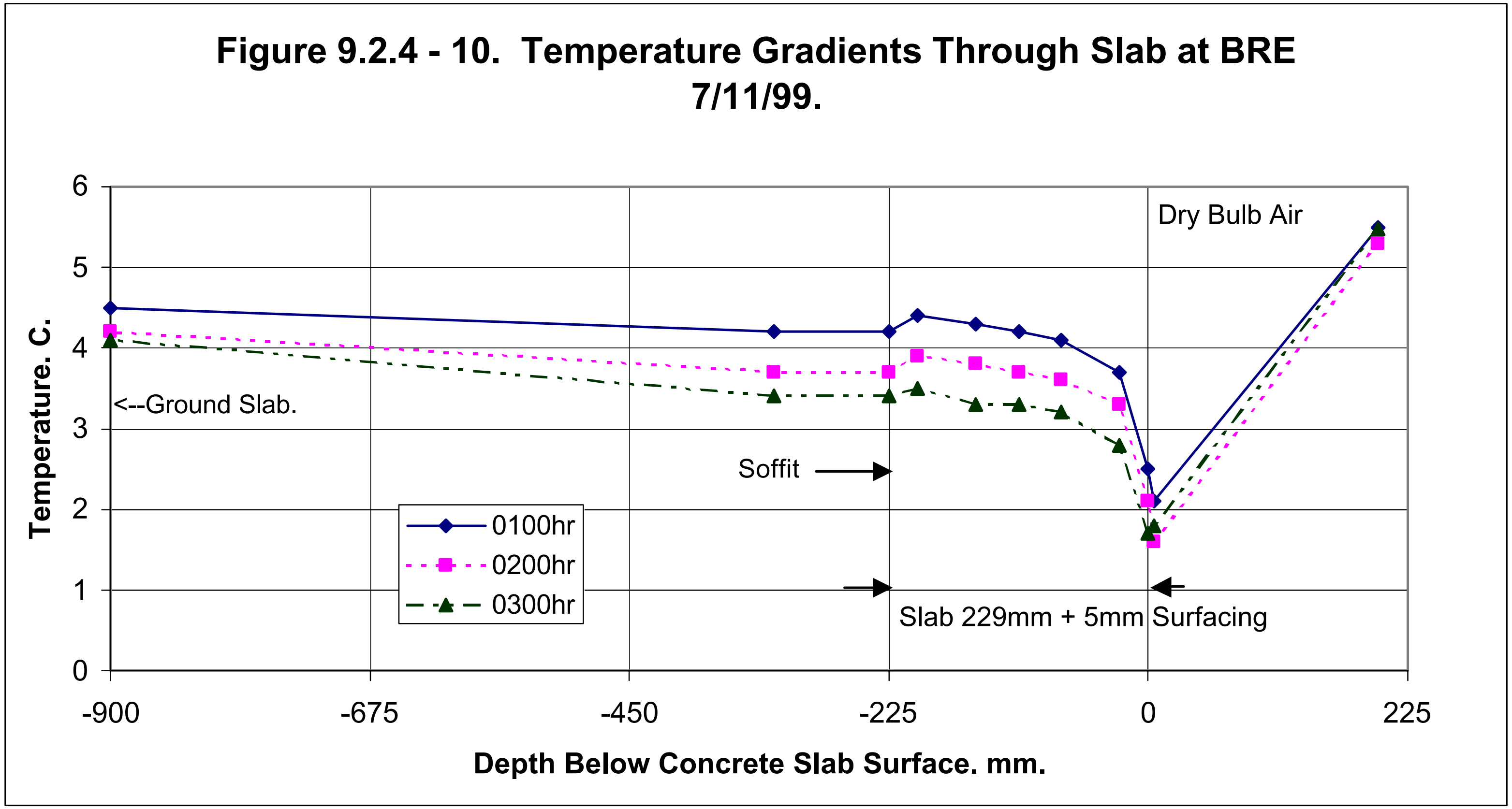


Figure 9.2.4 - 10. Temperature Gradients Through Slab at BRE 7/11/99.
Temperature. C.
Depth Below Concrete Slab Surface. mm.
Dry Bulb Air
<--Ground Slab.
Soffit
Slab 229mm + 5mm Surfacing
0100hr
0200hr
0300hr
6
5
4
3
2
1
0
-900
-675
-450
-225
0
225

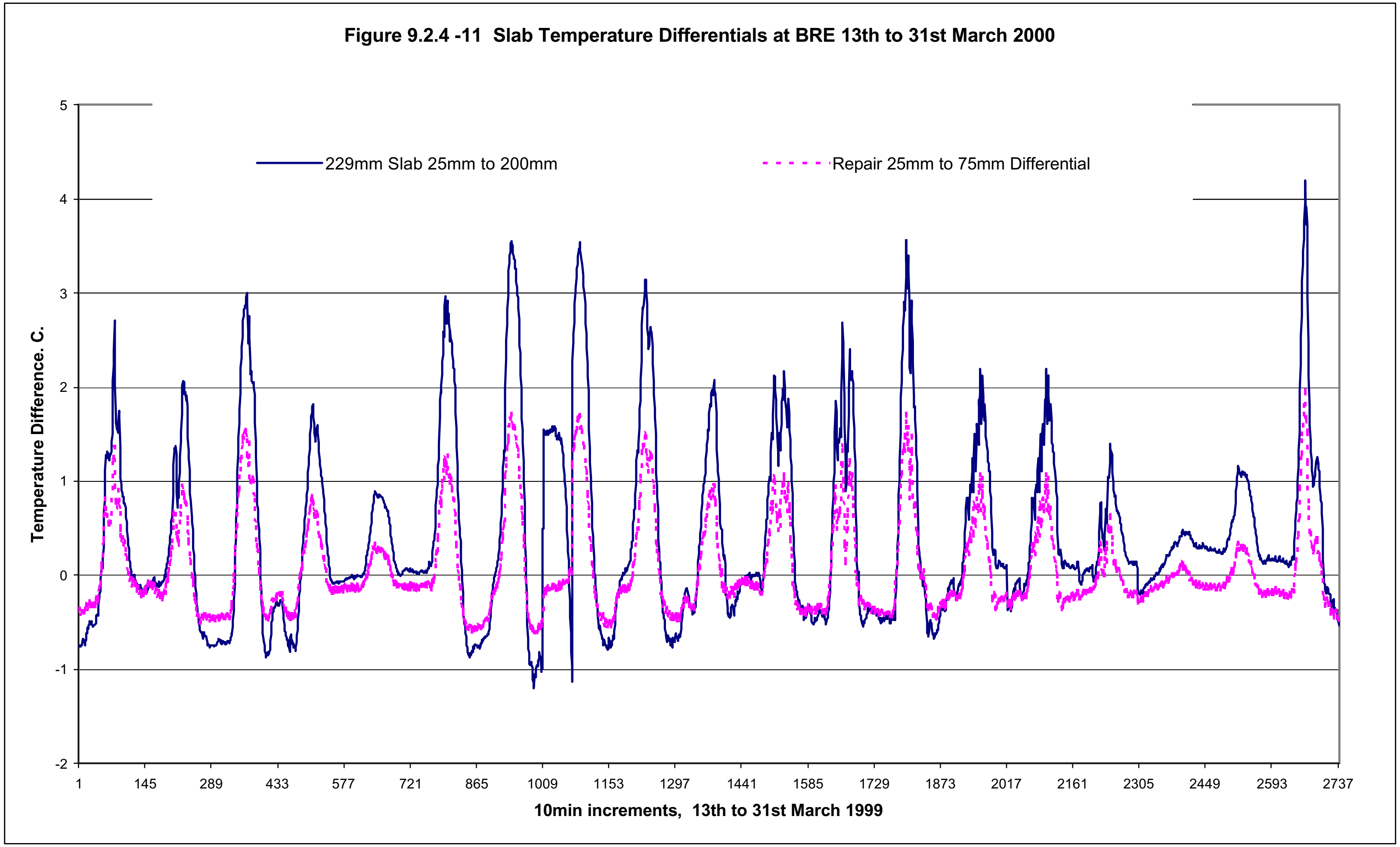


Figure 9.2.4 -11 Slab Temperature Differentials at BRE 13th to 31st March 2000
229mm Slab 25mm to 200mm
Repair 25mm to 75mm Differential
Temperature Difference. C.
5
4
3
2
1
0
-1
-2
1
145
289
433
577
721
865
1009
1153
1297
1441
1585
1729
1873
2017
2161
2305
2449
2593
2737
10min increments, 13th to 31st March 1999

The following values of temperature effects during the period of cooling to 3am on the 20th March 1997, (based on matching weather 6th - 7th November 1999) have been carried forward to the calculations of reactions, column moments and stresses in the Pipers Row structure.

-1.0°C Differential temperature between the top (cold) and bottom (cool) face of the top slab

with

-2.0°C Differential temperature between the average temperature of the top slab and the average temperature of the slab below. This is estimated from the difference in surface temperature of the slab at BRE and the ground slab below.

The weather before the failure in March 1997 would have been similar to the measured cycles of differential temperature in March 1999 which approximates to cycles on 4 days per week of:

3.0°C to -1.0°C Differential temperature between the top and bottom face of the top slab

with

5.0°C to -2.0°C Differential temperature between the average temperature of the top slab and the average temperature of the slab below.

For the repair the normal March cyclical temperature range between 25mm and 75mm deep would have approximated to a cycle, on 4 day per week, of 1.5°C to -0.5°C.

9.2.5 Magnitude of Thermal Effects on Column Perimeter Shears

In carrying out the overall ANSYS plate structural analysis, Amey Vectra included two thermal unit load cases which gave V_t reactions, M_x and M_y moments and shear stresses at perimeter nodes for all column locations. These runs were based on a short term E of 20.5kN/mm^2 and the uncracked stiffness of the slab, which may overestimate the load effects. The results should be regarded as indicative of the relative magnitude of the effects rather than precise.

The two load cases evaluated were:

LC8 For the average temperature of the top slab 10°C above that of the slab below. With the slab fixed at the ramp, this expansion moves the column tops away from the ramp with the resulting column moment increasing the shear on the side towards the ramp and reducing it on the other side.

LC9 for unit thermal conditions of a 10°C temperature difference between the top and bottom surfaces of the top slab. This day time condition hogs the slab to reduce the reactions on the inner columns H2 and I2 and increase it on the edge columns.

These unit temperature conditions have then been adjusted pro rata for the temperature differentials for the day and night conditions. The values of the change in effective shear V_{eff} (ie including moment enhancement) are given relative to the most adverse V_{eff} for the 'Reference Dead Load' at the time of failure (*Ref DL* = (LC1 + LC5)/2), see 9.2.2. This gives an indication of the sensitivity of punching shear to these thermal effects. However, as the thermal effects influence column moments as well as reactions, the interaction with other load effects on the peak shear stress on the four faces is more complex and is discussed further in the section on stress distributions below.

The differential temperatures between the top slab and the slab below (LC8) and within the top slab (LC9) have similar magnitudes of adverse effect on the shears around the columns. The increased shears from LC8 largely arise from the moments induced. The extra shears in the LC9 case result from the change in vertical reaction. The unfactored (λ_f =1.0) values of *Veff* and % change relative to *RefDL* are given in Table 9.2.5 -1.

Table 9.2.5 - 1
Increase in V_{eff} and % change relative to the *RefDL* from temperature effects

Condition	Unfactored (γ_f =1.0) *RefDL* + Temperature V_{eff} kN.	Most Adverse % of *RefDL* V_{eff} %
For Column H2		
RefDL only	244	100%
Collapse 20/3/97 and Low of Cycle Top Slab DT -1.0°C with Top Slab - 2.0°C to Slab below.	250	102.5%
March Thermal Cycle High Top Slab DT +3.0°C with Top Slab +5.0°C to Slab below.	239	98.0%

Table 9.2.5 - 1 (cont.)

Increase in V_{eff} and % change relative to the *RefDL* from temperature effects

Condition	Unfactored (γ_f=1.0) *RefDL* + Temperature V_{eff} kN.	Most Adverse % of *RefDL* V_{eff} %
For Column I2		
RefDL only	275	100%
LC8 -10.0°C from slab below	326	119%
LC9 -10.0°C in top slab.	340	124%
Extreme DT Max Day Top Slab DT 21.2°C with Top Slab +8.0°C to Slab below.	210	77.0%
Extreme DT Min Night Top Slab DT - 4.4°C with Top Slab -6.0°C to Slab below.	310	113.5%
Collapse 20/3/97 and Low of Cycle Top Slab DT -1.0°C with Top Slab - 2.0°C to Slab below.	285	104.2%
March Thermal Cycle High Top Slab DT +3.0°C with Top Slab +5°C to Slab below.	257	93.8%
For Column J2		
RefDL only	94	100%
Collapse 20/3/97 and Low of Cycle Top Slab DT -1.0°C with Top Slab - 2.0°C to Slab below.	92	98.6%
March Thermal Cycle High Top Slab DT +3.0°C with Top Slab +5°C to Slab below.	120	128.5%

The interior columns H2 and I2 have similar sensitivities to temperature effects, the differences arise from the difference in the edge span lengths. J2 being an edge column has a reduced reaction at night for the conditions prior to the collapse.

Over the period leading up to the collapse deterioration had weakened the slab to give a strength little more than the dead load reaction. The daily temperature cycles would have been acting, prior to coning off, with the cycles of live load from daily use. These cycles of load would have acted to progressively initiate and spread cracking at the three critical columns H2, I2 and J2 further weakening them. It would seem to be purely chance that the collapse was not triggered by the loading from vehicles during the weeks prior to coning off, rather than the empty car park conditions at 3am. The reducing loads at J2 at night compared to the high daytime increase in both vehicle and thermal load effects indicates that it was unlikely that the punching failure initiated there.

The overcast day condition gives increased V_{eff}, with full 'Medium' vehicles, of up to:

13% at H2, 10% at I2 and 37% at J2 above the *RefDL* condition.

compared to temperature cycles which at night give:

+2.5% at H2, +4.2% at I2 and -1.4% at J2 above the *RefDL* condition.

and by day up to:

-2% at H2, - 6.2% at I2 and +28.5 % at J2 above the *RefDL* condition.

The BS8110 load combinations on buildings (2.4.3.3) do not require temperature differentials to be considered with or without live load. The temperature differentials in a car park roof slab are limited as the soffit is exposed to air temperature, in contrast to the controlled interior of a building where more severe differentials can arise.

In BS5400 ultimate design the concrete dead load (γ_{fL} 1.15) and superimposed load (γ_{fL} 1.75) are considered for highway bridges with differential temperature (γ_{fL} 1.0), but with coincident HA vehicle loading(γ_{fL} 1.25). The restraint of movement condition in BS5400 (γ_{fL} 1.3) corresponds to the differential in temperature between the top slab and the slab below in a car park.

The current simplified BS 8110 requirements are likely to be sufficient only when all parts of the structure have ductile failure modes so local overstresses can be redistributed. Temperature effects need to be quantified in the assessment of all structures which contain brittle elements. This should be made clear in Codes of Practice and guidance for design and assessment.

9.2.6 Wind Loading and Horizontal Forces

Wind load was light at the time of the collapse. Meteorological reports gave 3m/s at 10m maximum at Birmingham Airport.

Horizontal forces applied to the structure were primarily resisted by the stiff ramp system, so they did not give rise to significant shear forces between slabs down the columns. The lift and stair towers were structurally independent of the floor slabs. The horizontal shears applied to the columns due to the difference in temperature between the top slab and the slab below have been considered in section above.

9.2.7 The Effect of Construction Tolerances on Column Reactions and Moments

BS8110 Clause 2.4.1.4 draws attention to the need to consider construction effects so that "compliance with limit state requirements is not impaired". In normal in-situ construction the slabs are cast onto the columns, so no misfit arises. With the lift slab construction at Pipers Row, the reactions when the slab was set on the columns depended on the tolerancing of the levels of the seatings on the columns, the variation in the sizes of wedges and packs and the setting of the shear head collars when the slab was cast.

Where structures are ductile, they are insensitive to misfit at the ultimate limit state, as redistribution occurs safely. However with the brittle characteristics of punching shear failure in a flat slab, without specially detailed reinforcement, the additional stresses from misfit can contribute to triggering a collapse.

Lift slab construction is based on casting the slabs at ground level and jacking them up precast columns and lowering them onto wedge seatings which support the shear head angle collars. The construction sequence and tolerances have been reviewed on the basis of the drawings and discussions with former BLS staff, see Appendix 1. This indicated that there was a potential lack of fit due to the tolerances in setting the slab relative to the inserts cast into the column. The likely range in vertical alignment would have been ±5mm for good quality construction at any column location, but could be as great as ±10mm.

The tolerances were such that it seemed unlikely that the four wedges at each column head would share the load equally. With the slab reaction concentrated on one or two wedges, a moment would be locked into the column with a redistribution of the shear stresses in the slab around the perimeter. Depending on the location of the high wedge relative to the other moments around that column this could either increase or reduce the peak shear stress and V_{eff}.

After all the slabs had been set on their wedges on the columns the gaps between the shear head collar angles and the columns were filled with a fine mortar to create a moment connection. Any subsequent applied rotations of the slab would then induce moments in the column and a redistribution of shears.

To check the actual construction, the retained column head with attached shear head and slab from I2 was sectioned as described in 8.2.7. This showed that at I2 almost all the load had been carried on wedge A (towards I1/H2) where the bearing area was bright and uncorroded. There was a small bright contact area on wedge B (towards I1/J2) indicating a small load on it. The other two wedges on the side towards I3 carried no load, as indicated by the corrosion which had formed on the whole area of the seating. The mortar infill at I2 provided a full moment connection.

It is not possible to determine the actual misfits at the columns and the consequent changes to the reactions and slab shears. However the potential sensitivity has been determined using the AV ANSYS plate analysis of the whole slab area. The effects of variations in the tolerancing of the column slab connections have been considered as two sets of unit load cases under slab dead load of 5.41kN/m^2 with a long term E of 10.25kN/mm^2 for the uncracked slab.

LC31 to 35:	with one column connection set high, relative to adjacent columns, by 5mm relative to the theoretical 'as-cast' on grade levels for each column G1, G2, H2, I2 and J2.
LC41 to 44:	with the pair of wedges on the side towards line 3 carrying all the load, prior to the grouting in of the column slab moment connection for each column G2, H2, I2 and J2.

As these are based on the long term E, they take into account the reduction in the effects of misfit with creep over time. Unfactored 5mm misfit at one location does not give the 'characteristic' condition appropriate for design or appraisal. This needs to consider possible misfits of ±10mm at the worst locations in a large structure and the likelihood of some of the adjacent locations being alternately high and low. The effect of vertical misfit at a column, with uniform seating on 4 wedges, changes the vertical reactions V with negligible effect on the column moments. The effects of the 5mm high vertical misfit are set out for H2 I2 and J2 in Figure 9.2.7 - 1.

Table 9.2.7 - 1
Increase in *Veff* and % change relative to the RefDL from 5mm Vertical Misfit at a Column

Condition	Unfactored *RefDL* + Misfit. V_{eff} kN.	Most Adverse % of *RefDL* V_{eff} %
For Column H2		
RefDL, perfect fit.	244	100%
LC31 G1 5mm high	249	102%
LC32 G2 5mm high	208	86%
LC33 **H2** 5mm high	304	**125%**
LC34 I2 5mm high	218	90%
LC35 J2 5mm high	249	102%
For Column I2		
RefDL, perfect fit.	275	100%
LC31 G1 5mm high	277	100%
LC32 G2 5mm high	284	103%
LC33 H2 5mm high	250	91%
LC34 **I2** 5mm high	317	**115%**
LC35 J2 5mm high	256	93%
For Column J2 (Based on M_x or M_y, excl. 1.25 factor).		
RefDL, perfect fit.	94	100%
LC31 G1 5mm high	94	100%
LC32 G2 5mm high	92	98%
LC33 H2 5mm high	100	106%
LC34 I2 5mm high	75	80%
LC35 **J2** 5mm high	109	**116%**

It is clear that the effects of vertical misfit on column reactions and shears, even with good quality control to limit it to ±5mm, is of the same order as the effect of live load for medium vehicles (ie 13% at H2, 10% at I2 and 37% at J2 above the *RefDL*) and the range from temperature effects.

There is no way in which these locked in stresses can be determined now. So the dead load column reactions at H2, I2 and J2 are indeterminate within a broad range set out in Figure 9.2.7 -2.

Table 9.2.7 - 2
Range of Column V_{eff} kN, for Levels of Vertical Misfit.

Misfit	-10mm	-5mm	0mm	+5mm	+10mm
Column H2	172	208	244	304	364
Column I2	225	250	275	317	359
Column J2 (Based on M_x or M_y, excl. 1.25 factor).	56	75	94	109	124

The breadth of this range makes it difficult to determine, from comparisons of load to deteriorated strength, at which column the collapse initiated.

The tolerancing of the construction and of the columns, wedges and shear heads make it improbable that any shear head was supported on more than one or perhaps two wedges. Depending on the location of the high wedge relative to the other moments around that column, this could either increase or reduce the peak shear stress. The wedge pairs are 150 mm from the centreline on line 2 giving a possible moment of $\pm 0.15V$ kN/m under slab erection dead load. The wedges are 65mm wide and typically 5mm apart, so if all the load is centred on one wedge, there is also a moment of $\pm 0.035V$ kN/m about the Line H, I or J axis.

Seating on one or a pair of wedges, substantially changes the column moments and shear distribution at the perimeter, with the vertical reactions V little altered. When most adversely combined with the *RefDL* reaction the moments and the corresponding V_{eff} to BS8110 are:

Table 9.2.7 - 3
Effect on V_{eff} of Uneven Seating on Wedges.

	V_t kN	M_x about Line H, I, J kN/m	M_y about Line 2 kN/m	V_{effx} kN	V_{effx} % V_t	V_{effy} kN	V_{effy} % V_t
H2	236	16.0	37.5	252	104%	294	123%
I2	271	13.8	42.6	285	104%	337	123%
J2	90	6.0	15.5	99	105%	115	124%

(J2 based on *Mx* or *My*, excl 1.25 factor, holes not considered).

If the wedge pair is high on the side adjacent to the higher stressed side of the shear perimeter the lack of fit of the wedges can increase BS8110 shear stresses by 4% about the x axis and 23 to 24% about the y axis. These enhancements apply to the actual reaction, so for example, for the case of I2 at 5mm high on one wedge pair the effective shear V_{eff} is increased to 388kN, 42% greater than the *RefDL* of 274kN for the perfect fit case.

In this section the BS8110 method of calculating V_{eff} has been used to examine the sensitivity of shear stresses to column moments. Because of the way in which the shear head angles transfer the moments and shears into the slab a more detailed evaluation of the shear distribution around the shear perimeter has been carried out using the AV ANSYS and BRE DIANA Analyses in Section 10.5.

Another potential cause of vertical misfit between column and slab is differential foundation settlement. This was considered immediately after the collapse. The form of the foundations, the general knowledge of the ground conditions in Wolverhampton expressed by Wolverhampton Council, and the lack of any reported signs of differential movements in other parts of the structure, particularly those below the failed slab, led to the conclusion that no significant differential settlement would have occurred.

The combined effects of vertical misfit and the setting of the wedges with the Lift Slab System was a major, but indeterminate, factor in determining the shear stresses at the time of failure. These misfits have too large an effect to be considered as secondary effects to be covered by the factors of safety. It is essential in both design and appraisal of any brittle structural element that these construction misfits are explicitly considered to determine a characteristic value and for an appropriate factor of safety to be applied when combined with other load effects.

9.3 EFFECT OF ANALYSIS ON COLUMN REACTIONS AND MOMENTS.

9.3.1 Young's Modulus, Flexural Cracking and Creep

The above idealised analyses of the structure under long term loads are based on a 229mm thick uncracked section with a long term E of 10.25kN/mm^2 for permanent loads and initial misfit and a short term E of 20.5kN/mm^2 for vehicle loads and temperature. For applied dead and live loads the E value, which has been applied equally to both columns and slab, does not effect the distribution of reactions and moments. However the magnitude of deflections and the magnitude of stresses from the misfit of supports and temperature differential effects are directly related to the assumed stiffness.

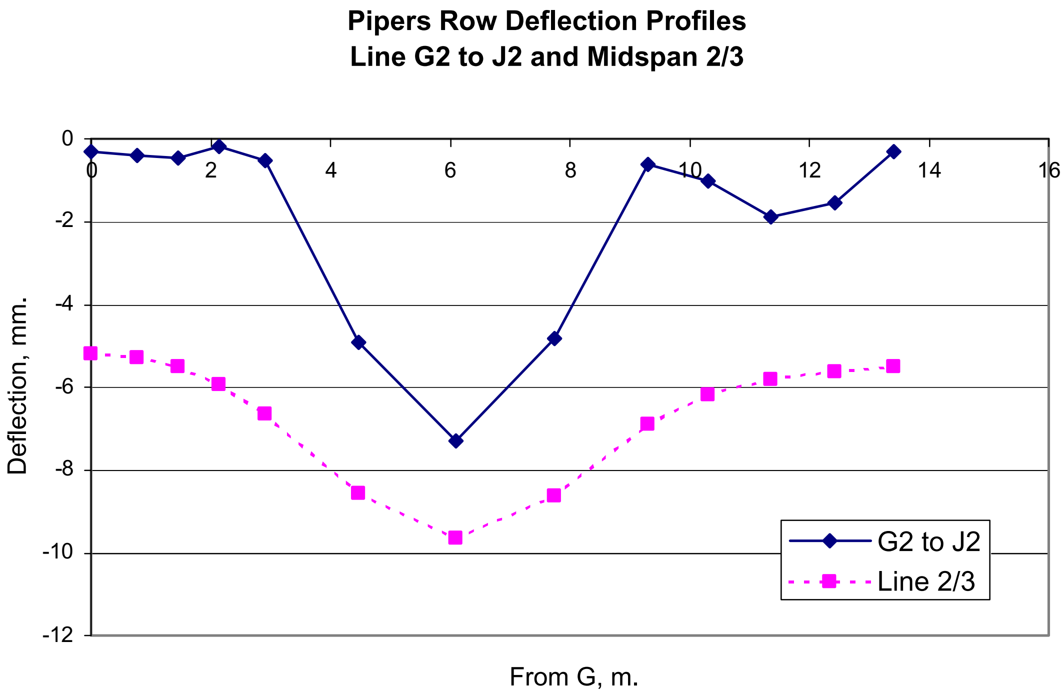
The stiffness of the actual structure varied from the idealisation in a number of ways, which need to be considered in interpreting the results of the idealised analysis. The testing of the concrete from Pipers Row showed a wide range of concrete strength and stiffness, Section 8.3.3, and these are compared in Table 9.3.1 - 1 with the assumed values used in the analyses.

Table 9.3.1 - 1
Comparison of analysis properties with test data

		Amey Vectra ANSYS	Failed Slab worst area	Failed Slab typical	Other Slabs
E short term	kN/mm^2	20.5	9	18	24
E long term	kN/mm^2	10.25			
Poisson's ratio		0.2			
fcu Design	N/mm2	21			
fcu Average	N/mm2		22	27	35
fcu Char.	N/mm2	21	15	20	28
Slab Thickness	mm	229		235	

The values of E assumed for analysis reasonably represent the typical concrete in the car park. However some areas of concrete within the failed slab had a significantly lower E and higher creep, making it softer in the both the short and long term. The variability of material within the failed slab is such that the location of all these 'soft' areas cannot be determined.

Figure 9.3.1 -1 shows the deflected profiles along the line through column centres from G2 to J2 and the parallel line at midspan between Lines 2 and 3. These are replotted relative to the design fall along Line 2. The design fall from G2 to J2 (201mm in 13.4m i.e. 1 in 67) should have been more then sufficient to prevent ponding on the slab. However actual sagging of the slab did create areas of ponding.



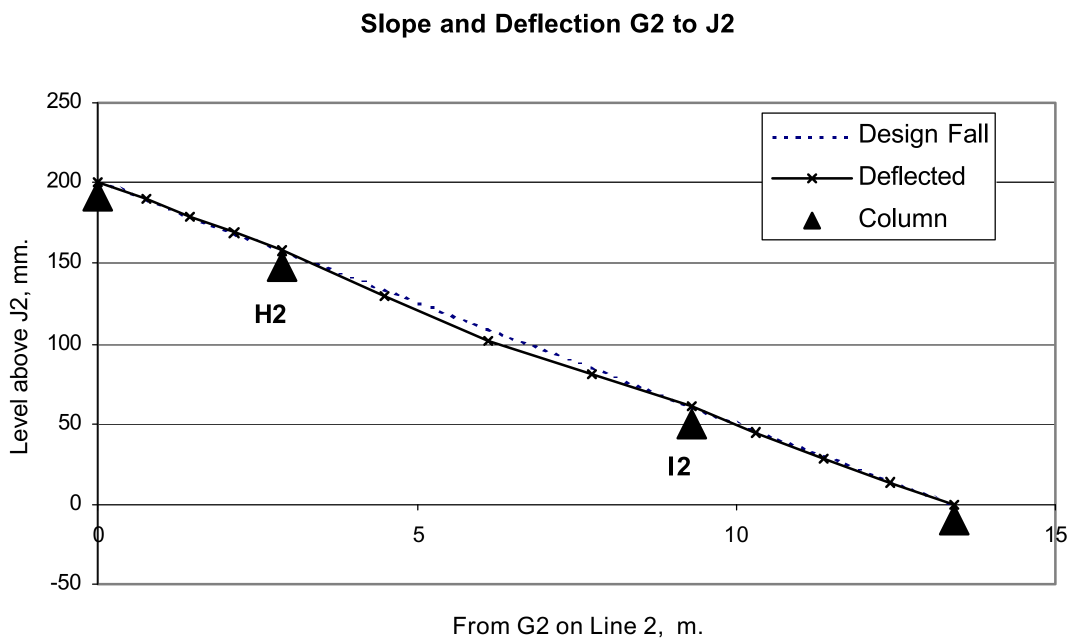


Figure 9.3.1 - 1 Deflections and Deflected Profile from AV ANSYS Cracked Slab run under *RefDL* 5.4kN/m^2

The precast columns were of $6000lb/in^2$ precast concrete ($41N/mm^2$) with a mean BS8110 short term E of $28kN/mm^2$. In consequence they would be 37% stiffer than assumed in the analysis and would proportionately increase the moments they attract, increasing the V_{eff} effective shear stress. The relative rigidity of the columns, which remained uncracked and undeteriorated, would have attracted increasing moments as the asymmetric effects of cracking, creep, deterioration and repairs developed in the slab. Reinforced concrete under significant flexure tends to crack, reducing its stiffness from the concrete or gross section normally assumed in analysis (BS 8110 2.5.2 a & b.). However the concrete between the cracks gives some tension stiffening so the cracked transformed section (2.5.2 c), with steel on one side of the neutral axis and concrete on the other, gives a lower bound value of EI.

With flat slabs the development of star cracking in the slab around columns is to be expected even under dead load, CIRIA 110 Section 2.1[24]. There are reports and photos by Wolverhampton taken in 1988 [H203], and by Sandberg in 1997, Figure 7.4 - 8, of well developed star cracking at some locations at Pipers Row in the lower levels where the bare slab permitted inspection. The top slab was covered by the surfacing which would have hidden star cracking. The examination of concrete attached to the I2 shear head clearly showed (corrosion on angle down to neutral axis) an old flexural crack across the face of long angle then radiating out as a star crack. It could be that the variation in severity of star cracking from column to column in part reflects the columns where the support is set high.

The effects of this local star cracking on column reactions and moments were examined by revising the stiffness of the elements around the columns in the AV ANSYS analysis down from the full depth concrete section to that reflecting the average cracked section at the '3*d*' perimeter on each face. This was done by reducing the flexural E, to give the appropriate EI, in the x and y axes (eg at I2 E_x was reduced to 0.53E and E_y to 0.29E) for the area within the '3*d*' perimeter. The full E was used for the elements incorporating the steel angles. The shear stiffness was not changed. In the BRE DIANA analysis the local area around I2 was modelled in detail including the distribution of reinforcement, the shear head angles and the cracking of the concrete, as described in Section 10.5.

This cracked analysis has a similar effect to the redistribution of moments from supports to the mid span permitted in BS8110 and has similarities to the cracked analysis in CIRIA 110 Appendix 1. However the reduction in column moments in BS8110 also relates to the correction of the too high values of *Mt* which can result from sub-frame analysis.

The checks by AV on the flexural capacity to CP114 and BS 8110 show adequate flexural steel in most locations under full factored design dead + live load. For the column strip steel over H2 and I2 the flexural steel was not highly stressed and no local yielding of top steel would have occurred in these areas, even under factored Dead + Live loads.

In the local areas of flexural overstress identified cracking would have redistributed moments to surrounding hogging areas and to the orthogonal reinforcement in the slab. As noted above the actual slab and surfacing dead load is only 87% of the design value and live loads would seldom have exceeded half the design value, so the extent of cracking leading to redistribution would have been limited except for the star cracking immediately around the column heads. There are some reports of soffit cracking developing (eg Sandberg Photo 3 - 30). For the analysis of the structure to determine the effect on the column reaction and moments from cracking it was assumed that, other than within the 3*d* perimeter, around the columns the slab retained it's uncracked stiffness.

Comparing the cracked analysis with the corresponding uncracked load case showed a redistribution of reactions from the columns H2 and I2 principally to the Row 1 edge columns. This reduced Reactions V by 7% at H2 and I2 with little change at J2. However when the increased transfer of moment to the columns was considered in calculating V_{eff} the reductions at H2 was 2.5%, at I2 4.5 % while at J2 the increase was 4%. This redistribution was significantly less than the reductions from the permitted moment redistribution on the BS8110 basis.

In the construction of Pipers Row the slab dead load reactions arose as the slab was set on the wedges, with only the moment from the uneven wedges. Then the moment connection was poured locking the columns to the slab, so that all subsequent rotations of the slab above the column would be resisted by the column. For this reason the surfacing and live load behaviour (LC6, LC7) have been analysed with column fixity compared to the basic 'as-lifted' dead load case LC1 on pinned supports. Because of the unequal spans the creep deflections of the slab will rotate the top of the column over time.

The long term deflections from creep effects are roughly equal to the initial deflections, though with the poor concrete at Pipers Row they may have been somewhat larger. This will cause column moments to develop equal to about half those for the case (LC5) with dead load applied with column fixity. To account for this the 'Reference Dead Load' *RefDL* has been taken as the mean of the LC1 and LC5 load cases. This slightly reduces (by 0% to 4%) the V_{eff} at H2, I2 and J2 relative to the pinned LC1 'as-lifted' case.

9.3.2 Comparison of Sub-Frame, Grillage and Frame Analysis.

The original BLS CP114 calculations determined moments in the slab as beams on pin supports at column locations. The reactions on the columns were derived by estimating the area supported. For CP114 the shear is calculated on the assumption that the load acts uniformly on the whole shear perimeter, so the transfer of the moment into the column is not considered.

For BS 8110 a range of analysis methods for flat slabs are permitted for determining the reactions V_t onto the columns and the M_t in both directions for the calculation of V_{eff}. A range of simplifying assumptions relative to sub-frame analysis are permitted particularly for uniform spans with UDL loading and for edge columns. Both H&S and AV have carried out assessments to BS8110 using sub-frame analysis to determine the reactions and column moments. CIRIA 110 “Design of Flat Slabs to BS8110” [24] sets out the basis and the advantages and limitations of different BS8110 approaches. It compares the sub-frame analysis followed by moment and reaction redistribution, with a more detailed grillage analysis and in it’s Appendix 1 with Regan’s experimental work.

AV carried out a grillage analysis following the principles set out in CIRIA 110 using STAAD. The values from this and the AV and H&S sub-frame analyses have been compared in Table 9.3.2 - 1 with the ANSYS figures.

AV and H&S have noted some differences in results from sub-frame analysis for CP114 or BS8110 have resulted from the range of reasonable assumptions on:

- The treatment of cantilevers beyond the outer columns (H&S calculations)
- The fixity assumed at the columns, both AV and H&S considered the all pinned case but in treating column fixity H&S assumed that edge columns would not provide fixity while AV considered edge columns fixed
- The redistribution of moments within the slab and from the columns into the slab with the consequent changes in V_t, M_x and M_y at columns.

Table 9.3.2 - 1
Variation in column reactions V_t, with analysis method

	kN/m^2	H2 V_t kN	I2 V_t kN	J2 V_t kN
AV *RefDL*	5.40	235	271	90
AV Design DL + LL ANSYS	8.62	368	423	141
AV BS8110 Grillage	8.62	368	416	151
AV CP114/BS8110 Subframe #	8.62	294	331	166
H&S adjusted to Superstress subframe #	8.62	293	331	168
BLS 1964 Design Max	8.62	431	431	192

redistributed with moments

Because the unequal spans lead to moments in the columns Pipers Row is more sensitive to the approach adopted for analysis and moment redistribution than a structure with regular spans. The BS8110 simplifications for the determination of the ductile deck flexural reinforcement may well underestimate the most adverse column reaction, moment and V_{eff} for the slab punching shear in similar irregular structures.

The BS5400 requirement in Clause 5.2.2 c) which only permits redistribution of moments if *"Shears and reactions used in design are taken as those calculated either prior to redistribution or after redistribution, whichever is the greater"* provides a more rational approach.

BS8110 Clause 3.7.1.2 permits a range of analytical approaches, including yield line analysis, and states *"In such cases the applicability of the provisions given in this section are a matter for judgement"*. In application, by engineers who are normally seeking maximum economy, possibly following simplified computerised routines, this clause may well result in values of V_{eff} being used which are too low for a brittle punching shear element. Both the analytical approach and the redistribution of moments need to be further researched to provide clarification for appraisal and design to prevent punching shear having a substantially higher risk of failure than for other structural elements.

9.3.3 The Effect of Deterioration and Repairs

All the above analyses assume that the slab had not deteriorated. The depth of deterioration, extending in some areas to 100mm deep in a 229mm nominal slab, would have led to changes in the distribution of reactions and moments in the columns. These changes are indeterminate because of our limited knowledge of the distribution of the deterioration and the way in which it changed the stiffness of the slab and the bond to the reinforcement.

By 1996 there was severe deterioration down to the level of the top steel around H2, I2 and J2.The deterioration would result in a loss of stiffness of the slab. Just 50mm of degradation on a 229mm slab halves the gross inertia of the section. The initial stiffness of the slab concentrates reactions on the inner columns H2 and I2 and their reactions would be reduced by deterioration with more load transferred to the edge columns in a similar way to the effect of star cracking. However, the star cracking is symmetric about columns, but deterioration would have been more variable, giving rise to increased column moments which may, depending on sign, have increased V_{eff} despite the reduction in V.

The magnitude of these changes in V_{eff} on H2, I2 and J2 would be unlikely to have changed V_{eff} at any of these locations by more than 5 to 10%, so this would have been, at worst, a lesser factor in initiating the collapse. The main effect of deterioration is on strength rather than load distribution.

When the repairs were carried out in 1996, the primary objective seems to have been rewaterproofing, with the degraded concrete removed and replaced only sufficiently for the resurfacing to be carried out. The corrosion and deterioration around the repairs at the time of collapse would have been only marginally less severe in 1996. This deterioration was left in place, though signs of its presence would have been obvious.

Repairs and propping

The theoretical and practical effects of a patch repairs on the distribution of stresses in a structure has been well researched [18-20] and guidance [38, 39, 40] is available. Three key factors have been identified:

- What was the stress state below the repair at the time at which the repair fully hardened (ie was propping used to reduce dead load stresses during repair)?

- How well do the Young's Modulus, creep, thermal and moisture movement strains of the repair match those of the concrete replaced?

- Does the repair to substrate concrete bond retain its integrity?

It has been reported that props were used under the slab, while the concrete was cut out and the repair was cast. However the distribution of the supports, their pre-load and the time for which these props were in place after the casting of the repairs are not known. Two conditions have been evaluated:

a) Full propping, pre-loaded to remove dead load stresses from the repaired area of slab until the well matched repair fully hardened which restores reactions and stress distributions to the before deterioration condition.

b) Unpropped, with the repairs carried out without effective propping, props prematurely removed or the repair debonded, so that the repair carries no dead load stress while the rest of the structure carries the additional dead load stresses redistributed during the cutting out.

The repair was reasonable match to good quality (40N/mm^2) concrete, but was stiffer and stronger than the concrete in the deteriorated areas of the slab. Its bond to the substrate concrete was poor with some areas never properly bonded to the deterioration weakened concrete around and under it. This bond may well have progressively deteriorated over the period up to the collapse due to thermal and moisture movements due to temperature and moisture gradients through the repair and substrate, frost in saturated concrete under the repair and the cycles of stress from vehicle and thermal load effects. The uncertainties of the propping pre-load, distribution and duration, make it probable that the repairs behaved similarly to the unpropped case.

Repair cutouts

The AV ANSYS cracked idealisation was further modified to reduce the thickness of the repaired areas to their cutout depth, in three stages for R1 (the H2 repaired area), for R2 (the H2 and J2 repaired areas) and then for R3 (the three repaired areas including I23 as shown on Figure 8.3.10 - 1).

Table 9.3.3 - 1 compares the effects of cutting out with results for the *RefDL* and star cracked cases. With poor repairs or support, the effect of the work in 1996 was to redistribute reactions and moments away from H2, and J2 to the surrounding columns, so that the reaction at I2 returns to close to its uncracked value. This would have tended to increase the risk of failure initiating at I2, rather than the other columns.

Table 9.3.3 - 1
Effect on max V_{eff} of cracking and repair cutouts, unpropped.

	V_{eff} Ref DL kN	V_{eff} Star cracked kN	V_{eff} R1 @ H2 kN	V_{eff} R2 @ H2, J2 kN	V_{eff} R3 @ H2, J2, I23 kN
H2	244	236	209	208	208
%	100%	97%	86%	85%	85%
I2	275	261	276	276	272
%	100%	95%	100%	100%	99%
J2	94	99	100	76	77
%	100%	105%	106%	81%	81%

(J2 based on M_x or M_y, excl 1.25 factor, holes not considered).

9.3.4 Redistribution of Reactions and Moments Following a Punching Failure

Progressive collapse is a particular risk with flat slab structures, as the failure of one column sheds its load to the neighbouring columns and applies the added reaction with a substantial moment increasing V_{eff}. AV considered three structure modifications to their cracked ANSYS idealisation under dead load. The plate elements around column H2 were reduced in stiffness so they 'failed' and carried negligible shear to that column. Then I2 and J2 were similarly 'failed' to evaluate the redistribution of their reactions. The figures in Table 9.3.4 - 1 are upper bound values of V_{eff} assuming that the slab flexural strength was sufficient to transfer shears.

Table 9.3.4 - 1
Effect on max V_{eff} of punching failures at H2, I2 and J2.

	V_{eff} *RefDL* kN	V_{eff} Star cracked kN	V_{eff} with H2 failed kN	V_{eff} with I2 failed kN	V_{eff} with J2 failed kN
H2	244	236	0	378	223
%	100%	97%		155%	92%
I2	275	261	356	0	340
%	100%	95%	129%		123%
J2	94	99	84	254	0
%	100%	105%	89%	270%	

(J2 based on M_x or M_y, excl 1.25 factor, holes not considered).

This pattern of redistribution suggests that a failure initiating at H2 might not trigger a second failure at I2, with only a 30% increase in V_{eff}. However a failure at I2 with a 55% increase in V_{eff} at H2 and a 171% increase at J2 would almost certainly have triggered the type of progressive failure that developed. A failure initiated at J2 might not have triggered a failure at I2 with the 23% increase in V_{eff}.

9.4 Load Effects and Load Factor γ_f Conclusions

The unfactored dead load at the time of collapse was only 5.4kN/m^2 ,which is 42.5 % of the BS8110 factored dead + live load of 12.55kN/m^2, which should have been resisted by the characteristic strength of the slab with the γ_m factor of 1.25 for shear. While strength deficiencies, evaluated in Section 10, substantially contributed to this short fall, the BS8110 partial factors γ_f of 1.4 and 1.6 were not sufficient to:

> *"take account of unconsidered possible increases in load, inaccurate assessments of load effects, unforeseen stress redistribution, variations in dimensional accuracy and the importance of the limit state being considered"*

The inadequacy of the blanket γ_f factor for Pipers Row and for a range of other flat slab structures results from:

1. The mode of failure being brittle and developing into progressive collapse. This requires that the assessments of load effects must identify the most adverse conditions including unforeseen stress redistribution. The simplifications and range of analytical options in BS8110 are appropriate for the design and assessment of the ductile slab in flexure, but not for punching shear. This is particularly important where slabs designed for shear to earlier codes and/or which are deteriorating, are being assessed. It should be noted that the current BS8110 punching shear design and detailing still does not eliminate the risk of a brittle failure mode.

2. Variations in dimensional accuracy in the Lift Slab construction method are an order of magnitude greater than for in-situ construction envisaged in the drafting of BS8110. The short end spans (G to H and I to J) at Pipers Row increases the sensitivity to this. While it can be argued that this should be considered in design or appraisal under Clauses 2.4.1.4 or 3.7.12, those clauses are ambiguous and this point is not always picked up in routine appraisal.

3. Flat slab punching shear failure is also particularly sensitive to differential settlements due to foundation movement from poor foundations, tunnelling, mining etc. These may be adequately covered by a blanket safety factor with ductile structural forms. For brittle elements these load effects should be identified and quantified explicitly in design and appraisal.

4. Thermal effects due to the restraint of the top slab inducing column moments and due to differential temperature in the top slab needs further consideration. Although they were possibly 'the last straw' that triggered the failure, their magnitude was not large at the time of failure. There are other exposure conditions and forms of structure where the effects of temperature on the stress distributions in brittle structures merits detailed evaluation rather than being wrapped up in the factor of safety.

While some developments in partial factors are under consideration for EuroCodes, this does not remove the need for revisions to BS8110 particularly for its application in appraisal.