Building Research Establishment Report

Large Panel Systems

The structure of Ronan Point and other Taylor Woodrow – Anglian buildings

Building Research Establishment

Department of the Environment
Building Research Establishment
Building Research Station
Garston
Watford
WD2 7JR

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SUMMARY

1. This report arises from the statement on Ronan Point and other Taylor Woodrow-Anglian (TWA) buildings made by the Minister for Housing and Construction to Parliament on 23 October 1984 about the implications of engineering advice on Ronan Point which had been received by the London Borough of Newham from their consultants.

2. The report is the result of an examination of the consultants' assessments of Ronan Point in the light of calculations, design recommendations, research information and documentary evidence on TWA buildings. It is concerned principally with the adequacy of the structure of Ronan Point to resist the effects of normal loads (self-weight, imposed loads and wind loads) and abnormal loads (fire or gas explosion). The main conclusions and recommendations relating to the adequacy of Ronan Point and remedial measures necessary to maintain it in a satisfactory structural condition are summarised below, together with the implications for other TWA buildings and the needs for research.

Ronan Point

3. It is concluded that Ronan Point has coped well with substantial normal loads and an abnormal load arising from a fire test since its reconstruction following the partial collapse in 1968. Engineering appraisal indicates that, although no structural distress has been observed and a margin of safety in respect of normal loads is present, the margin locally at the ends of the H2 joints in the flank walls of the lower storeys of the building is lower than that generally considered acceptable for buildings of this type. Possibilities for remedial measures are the repair, reconstruction or strengthening of the H2 joints at highly stressed locations, more comprehensive strengthening of the building by constructing an additional loadbearing system at the flank walls up to the eighth storey level or the removal of eight storeys from the top of the building.

4. The strengthening of Ronan Point during reinstatement after its partial collapse in 1968 is considered to have substantially raised the threshold at which an explosion or other abnormal load could cause progressive collapse. The risk of a gas explosion causing progressive collapse is judged to be remote. Measures to prevent the use of liquefied petroleum gas (LPG) for space heating in the building would make that risk more remote and should be taken. If the widespread use of LPG cannot be prevented, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2.

5. The risk of accidental fire inducing progressive collapse directly resulting in loss of life is remote. Enhancement of the structure in relation to local overstressing of H2 joints at the ends of the flank walls in lower storeys caused by normal loads would provide more capability for accommodating the structural effects of accidental fires. Further safeguard against these structural effects could be obtained by addition of suitable fire protection to ceilings. The gaps found at junctions between cladding panels and floor slabs should be closed by a suitable joint to prevent the passage of smoke and fire gases between adjacent flats.

6. Measures should be considered to safeguard the non-loadbearing cladding against fixing failures and deterioration. In view of the likely difficulties of extensive inspection, additional fixings to the building and between the two concrete skins of the panels would provide a practical precaution. The application of suitable protective coatings may provide a means of extending the life of the components most susceptible to deterioration. Further investigation of rain penetration is suggested.

Other TWA buildings

7. The conclusions drawn from the assessment of Ronan Point are likely to apply to some extent to all other TWA buildings and action is desirable to check the extent where that is not known already.

8. Most 'Type A' buildings are likely to have acceptable margins of safety in respect of normal loads in the H2 joints if they are soundly constructed. The H2 joints in buildings of 14 or more storeys should be appraised. Consideration should be given to the appraisal of the H2 joints in other 'Type A' buildings having regard in particular to their height and plan arrangement.

9. Those responsible should take steps to prevent the use of LPG for space heating in 'Type A' buildings over 6 storeys in height. Where the widespread use of LPG nevertheless continues, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2.

10. Consideration should be given to the appraisal of TWA buildings of 6 or less storeys in height in respect of their robustness and resistance to progressive collapse caused by abnormal loads, particularly where piped gas is installed or where LPG is used for space heating.

11. All TWA buildings should be appraised to establish whether the joints between panels can resist the spread of fire and fumes adequately.

12. The flank wall joints in 'Type B' TWA buildings are likely to be adequate. Confirmation of the design and construction quality should be considered only if additional assurance of adequacy is desired.

13. Tests on H2 flank wall joints and measurements on a 'Type A' building during demolition are outlined which would assist in confirming the adequacy of remedial measures. Other research to improve information relating to large panel system buildings more generally is proposed together with research on inspection methods.

Note on units:

SI units are used except where reference is made to a period at which the relevant quantity was expressed in Imperial Units. Occasionally reference is made to reports in which 'p.s.i.' was used instead of 'lbf/in^2'; the former usage has been retained.

1 lbf/in^2 (p.s.i.) = 6.89 kN/m^2

THE STRUCTURE OF RONAN POINT AND OTHER TAYLOR WOODROW-ANGLIAN BUILDINGS

1. INTRODUCTION

1.1 Ronan Point, a 23-storey tower block built by Taylor Woodrow-Anglian Limited, suffered a partial collapse in 1968. The subsequent Tribunal of Inquiry[1], found that an explosion of town gas blew out concrete panels forming part of the load-bearing flank wall of one flat. As a result there was progressive collapse of the south-east corner of the block.

1.2 The Tribunal made a series of recommendations affecting system-built blocks of flats over 6 storeys in height. These included measures to strengthen Ronan Point itself; measures to appraise and, if needed, to strengthen existing buildings; and measures to be taken in designing new buildings. In addition there were interim recommmendations about gas disconnection.

1.3 Following these recommendations the then Ministry of Housing and Local Government issued advice to local authorities in Circulars 62/68 and 71/68[2].

1.4 The advice to authorities was to appraise all their blocks over 6 storeys in height which were built of large pre-cast concrete panels to form load bearing walls or floors or both in order to consider whether they were susceptible to progressive collapse. In considering whether strengthening was necessary (either by providing alternative paths of support to carry the load, assuming the removal of a critical section - any defined element - of the load-bearing walls, or by producing a form of construction of such strength and continuity as to maintain the stability of the building against forces liable to damage the load-supporting members), they were to provide for forces equivalent to a standard static pressure of 5 lbf/in^2 where gas was to be used. When it was not to be used, the forces could be halved, ie to an equivalent standard static pressure of 2 1/2 lbf/in^2. The standards set applied also to new design pending the revision of Building Regulations and Codes of Practice. In 1970, provisions to resist progressive collapse based on the 5 lbf/in^2 standard in buildings of 5 or more storeys were introduced in Section D17 of the Building Regulations[28].

1.5 The report of the Inquiry warned in specific terms about the dangers of storing explosive substances such as liquefied petroleum gas (LPG) and this warning was repeated in more general terms in Circular 62/68[2)].

1.6 Following discovery of small gaps at the junctions between non-load bearing cladding panels of the facade and the floor panels in Ronan Point, the London Borough of Newham decided early in 1984 to evacuate the block. The Building Design Partnership (BDP) was appointed by the Council to investigate the problem and to report on any defects and incipient or actual deterioration of the fabric. The brief was later extended to include investigations of the structure of Ronan Point in its current condition with reference to structural stability, including resistance to wind, gas explosion and the effects of fire. Subsequently the Council appointed a second firm of consultants, Thomas Akroyd, and retained the Building Research Establishment (BRE) to carry out vibration tests, a fire test and measurements of cladding movement in support of the consultants' investigations.

1.7 Both BDP and Thomas Akroyd submitted first reports[3,4] to the Council in September 1984 summarising the findings of their investigations to date. In the same month a report on Ronan Point by Mr Sam Webb[5] on behalf of the Newham

Tower Block Tenants' Association was produced. BRE reports on the vibration tests[6], cladding panel movement[19] and fire test[7] were submitted in September, November and December 1984 respectively. Two further reports[8,9] were made to the Council by the consultants in November 1984. Subsequently the Council took the decision to demolish Ronan Point and five similar blocks, subject to financial considerations.

1.8 Following the submission of the consultants' reports in September the Minister for Housing and Construction requested a view by the BRE of their technical findings. On 23 October the Minister made a statement to Parliament[10] which included the following reference to the BRE:

> 'The BRE is evaluating these reports and discussing them with the consultants as a matter of urgency. Its provisional view is that the risk that progressive collapse of the building (as opposed to localised failure) could be caused by fire is remote. Nevertheless, it considered that it is necessary to investigate more fully what if any measures are necessary if buildings of this type are to remain in satisfactory structural condition in the longer term.'

1.9 On 26 October 1984, the Department of the Environment wrote[11] to Local Authorities with a copy of the Minister's statement and referred to BRE work related to Ronan Point as follows:-

> 'The BRE is beginning a programme of work to investigate the problems of large-panel systems of construction including Taylor Woodrow-Anglian and to provide advice on appraisal and remedial measures.'

> 'Further guidance to local authorities owning Taylor Woodrow-Anglian buildings will be issued, as necessary, in the light of BRE's evaluation of the evidence.'

1.10 This report is the result of an examination of the consultants' reports on Ronan Point in the light of discussions with them, calculations, design recommendations, research information and documentary evidence on Taylor Woodrow-Anglian (TWA) buildings. Conclusions and recommendations are made relating to the structural adequacy of Ronan Point, and to remedial measures necessary to maintain it in a satisfactory structural condition. The implications and needs for structural appraisal and remedial measures on other TWA buildings are discussed. Needs for further research are also described.

2. TAYLOR WOODROW-ANGLIAN BUILDINGS

Ronan Point

2.1 Ronan Point comprises 22 floors of flats, built in the Larsen Nielsen system, resting on an in-situ concrete podium containing garages. It was the second of nine such blocks to be built for the London Borough of Newham.

2.2 The Larsen Nielsen system, for which Taylor Woodrow-Anglian Limited were the United Kingdom licensees, uses large prefabricated reinforced concrete panels to make up the loadbearing walls and floors of the structure. A succinct account of the general structural form and method of construction of Ronan Point is given in the report of the Inquiry by the Tribunal[1].

The stock of Taylor Woodrow-Anglian (TWA) buildings

2.3 Information on the numbers, heights and locations of blocks was provided to BRE by Phillips Consultants Limited who were the consultants to Taylor Woodrow-Anglian Limited on the Larsen Nielsen system. It shows that there are 46 other blocks over 6 storeys in height with joints between panels similar to those of Ronan Point (specifically the flank wall H2 joint) and that all these blocks, designated 'Type A' by BDP, were strengthened after the collapse at Ronan Point to withstand an equivalent standard static pressure of 2 1/2 lbf/in^2 (Figure 1). It is understood that gas was not reconnected to these blocks after 1968. Figure 2 indicates that there are 43 blocks (designated 'Type B' by BDP) built subsequently with a different H2 flank wall joint designed to resist forces equivalent to a standard static pressure of 5 lbf/in^2.

2.4 There are also many more blocks of 3, 4 and 6 storeys in height.

2.5 All blocks over 6 storeys in height are in the Greater London Council area apart from blocks in Sunderland (7 blocks), Gateshead (1 block) and Leicester(1 block).

2.6 No other general distinguishing structural features have been determined, apart from number of storeys and type of H2 joint.

2.7 Early TWA blocks, such as those at Morris Walk, Greenwich, may probably be considered as 'Type A' although some details of the H2 joint are not identical to those at Ronan Point. In some high blocks, eg those at Hammersmith and Fulham, the two 'halves' (see 5.11) of the buildings are much further apart with a separate structure for lifts and access connected at each floor to each 'half'. The 'halves' contain an internal longitudinal shear wall extending for part of the length of the building.

2.8 It appears that all the flats in TWA buildings are contained within the precast panel construction. Although this construction may spring from ground level, it frequently starts from the top of one or two-storey in-situ reinforced concrete construction, as at Ronan Point. Thus Ronan Point is variously classed as 22-storeys (of precast construction) in height, 23-storeys to include the ground floor storey, or even 24-storeys because the ground floor storey height is approximately twice the standard storey height of the flats above.

3. THE CONSULTANTS' REPORTS ON RONAN POINT

3.1 The BDP summary report[3] reveals that voids have been found in the in-situ concrete as well as those already known to be present in the dry-pack mortar in many of the H2 joints between the loadbearing flank walls and floor panels. The structural behaviour of these joints and the implications for the performance of the Ronan Point structure as a whole under the effects of normal loads (ie dead, imposed and wind loads) and abnormal loads (specifically gas explosion or fire) were the major matters discussed by both consultants.

3.2 Both BDP[3] and Thomas Akroyd[4] consider that Ronan Point is not likely to be adversely affected by anticipated normal service loads including loads arising during gales such as could occur once in 50 years, but they cast doubt on the ability of the building to resist a gas explosion, eg arising from the storage or use of liquefied petroleum gas (LPG) in cylinders. They also confirm that there is a need for an effective fire-stop joint to close the gaps at the junctions between the cladding panels and floor slabs.

3.3 BDP consider[3] that in the event of a severe fire in two adjoining rooms, the H2 joints as strengthened after the collapse in 1968 'are required for security under normal loading conditions and the security of the joint under the effect of fire is uncertain.' Their concern is that lateral displacements and rotations at the joints may lead to transfer of load to untested and possibly defective zones of joints. They conclude that 'The defects are such that, in our opinion, major repair works are required before the building can be considered to have adequate structural strength in the long term.' Thomas Akroyd concludes[4] that 'The dry-pack mortar to the various joints is generally poor, but can be made good without difficulty. The quality of the H2 joint is less than perfect, but is not of particular importance, since the joint has been strengthened', and that 'The fire resistance of the various parts of the structure, being of concrete is in accordance with the structural fire resistance required for such a building.' In the further report(9) which considers structural performance under fire or explosion in greater detail, Thomas Akroyd conclude that the effects of thermally-induced stresses on the H2 joint in a fire are not a problem.

3.4 The report[9] confirms the earlier conclusion that there is doubt as to the ability of the building to resist a gas explosion and suggests that, because the use of LPG by tenants cannot be prevented, the building be strengthened to resist progressive collapse arising from damage following an abnormal load equivalent to a standard static pressure of 10 p.s.i.

3.5 BDP[3] discuss the implications of their findings on Ronan Point for the other five 'Type A' TWA buildings owned by the London Borough of Newham. They state 'It first needs to be confirmed that the defects encountered in Ronan Point are also present in the other five buildings' and then advise on actions required if similar defects are found. Actions are given in two categories, 'those necessary to ensure adequate safety during a reasonable and orderly decanting of tenants....... and those necessary to provide for the longer term, say the next thirty years'.

3.6 BDP[3] consider the detailing of the horizontal flank wall joint in 'Type A' buildings is such that satisfactory repairs to the defects would be difficult to effect. The construction of structural strengthening frames is recommended therefore at the flank walls of Ronan Point, detailed to collect the floor and wall loadings, and adequately tied through the building with ties buried in the depth of the floor screeds. It is stated that such structural frames would not only provide adequate structural strength against all existing loads, but could be designed to increase the resistance to pressure from 2 1/2 to 5 p.s.i.

3.7 A number of schemes of remedial works are discussed by Thomas Akroyd[9] for strengthening the building to resist a pressure of 10 p.s.i (including demolition and rebuilding of the flank walls, construction of internal reinforced concrete walls to resist blast, tying-back existing panels, the introduction of venting or the construction of internal or external frames tied through the building). They conclude that the construction of frames would be viable, the internal frame being favoured because it would cause less problems for foundations and would be visually unobtrusive.

3.8 BDP[3] also make recommendations relating to other, less major, aspects of the construction where structural shortcomings are considered to be present:

1. For the non-loadbearing cladding panels under high local wind loading, overstressing of some components of the restraint fixings and of the transom zone in certain panels is identified. Connections between the two leaves of some panels may be over-stressed, it is suggested, due to

cyclic thermal movements. The provision of additional fixings for the panels is recommended.

2. Local zones of incomplete hand-packed mortar are reported in the joints of the lift shaft walls and simple repair to complete the mortar filling is recommended.

3. Local repairs to make good zones of loose or missing mosaics are recommended for the mosaic faced panels.

4. For the exposed aggregate panels forming the outer leaf of the flank walls where an instance of corrosion damage was observed, the application of a protective coating is recommended to arrest progression of carbonation and protect in the long term against the onset of corrosion in zones where the cover to reinforcement is low.

3.9 Thomas Akroyd[9] comment on the design of the fixings of the non load-bearing cladding panels 'the fixings of the panels to resist wind pressure is not now an arguable issue, since the building was strengthened and where, as with the window panels, no strengthening was carried out, then there is an adequate factor of safety against the maximum wind gust pressures likely to occur'.

Thomas Akroyd[4] reported investigations into the possibility that cyclic heating and cooling of the floor slabs may have adversely affected the integrity of the structure over the years. They concluded that 'underfloor heating has no effect on the structure'.

3.10 BDP[3] do not express concern about the adequacy of the flank wall joints in 'Type B' buildings. They state 'the in-situ flank wall joints are much bigger, contain interlocking reinforcement connecting the units and vibrated concrete was specified and practical. Such joints will accept eccentric loading and are less sensitive to any deficiencies which may exist in the hand-packed joints, providing that the in-situ concrete is confirmed to be solid. The condition of the joints should be checked'.

3.11 In considering the feasibility of the refurbishment of Ronan Point, BDP[8] give estimates of cost for the works separated into items considered essential for the reoccupation of the block, and those which are identified as highly desirable for effective management and to bring the block up to acceptable modern standards. The estimated costs are:

Essential works	£3,144,000
Essential plus desirable works	£5,724,000

The essential works relate to items of strengthening and refurbishment. It should be noted that the estimated costs of items related to structural adequacy and fire safety - the primary matters being considered in this report - amount to about £1,202,000, ie only about 38% and 21% of the totals estimated for essential works and essential plus desirable works respectively. Thomas Akroyd[9] estimates the costs of strengthening the building to resist a pressure of 10 p.s.i. at £845,000, ie excluding attention to the gaps between cladding panels and floor slabs.

4. BRE INVESTIGATION

4.1 A building may reach a condition where it cannot be judged to be structurally adequate[12] due to one or more causes:

weakness of structure previously unidentified,
overload of normal dead, imposed and wind loads,
abnormal loads arising accidentally, eg fire or explosion, or
weakness arising from degradation.

4.2 The provisional BRE view of the structural adequacy of Ronan Point reached in early October 1984 after preliminary examination of the consultants' first reports[3,4] is given in Annex A in relation to each of the possible causes of inadequacy given above.

4.3 Subsequent BRE investigations are summarised in the following sections of this report. They are based on:

(1) assessments of structural behaviour (in relation to the normal and abnormal loads which might occur) on the basis of structural engineering calculations, available design recommendations, research information and test data,

(2) detailed study of the consultants' reports, discussions with them, and examination of structural engineering calculations provided by BDP.

4.4 The main categories of load to which the structure of a building may be subjected are normal service loads (dead, imposed and wind loads) and abnormal loads such as may be caused by fire or explosion. The distribution of these loads on any building is complex both in space and time. Likewise the response of a structure, particularly a large one such as Ronan Point, to loads is complex. It is therefore necessary to make simplifying assumptions in order to analyse the likely behaviour and thus assess structural adequacy, including safety. Such assumptions should be as realistic as possible while tinged with an appropriate degree of conservatism, bearing in mind the consequences should failure occur. When designing or appraising a structure the engineer justifies his assumptions on the basis of experience (his own with existing structures and as embodied, for example, in the recommendations of appropriate codes of practice), experimental evidence and economic factors. There are usually several appropriate ways of expressing and judging the safety of a structure relating to the many different factors affecting it, such as the uncertainties associated with the way loads act and are distributed within the structure, the quality of the construction and the analytical tools available. It is to be expected therefore that different engineers may approach the issues of analysis and safety in a variety of ways. Broadly similar conclusions may still be reached, although qualified in different ways. Such is the case in the assessments of Ronan Point considered here.

4.5 A detailed inspection of any building is almost certain to reveal a number of differences between the actual construction and the specification. Certain tolerances are recognised in design procedures but gross differences are not. The task of appraisal of an existing building[27] is to assess whether departures exceed those that can reasonably be accepted, and in some cases whether the 'as designed' structure is satisfactory in current circumstances.

4.6 For convenience the structural behaviour of Ronan Point is considered below, under normal loads and abnormal loads separately, although design and analysis must comprise an integrated approach to both considerations.

5. STRUCTURAL BEHAVIOUR UNDER NORMAL LOADS

5.1 The analysis of behaviour is considered broadly in the order: loads to be resisted by the structure; the stresses induced by these loads at particular locations in the structure; assessment of the maximum stresses which the particular locations could carry with safety and some ancillary considerations. Conclusions are drawn from this process, before the consideration of abnormal loads in Section 6.

Loads

5.2 Normal loads are dead (self-weight of structure), imposed (occupancy and snow) and wind loads, as defined in BS 6399[24] and CP3 Chapter V[22] for the particular geometry of building. An overload produced by exceptional variations of normal loads beyond those defined or covered by safety factors is such a remote possibility that it can be ignored.

5.3 Since the combined effect of normal loads would be greatest in the panel structure immediately above the in-situ concrete podium, the adequacy of the lower levels of the structure is likely to be the most critical. The podium joint (H23) consists of direct bearing of wall panel through dry-pack mortar and in-situ concrete onto the reinforced concrete slab and is not considered cause for concern (but see 5.40). Most concern centres on the horizontal (H2) joints at the top of the walls to the first-floor flats, the lowest level at which these joints occur - a height of 9.2 m above the ground. The loads calculated at this level in the east flank walls by BDP and BRE are given in Table 1, together with corresponding available values from the original calculations of Phillips Consultants Limited.

5.4 The resulting differences in dead load are small and arise from the consideration given to such factors as the accuracy of dimensional information, plant and equipment, allowance for window and door openings, and the weight of post-1968 strengthening. The building does not have a longitudinal axis of symmetry, as the central corridor divides it into two unequal parts. For some aspects of behaviour this causes less favourable conditions in the larger easterly part, and therefore the following assessment concentrates on this part of the building.

5.5 The derivation by BRE of the dead and imposed loads is given in Annex B. These loads include the weight of 21 storeys. At higher levels the loads would be lower pro rata and the implications are considered in section 9. The basic assumptions made in these calculations are considered to be unexceptional. The values obtained were comparable to those determined by others. The slightly more conservative values by BDP[3] were used in the subsequent calculations of stresses. A crude but satisfactory independent check on the mass of the building was obtained from the vibration tests[6]: the measurement indicated a mass of 11.0×10^6 kg which, when converted to average dead load stress at second-floor level gave a value of 4.2 N/mm^2, very close to the 3.7 N/mm^2 value determined using Annex B and the 4.7 N/mm^2 value in paragraph 5.19.

TABLE 1: LOADS VARIOUSLY ASSUMED TO ACT AT THE LEVEL OF THE LOWEST H2 JOINT OF THE NORTH EAST FLANK WALL OF RONAN POINT

SOURCE OF CALCULATION	DEAD LOAD ON FLANK WALL (MN)	IMPOSED LOAD ON FLANK WALL (MN)	TOTAL WIND LOAD ON FACE OF BUILDING (MN)	WIND MOMENT ON WHOLE BUILDING (MNm)	WIND MOMENT ON FLANK WALL (MNm)
PHILLIPS CONSULTANTS					
(a) ORIGINAL	5.35	0.44	1.56	39.9	3.39
(b) TRIBUNAL NOTE	5.21	0.46	1.56	41.4	(1.31)+
BDP[26]	5.52	0.59	1.35	40.99 (56.11)*	3.71
BRE**					
(a) CP3: 50-year	5.00	0.29	1.56	38.2	2.29
(b) DYNAMIC ANALYSIS: 50-year	5.00	0.29	1.98	48.3	2.90

+Applies to part of wall only - therefore ignore.

*Proposed for consideration, based on S3 = 1.17.

**Wind effects calculated for a building height of 66 m to top of plant room (see Annex E), not 62.5 m to top of parapet as Annex C.

5.6 The higher values for imposed load in Table 1 arise because no account was taken of the reduction in floor load allowed by BS 6399[24] for buildings of more than 10 storeys. However, as the imposed load is only about 5-10% of dead load, total vertical load will be relatively insensitive to variations in imposed load.

5.7 The derivation of the wind loads by BRE on the basis of the BS Code of Practice CP3[22] is given in Annex C in relation to a 50 year return period wind (the value taken for the design of most buildings). For the sake of comparison Annex C also gives the loads for a 500-year return period wind and for the maximum windspeed experienced by the building.

5.8 The derivation of the wind loads by BRE on the basis of a more precise method which takes account of any dynamic behaviour of Ronan Point is given in Annex D using a 50-year return period wind. This more sophisticated analysis to determine the wind moment likely to act on the whole building, taking account of any dynamic behaviour, gives a value 26% greater than determined using CP3[22]. This is not an adverse reflection on CP3 as safety factors could normally be expected to make allowance for this increase. Nevertheless, the higher value has been used also in the further analysis of the stresses arising in the structure from wind loads.

5.9 By comparison with a basic wind speed of 38 m/s assessed for Ronan Point from CP3[22], meteorological records from the London Weather Centre suggest that Ronan Point had experienced by late 1984 a maximum windspeed of 36.5 m/s at an effective height of 38 m, which is equivalent to a maximum basic windspeed of 32.6 m/s. Calculations based on CP3 indicate that the resulting wind moment on the building was 74% of that derived for design against a 50-year return wind speed (Annex C).

5.10 To assess wind loads BDP[3] adopted the assumptions given in Annex C using a return period of 50 years. To investigate the possible effects of even more severe wind loading, they considered a statistical factor to increase the return period of the design wind speed from 50 to 500 years, an increase in speed of 17%. Whilst it results in a 37% increase in base moment, it only increases total stresses by about 8% indicating the relative insensitivity of stresses to extreme wind loads. Although the principle of considering a greater return period is acceptable, it is considered that such an increase is over-conservative for the assessment of an existing residential building such as Ronan Point. It is thought that many engineers might consider a 100-year return period appropriate for the design of a new building of this type today. It is however a matter for engineering judgement and a question of balancing the importance of the building and the consequences of its failure in the wind against the sensitivity to the forces exerted by extreme winds.

Moment on flank wall produced by estimated wind load

5.11 Before the stress in the NE flank wall due to wind load can be determined it is necessary to estimate the proportion of the total overturning moment carried by the wall. This proportion will depend on the structural interactions and stiffnesses of the different parts of the structure. Essentially the building consists of two rigid panel structures connected by corridor slabs which tie the two blocks together. There are no cross walls across the corridor and therefore there can be very little vertical shear transmission between the two structures. On this basis the proportion of the total wind moment carried by the NE flank wall is examined in Annex E for different assumptions regarding the walls which are contributing to the inertia of the cross-sections. A best estimate for the wind moment taken by the wall is 6% of the total and this value is used subsequently in determining the stresses in the flank wall. That value is to be compared with the more conservative 9% taken by Phillips Consultants Limited in their original design and adopted by BDP[3] in their assessment.

Overall stability

5.12 In addition to considering stresses induced through the structure it is essential that the building as a whole remains stable. Wind loads may be resisted by shear and flexural resistance of the walls and wind moments are counterbalanced by the dead and imposed floor loads. For the E-W direction, resistance is produced mainly by the cross walls acting in shear as vertical cantilevers bending in their own plane. For the N-S direction it is produced mainly by the longitudinal corridor walls acting both in horizontal shear and bending in their plane.

5.13 BDP[3] did not comment specifically on any of these aspects of overturning. Calculation shows there is no cause for concern. No detailed reassessment of foundation capacity is made in this report since there are no prima facie grounds for concern. The vibration tests of the structure[6] support these views.

Local wind loads

5.14 The higher wind pressures experienced by cladding, particularly at corners of the building are derived also in Annex D. The Tribunal[1] placed much emphasis on the underestimation of the wind loads considered in the original design by comparison with more modern data on wind speeds and the consultants' reports[3,4] amplify this point. It is true that current codes would lead to higher design wind loads than indicated in earlier codes for the local pressures experienced by cladding.

5.15 Local wind suction on the flank wall panels was designed to be resisted by a combination of friction and tying in the H2 joint. BDP[3] consider that the resistance is acceptable but only by relying on some of the additional restraint provided by the steel angles installed as part of the strengthening measures after 1968. Thomas Akroyd[4] reach a similar conclusion, although with some differences of emphasis.

5.16 The available information on the design and condition of the non-loadbearing cladding is assessed in Annex G where it is concluded that the design is adequate. It is also concluded that, in view of the difficulties of inspection and guaranteeing the presence of adequate fixing, the provision of additional fixings of the panels to the building and between the two skins of the panels, as recommended by BDP[3], is a practical long-term solution.

Calculation of stresses in NE flank wall, as designed

5.17 It is clear from the plan form of the building that the highest stresses will be induced when the wind is acting on the long north/south faces. To calculate the stresses it is necessary to consider combinations of the dead, imposed and wind loads and the same procedure as that adopted by BDP is used. Since the line of action of the vertical loads does not intersect the lateral neutral axis of the wall a moment on the wall results. Stresses are calculated therefore using the following method assuming the loads act through the longitudinal centre line of the inner leaf. It is also assumed that behaviour is linear elastic and the three individual panels which make up each storey-height flank wall act compositely together. The values obtained are given in Table 2.

Stress (D(Dead) + L(Imposed)) = $P/A + M_1y/I$

where P is the total vertical load
A is the cross sectional area of the wall
M_1 is the moment due to asymmetry
y is the distance from the neutral axis to the position considered - this is taken as the distance to the appropriate end of the wall
I is the inertia of the wall about its neutral axis

Stress (Wind load) = M_2y/I

where M_2 is the moment in the flank wall produced by the wind load.

5.18 Two cross-sections of the wall are considered in the calculation of stresses:

(1) The 'solid' section which is 8.61m x 0.15m
$A = 1.29\ m^2$ $I = 8\ m^4$

(2) The 'reduced' section above the window opening where the effective opening has been taken as 1.1 m starting 0.9 m from the corridor end.
$A = 1.13\ m^2$ $I = 6.42\ m^4$

The calculations consider the full section at the H2 joint and assume that the neutral axes are in positions as appropriate to the different sections of the wall and do not take account of the corridor walls which would tend to reduce the stresses in the parts of the flank wall adjacent to the corridor.

TABLE 2: CALCULATED PEAK STRESSES AT THE END OF FLANK WALL IMMEDIATELY BELOW LOWEST H2 JOINT

Load combination	Wall section considered	Resulting stresses West N/mm^2	Resulting stresses East N/mm^2
D+L	solid section	3.8	5.6
D+L+W1	solid section	2.6	6.8
D+L+W2	solid section	2.3	7.1
D+L	reduced section	6.1	4.9
D+L+W1	reduced section	7.8	3.5
D+L+W2	reduced section	8.2	3.1

W1 is wind load arising from the CP3 50-year return period wind (1.56 MN) - Annex C - and, W2 (1.98 MN) is that derived from the more detailed analysis - Annex D.

5.19 The stresses are lower than those calculated by BDP (even when using the higher wind load W2) as a result of reducing the window size to that shown on the original design drawing and reducing the proportion of wind load carried by the flank wall (see 5.11 and Annex E). It appears that the various reasonable assumptions about load sharing are more likely to lead to calculated peak stresses lower than those given in Table 2 rather than the reverse. It is worth noting that the effect of the window is for higher stresses to be induced by easterly than westerly winds. As a basis for comparison, the dead and imposed loads distributed uniformly along the flank wall without openings would give an average stress of 4.7 N/mm^2 using the BDP loads in Table 1.

Basis of design of concrete panel structures to resist normal loads

5.20 Following the guidance of CP114[14] and CP116[15], the calculated stresses should not exceed appropriate tabulated permissible stresses. The permissible values have been obtained by reducing the nominal compressive strength of the concrete by a factor of safety. It is arguable, however, that for lightly reinforced concrete, particularly in the upstand of panels, the guidance of CP111[20] for plain concrete is more appropriate.

A different approach is adopted in the more modern limit state, partial factor, format of CP110[16] whereby the allowances for uncertainty and variability are ascribed both to loads and to strengths. The latter code also treats combinations of loads, particularly those including wind, in a different manner. The guidance of the codes on acceptable stress levels is reviewed in Annex F.

5.21 For convenience of comparison of the available information, the BRE appraisal has been based on permissible stresses. This appraisal makes use of other relevant information about Ronan Point itself but is not concerned specifically with design guidance and practice at the time of Ronan Point's construction.

Permissible stresses

5.22 Approaches to the assessment of permissible stresses relevant to an existing building such as Ronan Point should take into account codified data, possibly modified in the knowledge of actual concrete properties rather than assumed or designed values, contemporary and current codes of practice, relevant experimental data and, in the case of a composite joint such as H2, the load path through the various elements making up the joint.

5.23 Although the Tribunal cited (see paragraph 112 of the Report of the Inquiry)[1] the H2 joint as an example of workmanship below the desired standard their concern appeared to centre on the joint's ability to resist lateral loads rather than vertical loads. Tests commissioned by the Tribunal (Ibid para 120) were directed at the same objective. Subsequent tests examined the strength under vertical load[21].

5.24 There is no issue about the ability of the wall panels themselves to sustain the induced stresses given in Table 2. Much of the recent site investigation and calculation by the consultants, therefore, has been directed at assessing the current in-situ strength and the actual load-carrying area and mechanism of the composite H2 joint, given its complex nature containing precast and in-situ concrete, dry-pack mortar and strengthening steel. These aspects are considered in detail in Annex F.

5.25 There is a broad concensus that the calculated stresses for design in direct compression should not exceed about 10-15 N/mm^2 on a suitably reduced cross-section, although locally within a cross-section higher stresses might be acceptable. The higher value is considered appropriate in this appraisal since the investigations of the structure of Ronan Point have removed a proportion of the uncertainties concerning the quality of the construction which are covered by the margins of safety used in design (see also 4.5), BDP incline to the lower value in respect of the wall panel concrete. For the in-situ concrete at the joint they consider much lower stresses, being primarily concerned with the behaviour of the composite joint for which only limited test data are available.

Comparison of induced and permissible stresses

5.26 Unanimous conclusions of all commentators are that the dead and imposed loads are being transmitted safely from the flank wall panels at second-storey level to those below, and in addition that the high wind load already experienced was transmitted safely without causing distress. The essential issue for normal loading is whether, by generally accepted standards, an adequate margin of safety exists under these conditions and whether it will remain for the life of the building.

5.27 A range of views has been formed by the various parties on appropriate values for the levels of stress in the H2 flank wall joint. Thomas Akroyd consider the shortcomings of the H2 joint as of no particular importance (5, although without bringing forward calculations), beyond the need for remedial work to make-good voids. They have other concerns regarding behaviour of the joint under abnormal loading (see Section 6) but not related to quality of concrete in the joint.

5.28 BDP consider Method A of Annex F, using the loads given in Table 1, and conclude that dead, imposed and wind loads impose stresses of over 20 N/mm^2, considerably above the permissible stresses, before making allowance for incomplete dry-pack. Their concern is that stresses are at the upper levels of what might be considered acceptable and thus view the presence of defects to be of primary significance. However if some of the beneficial allowances of Method B of Annex F are taken, and without taking the mitigating effects of alternative load paths into account, an induced stress of 8.2 N/mm^2 (D+L+W2, reduced section in Table 2) is increased to over 13.0 N/mm^2, still about the upper value of permissible stress of 15.1 N/mm^2 deduced from Annex F, Method 1. The peak stresses at the extremities of the walls are significantly influenced by the presence of the window adjacent to the corridor and the lateral eccentricities of the loads. It should be noted that these calculated peak stresses are likely to be reduced in practice by local redistribution.

5.29 In both cases (Methods A and B) taking an assumption that sound dry pack is present over only half the width under the panel would double the stresses in the dry-pack. An examination of the available data from the BDP inspections of the flank wall has indicated that it is conservative to assume that only a half width of dry-pack is sound (Annex L). Unless the voids are uniformly distributed across the width of the wall, incomplete dry-pack, together with voids in the in-situ concrete, will lead to local stress concentrations in the composite precast/in-situ zone giving rise to larger stress peaks internally. Although these stresses are still below the failure stress, the margin of safety locally is regarded as unacceptable.

5.30 It is important to note that these maximum stresses arise only at the second-storey level and there only at the ends of the flank walls. Stresses will be lower at locations within the wall lengths and also increasingly less for storey heights above the second. The high stresses occur therefore only at a few locations. It should also be noted that these locations, the lower corners of the building, are those where local damage, eg joint failure, is thought to be one of the less critical in respect of inducing a progressive collapse (see Section 6 and Annex H).

5.31 The maximum stresses above may be compared with failure stresses in the region of 25 N/mm^2 obtained in the only tests[21] for which information has been found. These tests were on well-made joints and, whilst the concrete strength in the wall panels of Ronan Point may well be above the specified value of 39 N/mm^2, the difference between the values calculated above and those measured in the tests supports the view that the H2 joints at some locations in the lower storeys of the structure should be considered to have an unacceptable margin of safety. Remedial measures would be needed if Ronan Point were to be brought back into service. They are discussed in Section 10.

Dynamic behaviour

5.32 As would be expected for a building of such proportions there is some dynamic effect of wind loads on Ronan Point (see 5.8 above and Annex D).

5.33 The overall stiffness of Ronan Point was found in the vibration tests(6) to be rather greater than the average suggested by the limited data for other high buildings. This measured stiffness provides a reassuring indication, in addition to that from its record in service, of the integrity of the building under normal loads. However, it does not give insight into the strength or margins of safety of the building against collapse under overloads or abnormal loads since there is no simple relationship between stiffness and strength of structures.

5.34 Certain features of the dynamic response are attributed to the reconstruction work after 1968.

Other structural joints

5.35 Attention has focussed on the 'Type A' H2 joint because it is likely to be the most critical. It receives lateral support from only one side and it was difficult to construct. It is also a critical joint for the performance of the building under abnormal loads (see Annex H), as was shown by the collapse in 1968.

5.36 Other relevant joints are H4 at internal cross walls, H7 at non-loadbearing partitions, H14 at corridor wall, H23 wall panels-to-podium, V3 at flank to corridor wall and V13 between flank wall panels. Each are considered in turn below.

5.37 H4 is reported to be generally of satisfactory quality[4]. By the nature of its design it would have been inherently easier to place and compact the in-situ concrete, so as to achieve the required quality, than in an H2 joint. In considering the ability of an H2 joint to transmit moment in a fire situation in which adjacent floor and wall panels may deform, it was shown in full-scale tests at the Building Research Station[25] that the H2 joint is capable of transmitting some bending moment. It is concluded that an H4 joint should have an even greater capability, particularly under the higher vertical load at lower floors. This behaviour will contribute beneficially to the overall performance of the structure.

5.38 H7 which was designed with a 'soft' joint at the top, is reported to have been constructed generally in accordance with design[4]. Evidence from the fire test(7) and the collapse (plates 1 & 2 of reference 1) shows that the non-loadbearing walls are capable of carrying substantial load. Under abnormal load conditions this is likely to be beneficial (see Annex H).

5.39 H14 is reported to be generally of satisfactory quality(4). It supports the edge of adjacent floor panels. This offers a marginal reduction of load on the highly stressed part of the flank wall (see above).

5.40 H23 consists of the inner leaf of the wall panel bearing across its full width through dry-pack mortar and in-situ concrete onto the reinforced concrete of the podium structure. Drawings show that during construction the panels were levelled on timber blocks 4" x 6" x 1 1/2" thick or equivalent shims before the gap below the panel was filled with mortar. Although the quality of this joint was not commented on by the consultants, it would have been easy to construct, as would similar joints at the base of the internal loadbearing crosswalls. This form of joint acts in simple direct bearing (without significant effects arising from different materials and misalignments as in the H2 joint). The peak induced stress at the end of the flank wall will be less than 9 N/mm^2, well within acceptable values, thus affording an adequate margin to accommodate any likely shortcomings in construction.

5.41 Joint V3 is a plain mortared vertical butt-joint and is of little structural significance other than in transmitting wind pressure directly from the flank wall to the longitudinal walls.

5.42 Joint V13, as designed, is a strong connection. All joints were inspected visually by the consultants and no instances of cracking were observed. Their construction internally between panels was not examined. In practice the vertical bar may not have been threaded through the loops projecting from each panel, or the bar may have been short or absent. It is understood that grout rather than concrete was used to fill the long narrow castellated void. Thomas Akroyd[4] suggests the joint would have been easy to form, and argues that a poorly constructed V13 would have serious structural consequences and show visible defects. However such joints failed in the collapse without apparent damage to remaining adjacent panels.

5.43 The performance of V13 is relevant to the assumptions inherent in 5.5 and Annex B concerning the load distribution in the flank wall. It is of possible significance for the behaviour under abnormal loads (Annex J).

Conclusions

5.44 Ronan Point has safely carried dead, imposed and wind loads amounting to a substantial proportion of full design normal loads without showing any signs of distress.

5.45 Assessment of the stresses in the H2 joints under normal dead, imposed and wind loads taking into account the quality of their construction, suggests that the margin of safety locally at the ends of the flank walls in the lower storeys of the building is unacceptable.

5.46 Remedial measures would be needed to enhance the margin of safety at the highly stressed locations if Ronan Point were to be brought back into use. Overstress of the H2 joints is conservatively judged to be restricted to the lower eight storeys.

5.47 In the light of the likely difficulties of extensive inspection, the provision of additional fixings of the non-loadbearing panels to the building and between their two concrete skins is considered a practical precaution.

6. STRUCTURAL BEHAVIOUR UNDER ABNORMAL LOADS

6.1 The loads envisaged under this heading include those generated by gas explosions and thermally-induced displacements, particularly those due to fire.

6.2 Although not stated explicitly, the requirements of the Building Regulations[28] relating to abnormal loads for which safeguards against progressive collapse should be provided, were based on gas explosions. The requirements were not meant specifically to provide safeguards against thermally-induced displacements due to fire, but they have the effect of giving some protection to occupants and the structure in those circumstances. The need for such safeguards was mentioned in Circular 62/68[2].

6.3 The philosophy of safeguard against fire effects in building structures rests on the use of compartmentation to restrict fire spread within the building (see Section 8) and protection of the structure itself from the weakening effects of fire, so that it will remain stable for a sufficient period to allow evacuation and fire-fighting.

6.4 It is clear from Circulars 62/68 and 71/68[2] and discussions by BDP[3,8] and Akroyd[4,9] that the issues, objectives of design and design solutions for abnormal loads are complex.

Basis for design

6.5 The structural design philosophy for dealing with abnormal (accidental) loads is that the extent of any damage should not be disproportionate to its cause, eg see Code of Practice CP 110(16). This approach accepts that it is uneconomic, if not impossible, to avoid damage from some events. It expresses the aim that a limited degree of damage should be accommodated without jeopardising the whole structure, ie the structure should be robust, rather than sensitive.

6.6 Building Regulations[28] place limits on acceptability by defining a notional limiting extent of damage and give a prescriptive solution. In general the designer uses the prescribed solution and does not need to identify appropriate abnormal loads[17].

Design Strategy

6.7 The possible elements of a strategy for design for abnormal loads are[12]:

(i) The provision of multiple, independent load-paths (redundancy).

(ii) The inclusion of devices to allow the building to avoid carrying load.

(iii) The provision of local or general increase in resistance to enhance overall strength above that necessary to resist ultimate design for normal loads.

(iv) The installation of environmental and performance monitoring and control systems.

6.8 Method A of Circular 62/68[2] is encompassed by (i) above. Venting of explosions and expansion joints are examples of (ii). Current tying recommendations are examples of (iii) which relates also to Method B of Circular 62/68. This approach effectively raises the threshold at which significant damage occurs. Feedback from use and the protection of the structure from unplanned loads, eg vehicle impact or misuse by occupants are encompassed in (iv).

6.9 Combinations of the elements (i) - (iv) are generally adopted in buildings, depending on the nature of the structure and the possible consequences of failure. The overall aim is to make the risk of disproportionate damage in service remote by the use of an optimum combination. In practice the determination of an appropriate strategy may be partly implicit in the design. Elements of the strategy, eg redundancy (i), may be a natural characteristic inherent in a particular design to provide for normal loads. There may be no need for further specification beyond the need for checking. Such checking may commonly take the form of a 'notional removal of structural elements' and examination of the remaining structure for its ability to remain stable.

Gas explosions - pressure generated in buildings

6.10 In an explosion burnt gas is usually able to expand into other parts of the building or escape by venting through openings to the atmosphere. Thus in practice the peak dynamic pressure is usually much less than the theoretical maximum and rarely exceeds 20 kN/m^2 (3.0 lbf/in^2). Nevertheless this pressure is likely to cause damage to brick walls and other traditional constructions. Figure 3, adapted from reference 18, is based on assessment of pressures in explosions of town gas or natural gas in buildings.

6.11 Explosion of a gas requires oxygen, as in air. A stoichiometric mixture is one which contains just sufficient gas to burn all the oxygen in the air in the mixture. It is in these circumstances that the theoretical maximum pressure may be approached if the explosion is totally confined.The proportion of propane in a stoichiometric mixture is 4.0 per cent by volume, for butane it is 3.1 per cent, for natural gas (predominantly methane) it is 9.5 per cent and for town gas about 20 per cent depending upon its exact composition. However, gas explosions will occur with gas/air ratios for propane between 2.2 and 9.5 per cent, for butane between 1.9 and 8.5 per cent, for natural gas between 5.0 and 15.0 per cent and for town gas between 4 and 30 per cent.

6.12 Consideration of these proportions and the air volumes in the rooms of TWA buildings indicates that for 'bottled' liquefied petroleum gas (LPG), ie liquefied butane or propane gas in cylinders, only that used for space heating or for building repair work could occur in sufficient quantity (ie cylinders containing 15 kg or more of liquefied gas) to lead, on leaking, to an explosion which might be of structural significance. Other 'bottled' gas such as that contained in cigarette lighters, aerosols and camping equipment could not present a threat to the structure.

6.13 The pressure variation in a gas explosion is complex and dynamic magnification can change its effect on a structure considerably. The maximum pressure which can be reached by a fully confined stoichiometric gas/air mixture is about 700 kN/m^2 (100 lbf/in^2). This far exceeds the pressure envisaged when dwellings are designed. Gas explosions rarely approach their maximum potential pressure since they will not be fully confined (Figure 3), nor will the gas/air proportions be optimum or even if they are, will they be well mixed.

6.14 There are two conceivable ways in which an explosion associated with an LPG cylinder and heating appliance might possibly occur. Firstly an accidental fire might heat a cylinder to such a extent that it would explode. However, the risk of an explosion arising in a fire in this way has been virtually eliminated in modern cylinders by fitting pressure relief valves designed to open and relieve internal pressure safely and without explosion. The gas emitted may well ignite and burn as a jet. As the pressure in the cylinder decreases the flame shortens, and the valve may close, causing the flame to extinguish. Should the external fire continue, pressure in the cylinder may rise again and a further cycle of discharge and burning of gas may occur. The heat generated by burning of the gas will be only a fraction of that resulting from the burning of the contents of the room.

Secondly, unburnt gas may leak from heating appliances and cylinders into the surrounding air producing a mixture which would explode on ignition. Modern heating appliances and cylinders are both protected by mechanical means designed to minimise leakage of unburnt gas. Construction of appliances is specified in British Standard 5258, Parts 10 and 11, 1980[34]. These precautions have ensured that the risk of leakage of significant quantities of unburnt gas is small.

6.15 LPG used for space heating indoors is butane. Explosions of LPG, if leakage occurs from cylinders used for domestic space heating, may produce different pressures compared to town gas or methane. In terms of burning velocity under standard conditions bottled gas comes between town gas and natural gas, and is nearer to natural gas. However, burning rates are accelerated by turbulence and hence explosion pressures are increased.

6.16 In this respect natural (and town) gas and bottled gas have different characteristics. Both town and natural gas are less dense than air so that there will be a tendency for the flammable volume to be in the upper part of the room. Should ignition occur, the most likely source for venting of the explosion will be breakage of windows which are in the upper parts of the walls. Thus the burning has direct and easy access to the exterior, without impinging significantly on the furniture.

6.17 Bottled gas is more dense than air so that any flammable mixture is likely to be in the lower part of the room. When ignition occurs, flame propagation will be influenced by the presence of furniture which will tend to generate turbulence. Where the most likely vent is at a relatively high level, ie a window, access to it is indirect. This situation leads to the possibility of more turbulent explosions and thus relatively high pressures.

6.18 These considerations suggest that marginally higher pressures may be generated in buildings by explosions of bottled gas than by those due to natural or town gas: but the circumstances are sufficiently different to require investigation before this opinion can be validated in respect of burning characteristics and volume of leakage.

Gas explosions - design to safeguard against disproportionate structural damage

6.19 The complex nature of pressures in gas explosions (in both time and space) has led to the use of equivalent uniform static pressures as a practical basis for designing structural resistance. The value of 5 lbf/in^2 (34 kN/m^2) was chosen[2] as a practical design target in the absence of unequivocal objective data.

6.20 The nominal 2 1/2 lbf/in^2 to which some TWA buildings over 6 storeys in height were strengthened after 1968 was a compromise based on a typical upward failing pressure of a ceiling slab in Ronan Point (see reference 1, para 124). Strengthening to higher pressures would have required redesigned floor slabs.

6.21 The value of 34 kN/m^2 is used in CP110 as a basis for design recommendations for tying and for structural elements which must retain their function under accidental situations (see (iii) in 6.7 above).

6.22 Figure 3 shows that higher pressures can occasionally occur. However, in view of the extreme impracticality of designing dwellings to resist the maximum 'stoichiometric' pressure, a balance has to be drawn between risks to life and property and the costs of minimising the risks. The requirements of the Building Regulations are intended to ensure the risk of disproportionate damage is negligible.

6.23 Despite the considerable research and attention by designers and Code of Practice committees to the problem of designing to resist abnormal loads there has not been the same widespread agreement on suitable detailed procedures as for normal loading. A significant part of the difficulty is that overall structural response under abnormal loadings is dependent on the form of the individual structure. Thus a '5 p.s.i.' building may have a higher threshold of local failure than a '2 1/2 p.s.i.' one of otherwise identical form and conditions of use. However, if the structural form or conditions of use are different, the subsequent response of the two to abnormal load might be quite different; in particular the risk of progressive collapse following local failure need not necessarily be different or less favourable in the '2 1/2 p.s.i.' building.

6.24 Despite the arbitrary nature of the '5 p.s.i' value, no clear evidence or prevailing professional view has come to light supporting the use of a higher pressure for normal buildings, as proposed for Ronan Point by Thomas Akroyd[10]. Although the judgement in the Trial in 1979[37] found that an equivalent static pressure of 84 kN/m^2 (12 p.s.i) was generated, the judgement in the Appeal was that the explosion was of unusual violence in a domestic setting[38].

6.25 The strengthening of Ronan Point and similar buildings to '2 1/2 p.s.i' after 1968 was designed to make the risk of disproportionate damage remote using the principle of 6.7(iii) above in conjunction with the mitigation afforded by principle 6.7(iv) in removing piped gas from such buildings.

6.26 Some 9% of the volume, or floor area, of the building was affected by the collapse in 1968 (Annex H). The stability of the remaining structure and its ability to resist normal loads was not impaired by the severe accidental load.

Gas explosions - remedial measures to safeguard against disproportionate structural damage

6.27 The various proposals by BDP[3] and Thomas Akroyd[9] for further remedial measures to Ronan Point may be classified according to the principles in 6.7:

(1) Additional structural frame - (i)

(2) Improved venting - (ii)

(3) Stronger tying-back of panels - (iii)

(4) Removal of cylinder gas - (iv)

6.28 BDP[3] consider that the strengthening of the H2 joints after the collapse in 1968 provides part of the security under normal local wind loads implying that a 'notional removal of structural element' check indicates the structure to be inadequate to resist progressive collapse following loss of a flank wall panel as a result of an explosion. Thomas Akroyd[9] forms a similar view. Thus the aim of the proposed additional structural frame[3,9] is to provide alternative load paths(6.7(i)) for abnormal loads and, in the case of BDP, for weak structure(4.1) in the form of understrength H2 joints.

6.29 Given that the uncertainties of the performance of the H2 joints under normal loads are removed by repair or provision of an alternative load path, it is considered that the likelihood of disproportionate damage from a gas explosion, already exceedingly small (remote), could be made more remote if desired by adoption of 6.27(4) above possibly together with (2) - see 6.30 and 6.31 below.

6.30 The risk of gas leaking sufficiently from an LPG cylinder to produce an explosive mixture capable, on ignition, of producing significant pressures on a building structure is small[17]. Such mixtures would not necessarily be ignited in every case. Management procedures to prevent LPG cylinders from being taken into Ronan Point by tenants or workmen, were it to be returned to use, would substantially reduce - although not eliminate entirely - an already small risk of occurrence of ignition of an explosive mixture in the building. The resulting risk cannot be quantified precisely but it is considered to be very small.

6.31 This very small risk of occurrence has then to be considered together with the risk that such a gas explosion would cause progressive collapse of the structure. This latter risk is judged also to be very small. In any particular case there is a very low probability that the pressures produced on the structure will be sufficient to remove part of the load-bearing construction so that it is no longer able to carry the normal loads present. The strengthening of Ronan Point to the 2 1/2 lbf/in^2 limit will have had the effect of substantially raising the threshold of load needed to remove a part of the load-bearing construction although the real extent of the increase cannot be quantified precisely. In addition, the location of the part of the construction which has been removed is unlikely to be at a trigger level (Annex H) where there is least resistance to the development of a progressive collapse. Also the enhanced continuity in the strengthened three-dimensional construction may well be effective (Annex J). These considerations taken together support the suggestion that the risk of progressive collapse following an LPG explosion in Ronan Point would be very small. The combination of this very small risk with that associated with the likelihood of a significant LPG explosion occurring leads to the conclusion that the risk of a progressive collapse following an LPG explosion in Ronan Point is exceedingly small (remote) and would be made more remote if steps were taken to prevent LPG cylinders being in the building. If the widespread use of LPG cannot be prevented by management measures or incentives, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2. The former alternative, removal of cylinder gas, is preferable since it would remove also the adverse effects on the living conditions in the flats arising from the production of water vapour and combustion gases when LPG is burnt. The requirement of the latter alternative may be difficult to meet in practice.

Thermally-induced displacements

6.32 Thermal movements arise from the effects of internal (underfloor) heating of the building, changes in atmospheric environment and accidental fires.

6.33 The first two effects occur in building structures in normal service. They are not usually considered explicitly in design, the effects generally being small compared to those of dead, imposed and wind loads. For Ronan Point, the consultants' considerations ranged widely to encompass all effects that might adversely affect the integrity of the structure. Attention focussed on the effects of underfloor heating on the overall structure, of movement of non-loadbearing cladding particularly in relation to fire containment, and of fire on the structure.

Effects of underfloor heating

6.34 Structural movements were measured by Thomas Akroyd[4] with only three adjacent floors heated. They report a maximum elongation of 3.5 mm and conclude that there was 'no effect on the structure '.

6.35 BDP[3] calculated the relative lateral movement of adjacent floors, when only one was heated to be 2 mm, as part of their assessment of the sensitivity of H2 joints to lateral movement. It is here assumed this movement is made up of 1 mm at each H2 joint at both north and south ends of the building. They consider both the additional horizontal forces and distortion induced at the wall to floor joints and also the increased stresses (estimated to be about 4 N/mm^2 on a reduced area) induced in the H2 joint by the wall panels tending to rotate relative to the floor. They conclude that the former can be accepted by the 'as built' connections, but that the latter must be added to the stresses caused by normal loads. The condition assumed - only one floor heated or not heated - is likely to occur rarely and it is thought the increase of stress would be smaller than 4N/mm^2 in practice.

Movement of non-loadbearing cladding

6.36 Measurements of cladding panel movement due to changes in ambient conditions were made by BRE[19] in support of the BDP investigations[3]. A range of movement of 2.5 mm at right angles to the length of the panel was reported. It is of no significance to the integrity of the panels as long as their fixings are sound (see Annex G).

Effects of fire on the structure

6.37 A fire in a flat in Ronan Point would cause adjacent wall units and the floor of the flat above to bow towards the fire as a result of the temperature gradient through the concrete. However, with present knowledge, the extent of the movement of the structure under various realistic types of fire cannot be precisely predicted. Whilst research would be needed to develop accurate methods of prediction, an engineering assessment of the potential effects on the structure can be made.

6.38 The construction of the wall and floor panels is reported[3] to provide fire resistance for periods in excess of one hour. It would appear to comply with the requirements of the Building Regulations[28]. The effect of thermally-induced movements on the joints between components is potentially the most critical aspect of the structure under fire load.

6.39 The extent to which these movements will be transmitted through the structure and affect the H2 flank wall joint for example, depends on amongst other things, the overall structural layout and restraints.

6.40 Measurements by BRE indicate[7] that a horizontal expansion of 5 mm could be generated in the floor above a severe but realistic fire (in which the crosswalls were protected). The movement may then be transmitted to an H2 joint, relative to the adjacent floors.

6.41 BDP[3] treat this event as in 6.35 above, deducing proportionately higher lateral forces and eccentric vertical stresses and include the circumstance of a severe fire arising simultaneously in two adjacent rooms - a remote eventuality. The fire test[7] was made by a simulated living room fire. Adjacent rooms in a flat would not have so much furniture and so would not provide such a severe loading. In the not unlikely event of fire-spread within a flat, there would be several minutes delay, and therefore some staggering of the peak effects in adjacent rooms. Statistical evidence suggests the risk of fire-spread between flats is negligible.

6.42 BDP[3] consider that the stress magnitude and distribution arising from a fire in two adjacent rooms are different to anything experienced so far. The stresses are thus considered unacceptable by BDP, in the absence of test evidence as to the behaviour of the joint when subjected to imposed distortions, since they would be superimposed on stresses already high due to normal loads, and sudden failure of an H2 joint could occur in compression. It is implied that disproportionate damage, ie progressive collapse, would follow.

6.43 Thomas Akroyd[9] considers this event only in the context of lateral restraint and conclude there is no problem at the H2 joints arising from thermal expansion by fire.

6.44 For abnormal load conditions such as may arise in fire it is normally considered valid to design to ultimate strengths without applying factors of safety, and not to consider all potentially adverse conditions to apply simultaneously. The BDP[3] assessment in respect of the structural effect of fire is based on the much more conservative assumptions appropriate to design against normal loads. They assume distortions due to fire would cause transfer of load from one part of an H2 joint to another which may be a zone of defective workmanship unable to support load. However Ronan Point has withstood the movement resulting from a severe fire ,and other fires are understood to have occurred in 'Type A' TWA buildings without precipitating even local structural failure. In addition, the chance of a fire occurring in a TWA building which would induce stresses in the structure of equal or greater magnitude than those developed in the fire test is very small. In view of the size and distribution of accidental domestic fires and the three-dimensional nature of the structure of Ronan Point, it is suggested that instability of the load-bearing walls is extremely unlikely to occur arising from fire. Overall it is concluded the risk of fire inducing disproportionate damage in Ronan Point directly resulting in substantial loss of life, is remote. This conclusion assumes the provisions in the building for fire containment are adequate (see Section 8). In addition if remedial measures are carried out to enhance the margin of safety at the highly stressed locations identified in Section 5 the ability to accommodate the structural effects of fire will be further increased.

6.45 However, should additional assurance be needed concerning the performance of the structure in fire, measures could be taken, using the principle of 6.7 (iv), to reduce the structural effects of fire on Ronan Point by enhancement of the fire protection. A simple method of enhancement - mentioned by Thomas Akroyd[9] - would be to fix a non-combustible insulating covering to ceilings in lounges (the likely locations of most rapid intense fire) and possibly also bedrooms and flank walls. Suitable materials could be confirmed by calculation and test, both as regards their thermal properties and also the requirement that they shall remain attached and effective during a fire, even though there may be some thermal movements of the slabs.

Conclusions

6.46 If ignition of an explosive mixture of LPG (butane) occurred following leakage from cylinders used for space heating the explosion might produce marginally higher pressures than explosions of town gas or natural gas. The risk of an LPG cylinder exploding in an accidental fire has been virtually eliminated in modern cylinders.

6.47 The strengthening of the joints of Ronan Point using angles will have raised the threshold at which an explosion could cause progressive collapse. Since the characteristics of a random gas explosion cannot be quantified precisely, objective design criteria cannot be set. There is no case for raising the design specification from 17 kN/m^2 to 68 kN/m^2, or even 34 kN/m^2 provided that effective steps are taken to prevent the use of LPG for space heating.

6.48 Ronan Point and other 'Type A' TWA buildings which were strengthened to the equivalent of 17 kN/m^2 standard static pressure should not be considered for use with piped gas. The risk that a gas explosion arising from the presence of an LPG cylinder would cause progressive collapse is considered to be exceedingly small. This risk could be made more remote if greater assurance of safety is required. To that end LPG cylinders for space heating should be removed from the buildings and steps should be taken to ensure LPG is not used by tenants or workmen in the future. If the widespread use of LPG continues the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2. The former alternative is preferable since it would remove the adverse effects on living conditions of water vapour and combustion gases produced when LPG is burnt.

6.49 Underfloor heating has no significant adverse effect on the integrity of the structure.

6.50 Movements of non-loadbearing cladding panels arising from changes in ambient conditions are of no structural significance provided their fixings are sound.

6.51 Fires which have occurred in TWA buildings have had intensities which were considerably less than that produced in the fire test[7] although their duration may have been longer.

6.52 The probability of fires occurring with peak intensity simultaneously in two adjacent rooms is remote and not a major consideration.

6.53 The risk of fire inducing disproportionate damage in Ronan Point - progressive collapse of part of the structure - resulting in substantial loss of life is remote.

6.54 Should additional assurance be needed concerning the performance of the structure in fire, enhancement of the fire protection to ceilings in lounges and possibly bedrooms and flank walls could be provided.

6.55 The effects on the structure of a fire in a single room are likely to be small, although not necessarily insignificant at joints when considered superimposed on the stresses arising from normal loads in the H2 joints at the ends of the flank walls in the lower storeys. Enhancement of structural adequacy relating to the effects of normal loads at these locations would provide more capability for accommodating the structural effects of fire. This conclusion is further supported by the consideration that applied stresses due to normal loads at the time of a fire, are most unlikely to approach the maxima assessed in Section 5.

7. DEGRADATION OF THE STRUCTURE

7.1 Degradation refers here to deterioration in material and structural properties with time (even though the overall load and environment does not change significantly) giving rise to loss of strength of the structure.

7.2 The potentially significant effects for Ronan Point are shrinkage and creep of concrete, overloading of erecting bolts and loss of corrosion protection to reinforcement.

Shrinkage and creep of concrete

7.3 Highly stressed concrete may continue to deform over many years. This will not be harmful necessarily unless load is transferred to areas which cannot sustain such an increase. It could be beneficial, for example in relieving high stresses on the precast concrete upstand of an H2 joint by transfer to adjacent in situ concrete.

7.4 It is likely that any shrinkage or creep movement in Ronan Point has subsided by now and need not be considered further.

Erecting bolts

7.5 It has been found[3] that the nuts on the erecting bolts used for levelling wall panels during constructon are frequently tight. They were not released fully after completion of 'dry-packing', as intended. Reported deformation of the bearing plates[3] over the nuts indicates that substantial load has been transferred through some bolts.

7.6 Tests by the Building Research Station in 1968[25] show that the two bolts in a flank wall panel could carry a load equivalent to the weight of six storeys of construction, over the length of the panel. Failure occurred by yielding of the steel bearing plates.

7.7 BDP investigations using a radar technique suggest that concrete in which the bolts are embedded is cracked in the lower part of the building[3]. Close correlation is reported between radar indications and by direct examination of cores (Annex L). An interim review of the radar technique is given in Annex K. The technique is thought to have value as a means of screening flank wall joints in TWA buildings for voids.

7.8 As with creep of concrete, any movement of panels resulting from excessive loading of bolts is likely to have led already to redistribution of load in a manner now stabilised. The precast concrete of the wall panels would be able to sustain any remaining localised high stresses around the shank of the erecting bolt.

Corrosion protection of reinforcement

7.9 Long-term protection of the steel reinforcement of the concrete structure depends on the maintenance of a passive environment by the concrete or the prevention of access by water and air.

7.10 The consequences of corrosion of the steel reinforcement would be progressive cracking and spalling of the surrounding concrete. Initially a hazard from small pieces of spalling concrete and a loss of appearance might arise. Continued loss of concrete and cross section of steel would lead eventually to a loss of structural integrity. Potentially such deterioration might occur to non-loadbearing cladding panels and the outer skin of flank wall panels, as well as the loadbearing walls.

7.11 The sampling pattern for chemical tests on concrete at Ronan Point has not been examined, nor have individual results of tests. BDP reports[3] low background amounts of chloride in the outer side of cladding panels. Chlorides do not appear therefore to be a potential problem in the cladding.

7.12 Carbonation testing is reported[3] to have been 'spread reasonably evenly over all four elevations'. The tests were made only from the exposed external surface, i.e on the cladding part of external components, and indicated carbonation depths up to 20 mm. Over two hundred measurements of cover to reinforcement were made. Cover was found to be generally in the range of 30-45 mm with some instances of only 20 mm. These values suggest that a small number of corrosion problems may be expected to occur in the cladding at increasing frequency over the next 10 years and beyond[30]. Protective coatings as recommended by BDP[3] should be considered as a means of reducing the rate of carbonation and thus increasing the life of the most vulnerable components. A detailed assessment of the measurements including consideration of carbonation depths from internal faces and at bearings would be needed to determine the extent to which coatings are needed. Local defects in mosaics and exposed aggregate panels would also need to be made good.

Rain penetration

7.13 Water, albeit in limited quantities, is necessary for carbonation and corrosion of reinforcement to proceed. External UK environments provide sufficient. Internal environments may be moist enough for carbonation to occur (probably more rapidly than externally) but too dry for corrosion of reinforcement to occur. Rain penetration (or condensation or spillages internally, eg in bathrooms and kitchens) may upset this balance and will promote corrosion of reinforced concrete inside a building if the cover is carbonated.

7.14 BDP[3] report no rain penetration through cladding and one instance of leakage at roof level. Thomas Akroyd[4] make no comment, whilst Webb[5] reports water penetration into the loadbearing structure, especially the H4 joint, and the roof. Such penetration could lead to long-term deterioration of the structural joints. A fuller investigation of these reports would be needed, if Ronan Point were to be returned to use, and action taken to prevent water penetration where it is confirmed to occur. Consideration of the likelihood of water penetration into the structure and the voidage and poor quality of the in-situ concrete in the H2 joints suggests measures might be considered, eg injecting a suitable material into voids, to enhance the protection of reinforcement in the joints. Water in the voids of floor slabs would not necessarily reach H2 joints since some voids adjacent to the joints were filled with concrete as part of the post-1968 strengthening measures.

Conclusions

7.15 The condition of the erecting bolts and the reported deformations of the bearing plates and local concrete cracking around the bolts are not considered to have a significant adverse long-term affect on the structure.

7.16 Corrosion of reinforcement and associated cracking and spalling of cladding panels can be expected to occur with increasing frequency over the next 10 years and beyond. The application of suitable protective coatings should be considered as a means of extending the life of the components most susceptible to deterioration.

7.17 Further investigation of rain penetration would be needed if Ronan Point were to be returned to use. Where rain penetration is confirmed action would be required to prevent it and thus inhibit long-term deterioration of the structure. Improvement of the protection of reinforcement in H2 joints against corrosion might be considered to give assurance of long-term durability.

8. FIRE CONTAINMENT

8.1 No shortcomings in the provision for fire containment between individual flats have come to light apart from the gaps observed at the junctions between the cladding panels and floor slabs.

8.2 The fire test was undertaken to determine the resistance to the penetration of flame and fire gases of a proposed joint system designed to close the gaps[7]. The joint system prevented flame spread through the gap but there was detail of the joint system could be made so that it would accommodate relative movements of the loadbearing walls and floor slab in fire conditions, thus preventing passage of any smoke.

Conclusions

8.3 Although smoke and fire gases may pass through small gaps, the passage of fire (flames) requires a certain minimum size. For a gap the depth of the floor slabs (11 in), it is likely that flame would be extinguished at a width less than 15 mm.

8.4 Gaps which occur in Ronan Point and other TWA buildings at the junctions between cladding panels and floor slabs are unacceptable since they could allow the spread of smoke and fire gases between adjacent flats.

8.5 The gaps could be effectively closed to the penetration of flame and fire gases by the installation of a joint such as that tested[7] but with a modified end detail to accommodate movements between the walls and floor slabs.

9. IMPLICATIONS FOR OTHER TWA BUILDINGS

'Type A' TWA buildings

9.1 There are over 80 'Type A' TWA buildings, 46 of which are over 6 storeys in height and were strengthened similarly to Ronan Point. These TWA buildings probably all have similar H2 joints in their flank walls to those at Ronan Point although differences in detail are known to be present in some cases. In addition to differences of storey height the buildings may well be different in plan structurally, and in other details of construction including the cladding panels. Consequently conclusions and recommendations in this report relating to Ronan Point may not apply directly to other 'Type A' TWA buildings. Nevertheless the conclusions drawn for Ronan Point are likely to apply to some extent and action is desirable to check the extent. Actions which might be considered are discussed briefly below.

9.2 Investigations of the buildings are desirable, if they have not been undertaken already, to determine the details and present condition of the structures and any needs for remedial action giving particular attention to:

(1) H2 joints in flanks walls, especially looking for poor quality in-situ concrete and dry-pack, voids and any signs of distress. Most 'Type A' buildings are likely to have acceptable margins of safety in respect of normal loads in the H2 joints of the lower storeys if they are soundly constructed. The H2 joints in buildings of 14 or more storeys should be appraised. Consideration should be given to the appraisal of the H2 joints in other 'Type A' buildings.

(2) the use of LPG for space heating, the desirability of preventing its use, and, if use cannot be avoided, the need to provide strengthening to resist forces equivalent to a standard static pressure of 34 kN/m^2,

(3) fire containment, especially checking for gaps at the junctions between cladding panels and floor slabs which may allow penetration of flame and fire gases, between adjacent flats,

(4) non-loadbearing cladding panels, checking the adequacy of fixings to the building and between their two leaves and the condition of the concrete, including carbonation depths, chloride contents, loose or missing parts of panels,

(5) rain penetration.

9.3 Assessment of the adequacy of the H2 joints as found will depend largely on the height and plan arrangement of the building. Clearly blocks which are the same height as Ronan Point, eg the five similar ones at Newham, are the most likely to need similar remedial treatments to the joints but depending also on their quality of construction. For those buildings of fewer storeys in height appraisal should seek to establish any areas of stress concentration in the 'as designed' condition and then make allowances for the adverse effects of poor quality construction in a similar way to that described in Section 5.

'Type B' TWA buildings

9.4 There are 43 'Type B' TWA buildings. Their flank wall joints are bigger than those in Ronan Point, contain interlocking reinforcement and are less likely to have been poorly constructed. Their design was checked and construction supervised by independent consultants. These factors all suggest that the design is likely to be adequate and construction of good quality. Confirmation of the design and the construction quality should be considered only if additional assurance of adequacy is desired.

TWA buildings of 6 or less storeys

9.5 An important difference between Ronan Point and TWA buildings of 6 or less storeys in height is that they were not strengthened after 1968 to enhance their resistance to abnormal loads although they were all of 'Type A' construction.

9.6 All buildings should be capable of resisting abnormal (accidental) loads, such as may arise from a gas explosion or vehicle impact, to the extent that the damage produced is not disproportionate to its cause. General guidance is given in Codes of Practice. For example, CP110[16] states,'The layout of the structure on plan, and the interaction between the structural members, should be such as to ensure a robust and stable design: the structure should be designed to support loads caused by normal function, but there should be a reasonable probability that it will not collapse catastrophically under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause'.

9.7 Specific requirements first appeared in circulars 62/68 and 71/68[2] and related only to buildings of over 6 storeys in height. Subsequently Section D17 of the Building Regulations[28] increased the scope to include all buildings of 5 or more storeys in height. There is no fundamental structural principle for a demarcation at any particular storey height. In favour of a demarcation is the proposition that vertical progressive collapse is inherently limited in low-rise buildings by the number of storeys, that such buildings by their nature will be designed and constructed in an inherently robust form, and also that a requirement to design them to comply with D17 would impose an unacceptable economic burden. However, the possibility in low-rise construction that the form of the structure may not be inherently robust[35,36] should not be ignored. In addition, the possibility of progressive collapse occurring horizontally (a 'domino' type effect) as a result of local damage caused by an abnormal load may need to be considered. Generally, the mechanisms required for horizontal progression in low-rise construction are more difficult to activate than those for vertical progression in high-rise construction.

9.8 It is concluded that appraisal of TWA buildings of 6 or less storeys in height should include consideration of their robustness and resistance against progressive collapse caused by abnormal loads, particularly where piped gas is installed or where LPG in cylinders is used for space heating.

10. STRUCTURAL APPRAISAL AND POSSIBILITIES FOR REMEDIAL ACTION

Structural appraisal

10.1 A major problem in determining the structural condition of a concrete building such as Ronan Point is to obtain access and make inspections at sufficient locations, especially the more critical ones. Techniques are available for determining factors relevant to the condition of the concrete, eg cracking, depth of cover, carbonation depth and chloride contents, and for making assessment of likely future performance[27,30]. The identification of voids in concrete, for example at joints, and the determination of the presence and condition of fixings across voids behind precast panels may present greater difficulties. Some techniques, eg optical probe, are available[27]. The radar technique used at Ronan Point was of value. It needs careful calibration against physical data, eg from cores, and, in its present form, a high level of expertise for its operation and interpretation of results. Evaluation of the technique is desirable to assess the range of its practical use and the possibilities for development for more general application to concrete structures (Annex K).

Remedial action on Ronan Point

10.2 Since the Council of the London Borough of Newham has taken a decision to demolish Ronan Point and five similar blocks, discussion of remedial action may perhaps be only of academic interest. However, the possibilities are discussed briefly below in case consideration is given to bringing the building back into use.

10.3 Remedial measures at Ronan Point are needed to enhance the margin of safety of H2 joints revealed at the ends of the flank walls in, say, the lower eight storeys in respect of capacity to carry normal loads. Essentially, there are four possibilities.

10.4 The first possibility is to repair or reconstruct the joints at the highly stressed locations in the lower storeys by making-good dry pack and in-situ concrete by filling voids and replacement of poor quality material, or by total replacement. This action may also be beneficial in providing assurance of long-term durability of the reinforcement in the joints, but is not needed for that reason alone since there is no evidence to suggest corrosion is progressing significantly. Repair would be difficult to carry out in such a way that the margin of safety against joint failure is clearly enhanced sufficiently. Total replacement of the in-situ concrete and dry-pack might be ineffective unless a sure way can be found to transfer the load to the new material: an underpinning-type technique might be considered.

10.5 A second possibility for enhancement of the margin of safety at the H2 joints in the lower storeys is to strengthen the joints at the highly stressed locations by the addition of specially-designed components of structural steel or concrete to the flank walls to provide a path for vertical load through the floor slabs.

10.6 A third possibility is to provide comprehensive strengthening by building an additional loadbearing system such as the structural frame tied through the building as recommended by the consultants[3,9]. It is only needed to enhance the margin of safety in relation to normal loads locally in the lower levels of the building, and would therefore only need to extend up to say, the eighth storey level. The detailed design of such a frame, which might be internal or external to the building, is not considered here.

10.7 The fourth and more radical possibility is to remove, say, eight storeys from the building thereby reducing the stresses in the H2 joint at the ends of the flank walls in the lower storeys due to dead, imposed and wind loads.

10.8 Steps should be taken to prevent the use of LPG for space heating. If the widespread use of LPG cannot be prevented by management measures or incentives, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2.

10.9 Mention has been made already (5.31) of the paucity of test data on H2 joints. A programme of loading tests in which the quality of construction of the joints simulates that found at Ronan Point, preferably together with measurements of strain during the demolition of a tall 'Type A' TWA building, would assist in confirming the adequacy of the remedial measures discussed above for Ronan Point.

10.10 Remedial actions to cladding panels are discussed in Sections 5, and 7 and in Annex G and to the gaps between cladding panels and floor slabs in Section 8.

Remedial action on other TWA buildings

10.11 Where appraisal of other TWA buildings indicates needs for remedial action, the principles discussed above may be appropriate.

11. NEEDS FOR RESEARCH

11.1 Research which could be completed in a short timescale useful to structural appraisals of TWA buildings in the near future are:

(1) Tests on sound and poorly constructed 'Type A' H2 joints, eg with and without voids in the dry-pack and in-situ concrete, to determine behaviour and vertical load-carrying capacities under varying degrees of distortion and lateral eccentricities of vertical load.

(2) Monitor lateral displacement and strains in walls caused by floor movement.

(3) Examine quality of 'Type A' H2 joints during demolition of a TWA block particularly for correlation with radar technique.

(4) Examine strain distribution in a 'Type A' H2 joint in the second floor of a tall TWA block during demolition together with measurements of strain distribution across floor and adjacent walls.

This research would produce information on the stresses and loadcarrying capacity at the more critical locations in the buildings and thereby assist in judging the extent of structural remedial measures that may be needed.

11.2 A number of areas where information is limited have been revealed during the preparation of this report which have implications for large panel system buildings more generally. Research is therefore desirable as follows:

(1) Tests to determine the pressures generated in LPG explosions and their structural consequences.

(2) Investigation of structural robustness, eg bridging action following removal of support from a wall panel.

(3) Investigations to establish whether there have been any significant changes in the incidence, nature and consequences of gas explosions in residential buildings.

(4) Investigation of performance of large panel systems in fires using theoretical analyses and tests on sub-assemblies including investigation of possibility of disproportionate damage in these circumstances.

(5) Determination of degree of structural coupling between 'halves' of blocks and dynamic effects of wind.

11.3 To assist in structural appraisals of large panel concrete structures, research is desirable aimed at improving inspection methods and guidance on their use (such as radar techniques for detecting the presence of voids or cracking).

12. MAIN CONCLUSIONS AND RECOMMENDATIONS

Ronan Point

12.1 Since its reconstruction following the partial collapse in 1968, the structure of Ronan Point has coped well with the loads to which it has been subjected. It has sustained substantial normal dead, imposed and wind loads without any signs of distress and it resisted the abnormal loads arising from a fire test satisfactorily.

12.2 Assessment of the stresses in the H2 joints under normal dead, imposed and wind loads taking into account the quality of their construction, suggests that the margin of safety locally at the ends of the flank walls in the lower storeys of the building is lower than that generally considered acceptable for buildings of this type. Remedial measures would be needed to enhance the margin of safety at highly-stressed locations if Ronan Point were to be brought back into use.

12.3 There are four possibilities for remedial measures to restore adequacy of the structure at lower levels in relation to normal loads. Repair or reconstruction of H2 joints in the lower eight storeys by making good the dry-pack and in-situ concrete by filling voids and replacement of poor quality material or by total replacement, would have a beneficial effect on structural adequacy. A second possibility is to strengthen the joints of the highly stressed locations by the addition of specially designed components to provide a path for vertical load through the floor slabs. A third possibility is to provide comprehensive strengthening by building an additional loadbearing system such as the structural frame tied through the building recommended by the consultants extending up to the eighth storey level. The fourth possible way of restoring structural adequacy is to remove the top eight storeys from the building thereby reducing the stresses in the H2 joints at the end of the flank walls in the lower storeys to within acceptable levels.

12.4 The strengthening of Ronan Point during reinstatement after its partial collapse in 1968 will have raised the threshold at which an explosion or other abnormal load could cause progressive collapse. The building should however not be considered for use with piped gas.

12.5 The risk that a gas explosion arising from the presence of a liquefied petroleum gas (LPG) cylinder would cause progressive collapse is considered to be remote. This risk should be made more remote by effective steps to prevent the use of LPG for space heating in the building in the future. Such steps are desirable also since the adverse effects on living conditions arising from water vapour and combustion gases produced when LPG is burnt would be removed. If the widespread use of LPG cannot be prevented by management measures or incentives, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2.

12.6 Accidental fires which might occur would seldom have intensities approaching that produced in the fire test. The probability of fires occurring simultaneously with peak intensity in two adjacent rooms is remote and is not a major consideration.

12.7 The risk of fire in Ronan Point inducing progressive collapse of part of the structure directly resulting in substantial loss of life is remote. Enhancement of the structural adequacy of the building in relation to the local overstressing of H2 joints at the ends of the flank walls in lower storeys by normal loads would provide more capability for accommodating the structural effects of accidental fires. Additional assurance concerning the performance of the structure in fire could be obtained by enhancement of the fire protection to ceilings.

12.8 The gaps which occur at junctions between cladding panels and floor slabs are unacceptable since they could allow the spread of smoke and fire gases between adjacent flats. The gaps should be closed. This could be done effectively by the installation of a joint such as that tested but with a modified end detail.

12.9 The condition of the erecting bolts and the reported deformations of bearing plates and local concrete cracking around the bolts are not considered to have a significant adverse affect on the structure.

12.10 Corrosion of reinforcement and the associated cracking and spalling of concrete cladding panels can be expected to occur with increasing frequency over the next 10 years and beyond. The application of suitable protective coatings should be considered as a means of extending the life of the components most susceptible to deterioration.

12.11 Further investigation of rain penetration would be needed if Ronan Point were to be returned to use. Where rain penetration is confirmed, action would be required to prevent it and thus inhibit long-term deterioration of the structure. Improvement of the protection of reinforcement in H2 joints against corrosion might be considered.

12.12 In the light of the likely difficulties of extensive inspection the provision of additional fixings of the non-loadbearing panels to the building and between their two concrete skins is considered a practical precaution.

Other TWA buildings

12.13 The conclusions drawn from the assessment of Ronan Point are likely to apply to some extent to all other TWA buildings and action is desirable to check the extent where that is not known already.

12.14 Most 'Type A' buildings are likely to have acceptable margins of safety in respect of normal loads in the H2 joints of the lower storeys if they are soundly constructed. The H2 joints in buildings of 14 or more storeys should be appraised. Consideration should be given to the appraisal of the H2 joints in other TWA buildings, having regard in particular to their height and plan arrangement.

12.15 Those responsible should take steps to prevent the use of LPG for space heating in 'Type A' buildings over 6 storeys in height. Where the widespread use of LPG nevertheless continues, the loadbearing system at the flank walls should be strengthened to provide for forces equivalent to a standard static pressure of 34 kN/m^2.

12.16 Consideration should be given to the appraisal of TWA buildings of 6 or less storeys in height in respect of their robustness and resistance to progressive collapse under abnormal loads, particularly where piped gas is installed or where LPG is used for space heating.

12.17 The flank wall joints in 'Type B' TWA buildings are bigger than those in Ronan Point, contain interlocking reinforcement and are less likely to have been poorly constructed. The design is likely to be adequate and the construction of good quality, particularly since the design was checked and the construction supervised by independent consultants. Confirmation of the design and construction quality should be considered only if additional assurance of adequacy is desired.

12.18 All TWA buildings should be appraised to establish whether the joints between panels can resist the spread of fire and fumes adequately.

12.19 A programme of loading tests on joints and measurements on a tall 'Type A' building during its demolition would assist in confirming the adequacy of remedial measures discussed for Ronan Point. For large panel system buildings more generally, research is needed into the incidence and characteristics of LPG explosions and fires and their effects on panel structures, and also into inspection methods.

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37. London Borough of Newham v. Taylor Woodrow-Anglian Ltd. Royal Courts of Justice, Queen's Bench Division, Judgement, 21 December 1979.

38. London Borough of Newham v. Taylor Woodrow-Anglian Ltd. Court of Appeal (Civil Division). Judgement, 9 July 1981.

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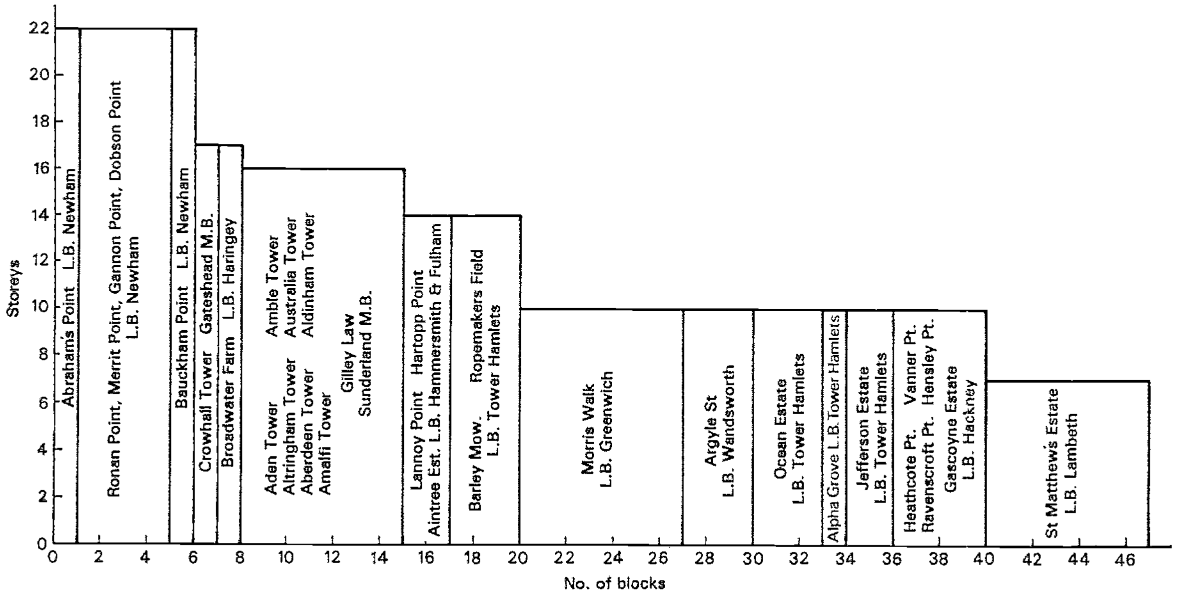


Figure 1. TWA buildings with 'Type A' H2 joint.

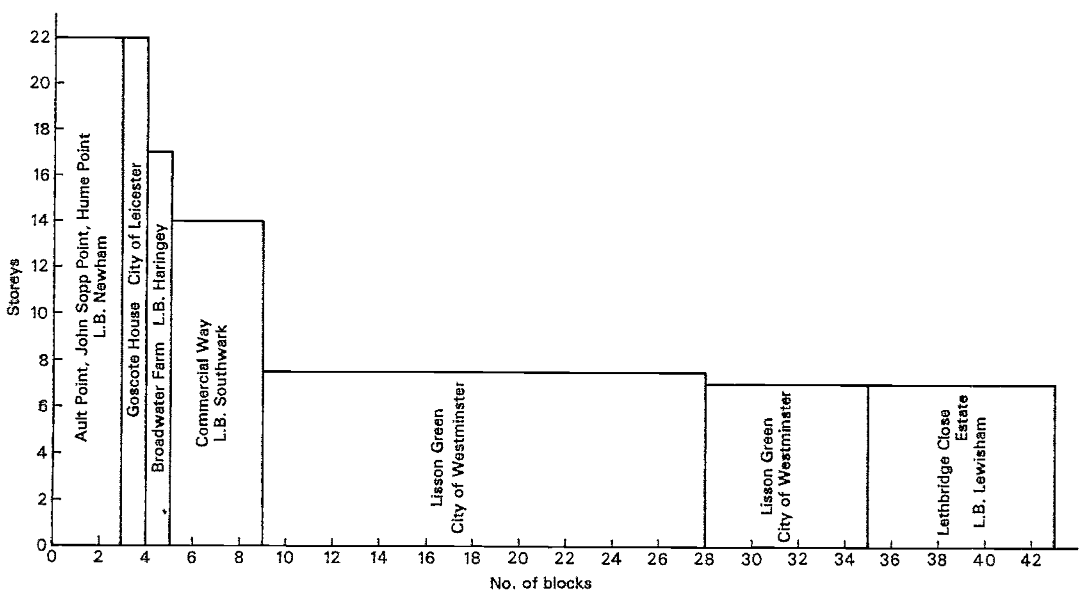


Figure 2. TWA buildings with 'Type B' H2 joint.

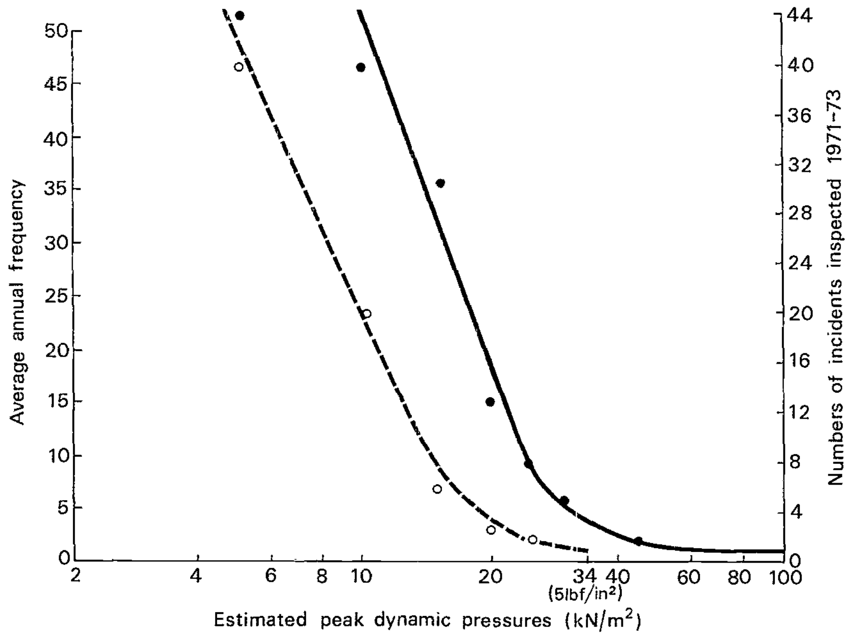


Figure 3. Frequencies with which given peak dynamic pressures may be reached or exceeded in significant accidental explosions.
(Full lines denote maximum estimates of the peak pressures; broken lines denote minimum estimates of the peak pressures)

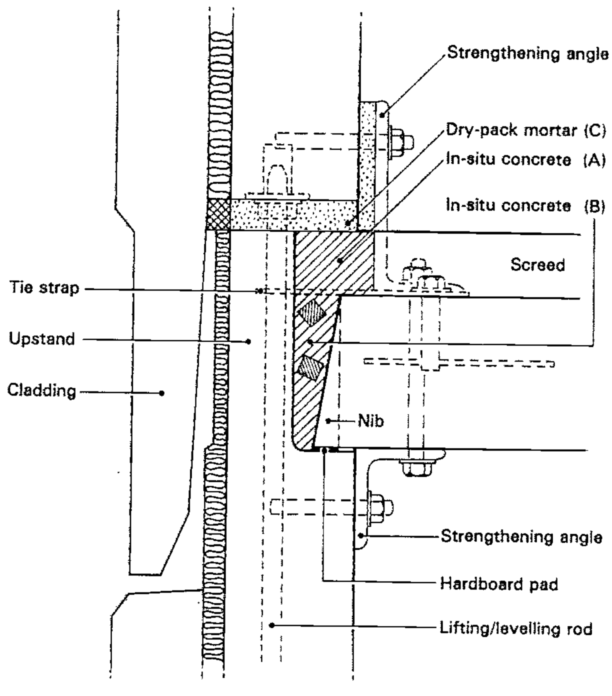


Figure 4. Simplified diagram of H2 joint.

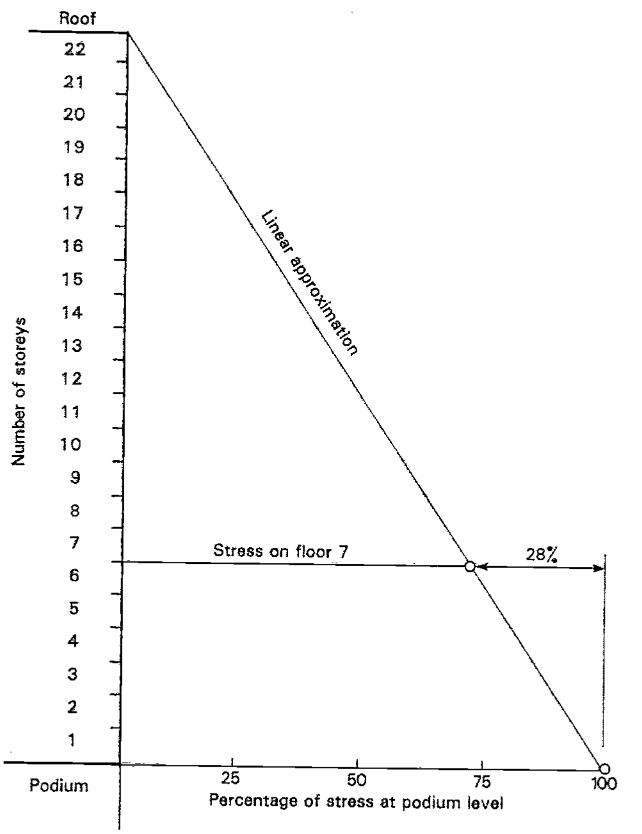


Figure 5. Variation of stress with level.

ANNEX A. PROVISIONAL BRE VIEW (OCTOBER 1984)

RONAN POINT

1. We have now examined the reports of BDP, Thomas Akroyd and Sam Webb.

2. The reports as written combine the issues of structural adequacy under normal service loads (occupancy and wind), fire, explosion and durability in a way which makes it difficult to identify the safety questions involved. This is not to say the reports or investigations are incorrect but rather that they present conclusions for which the intermediate stages of logic have not been fully explored. Our initial comments are given separately below for each issue.

Structural adequacy under normal service loads

3. Ronan Point has coped well with the normal loads occurring since it was built. There is a general concensus of opinion in the reports that for normal loading conditions, including wind, there is no need for concern and we would agree with this point of view. In fact the dynamic tests show that the overall response of the building is likely to be better than anticipated in conditions of high wind-loading.

Fire

4. The behaviour of the joints (H2 type in particular) in conditions of fire is not a matter which can be resolved by calculation or speculation. The philosophy of safeguarding against fire effects in structures rests on the use of compartmentation to restrict fire-spread within the building and protection of the structure itself from the weakening effects of fire so that it will remain stable for a sufficient period to allow evacuation and fire-fighting. The maintenance of full structural performance in fire conditions is not a usual basis of design. Thus we would question the suggested need for strengthening to cope with fire conditions and feel there is no need for immediate action as far as Ronan Point (it is evacuated already) or other similar buildings are concerned since the risk is not proven. We would suggest instead that enhancement of the fire protection should be considered (rather than strenthening) as the simplest way of removing doubts about the performance of the buildings in fire. FRS has suggested that 13 mm plasterboard placed on the ceilings in lounges (the likely locations of most rapid intense fire) and possibly bedrooms would be the simplest way of enhancing fire protection and reducing structural forces, rotations and movements caused by fire. This type of enhancement merits further investigation.

5. The compartmentation of the buildings has only been questioned at the junction between the cladding panels and floor edges. The fire stop tested at Ronan Point performed satisfactorily as far as penetration of hazardous quantities of combustion gases is concerned, and FRS have indicated verbally that a minor modification of detailing at the corners should prevent the penetration of small amounts of smoke as occurred in the test. (The FRS report is expected shortly.)

Explosions

6. The strengthening of the joints of buildings of this structural form will have the effect of raising the threshold of explosion which may cause progressive collapse. Since characteristics of a random gas explosion cannot be

quantified and specific design criteria cannot be set, we can see no advantage in raising the design specification from 2 1/2 p.s.i to 5 p.s.i. Furthermore the original tribunal report ruled that the structure was unsuitable for use with gas and it is questionable whether action should be taken now on the assumption that gas use may resume. To this end we would agree that action should be taken to ensure as far as is practical, and without causing alarm, that bottled gas is removed and not brought into all similar buildings, ie all those strengthened to the 2 1/2 p.s.i limit. Clearly the possibility of a bottled gas or other similar explosion cannot be ruled out entirely but in the event of that occurring, we feel that a progressive collapse is extremely unlikely due to its location being random.

Durability

7. Precast components

We agree with the proposals to coat the most vulnerable parts of the structure, especially the flank walls, to inhibit carbonation. There appears to be no need for further action on the general durability of the components.

8. In-situ joints

There is good evidence to support the case that the joints are not completely filled (especially the H2 joints) and we feel that this is not satisfactory from the point of view of durability and hence of the building's safety and habitability in the long term. Although the BDP report suggests that making good the defects in the joints would be difficult, we feel they should be filled to enhance protection of the joint steel against corrosion and that grouting techniques may be feasible. This action would probably also bring a bonus in providing some improvement in the load-carrying capabilities of the joints. The reports of water penetration should be investigated in more detail.

9. The overloading of the erecting bolts

It would seem likely that the erecting bolts are receiving a substantial proportion of the vertical load which may be causing local distress. This is not regarded as an immediate hazard but should be monitored in the future as part of the future care of these buildings.

10. Non-loadbearing panels

Extra fixings of the non-loadbearing panels will be desirable if the original fixings are deficient or have been omitted. Deficiencies are referred to in the restraint fixings and connections between leaves in relation to stresses induced by wind and thermal effects. We doubt that fatigue failure of connections due to thermal effects is a real possibility. In fixing these panels back to the building consideration will need to be given to the effect of limitation of the thermal movements.

ANNEX B. DERIVATION OF DEAD AND IMPOSED LOADS AT SECOND-FLOOR LEVEL BY BRE

Assumptions

B1. A number of assumptions are made about the sources of the vertical loads;

(1) average mass per unit volume of precast concrete wall panels is 153 lb/ft^3;

(2) average mass per unit area of precast concrete floor units is 87.7 lb/ft^2;

(3) floor slabs are 1-way spanning, ie half mass of slab and imposed load for half the area of the slab is carried by the flank wall;

(4) the weight of the partitions on the floor slab is carried by the cross wall and the flank wall in inverse proportions to their distances from these walls;

(5) no mass from the tank room is distributed to the flank wall;

(6) additional loads arising from strengthening post-1968 are ignored;

(7) imposed loads are as BS 6399 Part 1:1984.

Distribution of load downwards and to the flank wall of the eastern part of the block.

B2. The vertical joints between the flank wall panels are assumed to be capable of transferring load. The weight of half of the adjacent non-loadbearing cladding panels is distributed across the full width of the eastern section of the flank wall. The window openings have no effect on the load distribution. The full width of the eastern section of the flank wall is considered at the H2 joint at the intersection of the second floor and the flank wall.

Calculated loads

B3. The values calculated are:

		tons	% of total dead load
(1)	Weight of flank wall (including parapet)	283.4	56.5
(2)	Weight of floor slabs (including roof)	161.7	32.2
(3)	Weight of partitions	21.2	4.2
(4)	Weight of face wall	35.1	7.0
	Total	501.4	99.8
(5)	BS6399: Part 1: 1984 imposed load (floors and roof)	28.7	

B4. The calculated loads in the flank wall at second-floor level are:

Load combination	Load kN	Load/unit length kN/m
Dead	4995 kN	561 kN/m
Imposed	286 kN	32 kN/m
Dead + imposed	5281 kN	593 kN/m

Discussion

B5. The calculated loads are compatible with those assumed in the original design and determined by BDP (see Table 1).

ANNEX C. DERIVATION OF WIND LOADS ON RONAN POINT

C1. BDP[3] derived the values below using CP3[22] as follows:

Basic windspeed	= 38 m/s
Factor S1	= 1.00

Height above ground level		S_2
9.21 m	10.00 m	= 0.90
10.00 m	20.00 m	= 0.96
20.00 m	30.00 m	= 1.00
30.00 m	40.00 m	= 1.03
40.00 m	50.00 m	= 1.06
50.00 m	62.50 m	= 1.08
61.00 m	66.00 m	= 1.09

Factor S3		= 1.00
Length of structure	(L)	= 23.67 m
Width of structure	(W)	= 17.93 m
Height of structure	(H)	= 62.53 m (to parapet)
		= 9.21 m (to second-floor level)
Force coefficients	CF 'y'	= 1.10
	CF 'x'	= 0.95

The above S2 factors relate to Category (1), Class C, (Open country with no obstructions). This is a conservative assumption, the terrain around Ronan Point is well built-up and Category (3), Class C, (Small towns, outskirts of large cities) would be more appropriate. However the conservatism in taking the Category (3), S2 factor is offset to some extent by adopting the procedure of dividing the height of the building into parts as permitted by CP3 (clause 5.5.2), although not strictly applicable to a building of this height-to-width ratio. This procedure leads to a total wind force of 1.32 MN, assumed to act at 30.47 m above second-floor level, which imposes a wind moment of 40.99 MN - see Table 1.

C2. Assuming the terrain to be Category (3) and using the S2 value for the top of the structure as CP3 recommends (clause 5.5.2) the following calculation is made:

Basic wind speed for design = 38 m/s

Maximum basic wind speed experienced to date = 32.6 m/s (based on measured speed of 71 knots in 1978 at an effective height of 38 m at the London Weather Centre).

Factor S1 = 1.00

Factor S2 = 1.03 (Category (3), class C, @ 64 m)

Factor S3 = 1.00 (50-year return period wind)

= 1.17 (500-year return period wind)

V 's' (50) = 39.1 m/s

V 's' (500) = 45.8 m/s

V 's' (Max) = 33.6 m/s

Length of structure	(L)	= 23.7 m
Width of structure	(W)	= 17.9 m
Height of structure	(H)	= 64 m

L/W = 1.32

b/d = 1.32: h/b = 2.7 : CF 'y' = 1.10

b/d = 0.76: h/b = 3.57: CF 'x' = 0.95

A 'y' = 64 x 23.7 = 1516.8 m '2'

A 'x' = 64 x 17.9 = 1145.6 m '2'

The calculated base shears (MN) arise from an assumed uniform loading over the height of the building, the centre of loading acting at a height of 0.5 H.

Worst-case cladding pressures would be assumed in design to act over a width of 0.25 x 17.9 = 4.5 m from each corner of the building.

C3. Conclusions

The calculated base shears (MN) and worst-case cladding pressures derived from paragraph C2 are:

	Base shear'y' MN	Base shear'x' MN	Shear'y' at 9.2 m MN	Cladding pressure kN/m^2
50-year return period wind	1.56	1.02	1.34	-1.12
500-year return period wind	2.15	1.40	1.84	-1.54
Maximum wind to date	1.16	0.75	0.99	-0.83

	Base moment'y' MN.m	Base moment'x' MN.m	Moment'y at 9.2 m' MN.m
50-year return period wind	49.9	32.6	36.7
500-year return period wind	68.8	44.8	50.4
Maximum wind to date	37.1	24.0	27.1

(Direction Y is E-W, X is N-S).

P

B

l

o

ANNEX D. ASSESSMENT OF WIND LOADS ON RONAN POINT, TAKING ACCOUNT OF DYNAMIC BEHAVIOUR

D1. Classification

Structural categorisation of Ronan Point by the Harris classification procedure[23] is used here to determine an optimum method for assessment of wind loading at the onset of structural distress, using both estimated and measured structural parameters for the building.

Entries in the 'Estimated' column below indicate values derived from best current knowledge. Entries in the 'Measured' column indicate values measured during the vibration tests on Ronan Point[6] and values derived therefrom. When there is no entry in the 'Measured' column, the 'Estimated' value is used in subsequent calculations.

<u>Data</u>

		Estimated	Measured
Lowest natural frequency	n1 =	0.76Hz	0.831Hz
Size parameter = $(H^2+W^2)^{1/2}$	1 =	68m	68m
Maximum acceptable deflection	x =	20mm	20mm
Damping ratio at x	zeta =	0.075	0.058
Basic mean wind speed	V_B =	21m/s	
Design mean wind speed (for $\theta=240^o$)	V_H =	26.4m/s/s	

<u>Calculation</u>

Structural reduced frequency	n $1/V_H$ =	1.96	2.14
Parameter F	F =	0.48	0.47
Longitudinal turbulence length	L =	116m	
Meteorological reduced frequency	n L/V_H	3.34	3.65
Parameter f	f =	0.467	0.591
Product fF	fF =	0.224	0.278

<u>Output</u>

Structural Class	Class =	A2	A2
Gust Factor	S_G =	1.69	1.69
Dynamic Amplification Factor	$gamma_D$ =	1.07	1.08

The classification of Ronan Point as Class A2, 'small static', using both estimated and measured structural parameters indicates that the dynamic amplification of the resonant component is approximately balanced by the decrease in load correlation over the size of the building, so that the wind loads on Ronan Point may be assessed as if the building were static and small. The degree to which the 'resonant effect' and the 'size effect' balance is expressed by the Dynamic Amplification Factor, gamma D. The 'Estimated' values above will be conservative.

D2. Choice of method

It is valid to assess the wind loading of Ronan Point by static methods. The use of Class C wind loads, as recommended by CP3[22] and given in Annex C does not take the dynamic amplification into account. The Harris structural classification indicates that Class A loads should be used, but increased by a factor of 1.07 (see D1). This approach has been used for the best estimates of wind loads in D3 following.

D3. Best estimates of wind loads

RONAN POINT	Best estimates of wind loads

Structural parameters :

Height	62.53 m
Width	23.67 m
Depth	17.93 m
BRE-Harris Structural Class	A2
Dynamic amplification factor	1.07

50-YEAR RETURN ESTIMATES

	Axis XX	Axis YY	Direction of highest windspeed	
Meteorological parameters :				
Wind direction	270^{o}	180^{o}	240^{o}	True
Class A (1-s) Design Gust	43.3	38.6	44	m/s
Class A design dynamic pressure	1.148	0.913	1.186	kN/m^2
Class C (16-s) Design Gust	37.4	34.0	38.1	m/s
Class C design dynamic pressure	0.857	0.708	0.889	kN/m^2
Loading coefficients :				
Overall force coefficient	1.087	1.087	1.043	
Worst local pressure coefficient	-1.30	-1.30	-1.00	
Structural dimensions :				
Face area	1480	1121	1480	m^2
Base moment of area	46275	35053	46275	m^3
Overall loads :				
Base shear : Class C	1.38	0.86	1.37	MN
Base shear : Class A	1.85	1.11	1.83	MN
Base shear : Class A2	1.98	1.19	1.96	MN
Dynamic increment = A2-C	0.60	0.33	0.59	MN
Dynamic increment / Class C	43%	38%	43%	
Base moment : Class C	43.1	27.0	42.9	MN.m
Base moment : Class A	57.8	34.8	57.2	MN.m
Base moment : Class A2	61.8	37.2	61.2	MN.m
Dynamic increment = A2-C	18.7	10.2	18.3	MN
Moment at 9.2 m : Class C	31.6	19.8	31.5	MN.m
Moment at 9.2 m : Class A	42.4	25.5	41.9	MN.m
Moment at 9.2 m : Class A2	45.3	27.3	44.9	MN.m
Local cladding pressures :	-1.493	-1.186	-1.186	kN/m^2

D4. Conclusions

These values represent the best estimates incorporating current knowledge without resorting to new experimental testing. The Class C values are the results that are obtained through the quasi-static approach of CP3[22] but using the best estimates for the parameters and not the Code values; that is to say, including the effects of atmospheric turbulence, but excluding the resonant response of the structure. The Class A and A2 results are the best estimates obtained through a quasi-static approach[23] uncorrected and corrected for dynamic resonance respectively, but without a full dynamic analysis. The difference between Class A2 and Class C represents an estimate of the resonant component.

The estimates relating to Class C are less than those derived from CP3 in Annex C, confirming that CP3 gives conservative results for large static structures. However, the degree of dynamic response more than offsets this conservatism, with the result that the best Class A2 estimates are 26% greater than the estimates from CP3 in Annex C. The partial safety factor for wind loads would be taken as 1.4 and is sufficient to absorb the resonant component. Under the current Code, a structure of this size would be designed as Class C as in Annex C and the resonant component absorbed into the partial safety factor on wind loads.

ANNEX E. ESTIMATION OF WIND LOAD CARRIED BY NE FLANK WALL AT SECOND FLOOR

Inertia calculations

E1. Consider the East and West sections of the building to act independently and to share the loads in proportion to their inertia. Consider just the cross walls which are assumed to be continuous, the inner walls being 7-inch and the outer walls 6-inch. This simplifies the cross-section to that shown in Figure E1 and corresponds to a total inertia of 114.5 m^4. In this scheme the inertia of the NE flank wall is 8 m^4, so the load carried by the flank wall is 7% of the full load.

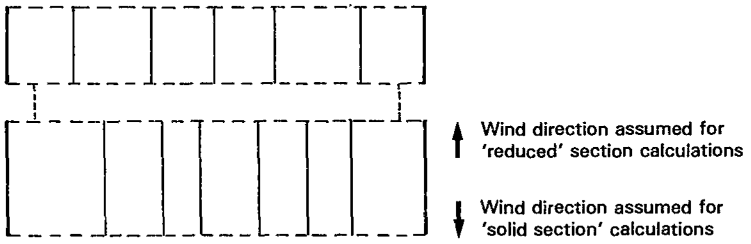


Figure E1. Simplified planform for inertia calculation.

E2. If the corridor shear walls are included in the above model as in Figure E2, the inertia is increased to 193.8 m^4. Due to a change of neutral axis the inertia of the flank wall is increased to 9.13 so the flank wall carries 4.7% of the wind load.

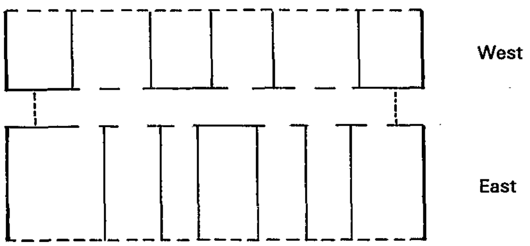


Figure E2. Modified planform for inertia calculation including corridor shear walls.

E3. From the dynamic tests on Ronan Point the fundamental natural frequency in the EW direction is 0.84 Hz. If a simple cantilever-type model is assumed for the building the inertia can be calculated if the mass, frequency and elastic moduli are known. The mass was measured in the tests and if an E value of 4×10^{11} N/m^2 is assumed (which seems reasonable from the measured characteristics of the concrete) then the inertia required is $154.4m^4$.

E4. Therefore a conservative assumption is that the NE flank wall takes 6% of the total wind moment. A value of 9% as used by BDP in their assessment is clearly on the 'safe' side. It is based on the same conservative assumption used for the original design by Phillips Consultants Limited which ignores any contribution to the stiffness from the corridor walls or party walls.

E5. BDP estimate the total wind moment on the basis of a 50-year return period wind to be 41 MN.m (Annex C) of which 9% is assumed to be taken by the NE flank wall, ie 3.7 MN.m (Table 1).

E6. From Annexes C and D the estimated wind loads are:

(a) 1.56 MN applied to the east face of the building - calculated on the basis of CP 3[22] and a 50-year return period wind.

(b) 1.98 MN applied to the east face of the building - calculated on the basis of the more precise method which includes dynamic effects.

E7. The proportion of the wind load carried above the second storey (9.2 m) level is 56.8/66.

The total moment resisted at second-storey level is therefore equal to wind load x $\frac{56.8}{66} \times \frac{56.8}{2}$

which gives 38.2 MN.m using the wind load of 1.56 MN, and
48.3 MN.m using the wind load of 1.98 MN.

(These estimates are on the conservative side due to the assumed building height (66 m to the top of the tank room) being larger than the height to the parapet (62.53 m)).

E8. The moment carried by the NE flank wall is determined by the relative inertia of the flank wall to the whole structure. For the calculation of stresses given in Table 2, Section 5, the estimate of 6% of the total wind moment was assumed to be carried by the flank wall (see Table 1).

ANNEX F. ASSESSMENT OF 'AS BUILT' STRENGTH OF H2 JOINT

F1. The importance of the H2 joint merits consideration in some depth so as to explore the limits of uncertainty and to achieve the best assurance of safety without undue conservatism. In the final result it will be seen that, whatever the numbers say, the judgement needed to interpret them is an important part of the process.

Methods of assessing allowable stresses in concrete

F2. The term specified strength relates to failure of a test cube and is used as a basis for design and specification of concrete elements. There is a number of ways in which a representative value for the allowable strength of concrete may be deduced from the cube strength, for design of a concrete component in a panel structure:

Method 1: CP111:1964

CP111:1964 - Structural recommendations for loadbearing walls - applies to plain (ie unreinforced) concrete. It recommends in Table 8 a maximum permissible stress of cube strength - 4 or 10.5 N/mm^2 whichever is the lesser. This stress may be increased by 20% (clause 330(a)) when the ratio of height to length of a wall is less than 1:2. Increases of up to 25% are permitted when eccentric loads or lateral forces are acting (clause 331) as long as the extra stress is caused by those effects. These two enhancements increase the maximum permissible stress to 15.1 N/mm^2 in certain cases. Local stresses caused by concentrated loads may be 50% greater (clause 332) if the full increases for aspect ratio and eccentric/lateral loads have not been invoked. There is no allowance for age at loading.

Method 2: CP114:1957

CP114:1957 - The structural use of normal reinforced concrete in buildings - recommends for designed mixes in Clause 303 (Table 8) permissible concrete compressive stresses of $0.253u_w$ for direct stress and $0.333u_w$ for bending stress where u_w is the 28-day works cube strength.

Walls are treated in a similar fashion to columns. By Clause 332 (a) the load carried by the concrete in an axially loaded column (or wall) is based on $0.253u_w$.

By Clause 322(c) the load carried by an eccentrically loaded column is based on $0.333u_w$. However, there is a proviso that, in this case, the load should not exceed the axial capacity of the member, ie two criteria must be satisfied in similar fashion to CP111.

As in CP111, stress increases of up to 20% apply in the case of long members or long lengths between openings.

Method 3: CP116:1965

CP116:1965 - The structural use of precast concrete - adopts an approach which differs from that of CP114 only in regard to the level of the permissible stresses. Thus the recommended values for permissible compressive stress, given in Table 3 of Clause 303 are $0.274u_w$ for direct stresses with a maximum of 2200 p.s.i, and $0.366u_w$ for bending stresses.

Axially and eccentrically loaded columns are treated in exactly the same fashion as in CP114 ie two criteria must be satisfied.

Table 19 of clause 324 adopts the same allowance for long walls and long lengths between openings as CP111 and CP114 but states that 'where a wall comprises a series of units without adequate joints the length shall be taken as that of the unit'.

Modifications may be made for age at loading (clause 303a and Table 7) up to a 24% increase and for the effect of wind forces (clause 307) a 25% increase.

Stress increases of up to 20% (Table 19) may be allowed, depending on aspect ratio, as in Method 1 above. A concrete wall is considered to be reinforced if the vertical and lateral reinforcement percentages exceed 0.2% (clause 324).

Method 4: CP110:1972

CP110 - The structural use of concrete - is a unified code applying to plain and reinforced concrete, both in-situ and precast. In particular clause 3.8 treats reinforced walls and clause 5.5 plain concrete walls. A direct comparison with the recommendations in CP111, 114 and 116 is difficult because CP110 gives methods for calculating capacity of wall directly, accounting for the same factors such as slenderness, wind load and age but not in a directly comparable way.

Method 5: In-situ testing of concrete

The strength and quality of existing concrete in a structure may be assessed by non-destructive testing methods or by sampling followed by laboratory testing. Non-destructive methods are given in BS4408 but there is also a substantial bibliography on other methods. The techniques used by BDP[3] have included measurements of ultrasonic pulse velocity (BS4408 Part 5), radar survey by Cambridge House Geotechnical Services and visual inspection, sometimes involving local removal of concrete. The methods did not provide estimates of strength; the asessment of quality is given in Annex L. Sampling methods have involved the taking and testing of cores in accordance with BS1881, subject to the guidance of BS6089:1981.

Method 6: Experimental tests

There are no standard procedures for full-scale tests on structural components or assemblages, although there are well-founded general procedures. A number of test programmes have been conducted specifically in relation to Ronan Point following the partial collapse in 1968[21,25].

There have also been tests on the redesigned H2 joint, referred to as 'Type B'[32] and similar experimental variations[33].

Details of joint

F3. The H2 joint is shown in Fig 4 in simplified form to indicate the essential concrete elements of floor and wall panels in-situ concrete (A and B) and dry-pack (C), the designations A, B and C as in the reports of radar investigations. The strength of each of these parts of concrete may be considered in turn using the most appropriate method of assessing the permissible stresses summarised in section F2.

Floor panels

F4. The specified cube strength of concrete (Phillips Consultants Limited) was 5700 p.s.i. (39 N/mm^2). The Tribunal (1, paras 108-9) expressed satisfaction with the quality of the units and the method of their production, although actual measurements of strength were not quoted. However the tests carried out by Imperial College[21] are understood to have used floor and wall components cast in the same manner and in the same factory as the actual units used for construction. This concrete had a 28 day cube strength of 62 N/mm^2: it is considered more appropriate to use this value.

Application of method 3 (see F2) suggests a permissible stress of the maximum allowed value of 15.2 N/mm^2 (not the calculated value of 17.0 N/mm^2) in direct compression. At the relative magnitude of the wind-induced stresses at Ronan Point (Table 2) an allowable enhancement of 25% leads to 19.0 N/mm^2. Further enhancement of 24% for increase in strength with age gives 23.6 N/mm^2.

Wall panels

F5. These were subject to the same concrete specification as the floor panels above. The corresponding strength in the tests[21] was 58.6 N/mm^2. The permissible stress using method 3 again is the maximum value of 15.2 N/mm^2 (not the calculated 16.1 N/mm^2). However, the only reinforcement is provided by the perimeter steel positioned to afford added strength during handling and it may be more appropriate to limit the permissible stress to 10.5 N/mm^2 as quoted in Method 1. Similar enhancements for wind effects leads to 13.1 N/mm^2. The age allowance of 24% in methods 2 and 3 is not quoted in method 1. However BDP, using method 2, do not consider it appropriate to compound the wind and age allowances. It does not seem unreasonable to do so as recent test results (see below) confirm that the concrete strength is substantially greater than the minimum specified; its application leads to 16.3 N/mm^2.

A further enhancement of 20% for a long wall is only justified by methods 1,2 and 3 if the vertical panel joints V13 can be considered to make the wall monolithic. The assumptions about stiffness in Annex E rely on continuity in the flank wall, as discussed in 5.40.

Three cores taken by BDP were tested by BRE in the laboratory. The mean estimated in-situ cube strength was about 60 N/mm^2 giving a permissible stress of 16.4 N/mm^2 in direct compression. This value takes account automatically of any change in strength with age, although other enhancements may be appropriate.

Upstand

F6. The upstand is cast integrally with the wall panel. Although the latter is essentially unreinforced the upstand is said to contain reinforcement near its edges. The shanks of the vertical levelling bolts may act as reinforcement although the cover to their inner surfaces is small. It may therefore be more justifiable to use method 3 and assume the maximum permissible stress of 15.2 N/mm^2 (calculated 16.4 N/mm^2), which with the wind enhancement becomes 19.0 N/mm^2, or age enhancement 23.6 N/mm^2 (cf 16.3 N/mm^2 for method 1, unreinforced concrete).

In-situ

F7. The in-situ concrete is shown in Figure 4 as two separate parts A & B for convenience of identification in the radar investigation of concrete quality; in fact the concrete was cast as one mass. Generally the floor screed, separated from the in-situ concrete by a polystyrene spacer, is an integral part of the precast floor unit although the strengthening measures after 1968 included removal of the screed on the floor panels adjacent to the flank walls and replacement by a reinforced structural screed. The screed is not part of the H2 joint.

It is not known whether the in-situ concrete was also specified as 5700 p.s.i. (39.3 N/mm^2) but in tests[21] an 8-day strength of 40 N/mm^2 was used. It is probable that a more realistic value for the mass of concrete would be 30 N/mm^2, allowing for the difficulties of site placing, compaction and curing, but ignoring any significant voids into which concrete was not placed, eg at the bottom of zone B.

While method 1 would suggest a permissible stress of 7.5 N/mm^2, some horizontal reinforcement is present and the concrete is closely confined between the stiffer precast concrete upstand and floor panels.

Dry-pack

F8. The dry-pack is a mortar with a nominal specification of equal parts of cement and sand, with just sufficient water to enable it to be packed into the gaps to be filled. The specified strength of the dry-pack was 5700 p.s.i.(39.3 N/mm^2) and test cubes made on site during construction confirmed that the minimum was obtained[1]. Further tests[1] showed, nevertheless, the potential for variability which might be exacerbated by the difficulties in practice of placing the dry-pack consistently as specified. Further information on the quality is given in Annex L.

In laboratory tests on H2 joints[21] a 4 day strength of 29 N/mm^2 was used. The thickness of dry-pack was required to be only 1 1/2 inch (38 mm) and in this confined state its effective strength will be very much greater than indicated by cube strengths. It is difficult to evaluate a reliable value but it is sufficient to accept that a significant reduction in the quality and/or amount of dry-pack could be tolerated without encroaching on necessary margins of safety.

However, poor quality or position of dry-pack may have adverse side-effects by inducing stress concentrations in the underlying in-situ and precast concrete of an H2 joint.

Load paths through joint

F9. If all the concrete elements in the joint were of the same strength and stiffness, the same stresses would occur in the joint as in the adjacent walls above and below and give no concern. In practice it is necessary to allow for a poorer quality of in-situ concrete, A, particularly in the lower part of the pocket. This produces an effectively narrower support area through the joint, putting higher stresses on the precast upstand or towards the end of the floor panel. In addition, the load from above may be applied in an eccentric or concentrated form as a result of reduced coverage of dry-pack.

Stresses in H2 joint - Method A

F10. The procedure used by BDP[26] makes a number of assumptions and identifies several potentially unfavourable influences which they consider should be taken into account. They consider:

1). the overhead load on the H2 joint is applied in the centre of the wall,

2). the floor does not transmit any load except through the support nibs which rest on hardboard pads,

3). the effect of dissimilar materials in the joint (precast and in-situ concrete) for the purpose of assessing effective width,

4). incorrect positioning of the floor on its seating,

5). tolerances on vertical alignment of walls,

6). the possibility of poor quality or missing in-situ concrete.

F11. BDP estimate that the following stresses may be produced at the West End of the second storey NE flank wall by loading D + L + W (Table 2):

1). Under 'as designed' conditions, average stress in 150 mm width of wall - *10.1 N/mm^2.

2). Stress in H2 joint allowing for a reduction in width of section, dissimilar materials and eccentricities - *21.1 N/mm^2.

*These are averaged values and do not represent the worst conditions considered by BDP.

Stresses in H2 joint - Method B (modification by BRE)

F12. It is considered that more realistic assumptions are justified for the appraisal of an H2 joint than those in F10 for Method A. The following aspects may be quantified to some extent and lead to a less pessimistic assessment of the likely stresses:

1) eccentricity of vertical load above joint,

2) likely neutral axis of effective section,

3) lateral restraint provided by floor,

4) alternative load paths,

5) effect of corridor wall.

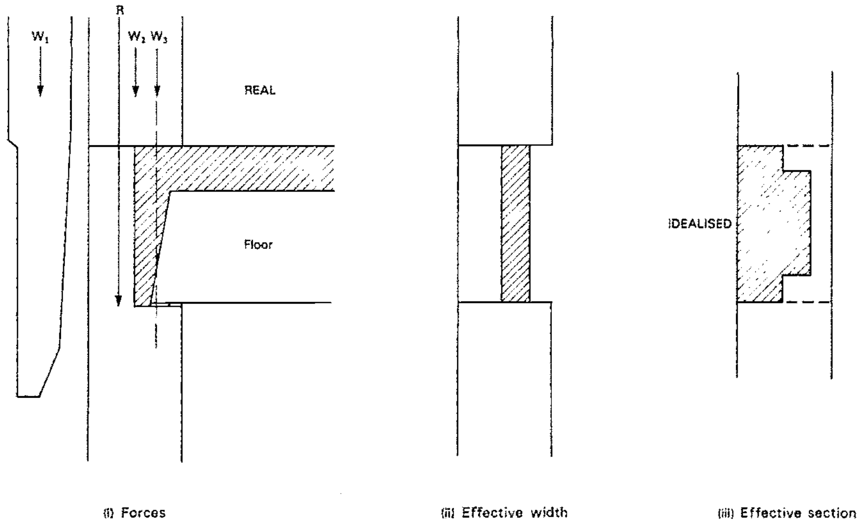


Figure F1. Idealisation of H2 joint.

Using the idealisations shown in Figure F1 the following stresses are estimated at the West End of the second storey under loading D + L + W2 (Table 2):

(1) Under 'as designed' conditions, average stress in 150 mm width of wall - 8.23 N/mm^2

(2) Stress in H2 joint allowing for a reduction in width of section, dissimilar materials and eccentricities - 13/4 N/mm^2

(3) Stress in H2 joint allowing for dissimilar materials, floor displacement of 20 mm and wall misalignment above joint level of 5 mm - 19.5 N/mm^2

The calculations have used the method of modular ratios to represent the dissimilar materials in the joint as a homogeneous width of precast concrete. The notional stresses calculated in (2) and (3) occur on the inner face of the joint and hence represent stresses of 13.4/1.5 (8.9 N/mm^2) and 19.5/1.5 (13.0 N/mm^2) occurring in the in-situ concrete (using 1.5 as the modular ratio).

Case (3) calculates stresses when all floors are displaced 20 mm towards the outer face and all walls above the joint on the second floor are displaced 5 mm inwards. This condition is unrealistic and does not occur in any construction where misalignment of walls and displacement of floors is random through the height of the building. It does show however that some misalignment would lead to stresses greater than case (2).

F13. Alternative load paths have mitigating effects on the induced stresses determined above. The floor is actually an integral part of the joint (point 3, F12) and is fixed partially by the in-situ concrete. Some of the in-situ concrete is likely to have filled the gaps under the unsupported nibs and under the section of the floor over the wall panel, hence improving the load carrying capacity of the joint.

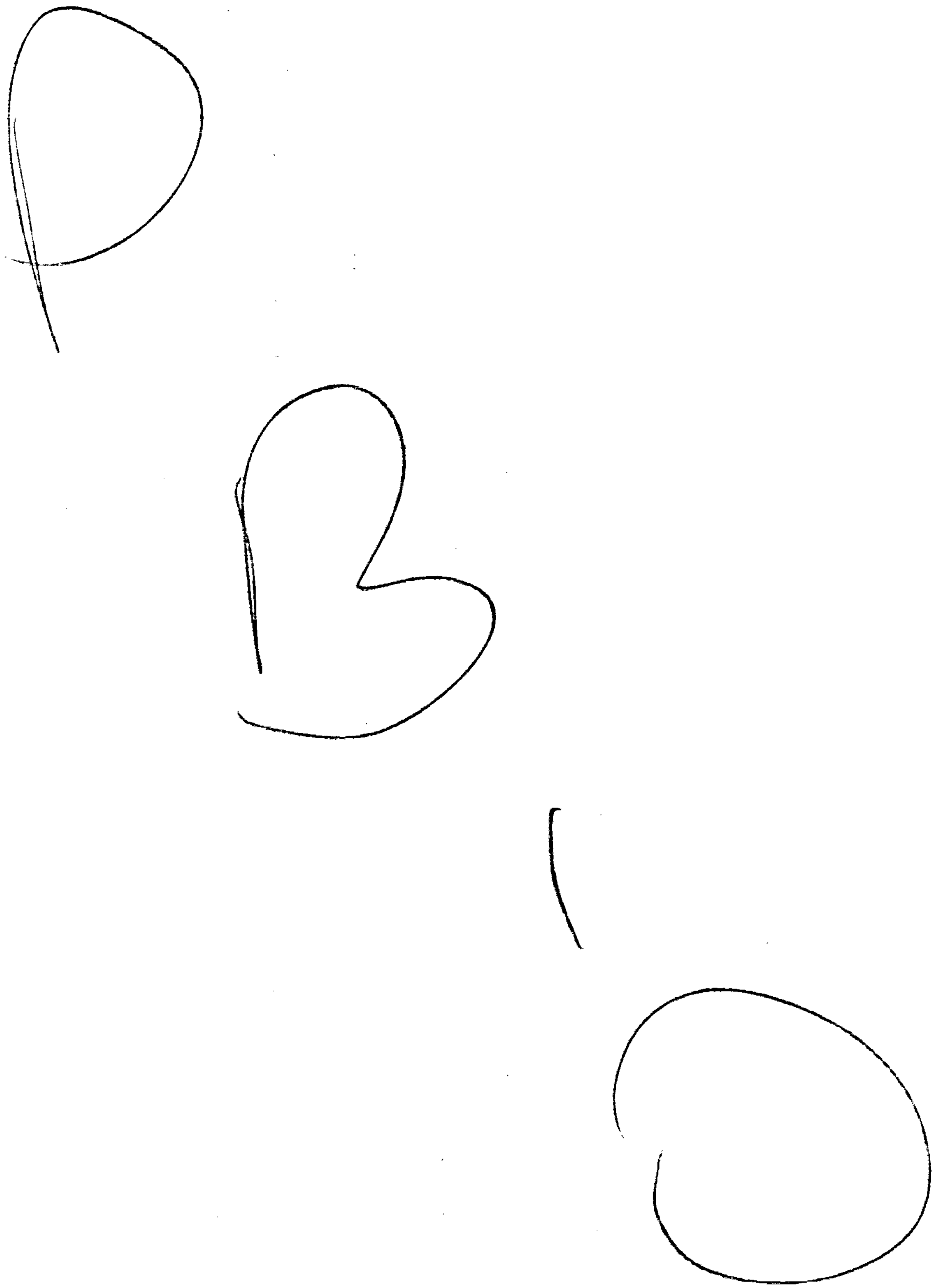
The high stress in the joint is produced principally by bending moments generated in the joint caused by misalignment of the line of action of the applied load and the neutral axis of the section. However, the floor is a rigid part of the joint and must be capable of carrying some of the moment, so reducing the stresses produced by the eccentric moment.

F14. Another path for part of the load (point 4, F12) may exist through the inner dry-pack and in-situ concrete onto the floor panels, thence through the nibs to the wall. The effect of floor misalignment on this load path should be of limited significance: outwards error would increase the in-situ concrete area over the floor, inwards error would give a wider zone of in-situ concrete and the posibility of better quality concrete.

The area of overlap of wall and end of floor is 10-15% of the total width. The resulting load through the composite upstand/in-situ concrete could be reduced by this amount, which could be important. The effect would be greater in areas where the dry pack is absent at the outer edge.

Whilst the strengthening angles are present primarily to give resistance to damage by abnormal loads, they provide some, albeit small benefit to the joints' ability to carry normal vertical loads.

F15. The maximum stress is at the West end of the NE (and SE) flank wall. At this position the flank wall joins a 7-inch corridor wall (point 5, F12) which is assumed to carry no load. In fact this corridor wall must carry load, if not through the V13 joint between walls, then through the floors at the H14 joint. For compatibility of deflections the stresses in the flank wall must be considerably reduced at its extremity, reducing linearly away from the end of the wall.



ANNEX G. NON-LOADBEARING CLADDING

General details

G1. The non-loadbearing cladding panels are positioned on the east and west elevations, supported and restrained in position at the flank and cross-wall locations only. The details of these panels were not considered by the Tribunal[1] but the description here is drawn from BDP[3].

G2. The available original drawings show the units to be of reinforced concrete construction, in section comprising a 4-inch thick inner skin, a 1-inch thick layer of polystyrene insulation and an outer skin 2 1/2 inches thick faced with mosaic. Each panel contains a large window opening with the sandwich form of construction extending from the underside of this opening to the bottom of the unit. The outer skin extends downwards to overlap the single-skin lintel section (called transom by BDP) above the window of the cladding panel of the storey below.

G3. The manufacturing details for the panels indicate that the inner skin was reinforced with a mesh positioned 1 inch from the inside of the face of the unit. The second sheet of mesh, at the same cover, was placed in the opposite face of the lintel section, but the original drawings contain no details of either of these meshes. The inner skin reinforcement was completed by the provision of two looped high-tensile bars in the front face of the mullion.

G4. The outer skin of the panels was also reinforced with a mesh, placed centrally in the concrete thickness, and the two skins were connected together with stainless steel bar reinforcement and wire ties. Full details of dimensions, profiles and the fixings are shown in Figure G1 (taken from reference 3).

Panel fixings

G5. Information available on the original construction shows that the vertical support of the panels was provided by steel-to-steel seating at the cross-wall and flank wall positions. The level of these seatings is approximately that of the underside of the window openings. Restraint fixings were located within the floor construction zone at the top of the panels again at the flank and cross-wall positions. There are slight variations to these latter fixings depending on the relationship between the cladding panel and the supporting wall, but basically all comprise stainless-steel bolts with anchor plates into mild-steel inserts, either placed within the in-situ make-up area of the floor construction or cast in the flank wall corner units. These are the only connections between the cladding panels and the structure. The original details allowed for a straight joint between the units and the floor edge, but adjacent panels were linked together vertically by projecting bars from the lower panel locating in the steel inserts in the upper unit. The horizontal joint between the inner skins of panels is at the finished floor level. It is shown as fibreglass packing sealed on the inside face by 3/4 inch thickness of dry-packed mortar.

G6. A different fixing detail occurs at the end of cladding panels adjacent to the flank walls (see section 4 of Figure G1). Firstly it is not clear how the junction could be executed on site unless the bolt and plate are cast into the panel in the factory. In that case the cladding panel would have to be slotted in on site, not generally leading to a tight restraint. More information is required about the nature and condition of this joint, variously designated V2 and V21. It should be noted too that this connection can afford no lateral restraint to the flank wall panel.

As built conditions

G7. Exposure of the fixings and seatings at a number of locations has shown them to be in accordance with the original details[3]. The general quality of the concrete and the condition and position of the reinforcement have been found also to be in agreement. The condition of the panels was generally good apart from areas of missing or damaged mosaic tiles.

G8. The H1 joint between the cladding panel and floor slab was designed to be packed with fibreglass and pointed with mortar. Some joints have been packed effectively solid with mortar; the possibility that this could inhibit natural movements of the cross-walls, floor and panel and induce stresses not considered in design should be assessed.

G9. Webb[5] considers this effect to be very serious on the basis of an incorrect analogy with pinching of brickwork panels between concrete floors. The fixings of the panels and their reinforcement would prevent a pinching failure. BDP have monitored some H1 joints and detect no significant vertical load transfer[3]. Thomas Akroyd[4] address the issue in some detail. They consider that, in the absence of signs of distress and with adequate panel fixings, no adverse effect upon the integrity of the structure arises. Overall, the available information suggests that solid mortar packing of H1 joints does not cause a significant structural problem. However the adequacy of the fixings of the non-loadbearing cladding panels needs careful assessment.

Bowing of panels

G10. Differential drying shrinkage can result in curvature of concrete sandwich panels. This occurrence is widely recognised and it is likely that curvature of the panels at Ronan Point occurred during the first year or two after manufacture. In addition to this permanent deformation which may be up to about 10 mm, changes in ambient conditions lead to cyclic movements, such as measured by BRE[19]. This bowing and movement is not likely to be new and any such cyclic movement is of no significance to the integrity of the panel as long as its fixings are sound.

G11. Although attention has focused on the gap formed at Ronan Point between the floor slab and a deformed panel, a gap may also be formed between a bowed panel which abuts an intermediate cross-wall, in other TWA buildings with different internal layouts. It is not clear that this gap would constitute a fire hazard in the context of the requirements for fire containment, but this storey-height vertical gap certainly constitutes an acoustic path between adjacent rooms, which may be bedrooms.

Resistance to wind loads

G12. The Tribunal recommended (1, para 145(a)): "The face panels should be better secured, aiming to make them withstand safely a suction of 65 lb/ft^2". It is not clear that any such action was taken but the recommended value of suction (equivalent to 3.25 kN/m^2) is certainly very much higher than the maximum local suctions indicated in Annex C (-1.12 kN/m^2 for CP3 50-year) or Annex D (-1.49 kN/m^2 for class A, accounting for dynamic effects).

G13. BDP[3] base their assessment on a local value of 2.1 kN/m^2 but the derivation of this higher value is uncertain. Even when considering a 500-year return period wind, the resulting value of 2.9 kN/m^2 is still less than the Tribunal value. Thomas Akroyd[4] quotes a local value of 2.07 kN/m^2 after increasing the 'probability factor' from 1 to 1.175: again, the derivation is uncertain. Thomas Akroyd notes[4] that "the fault in the original design was not in the use of an out-of-date Code, but in the design of the cladding panels close to the corners of the buildings, where they should have been designed for higher wind forces". After considering the strength of the fixings Thomas Akroyd conclude satisfactorily that "the factor of safety against failure of the panel due to wind suction is thus 2".

G14. Although BDP conclude[3] that "under certain conditions of loading some components of the restraint fixings together with the transom section (of the larger lengths) of the panel unit appear to be overstressed", it seems reasonable that use of a design suction of 1.49 kN/m^2 (Annex D) could lead to a satisfactory appraisal.

G15. Despite the relatively satisfactory report on the actual condition of the fixings BDP[3] state:

"Our assessment of the original design and detailing of the cladding panel supports and restraints is that the very limited number of fixings provided makes them particularly vulnerable to adverse variations from the specifications or poor workmanship such as badly-made welds, loose bolting etc.

No amount of sampling would eliminate the possibility of an incorrect component or poorly-made fixing. None of the present panel fixings can be seen without extensive cutting of the structure or the panel itself and only a limited amount of cutting and exposure of fittings is practicable.

As a consequence we are of the opinion that additional fixings should be provided, linking the entire panel thickness back to the slab edge and the loadbearing cross-walls. The introducton of this measure would obviate the need for further extensive surveys of the panel connections and would reduce, where necessary, the stresses of both fixings and panel to within normal limits."

G16. Given the undoubted difficulty of comprehensive inspection, this approach represents a practical precaution for the long-term, which has the benefit of removing any further concerns, provided the remedial measures adopted in detail use appropriate techniques and materials (see below).

G17. BDP's recommendation for additional fixings also extends to the connection between the two concrete skins of panels. They consider that high bending stresses may be induced in the stainless-steel connections by frequent ambient temperature variations, and that further tests are necessary to check whether fatigue may be a problem. A sandwich panel with an insulating core, whether integral with the adjacent leaves or as a spacer together with metal connections (ties) between the leaves, presents a complex problem for rigorous stress analysis, particularly when time-dependent thermal response has to be considered. In some large panel systems it is understood that existing designs can be justified, or indeed were designed, only by full-scale test.

G18. In the additional context of fatigue, the likely stress distribution in the ties is more complex than and different from that for which fatigue testing is usually performed. However, over a period longer than the life of Ronan Point, there is no field evidence that appropriate grades of stainless steel have been subject to failure by fatigue. At the stress levels likely to be reached it is considered that fatigue is unlikely to be a problem in the life of such a building.

G19. Again, in view of the difficulties of inspection and guaranteeing the presence of an adequate number of ties of the right quality and location, additional tying may be considered a practical precaution.

G20. The design of remedial measures should ensure, in addition to choice of a suitable strength and quality of material, that the installation can be carried out reliably and that no adverse effects are created by introducing a fixing system for which the panels were not designed originally. In particular the influence of rigid fixings or fixings in the central portion of a panel should be considered carefully.

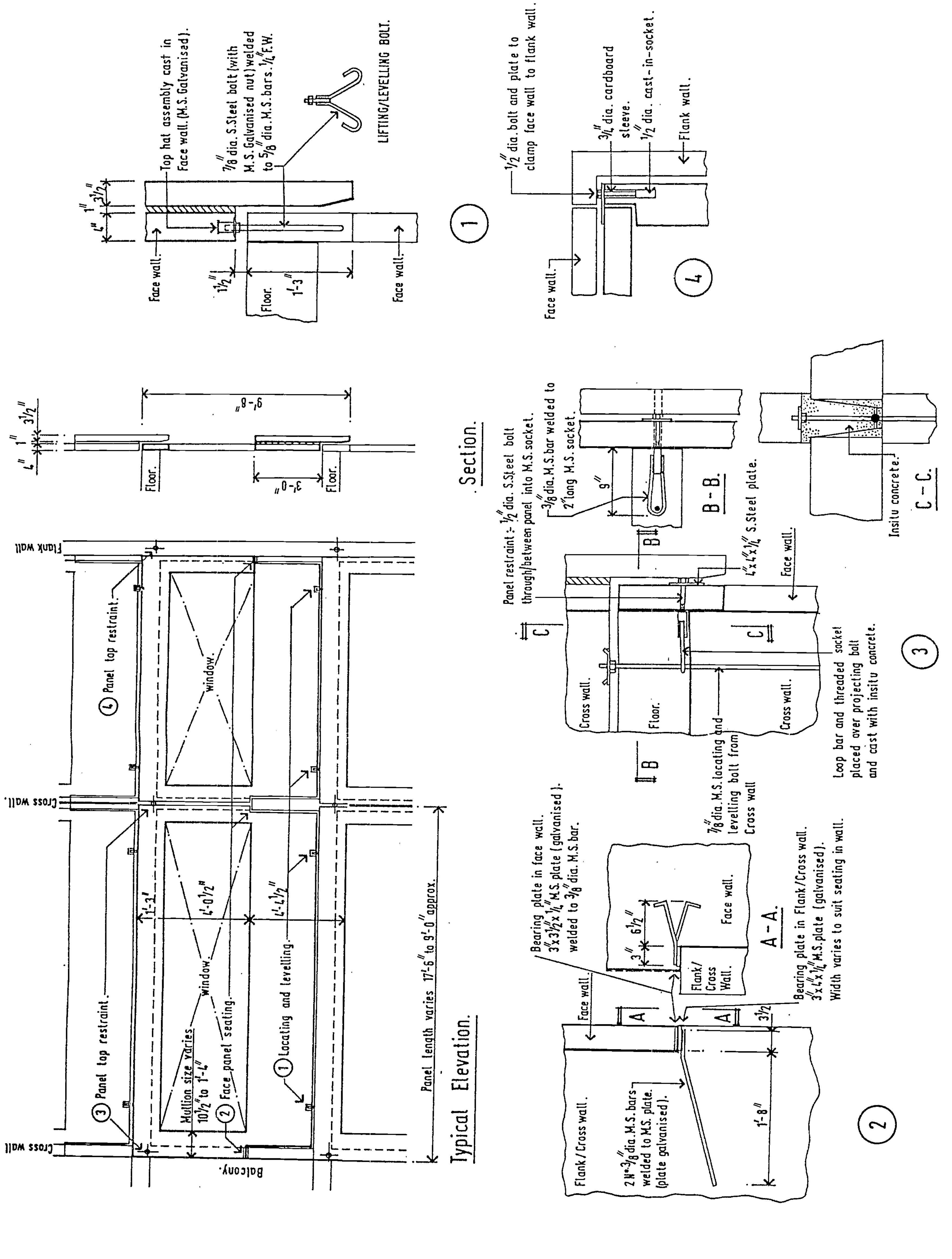


Figure G1.   Details of non-loadbearing cladding.

ANNEX H. STRUCTURAL BEHAVIOUR UNDER ABNORMAL LOADS

General

H1. The following discussion is based on insights into the behaviour of multi-storey panel structures under abnormal loads obtained from BRE tests on models[29].

H2. A major concern in the performance of large complex (in the sense of consisting of many elements) structures is their predisposition to progressive collapse. In this context, progressive collapse means a collapse with a failure front which moves away from the initial trigger or local overload (eg gas explosion) to envelope portions of the structure significantly larger than the part directly damaged by the initial load. The direction and extent of a progressive collapse depends upon structural form, ie the disposition of structural materials in the building and the pattern of weak and strong zones/joints, and on the facility with which the potential energy of the building can be released to motivate the failure front.

Progressive Collapse in High-rise Large Panel Structures

H3. In high-rise large panel load-bearing wall structures, the number of possible physical mechanisms of collapse is limited. The critical mechanisms will undoubtedly involve vertical progressions of failure fronts. These mechanisms may be separated for discussion into those which could be generated by a trigger at low levels and those which could be triggered near the top of the building (see Figure H1).

Low trigger-induced mechanisms

H4. The failure mechanism must involve shearing through door/window lintels or along vertical joints (Figure H2 (i) & (ii)) including either shearing of all the adjacent floor-to-wall joints in the wall (Figure H3) or shearing of the floor-to-floor joints (Figure H4) or the floor panels themselves at each storey above the trigger level. These mechanisms could all be followed by the collapse of floor panels either all at once or with the failure front moving up from the trigger level or down from the top of the building.

H5. In walls with weak wall-to-wall horizontal and vertical connections, each panel above the trigger level could act independently and drop off sequentially from the trigger level to the top rather like a zip fastener being undone (Figure H2(iii)). The floors might fold down (as happened near to the top of Ronan Point in 1968) or fall off.

High trigger-induced mechanisms

H6. If sufficient dynamic debris load is generated by the trigger, floor-to-wall joints could fail sequentially in shear from the trigger level downwards - the walls may not be affected and thus remain in place. (Figure H5 (i))

H7. In walls with strong vertical ties and relatively weak wall-to-floor ties, a sufficiently large trigger explosion could result in a flank wall peeling off like a banana skin and in so doing releasing the floors either to span in some diagonal fashion or to collapse progressively following the walls (Figure H5 (ii)).

H8. Potential trigger accidents which occur around the mid-height section of a building could generate high trigger failure modes below the trigger level and low trigger modes above the trigger level.

H9. There is, in general, no connection between the mechanisms of failure directly generated by an abnormal load and a subsequent progressive failure of a large section of the structure. This is principally because the energy source for primary failure (trigger) is generally external to the structure while the secondary failure reflects the conversion of potential energy in the structure to kinetic energy. There is consequently no simple design approach to deal with protection against local damage and control of subsequent progressive collapses. If however each mechanism described above is considered separately, fairly simple non-dynamic analysis should suffice to assess the effect of any potential trigger load. The assessment of local damage caused by an abnormal load is a different matter. It is probable that some form of dynamic analysis is essential unless an arbitrary static load is assumed (cf 34 kN/m^2).

Importance of trigger location

H10. Real buildings always have a significant degree of three-dimensional continuity. The effects of this continuity are less around corners. As a basic principle, the higher up the building local damage occurs, the fewer potential alternative paths there are to redistribute loads. Consequently damage high up a building near a corner is most critical. However, damage right at the top is less critical since there is no potential energy source from a mass of elements above the damage site. Damage at the bottom corner is probably the least critical unless there is an exceptional combination of weak shear joints between wall panels and floor panels.

Potential trigger loads

H11. Abnormal loads which might trigger progressive collapse in high-rise large panel structures include explosions (high explosives, piped gas and cylinder gas), impacts (aircraft and vehicles) and fire.

Explosive loads

H12. The local effects of an explosion on a structure will depend firstly, on the degree of venting achieved through windows, curtain walls etc which combined with the nature of the explosion, will determine the energy available to cause structural damage. Secondly, the local effects will depend on the local dynamic properties, which will control the influence of any particular explosion expecially where dynamic magnification of the effect of the applied pressure is possible.

The variety of possible loadings including peak pressure levels, released energy and rates of pressure increase when combined with local in-situ natural frequencies, makes the establishment of prescribed design rules on any other basis than pragmatism virtually impossible.

Impact

H13. There are two principal sources of impact, aircraft and road vehicles. Impact by road vehicles is the most likely source of damage[17] and will normally occur at very low levels of the structure. It is possible to design explicitly for a notional impact of a prescribed vehicle, travelling at a prescribed speed. The impact of a large aircraft on a building would involve so much energy that it is quite impractical to design explicitly against such an occurrence. Fortunately it is rare. Light aircraft would have a similar effect to a high-level explosion.

Fire

H14. It has been suggested that fire could be a potential trigger for a progressive collapse[3]. There seem to be two possible ways in which this might occur. Firstly, if the fire is sufficiently intense conceivably it could directly reduce the load carrying capacity of the adjacent concrete elements. However in view of the fire resistance of floor slabs and wall panels on TWA buildings this is unlikely to cause other than local damage such as occurred in the fire test[7]. For wall panels, loss of capacity would be the nearest real situation related to the notional removal of elements referred to in the Building Regulations[28]. It is considered that significant losses of capacity are unlikely to occur and are equally unlikely in the event of their ocurrence to precipitate progressive collapse.

H15. Secondly, a fire would cause local thermal expansion of the adjacent concrete components. It is difficult to see any critical effect on the walls or floor themselves of such expansion. However the possibility needs to be considered that expansion of a floor slab could deform the vertical line of the external load bearing walls sufficiently to critically lower the local buckling load. Such a possibility seems reasonable if the structure is treated as though it were two-dimensional. However, in view of existing knowledge of the size and distribution of domestic fires and the three-dimensional nature of the structures of TWA buildings it is suggested that such a buckling failure is extremely unlikely to occur (see Annex J).

Concluding comment

H16. It is worth recalling that in spite of the very great numbers of high-rise large panel structures which have been built world-wide and in spite of the many explosions, impacts and fires which have occurred in this large population of buildings, the failure of approximately 9% of the volume of the Ronan Point in 1968 is the only example of progressive collapse in this type of structure whilst in service which has been recorded.

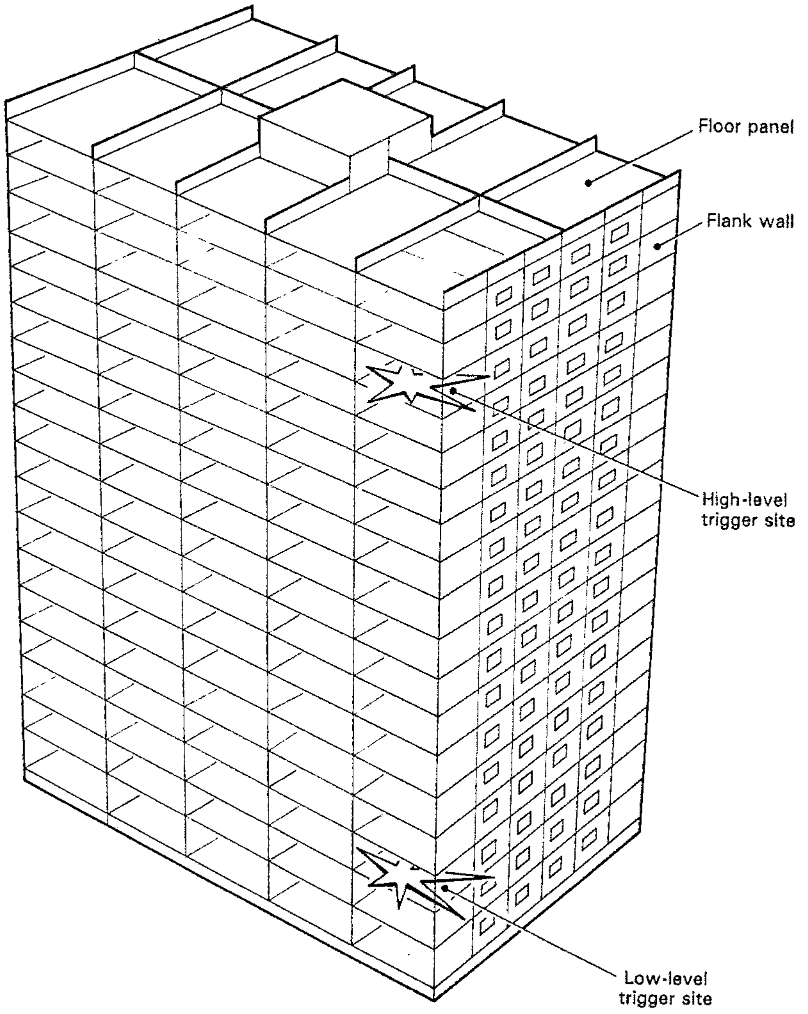


Figure H1. Location of trigger sites in a typical panel structure.

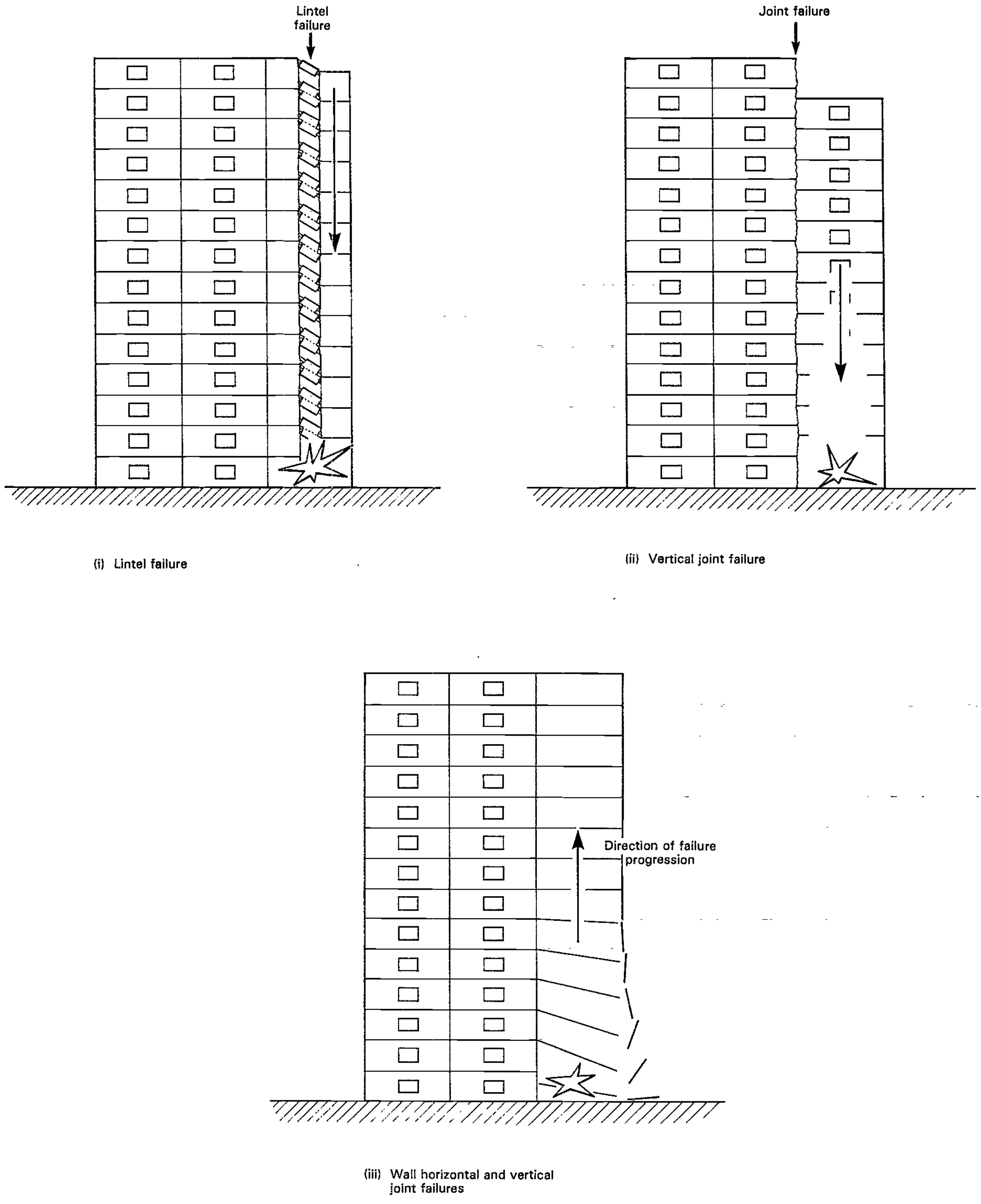


Figure H2. Possible failure modes of flank wall associated with low-level trigger site.

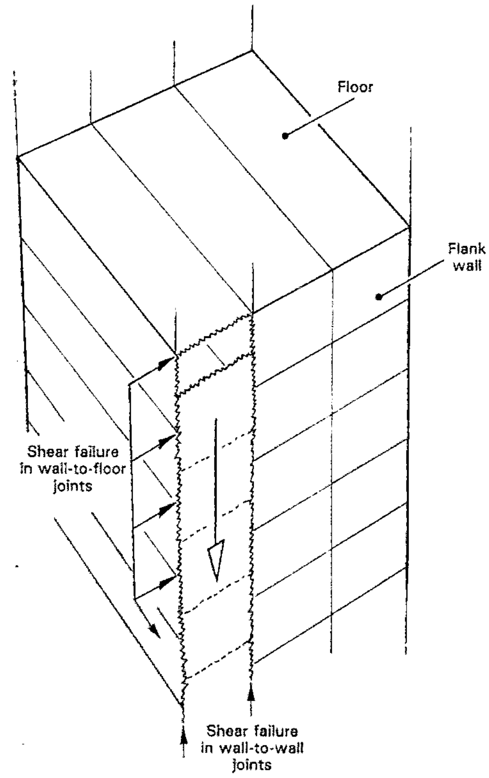


Figure H3. Failure mode with combination of wall-to-wall and wall-to-floor joint failure for low-level trigger site.

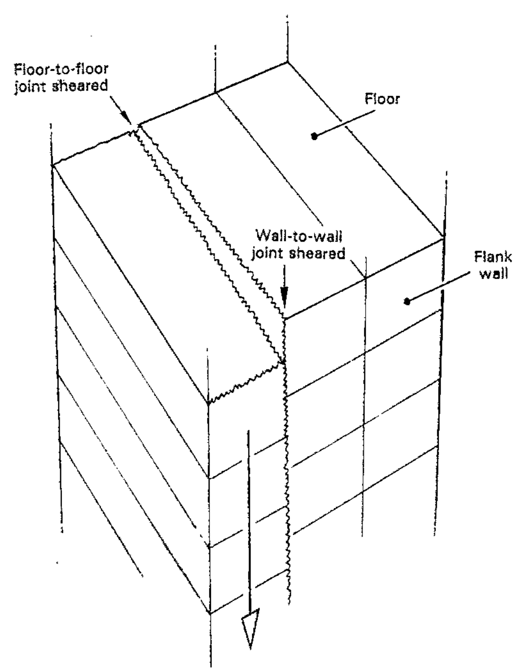


Figure H4. Failure mode with combination of wall-to-wall and floor-to-floor joint failure for low-level trigger site.

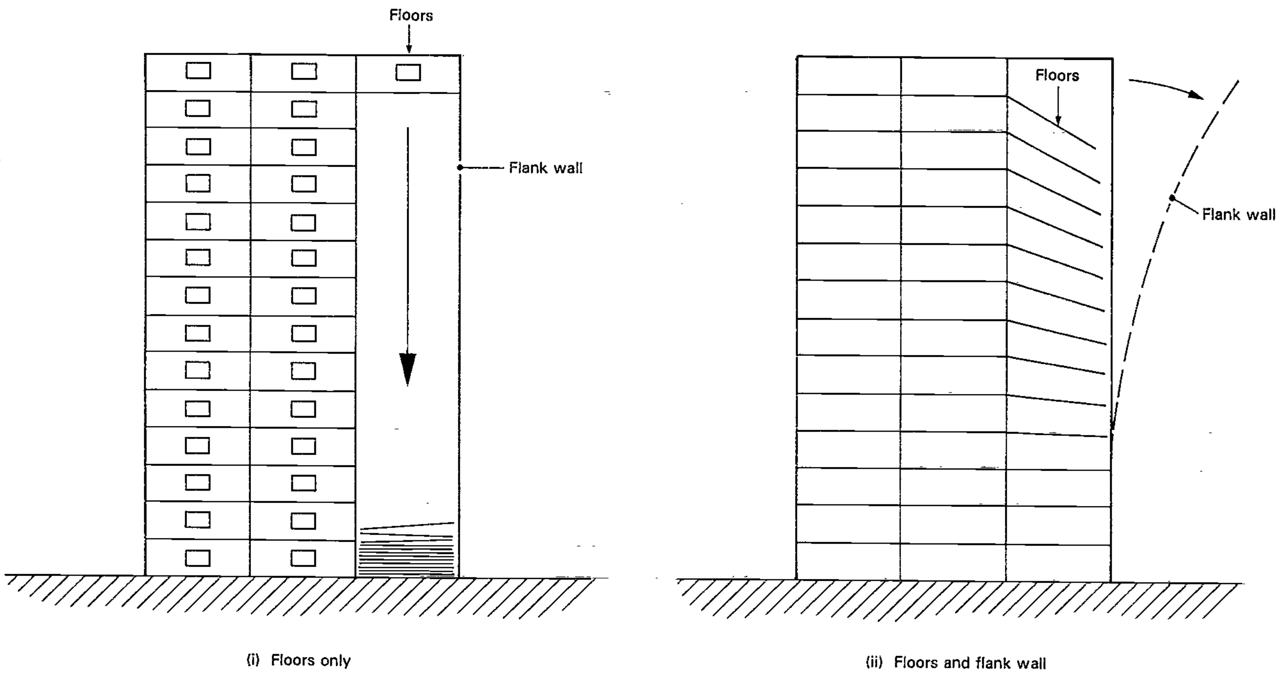
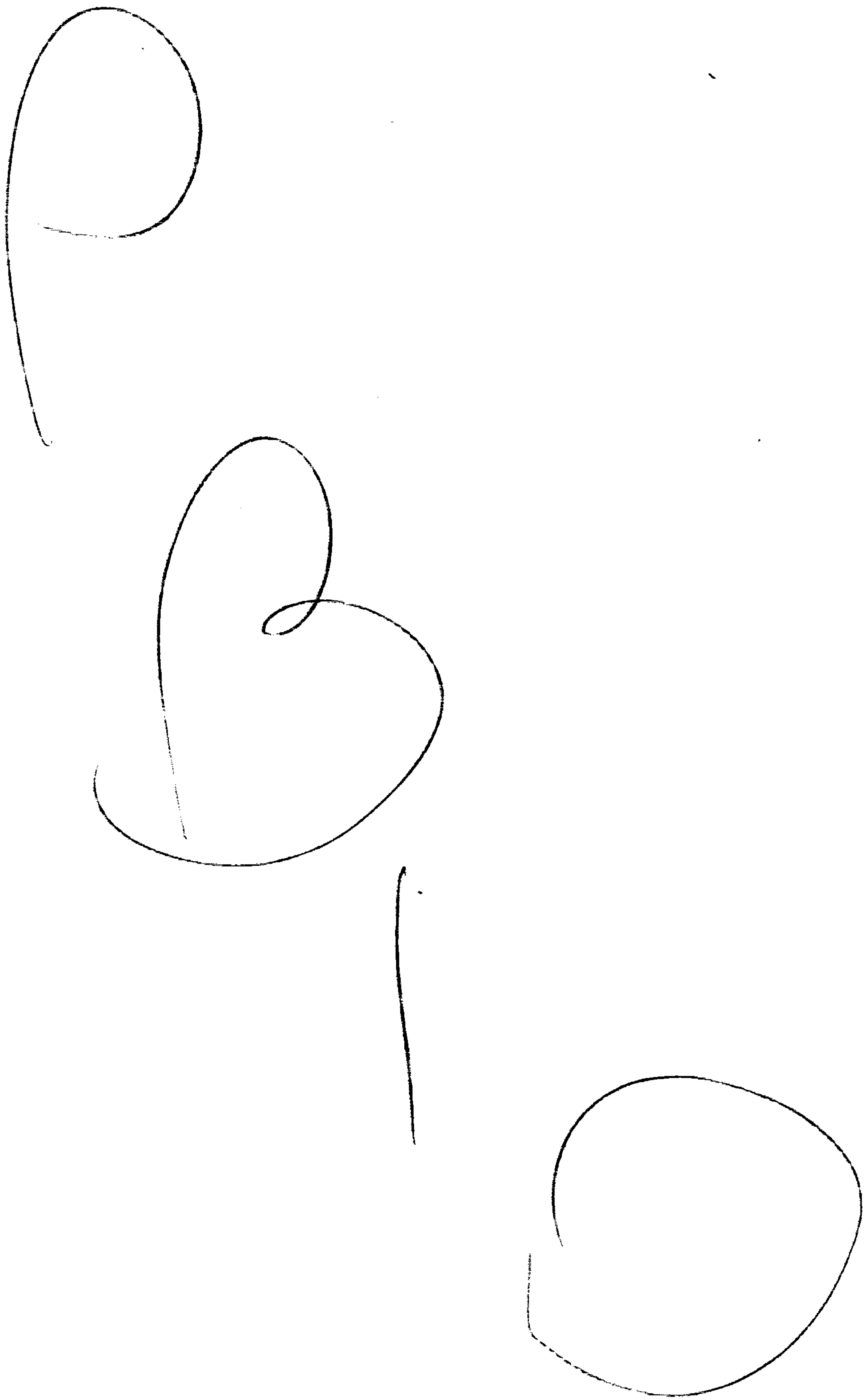


Figure H5. Possible failure modes associated with high-level trigger site.



ANNEX J. EFFECT OF NOTIONAL REMOVAL OF STRUCTURAL ELEMENTS

General

J1. The conceptual device of the notional removal of structural elements is a useful aid to the designer wishing to establish a required degree of continuity for a new structure[29]. In real structures, the removal of any element by accidental means, ie not by sawing or chemical attack, must lead to a great variety of consequential damage to neighbouring elements. Such damage necessarily depends upon the nature of the abnormal/accidental load and upon the strength and disposition of connections between the failed element and its neighbours. Careful consideration of the possibilities is therefore needed in an assessment of the potential performance of an existing structure which employs the device of notional removal of structural elements, since the consequential damage associated with actual element removal cannot be taken into account explicitly.

J2. Notwithstanding the preceding comment, a consideration of the effect of notional removal of structural elements can lead to some assessment of the effectiveness of the continuity between structural elements in 'bridging over' local damage in a TWA building such as Ronan Point.

Loss of floor panels

J3. The loss of any individual floor panel, ie AF, BF and CF (Figure J1) would lower the vertical load carrying capacity of the adjacent walls AW, BW & CW. However, since there are strong connections between the individual wall panels, buckling in a stack of single panels could not occur. If all these floor panels are removed, since there is only a weak shear connection between the flank wall and the corridor wall, it is possible to envisage the combined shear and buckling failure mode illustrated in Figure J2. There is a nominal bearing of the adjacent floor slab onto the corridor wall which would clearly resist this failure mechanism as would the non-loadbearing walls in the building.

Loss of wall panels

J4. In the event of the loss of any individual panel there is sufficient reinforcing steel to allow bridging or cantilevering over the gap.

J5. The case of two panels being removed is not quite as clear, nevertheless it is probable that diaphragm action of the floors above the damaged site will carry load around the gap especially since the reinforced screed and strengthening angles would be able to contribute. Shear failure is a possible mode in the event of BW and CW being removed.

J6. The third possibility of all three panels being removed would suggest a failure mechanism similar to that illustrated in Figure J2, if the panels below are involved, or as in Figure J3 if those below are not involved.

J7. The loss of internal panels is less serious than those considered above.

Discussions

J8. The above considerations are essentially qualitative and based on insights into the behaviour of panel structures under abnormal loads obtained from tests[29]. They indicate that the additional strengthening in the outer bays of Ronan Point, ie strengthening angles and reinforcement concrete screed, will have provided a substantial degree of continuity in the structure on the basis of the conceptual device of notional element removal from the three-dimensional construction.

J9. A less optimistic view of the beneficial effects of continuity following element removal is taken by the consultants[3,9]. Fortunately this aspect of performance is not the sole consideration in the assessment of the risk of progressive collapse (see 6.30-31).

J10. The available information on the performance of structures following local damage arising from abnormal loads, eg gas explosions, is limited. Research is needed to determine more clearly the circumstances in which progresssive collapse may follow local damage (Section 11).

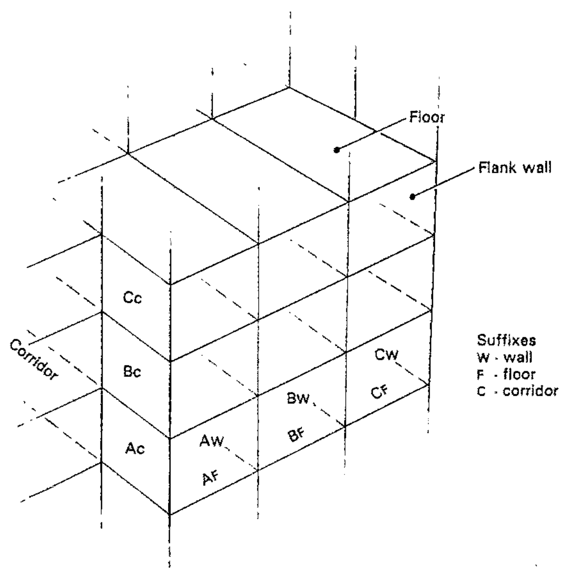


Figure J1. Section of flank of block.

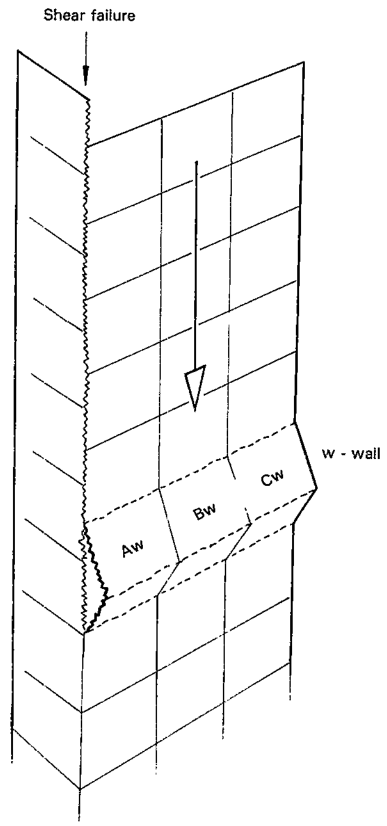


Figure J2. Combined shear and buckling failure of flank wall.

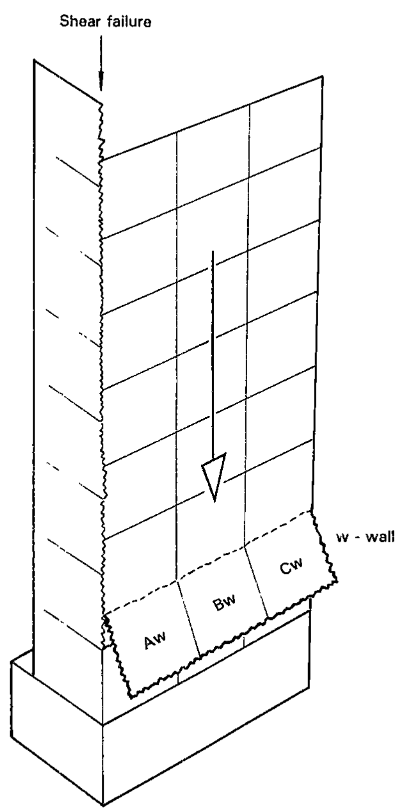


Figure J3. Crushing failure only, at bottom of flank wall.

PB10

ANNEX K RADAR TECHNIQUE

Background

K1. The apparatus has been demonstrated to BRE by Cambridge House Geotechnical Services and consists of an antenna which generates a signal and also functions as a receiver for the reflected signals. In essence the apparatus works on a classic radar principle, ie it sends out a known signal and then records the reflective signal. Different objects and configurations produce different returned signals and therefore can be distinguished from one another. In addition, metallic objects can absorb and then reradiate an extra signal thus producing further information.

One detailed difference between this system and typical aircraft radar, is that a continuous signal is not swept over an area but a series of pulses is produced and the source (antenna) is physically moved across the area being investigated.

Limitations

K2. In common with all waveform detection methods, the smallest physical change which can be detected with certainty is governed by the wavelengths of the radiation, in this case about 5 mm. It is possible to get below this level, say down to 3 mm by looking at phase change information but it would seem that this is not so straightforward.

K3. In order to gain sufficient information at a particular depth or about part of a multi-layer system, it is necessary to tune the antenna so that the unwanted signals from other parts of the construction are diminished, otherwise they may swamp the signal coming from the point of interest. This necessitates changing antennae to investigate different depths within construction.

K4. Since all that is recorded is a reflected signal shape which does not have any corresponding physical form of the item being investigated, the interpretation of the results has to be done in relation to a previously known response to an established condition. Therefore, for any given situation the use of the instrument will usually be restricted to circumstances in which the 'normal' condition can be established.

It may be that with experience it would be possible to characterise in general terms a number of specific types of defect.

K5. The system can only work for essentially dry construction. Water attenuates and diffuses a signal and reduces the level of response. The presence of a lens of water in dry construction may well not prove a problem although this has yet to be determined.

K6. The longer the wavelength used, the greater penetration into concrete that can be achieved (up to 2 m in dry concrete) but as the wavelength increases the resolution diminishes.

K7. Steel acts as a complete barrier and will shield anything behind it. It could not be used for example to detect voids in metal-sheathed post-tensioned ducts but could be used where PVC ducting had been incorporated.

Potential

K8. Despite the limitations of this technique it has in principle possible application for detecting a number of different types of defects in concrete construction. Many of the limitations would not restrict its use in this area. Its success is likely to hinge on its resolution properties and the extent to which special expertise, as opposed to simply a trained observer, is required for interpretation of the readings.

Future Action

K9. BRE propose to evaluate the apparatus under controlled conditions to assess: (a) if the apparatus in its present form is sufficiently well developed to be of wide practical use; and (b) the apparatus could be improved so that a less expert operative could be used.

This exercise may also include further assessment of the information obtained so far from the investigations on TWA blocks in London, including the physical data acquired to help validate the radar technique.

There are very few sets of this apparatus in existence and at the moment the cost of the apparatus is approximately £50,000. In the present form and level of development of the equipment, an expert user is essential.

ANNEX L. ASSESSMENT OF THE 'AS BUILT' QUALITY OF H2 JOINTS

Ultrasonic testing

L1. The conclusions reported[3] (4.3.2) in relation to the flank walls are:

"1. the precast concrete is consistent in quality.

2. the transit times for pulses through the H2 joint are significantly greater and more variable than those for the same distance through the precast units. This indicates that discontinuities or lower quality concrete or both are present: no conclusion can be drawn however as to the strength of the joint."

Removal of the skirt of the flank wall panels at 2nd and 3rd storey level of the north wall.

L2. This was carried out to permit inspection and photographic record of the dry-pack. The width and depth of missing dry-pack, as visible from the outside at 2nd-floor level, was measured along the 88% length of the north flank wall (91% of NE and 85% of NW) which had been exposed. The percentage voids for each of the six panels ranged from 3.7 to 23.3% : on average 15.6% of the area of the north-east wall examined was void, and 13.2% of the north west (BDP imply 28 and 22% respectively). However, most of the voidage occurred around the levelling bolts, only a few percent occurring between their locations.

L3. Measurements at 3rd floor level have not been seen in the same detail by BRE but a summary diagram gives the same impression, possibly slightly higher voidage, both between and around the levelling bolts.

Drill survey

L4. The purely visual observations in L2 and 3 above give no real indication of the quality of the dry-pack. A drill survey was carried out using a hand-held hammer drill with a 15/16-inch diameter bit. The resistance to penetration over a depth at which constant pressure could be maintained was assessed against a 7-point scale. Some twenty holes were drilled in the north flank wall from the inside at every floor level from 3 to 22, except for 8-10 where the underfloor heating experiment was in progress. Access was restricted to locations not covered by strengthening angles : locations of high stress at ends of walls were not accessible. About one third of the holes was inspected by a remote optical probe as a visual cross-check on the quality and extent of hard-pack. Comparison at level 3 showed, not unexpectedly, that an outermost zone of dry-pack was of low strength even though visibly present and intact. The measurements and corroborative observations were synthesised to give a visual picture of the voidage at each level.

L5. BDP conclude that:

"The width of the dry-pack mortar varied considerably from full width (150 mm) solid hard-pack to total void. However the average width was approximately half-way (ie 75 mm). The results obtained from the drill resistance were in general verified by random borescope observations throughout the North Flank Wall and also by the physical observations from the outside of the 2nd and 3rd floors. (Made possible by the removal of the external cladding.)"

Assessment of BDP site studies

L6. While the assessment of drill resistance by the operator is likely to be variable and subjective the overall results have been presented and quantified in only three grades viz, void, very soft and all other ratings. It is felt to be a reasonable method of representation in the light of the optical correlations.

It is noticeable that of the 18 levels investigated in the north flank wall the most complete filling was found at level 2, at which the highest stress might occur. At the end of level 2 it was possible to inspect the full dry-pack area, and void was found there around the levelling bolt. However this would be the easiest place to insert new packing if thought necessary.

Apart from a few locations at 21 and 22 floors the dry-pack was always found at the inner surface and generally over the inner part of the joint, ie over the in-situ concrete. Generally for the first 6 floors the voidage does not much exceed 30% on average.

The evidence of the NE wall at level 15 suggests that complete packing could be achieved. It need not be a presumption that dry pack in all TWA blocks is likely to be inadequate even though inspection may still be prudent.

L7. It may be more appropriate to concentrate on the condition of the dry-pack at the corridor ends of the NE and SE flank walls. Although the ends of the walls were not accessible, except at level 2, the proximity of the levelling bolt is likely to have inhibited good quality packing there.

Cracks

L8. Two cracks in the upstand, one vertical and one horizontal, were visible where the skirt had been removed. It is not apparent that they have been caused by overstressing.

Radar testing

L9. BDP[3] summarise their findings as:

"1. The testing was carried out from the outside of the building, through the outer skin, to examine the zones A, B and C for cracks, voids or other defects.

2. Due to the barrier of the outer leaf, the equipment was used near the limit of its capability and 16 cores were taken to obtain the true status to identify positively the anomalies indicated by the radar results. Close correlation was obtained between radar results and the core data."

L10. BDP[3] give more details of the defects as :

"A. Radar readings indicate the presence of 'star' cracking around the lifting bolts/levelling assembly at all panels at 2nd-floor level. These are confirmed by cores 1, 2 and 3.

A.[1] The vertical cracks are also indicated above windows generally up to the 13th floor level with one instance in the north east flank at the 16th floor. These are not visible within the cores or core holes and may be micro-cracks.

B. Voids are generally present in the in-situ concrete under the lower of the square twisted steel bars which is located in the gap between the projecting nibs of the floor units and the wall units. All of the evidence (including visual examination of cores and inside of the 16 core holes) indicates that the upper zone of the in-situ concrete is sound and the lower zone below the upper reinforcing bar is poorly compacted and that voids of up to 12 mm are general below the lower reinforcing bar, with some instances of larger voids.

C. From the radar results, voids in the hand-pack are indicated of up to 20% of the inner leaf thickness. However, the radar signals are affected by the presence of fibreglass and the actual voids may be greater."

L11. There appears to be reasonable correlation at 2nd-floor level between the external visual inspection of the hand-pack and the voidage deduced from radar technique, that is, up to 20%. Although this technique does not measure strength the apparent volume of dry-pack present is much greater than the proportion judged to be of relevant strength by the drill survey.

L12. The observations in L10 above on the in-situ concrete and the extent of voiding at the base of the pocket are the only data available from recent investigations. Photographs of the in-situ concrete recovered from the debris in 1968 tend to corroborate the findings.

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