Please note the contents of these documents contain detailed descriptions and diagrams of Grenfell Tower. This could be upsetting for some.

This version of the document has therefore been created with any photographs of fire damage or the interior of the Tower removed to minimise the amount of potentially upsetting or distressing information within it. A copy of the original documents with photographs can be provided on request.

Please take care when reading or circulating these documents.

The Grenfell Health & Wellbeing Service is a free and confidential local NHS service for children and adults affected by Grenfell. To talk to someone, you can get in touch by phone on 020 8637 6279 or by e-mail Grenfell.wellbeingservice@nhs.net.

This document and its contents have been prepared for the Ministry of Housing, Communities, and Local Government. For further information, please contact GrenfellTowerSite@communities.gov.uk

Jacobs

Independent Peer Review
of
Engineering Advice - Grenfell Tower
for
MHCLG

Dr J M Roberts FREng BEng PhD FICE FIStructE

Senior Director of Structural Engineering, Jacobs UK Limited

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A Introduction

1 This report provides an independent peer review of the engineering advice provided to MHCLG in the aftermath of the fire at Grenfell Tower in June 2017. The issued brief for the peer review is at Appendix A, but it is specifically noted that the brief might need to be amended to cover other issues, if this was considered to be appropriate by the author. In the event I have not proposed any additions to the brief (I did not consider any were required), but I have been provided with further documents which are referenced as relevant in this report.

2 The review has been carried out (and the report has been written) by me, Dr John M Roberts. I am a chartered civil engineer and a chartered structural engineer, and I am Senior Director of Structural Engineering at Jacobs UK Limited; a copy of my CV is at Appendix B. In accordance with the corporate policy of Jacobs the report has been internally reviewed, checked and approved for issue, but nevertheless it has the status of an "expert report" written by the author.

3 I have read through the entirety of the documents listed in the brief at Appendix A, and I provide a commentary on each of those which I consider to be relevant to the purpose of this review in Section C of this report[1]. I note that no detailed check has been made of the calculations themselves. The review of any calculations considers the method(s) adopted for any analysis and/or design, and the source (but not the accuracy) of the input data, and also takes account of whether the calculations in question have been properly signed off as having been checked and approved by the originating party. I have not yet visited the site[2] – I consider that the submitted documents provide me with a sufficient description of the building and its condition for the purposes of this report.

4 The *engineering advice* which is peer reviewed here comprises the advice provided – principally by Atkins – to MHCLG in respect of the current structural condition of the building (taking account of the significant propping and other temporary works either already carried out or proposed to be carried out), and also the engineering advice in respect of demolition of the structure in the future.

5 For the avoidance of any misunderstanding this report is not intended to provide any input into the technical advice being provided to either the *Grenfell Tower Inquiry* (Chairman Sir Martin Moore-Bick) or the *Independent Review of Building Regulations and Fire Safety* (Chair Dame Judith Hackitt). It is intended to inform the Secretary of State for MHCLG as to whether an independent review provides confidence in the advice and recommendations that have been received by MHCLG to date.

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[1] I also commented on three further documents supplied after the briefing list was provided, these being at C3, C11 and an updated version of the report reviewed at C10.

[2] However, I plan to do so on Monday 17th May, and I will revise this report if anything I observe on site causes me to change my opinions.

B Background information

6 Grenfell Tower is a 25-storey residential development that was completed in 1974, situated some 6 km due west of central London. It was initially designed *circa* 1966 – 1968. The partial collapse of the 22-storey residential block Ronan Point, in East London, occurred in May 1968 and led to significant changes in structural engineering practice (and changes to Part A of the Building Regulations, which were amended in 1972). Ronan Point highlighted the vulnerability of certain types of structure to what is termed "disproportionate collapse" where a relatively limited overload or occurrence of damage could lead to a progressive collapse of a much larger part (if not the whole) of a building structure. I understand that the design of Grenfell Tower took account of (and presumably complied with) the disproportionate collapse prevention requirements of the amended1972 Building Regulations, and that the construction of Grenfell Tower may have been delayed for this reason.

7 Grenfell Tower is largely[3] an *insitu* reinforced concrete framed structure, where the concrete is cast into formwork on the site. An *insitu* concrete frame structure will, in general, form a robust and "continuous" structure. As such, it different from many other multi-storey residential blocks constructed in the mid-to-late 1960s and the early1970s (such as Ronan Point) which utilised largely precast panels joined together with often ineffective – or certainly not particularly robust – connections. A photograph taken during the construction of Grenfell Tower is shown at Figure 1 below.

[3] There are some spandrel height precast concrete panels below the window levels around the perimeter of each floor, but the main core, columns and floor structures are all *insitu* concrete.

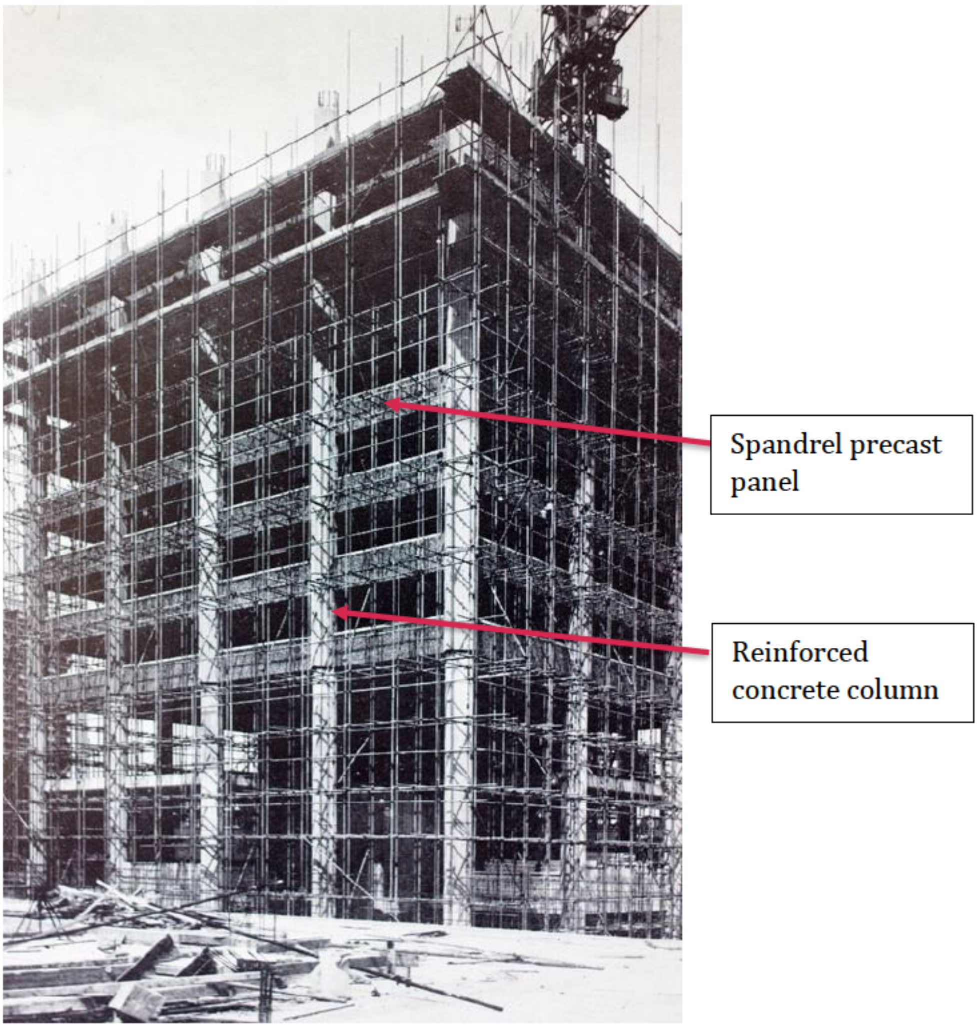

Figure 1. Grenfell Tower – reinforced concrete frame during construction

8 Grenfell Tower is *circa* 67.3m tall and 22.0m square. The layout of the top 20 storeys of Grenfell Tower (see Figure 2, below), level 4 to level 23, comprised 6 separate flats on each floor level (see Figure 3, below). In accordance with practice at the time, each flat was separated from adjoining flats (and from the central core area where the staircases and lifts were housed) by a concrete "compartment wall" (or party wall), and from the flats, below and above, by a concrete "compartment floor". The compartment walls and compartment floors and the structure as a whole had to be designed to provide a <u>specified period of fire resistance.</u> In a reinforced concrete structure this is achieved by a combination of the thickness of the concrete member and the so-called "cover" to the steel reinforcement which is embedded in the concrete. The thickness of the "cover" provides insulation to slow down the otherwise rapid rise in temperature that occurs when steel is heated in a fire - a crucial issue since steel loses strength at high temperatures. Concrete itself performs relatively well at high temperature but it expands and can ultimately crack or fracture, and it is very vulnerable as it cools down following a fire event.

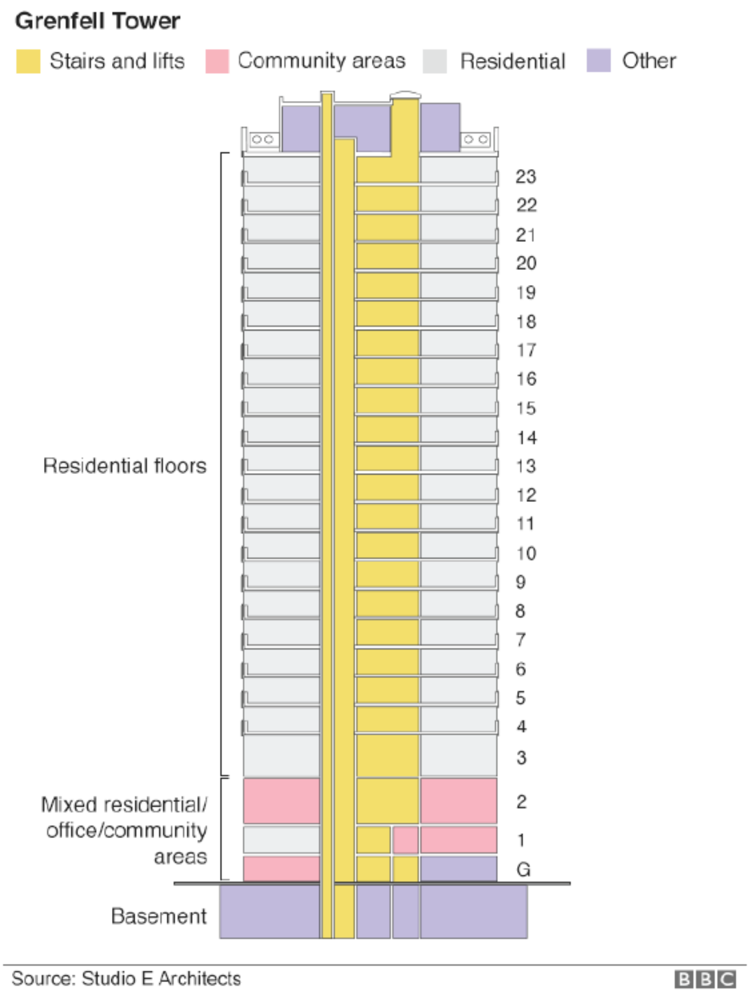

Figure 2. Section with floor level referencing (from BBC website)

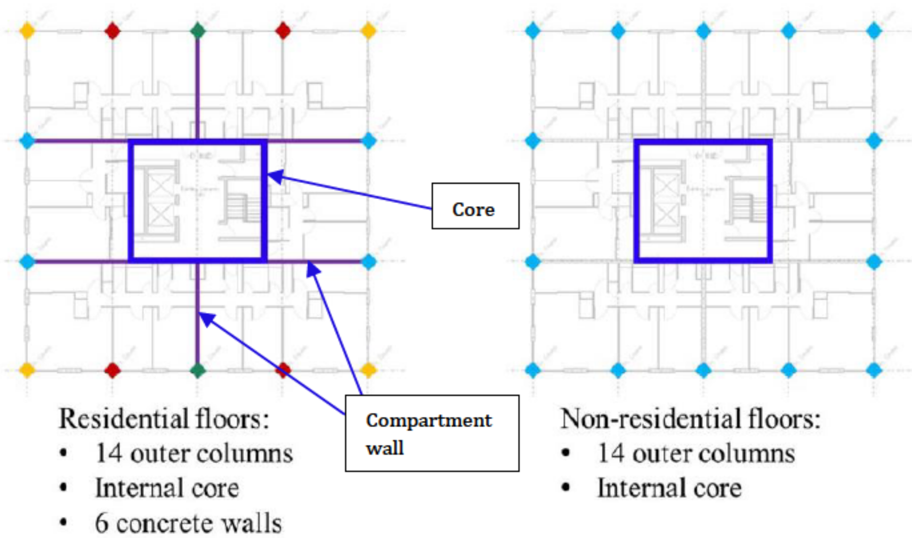

Figure 3. Typical structural floor plans (taken from Arup Report see C1)

9 The specified period of structural fire resistance for Grenfell Tower was set out in the Building Regulations 1972 at Part E “*Structural Fire Precautions*”.

Grenfell Tower is classified as "*Other Residential*" and is higher than 28m, and thus, in accordance with Table A to Regulations E5 :-

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TABLE A TO REGULATION E5 *Part 1—continued*

Purpose group	Maximum dimensions			Minimum period of fire resistance (in hours) for elements of structure(*) forming part of—		
	Height (in m)	Floor area (in m^2)	Cubic capacity (in m^3)	ground storey or upper storey	basement storey	
(1)	(2)	(3)	(4)	(5)	(6)	
III (Other residential) Building or part(†) having not more than two storeys	No limit	500	No limit	½	1	x
Building or part(†) having three storeys	No limit	250	No limit	1(*b*)	1	
Building having any number of storeys	28	3000	8500	1	1½	
Building having any number of storeys	No limit	2000	5500	1½	2	

the required period of fire resistance was $1^1/_2$ hours for all the structural elements (apart from the basement where 2 hours was required)

10 The logic behind these requirements was that this would provide enough time for anyone in the building to escape, and also would allow enough time for the initial fire fighting stage to take place and for the fire to be extinguished in the compartment in which it started. As is now well established from what happened at Grenfell Tower (and at other documented fires in multi-storey buildings) the weak link in this strategy was that a fire could exit the external façade via the window aperture and potentially then enter the compartment above, again through the window aperture. (At Grenfell Tower, as is now well-known, this strategy was compromised by the fact that combustible materials had been attached as cladding to the external façade during the 2011 refurbishment).

11 Notwithstanding the nominal $1^1/_2$ hours period of fire resistance noted above, I understand that the Grenfell Tower fire burned for around more than 24 hours in total[4] from when the fire started in the early morning of 14^{th} June 2017. Many of the floors above level 4[5] (where the fire is known to have started, see Figure 2 above) suffered an effective "burn out" – where every item of combustible material was consumed by the fire.

12 Many buildings are unable to survive a long-period fire, let alone a burn out, without collapsing. The most notorious recent example is, of course, the twin towers of the World Trade Centre in New York, which completely collapsed

[4] Although I note that the Arup Report (see C1 below) estimated that the intense fire <u>in any individual flat</u> would have only lasted between 30 minutes and 2 hours.

[5] The floor numbering system used on the building varies. Some references have used "level 1" for the first floor of the original residential accommodation, shown as level 4 on Fig 2. This report uses the floor numbering shown in Fig 2.

due to fire (and not due to the impact of the planes) after 105 minutes (WTC 1) and 56 minutes (WTC 2) respectively[6]. The structure of the WTC towers was significantly different[7] from the structure of Grenfell Tower and the fire event was also significantly different[8] but nevertheless the Grenfell Tower structure survived a massively extensive and long-lasting fire without collapse.

13 As far as I am aware the original design calculations and drawings for the Grenfell Tower structure have not been located, and it must be a reasonable assumption that they will not now be available to assist in any evaluation of the structure following the fire. For a reinforced concrete structure this is particularly problematic, as the diameter, spacing and location of the embedded steel reinforcement (which is a key parameter for any structural assessment) cannot easily be determined without destructive opening-up. I comment later on how this has been dealt with in the structural assessments that have been carried out, but, in my opinion, it is inevitable that a conservative approach to assessing the current residual strength of the structure will need to be taken, if only for this reason.

14 As a general comment, the key issue[9] currently facing the damaged Grenfell Tower structure is the risk of severe wind loading from a storm. Buildings are designed to perform safely in the event of the "worst" predicted gust of wind (lasting just a few seconds) that is predicted to occur during a 50-year lifespan[10]. Wind forces cause lateral loads on buildings, and in tall buildings such as Grenfell Tower this is likely to dominate the structural design, rather than the more obvious vertical (or gravity) loading condition which derives from the self-weight of the building and the vertical loads imposed on the floors from the "use" of the building. Generically the ability of a structure to withstand wind loading is described as lateral stability and I will use that term in this report.

15 Added to concerns about lateral stability is the medium-to-longer term issue of deterioration of the concrete structure. The building is no longer heated and is almost certainly not fully weathertight or watertight. The existing damage to the concrete structure resulting from the fire has led to exposed reinforcement, particularly on the underside of the floor slabs and to some of the columns. Even where not directly exposed, the concrete "cover" (see paragraph 8 above) to the reinforcement may have been reduced. In both situations the steel reinforcement is left vulnerable to corrosion, and

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[6] This information is extracted from "*Final Report on the Collapse of the World Trade Centre Towers*" NIST September 2005

[7] The WTC towers were essentially a steel framed structure.

[8] The WTC fires were ignited simultaneously across the entire floor plate of several floor levels with aircraft engine fuel.

[9] The building is of course unoccupied and hence is in a different situation from residential buildings where other considerations apply. The fact that workers are currently able to access the building can be managed to ensure that, in the event that a severe storm is anticipated, the building can be evacuated quickly.

[10] Statistically this is represented by a wind speed that has a 0.02 risk of occurring in any 12-month period of exposure.

corrosion leads to a loss of the steel cross sections and a consequential reduction in structural strength[11].

16 Temporary propping was installed in Grenfell Tower immediately after the fire (the Stage 1 propping), and this has since been extended (the Stage 2 propping) and is soon to be extended again (the Stage 3 propping). Much of the focus of the engineering advice centres on the efficacy of the temporary propping; this will be reviewed in this report.

C Detailed Review of Engineering Advice and other Relevant Reports

C1 Arup Report "*Grenfell Tower – preliminary review of post-fire stress state*" dated 30th June 2017.

17 This report was issued about two weeks after the fire and gave advice to The Royal Borough of Kensington and Chelsea ("**RBKC**"), the then owners of Grenfell Tower. The intention of the report was to assist the Dangerous Structures Surveyor to understand and evaluate the safety of the structure following the fire damage, and to provide guidance in respect of the initial temporary propping required. As such it remains a relevant document to this date, as the Stage 2 propping installation (which remains in place) was almost certainly conditioned by this report[12].

18 In my opinion the report is (remarkably) competent considering the extremely short time – around two weeks - that had elapsed since the fire. It provides a good general assessment of the existing structure and a very good description of the likely affect that the fire will have had on the structure.

19 In Section 2 "*The Structural System*" it is reported that "*There are RC separating cross walls between the flats above level 04 slab (level 01 on the fit-out drawings). These are assumed to be non-structural as they do not go to basement, however they may attract vertical loads by helping to support the slabs*". In my opinion the RC (reinforced concrete) cross walls are likely to have been designed as structural elements and are almost certain to be carrying vertical loading; however, I doubt that this makes any material difference to the conclusions of the report.

20 Also, in Section 2, it is stated "the *reinforced concrete design code of practice CP110 was published in 1972 and is a limit state code. This code of practice superseded CP114 which was a working stress code. As this was a time of transition from one code to another it is not clear which code the building was designed to, but it is expected that for typical elements the design from the two codes of practice would be similar".* I agree that reasonably similar designs

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[11] Reinforced concrete works as a composite material, and the steel reinforcing bars, which are normally embedded in the concrete, bond with the steel to allow the load to be carried. This when the bars are already exposed, or become exposed due to corrosion, the bond no longer exists, and the material can no longer work as "reinforced concrete".

[12] The Stage 1 propping was installed as the first action after the fire fighting ceased, and I understand that it was carried out without any reliance on this report.

are likely to have resulted from the use of either code, but, in my opinion, it is far more likely that the design will have been carried out to CP114, based on my working experience as a structural engineer from that time.

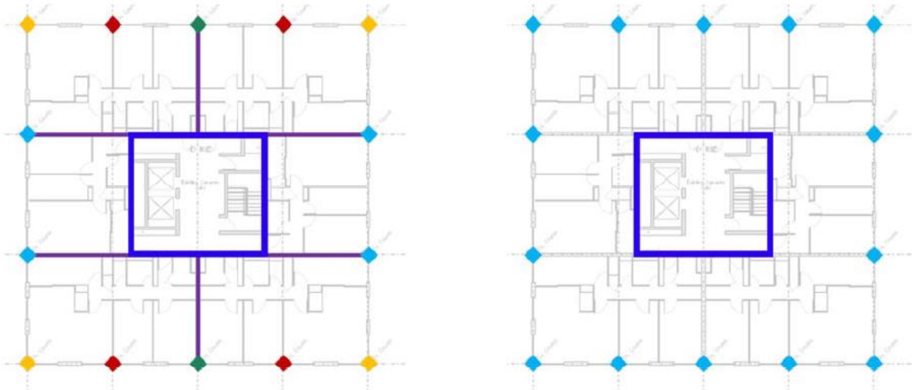
21 The report provides a very detailed analysis of the "progress" of the fire through the building height and across the building elevations, and it gives a logical and authoritative assessment of the time periods and temperatures to which any individual element of the construction is likely to have been exposed. I can see no reason to take issue with any of these findings or with the assessment that follows from them. I also note that such a retrospective time analysis has not been repeated by any other party, to the best of my knowledge.

22 In section 4.1 it is recommended that "*an extensive set of cores are taken from across the building once it is made safe to aid future analysis and understanding of the overall structural response*". A set of cores has been taken[13], but I note in any event that the recommendation was made primarily to provide data for comparison with any future fire damage to "other" concrete structures as well as a necessity for understanding the situation at Grenfell Tower itself.

23 The structural assessments given in the report assume that all the high yield ("square twisted") steel reinforcement bars in the columns will have been inadvertently "annealed" during the fire and will now have a strength equivalent only to ordinary mild steel. In my opinion this is an appropriate assumption to make for simplicity – although it is almost certainly too conservative for the reinforcement in the columns in the lowest few storeys, where there was no evidence of fire damage to the concrete.

24 Figure 3 from the report provides a useful summary of the column locations for the upper storeys ("residential") and lower stories ("non-residential") and I have already reproduced it here as Figure 3 (above) and this is repeated for convenience below:-

[13] A number of cores have been taken and these have all been petrographically examined and some have been strength tested – see Report C3.



Residential floors:
- 14 outer columns
- Internal core
- 6 concrete walls

Non-residential floors:
- 14 outer columns
- Internal core

Figure 3. Floor plan layouts showing the structural supports (walls and "diamond shape" columns)

25 The regular layout, with a central core (the dark blue "box" formed from 4 walls), 14 external columns, and (on the residential floors only) the 6 internal party walls (purple/grey colour) provides a straightforward structural layout and allows a reasonably simple assessment to be made of the structural behaviour.

26 The structural assessments provided in this report use a reasonable assumption as to the self-weight of the building itself and a reduced allowance for an imposed load[14] together with reduced load factors[15]. The full listing of the "*Assessment Assumptions*" is set out at Section 10 of the report. In my opinion these assumptions, taken together, gave a realistic basis on which to carry out an initial assessment of the remaining structural capacity of the damaged building.

27 The assessment, on this basis, shows that some columns that have physical damage need to be "relieved" of some loading, and the floor slabs generally have very limited load carrying capacity where the reinforcement is exposed and not effectively bonded to the concrete. In my opinion these conclusions appear to be what one might reasonably expect from a structure of this type subjected to this damage. I note, in particular, that the residual strength of the structure benefits from a very generous column size which has been used throughout [16].

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[14] An allowance of $1kN/m^2$ is used rather than the standard value of$1.5kN/m^2$ which applies to residential accommodation.

[15] Load factors of 1.1 are used in place of the standard values of 1.4 for dead load (self-weight) and 1.6 for imposed load.

[16] It is common practice to adopt a standardised column size and then use different amounts of steel reinforcing bars within the column cross section, depending on where the column is located on plan and in elevation. Note that load carried by columns is cumulative, and therefore increases storey-by-storey down the building. The columns at Grenfell Tower are an unusual "irregular octagon" cross section, based on a 960mm square column with the four corners cut off.

28 At Section 9 "*Conclusions and Next Steps*" the report notes that any floor slab which is permanently deflected will have little if any reserve of strength and should be "propped". Propping is also advised at physically damaged columns (several are listed). In overall terms propping is also recommended "bottom to top" including props below the underside of the "cross walls" (the concrete compartment walls, the purple / grey walls shown on Figure 3 above)[17]. In my opinion, however, propping below the concrete compartment walls is not actually necessary, provided that the six columns at the external ends of these walls are undamaged between ground level and the underside of Level 4, which I understand to be the case.

C2 Michael Barclay Partnership ("**MBP**") Report "*Structural Condition Study – Short to Medium Term Recommendations*" dated March 2019.

29 This report was prepared for the District Surveyor of Harrow who, by agreement between RBKC (see paragraph 17 above) and the London Borough of Harrow, was acting as the Dangerous Structures Surveyor under the relevant regulations.

30 The report relies upon a survey of the fire damaged structure "*Grenfell Tower. Fire-damage investigation of key structural concrete elements*" carried out by RSK and also dated March 2019 (see C3).

31 The report notes that the building was classified as a Dangerous Structure and that "immediately after the fire" the structure was "*inspected and made safe*" by a specialist contractor with the installation of temporary propping to the floor slabs. Subsequently, an external fully-sheeted scaffold has been installed, which the report describes as not being fully water-tight. The report summary states that the building (in March 2019) "*is currently considered "safe" but has to retain its Dangerous Structure status until such time that the temporary propping can be removed, or the building is demolished*"

32 The purpose of the report is stated to be (i) to provide recommendations to the Borough Surveyor as to retention or deconstruction of the building and (ii) to consider any additional works required in order to retain the building "*as it stands now*" for the next 1 to 3 years.

33 Notwithstanding the fact that the ownership of (and responsibility for) the building has now been transferred from RBKC to MHCLG, the report provides an important part of the narrative of investigations and engineering advice that exists for Grenfell Tower, and the fact that the building remains classified as a Dangerous Structure needs to be considered in any future actions taken.

34 The report states that the structure is so badly fire damaged above level 10 that demolition of this section of the structure is inevitable. In respect of the structure below level 10 it concludes that this could be repaired or replaced

[17] The compartment walls start at Level 4, and thus propping is advised on the line of these walls down to basement level. Currently the propping does not extend down to basement level, as discussed later.

"*if it is cost-effective to do so*"[18]. However, whether or not it is <u>cheaper</u> to demolish rather than retain the structure below level 10 does not appear to be the key criterion that will be now used to make such a decision, and therefore the conclusions set out in this report need to be treated with a degree of caution.

35 At page 5 the report states that "*It is understood that the original footprint of the building took an open form between Ground and 4th Floor Levels and that floor slabs were retrospectively added at some later date using steel beams and concrete slabs on permanent metal-decking formwork.*" This fact needs to be considered carefully in respect of any subsequent advice, as it appears to be an important point which could have serious effects on either demolition or retention proposals.

36 Also, on page 5, the report states "*The principal vertical structural elements comprise perimeter columns, cross-walls, and a central core containing a stair and lift shafts: The cross-walls do not extend below 4th Floor Level and so are not considered to contribute to overall sway stiffness. However, it is thought that the crosswalls may perform the function of "outriggers" and offer vertical support to the floor plates.*" In my opinion this is not a completely accurate assessment of how the structure is likely to perform (although it may well be accurate as an assessment of how the structure was originally designed). If the crosswalls act as "outriggers" (and in my opinion they will) then they will contribute to the overall sway stiffness if correctly analysed. However, this will not necessarily be a material issue unless the existing structure is retained to any significant height above the 4th floor level.

37 On page 6 ("*Fire Damage*") it is stated that emergency propping was installed immediately after the fire in June 2017 and that "*Details of this propping are contained in a separate report by DeconstructUK*".[19]

38 The assessment of the fire damage to the concrete structure is stated to have been carried out in accordance with the recommendations in the Concrete Society Report "*TR 68 Assessment, design and repair of fire-damaged concrete structures*", which was published in 2008. In my opinion this is the authoritative UK source document for assessing fire damaged concrete structures. TR68 defines five classes of damage to concrete following a fire *viz* DC0[20] (not affected by the fire) to DC4 (where very severe damage has occurred). I note that the damage classification of the Grenfell Tower structure has been carried out by RSK, and it is presented in their report (see paragraph 30 above), rather than directly by MBP.

39 On page 9 ("*Key observations on RSK Report*") MBP state that the damage classifications to the floor structure at levels17 to 23 are so severe that "*demolition and replacement is the only practical option*", whereas at levels 11 to 17 they state that the damage – whilst severe -is capable of being strengthened / repaired. A similar conclusion was reached for levels 4 to 10,

[18] Executive Summary page 3

[19] I have not reviewed this report, which has effectively been superseded by the Stage 2 and (about to be installed) Stage 3 propping

[20] DC0 means "Damage Class 0" and so on.

except that here the strengthening / repair works would be more limited and were estimated to be required to about 10% of the floor areas. Although not stated, this clearly means that at levels 11 to 17 significantly more strengthening / repair work would be required if these floors were to be retained, and in fact on page 10 it is then noted that more than half of the damage classifications at levels 10 to 14 were the most severe rating (DC4)

40 The conclusions reached in this report (and - presumably - accepted by the District Surveyor of Harrow in his capacity of the appointed Dangerous Structures Surveyor for Grenfell Tower) are (i) that "*the Building is demolished to Level 10 as soon as practical and the temporary propping system removed*"; and (ii) that "*a cost-benefit analysis is undertaken for retaining and protecting the Building below level 10 versus extending the demolition from Level 10 to Ground.*"

41 Although these two recommendations[21] have not (yet) been enacted, it remains the case (as far as I am aware) that if any part of the building is to be retained for any kind of re-use (even if without public access inside it), then the Dangerous Structures Surveyor would have to first withdraw the Dangerous Structure classification that currently applies to the building. As such, advice from MBP is likely to be sought again by the District Surveyor, and this needs to be borne in mind in any decision taken about the future of the building.

C3 RSK report "*Grenfell Tower. Fire-damage investigation of key structural concrete elements*" dated March 2019 (reference 1280180-01 (02))

42 This is essentially a factual report of inspections and testing - the engineering assessment of the results of the investigations has been provided by MBP (see Report C2 above), and has also been used (even if supplemented by other inspections) by other parties as well.

43 The investigations were carried out in May and June 2018 i.e. around one year after the fire; the delay is understood to have been due to the fact that the building was under the control of the Metropolitan Police, as it was classified as a crime scene[22]. By the time of the investigation, temporary propping (Stage 1 and Stage 2) had been installed and the building had been externally scaffolded and sheeted. The external scaffolding and sheeting did not extend across the existing roof and, as a consequence, the building – although significantly protected from weather – was not fully watertight. Concrete that has been designed for "internal" conditions will usually deteriorate with medium- to long-term "external" exposure[23] and during the investigation RSK reported on some water penetration to a limited number

[21] I am not commenting on these recommendations, since they do not explicitly form part of the engineering advice being provided to MHLG

[22] Even in May / June 2018 access to some of the apartments was not possible as the police investigations had not been completed.

[23] Mostly from intermittent water ingress (wetting and drying), which triggers corrosion of embedded steel reinforcement and in turn causes cracking and delamination of the concrete "cover" to the reinforcement.

of internal locations. In my opinion however, since the RSK investigations were dealing predominantly with fire damage to the concrete structure, any effects due to deterioration from weather exposure are unlikely to have been a material consideration for this report.

44 The factual results comprise both visual examinations and site or laboratory testing of samples. The visual examinations will clearly depend on the knowledge and expertise of the inspectors, but I note that the results have been classified by reference to standard published descriptions[24] commented on below; the site and laboratory tests are less subjective, except to the extent that the sampling has been determined also by the inspectors. I do not have any comments with respect to the accuracy of the results.

45 RSK have carried out a comprehensive survey of all accessible rooms on floors 4 to 23[25], and on the roof, of the building. Bearing in mind that the fire is known to have started in an apartment on floor 4[26] and progressed (as is normal) upwards, it seems certain that the overall coverage of the visual inspections will have been good.

46 One specific comment about the presentation of the visual survey results is that many of the findings have been "averaged" across floors, and in some cases, across the three main groupings of floors (lower, middle and upper) used in the report. Particularly in the lower floors group (levels 4 to 10) which suffered less damage than the middle and upper floors, this averaging can provide a potentially misleading impression as to the severity of the damage. In general, it is my opinion that, as well as the average damage, the worst cases of damage are a key feature, and where there are only a limited number of the most severe category of damage (DC4, see paragraph 38 above) calculating an average can conceal these, due to the large number of lower classifications which serve to take the average down to a low value. It is therefore my opinion that the report could be considered to be optimistic in respect of the conclusions that have been drawn about where remedial work are required, and about the balance between the feasibility of remedial works compared to the desirability of demolition.

47 The report tabulates the damage locations as columns, walls, floor slabs and slab soffits. It is very important to understand what these last two categories mean. The structural elements which provide both the vertical load capacity and the lateral stability for the building are the columns, the walls[27] and the floor slabs. In the report, the term "*floor slab*" relates to visual inspections on the top surface of the floors – which in fact comprise a non-structural "sand and cement" floor screed topping[28] - and the term "*soffit slab*" relates to visual inspections on the underside of the floors (i.e. the underside of the actual structural concrete slab). Thus, the key parameter in assessing the

[24] See also paragraph 38 above
[25] In the RSK report the floor numbering shown on Figure 2 is used
[26] Flat 16 on the 4th Floor. Flat 16 was a two-bedroom flat at the North East corner of Grenfell Tower.
[27] Both the core walls and the compartment (or "party") walls
[28] This is what I understand to be present

structural damage are the reports relating to the *soffit slab* and not the *floor slab* which might be counter-intuitive. (I am not suggesting here that MBP have misunderstood this distinction – all structural engineers will understand the situation I describe above).

48 In my opinion the Schmidt hammer tests reported are of relatively limited value and the results should be treated with care, as they are unlikely for several reasons[29] to provide any degree of accuracy in terms of a reliable indicator of concrete strength. In any event the Schmidt hammer results from the "floor slab" are tests carried out on the screed topping layer (see above) so these tests in particular cannot, in my opinion, give any guidance as to residual concrete strength (after the fire) in the floor slabs themselves.

49 The most reliable tests for assessing the residual strength of the concrete is provided by the concrete core tests listed at Table 2.4 (page 28 of the report). In total 25 cores were extracted from the concrete; 3 of the cores were from an "undamaged" location on level 4 and the remaining 22 samples were from locations spread throughout the "fire damaged" sections of the building, from columns, walls and from the underside (soffit) of the floor slabs. The results of visual inspections of the cores, petrographic examinations of the material in the cores, and strength tests on samples prepared from the cores are given in Section 4 of the report. In summary the concrete can be classified as being originally of good quality, with a well balanced and compacted mix; the fire damaged samples could be reasonably well identified from both the visual examination and the petrographic inspection of the cores. The strength tests need to be carefully considered as the tested sections of the cores were (deliberately) selected to avoid fire damaged sections. This means that any assessment of the residual strength of the concrete needs to first discount any parts of the concrete structure damaged by the fire. Typically (in the 22 cores extracted from fire damaged zones) the damage penetrated up to about 50mm depth from the surface exposed to fire[30]. The strength tests on the residual (undamaged) sections of the concrete structure varied from just under 30N/mm^2 at the upper floors to around 40N/mm^2 at the lower floors, which in my opinion indicates a material reduction in strength in the upper parts of the building even where visually / petrographically the concrete appeared undamaged. However, these strengths still indicate a <u>reasonable quality residual concrete material</u> underneath the fire damaged external surface layers[31].

50 The strength tests on samples of steel reinforcement are, at first sight, also encouraging, as they provide confirmation that the steel reinforcement post-

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[29] They estimate "strength" by an empirical relationship between the hammer rebound results and concrete cube tests, and the relationship should be validated from calibration tests on the exact concrete mix used for the building (which cannot now be done). As such the results only provide an indication of relative strengths, rather than a specific value.

[30] Although the fire damaged concrete in one case at least extended right through the wall thickness, which implies that the whole wall would have to be discounted.

[31] Structural grade concretes in the early 1960s were typically planned to have strengths of 21 to 30N/mm^2 (see CP114:1969 Table 5 *Standard Mixes*). These are 28 day "minimum" <u>cube</u> strengths and are related to, although not directly comparable with, small diameter <u>cylinder</u> tests carried out after 50 years.

fire is of the same order as the nominal steel strength beforehand. However, two points need to be borne in mind here. Firstly, steel reinforcement in many of the floor slab locations is now exposed (no longer embedded in the structural concrete) and is very vulnerable to corrosion and consequential loss of section area. The actual strength of reinforcement is calculated from the strength[32] multiplied by the steel area, so a loss of area causes a direct loss of the load carrying capacity of the structure. In addition, since reinforced concrete as a structural material relies on "composite action" between the steel and the concrete, when the steel is no longer embedded in the concrete this composite action can no longer occur. Secondly, the tests reported for the column steel reinforcement are not for samples of the main vertical reinforcing bars in the columns but are for the secondary transverse reinforcement called binders or links. These secondary reinforcing bars are reported as small diameter smooth round mild steel bars – but the main reinforcement in the columns is large diameter twisted high yield bars. As noted at paragraph 23 above in Report C1 the twisted high yield bars would have a higher initial strength, but following the fire are likely, in many locations, to have a reduced strength.

51 Section 5.3 ("*Potential for Repairs*") provides a useful summary of the extent of the fire damage, where in overall terms about 30% of the concrete falls into the category DC3 or DC4. Both these damage categories are associated with a requirement for major structural remedial works, and it is stated in the guidance document used for this report[33] that re-construction may be a more appropriate approach to remedial works than attempting *insitu* repairs. The summary also provides a reminder that whereas most of the DC3 and DC4 damage locations are at upper floor levels, some are found at lower floor levels as well. This is a factual observation, but it has an important bearing on any engineering assessment made using this or subsequent survey inspections.

C4 LB of Harrow "*Dangerous Structures Letter*" dated 24th August 2020

52 This letter is from the appointed Dangerous Structures surveyor / advisor[34], [redacted], of the London Borough of Harrow, to provide an update to MHCLG as the owner (by that date) of the building. The building had been declared as a Dangerous Structure in June 2017 following the first inspection he had made on 18th June 2017. The building, some 3 years on, remained a Dangerous Structure and [redacted] explains the statutory duty which that classification places on the owner / the local authority. The classification prevents the building being used until it is either repaired to the satisfaction of the appointed surveyor or demolished[35]. In the absence of either action

[32] Strength here is expressed in terms of a stress - a force per unit area and not in terms of a force (or load)
[33] Concrete Society Report "*TR 68 Assessment, design and repair of fire-damaged concrete structures*" see paragraph 38 above.
[34] He uses both descriptions in the letter.
[35] He uses the term "deconstructed" rather than "demolished".

(as currently applies here) the appointed surveyor has a statutory duty to ensure the building is temporarily made safe or, if that cannot be done, an exclusion zone is enforced to avoid any risks to life. The surveyor has wide-ranging powers under the relevant legislation[36] to enforce these requirements.

53 The letter gives an unequivocal recommendation in terms "*However, as the Dangerous Structure advisor my recommendation is that the structure should de-constructed at the earliest opportunity*". In my opinion and experience a recommendation from a Dangerous Structure surveyor carries very significant weight.

54 The letter itself has as an appendix a letter of the same date from MBP, the structural engineer giving advice to the surveyor. The letter was written after a joint visit to Grenfell Tower on 14th August 2020 by [redacted][37] from MBP and [redacted], the surveyor. MBP give a summary of the history of this issue from June 2017 onwards, and they particularly note that (i) it was severely damaged during the fire; (ii) the building's stability is currently reliant on temporary propping; (iii) that the current exposure conditions are allowing further deterioration to occur; and (iv) that the (inevitable) demolition will become "*more dangerous*" the longer it is delayed. The MBP letter concludes by stating "*the need to demolish[38] the building is a fact, not a choice.*"

55 I have no comments to make on the letter from either party. In my opinion each provides a reasoned analysis of the situation from the perspective of this building subject to the Dangerous Structures notice.

C5 Atkins Report "*Grenfell Tower. Final Design Validation*" dated 4th June 2020

56 I note that the various documents and reports prepared by Atkins for MHCLG are reviewed in this report in date order.

57 The Final Design Validation ("**FDV**") report is a 115-page document which provides an engineering assessment of the stability of Grenfell Tower after the fire and after the installation of Stage 3 temporary propping. Here "stability" can be defined as an ability of the structure to safely carry the loadings to which it is currently subjected i.e. to deal with (i) the vertical loading from the self-weight of the building and from a loading allowance related to access / inspections and necessary short term remedial works (not long term occupation) together with a snow loading allowance on the roof; and (ii) the lateral loading from wind forces.

58 Of particular importance is the fact that the entire assessment is based on the assumption that temporary propping ("**Stage 3 propping**") is installed throughout the entire height of the building from basement level to the

[36] Sections 77 and 78 of the Building Act 1984. I assume that the action has been taken under Section 78 "by agreement" with the owner.

[37] [redacted]

[38] I note that elsewhere in the letter "*demolish*" is described as demolition at least down to level 10

underside of the roof. Whilst the propping currently in place (Stage 1 and Stage 2 propping) extends down to the Ground Floor level (see Figure 2 above)[39], the proposals for the Stage 3 propping are that it is initially installed from Level 4 upwards ,and that it will only be installed <u>below</u> Level 4 if the demolition of the building then does not commence.

59 Atkins state "*If the propping strategy changed such that propping started on a suspended slab level rather than at Basement Level, a high-level investigation of the consequence of this change could be carried out through modification of the analysis model used for the FDV*". Since, as I note above, the Stage 3 propping as currently planned to be installed will start at a suspended floor level (Level 4) then this report is assumed to have represented a final option if the Stage 3 propping is to be extended down to basement level. (A subsequent report by Atkins (see C6) does however deal with the case where the Stage 3 propping is to be installed from Level 4 upwards).

60 The Executive Summary highlights the fact that the assessment is based on the assumption that no propping load is carried by the existing floor slab structure – it follows from this assumption that no propping loads are carried by the columns or the walls either (although this is not stated). This clearly cannot be the case <u>unless or until the propping is extended down to basement level</u> (which is what this report assumes). I also note that this assumption appears to be based upon an assumption made by the Temporary Works (propping) designer and is "*not investigated /validated further*" in the FDV. In my opinion it is a reasonable assumption as far as the Stage 3 propping design (from basement level upwards) is concerned Again it is necessary to see the later report (C6) to understand that this assumption will not be correct if the propping starts at Level 4, since it is obvious that, in this case, the propping loads have to be carried by the floor slabs.

61 The major "unknown" in the overall structural assessment is the lack of precise knowledge as to the size and number of the reinforcing bars, particularly in the columns. This is explicitly acknowledged at the end of the Executive Summary on page 6. The code of practice which in my opinion would have been used for the original design (CP114 1969 see paragraph 20 above) requires that columns have a minimum reinforcement quantity of 0.8% (by cross sectional area) and a normal maximum of 4% (by cross sectional area); this at least provides a framework for the range that is likely to have been used. Since the column sizes do not vary with the height of the building a reasonable approach would be to assume 0.8% at the upper level columns increasing in batches to say 3%[40] at the lower levels. I comment later on the assumptions made in the report, and I also note that further investigations have been carried out to determine some of the column reinforcement bar sizes and spacings.

62 At section 1.2 "*Relationship to the Initial Design Validation (IDV)*" it is explained that the FDV replaces entirely the Initial Design Validation ("**IDV**")

[39] So that there are 5 unpropped stories at the bottom of the building.
[40] 3% as a conservative value, instead of 4%

which was prepared in July 2019, and that the FDV therefore "*acts as a single standalone document*". As such it is my opinion that I do not need to carry out a detailed review of the IDV.

63 Section 1.3 provides an extremely important qualification to the FDV, which is that the assessment is only of the performance of the "primary structure" as it now exists (which here means the fire-damaged existing reinforced concrete comprising floor slabs, columns and walls). In effect this assessment therefore assumes that the installed propping has been correctly designed to carry the loading which is "allocated to it" (my wording). This is a reasonable approach to take, but there are two ensuing *provisos*. Firstly, and rather obviously, that the propping design has been properly carried out. Secondly, and more importantly, that the allocation of loads, as between the propping and the residual existing structure, is reasonable and has been followed by both this assessment (for the existing structure) and by the Temporary Works designer for the propping design calculations[41]. This *provisio* (although not stated exactly as I have set out above) is explained at the first bullet point in Section 1.3.

64 Atkins list the sources of information used to inform their assessment at Section 2. The survey information listed as document 1 is a slightly earlier version of the RSK survey report which I have reviewed at C3 above, but I understand that the information in the March 2019 version I have reviewed is essentially identical to that in the February 2019 version listed by Atkins.

65 The geometry (dimensions) of the concrete elements, and the assumed contribution of the concrete elements to structural performance, is set out at Section 3.4 *Geometry* of the report. I have no access to any independent means to confirm the element dimensions, but they are based on a combination of record drawings and site data and appear reasonable. In my opinion, the assumptions Atkins have made as to the contribution of the concrete elements to structural performance are correct / appropriate (and here I note that the Atkins assumption differ from those set out in the Arup report, see C1 above).

66 The assessed material strengths used by Atkins are set out in Section 3.5 *Material Properties*. They have used as the base data the test results obtained by RSK (see C3 above) on which I have already commented. Here Atkins re-convert[42] the residual concrete strengths obtained from the RSK laboratory tests back from (standard) cube tests to equivalent cylinder test results, so as to enable the strengths to be used with the current code of practice used for the assessment (see later). As a reminder, the RSK strength tests were

[41] This is to ensure that any loading that exists is carried by one of the two possible methods. It would be acceptable for the total loading to be allocated so that more than 100% is carried (e.g. by assuming 60% in one component and 50% in the other) but not so that it less than 100% is carried (e.g. by assuming 40 % in one component and 50% in the other).

[42] The RSK tests were carried out on different diameters of cylinder cores cut from various locations on the building and these results were converted (by RSK) to equivalent cube strengths. Atkins have here converted the cube strengths back to standard diameter (100mm) cylinder strengths to match with the structural code of practice they are using for the assessment (which utilises cylinder strengths).

carried out on nominally "undamaged" (i.e. visually undamaged) sections of concrete from locations subjected to the fire and, in my opinion, they represent the best data available on the residual strength of the "intact" (nominally undamaged) concrete in these zones.

67 The concrete strengths calculated by Atkins have been averaged out across all results for each type of element (slabs – 27N/mm^2; walls 18N/mm^2; columns 23N/mm^2) whereas RSK provided values grouped into lower floors, middle floors and upper floors. There is a theoretical advantage of using variable strengths graduated by floor height, but equally the approach selected by Atkins uses a larger data set for each strength calculation. I am content with the approach that Atkins have used here; within the range of the other key parameters that apply, the apparent added accuracy of variable strengths is almost certainly illusory. (I note that the values set out above are not on the same basis as, and therefore not directly comparable with, the "bare" results from the RSK data, which I summarise at paragraph 49 above).

68 The reinforcement steel strengths calculated by Atkins are set out at Section 3.5.2 *Reinforcement*. As with the concrete strengths the base data used is the series of laboratory tests on site obtained samples of steel reinforcing bars carried out by RSK. Here Atkins have only considered the test results for mild steel round bars used as main reinforcement – so they have excluded the test results on the 8mm diameter secondary reinforcement, and they have also excluded the single test on a square twisted bar sampled from a column. They have then conducted a statistical analysis of the results to estimate a "95% characteristic strength"[43] which is a value needed for use in current structural codes of practice. In my opinion the sample size is perhaps too small to allow a proper statistical value to be derived, but the characteristic strength that is calculated (a 0.2% proof stress of 241N/mm^2) is a sensible value to use.[44]

69 I note that in the absence of any other data Atkins must have used this reinforcement strength for the square twisted reinforcement bars in the columns, although they do not state this or explain why this has been done. I note that square twisted bars – which are normally significantly stronger[45] than mild steel round bars – effectively reduce in strength after severe heating in a fire back to typical mild steel values (see paragraph 23 above). Hence, in my opinion, what I understand has been assumed by Atkins regarding the column reinforcement strength is acceptable.

70 The assumptions made by Atkins as to the loading applied to the concrete structure are set out at Section 3.7 *Loading*. The wording used here is potentially confusing – Atkins state "*Therefore, only the effect of live load being applied to the slabs post-prop installation, is considered*". This

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[43] The average value of the set, less 2 x the standard deviation of the set. With a large enough sample 95% of the results are expected to be above the characteristic value.

[44] At the time Grenfell tower was designed mild steel reinforcement had an implied value of about 255N/mm^2 (140N/mm^2 /0.55)

[45] They are normally termed high yield steel

presumably means that the installation of the Stage 3 propping is "neutral" in effect i.e. that each slab carried its own dead load before the propping was installed, and that no load is transferred into the propping until live load is applied to the floor slabs. This is how I interpret Section 5.1.2 "*Staged Analysis*" where it is stated "*A staged analysis is used to determine the stresses in the structure. In the staging process the structure is first analysed with no props in place, the propping is then introduced and additional loading applied. In this manner the load share between primary structure and propping is captured in assessing the impact of additional loading on the structure after the installation of the Stage 3 propping*".

In practice some dead loading is likely to be currently carried by the Stage 1 and Stage 2 propping[46], and this propping will be removed as (or after) the Stage 3 propping is installed, which means that the Stage 3 propping may well have to carry some of the existing dead load.

71 The imposed loading used by Atkins in their analysis is considered to be acceptable, although possibly it is slightly conservative for a long-term situation[47]. They have used an imposed load of 1.5kN/m^2 for any six sequential floor levels and a loading of 0.6kN/m^2 for all other floors and the roof. I note that the original design of Grenfell Tower would have almost certainly used an imposed loading of 1.5kN/m^2 as well, since it was (and still is) the accepted value for residential use[48]. To give an approximate indication of these imposed loads, noting that the gross floor area is 22m x 22m (i.e. 484m^2), then 1.5kN/m^2 gives a load per floor of 726kN or 74 tonnes. 6 floors at this loading and the remaining14 floors at 0.6kN/m^2 totals over <u>850 tonnes of imposed load</u> allowed for in the assessment in the building as a whole[49]. For comparison purposes the self-weight of just the floor slabs (excluding the columns and the walls) in the 20 original residential floors is about <u>5,500 tonnes</u>.

72 As noted later at C11, the propping design by Cantillon may have used a slightly different imposed load arrangement, but the Cantillon design has also allowed for the plant and equipment loads from a demolition scheme, so is not directly comparable with the Atkins analysis

73 As I have noted already at paragraph 14 above, wind loading is a key design parameter. The existing (damaged and propped) structure is almost certainly most at risk currently from wind loading during a storm. The FDV deals with wind loading at Section 2.7.2 and 2.7.3. At 3.7.2 Atkins simply refer to the current wind loading Eurocode and note that London is allocated a "wind speed" of 21.4m/sec. At 2.7.3 a load factor for wind load is stated to

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[46] Depending on the sequence of installation and the degree to which the propping was "tightened".

[47] By this I mean that the loads currently will be lower than the imposed load assumed for the calculation. This is obviously a safe assumption to make, and will prove helpful when loadings from any demolition scheme are considered.

[48] This value is also specified in the Building Regulations, then and now, for residential accommodation generally.

[49] This is a generous value, as I have not deducted the "voided" floor areas such as lift shafts and vertical risers. Nevertheless, it gives a useful "ball park" value.

be 1.2. In my opinion this is the correct approach to dealing with wind loading and the description is clear to structural engineers, although it might not be so for other parties. By way of explanation, the relevant Eurocode is EN 1991-1-4 :2005 together with the UK National Annex. This code allows for a calculation of the worst wind speed likely to occur annually with a 0.02 probability. Since 0.02 = 1/50 it is often loosely referred to as "50-year wind" or a wind speed likely to occur on average only once in any 50-year period of exposure[50]. The wind speed quoted by Atkins comes from a wind speed map of the UK in the Eurocode which sets out the hourly mean wind speeds[51] for all UK locations. The hourly mean wind speed is measured at 10m above ground in flat open unobstructed countryside 100m above sea level and away from the coast with. Naturally a lot of adjustments are required to convert this to the worst 3- to 15-second long gust velocity[52] during the worst storm in 50 years at a specific urban location such as Grenfell Tower. A key parameter affecting the gust velocity is the height of the building – wind speeds are a lot higher at 66m above ground compared to 10 m above ground. As an approximate value, the "50-year" worst gust velocity at the top of Grenfell Tower is likely to be about 40 to 45m/sec (90 to 100mph). The final step in the process is converting the gust velocity into a wind pressure, which is a loading intensity with the same units as dead load and imposed load i.e. kN/m^2.

74 Atkins correctly note that the surface area exposed to the wind is currently larger than the original façade area of Grenfell Tower, because the wind force acts on the sheeting material of the external scaffolding and this scaffolding is assumed to be supported laterally from the existing building.

75 The code of practice used to carry out the assessment is stated at Section 3.8 *Design Codes for the Purposes of Assessment* to be Eurocode 2. This is the current UK code used for design of reinforced concrete buildings. Engineers (including myself) are often concerned about carrying out a structural check using a different code to that used for the original design; however, the FDV is not a check on the existing design as such, and in any event modern structural computer analysis and design programmes are not actually available to comply with the code (CP 114) which I consider was used for the original design. I am therefore of the opinion that the approach taken here is acceptable. I also note that Atkins are using CP114 to arrive at estimated values for minimum reinforcement quantities, and for the concrete cover to the reinforcement; again I consider this to be the correct approach.

76 Section 4 of the report gives the information about the damage to the concrete which has been used in the analysis of the residual concrete structural capacities. The damage data comes primarily (as already noted)

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[50] It is often described by structural engineers in this way. It is an "approximately correct" translation of "*0.02 annual probability wind speed*", and is perhaps is easier to understand.

[51] As the term implies this is the average wind speed over a 1-hour period. Within this hour much faster gusts of wind will occur, and it is these gusts which are of interest for structural design.

[52] Velocity is a vector quantity, with the direction being relevant, unlike speed which is a scalar quantity and independent of direction.

from the RSK survey report from February 2019 (see paragraph 64 and section C3 above). This has been supplemented by a survey carried out by Atkins in February 2020 which is summarised at Section 4.2 "*Atkins structural damage survey*" and reported in detail in Appendix A. It is worth considering the level of detail which is recorded on the Atkins survey – Figure 4 below is a copy of Figure 4.7 of the FDV:-

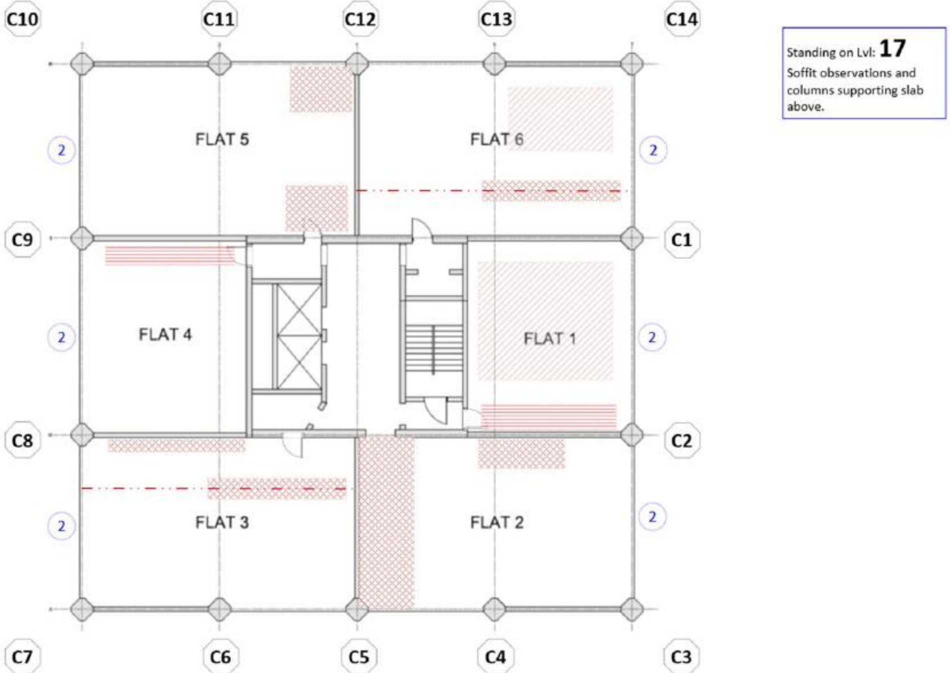

Figure 4. Example from Atkins February 2020 visual survey

77 As can be seen from the above example, the Atkins visual survey provides a more specific record of exactly where the damage on any particular floor occurs, as compared to the overall single classification of damage that has been used by RSK. Then, in Section 5.2.2. of the report Atkins explain that each individual damage zone can be associated with a group of elements from the standard "floorplate finite element mesh" which they show at their Figure 5.3.

78 A decision as to exactly what (reduced) "structural performance[53]" value to associate with each finite element still remains essentially a matter of engineering judgement, but as Atkins point out, correctly in my opinion, at Section 5.1.1. the <u>relative</u> values between different sections of the slab are probably more important than the <u>absolute</u> values in building a reasonably accurate model of the structure. This means that a consistent approach needs to be taken to selecting these values. Nevertheless, when considering the results of any analysis, the absolute values are important, since the absolute values have to be used to assess whether a particular section is overstressed.

[53] For a floor slab finite element, the relevant parameter is the bending rigidity or "EI" value where E = Youngs Modulus, and I = the Second Moment of Area of the cross section. Both E and I are potentially affected by the fire damage.

79 One important feature of the FDV is that the analysis has been carried out on what is termed a "stage" basis, the first stage being a model of the (fire damaged) structure without any propping included.

80 The important topic of lateral stability is dealt with at Section 5.2 "*Global Stability Assessment*" of the report. Here Atkins confirm that the reinforced concrete party walls have been explicitly included in the analysis model, as is shown graphically at their Figure 5.5 which is reproduced below as Figure 5:

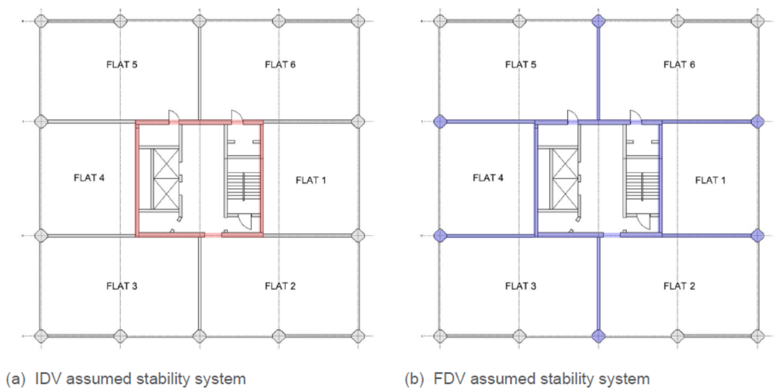

Figure 5. Stability system (above Level 4)

81 In my opinion the FDV model as shown on the right-hand side is correct, and, as such, should give a much "less conservative" result than the IDV[54] model shown on the left-hand side. It is important to note that the contribution of the concrete party walls to the global stability of the building is dependent largely on the effectiveness of the 6 columns[55] on the external perimeter <u>below level 4.</u> Atkins do not actually give any quantitative value for this increase in lateral stability – they merely state (correctly) that since the model assumed for the IDV assessment was shown to be satisfactory, then the model used for the FDV must obviously be satisfactory "by inspection".

82 A question mark appears to exist in respect of how wind loads are transmitted from the external sheeted scaffolding into the existing concrete structure and thence are carried by the stability system of the combined central core and the six party walls. This is set out at Section 5.2.2.2. of the report and appears to be an unresolved issue at the time the report was written, with Atkins stating that this is an issue for the Temporary Works Designer (TWD). I understand that this has now been resolved by the design work carried out by Cantillon.

83 Section 5.3 of the report focusses on the severe damage to Column C10 and conducts a parametric study to investigate whether the damage has likely caused additional loading to be carried by the adjacent columns (particularly C9 and C11) which would then have overloaded them. The assumptions as to the extent of structural damage to C10 at level 13 seem to me to be very

54 IDV is Initial Design Verification see paragraph 62 above.
55 Shown coloured blue on the right-hand diagram at Figure 5.

pessimistic[56] but the results indicate that C9 and C10 are not massively overloaded (see Atkins Figure 8). In my opinion this illustrates the inherent robustness of the reinforced concrete columns on this structure – they are (fortunately) unusually large, very possibility as a result of the design having been revised following the collapse of Ronan Point (see paragraph 6 above).

84 Section 5.4.2 of the report sets out the assumptions as to the reinforcement in the columns. I refer back to my paragraph 61 where I gave my own view on this key consideration. In Section 5.4.2 Atkins make the assumption that all the columns (which are physically the same cross-section throughout the height of the building) are reinforced with a minimum amount (0.8% of the cross-sectional area) of reinforcement throughout the height of the building. I feel that this is a very conservative assumption, although one that is obviously safe to make[57].

85 At Section 5.4.3.1 Atkins state "*The findings are consistent with those of the IDV, namely that at lower levels for the highest loaded of the columns it cannot be shown that the full section, i.e. undamaged by fire, can resist the applied loads. However, this is based on minimum reinforcement and it is likely that the reinforcement amount would have increased at lower levels. Further, it is also noted that the highest loaded columns are also those that support the fire walls*[58] *and as such it is likely that they would have been designed with an increased level of reinforcement*." In my opinion it is almost certainly the case (as I note above) that the minimum reinforcement assessment is very conservative and hence these columns are almost certain to have more reinforcement and thus are almost certain, in practice, to be capable of safely carrying the imposed loads.

86 Section 5.5 "*Slab assessment (gravity loading)*" provides the assessment of what is perhaps the most critically damaged parts of the building, the floor slabs. The damage is unsurprising, in that in a severe fire inside a contained "concrete box"[59], the underside of the roof of the box (i.e. the floor slab at the level above) will inevitably be more affected by heat than any other element of the structure, simply because "heat rises". It will be recalled that the damage to the underside of the floor slabs was identified as a critical issue from the very first inspections carried out in the immediate aftermath of the fire, and that a permanent deflection of up to around 100 to 150mm was evident at numerous floor levels.

87 The basis of the assessment is that

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[56] As a worst case assuming only 0.01% (effectively zero) of the C10 cross section area remains, and even as a best case only 10%.

[57] In theory this is correct. But in the 1960s and 1970s structural designers did sometimes "break the rules" and only provide 0.8% reinforcement to the external strips (say 150 or 200mm wide) of large concrete cross sections – which could result in even less reinforcement in these columns.

[58] (Fire walls = party walls). See paragraph 81 above where I make the same observation.

[59] Treating each apartment as a concrete box with a floor, concrete walls on three of the four sides and a concrete "roof" above.

(i) calculations are carried out to derive bending moments and shear forces in the slab at each floor level, using just the dead load of the structure (see paragraph 70 above);

(ii) this first stage analysis is then used to compare the predicted bending moments and shear forces with the (reduced) moment capacity and shear capacity[60] to see if the structure is capable of supporting its own dead load;

88 In carrying out this first stage assessment Atkins state at 5.5.1 "*Where tension is likely to have been developed at the top of the slabs, e.g. at supports and hogging regions, it has not been possible to assess the resistance to such stresses due to a lack of information on the top reinforcement in the slabs*". I do not understand exactly what affect this has on the results – it is implied that Atkins has assessed that no structural damage to the top steel and the associated concrete has occurred and therefore a full strength has been assumed at these locations. The actual reinforcement sizes are not known, and therefore, as is stated at 5.5.2 "*a conservative assumption has been made of top reinforcement in the slabs as this is currently unknown*". I assume that this is the reason why some areas of overstressing are indicated. Generally, these are adjacent to the locations where the slab itself requires propping to deal with damage to the bottom reinforcement / bottom concrete, and hence the slab will "deal with" the potential overstressing by re-distributing the loads into the propping.

89 Section 6 "*Structural Degradation*" gives a summary of the recommendations by Atkins to prevent further structural weakening, i.e. (i) roof waterproofing; (ii) concrete repairs / reinforcement exposure; and (iii) perimeter protection. I agree with the proposals for (i) and (iii), but in my opinion it is unnecessary to consider carrying out repairs to concrete and / or protection to exposed reinforcement, unless the building is going to be left *insitu* for a number of years.

90 Section 7 "*Conclusions and next steps*" provides a summary of the FDV and suggestions as to what is required going forward. I am satisfied with what is set out here, noting that (as is stressed by Atkins) the FDV is predicated on the assumption that Stage 3 propping (reaching down to basement level) is installed. Since this has not yet been implemented (the current propping is Stage 2 and commences at Ground Floor level, and the initial installation of the Stage 3 propping will commence at Level 4) it is an important point that must be kept in mind at all times.

91 The Appendices to the FDV contain the following information:

Appendix A Fire Damage Survey for each floor slab (see paragraph 76 above)

Appendix B Structural Analysis Output. This comprises pictorial results for slab bending moments (four diagrams for each slab[61]) and slab shear

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[60] The shear capacities in all cases exceeded the shear forces – this is the usual situation in floor slabs, where moment capacity is the critical structural case

[61] Hogging and sagging moments, each in the X and Y directions

forces. The identification of the floors is by "Cassette"[62] and ranges from Cassette 05 to Cassette 24. The presentation contains only the output and so cannot at this stage be checked, but Atkins have confirmed that their standard QA checking / approving procedures have been applied to all calculations presented in the report.

Appendix C Interpretation of analysis. This provides, by a diagram for each floor level, the locations where propping of the floor slabs is required (based on the first stage structural analysis, where no propping is modelled, and where the floor slab cannot safely support its own self weight).

C6 Atkins Report "*Grenfell Tower. Propping Risk Mitigation Study – Part 1: Stage 3 propping from level 4 upwards*" dated 3rd July 2020

92 This document is a technical note prepared to give further data on the Stage 3 propping solution which was assumed to be installed as a prerequisite of the Final Design Validation (FDV) document reviewed at C5 above. The FDV assumption was that a full "down to the basement level" propping system was installed. The current propping layout is termed the Stage 2 propping, and the FDV identified the propping required to provide support to the building, based on a detailed structural analysis of the damaged slabs subjected to a selected (short-term) imposed loading regime.

93 This document provides an assessment of the "partial" installation of the Stage 3 propping layout, which (initially at least) excludes the installation of any propping below Level 4[63].

94 The assessment makes use of the analysis model already described by Atkins and used in the FDV (see C5 above) which had modelled propping down to basement level. For this technical note the overall model simply excludes the props between basement level and the underside of Level 4. As with the FDV analysis, the imposed loading that has been used for the analysis is 6 floors carrying $1.5kN/m^2$ and the remaining 14 floors carrying $0.6kN/m^2$. Here, however, the six heavier loaded floors are floor levels 18 to 23 whereas for the FDV the description is that any 6 consecutive floors can be loaded with the higher value of imposed load (giving 15 options, i.e. floors 4 to 9 inclusive, floors 5 to 10 inclusive, and so on up to 18 to 23 inclusive).

95 It is an established structural engineering principle that a suspended floor in a multi-storey building, which is not itself fully loaded, can be used to provide a degree of support to the deadweight of a floor above. It is this principle which allows formwork to be propped from (say) level X whilst the concrete at level (X+1) is cast. During construction, a newly-cast concrete floor is unlikely to be strong enough[64] to carry the dead weight of the floor above, and hence the propping is carried down to perhaps 2 or 3 floors below in order to spread out the loading.

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[62] See Section 5.1 of the report.

[63] Except for localised propping down to basement floor level around Columns C10 and C11.

[64] Sequential floors are often cast at one- to two-week intervals, but concrete takes around four weeks to reach its nominal strength.

96 The very real complexity of this situation is that the loading that is transferred into the prop depends on (i) the exact installation method (i.e. how "tight" the prop is; it can be inserted to just touch the concrete floor above, or it can be deliberately "preloaded" by screwing it up to a much tighter condition); (ii) the order in which the props are installed (each subsequent prop will relive the earlier nearby props of some load if the props are being preloaded); and (iii) the relative stiffness of the props compared to the structure being propped.

97 Typically, if the props are installed to "just touch" the slab above, then no significant dead load is transferred to the propping system, but the imposed load is carried largely by the propping[65]. However, it is frequently the case (as here) that some of the dead load needs to be carried by the propping as well, which necessitates the preloading referred to above.

98 Atkins have considered the complex and difficult issue of how to analyse the loading actually carried by the propping (see the two paragraphs above) which they describe in terms "*Where the propping is assumed to support a percentage of the dead load, it is assumed that the propping is prestressed (or similar) in order to relieve the stresses in the slabs*". They achieve this by way of a parametric study, looking at three options. These are set out in their Table 1 which I show below:-

Table 1 - Load cases

Construction stage	Load type	Load case 1	Load case 2	Load case 3
1 (no propping)	Dead	75%	50%	25%
2 (props installed)	Dead	25%	50%	75%
2 (props installed)	Live	100%	100%	100%

99 The terminology used here could be misunderstood; (*no propping*) means the proportion of the dead load which remains carried by the slab structure <u>after the props have been installed</u>, and (*props installed*) means the proportion of the dead load carried by the props. The range used in the study (75/25 to 25/75) cannot be considered as a precise limit but, in my opinion, is likely to be a reasonable estimate of the split of loadings between the slabs and the props.

100 As an illustration of the complexity that results from even this simplified analysis, Atkins look at one row of 18 props which stretches across Flat 2 and Flat 3[66] and provide the results at their Figure 2 "*Axial forces in props from Level 4 to Level 24*" for Load Case 2 (see Table 1 in paragraph 98 above). This clearly shows how the load in the props increases (as one would expect) storey by storey downwards until the mid-height storeys, where the load then reduces again as the props effectively become supported by the less-damaged floors from about Level 10 down to Level 4 (the starting level for the slab props).

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[65] This does depend as noted above on the relative stiffness of the propping compared to the existing structure

[66] These are identified at Atkins Figure 3 *Key Plan for Sections* as Section 4-4

101 Appendix A of the report gives full results for all 18 cross sections and covers all props across the whole building[67] but I note that these results, all of which show a similar loading profile to that described at paragraph 100 above) are only for Load Case 2 (50/50 split).

102 The report is described as being only a "*proof of concept*" study that propping, starting at Level 4, is technically possible. Whilst it would, in my opinion, have been very helpful to see the variation in results when Load Case 1 (75/25 split) and Load Case 3 (25/75 split) are used instead, I note that further work on the propping design has now been carried out by the appointed Temporary Works designer, Cantillon (see C11).

103 At Section 4 "*Long-term resilience*" Atkins state that propping from Level 4 upwards should be regarded as a short-term option, due to the inherent uncertainties is relying long-term on an existing (damaged) structure to provide a sound "foundation" for the propping system (these are my words). I agree with this opinion.

C7 Atkins Report "*Summary of Findings*" dated 16th July 2020

104 As the title suggests, this is a summary of the advice already issued by Atkins dealing with (i) the damage to the structural elements of the building; (ii) the current and proposed propping; and (iii) the proposed demolition[68].

105 The detailed information concerning issues (i) and (ii) has already been reviewed at sections C5 and C6 above and will not be repeated here.

106 The report summarises Atkins opinion on the demolition[69] at Section 3.3 "*Considerations relating to start of deconstruction works*" where their advice is "*to carry out deconstruction at the earliest opportunity*". I agree with the reasoning given for this advice as well as the advice itself.

107 Appendix A to the report provides in a convenient format a series of graphic photographs illustrating typical examples of the widespread fire damage to the structural elements of the building, particularly the reinforced concrete slabs.

C8 Atkins Report "*Update on design works*" dated 13th December 2020

108 The design works, referred to here, relate to the design of the propping system, and specifically to the design of the Stage 3 propping system. I note that Atkins are not carrying out the Stage 3 propping design (and did not carry out the Stage 1 or stage 2 propping design either). As such this report provides a description of the propping systems already installed (Stage 1 propping, augmented by Stage 2 propping) and comments on the Stage 3 propping design "by others"[70].

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[67] Nominally 18 prop locations at each cross section, with 18 cross sections – 324 props – but there are no props in the central core, so the actual number 48 less than this (276 prop locations, or 5,520 actual storey height props).

[68] Here termed "deconstruction".

[69] Similar advice is also given at Section 3.1 "*Ongoing risk associated with long-term propping of the structure*"

[70] The Stage 3 propping design is being carried out by Cantillon.

109 The presumed arrangements for the Stage 3 propping are now formalised as being propping from Level 4 upwards only, although it is noted that if the demolition of the building does not commence as the installation of the Stage 3 propping is completed, then the propping will have to be extended down from Level 4 to the basement level.

110 The current engineering design validation for the propped building (as opposed to the propping design itself) is contained in the Atkins FDV Report (see C5 above) dating from June 2020. The FDV is based on Stage 3 propping extending down to basement level; this difference in assumptions needs to be kept in mind when the totality of the engineering advice is being considered, although it is noted that the report reviewed at C6 above does look specifically at the "Level 4 upwards only" propping provision.

111 Atkins note that the Stage 3 propping design is "*currently being completed*" (December 2020) by Cantillon, and is being independently checked by MBP and reviewed by Atkins themselves. It is stated that both MBP and Atkins have "*carried out separate independent analysis to verify Cantillon's work*". In the case of Atkins this is referring to the Report from July 2020 "*Grenfell Tower. Propping Risk Mitigation Study – Part 1: Stage 3 propping from level 4 upwards"* which I have reviewed above at C6. I review the Cantillon Stage 3 propping design (which includes confirmation that an independent check has been carried out by MBP) at section C11 below.

112 At Section 2.4 "*Findings to date*" Atkins summarise what they understand from their reviews of the Cantillon design and from their discussions with both Cantillon and MBP. The conclusion that they reach is that the Cantillon design will provide a satisfactory solution to propping the building on a short term basis (defined as being up to July 2022, when the installation of Stage 3 propping from level 4 to the underside of the roof is planned to be complete) and that this propping will allow for (a defined method of) demolition to take place. Atkins note that they had not (at the time this report was prepared) seen the final design documentation from Cantillon, but I am informed that they have now seen and reviewed this design (see section C11 below).

113 The report explains that the propping design assumes that the last storey of props (from Level 24 to the underside of the roof slab) will not be taken up fully to the underside of the roof slab (approx. 3mm gap to be left). All the other props, below this level, will be installed "hand-tight". Bearing in mind the wide range of potential load sharing between the existing structure and the propping, as assumed by Atkins in their report "*Grenfell Tower. Propping Risk Mitigation Study – Part 1: Stage 3 propping from level 4 upwards*" (see C6 above), it is important to understand that this is how Cantillon have dealt with this design complexity.

114 In Section 3.2 "*Considerations relating to start of deconstruction works*" it is stated "*Given the levels of damage to the primary structure it is not seen as practicable to remediate all or part of the damaged structure to bring it back in to use. As such it is expected that the superstructure, i.e. that part of the building above ground floor, will have to be deconstructed*". I do not recall

seeing this stated so explicitly in any previous reports and the statement is not accompanied by any detailed explanation of why this opinion has been given.

115 However, I agree with this opinion, and the explanation I would give relates primarily to the damage to the floor slabs, where the bottom layer of steel – in numerous locations – is de-bonded from the concrete and exposed, and the slabs have deflected by up to 150mm. In my opinion this cannot be repaired *insitu* in any sensible way that would restore the ability of the reinforced concrete floor slabs to support its own self weight[71]. The concrete floor slabs would therefore have to be removed and re-constructed, which is an incredibly difficult and extremely risky operation to carry out unless the whole building is demolished first.

116 Appendix A to this report contains the same 10 photographs showing typical damage to the existing structure which were at Appendix A of Report "*Summary of Findings*" dated 16th July 2020 (see C7 above) and so will not be commented on here.

C9 Atkins Report "*Summary Note*" dated 22nd February 2021

117 This relatively short report does not provide any new technical information or assessment; it simply (as the title suggests) summarises the "structural engineering view" as it stands at the beginning of 2021.

118 However, what it does provide is some further clarity on the time scales related to the completion of the Stage 3 propping installation[72] and the (anticipated) earliest start date for either a full or a partial demolition.

119 It is now recommended that the Stage 3 propping is installed from the Level 4 upwards during the next year or so, with completion aimed for June 2022. In June 2022 it is further recommended that either (i) the demolition starts in July 2022; or, if not, (ii) the Stage 3 propping is extended down from Level 4 to the basement. A clear preference is expressed for option (i), with option (ii) being considered as a contingency arrangement only.

120 It is noted that the Stage 3 propping design takes account of a proposed method for demolition (and thus should not need to be altered or amended when any demolition work commences, other than to remove the propping progressively as the demolition work is carried out (from the top down).

121 I am in accord with these recommendations, and in particular I consider that demolition should commence as soon as practicable.

C10 RSK Report "*July & December 2020 durability surveys and testing*" dated February 2021[73]

[71] Remedial work to the floor slab could (with great difficulty) be carried out which, when complete, could then support imposed load, but the slab would have to remain propped for its own self weight and would still be seriously deformed with a downward deflection.

[72] The Stage 3 propping design is described at section C11 below – but see paragraphs 111 and 112 above.

[73] I also include here a review of the March 2021 update to this report, see paragraph 132 below.

122 This document records the results of investigations into the corrosion of steel reinforcement[74] at Grenfell Tower, stated to be generated, at least in part, by reports of increased humidity inside the building since the installation of the Monarflex covering to the external scaffolding.

123 Three different techniques have been used to provide data for considering the likely progression of reinforcement corrosion, these being (i) half-cell potential monitoring; (ii) carbonation measurements; and (iii) chemical tests for chloride ion content. The data measurements were taken at 28 test panel locations; 3 locations were designated as control panels and were located in areas undamaged by fire, and the remaining 25 test panels were distributed across the floor levels and across the three main structural elements (slab soffits, walls and columns).

124 Half-cell potential monitoring works on the basis that rusting of reinforcement is an electrochemical reaction. An exposed section of reinforcing bar is connected to a standard half-cell[75] and a reading is taken. These readings are repeated on a grid layout across an area of reinforced concrete and the plotted readings provide an indication of where "corrosion activity" is most likely (the higher the reading the more likely the corrosion).

125 Carbonation measurements are a standard test to determine if rusting corrosion is likely to occur in reinforced concrete. Concrete is a heavily alkaline material (pH 12 to 13) and the electrochemical reaction that leads to rust formation cannot occur in an alkaline environment. However, atmospheric carbon dioxide reacts with concrete and gradually (usually over a period of many years if not decades) neutralises the concrete, which allows rust corrosion to commence. The depth to which the carbonation has penetrated from an exposed concrete surface is a very good indicator of when reinforcement corrosion might start to occur – when the so-called "carbonation depth" exceeds the cover to the reinforcement then the alkaline protection ceases, and if water and oxygen are present, even in quite small quantities[76] then rusting can commence. It is self-evident that concrete exposed to an external environment will be much more vulnerable to rusting reinforcement, as carbonation will be faster, and water / oxygen are more readily available to penetrate the concrete and reach the reinforcement.

126 Chlorides in concrete have a seriously deleterious effect on reinforced concrete, as a high chloride content also allows corrosion to reinforcement[77]. Chlorides can either be introduced into the (wet) concrete mix during construction[78] or they can be taken in to hardened concrete later (for

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[74] The steel reinforcement forms the "reinforcement" element of reinforced concrete.

[75] The RSK tests used a copper / copper sulphate electrode; advice from The Concrete Society is that the preferred reference electrode is silver / silver chloride.

[76] The concrete forms the electrolyte in the chemical reaction.

[77] Chlorides are acidic and hence neutralise the alkalinity of the concrete

[78] During the 1960s and 1970s calcium chloride was often added to concrete to speed up the curing process. It has effectively been banned from use as an additive in concrete since 1977, although small quantities exist naturally in some types of aggregate

example by wetting the concrete with salt-laden water, such as a de-icing liquid).

127 The RSK tests mapped the half-cell potential in each of the 28 test areas and then tested for carbonation and chloride content at the "worst reading spot" in each panel. I understand that it is intended to repeat these tests at quarterly intervals to assess if the corrosion potential is worsening, although the report in fact only provides results for two sets of readings (July 2020 and December 2020)

128 A large number of the half-cell potential readings were below the threshold value where corrosion might be expected to be imminent, which appears to indicate (in my opinion) that, where the reinforcement is still properly embedded in the concrete (even where the concrete has been subjected to fire damage) then some degree of protection from the normal source (alkalinity of the concrete paste) still exists. This conclusion is backed up by the carbonation tests which do not show any location where the depth of carbonation exceeds the concrete cover dimension.

129 The average readings of half cell potential show a slight increase between July 2020 and December 2020. If this is confirmed as a trend by subsequent readings, then this would indicate that the potential for concrete deterioration from corroded reinforcement is increasing with time. Essentially this is bound to be the situation over the medium to long term. Once the rusting process starts it becomes impossible to stop it progressing, since only very small quantities of water and oxygen are then needed for corrosion, that has commenced, to continue[79].

130 Nevertheless, the key point in Grenfell Tower is that there is a substantial proportion of reinforcement (notably on the slab soffit) where the reinforced is already exposed, as the concrete cover spalled off during / after the fire. This reinforcement is already corroding (since it has no protection from concrete alkalinity), and this corrosion cannot now be effectively stopped in an unheated and less than watertight building.

131 This RSK report does not provide any engineering advice in respect of the potential effect of reinforcement corrosion, but it allows structural engineers to understand the background situation which will result in long-term irreversible damage to the structural integrity of the (already damaged) building.

132 I have also reviewed an update to this report (dated 31st March 2021) which provides the results from the repeat quarterly survey carried out in March 2021. This shows that very little has changed as between December 2020 and March 2021.

C11 Cantillon "*Technical Note Grenfell Tower – Design Brief. Stage 3 Propping and its Launch from fourth floor*" dated 12th April 2021

133 This document provides a detailed assessment of the Stage 3 propping and its interaction with the (fire damaged) existing structure. The assessment is (in

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[79]Significantly higher water and oxygen quantities are needed to allow corrosion to commence.

many ways) very similar to that already provided by Atkins in their FDV Report and their PRMS Report (see C5 and C6 above), but I note that it is a document prepared by the firm which is responsible for installing the propping, and it provides a very specific and detailed layout of the actual propping to be installed.

134 As with the Atkins reports noted above, a model is created for the existing fire damaged structure, using the existing survey information from RSK (see C3 above). The Cantillon model assumes that the floor slabs are two-way spanning slabs supported in each case on all four sides (either by the concrete party walls, the core walls, or – around the perimeter edge – the precast concrete spandrel panels[80]). This differs from the Atkins model which assumed the floor slabs were supported at the perimeter edge only by the columns. I consider that it is more likely that the floor slabs are supported by the spandrel panels; the Atkins model is therefore a very useful comparison which is likely to be conservative.[81]

135 The fact that two clearly different analyses and assessments have been carried out, both of which indicate that the Stage 3 propping / damaged structure combination is satisfactory, is a helpful situation in dealing with this complex issue.

136 The Cantillon calculations are signed off as having been subject to a fully independent check by MBP.

137 I note that the Cantillon calculations include specific allowances for a particular method of demolition – a detailed description of which is given. In my opinion the demolition method described is appropriate and is likely to be followed by any appointed demolition contractor, but with the obvious proviso that a demolition contractor will need to be independently satisfied that the work can be carried out safely within the constraints of the Stage 3 propping.

138 The propping sequence is that work starts at 4th floor and progresses on a floor by floor basis up through the building. It is specified that when the propping reaches the underside of roof level then the demolition of the building should commence, otherwise the propping then will be extended downwards from the 4th floor to the basement level. The design is based on the Stage 3 props being installed just hand tight i.e. there is no deliberate preloading of the propping.[82]

139 In my opinion the design and specification of the Stage 3 propping as set out in this document is well ordered and competent.

C12 Miscellaneous other technical documents

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[80] The edge spandrel panels span between the columns, and the floor slabs are cast onto reinforcement starters protruding from the bottom edge of the spandrel panel.

[81] If the analysis shows that the propped structure is satisfactory using the Atkins model, then it is bound to be satisfactory when assessed using the Cantillon model.

[82] It is likely that some dead load transfer will occur as Stage 1 and Stage 2 propping is sequentially removed.

140 I have reviewed the other documents listed in the brief (see Appendix A) which in general comprise factual reports from site inspections and the like. I am not providing any detailed commentary on these documents as they do not provide engineering advice.

John Roberts

Dr J M Roberts

Senior Director of Structural Engineering, Jacobs UK Limited

12th May 2021

Appendix A

Remit for Independent Peer Review of Engineering Advice

a) Introduction and Purpose

The fire at Grenfell Tower in June 2017 resulted in a tragic loss of life and caused extensive damage to the building. Since this time a number of measures have been taken to monitor the building's condition and assure its ongoing stability and safety with several recommendations made about the future of the Tower, about which no decision has yet been made. This has been led by the Dangerous Structures Surveyor and MHCLG's appointed engineers and supported by a number of specialist firms. This body of work is collectively referred to as "engineering advice". Further background is included in Annex A to this document.

MHCLG is looking to appoint an independent engineering expert to undertake a peer review as an independent assessment of the engineering advice provided to date and offer a second opinion if this advice is robust, sound and well founded.

In addition, certain designs and calculations have been undertaken to assess and assure the current and ongoing stability of the Tower. The peer review will also consider if the basis of this work is sound. This may involve inspection of various calculations undertaken to date, however this review is not intended as a detailed design check but an assessment of the overall methodology and conclusions drawn.

b) Scope of Review

The consultant is to use its own discretion in determining the methodology to be adopted but, by way of guidance, the scope of activities is likely to include some or all of the following:

1) A site visit and inspection of Grenfell Tower including a review of the management, maintenance and inspection procedures in place.

2) Documentary evidence review of items listed in Annex A, which includes:

- Structural analysis reports and supporting information
- Inspection reports, tests and results
- Historical data including building movement monitoring humidity and other such information
- Inspection of design calculations and checking procedures
- Review of programme timelines and other such information that may inform your recommendations

3) Interviews with project team members including the Dangerous Structures Surveyor, engineering consultants, MHCLG staff, contractors, specialist consultants and suppliers and other relevant individuals

c) Management of the Review.

It is anticipated that the review will be undertaken by a small team of relevant experts led by a senior member who will be the main point of contact with MHCLG, reporting directly to the Deputy Director for Site Management in MHCLG.

All staff engaged on the project must have relevant qualifications and credentials to undertake the work. In addition, regular progress updates will take place during the review to discuss emerging findings with the SRO.

d) Deliverables

The deliverable for this commission will be a concise report which will include as a minimum:

- An Executive Summary
- A brief overview of the review methodology
- Key findings
- An opinion regarding the advice and recommendations provided by the engineering consultants to date
- Any other relevant conclusions and recommendations

e) Timescale

This review must be undertaken within a relatively short time frame, but without compromising thoroughness and rigour. The draft report should be made available within two weeks of appointment, with the final report issued no later than three weeks from appointment following review by MHCLG.

f) Conflict of Interest

The service provider is required to declare any previous engagement with Grenfell Site and Programme and any previous or on-going engagement with Engineers and Specialist firms being the subject matter of review. Where a conflict of interest exists or may exist or arise during the service provision, this should be notified to MHCLG as soon as the service provider becomes aware of it. Transparency is key in the event that conflict of interest cannot be avoided entirely. The service provider is required to submit a written statement on how it proposes to avoid or manage both perceived and actual conflicts.

ANNEX A

Report	Author	Date
Summary Report - Structural and Fire Considerations	Arup	June 2017
Structural Feasibility Report	MBP	March 2019
Initial Design Validation	Atkins	July 2019
Final Design Validation	Atkins	June 2020
Summary of Findings	Atkins	July 2020
Letter from Harrow District Surveyor (Dangerous Structures Engineer)	District Surveyor	Aug 2020
MBP Letter	MBP	Aug 2020
Safety Inspection Reports	DeRisk	Monthly since 2017
Safety Inspection Reports	RSK	Monthly since December 2020
Asbestos Survey	DeRisk	Feb 2019
Executive Asbestos Summary	DeRisk	May 2019
Weekly Site Support Records	Atkins	Weekly since Sept 2020
Summary Note	Atkins	Feb 2021
Condensation Mitigation Note	Atkins	Feb 2021
Condition survey and leak detection survey	RSK	Feb 2021

Appendix B

Jacobs

Dr John Maxwell Roberts

Senior Director Structural Engineering / Chartered Engineer

Personal Details

Length of service in the profession: 47 years

Year joined Jacobs: 1981

Jacobs office location: Manchester

Summary Biography

John is one of the UK's leading structural engineers with over 45 years of experience in major building projects. He was President of the Institution of Structural Engineers in 1999 to 2000 and has held a number of other appointments in the profession. He has served as Chairman of the Joint Board of Moderators which approves the standards of university engineering courses. He has experience of acting as an advisor to various government bodies (including giving evidence to a select committee), as well acting as an expert in commercial disputes and criminal legal cases.

John has wide ranging technical expertise in structural works in both new construction (and particularly in large steelwork structures) and in refurbishment and restoration of existing buildings. He has specialist knowledge of dynamic loadings, structural vibration issues, and fatigue damage. He has written a number of published articles and contributed to a well-known textbook on structural design. He also has specialist knowledge of construction contracts and procurement options.

Key Skills/Areas of Expertise

- Structural engineering design of new buildings
- Assessment and re-use of existing buildings
- Theme park rides and attractions
- Cable driven light train systems
- Cable car systems

Education, Qualifications, Registrations and Certifications

- Bachelor of Engineering (BEng) (1st Class Honours), Sheffield University, 1969
- Doctor of Philosophy (PhD), University of Sheffield, 1972
- DEng Honorary Doctorate, University of Sheffield, 2006
- DTech Honorary Doctorate, University of Brighton, 2008

Memberships and Affiliations

- Fellow of the Royal Academy of Engineering (FREng), 1995 to Present
- Fellow of the Institution of Civil Engineers (FICE), 1989 to Present
- Fellow of the Institution of Structural Engineers (FIStructE), 1989 to Present
- Visiting Chair in Principles of Engineering Design, University of Manchester, 2003 to Present
- Royal Academy Visiting Professor in Engineering Design, University of Manchester, 1995 to 2003
- Chairman, Joint Board of Moderators on behalf of ICE and IStructE, 2010 to 2014
- Former Member of Council BCSA (British Constructional Steelwork Association), 2000 to 2007
- Member of Council and Executive Committee, Steel Construction Institute, 1995 to 2007
- President, Institution of Structural Engineers, 1999-2000
- Chairman, Structural Engineers Registration (SER) Limited (a not-for-profit company set up by ICE and IStructE), 2011 to 2018
- Chairman, Steel Construction QA Scheme Ltd, 1994-1999

- Chairman of Trustee Board, Institution of Structural Engineers Pension Scheme, 2013 to date

Achievements/Awards

- Supreme Award (for British Airways i360) The Structural Awards, 2017
- Special Award (for the London Eye) The Structural Awards, 2001
- Special Award (for the Pepsi Max Big One at Blackpool Pleasure Beach) The Structural Awards 1995
- Gold Medal, Institution of Structural Engineers, 2005
- James Forrest Medal Institution of Civil Engineers, ICE, 1973
- The Lancashire and Cheshire Branch Prize Institution of Structural Engineers, 1984/85 and 1988/89
- The Sir Arnold Waters Medal, Institution of Structural Engineers, 1984/85
- Derrington Construction Medal, Institution of Structural Engineers, 1998/99
- British Iron & Steel Prize, University of Sheffield, 1969
- Mappin Medal, University of Sheffield, 1969

Languages

- English (mother tongue)
- French (basic)
- German (basic)

Employment History

- 1981 to Present, Jacobs
- 1974 to 1981, Bertram Done & Partners
- 1972 to 1974, Sir Alfred McAlpine & Sons Limited

Published Papers

- "*The behaviour of steel structures under impact loads*", Institution of Civil Engineers Associate Members & Student Paper Competition 1971/72.
- "*The response of steel struts to impact overload*", PhD Thesis, University of Sheffield, 1972
- "*Structural Work at Rihand Power Station, India*", Institution of Structural Engineers, Lancashire & Cheshire Branch, April 1985.
- "*Alton Towers Monorail" (a "fast track" fast track?)*" Journal of the Institution of Structural Engineers, Vol. 68 No.1 Jan. 1990.
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- "*Fast and Furious: Fairground, of the Future*" Royal Academy of Engineering lecture at the Annual Meeting of the British Association for the Advancement of Science; September 1977.
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Other Information

John regularly gives talks and delivers lectures to both university students (Manchester, Sheffield, Brighton, Leeds, Durham, Bath) and professional civil and structural engineers in the UK and internationally