FMS

Mater Dei Hospital | Malta

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MAU building executive summary

The MAU building at Mater Dei Hospital, Malta is a reinforced concrete structure, rectilinear on plan. The lateral stability of the building is provided by shear cores and moment frame. The building was built and designed in phases. The first phase of the building was constructed in 1996, the second phase of the building (additional floors) were designed and constructed prior to 2002. The contractor for both phases was Skanska Malta JV.

The vertical extension of the MAU building at Mater Dei Hospital was proposed as a suitable solution to the growth of the hospital. As part of the due diligence for the extension of the MAU building concrete cores were taken from the most critically loaded elements the columns. These core compressive results came back lower than anticipated.

Arup were tasked with reviewing the existing core results, carrying out further testing via an independent third party (CRL), reviewing the impact on the structure of the reduced concrete strength, and reviewing the structure under seismic conditions.

The original design characteristic cube strength of the MAU building is 30MPa. Evaluating the concrete core data available and reviewing all relevant standards and technical guidance, a characteristic cube strength used for evaluating the building of 18MPa is deemed appropriate. Whilst no specific checks have been made, by inspection and engineering judgement the addition of floors to the MAU is not considered feasible and hence not recommended.

The gravity assessment of the building assuming the measured characteristic cube strength of 18MPa highlighted that there are 21 columns and 25 beams that do not comply with the ultimate limit state criteria, and are currently working with a reduced factor of safety. Strengthening works to those columns and beams are required.

The MAU building has been reviewed against the original design intent for the pre-2007 extension with a PGA=0.06g ($0.6 m/s^2$). Over 50% of the columns fail the seismic checks. Remedial works are required to ensure compliance. The remedial works required are extensive and a number of options have been assessed to determine a cost effective and practical solution noting the critical on-going operational nature of the building.

Several columns in level 7 of D1.1 require remedial works due to spalled concrete which is affecting the strength and durability of the columns. The columns need repair works to prevent further deterioration of the columns.

The remedial works are expected to be in the form of additional shear walls, reduction of plantroom loads and bracing to stabilise the platroom roof structure to deal with seismic loads.

Further investigation is required into the carbonation levels of the concrete, the fixing condition between the blockwork walls and the structure.

Contents

Appendices

1 Introduction

The Mater Dei Hospital is the main public hospital for the people of Malta, located in Msida. The hospital is the designated trauma centre for the islands and is an acute general teaching hospital. This report relates to the Medical Assessment Unit (MAU) building.

Figure 1 Plan view of hospital

In mid-2014 FMS (Foundation Medical services) proposed the extension on the existing MAU building by at least an additional storey. To determine if the extension was structurally viable, concrete core testing was carried out on a select few columns. The results came back lower than expected, raising concerns about the integrity of the structure. Additional core testing was carried out by alternative testing houses, with inconclusive results.

Arup were commissioned by FMS in September 2014 to carry out further testing of the concrete and provide advice on the structural condition of the building. This report describes our findings.

2 Scope

The structural review was split into four phases. This report covers Phases 1-3.

Phase 1: MAU Critical Safety Review
Phase 2: MAU Concrete and Geotechnical Testing
Phase 3: MAU Analysis and Recommendations

Phase 1 Critical Safety Review

Phase 1 of the review was a high level review of the existing design drawings and calculations, concrete test reports, and construction records. A site visit was carried out by Edward Hoare and Richard Hill. A memo was issued and can be found in the appendices to this report.

Phase 2 Concrete and Geotechnical Testing

Phase 2 of the review focussed on concrete testing. The scope of the testing work was agreed with FMS and the testing was supervised by Richard Hill and Natalie Dumbrell. We reviewed the results of the tests together with the available information from previous tests. The outcome of that review was a proposed figure for concrete strength, which could be used in Phase 3.

Phase 3 Analysis and Recommendations

Phase 3 of the review was to carry out analysis of the MAU building structure. The review includes an assessment of the structure under both gravity and seismic loads, assuming:

1. The original concrete design strength;
2. A reduced concrete design strength as proposed in Phase 2.

3 History of Mater Dei Hospital

To help understand the existing structure, a review of its construction history was undertaken. The objective was to identify changes to the design team and design brief, to help with interpreting the construction documentation. The following is a brief summary, but a more detailed version is included in the appendices:

The same contractor, Skanska MJV, was used throughout the history of the main hospital between 1995 and 2004. There were several structural engineers involved over that time.

Ortesa Spa started design in 1993 and had their contract terminated in mid- to late-1996.

It is unclear who the responsible structural engineers were, between 1996 and 2000. New designers (Norman and Dawburn) were appointed in July 1998 and had their contract terminated in October 1998. In 2000 Skanska MJV were appointed to be the main contractor on a design and build contract and it is assumed that they appointed structural engineers as necessary.

4 Document review

We have reviewed construction information supplied by FMS. Although our review was extensive, the volume of information involved means that not all potentially-relevant documentation was received or reviewed. There was enough information to obtain an understanding of the construction information to inform our review of the structure.

4.1 Drawings

The following drawings were reviewed, though none were complete sets:

- ‘As Built’ structural drawings for the extension of block D1 in 2007. Included details of connections between new and existing structures.
- ‘For Construction’ structural drawings for block D1. Included some reinforcement details.
- ‘Pre-construction’ structural drawings for block D1. We believe that these were superseded by drawings from the ‘For Construction’ set.

Table 1 Summary of design information

Element	Dimensions?	Reinforcement details?	Comment
Foundations	Yes	Yes	Details for 1996 construction Details for 2007 construction
Columns	Yes	Yes	Details for 1996 construction Details for 2007 construction
Slabs/Predalles	Yes	Yes	Shop drawings not present Some details for 1996 construction – obtained through email documentation. Details for 2007 construction
Beams	Yes	Yes	Details for 1996 construction Details for 2007 construction
Walls	Yes	Yes	Details for 1996 construction Details for 2007 construction

4.2 Calculation reports

We have reviewed a calculation report for block D1.3. The majority of the report was output from a computer programme (SAP 90). There was some information on column, beam and slab design.

“Design Parameters for Oncology to cater for seismic actions” dated 17/11/2014 produced by Dr Pierre Farrugia, Perit.

4.3 Construction information

The following information relating to construction processes was reviewed:

- Skanska pour dates: various dates from April 1996 to September 1996

- 5 no. monthly reports by Skanska: November 1995 to March 1996. Included site progress photos, weather records and site activity records.
- Concrete cube test certificates: dated between 1996 and 2000. These showed the general location and elements poured.
- Letters between Skanska MJV and FMS discussing placed concrete.
- Mix designs for Mixer, tal-Maghab and Blokrete, all dated 1995/1996.
- Site photos, taken during construction.

4.4 Concrete core testing reports

The following reports relating to concrete core tests were reviewed:

- Celtest: Certificates dated 12 August 2014;
- Terracore: Interpretative report for testing conducted on concrete samples, dated 25 August 2014;
- Solidbase Laboratory Ltd: Core test assessment for concrete columns at level 8, dated 31 August 2014;
- Innovative Architectural Structures: *Analysis of Concrete Structure Beneath Proposed Extension*, dated 11 September 2014;
- Terracore: Laboratory test certificates for concrete columns, identifying the cover to reinforcement.
- Sandberg: Arup-commissioned concrete core test certificates, dated 31 October and 10 November 2014.

4.5 Geotechnical information

The following geotechnical information has been used to determine the site classification for the seismic review.

- Uniaxial compressive strength tests dated Sept/Oct 1994 from Harrison and Company; and
- *Revised borehole logs – made ground investigation* dated 29/11/94 from D.Duca on behalf of Harrison and Company;

5 Existing structure

From the history and document reviews, we established how we believe the MAU building structure was developed. Our understanding of the sequence, and related aspects of the construction, is presented here.

5.1 Fundamentals

The MAU building houses the emergency department, hyperbaric chamber, and Medical Assessment Unit. It is rectilinear on plan, measuring approximately 40x65m. There are 4 no. storeys referred to as levels 8, 9, 10 and 11. Level 8 is the basement. Level 11 is the roof, which includes 2 no. plant rooms of similar construction to the main building that cover a significant area of the roof.

The building is of reinforced concrete construction, comprising precast concrete units (“predalles”) with an in situ concrete topping. The predalles were installed as 60mm thick flat plates, to which void-formers were added before in situ concrete was placed, bringing the total thickness to 400mm (levels 8-10) or 480mm (level 11 and plant room roof).

There are in situ reinforced concrete beams running between columns in the primary direction with perimeter beams. The beams are the same depth as the predalles and provide an in situ connection between the beams and the slabs. The columns are generally 450mm square throughout the building, founded on shallow pad footings bearing on rock.

5.2 Construction history

The building on the design drawings is divided into two sections referenced as D1.1 and D1.3. Construction appears to have taken place in a phased manner for both buildings, as discussed below.

The first phase was constructed between 1995 and 1998 by Skanska MJV. It is unclear what the exact dates for the construction of the second phase were. It is believed to have been built between 2002 and 2007. The second phase contractor is assumed to be Skanska JV as there are 'As Built drawings' for the second phase of construction with a Skanska stamp on.

In 1996, construction of D1.1 levels 8, 9 and 10 commenced over part of the footprint, between gridlines 01/D1.1 and E/D1.1 (shown green in Figure 2). The remainder of D1.1 was built between 2002 and 2007.

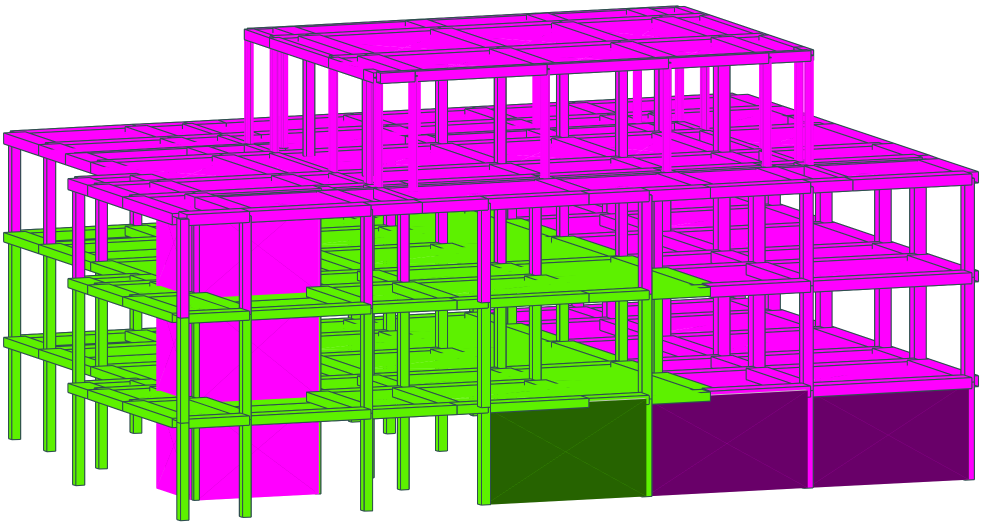


Figure 2 Block D1.1 construction sequence. Green: commenced in 1996. Pink: completed by 2007.

D1.3 Level 8, 9 and 10 were built in one continuous operation in 1996 (shown pink in Figure 3). By 2007, Level 11 (including two plant rooms), separate stair and lift cores as well as some minor structural amendments were complete.

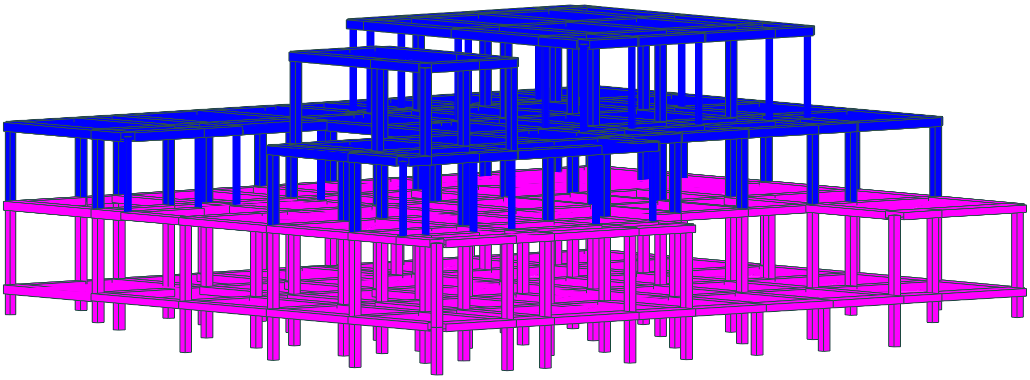


Figure 3 Block D1.3 assumed construction sequence. Pink: commenced in 1996. Blue: completed by 2007. Lift and stair cores not shown.

5.2.1 Design history

The design of the first phase of the building was by Ortesa Spa, an Italian-based company. The design was carried out in accordance with Italian Standards.

The second phase of the building was designed in accordance with German Standards. It was carried out under a design and build contract with Skanska MJV. We did not find any documentation showing who the designers were.

5.2.2 Concrete suppliers

There were four concrete suppliers for the main hospital building:

- Blokrete
- Tal-Maghtab
- Mixer Ltd. (formerly Planka Ltd.)
- Devlands

There is evidence from the concrete cube crushing test certificates that at least two of the suppliers (Devlands and Blokrete) supplied concrete to blocks D1.1 and D1.3. The records are incomplete, so other suppliers might have been used.

5.2.3 Concrete mix design

We found records for three of the four concrete suppliers for 30MPa cube characteristic strength. Table 2 summarises the three concrete mix designs, all of which show water:cement ratios between 0.50 and 0.55.

Table 2 Concrete mix design for C30 concrete (cube) – main hospital

	Blokrete	**Tal-Maghtab**	**Mixer Ltd.**
Aggregate crushing value	32.4	30.4	26-30
Cement (kg/m³)	345	380	350
Water (l/m³)	172	210	170
Sand (kg/m³)	712	670	800
Aggregate kg/m³	1069	1670	1070

6 Original design

6.1.1 First phase

The document review indicated that the Standards used in the first phase of design were:

- Decreto Ministero Lavori Pubblici Italiano 14-02-1992 Technical specifications for the execution of structures in normal and prestressed reinforced concrete and steel structure)
- Decreto Ministero Lavori Pubblici Italiano 24-01-1986 Technical specifications relative to buildings in seismic regions
- Decreto Ministero Lavori Pubblici Italiano 12-02-1982 Special criteria for safety verification of building and load and overloads
- Decreto Ministero Lavori Pubblici Italiano 11-0301988. Technical specifications regarding design and foundations execution.
- UNI 9502 specifications. Analytical procedure for the evaluation of fire resistance of normal or prestressed reinforced concrete elements

6.1.2 Second phase

The second phase of the MAU building was based on a tender which referenced German Standards (DIN 4149 for Zone 2). We did not find any further information on the Standards used.

6.2 Gravity loading

Table 3 shows the loading used in the design of the first phase of the MAU building. We did not find any mention of the treatment of facades or partitions, nor of specific loading for the plant room, in the calculation report.

Table 3 First phase loading

Level	Superimposed Dead Load (kPa)	Imposed Load (kPa)
Level 11, 12	No information	No information
Level 10	3.5	1.0
Level 9	3.0	3.5

It should be noted that the original design of Level 10 was for an imposed load of 1.0kPa, which is typically an access and maintenance load. This level is now occupied by the operational parts of the hospital, where an imposed load of 3.5kPa would be consistent with the allowance for level 9.

The calculation report states that concrete density of 24kN/m³ was to be used in self-weight calculations. 24kN/m³ is the typical value for the density of concrete.

6.3 Seismic loading

The first phase of the building was designed to Italian Standards. The reference peak ground acceleration (PGA, a_{gR}) on rock was taken as 0.04g, with an Importance Factor γ_I=1.4, leading to a design a_g = 0.04g x 1.4 = 0.056g.

The second phase of the building was designed to German Standards, the PGA was taken as 0.6m/s²=0.061g with an Importance Factor γ_I=1.4, leading to a design a_g = 0.06g x 1.4=0.084g.

A more detailed discussion of seismic variables can be found in section 9.2.

6.4 Design material parameters

6.4.1 Concrete

The concrete characteristic cube strength used in the design of columns, beams and in situ slab elements was f_{cu} = 30MPa. The precast portion of the predalles design was based on f_{cu} = 35MPa.

6.4.2 Reinforcing steel

The yield strength of the steel was f_{yk} = 440MPa.

6.5 Geotechnical parameters

From the review of the geotechnical information provided, the ground condition used in the seismic assessment is type A rock.

7 Existing condition

7.1 Construction

Opening-up works were carried out in several areas, exposing the reinforcement. A Hilti Ferro Scanner was also used in several locations to check the spacing and size of bars.

Our observations matched the reinforcement shown on the drawings. Although these observations were limited, they provide a degree of confidence that the concrete was built in accordance with the design intent.

7.2 Condition

7.2.1 Concrete

In the service tunnel to block D1.1 evidence of previous concrete repairs was found. Those repairs had not been effective in stopping corrosion of the reinforcement. As a result, corrosion of the reinforcement had continued, leading to spalling of the repair mortar and a progressive reduction of column capacity.

Figure 4 Areas of corroded reinforcement

7.2.2 Masonry

Figure 5 cracking in brickwork around plant room D1.3

Minor cracking was observed to some external masonry walls, particularly to plant rooms at roof level. The cracks are in keeping with movement-related cracking (i.e. frame deflections and thermal movements) and we do not believe that they are a structural concern for the wall.

7.3 Concrete strength

7.3.1 Core test history

Prior to Arup's involvement FMS (via the consultant Innovative Architectural Structures, iAS) commissioned three testing companies to carry out compression strength tests on the columns in the MAU.

It is understood that Terracore were the initial testing company used to test the columns in the MAU however the high variability in the results lead to the appointment of two other firms to understand if the Terracore results were correct or whether the variance was a result of testing procedure or apparatus used. The subsequent firms were Solidbase Laboratory Ltd - a Maltese company and Celtest - a UK based company.

The Solidbase results were in general higher than those reported by Terracore and the Celtest results were considerably lower.

In September 2014 FMS appointed Arup to undertake concrete testing to determine the following;

- Carry out independent core strength testing on the MAU.
- Evaluate the existing test results from Terracore, Solidbase and Celtest to determine if any of the results can be relied upon.

In order to independently evaluate the concrete strength Arup appointed Concrete Repairs Limited (CRL) to carry out site sampling and Sandberg to undertake the laboratory tests. As a result Arup have been able to trace the samples from start to finish.

7.3.1.1 Arup results – compressive strength

22 number 69mm diameter concrete cores were taken and prepared and tested in accordance with EN 12504 and EN 12390. The cores were tested in a dry condition. The interpretation of results is in accordance with EN 13791.

The maximum core strength was 47.0MPa, the minimum core strength was 22.4MPa and the characteristic core strength was 21.0MPa (using EN 13791, approach A). Converting this to a characteristic cube strength used for evaluating the building gives 25MPa.

Table 4 Sandberg compressive core test results

Building	Level	Element	Grid Reference		Company	Corrected in situ core strength (MPa)
D1.3	8	column	4	B	Sandberg	22.4
D1.3	9	column	1	E	Sandberg	22.9
D1.3	8	column	5	C	Sandberg	24.0
D1.3	8	column	4	A	Sandberg	26.5
D1.1	8	column	3	C	Sandberg	27.8
D1.3	8	slab	8	E	Sandberg	28.0
D1.1	8	column	2	C	Sandberg	28.1
D1.3	8	column	5	D	Sandberg	28.2
D1.1	8	column	4	C	Sandberg	28.3
D1.3	9	column	9	D	Sandberg	32.0
D1.3	8	column	6	E	Sandberg	32.8
D1.1	8	column	5	B	Sandberg	33.3
D1.3	8	column	5	C	Sandberg	34.1
D1.3	9	column	8	D	Sandberg	34.6
D1.1	9	column	3	E	Sandberg	34.8
D1.3	8	column	3	C	Sandberg	35.7
D1.3	8	column	3	C	Sandberg	38.6
D1.3	8	column	7	D	Sandberg	38.6
D1.3	8	slab	8	E	Sandberg	40.7
D1.1	9	wall	4	D.5	Sandberg	42.0
D1.1	9	column	3	D	Sandberg	44.4
D1.1	9	slab	3	D.5	Sandberg	47.0

7.3.1.2 Terracore

According to the iAS report Terracore tested 95 no. cores, although only 13 no. completed test certificates (for cores tested in a dry condition) were included in the Terracore report provided (report ref J2085, dated 25 August 2014). The cores were tested in a both a wet and dry condition and the diameters ranged between 58 and 64mm.

For the 13 core samples with certificates, the maximum core strength was 36.0MPa, the minimum core strength was 17.7MPa and the characteristic core strength was 21.7MPa (using EN 13791, approach B because less than 15 cores are available). Converting this to a characteristic cube strength used for evaluating the building gives 26MPa.

However the remaining 82 cores cannot be fully ignored. The maximum core strength from all of Terracores tests was 42.9MPa, the minimum core strength was 7.2MPa and the characteristic core strength was 11.2MPa (using EN 13791, approach A). Converting this to a characteristic cube strength used for evaluating the building gives 13MPa.

It is clear from reviewing the 13 certificates that the moisture content of the core is significant and the saturated cores are typically 70% the strength of the air dried cores. Ordinarily the expected decrease in strength between wet and dry cores is 10-15%.

Without the test certificates for the remaining 82 cores it is not possible to determine which cores were tested dry and which were tested wet.

Looking at columns where cores were taken by both Arup and Terracore there is poor correlation. At locations where cores were taken by Arup, Terracore and Solidbase. There was good correlation between Arup and Solidbase results and not with the Terracore results. Without the Terracore certificates is it not possible to determine whether the variance in the Terracore results are a result of testing procedure or apparatus used.

It is not possible to determine which of the Terracore results can be used in conjunction with the Arup results and as such it is not considered appropriate to incorporate the Terracore results with the Arup results in order to determine a characteristic cube strength used for evaluating the building.

7.3.1.3 Solidbase

Solidbase carried out testing on 30 no. cores in August 2014. The cores were tested in a dry condition and the diameters ranged between 58 and 64mm. Test certificates for all cores were provided in their report “Report No: 18”, dated 31 August 2014.

The maximum core strength was 43.5MPa, the minimum core strength was 17.3MPa and the characteristic core strength was 15.0MPa (using EN 13791, approach A). Converting this to a characteristic cube strength used for evaluating the building gives 17MPa.

The certificates provide the appropriate detail to enable ‘like-for-like’ comparison with the Arup results. Looking at columns where cores were taken by both parties there is good correlation. As such it is considered appropriate to incorporate the Solidbase results with the Arup results in order to determine a characteristic cube strength used for evaluating the building.

7.3.1.4 Celtest

Celtest carried out testing on 7 no. 63mm diameter cores in August 2014. The cores were tested in a saturated condition. The iAS report compares the Celtest results with results from Terracore for the same columns (4 No. locations).

The Celtest results vary between 47%-66% of the Terracore strengths. This variance between wet and dry samples is similar to that established in Terracore's own tests.

The Arup and Solidbase tests are based on dry samples and as such the Celtest results cannot be combined.

7.3.2 Characteristic compressive strength

As discussed in section 7.3.1, the unknowns surrounding the Terracore results, and their poor correlation with Arup and Solidbase results make then unsuitable for incorporation with the Arup results.

There is good correlation between Arup and Solidbase results. Combining the two sets of results the characteristic cube strength used for evaluating the building gives 20MPa (using EN 13791, approach A for a total of 52 cores).

The approach in EN 13791 determines the characteristic cube strength by determining the mean (average) of the results and subtracting the standard deviation of the sample with a multiplier (1.48). Characteristic strengths in Eurocodes are based around the statistical reasoning that 95% of the material used in the building will have a strength above the characteristic value. For an infinite number of values standard statistical tables would recommend taking a multiplier of 1.64 on the standard deviation and not 1.48 as used in EN 13791. This is recognised in BS 6089 which is the UK complementary guidance to EN 13791. In BS 6089 for populations greater than 121 cores a multiplier of 1.64 is used for lesser numbers of cores the multiplier values in line with standard statistical tables.

For the 52 number Arup & Solidbase cores a multiplier of 1.7 should be used. This results in a characteristic cube strength of 18MPa.

Evaluating the data available and reviewing all relevant standards and technical guidance a characteristic cube strength used for evaluating the building gives 18MPa is deemed to be appropriate.

This value is clearly less than the specified design characteristic strength of 30MPa. The impact that the lower strength concrete has on the performance of the MAU is discussed in the subsequent chapters.

The subsequent chapters evaluate the building in its current form. Whilst no specific checks have been made looking at additional floors by inspection and engineering judgement the addition of floors to the MAU is not recommended.

7.3.3 Observations of the concrete matrix

7.3.3.1 Petrographic results

The petrographic examination of the concrete samples (taken by Arup and CRL) revealed that the concrete comprised a portland cement based binder with limestone aggregate and limestone fines and possibly limestone powder cement replacement. The high limestone content explains the white appearance of the concrete.

The concrete typically exhibits good compaction with negligible to few voids up to 5mm and commonly less than 2mm across. The estimated water : cement ratio was 0.6 (by comparison to a reference specimen). This is slightly higher than the specified design water : cement ration of 0.5 taken from original design mix records.

From the petrographic examination it was noted that the concrete appeared generally porous. Site observations support this view.

7.3.3.2 Carbonation

The iAS report presents carbonation readings taken from 10 columns in Level 8 of block D1.3. Seven of the ten locations had carbonation in excess of 40mm.

The Arup/ CRL tests in October 2014 similarly showed that in a significant number of test locations the carbonation depth was well advanced and had reached the depth of the steel reinforcement.

The carbonation can be clearly seen by a change in colour of the concrete, as shown in Figure 6 below.

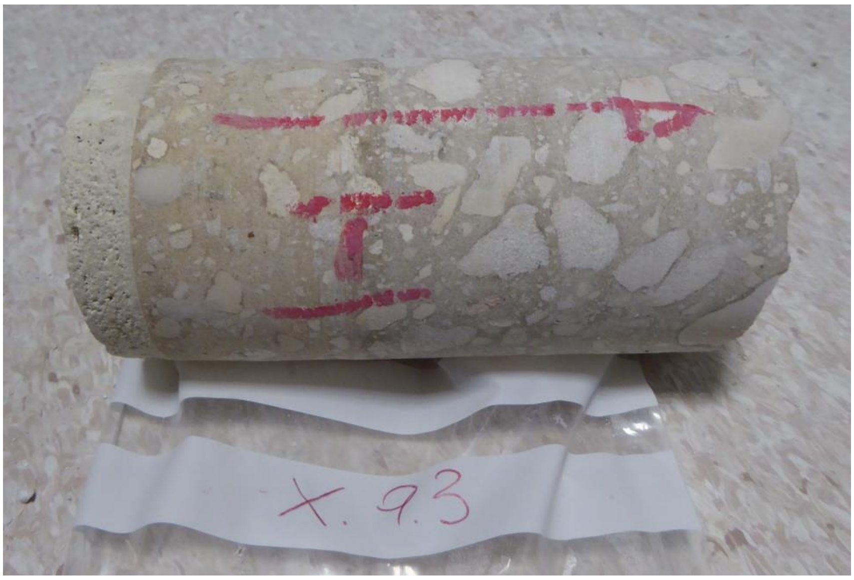


Figure 6 Concrete core from level 9 from MAU showing change in colour

This level of carbonation is considered high for good quality concrete of this age. Typically for concrete 20 years old carbonation would be expected to be less than 20mm.

High levels of carbonation can be attributed to the following;

- Higher water : cement ratio
- Poor compaction
- Porous materials

The consequence of the advanced carbonation is the increased risk of corrosion to the reinforcement in the concrete elements. This is particularly important in the Level 8 areas and Level 11 plant rooms where the columns are in high humidity environments. In level 9 and 10 the columns are typically plastered and are within the climate controlled conditions of the hospital wards and operating theatres, here the humidity is unlikely to reach levels high enough to initiate significant corrosion.

There is no direct correlation to rate of carbonation and strength. However, low strength concrete typically has higher rates of carbonation but these are for the three reasons mentioned above. There is therefore a secondary link between strength and carbonation.

It is not accurate to say that as concrete carbonates it loses strength. To the contrary the carbonated surface of concrete exhibits a higher compressive strength than the uncarbonated concrete within the depth of the element.

7.3.3.3 Aggregate type

It is understood that concrete in Malta should contain only Tal-Qawwi aggregates. Visual inspection of the cores identified some inclusion of Tal-Franka. The figure below highlights the aggregate consistent with Tal-Franka.

This may contribute to the low strength observed in the cores. The construction documentation includes mix designs and cube tests from 1996-2000. It is not known if the cubes tested included Tal-Franka aggregate, however if it is believed that the cubes were made of the same concrete supplied to the columns then the cube crushing results would reflect a mix containing Tal-Franka.

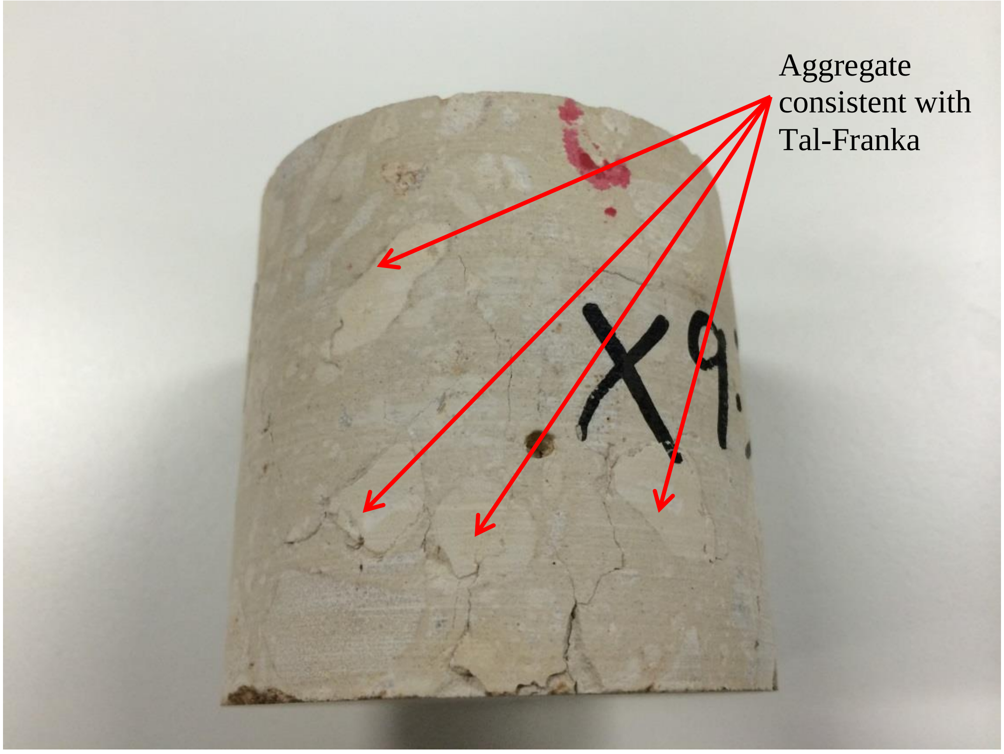


Figure 7 Core showing aggregate consistent with Tal-Franka

7.3.3.4 Chlorides

Chlorides represent a risk to reinforced concrete structures because they have the potential to break down the protective passivation layer that surrounds the reinforcement, causing it to corrode. Chlorides can be present internally in the concrete or they can migrate into the concrete from external sources.

The chloride results were all negligible to low which implies that corrosion due to chloride in the concrete is very unlikely to occur.

7.3.3.5 Summary of concrete properties

The characteristic strength as determined by core sampling (18MPa) is significantly below the specified design strength (30MPa). This is most likely due to poor construction (Aggregate type, water : cement ratio, poor compaction) and is not in Arup's opinion a result of any deterioration over time.

The impact of strength is discussed in the subsequent chapters.

The durability of the structure is at risk due to the quality of the concrete. In particular columns in high humidity environments are at risk of corrosion to the reinforcement and a regular inspection and repair regime will need to be implemented.

8 Structural assessment - gravity

The columns and beams of the MAU building (Blocks D1.1 and D1.3) were reviewed for the loads shown in Table 5. The load combinations are as described in *Eurocode 0: Basis of Structural Design*. These are gravity loads (self-weight plus imposed live and dead loads) only. The capacities of the beams were reviewed against *Eurocode 2: Design of Concrete Structures*.

Table 5 Assumed loading of D1.1 and D1.3

Level	Predalles self-weight (kPa)	Superimposed Dead Load (kPa)	Imposed Load (kPa)
Level 9, Level 10	4.5	3.0	3.5
Level 11 Plant room	7.5	3.0	3.5
Level 11 roof, level 12	7.5	3.5	1.0

Table 6 Material properties assumed in review

Property	Material value
Concrete – characteristic cube strength	
Original design	f_{cu}=30MPa
Current condition	f_{cu}=18MPa
Concrete density	24kN/m³
Steel	f_{yk}=440N/mm²
Steel material factor	γ_s=1.15
Steel yield design	f_{yd}=383N/mm²

8.1 Columns

8.1.1 Working stress check

An initial working stress check was carried out for the columns. This was comprised of the unfactored loads shown in Table 5 multiplied by the area of floor that each column supports. This calculation provided a stress in the columns, which was compared with the adopted design stress: f_{cu} = 18MPa.

All of the columns at level 8 were found to be working below the adopted design stress.

8.1.2 Ultimate limit state check

Checks were carried out at the Ultimate Limit State (ULS), which is similar to the working stress check but includes the standard factors of safety.

21 no. columns failed the ULS checks based on the adopted design stress, and f_{cu}=18MPa.

Table 7 Columns overstressed at ULS

	D1.1	D1.3
f_{cu} =18MPa	2	19
f_{cu}=30MPa	0	1

8.2 Beams

8.2.1 Working stress check

Eight beams were found to fail the ULS checks based on the unfactored working stress. Whilst there appears to be no immediate safety concern, based on site observations of the beams in question, they are working close to their capacity. We recommend strengthening works and/or additional columns to those beams in order to provide suitable factors of safety against failure.

8.2.2 Ultimate limit state check

Allowing moment redistribution (which helps to reduce peak stresses) of up to 30% in beams, 25 no. beams failed the ULS checks based on the factored design stress.

The beams have not been checked for deflection under imposed loads, as excessive deflections lead to damage to finishes rather than safety concerns.

8.3 Gravity conclusion

There are no columns that fail the working stress checks and there are 8 beams that currently fail the working stress check. There are no visible signs of distress, and we believe that the beams are working with reduced factors of safety.

There are 25 beams and 21 columns that require strengthening work due to the reduced concrete strength. The strengthening options are covered in more detail in section 10.

9 Structural assessment - seismic

9.1 Introduction

The assessment of the MAU building was carried out in accordance *Eurocode 8: Design of Structures for Earthquake Resistance – Part 3 - Assessment and Retrofitting of Buildings* (EC8-3) with reference as necessary to *Part 1- General rules, seismic actions and rules for buildings* (EC8-1).

9.2 Assessment criteria

9.2.1 Limit states

EC8-1 requires that the structure should be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. i.e. the building will stand up after a major earthquake event but may not be operational. (No Collapse – design return period = 475 years)

EC8-1 also requires that a structure should be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the limitations of use, the cost of which would be disproportionally high in comparison with the costs of the structure itself. i.e. the building will stand up after a less major earthquake and should be operational. (Significant Damage – design return period = 225 years)

In EC8-3 there are three limit states which the building can be assessed against. These limit states define the level of damage in the buildings. These are Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). It is worth noting that the definitions associated with the limit states to EC8-3 are different to the limit states in EC8-1.

Significant Damage (EC8-3). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out of plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair. Basic return period of 475 years, corresponding to a probability of exceedance of 10% in 50 years.

Damage Limitation (EC8-3). The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures. Basic return period of 225 years, corresponding to a probability of exceedance of 20% in 50 years.

For the review of the existing hospital building checks were carried out against Significant Damage and Damage Limitation to EC8-3.

9.2.2 Defining earthquake event

Having identified the limit states that the building will be checked against it is important to identify the design seismic event the building will experience. A design seismic event in Eurocode 8 is based on a '*reference peak ground acceleration on rock*'(PGA, a_{gR}) which is defined in the country specific national annex The reference PGA in Eurocode 8 is expressed as a_{gR}=value * g, where g = gravitational constant 9.8m/s². Unlike the Richter and moment magnitude scales, the PGA is not a measure of the total energy (magnitude, or size) of an earthquake, but rather of how hard the earth shakes in a given geographic area (the intensity).

The PGA is modified to obtain a '*design ground acceleration on rock'* with a value represented as a_g. The modification of the PGA is by an importance factor γ_I. The values for γ_I are defined in EC8-1 clause 4.2.5 and table 4.3. For a hospital building the importance class is IV and the recommended value of γ_I = 1.4.

It is worth noting that the Mater Dei hospital is founded on rock, and the a_g value needs no further modification for the ground type.

The PGA is associated with a return period. The higher the return period the greater the PGA for example a return period of 2500 years will have a greater seismic event associated with it compared to a 475 year return period. By modifying the PGA with an importance factor the return period of the seismic event increases, hence the building is designed for a seismic event that has a lower probability of occurring.

Table 8 Seismic events

	PGA (a_{gR})	**Design peak ground acceleration (a_g)**	**Analysis case**
1st phase MAU	0.04g	0.056g	
SHARE data / 2nd phase MAU	0.06g	0.084g	1
Oncology tender documentation	0.10g	0.14g	2
Oncology design documentation	0.085g	0.12g	

The design peak ground acceleration shown in the above table is used for the *Significant Damage* to EC8-3 review. The design peak ground acceleration (a_g) value was reduced by a factor of 0.5 to review the structure against the requirements for *Damage Limitation* to EC8-3.

9.2.3 Seismic Review

The review of the MAU building followed two phases:

1. Review against SHARE/2nd phase MAU seismic event: a_g=0.084g.
2. Review against highest design peak ground acceleration value: a_g =0.14g

At both phases the requirements for *Significant Damage* and *Damage Limitation* was assessed Table 9 summarises the different a_g values for each limit state. The review included a deformation and strength check of the elements.

Accidental torsion in accordance with EC8-1 was incorporated into the analysis, as it increases the moments on the perimeter columns.

Table 9 Analysis cases summary

	Analysis Case 1	**Analysis Case 2**
PGA (a_{gR})	0.06g	0.10g
a_g	0.084g	0.14g
Limit state		
Damage Limitation ground a_g	0.084g	0.14g
Significant Damage a_g	0.042g	0.07g

9.3 Analysis approach

For a seismic event a building behaves in a different manner to that of gravity loading. With gravity loading the load can be traced vertically down to the ground. For seismic loading the load is applied horizontally to the building, which is then transferred into the vertical structure via the formation of plastic hinges.

To carry out a seismic assessment on a building the location of plastic hinges needs to be anticipated. Plastic hinges are typically found in the least stiff primary elements. Typically in the MAU building the columns are 450mm*450mm and the beams are 1000mm wide by 400-480mm deep. The columns are less stiff than the beams hence the plastic hinge will form in the columns before the beams and hence will fail before the beams.

For the assessment of the structure to withstand a seismic event, the capacity of the columns was the main focus of the review. The beams were also assessed under gravity loading, and those beams which are close to their capacity will require strengthening works to be able to carry the seismic load. During the seismic review of the beams it became apparent that the ULS gravity condition was the governing limit state. If the beams complied with ULS then they would comply with seismic.

The review focused on the capacity of the columns for strength including bending and axial loads and deformation capacity, i.e. how much an element can rotate prior to the element failing, and how much additional load the columns can take from a seismic event prior to failing.

9.4 Capacity

The capacity checks were carried out assuming a variety of characteristic concrete strengths to gauge the impact concrete strength has on the capacity of the building. For loading assumptions refer to Table 5 and material assumptions Table 6.

9.4.1 Strength capacity

The strength capacity check of the main structural elements (columns and beams) were carried out in accordance with EC8-3 and EC2-1-1. The column elements were checked for their working (unfactored) load capacity and plotted on an M/N curve for that particular column type (i.e. the blue dots shown below). This initially determined whether the columns were working above their capacity. For each column the seismic loads were then plotted against the gravity loading. This helped to identify which columns fell outside the M/N curve, under both gravity and seismic conditions. Figure 8 shows an example of the curve, all the columns of this particular type fall within the M/N curve and therfore within the design capacity.

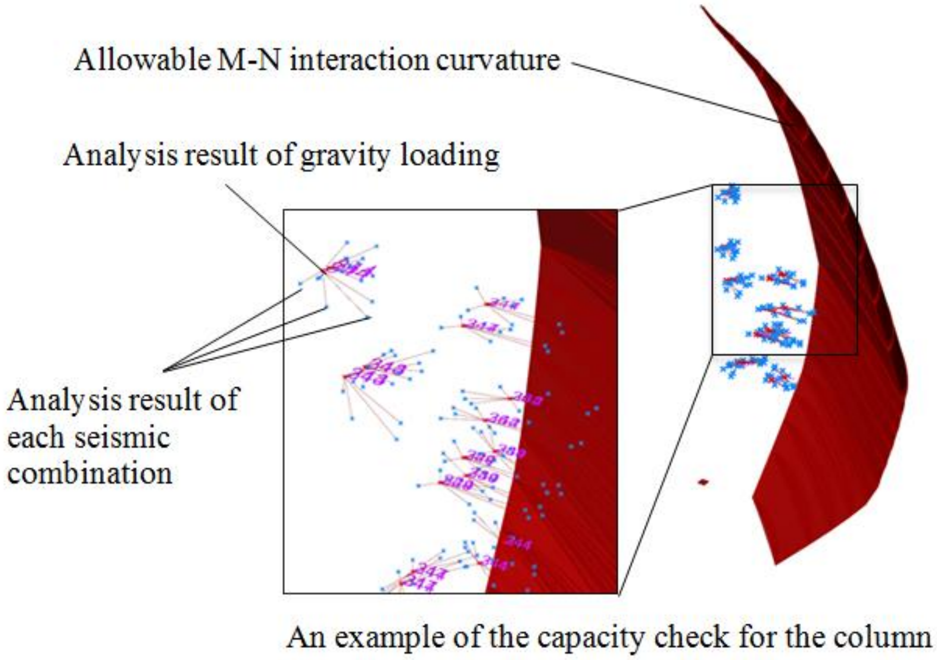


Figure 8 Example M/N plot for strength capacity

Analysis cases 1 and 2 were evaluated, under the significant damage condition (Table 9) as per EC8-3 to investigate the impact the seismic event has on the building. This will help to determine the extent of remedial works required to bring the compliance up from the seismic event at analysis case 1 to analysis case 2.

Table 10 Summary of number of columns over stressed at strength check with no accidental torsional effects.

	Analysis case 1	Analysis case 2
Block D1.1	8	14
Block D1.3	20	28

Additional analysis was carried out taking into consideration the impact of the accidental torsional seismic effect in accordance with EC8-1 shown in Table 10. The number of columns that failed the strength check increased compared to those shown in Table 11

Table 11 Summary of number of columns over stressed at strength check including accidental torsional effects and assuming f_{cu}=18MPa.

	Analysis case 1
Block D1.1	35
Block D1.3	33

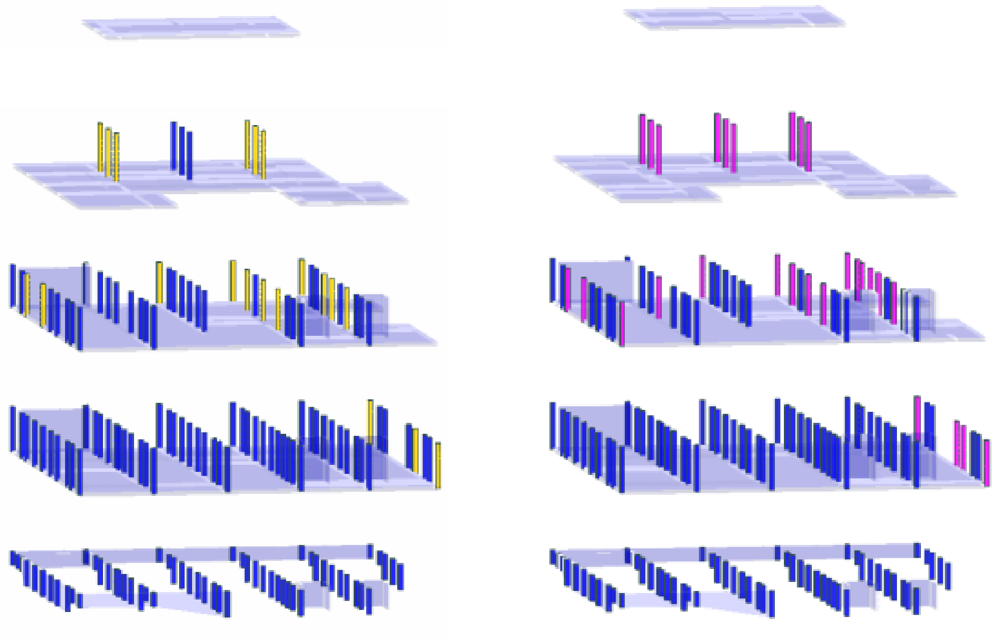


Figure 9 shows and example for block D1.3 with yellow and pink columns representing over stressed in the strength condition under different assessment criteria.

9.4.2 Deformation check

The deformation checks were carried out in several stages. The first stage was to check the rotation of the element under gravity loading. If the element did not comply with the limits set for rotation then they would not comply under seismic loading. The deformation of the columns was checked to both the significant damage and damage limitation limit state.

Table 12 Summary of number of columns over rotated at deformation check incorporating accidental torsional effects

	Limit State	**Gravity loading**	**Analysis case 1**	**Analysis case 2**
Block D1.1	Significant Damage	0	15	20
Block D1.1	Damage Limitation	6	63	63
Block D1.3	Significant Damage	8	38	71
Block D1.3	Damage Limitation	30	90	129

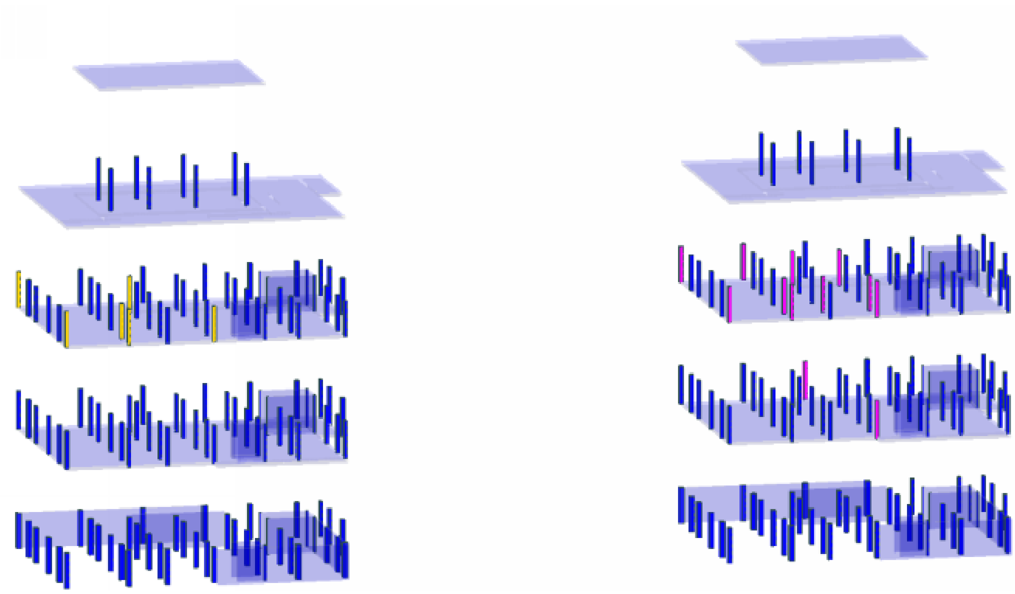


Figure 10 an example of the columns over rotated under significant damage limit state for one seismic load case.

9.4.3 Impact on concrete strength

As part of the review the impact concrete strength had on the deformation capacities was investigated. Analysis case 1 was assumed which has a_g=0.042g/0.084g. The concrete strength was altered to f_{cu}=18MPa and f_{cu}=30MPa.Table 13 show results based on no accidental torsion. The number of columns that fail the deformation checks will increase when accidental torsional effects are incorporated.

Table 13 Comparison of concrete strength on rotational capacity for block D1.3 ignoring the impact of accidental torsion on the building

169 columns in total in D1.3	**No. of columns over stressed in strength condition under seismic loading**	**Number of over rotated columns under Damage Limitation**			**Number of over rotated columns under significant damage**		
Concrete grade	Strength	Gravity	seismic	Total %	Gravity	seismic	Total %
f_{cu}=18MPa	35 (21%)	42 (25%)	25 (15%)	40%	9 (5%)	22 (13%)	18%
f_{cu}=30MPa	20 (12%)	28 (17%)	20 (12%)	29%	7 (4%)	17 (10%)	14%

Table 13 shows that concrete strength plays a big part in the percentage of columns that fail the capacity checks in accordance with EC8-3. The values shown above are representative of one particular seismic event.

Table 14 Comparison of concrete strength on rotational capacity for block D1.3 incorporating accidental torsional effects

169 columns in total in D1.3	**Number of over rotated columns under Damage Limitation**	**Number of over rotated columns under significant damage**
f_{cu}=18MPa	90 (53%)	38 (22%)
f_{cu}=30MPa	89 (53%)	37 (22%)

As can been seen by comparing Table 13 and Table 14, accidental torsion has a significant impact on the demand of the columns for rotation.

9.5 Seismic conclusions

9.5.1 Impact of seismic event

As can be clearly seen in section 9.4.1 and 9.4.2 the size of the seismic event has an impact on the number of columns that fall below the compliance criteria of EC8-3.

Block D1.1 has a greater proportion of columns in compliance with EC8-3 than D1.3. This is due to D1.1 having more wall structures e.g lift shafts and stair cores than D1.3, and being generally smaller in plan.

Strength

There are several columns that will need remedial works to bring the strength capacity in line with EC8-3, based on fcu=18MPa. Table 10 summarises these columns based on the seismic event.

Significant Damage

To bring the MAU building into alignment with the significant damage limit state will require remedial works for a high proportion of the columns.

Damage Limitation

To bring the MAU building into alignment with the damage limitation limit state will require remedial works for a high proportion of the columns.

9.5.2 Impact on concrete strength

As shown in Table 13 the concrete strength has the greatest effect on the Damage Limitation limit state. With f_{cu}=18MPa, 40% of columns (ignoring accidental torsion effects) would require remedial works for the damage limitation compared to the original assumed f_{cu}=30MPa where 29% of columns would require remedial works. A higher number of columns would require remedial works if accidental torsion effects were taken into consideration.

9.5.3 Impact on accidental torsion effects

The accidental torsional effects have a significant impact on the demand on the columns. By providing a stiffer structure in particular around the perimeter will reduce these effects.

10 Remedial works

As highlighted in sections 7, 8, and 9, the structure is deficient in several areas, and remedial works are recommended. The remedial works are required to the building due to durability of the level 7 columns and high risk of imminent durability issues to level 8 columns, a reduced concrete strength impacting the capacity of columns and beams, and poor seismic performance.

Section 10.1.1 makes recommendations for the columns at level 7 of D1.1.

Possible solutions for the strengthening work for both gravity and seismic loading are summarised in Table 15. The columns require strengthening in the axial capacity and an increase in the rotational/deformational capacity for seismic and gravity conditions. 25 beams require an increase in bending capacity.

Table 15 Possible remedial options

	Member Reinforcement approach			**Force redistribution approach**		
	Concrete jacketing	**Steel Jacketing**	**FRP plating and wrapping**	**Additional steel bracing**	**Additional concrete wall/columns**	**Retrofit to existing walls**
						See sketch SK-
Structural capacity benefits	Increased flexural, and shear strength. Increase in bearing capacity. Increases deformation capacity	Increase shear strength	Increase shear strength. Increased bending strength in beams.	Increases stiffness of whole building, which in turn reduces deflection and flexure of individual elements.	Walls will increases stiffness of whole building, which in turn reduces deflection and flexure of individual elements. Columns will reduce deformation in localised areas	
Change to structure	Columns will increase in size by at least 200mm in each direction	Columns will increase in size	Minimal	Will introduce new vertical elements within the building	Will introduce new vertical elements within the building	Minimal
Impact on hospital	Each column would need to be exposed and drilling would be required through the slabs.	Each column would need to be exposed	Beam soffits would need to be exposed, and the FRP stuck on. For the beams that require remedial works	Locations for the bracing will need to be investigated and continuous throughout height of building. Localised work	Localised impact.	Perimeter and internal masonry walls will require retrofitting. Possibly localised to discrete areas

10.1 Columns

10.1.1 Concrete repair

The columns in the basement of D1.1 as shown in Figure 4 will need to be made good. The spalled concrete will need to be broken back to sound (uncarbonated) concrete, and the reinforcement will need to be treated prior to the concrete being repaired. The high carbonation levels from the testing undertaken indicates that most of the columns are now at risk of corrosion. This can be dealt with by adopting a regular inspection and repair regime. The time between inspections can be increased by the application of appropriate anti-carbonation coatings. An appropriate strategy will be proposed as part of the remedial works stage.

10.1.2 Gravity

The columns that are failing under ULS gravity condition, additional checks such as concrete core testing can be carried out to confirm the column strengths. An alternative approach is to reduce the dead load factors applied to the self-weight of the structure as more is known about the existing condition. If the further test/checks outlined above do not bring the columns load within the ULS capacity then strengthening works would be required to reinstate code margins of safety. Alternatively if the Hospital will accept restrictions on future change of use (i.e. not increasing the building weight or load within) then an accepted pragmatic approach is to accept reduced factors of safety without compromising safe usage.

10.1.3 Seismic

As previously identified in other sections, the columns require strengthening and an increase in the rotational capacity. The most appropriate solution for increasing both strength and rotational capacity of the columns will be to apply a concrete jacket to each column that is currently failing the EC8-3 requirements.

The concrete jacket negates any existing weakness in the concrete columns.

The concrete jacket will increase the size of the columns, and increase the stiffness. Increasing the stiffness of the columns will increase the seismic loading onto the columns which will then need to be checked to ensure the loading onto the column will be within the increased capacity. There will be a number of cases where the concrete jacket will not be able to be big enough to increase the strength and deformation capacity so it is recommended that additional vertical structure will need to be introduced.

Introducing additional vertical structure such as new columns, walls, and bracing will increase the stiffness of the building. The new walls/bracing will attract more of the seismic loads and reduce the seismic loads on the columns.

With the high proportion of columns that require strengthening for a seismic event Arup introducing additional concrete walls to improve the stability system and relieving the columns of the shear forces and moments imposed under seismic events. Reducing weight of the plantroom structure on the roof and providing a system of new bracing will improve seismic performance.

10.2 Beams

For the beams to carry the seismic load, they need to be able to carry the ULS load. There are several beams that are currently at their gravity load capacity and will need to be strengthened.

The two most appropriate solutions for the beams are for the introduction of columns on long spans, and carbon fibre strips to the underside of the beams to increase the bending strength of the beams. Reducing load will also be considered. During the remedial works stage the most appropriate solution for the beams will be put forward.

10.3 Masonry walls/façade

The masonry walls that form infill panels and the façade should be tied back to the structure. There is currently no evidence to confirm whether the walls have been designed and detailed or built with ties tying the walls back to the concrete frame.

All walls will require inspection to confirm the existence of ties to the concrete frame, and for the façade between the limestone blocks and the internal blockwork skin. If these ties are not present then remedial ties will need to be installed.

As part of the review for the strengthening of columns, the possibility of strengthening the perimeter walls in such a way which will increase the stiffness of building thus reducing the demand on the columns was raised as a practical solution. An initial scheme has been drawn up and shown in SK-017 included in Appendix B.

The blockwork was not originally designed to carry seismic loads. The façade to the MAU building is a double skin block and limestone wall which adds inherent stiffness to the structure and so is a likely path for seismic loads to take regardless of the design intent. Based on the higher seismic design figure, the blockwork potentially provides an adequate load path and removes the need to strengthen the majority columns for seismic loads.

The remedial works to the blockwork need to ensure an adequate connection between the panels of blockwork and the concrete structure. There is a possibility that the blocks may also be hollow in some or all areas (not investigated on site), and to achieve a suitable load path the blocks may need to be grouted up via holes drilled in the sides.

10.4 Summary

Extensive work is required to bring the MAU building up to standard, the anticipated costs of the remedial work is likely to be in a range of €3.0 million to €5.0 million. A variety of solutions have been investigated and discounted due to the suitability of carrying out these options whilst ensuring minimum disruption to the daily operation of the A&E at Mater Dei Hospital.

11 Next steps

We recommend that a system of monitoring should be put in place to monitor and review proposed changes to loading and to observe (e.g. through an annual one day site visit) any signs of distress in the structure that might become apparent in the future. Both an FMS representative and a structural engineer would be needed to carry out each role.

The concrete durability is of concern. Testing indicated that the carbonation front has reached the steel reinforcement and as such there is a high risk that in areas of high humidity (un-occupied areas of level 8 and level 11 plant rooms) the reinforcement will corrode ultimately leading to a reduction in element strength if left untreated. A regular inspection and repair regime should therefore be adopted.

The design remedial works for the MAU building will need to be developed into a series of options for tender. It is envisaged that the options will be developed further by a suitably qualified team of local engineers and contractors, in conjunction with the hospital. As the hospital is a working building and will need to remain fully operational during installation of remedial works, the hospital will need to be consulted on the appropriate staging of the remedial works, and this will need to be taken into careful consideration for the remedial works design.

Appendix A

Oncology and MAU Phase 1 Memo

File Note

ARUP

To	James Camenzuli	Date	6 October 2014
Copies	Alan Comerford, John Papagiorcopulo	Reference number	238866-00
From	Ed Hoare, Richard Hill, Tilly Langley	File reference	4-05-07
Subject	Mater Dei – Accident and Emergency Building – Stage 1 Report		

1 The Building

The accident and emergency building is rectilinear on plan, approximately 40 m by 65m. The building comprises 4 stories referenced level 8, 9, 10 and 11. Level 8 is a basement level. Level 11 is the roof level, at this level there are two plant roofs of similar construction to the main building.

The building is of reinforced concrete construction comprising precast concrete floor units (known as predalles; a thin precast panel with insitu reinforced concrete topping), supported on insitu reinforced concrete beams and columns. The columns all appear to be the same size throughout the height of the building (450mm square) and are understood to be founded on shallow pad footings bearing on to rock.

The building on the design drawings is divided in to two sections referenced as D1.1 and D1.3. It is understood that construction was phased for D1.1 with construction commencing in in 1996 of L8, L9 and L10 over part of the footprint (the southern bays gridline 01/D1.1 to E/D1.1) before the north two bays where added along with L11 over the entire footprint of D1.1 in 2006/7. D1.3 is understood to have been built in one continuous operation in 1996.

The A&E building is a reinforced concrete frame building. The horizontal structure consists of reinforced concrete slabs made up of predalles. The predalles are a precast floor system which have a precast concrete biscuit (60mm thick) with polystyrene void formers with a cast insitu concrete layer on top. The predalles are supported via cast insitu concrete beams. The beams are supported by a series of cast insitu concrete columns and walls. The columns and walls are supported on pad footings which sit on rock.

Lateral loads appear to be resisted by the frame action of the beams and columns, known as a moment frame refer to Appendix A for further observations.

2 Original documentation

Design information was supplied by FMS.

2.1 Drawings

1. ‘As Built’ structural drawings for the extension of block D1 in 2007 – includes details between connections of new and old building
2. ‘For construction’ structural drawings for block D1 – includes some reinforcement details
3. ‘Pre-construction’ structural drawings for block D1 – we believe that these have been superseded with the drawings from the ‘For Construction’ set

Table 1 Summary of design information

Element	**Dimensions**	**Reinforcement details**	**Comment**
Foundations	Yes	Yes	Details for 1996 construction Details for 2007 construction
Columns	Yes	Yes	Details for 1996 construction Details for 2007 construction
Slabs/Predalles	Yes	Yes	Shop drawings not present Some details for 1996 construction – obtained through email documentation. Details for 2007 construction
Beams	Yes	Yes	Details for 1996 construction Details for 2007 construction
Walls	Yes	Yes	Details for 1996 construction Details for 2007 construction

2.2 Calculation report

There is a calculation report for block D1.3, the information in this report is an output from a computer programme (SAP 90). There is some information on the columns, beams and slab design.

2.3 Design information

The following section is a summary of the design information extracted from the calculation report

Standards

4. Building designed to Italian standards:
    - Decreto Ministero Lavori Pubblici Italiano 14-02-1992 Technical specifications for the execution of structures in normal and prestressed reinforced concrete and steel structure)
    - Decreto Ministero Lavori Pubblici Italiano 24-01-1986 Technical specifications relative to buildings in seismic regions
    - Decreto Ministero Lavori Pubblici Italiano 12-02-1982 Special criteria for safety verification of building and load and overloads

- Decreto Ministero Lavori Pubblici Italiano 11-0301988. Technical specifications regarding design and foundations execution.
- UNI 9502 specifications. Analytical procedure for the evaluation of fire resistance of normal or prestressed reinforced concrete elements

Loading

- Overaloads (imposed) design loads – no mention of partitions

Level	Superimposed Dead Load (kPa)	Imposed Load (kPa)
Roof	3.5	1.0
Other floors	3.0	3.5

- Presence coefficient S is assumed to be:
  - Accidental loads ponding floors S=0.50
  - Accidental loads roofing floors S=0.33
- Wind loading – not mentioned

Seismic

- Seismic design data
  - Class III
  - Seismicity grade S=6
  - C=0.04 (Seismicity intensity factor)
  - R=1 (Response factor)
  - ε=1 Foundation factor
  - I=1.4 seismic protection factor
  - B = 1 for building D1.3

Load combinations

- Load combinations – note the design is to allowable stresses

Combination	Permanent overloads	Live loads	Seismic x	Seismic y
I	+1	+1		
II	+1	+1	+1	
III	+1	+1	-1	
IV	+1	+1		+1
V	+1	+1		-1

- Foundations concrete 28 days 25N/mm² (cube)
- Other elements concrete 28 days 30N/mm² (cube)
  - Note that 30N/mm² cube strength is equivalent to 25N/mm² cylindrical strength
- Steel fyk = 440 N/mm², ftk = 550 N/mm²

2.4 Construction information

5. Monthly reports for March/April 1996 including:
    6. some overall site photos
    7. Construction programme – construction for block D1 began late 1995
8. Pour dates for several weeks in March/April/May 1996
9. Concrete cube results for the entire site for dates between March - June 1996
10. Concrete core testing results for 2014

2.5 Summary

In general a good level of information exists which will simplify one part of the appraisal process (i.e. confirming what is built). From the initial walk through on 22 September 2014 the column size and spacing appeared consistent with the drawings and FMS confirmed that the drawings were accurate.

To complete the design review of the structure we require confirmation of the wind loading and earthquake loading onto the building.

3 2014 concrete core results

Between July and August 2014 a series of 64mm diameter cores were taken from columns to evaluate the insitu compressive strength. A report by iAS, dated 09 September 2014, provides a summary of the testing and an interpretation of the results. The aforementioned report has been reviewed as part of this Stage 1 study and will be used to inform the subsequent testing and appraisal. Whilst the iAS report provides a clear description of the tests, having sight of the original test information (on which the iAS report was written) is essential to enable us to understand which of the existing results can be relied upon and to ensure they can be consistently compared.

Please provide the original, raw, test reports from Terracore, Celtest and Solid Base as soon as possible as it will help inform the number and locations of future tests which could have a beneficial impact on the disruption (dust, vibration and noise) to the hospital.

Summary of results

Level 8

- D 1.1 (first stage) – 1 column tested (out of approx 14 columns)
- D 1.1 (2nd stage) – 1 column tested (out of approx 20 columns)
- D 1.3 – 30 columns tested (approx 60% of columns)

Level 9

- D 1.1 (first stage) – 1 column tested (out of approx 14 columns)
- D 1.1 (2nd stage) – 1 column tested (out of approx 20 columns)
- D 1.3 – 36 columns tested (approx 70% of columns)

At level 8, 20 of the 42 core results fell below the specified design cylinder strength of 25N/mm^2 (C25/30 concrete).

At level 9, 18 of the 44 core results fell below the specified design cylinder strength of $25N/mm^2$ (C25/30 concrete).

At level 10, all 3 cores exceeded $25N/mm^2$ (C25/30 concrete).

At level 11, the single core taken exceeded $25N/mm^2$ (C25/30 concrete).

Three test companies where used and the iAS report highlights discrepancies between the results from each company and correctly questions the validity of the test results. It is possible that the results can be influenced by the workmanship in extracting the cores as well as the transport and preparation of the cores and the calibration of the testing rig. It is not possible to know with confidence whether the existing results have been affected by such factors as such future testing is proposed for comparison to the existing results.

There are clear differences in results from the same column, this could be explained by 'testing' errors but could also be due to the core locations within a column. During the initial visit it was observed that columns had construction joints within the height of the column, meaning that the bottom half was constructed at a different time and from a different batch of concrete. A core taken from the bottom section could easily have a different strength to a core from the top section. Additionally if one of the cores was taken immediately below this joint there is a higher probability of a low core result because the top of a core is often less well compacted.

Whilst there could be 'testing' variations, there is sufficient evidence to confirm that some of the columns have a lower strength than the specified $25N/mm^2$ cylinder strength. This may not be of immediate concern as the column may not require $25N/mm^2$ strength to perform safely. This will be determined at the next stage.

4 The initial visit

Richard Hill and Edward Hoare of Arup undertook a visual inspection of the A&E building on 23 September 2014. The building was in functional use as an A&E department and significant areas of the structure were hidden by suspended ceilings and finishes making it difficult to inspect the main structure. All levels were accessed at the time of the visit, whilst some rooms were not entered, enough of each level was seen to achieve a representative understanding of the layout and condition of the building. There was no notable distress observed to the floor finishes (large ceramic tiles) and plastered wall finishes which would be a tell-tale of underlying structural issues.

Spot checks of column sizes and approximate location was undertaken and showed good correlation with the drawings that available. It is understood from FMS that a full survey had been carried out and had shown that the drawings reflected the as-built structure. ***This survey has been added to our Information Required Schedule for completeness, but we are now working with the understanding that the general arrangement drawings can be relied upon.***

In the area of Level 8 where the poor concrete test results were found, the hospital had installed propping to alleviate some of the load from the columns – refer to page 15 of Appendix A. This seems a sensible approach until such time when the structural review is complete.

As discussed above at the time of the visit there was no visual sign of distress in the elements observed indicating that the building has been performing adequately under the current loading condition. However given the concrete core results it is likely that the columns will be operating with a lower safety margin than code recommended limits. Remedial works are likely to be required to reinstate an acceptable safety margin under normal vertical loading and to ensure acceptable performance in a seismic event. At this stage it is not possible to determine how many structural elements will be affected.

5 Next steps - Stage 2

5.1 Concrete testing

Further testing is required to independently validate the testing to date and to look to answer the variations in test results. A testing schedule will be developed at the outset of the next phase taking in to account the 24 hour, 7 day working of the hospital and the busyness of the A&E.

Most tests will be focussed at Level 8 as this is where the columns are most highly stressed and understanding the material strength is most critical.

Some tests will be required at the upper levels. Other ‘less-intrusive’ testing will be explored, for example using Schmidt (rebound) hammers to allow comparison evaluation of adjacent columns. The Schmidt hammer will not be used to give strength values for use in calculation it is possible that there may be a sufficient correlation between the poor core results and Schmidt hammer readings to better understand the number of columns which may have lower strength concrete.

5.2 Opening up work to validate design drawings

The building should be designed and built to resist seismic loading. The existing drawings indicate the building was designed for seismic loads and the steel reinforcement drawn in such a way to be able to accommodate seismic loads. In order to confirm that the building has been built in accordance with the drawings some breaking out of concrete to expose reinforcement will be required.

Critical areas are at joints between floor slabs and beams and between beams and columns. The disruption, noise, dust and vibration will need to be carefully considered and the extent of opening up balance against the confidence in the design information.

5.3 Design appraisal

Focus needs to be on identifying the seismic performance requirements and establishing the minimum concrete strength required in key elements.

6 Additional information required

Below is a list of additional information required to be able to progress stage 2 of Arup's review.

11. To complete the design review of the structure we require confirmation of the wind loading and earthquake loading onto the building.
12. To understand the pattern behind the concrete results it is helpful to understand the construction sequence of block D1 with dates, and in particular D1.1 and D1.3. If there are any more construction records – including photos for D1.1 and D1.3 they will be very beneficial to the structural appraisal.
13. Please provide the original, test reports from Terracore, Celtest and Solid Base as soon as possible as it will help inform the number and locations of future tests which will assist in minimising the level of disruption (dust, vibration and noise) to the hospital.
14. Please provide the 'as-built' survey information to enable Arup to gain confidence in the current geometry.

Appendix A

Summary of Observations

Mater Dei Hospital

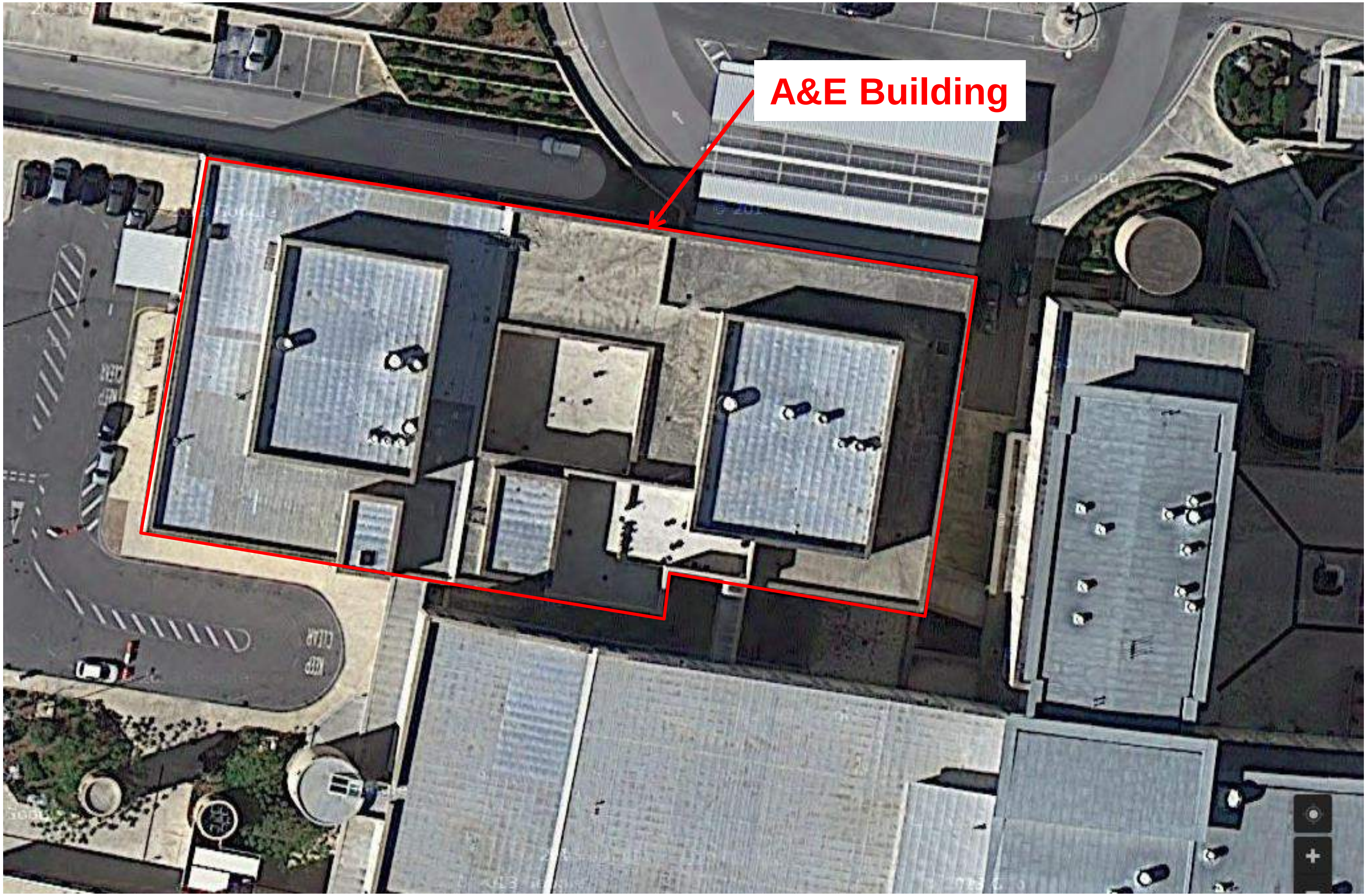
Appendix A
Summary of observations

ARUP

Mater Dei Hospital

ARUP

Accident and Emergency Building



ARUP

A&E Level 8

Cores = 64mm diameter

Col Ref	Level	Block	Stress at Failure N/mm2 Terracore Phase 1	Stress at Failure N/mm2 Terracore Phase 2	Stress at Failure N/mm2 Terracore Phase 2	Stress at Failure N/mm2 Solid Base	Stress at Failure N/mm2 Solid Base		
08-1.3-B-05	8	1.3		16.0		26.9	27.9	25.5	27.2
08-1.3-B-05A	8	1.3		16.9	23.8				
08-1.3-B-06	8	1.3		20.5					
08-1.3-B-07	8	1.3		27.1	17.6	28.1	27.3	26.7	30.2
08-1.3-B-08	8	1.3		31.4					
08-1.3-C-01	8	1.3		32.4					
08-1.3-C-02	8	1.3		28.9					
08-1.3-C-03	8	1.3		18.8		41.8	41.3	42.6	41.6
08-1.3-C-04	8	1.3		13.5					
08-1.3-C-05	8	1.3	11.4			28.5	34.7	31.1	19.8
08-1.3-C-06	8	1.3		28.4					
08-1.3-C-07	8	1.3		27.1					
08-1.3-C-08	8	1.3		29.1					
08-1.3-D-01	8	1.3		26.6					
08-1.3-D-02	8	1.3		29.1					
08-1.3-D-03	8	1.3	25.1						
08-1.3-D-03A	8	1.3		14.2	33.2				
08-1.3-D-04	8	1.3		12.9		23.6	22.1	26.9	21.6
08-1.3-D-05	8	1.3		15.1					
08-1.3-D-06	8	1.3	26.7			47.0	43.4	46.9	50.7
08-1.3-D-07	8	1.3		32.2					
08-1.3-D-08	8	1.3		22.4					
08-1.3-E-01	8	1.3		26.9					
08-1.3-E-02	8	1.3		23.9					
08-1.3-E-03	8	1.3		19.2					
08-1.3-E-04	8	1.3		18.6		23.1	26.5	20.8	22.0
08-1.3-E-05	8	1.3		23.3					
08-1.3-E-06	8	1.3		17.6					
08-1.3-E-07	8	1.3		32.8					
08-1.3-E-08	8	1.3		29.1					
		number of cores	3	27	3	7			

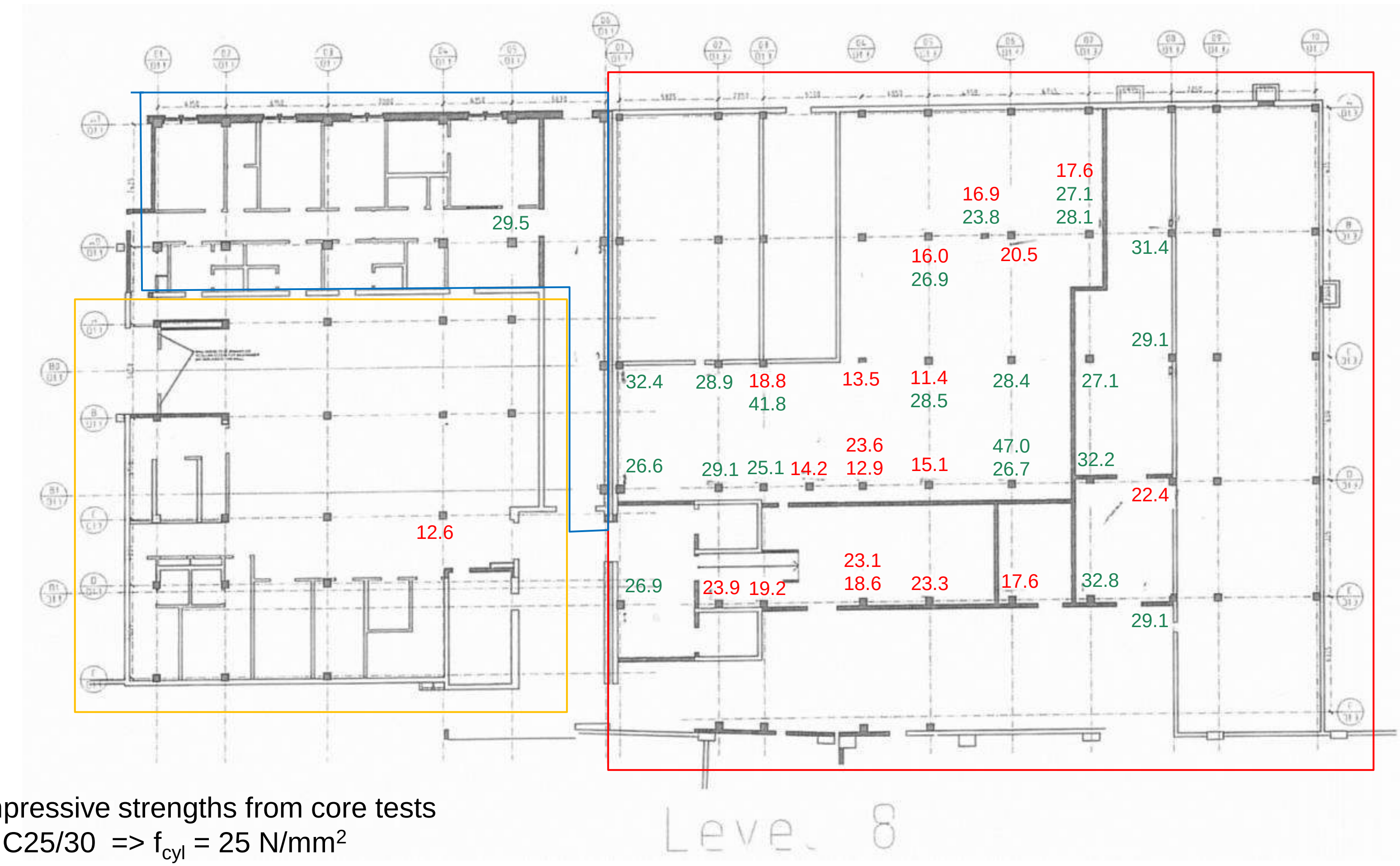
Note: values have been multipled by 1.17 to take account of the 0.85 factor

Design Specified Cylinder Strength, fck,cyl =	25	N/mm2
fck - 4 =	21	N/mm2

	Mean	SD	number of cores	$t_{0.05}$ factor	fck,is
Terracore	23.4	6.5	33	1.7	12.3
Solid Base	31.3	9.3	7	1.94	13.2
All	24.8	7.6	40	1.69	11.9

A&E Level 8

Typical columns 450mm x 450mm

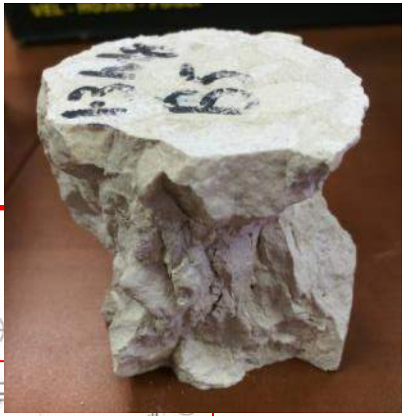


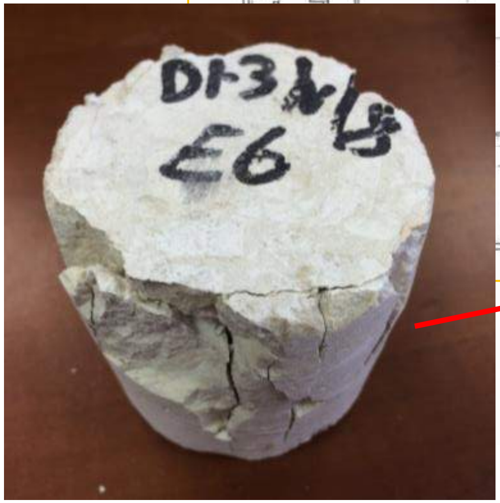
Compressive strengths from core tests
f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

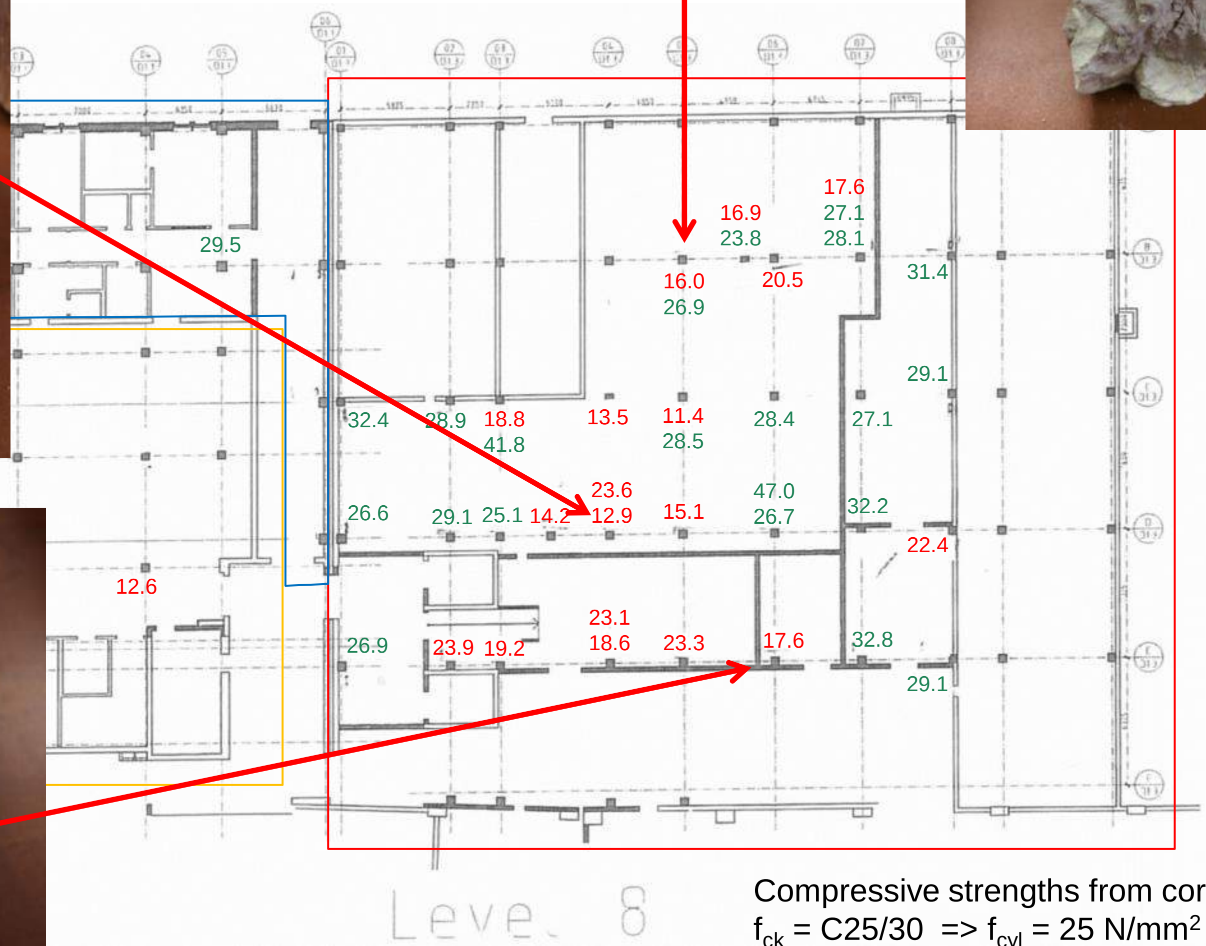
< 25 N/mm^2 > 25 N/mm^2

ARUP

A&E Level 8







Compressive strengths from core tests
f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

< 25 N/mm^2 > 25 N/mm^2

ARUP

A&E Level 9

Cores = 64mm diameter

Col Ref	Level	Block	Terracore Phase 1 Stress at Failure N/mm2	Terracore Phase 2 Stress at Failure N/mm2	Terracore Phase 2 Stress at Failure N/mm2
09.1.3-B-08	9	1.3	16.5		
09-1.3-B-01	9	1.3		30.5	41.8
09-1.3-B-02	9	1.3		23.5	
09-1.3-B-03	9	1.3		26.2	
09-1.3-B-04	9	1.3		17.8	25.6
09-1.3-B-05	9	1.3		30.9	31.1
09-1.3-B-05A	9	1.3		23.6	26.1
09-1.3-B-06	9	1.3		46.9	
09-1.3-B-07	9	1.3		33.4	
09-1.3-B-08	9	1.3		16.5	
09-1.3-B-09	9	1.3		32.1	
09-1.3-C-02	9	1.3		22.7	
09-1.3-C-03	9	1.3		24.1	27.8
09-1.3-C-04	9	1.3		8.5	10.1
09-1.3-C-05	9	1.3		43.3	
09-1.3-C-06	9	1.3		34.9	
09-1.3-C-07	9	1.3		36.5	33.3
09-1.3-C-08	9	1.3		23.1	40.2
09-1.3-C-09	9	1.3		50.5	
09-1.3-D-01	9	1.3		26.4	26.6
09-1.3-D-02	9	1.3		34.1	30.6
09-1.3-D-03	9	1.3		19.9	21.4
09-1.3-D-04	9	1.3		32.0	
09-1.3-D-05	9	1.3		37.8	42.4
09-1.3-D-06	9	1.3		23.1	20.5
09-1.3-D-07	9	1.3		24.7	
09-1.3-D-08	9	1.3		30.9	
09-1.3-D-09	9	1.3	20.4		
09-1.3-E-01	9	1.3		11.8	11.6
09-1.3-E-02	9	1.3		29.9	26.5
09-1.3-E-03	9	1.3		34.8	32.9
09-1.3-E-04	9	1.3		37.1	
09-1.3-E-05	9	1.3		40.5	34.9
09-1.3-E-06	9	1.3		40.8	
09-1.3-E-07	9	1.3	19.8		
09-1.3-E-08	9	1.3		29.2	
09-1.3-E-09	9	1.3		41.4	
		number of cores	2	27	13

Note: values have been multipled by 1.17 to take account of the 0.85 factor

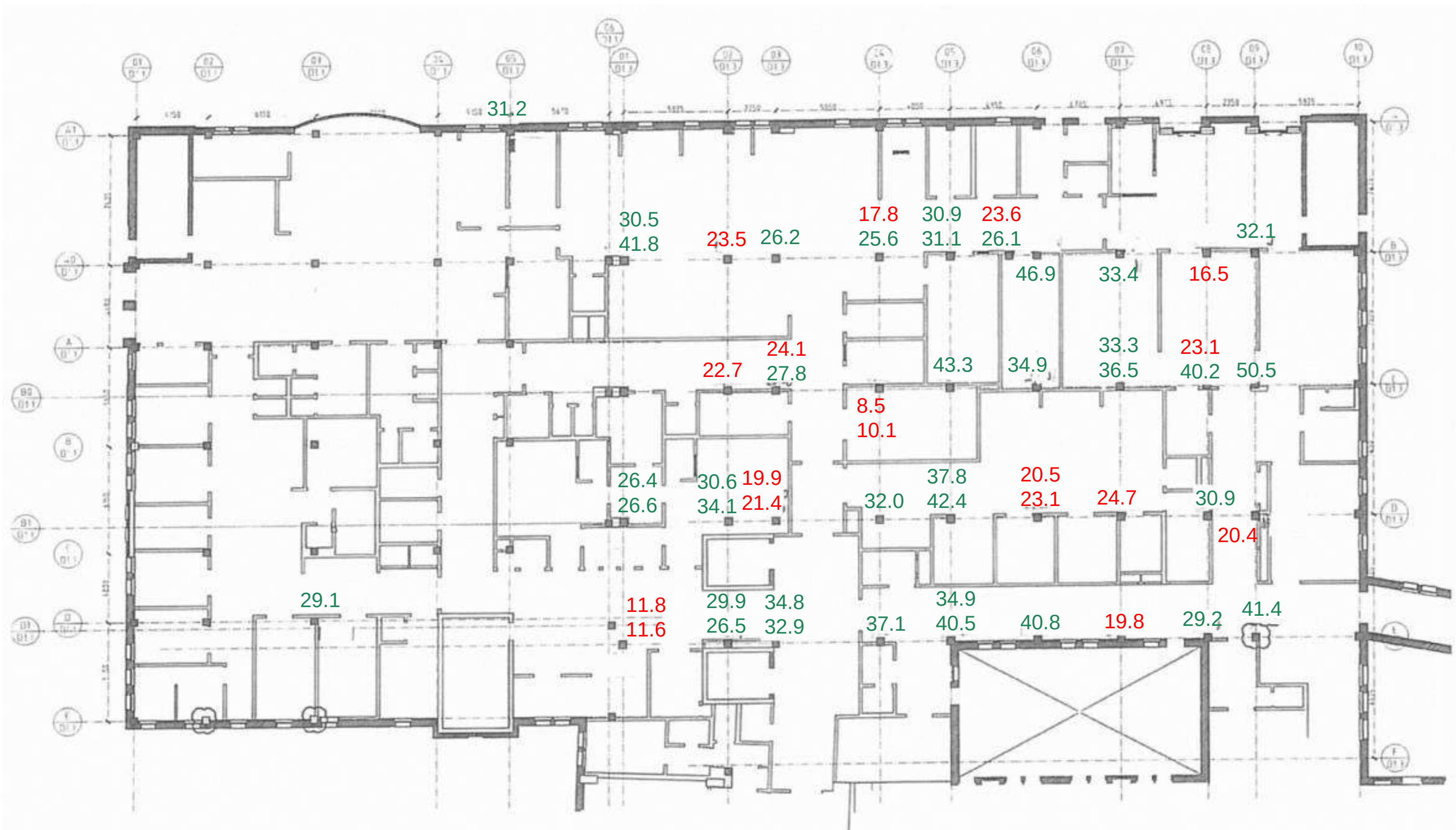
Design Specified Cylinder Strength, fck,cyl =	25	N/mm2
fck - 4 =	21	N/mm2

	Mean	SD	number of cores	$t_{0.05}$ factor	fck,is
Terracore	28.9	9.5	42	1.69	12.9

ARUP

A&E Level 9

Typical columns 450mm x 450mm

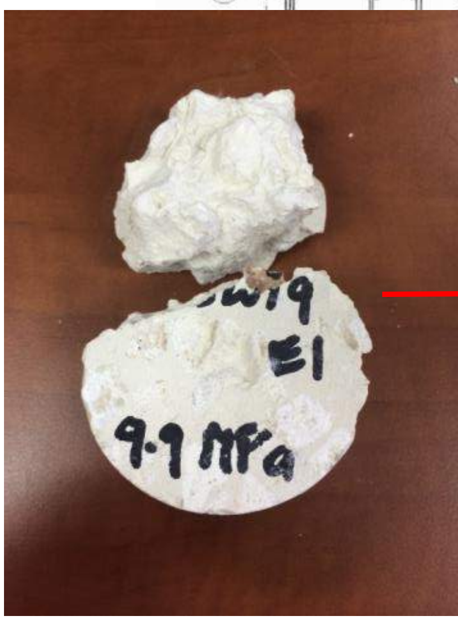


Compressive strengths from core tests

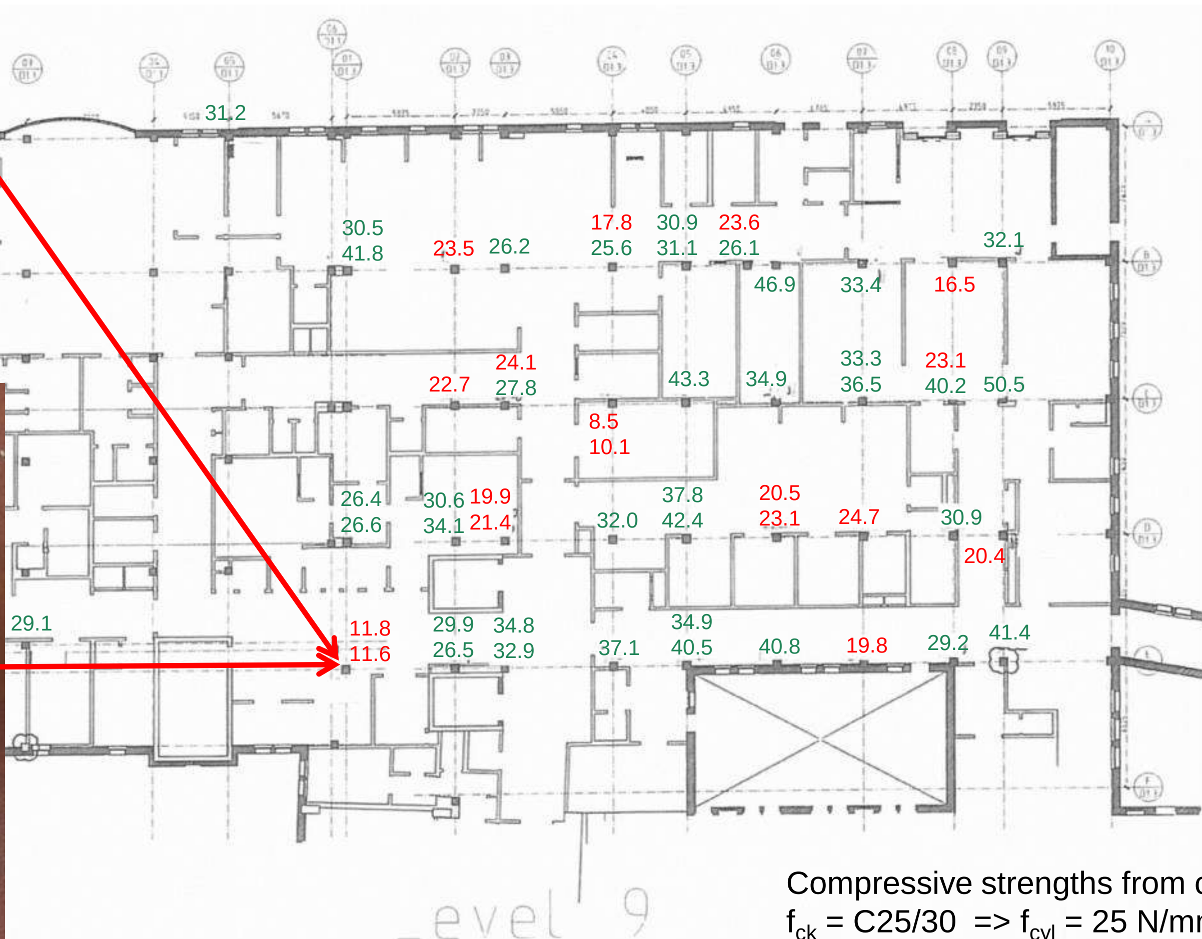
f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

< 25 N/mm^2 > 25 N/mm^2

A&E Level 9



Cores = 64mm diameter



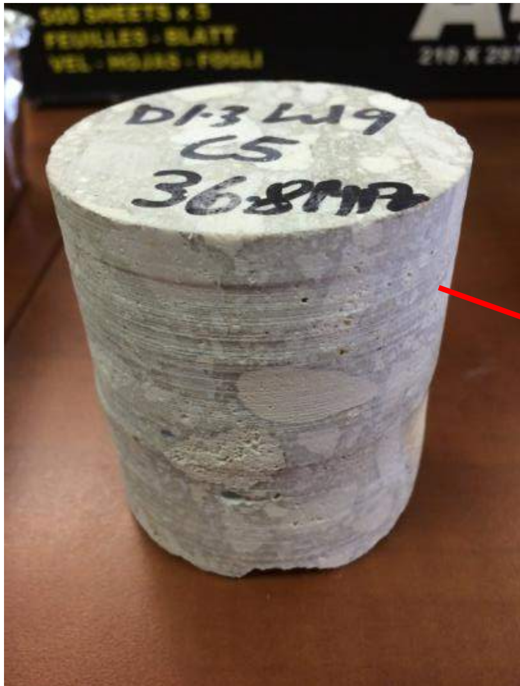
Compressive strengths from core tests

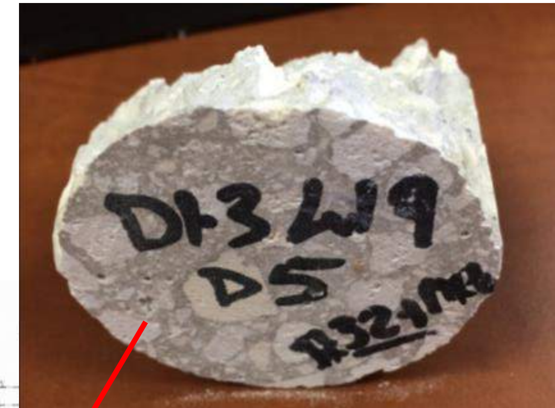
f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

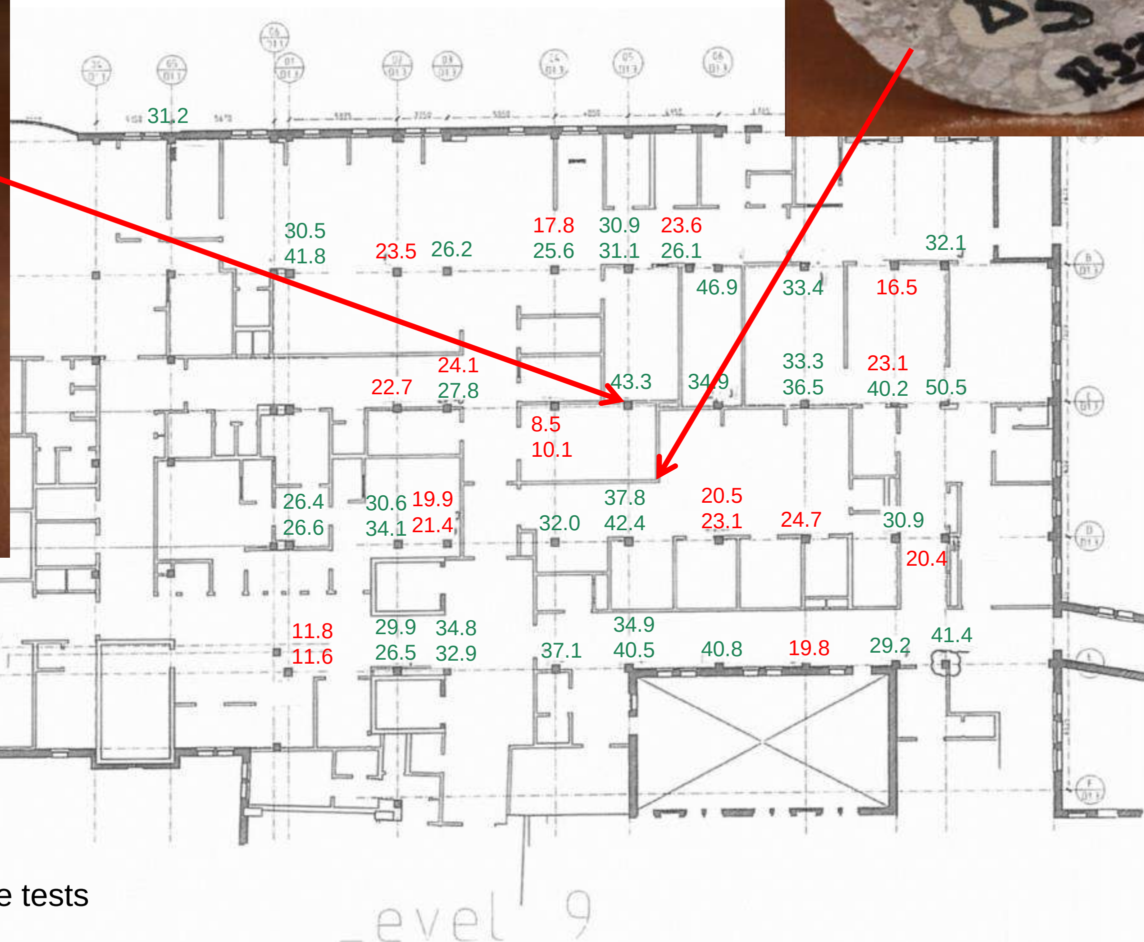
< 25 N/mm^2 > 25 N/mm^2

ARUP

A&E Level 9







Compressive strengths from core tests
f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

< 25 N/mm^2 > 25 N/mm^2

ARUP

A&E Level 10

Cores = 64mm diameter

Col Ref	Level	Block	Terracore Phase 1 Stress at Failure N/mm2
10-1.3-B-06	10	1.3	34.7
10-1.3-E-04	10	1.3	36.1
10-1.3-D-03	10	1.3	25.1
		number of cores	3

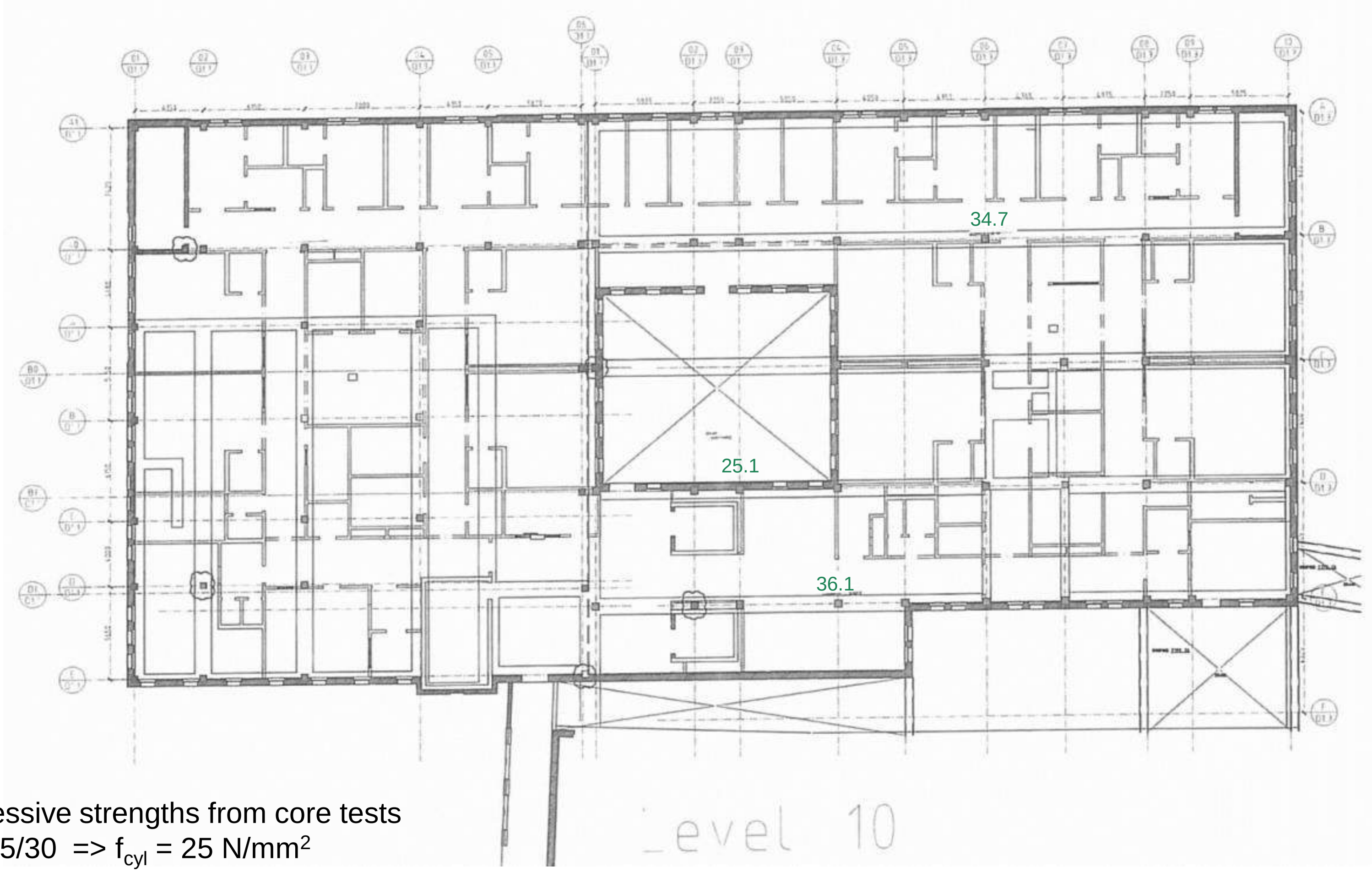
Note: values have been multipled by 1.17 to take account of the 0.85 factor

Design Specified Cylinder Strength, fck,cyl =	25	N/mm2
fck - 4 =	21	N/mm2

	Mean	SD	number of cores	$t_{0.05}$ factor	fck,is
Terracore	32.0	6.0	3	2.92	14.4

A&E Level 10

Typical columns 450mm x 450mm



Compressive strengths from core tests
f_{ck} = C25/30 => f_{cyl} = 25 N/mm²

< 25 N/mm² > 25 N/mm²

ARUP

A&E Block D 1.1 Level 8-11

Cores = 64mm diameter

Col Ref	Level	Block	Terracore Phase 1 Stress at Failure N/mm2
08-1.1-A0-05	8	1.1	29.5
08-1.1-B-04	8	1.1	12.6
09-1.1-A1-05	9	1.1	31.2
09-1.1-D-03	9	1.1	29.1
10-1.1-A0-03	10	1.1	30.5
11-1.1-B-03	11	1.1	43.1
		number of cores	6

Note: values have been multipled by 1.17 to take account of the 0.85 factor

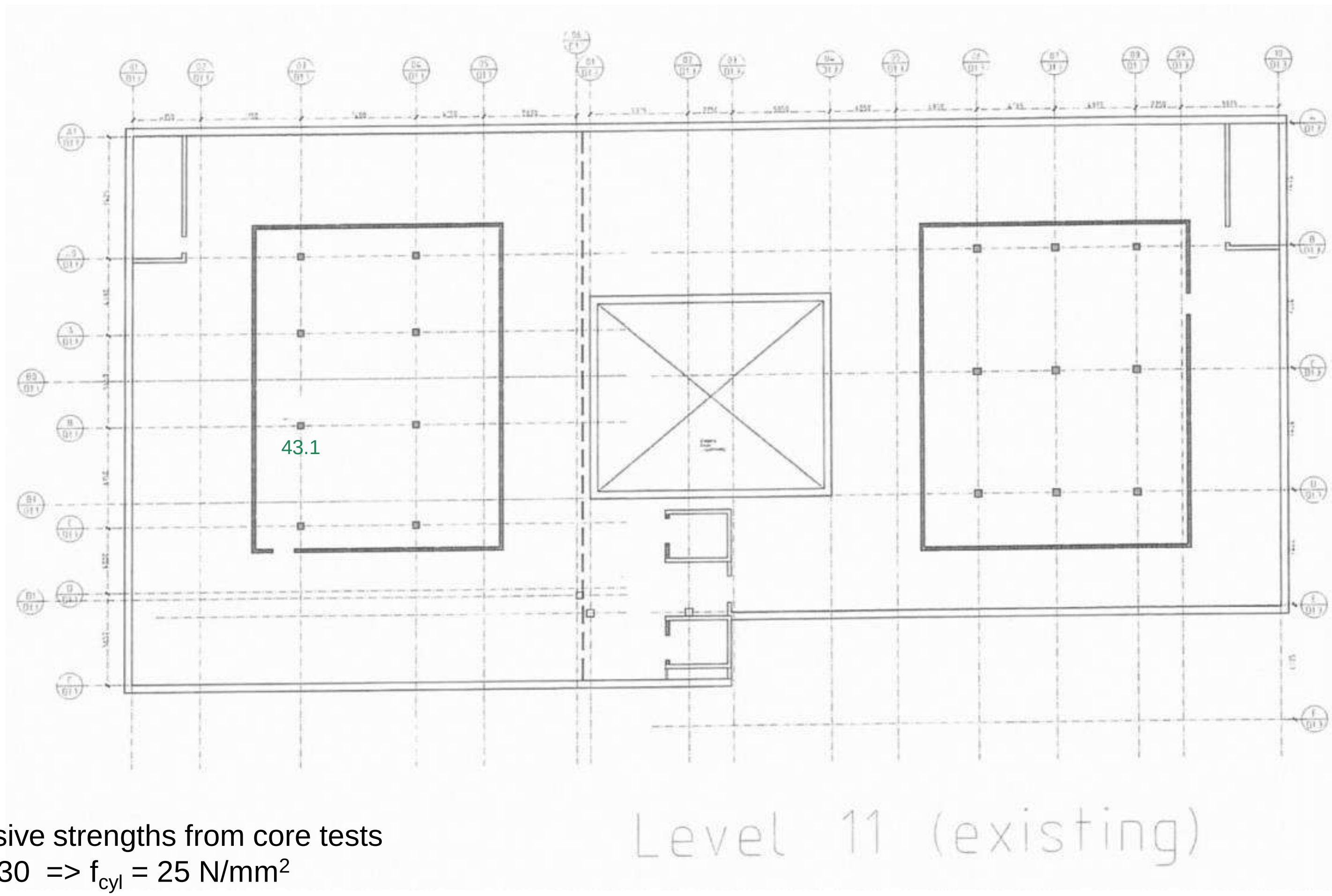
Design Specified Cylinder Strength, fck,cyl =	25	N/mm2
fck - 4 =	21	N/mm2

	Mean	SD	number of cores	$t_{0.05}$ factor	fck,is
Terracore	29.3	9.7	6	2.02	9.7

ARUP

A&E Level 11

Typical columns 450mm x 450mm



Compressive strengths from core tests

f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

< 25 N/mm^2 > 25 N/mm^2

ARUP

Figure 1: Multiple cores in single column

Figure 2: Repaired core

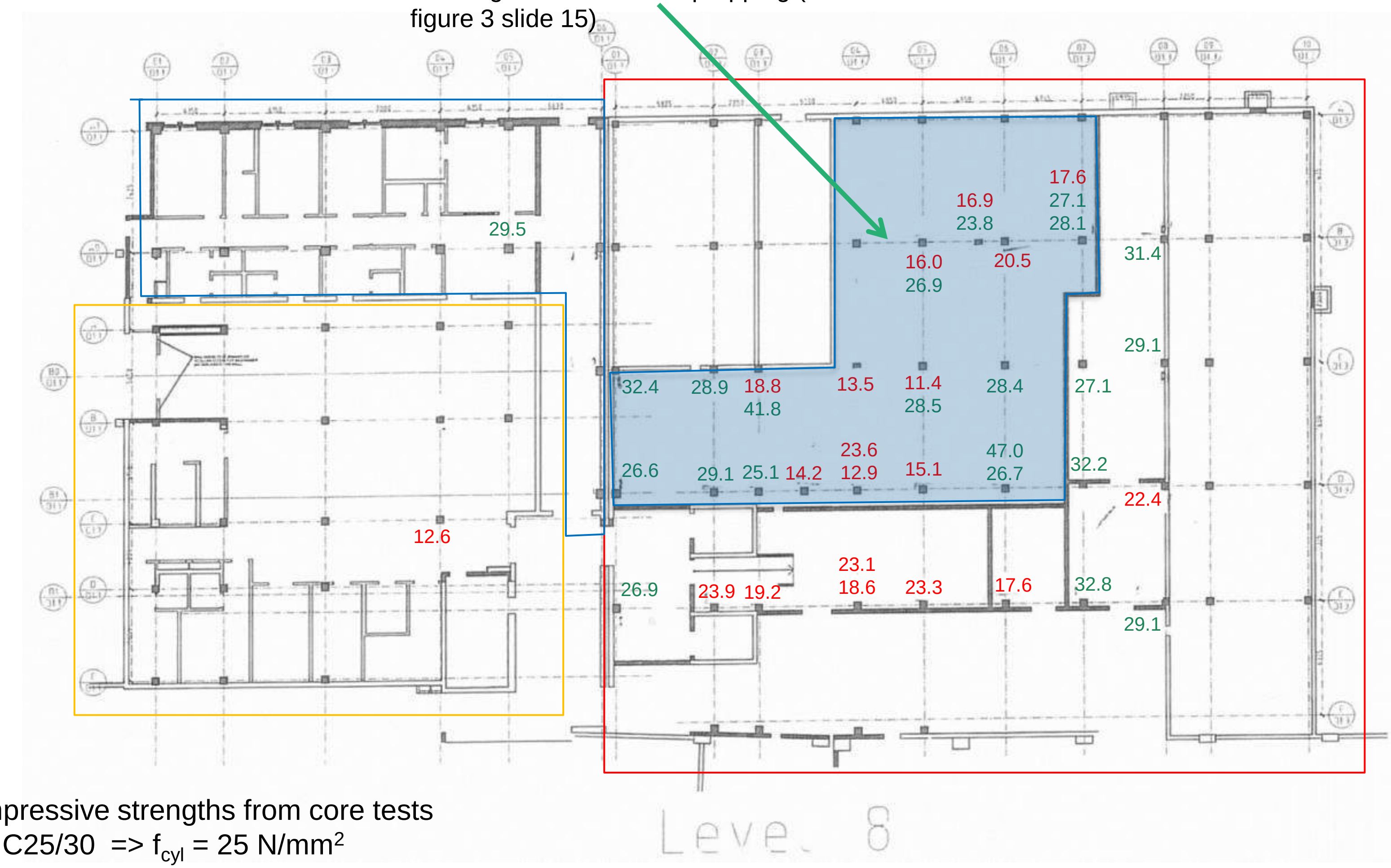
Figure 3: Propping to underside of Level 9 building D1.3

Figure 4: ‘White’ concrete column (D1.3)

A&E Level 8

Typical columns 450mm x 450mm

Shading denotes area of propping (refer to figure 3 slide 15)



Compressive strengths from core tests

f_{ck} = C25/30 => f_{cyl} = 25 N/mm^2

< 25 N/mm^2 > 25 N/mm^2

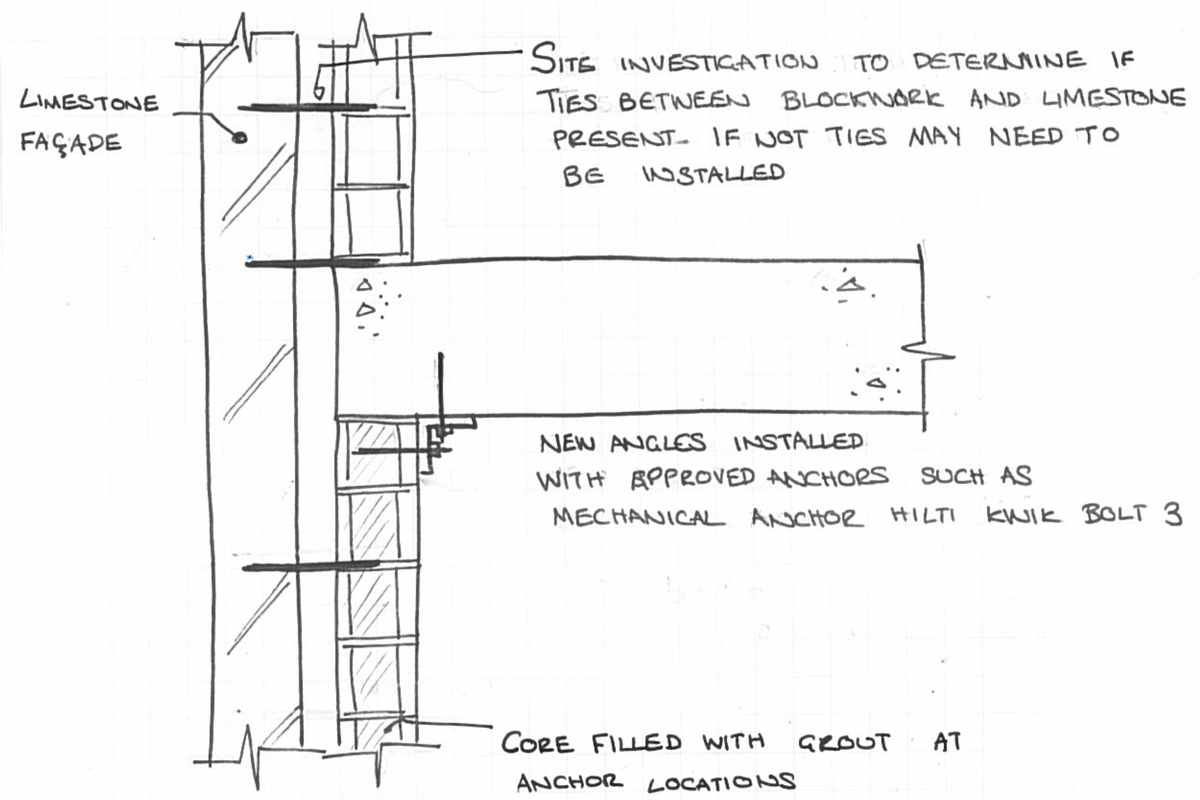
ARUP

Level 8 column

Level 11 column

Appendix B

Proposed remedial works to external wall



PROPOSED EXTERNAL WALL SEISMIC RETROFIT.

NTS

INITIAL ASSUMPTIONS.

- SEISMIC LOADS RESISTED BY WALLS ONLY INCLUDING INTERNAL WALLS
- SEISMIC LOAD TRANSFERRED VIA SHEAR FROM SLAB INTO WALLS.
- APPROXIMATE SEISMIC FORCE = 1200 IN BOTH DIRECTIONS (ON D1.1)
- ASSUME ONLY ONE WALL ON SHORT PERIMETER ACTING = APPROX 30m => 40 kN/m
- Assume 15.9 mm diameter Hilti Kwik bolt 3 EMBEDMENT DEPTH 102mm ∴ NEED ONE BOLT AT 1200mm CENTRES MINIMUM.
- ASSUME CORES WITH BOLTS LOCATED ARE FILLED PRIOR TO INSTALLATION.
- ALL LEVELS TO BE RETROFITTED
- STRENGTHENING TO COLUMNS NOT SHOWN
- ADDITIONAL INTERNAL WALLS MAY BE REQUIRED.
- ADDITIONAL COLUMNS NOT SHOWN.
- ASSUME 50% OF HOLLOW BLOCKWORK FILLED.

ARUP	Axo Sheet
Sketch Title	PROPOSED REMEDIAL WORKS TO EXTERNAL WALL.
Job Title	MATER DEI
Job No. 238866	Sketch No. SKS-017 Rev.
Made by [signature]	Date 4/12/14 Chd.

Appendix C

History of Mater Dei hospital

Appendix C – Construction History

The following is a summary of the history of the Mater Dei Hospital, it was pulled together from news stories published in The Malta Independent between 24 and 26 October 2004.

C1.1 History

C1.1.1 First phase

In early 1990s a new hospital – now called the Mater Dei Hospital – was put to concept. The new hospital was to complement the 50 year old St Luke's Hospital, Pietà, Malta. St Luke's hospital was to undergo an extensive programme of refurbishment. The new hospital was to be a 480 bed, state of the art specialised hospital to operate in the fields of diabetes, cardiology, degenerative diseases and other chronic illnesses prevalent in Malta and other Mediterranean countries. The new hospital was also to have a strong research and teaching aspect.

In 1990 the Foundation for Medical Sciences and Services (FMSS) was established by the Maltese government as an autonomous body of a non-commercial nature. This body was to provide health care services, promote medical studies through teaching and collaborate with other similar bodies.

In 1991 FMSS along with other Italian medical and research centres set up the Monte Tabor Foundation-Malta (MTFM), an Italo-Maltese centre for the promotion of scientific research and health, educational and training services.

On 15 July 1992 a letter of intent was signed between FMSS and MTFM specifying the building of a hospital to complement St Luke's Hospital, Pietà, Malta. At this point in time the hospital was referred to as 'The San Raffaele Hospital (Malta)'. The hospital was to be a specialised hospital of 450 beds.

FMSS were to provide the land, construction of structure, and provision of all equipment including medical and sanitary. MTFM were to be responsible for the design and construction supervision as well as the operation of the hospital.

The design of the hospital started in 1993, the designers were ORTESA Spa who were related to the co-founders of MTFM.

On 12 September 1995 there was construction agreement between FMSS and Skanska International Building Ltd., Blokrete Ltd, Devlands Ltd and Cassar, Grech, Ebejer & Partners. The group of companies is referred to as Skanska Malta Joint Venture (Skanska MJV). Construction commenced on the San Raffaele Hospital project on 10 October 1995.

During 1996 Skanska MJV maintained that Ortesa's design was incomplete and or needed adjustment. In July 1996 Skanska MJV put an offer to FMSS to finalise Ortesa's drawings, FMSS declined the offer. In August 1996 FMSS decided to suspend payments to MTFM due to problems with Ortesa.

C1.1.2 Second phase

In October 1996 the new Labour government commissioned a report to be able to make informed decisions about the hospital. The report recommended a new medical brief to increase the size of the San Raffaele Hospital to 980 beds, change the hospital from a specialised teaching hospital to a general acute hospital, replace St Luke's Hospital, terminate the contract with Ortesa, and keep Skanska MJV to continue construction of the San Raffaele Hospital based on Ortesa's drawings.

In April 1997 FMSS were instructed to terminate contracts with MTFM.

In March 1998 the foundation of medical services and sciences were separated into two separate organisations. The Foundation of Medical Services (FMS) which retained responsibility for health sector services and Foundation for Social Welfare Services (FSWS) which took over social welfare services.

In July 1998 Norman and Dawbarn were chosen as new designers for the new hospital.

C1.1.3 Third phase

In September 1998 the San Raffaele hospital was re-evaluated due to the election of a new government administration. The new administration decided to amend the 2nd brief of 980 beds to 650 beds with a possible extension of 825 beds. The new administration also terminated the contract with Norman and Dawbarn. The new administration decided to change the contract to a design and build style contract.

In February 2000 Skanska MJV and FMS enter into an agreement for the building, finishing and commissioning of the new hospital under a design and build contract.

In 2004 disputes about the completion date and costs of the hospital were ongoing between Skanska MJV and FMS