CORMAC

Moorfield Car Park, Truro

Structural Assessment

Infra23-178-CSL-SGN-SW824447-RP-S-002

Mott MacDonald ref: 004STR-MMD-XX-XX-T-S-0003

Cormac | Infrastructure Design - Structures

Issue & Revision Record						
Revision	**Date**	**Originator**	**Checked**	**Authorised**	**Purpose of Issue**	**Nature of Change**
P01	April 2024	[redacted]	[redacted]	[redacted]	DRAFT for initial comment	
P02	May 2024	[redacted]	[redacted]	[redacted]	For acceptance	Updated for comments
P03	May 2024	[redacted]	[redacted]	[redacted]	For acceptance	Updated for comments

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1 Introduction

1.1 Scheme background

1.1.1 Mott MacDonald (MM) has been commissioned as part of the Cormac Strategic Partnership to undertake a structural assessment of the existing Moorfield Car Park in Truro. This is due to ongoing concerns regarding the condition of the structure and its suitability for continued use.

1.1.2 This report summarises the structural assessment undertaken for the car park, its findings and recommendations.

1.2 Reference documents

1.2.1 The report should be read in conjunction with the corresponding Basis for Structural Assessment document, 004STR-MMD-XX-XX-T-S-0002 Rev P04.1 March 2024.

1.3 Report structure

1.3.1 The report structure is as follows:

- Section 2 – Description of the Structure
- Section 3 – Assessment Method
- Section 4 – Assessment Results and Commentary
- Section 5 – Conclusions and Recommendations

2 Description of the structure

2.1 Description of the structure

2.1.1 Moorfield Multi-Story Car Park is a split level car park with nine decks, constructed in 1970-71. The initial design life of the structure was likely 50 years based on standards for the time. As such the structure is life expired.

2.1.2 Each deck consists of a waffle type in-situ reinforced concrete (RC) slab which span between in-situ reinforced concrete columns. The decks are cantilevered externally around the perimeter.

2.1.3 The in-situ deck comprises a 300mm thick waffle slab using 200mm Mills M Mould, i.e the voids are 200mm deep with a 100mm thick slab. The car park grid is typically 7.2m x 8.8m. The northern and southern edges of each deck are cantilevered for a length of 3.5m.

2.1.4 The RC columns are typically 300mm x 600mm with longer dimension along North-East direction.

2.1.5 Other elements of the car park include parapet walls, which are predominantly precast RC panels, and three stairwells. The decks are arranged as shown in the following figures. Steel columns were added to support the central ramps circa 2018.

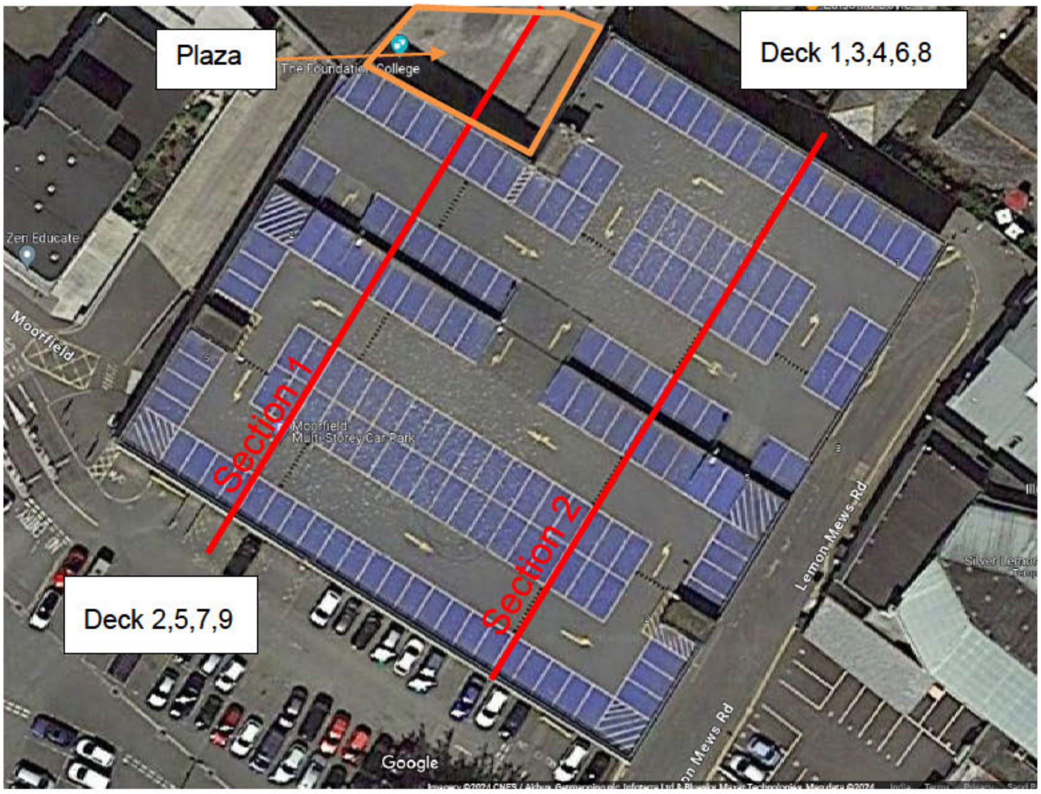

Figure 2.1 Moorfield MSCP

2.1.6 Deck 1 is partially located underneath Deck 4 but at the Northern end there is a section that is underneath a small plaza area surrounded by retail units.

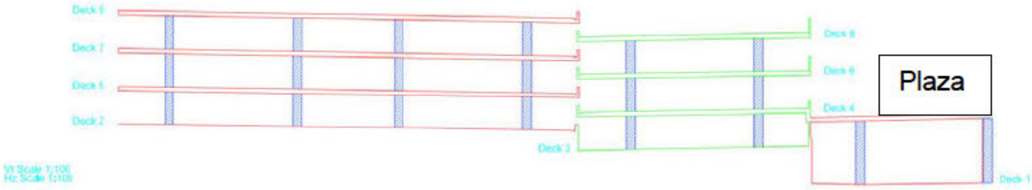

Figure 2.2 Car Park Cross Section 1

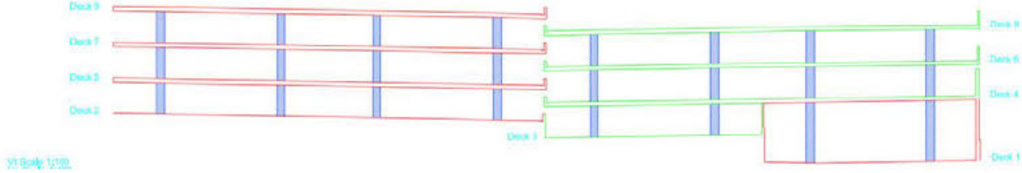

Figure 2.3 Car Park Cross Section 2

2.1.7 Deck 4 supports a small toilet block along its northern edge.

2.1.8 The structure is supported on piled foundations.

2.1.9 There are RC ramps providing access between the two halves of the structure. There are transverse "flexcel" joints at the centre of these ramps providing structural separation of the two halves of the car park.

2.1.10 In 2017/2018 steel props were added to the ramps

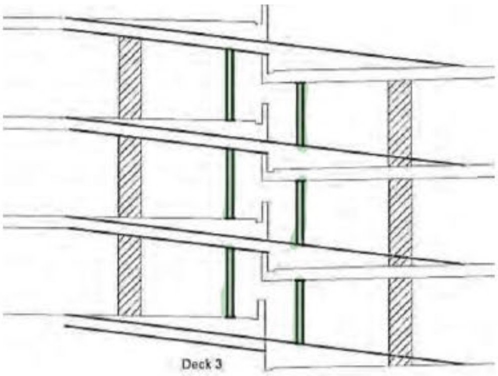

Figure 2.4 Ramp props

2.2 Previous assessment summary

2.2.1 There are no records available to demonstrate that the structure has been statically assessed previously. Therefore, the existing capacity of the structure is unknown.

2.2.2 In 2021 Airey and Coles Consulting Engineers produced a structural report that reviewed the available inspection reports and survey information. This qualitative report concurred with the findings of the survey which concluded:

2.2.3 *"Based on the age of the structure (50 years), the original design life at construction (50 years), the modified design life as recommended by the Institution of Civil Engineers (30 years) and the lack of detailed original design drawings – the structure has a maximum remaining design life of 5 years with 6 monthly ongoing inspections. The report recommends that the structure should be demolished at the earliest convenient opportunity."*

2.3 Monitoring and inspection summary

2.3.1 The structure has been regularly inspected over recent years with six monthly inspections since June 2020 and interim inspections more frequently to monitor defects for worsening.

2.3.2 Since December 2023 additional monthly monitoring on the cracks within the deck soffit has been completed to investigate rate of growth of these cracks.

2.3.3 These have identified elongation of 15 of the 68 cracks surveyed. Elongation is not following any pattern with cracks recorded as stable and then increasing in one month by up to 100mm (average 45mm). An extract from the April survey is shown in Figure 2.5. No recordings of the width of cracks has been recorded to date.

Level 3		Crack Marked						
	05/12/2023 14.0°C	L3 B101	L3 C81	L3 C82	L3 E91	L3 F91	L3 G91	L3 H91
	12/12/2023 13.4°C							
	20/12/2023 12.3°C						+72	+21
	03/01/2023 8.0°C							
	15/02/2024 14°C		+25					
	18/03/2024 13°C	+55						
	16/04/2024 10.0°C							

Level 4		L4 B11	L4 C21	L4 D21	L4 E21	L4 G22	L4 G21	L4 G23	L4 J31	L4 G51	L4 H51
	05/12/2023 14.0°C	Crack Marked									
	12/12/2023 13.4°C		+7				+50				
	20/12/2023 12.3°C										
	03/01/2024 8.0°C										
	15/02/2024 14°C					+20					
	18/03/2024 13°C										
	16/04/2024 10.0°C										

Figure 2.5 Crack monitoring results

2.3.4 Intrusive Site-Investigation Report (004STR-MMD-XX-XX-T-S-0005) provides details of the investigations carried out in February 2024 which included chloride, carbonation and Schmidt hammer testing, ferro scans and breakouts. This information is used to support section information used in the structural assessment.

3 Assessment method

3.1.1 The assessment was undertaken in accordance with the Basis for Structural Assessment document, 004STR-MMD-XX-XX-T-S-0002 Rev P04.1.

3.2 Assessment standards or guidance

3.2.1 The quantitative assessment of the RC superstructure has been undertaken in accordance with Eurocodes and UK National Annexes. The foundations have not been assessed.

3.3 Method of analysis

3.3.1 The assessment methodology and parameters used are as specified in the Basis of Assessment. The assessment is focused on the capacity of the structure under vertical loading and consider a section through a critical bay. Assessment is based on the equivalent frame method.

3.3.2 Sections were constructed in Autodesk Robot to calculate load effects and supplemented by excel and Tekla Tedds calculations.

3.3.3 Figure 3.1 shows the location of the three slices that are modelled (red lines), a transverse slice running approximately N-S, a transverse slice through the plaza and a longitudinal slice through E-W.

3.3.4 The assessment has been targeted in its approach to initial key areas to understand the capacity. A section has not been taken through the ramp, nor have the columns at the entrance to Deck 1 been considered (Figure 3.2). These columns differ from the others at Deck 1 as they continue to the underside of Deck 6 with beams connecting them to Deck 4. There is limited as built information for these beams and columns.

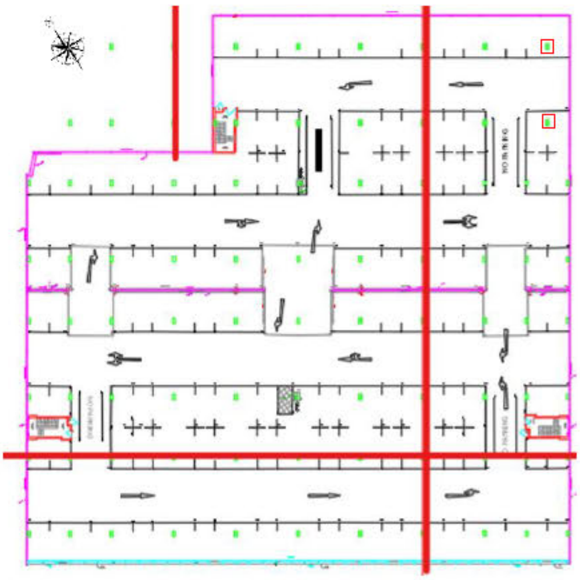

Figure 3.1 Location of slices

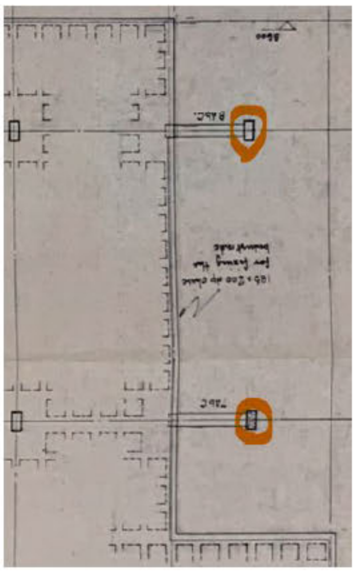

Figure 3.2 Level 1 columns

3.3.5 Figure 3.3 shows the transverse section in Autodesk Robot, the frame connections are fixed with pinned supports.

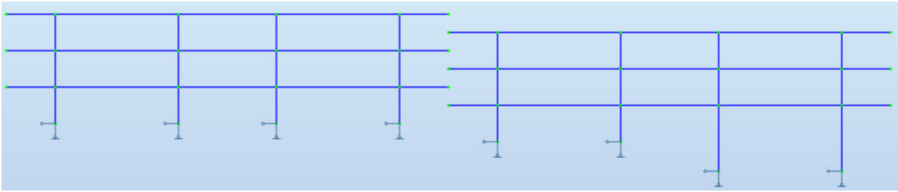

Figure 3.3 Idealised section

3.4 Assessment loading

3.4.1 The structure has been assessed for:

- Dead loading of the structural frame.
- Superimposed dead load of finishes and services.
- Imposed loading cases included:
  - Vehicle Uniformly Distributed Loading (UDL) of 2.5kN/m^2 and 2No. pattern cases in accordance with Eurocodes to create alternative bending arrangements in the slab.
  - Vehicle loading reduction to include only 2 in 3 bays occupied was also applied, with vehicle running lanes still applied as standard.
  - Loading on the roof was removed as in the current operational scenario.
  - Pedestrian loading of 5kN/m^2 over the plaza.
- Snow/maintenance loading was considered but did not contribute to the critical load cases.
- An accidental case was included with a vehicle impact load of 50kN applied to isolated columns at 0.5m from the base in accordance with BS EN 1991-1-7.

Partial factors of 1.35 for dead load and 1.5 for variable load have been applied in the assessment in accordance with Eurocodes. It is recognised that the historic BS8110 combinations are lower with 1.15 and 1.35 respectively and that as set out in Appendix 10 of Appraisal for Existing Structures (Third edition) there can sometimes be opportunities to use BS8110 values or to reduce these further. As such this the suitability of this has been considered.

For dead load this is typically suggested when elements have been accurately surveyed and actual loading from the structure is fully understood. In this case given that as-built records are incomplete, and a full measured building survey has not been carried out it is felt that a reduction is unsuitable.

For imposed loading a partial factor of 1.5 has been retained as there is recognition that the car park can be highly utilised on all open levels. Additionally, as set out in recent IStuctE guidance on car park design (June 2023), vehicles are increasing in size and weight due to trends and uptake of electric vehicles and for newer car parks a $3kN/m^2$ load could be considered. Based on the size of this car park, the height restrictions and that the EV charging points are outside, $3kN/m^2$ has not been considered but this is mitigated partly by not reducing the partial factor.

Check of the deflection of the slabs under dead and imposed load has been completed with serviceability limits of span/250. Check of the crack potential based on stress within the section is also completed using Concrete Centre Eurocode sheets (BS EN 1992-1).

3.5 Material properties

3.5.1 For the purpose of this assessment the properties have been taken as follows:

- Concrete strength for columns and slabs: C25/30
- Reinforcement for columns and slabs: $410N/mm^2$

3.5.2 Material factors have been applied in accordance with Eurocodes. Due to the age and condition of the structure it has not been felt prudent to refine these values. A Deterioration Corrosion Risk Assessment has been produced by MM following review of carbonation and chloride testing and found the results would indicate that a significant proportion of the reinforcement has been susceptible to corrosion for an unknown period of years.

3.5.3 Within the assessment of results commentary is made regarding the potential impact of low cover and elevated chloride and carbonation levels on potential reduction in capacity.

4 Assessment results and commentary

4.1 Headline results

4.1.1 **Column Capacity**

4.1.2 With regards to the columns the majority of them had sufficient capacity for loading as set out in Table 4.1. This is based on uniaxial and biaxial loading.

4.1.3 The exception to this was those under the plaza where the columns have a utilisation ratio higher than 1 in uniaxial and biaxial bending, which is considered as failed. These columns are subject to high moment effects. It is of note that these columns are also taller (4.7m) than the rest of the car park columns (2.6m).

4.1.4 There are some other columns primarily in the upper floors that also fail in biaxial loading, this is due to them having considerably less axial loading. However when no parking is applied to the upper deck these columns pass.

4.1.5 Utilisation shown in Table 4.1 were calculated based on biaxial bending as per Cl 5.8.9 of EN 1992-1-1:2004+A1:2014. Refer to Appendix B for further results tables.

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1,0$$

4.1.6 The calculations for design moment (M_{Ed}) include consideration of second order moment effects.

4.1.7 The columns pass in shear and under accidental load cases.

4.1.8 **Slab Capacity**

4.1.9 For the slab below the plaza utilisation is greater than 1 for ULS loading of the structure, with 5kN/m^2 pedestrian load and associated dead and super imposed dead loads. There is also high utilisation for service stresses in accordance with BS EN 1992-1 cl 7.4.2(2) with a high span to depth ratio, which indicates a deflection failure. There is a potential utilisation ratio higher than 1 of the end span (that over the roadway) also in failure. This is in the case where only 1 bar is present in each rib. This is something not shown in the drawings but picked up on site in some of the investigations, this is discussed further in section 4.3.

4.1.10 Refer to Table 4.2 for slabs with full 2.5kN/m^2 loading and Table 4.3 for values where only 2 in 3 bays are loaded.

4.1.11 All slabs at column locations have a utilisation of greater than 1 in punching shear, which is considered as failed, even with reduced vehicle loading to two vehicles in every bay of three spaces.

4.1.12 Refer to Appendix B for further results tables.

4.2 Results

4.2.1 Deck locations correspond to the deck that the the columns are on. i.e deck 1 columns are those visible when on deck 1 and are supporting deck 4 above. Deck 6 columns are supporting deck 8.

Column utilisation									
Location	Case	Applied moment		Rebar	Section capacity		Utilisation	Result
		$M_{yy,Ed}$	$M_{zz,Ed}$		$M_{yy,Rd}$	$M_{zz,Rd}$		
Deck 1	Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	70	104	4H25	282	124	0.93	Pass
Deck 1	Car Park_Lower Story (Max Moment Comb)- FULL LOAD	89	101	4H25	324	140	0.84	Pass
Deck 3	Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	43	62	4H25	282	125	0.45	Pass
Deck 6	Car Park_Upper Story (Max Bending Comb) – FULL LOAD	219	65	4H25	288	122	1.27	Fail
Deck 6	Car Park_Upper Story (Max Bending Comb) – 2/3 bays loaded	205	68	4H25	280	119	1.29	Fail
Deck 6	Car Park_Upper Story (Max Bending Comb) No cars 8&9 floor	84	42	4H25	257	109	0.71	Pass
Plaza (Deck 1)	Plaza (Max Axial Load Comb)- FULL LOAD	266	114	4H25	344	151	1.46	Fail
Plaza (Deck 1)	Plaza (Max Moment Comb)- FULL LOAD	375	105	4H25	277	118	2.26	Fail
Plaza (Deck 1)	9. Plaza (Max Axial Load Comb)- SDL = 0.2 kPa	238	63	4H25	336	147	1.06	Fail
Plaza (Deck 1)	10. Plaza (Max Moment Comb)- SDL = 0.2 kPa	346	33	4H25	269	114	1.28	Fail
Plaza (Deck 1)	11. Plaza (Max Moment Comb)- IL = 2.5 kPa	346	33	4H25	252	107	1.30	Fail

*Fail only with one bar, as such subject to validation of bars

Table 4.1 Column utilisation

4.2.2 Typical end span fails SLS case with both 1 and 2 bars, and flexure with 1 bar. Cases with one bar per rib and two bars per rib have been considered based on the survey where only one bar was identified in some locations. This is discussed further in section 4.3.

Slab utilisation – Full vehicle loading							
Midspan sagging		ULS utilisation ratio				ULS	SLS
Typical end span	Transverse	0.55	(2bars)	1.08	(1bar)	FAIL*	FAIL
Typical internal span	Transverse	0.33	(2bars)	0.64	(1bar)	PASS	PASS
Typical internal span	Longitudinal	0.39	(2bars)	0.76	(1bar)	PASS	PASS
Plaza midspan	Transverse	1.15	(2bars)	2.25	(1bar)	FAIL	FAIL

Hogging @ end of beam (column face) 0.3/0.15m					
Cantilever	Transverse	0.71	(4 bars)	PASS	FAIL
Typical internal	Transverse	0.35	(4 bars)	PASS	PASS
Typical internal	Longitudinal	0.57	(3 bars)	PASS	PASS
Plaza	Transverse	1.09	(4 bars)	FAIL	PASS
Hogging in waffle slab @ 0.9m from column centre – transition zone					
Typical internal	Transverse	0.32	(2 bars)	PASS	PASS
Cantilever	Transverse	0.92	(2 bars)	PASS	PASS
Typ internal	Longitudinal	0.35	(2 bars)	PASS	PASS
Plaza	Transverse	1.25	(2 bars)	FAIL	PASS

Table 4.2 Slab utilisation

Slab utilisation – Reduced vehicle loading (2/3 spaces)							
Midspan sagging		ULS utilisation ratio				ULS	SLS
Typical end span	Transverse	0.55	(2bars)	1.08	(1bar)	FAIL	FAIL
Typical internal span	Transverse	0.27	(2bars)	0.64	(1bar)	PASS	PASS
Typical internal span	Longitudinal	0.39	(2bars)	0.76	(1bar)	PASS	PASS
Plaza midspan	Transverse	1.15	(2bars)	2.25	(1bar)	FAIL	FAIL
Hogging @ end of beam (column face) 0.3/0.15m							
Cantilever	Transverse	0.64		(4 bars)		FAIL	PASS
Typical internal	Transverse	0.32		(4 bars)		PASS	PASS
Typical internal	Longitudinal	0.56		(3 bars)		PASS	PASS
Plaza	Transverse	0.53		(4 bars)		FAIL	PASS
Hogging in waffle slab @ 0.9m from column centre – transition zone							
Typical internal	Transverse	0.30		(2 bars)		PASS	PASS
Cantilever	Transverse	0.84		(2 bars)		PASS	PASS
Typical internal	Longitudinal	0.33		(2 bars)		PASS	PASS
Plaza	Transverse	1.25		(2 bars)		FAIL	PASS

Table 4.3 Slab utilisation reduced vehicle loading

Punching Shear		
Load case	Utilisation	Result
Permanent Load	0.95	PASS
Full Load	1.44	FAIL
Reduced Load	1.34	FAIL

Table 4.4 Punching shear

4.3 Commentary

4.3.1 A visual representation of the failure locations is shown in Figure 4.1.

*flexure failure only applies if there is one bar in each rib, it passes if there are two.
Visualisation overlain on previous inspection report sketch, ignore green and red hatches.

Figure 4.1 Visualisation of assessment

4.3.2 The Phase 1 inspection, summarised in memo 100105991(4)-MMD-MO-XX-RP-S-0001(B), identified cracking within the structure that is of note due to the findings of the assessment and the similarity of the zones affected:

- Cracking running longitudinally down the underside of deck 8 and 9. These cracks extend the majority of the length of the car park and occurs within the "end spans" (shown in red in Figure 4.2). This is noted as an area of concern in this assessment. This longitudinal cracking is not present within the lower decks at this time.
- Cracks extending radially from the column heads within the deck of the waffle section. This occurs around columns on all decks, with the

frequency and length greatest under the plaza and on the upper decks. This is noted as an area of concern in this assessment.

- Cracking within the soffit of the deck under the plaza, extending from the column heads. Cracks have evidence of water seepage and repairs.
- Areas of cracking / spalling on some columns and impact damage.

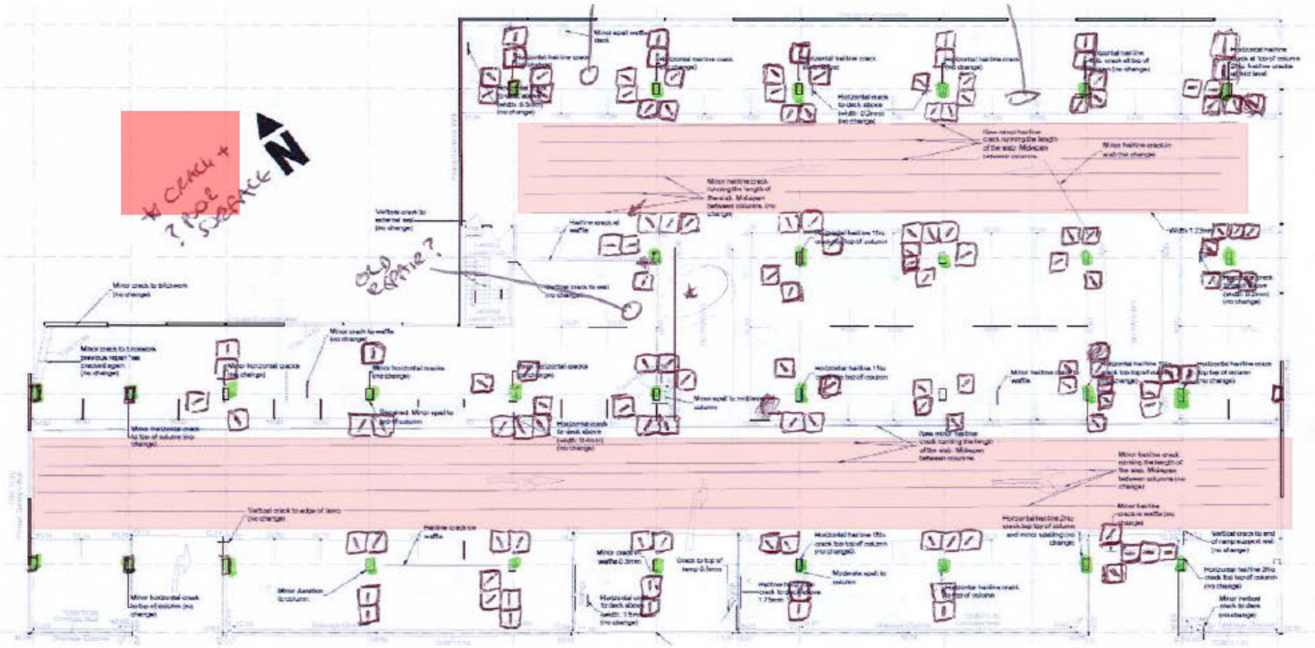

Figure 4.2 Deck 6 (underside of deck 8) inspection findings Nov 23.

4.3.3 Columns

4.3.4 Under the existing load scenario with no vehicle loading on the top decks (8 & 9) all columns within the car park, excluding the plaza, pass assessment. Retaining these top decks clear of vehicles is recommended as without this some of the columns have utilisations of greater than 1 in biaxial bending.

4.3.5 Under permanent loading only, the columns have utilisation of less than 1 and pass assessment.

4.3.6 Columns had sufficient capacity for Impact load of 50kN.

4.3.7 From visits to the car park it is clear that the columns are subject to minor impacts and, although this does not look to cause failure of the structure, the exposure of the reinforcement and repeated damage can lead to reducing capacity of the bars. If the car park spaces are adjusted to reduce the number of spaces between bays this may be favourable as there is an opportunity to move the spaces slightly away from the columns. This would lead to slightly higher midspan bending than a case with two spaces adjacent to the columns.

4.3.8 Slabs

4.3.9 The long “end” span over the carriageway is over utilised in SLS failure for stress, as is the cantilever span. This is as calculated in Concrete Centre spreadsheets based on span to depth ratios and permissible stress. The modelled deflection was maximum 10mm over 3.5m.

4.3.10 There is a potential concern regarding flexure in these 8.8m “end” spans as the site investigations identified some instances of ribs having only one 20mm bar in the bottom of the section instead of two. As identified in Table 4.2 & Table 4.3 if this is the case then the section has a utilisation greater than 1. Exact locations where there is only one bar have not been fully mapped and should it only apply to the shorter spans then the flexural failure would be removed. Investigation of these is recommended.

4.3.11 Reducing the loading on the decks by one in three car park spaces does reduce the utilisation of the structure but it is less than initially anticipated prior to the assessment as some of the worst case load effects were driven by the pattern load cases which stagger the loading within the space to create peak effects. Utilisation of the structure still exceeds 1 under the reduced load case.

4.3.12 Under permanent load the slabs have a utilisation less than 1.

4.3.13 Punching shear

4.3.14 All column heads modelled fail in punching shear under both full loading and loading with two in three spaces used.

4.3.15 There is no record of punching shear reinforcement, and it was not identified in the scans. As such assessment was completed on the basis of no additional shear reinforcement for punching shear. It may be that there is additional reinforcement as some of the scans were inconclusive which could reduce this utilisation. Locating shear reinforcement using non-destructive methods is very difficult and breakout is not recommended.

4.3.16 Under permanent loading only, the slabs have a utilisation of 0.95 in punching shear, which although lower then 1 is still fairly high. This does however mean that where the decks are closed to live loading by vehicles (such as decks 8 & 9) they do just pass assessment.

4.3.17 Punching shear is driven by the load effects in the slab and is not a result of the weight of the structure above. As such removal of upper decks of the car park would not remove the failure from the remaining levels. Therefore removing any unused decks is not suggested as a mitigation based on the assessment.

4.3.18 <u>Plaza</u>

4.3.19 The plaza loading is greater than that of the other deck areas with $5kN/m^2$ pedestrian load and $1.35kN/m^2$ super imposed dead load (SDL) applied. This higher SDL load was applied as there is an unknown depth of fill and surfacing on top of the slab. There may be an opportunity to reduce this loading following investigation of the fill/surfacing depth. Additionally a reduction in the pedestrian loading could be considered if the area is fenced off or if it can be shown that the area only has light pedestrian usage.

4.3.20 Reduction of the SDL super imposed and imposed load was investigated (Case 9-11 in Table 4.1), but still record failure of the column under these reduced load cases.

4.3.21 As there is only one column section provided in the as-built information and limited investigation of these columns was carried out it may also be considered to carry out additional ferro scanning and breakouts on these longer columns. The Ferroscan that was undertaken did identify two bars within the corner however it was inconclusive if these were starter bars or not, and as such they have not been relied upon in the calculation as it is likely a lap zone. This may resolve the failure in the column.

4.3.22 For the slab, additional tables in Appendix B show that under the reduced superimposed load, which may apply if the fill thickness is lower then currently assumed, the slab still has a high utilisation. A reduction in pedestrian load would result in a utilisation of below 1.0.

4.3.23 <u>Progressive collapse</u>

4.3.24 Analysis of progressive collapse has not been carried out as part of this assessment. However from a review of the form and known failure modes for these types of structures there is a risk of progressive collapse either vertically or horizontally through the car park, and example of which is shown in Figure 4.3.

4.3.25 If a slab were to fail in punching shear, the additional loading on the slabs below due to the weight of the failed deck would have potential to cause progressive vertical failure.

4.3.26 Similarly, the failure could progress horizontally along the deck length, as the failure of the slab at one column increases the load at the adjacent column, causing a further punching shear failure.

4.3.27 Modern slab construction is typically designed with a minimum amount of tying reinforcement to increase resilience against this kind of failure. Due to the limited information it is not possible to determine if this has been included, and so there is a risk of progressive collapse if the potential failure modes are not mitigated.

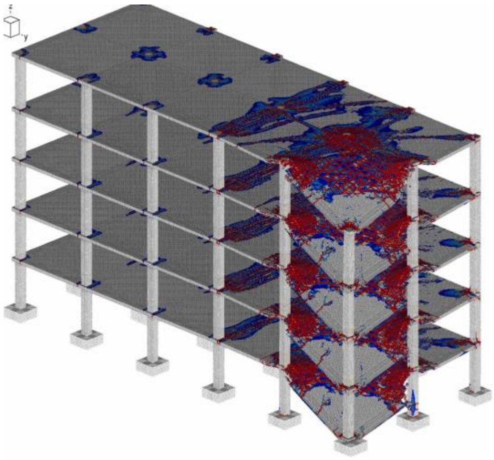

Figure 4.3 Progressive collapse example (theconstructor.org)

4.3.28 Global Stability

4.3.29 Assessment of the global stability has not been completed.

4.3.30 Condition

4.3.31 The section capacities calculated in this assessment are based on the record information for the structure. The Deterioration and Corrosion Risk Assessment (004STR-MMD-XX-XX-T-S-0005) identified that a significant proportion of the reinforcement has been susceptible to corrosion for an unknown period of time, particularly in the columns. Refer to Corrosion Risk Assessment for further discussion of findings.

4.3.32 There is potential that there has been corrosion of the reinforcement and reduction in the capacity of some of the elements, in particular in locations of cracks with evidence of water ingress. It is difficult to quantify the extent of this impact.

4.3.33 Where areas have an utilisation of greater than 1 this would only worsen if the capacity is reduced due to corrosion.

4.3.34 For much of the rest of the structure the utilisations are <0.9 and in some cases lower than 50% as such a reduction in capacity associated with even carbonation induced corrosion of the bars of a couple of mm may not cause these areas to become overstressed but would increase the utilisation.

4.3.35 An example of this is for mid span sagging where a reduction in the bars from two 20mm bars to two 16mm bars sees a reduction in capacity from 52 to 33.5kNm This would see the following changes, with increasing utilisation for all sections with continued failure under the plaza.

Slab utilisation – Reduced vehicle loading (2/3 spaces)						
Midspan sagging		My/width (kNm)	(2 * 20mm bars)		(2 * 16mm bars)	
			Capacity (kNm)	Utilisation	Capacity (kNm)	Utilisation
Typical end span	Transverse	28.4	52	0.55	33.5	0.84
Typical internal span	Transverse	14.2	52	0.27	33.5	0.42
Typical internal span	Longitudinal	20.2	52	0.39	33.5	0.60
Plaza midspan	Transverse	59.4	52	1.15	33.5	1.77

Figure 4.4 Revised load effects

4.3.36 In the case of chloride induced corrosion within crack locations pitting opposed to even corrosion occurs which can cause significant localised section loss. The above utilisation reduction calculations are not applicable for chloride induced corrosion as the reduction in capacity is likely to be significantly greater. Areas of chloride ingress and corrosion have been observed, refer to the Corrosion Risk Assessment report.

4.3.37 As noted above removal of any closed decks is not considered to be a mitigation against the potential failure modes, however if the condition of these upper decks did deteriorate significantly then this could be considered from a safety case.

5 Conclusions and recommendations

5.1 Conclusions

5.1.1 The car park was constructed in 1970-71 and based on standards for the time is expected to have had a 50 year design life. As such the structure is life expired by three years. This is extended to 23 years life expired when considering the Institution of Civil Engineers document “Recommendations for the Inspection, Maintenance and Management of car park structures”, 2nd Edition which suggests a car park of this time with limited record information may have a reduced design life of approximately 30 years.

5.1.2 The following conclusions are made from the assessment for the as-built capacity:

Columns

- The majority of the columns have sufficient capacity in uniaxial bending.
- The columns under the plaza have a utilisation ratio greater than 1 which is considered over capacity.
- In biaxial bending the upper deck columns, those supporting decks 8 and 9, fail under full and reduced (2/3 spaces) loading. This is due to lower axial loading on these columns. When these decks are not subject to vehicular loading, as in their current closed state, they pass. All other columns pass biaxial checks in both full and reduced load cases.
- The columns are sufficient for 50kN impact loading.
- The columns pass assessment for permanent load only.

Slabs

- The 8.8m end span and cantilever spans are subject to increased stresses and exceedance of the capacity under SLS.
- Subject to validation of the reinforcement within the ribs this area may also exceed the flexural capacity for the “end” 8.8m spans if only 1 bar is present in each rib.
- The slab supporting the plaza has insufficient capacity in flexure. However due to potential conservatisms in the assessment it is suggested this area should be subject to further investigation of the reinforcement and the fill depth.
- The slabs pass assessment for permanent load only.

Punching shear

- Investigations were inconclusive as to the size and spacing of punching shear reinforcement over the columns as such the assessment has been carried out without shear reinforcement. The assessment indicates that the slab loading exceeds the punching shear limits in all locations under both full loading and that due to 2 in 3 spaces being occupied.

- Under permanent load only the slab passes assessment for punching shear.

5.1.3 Cracking identified during inspections of the structure is noted as an area of concern in this assessment.

- Running longitudinally down the underside of deck 8 and 9. These cracks extend the majority of the length of the car park and occurs within the "end spans".
- Radially from the column heads within the deck of the waffle section. This occurs around columns on all locations, with the frequency and length greatest under the plaza and on the upper decks.
- Under the plaza.

5.1.4 Carbonation and chloride testing has indicated that areas of reinforcement are susceptible to corrosion. This could further reduce the capacity of the structure.

5.1.5 The structure passes assessment for permanent loading only. This means that in the case of closure of the car park the outdoor parking spaces can be utilised.

5.1.6 <u>Summary</u>

5.1.7 The car park has exceeded its original design life. There are signs of ongoing uncontrolled cracking to slab soffits, and evidence of carbonation and chloride ingress to concrete which suggest ongoing degradation of the structure. Assessment of the main slab and columns indicates that the structure was designed with minimal, if any, spare capacity. As such, ongoing loading of the slab and further degradation of reinforcement by carbonation or chloride induced corrosion could lead to failure of the slabs.

5.1.8 The potential failure modes identified are:

- Punching shear failure.
- Flexural failure.

5.1.9 <u>Likelihood</u>

- Punching - No punching shear reinforcement was located as part of the non-intrusive investigations performed as part of this study. Therefore it is not possible to ascertain the type or specification of punching reinforcement around columns without record information. A safe assumption is therefore that the structure was designed without residual capacity, and that corrosion past that expected in the original design life period has already occurred, leaving the structure at risk of collapse without intervention. Punching failure is a brittle failure with no visible warning and so it is not possible to predict when this could occur.

The report "Enhancing the Whole Life Structural Performance of Multi_Storey Car Parks (Office of the Deputy Prime Minister), 2002" identified that Inadequate factors of safety in old codes and in BS8110 12 make this a particularly sensitive failure mode for structures of this age.

Flexure - There is evidence of soffit cracking at positions of high flexural stresses, indicative of potential overstressing of reinforcement, either by overloading compared to the original design, or by ongoing degradation of reinforcement due to corrosion. These cracks will in turn provide further pathways for corrosion. Flexural failure is a ductile failure and so collapse is most often preceded by significant visible cracking and deflection of the slab. Closure of deck 8 & 9 has been a reasonable action in response to recording these cracks.

Similar cracking on other decks would indicate concerns regarding the flexural capacity and should result in further deck closures to address this failure mode. Other critical issues identified have raised additional concerns and should also be addressed prior to this becoming the remaining trigger point for closure.

5.1.10 Based on this assessment, available information and condition of the structure the car park slabs are considered to be at high risk of failure.

5.2 Recommendations

5.2.1 **Based on this assessment it is recommended that the car park is closed due to the high risk of failure and then demolished at the earliest convenient opportunity.**

5.2.2 Whilst it is recognised that the car park is a critical part of the infrastructure within Truro, potential continued operation of the car park is a safety risk and mitigation measures should therefore be considered to reduce the high risk to a level that is acceptable to the Council:

1. Close the car park – demolish in line with previous advice. Assessment of the interim derelict state could be considered. This would achieve the lowest residual risk.

2. Further investigations and assessment:

- Locate further record information for punching shear reinforcement to allow assessment.
- Further non-destructive investigations (Ferroscan and GPR surveys) to determine presence of punching shear reinforcement on slab around columns. There is a risk that this may still provide inconclusive results with regards to punching shear reinforcement layout.

- Further non-destructive investigations (Ferroscan surveys) to determine the number of bars within the waffle slab ribs (to verify risk of flexural failure).
- Further detailed assessment on risk of ongoing corrosion to top and bottom surface of slab around columns via half-cell surveys (to determine level of risk of failure in punching shear).
- Re-assessment of frame with further reduced loading:
  - Further reductions in cars per bay;
  - Further closure of decks;
  - Removal of load from the cantilever spans.
- It is unlikely that investigations and assessment alone will be enough to allow the car park to remain fully open without risk, and interim remedial action may be required subject to option progressed.
- Further investigation works under the plaza, of the plaza slab built-up (fill and surfacing thickness) and of the ribs on the 8.8m spans throughout the structure to confirm section properties. Review of the assessment based on these findings.
- Investigation of the double height columns, and beams at the entrance to Deck 1 to confirm geometry and reinforcement details.
- Carry out a risk-cost-benefit analysis (residual risk vs cost of further investigations / assessment and implementation of mitigation measures vs income from car park and value of critical infrastructure asset).

To mitigate the risks during period of further investigation and assessment it is recommended the car park is closed.

3. Mitigate risk of failure

- Punching shear – Construct new punching shear heads, as shown in Figure 5.1 to strengthen the structure, or installation of props adjacent to column locations.
- Flexural failure – Load reduction and/or soffit strengthening options such as construction of down stand beams or installation of fibre reinforced polymer (FRP) strengthening methods to the underside of the frame.
- Repair areas of spalled concrete and seal all visible cracks to reduce potential pathways for ongoing corrosion and apply electrical chemical technique to halt ongoing corrosion, such as cathodic protection or corrosion inhibitors.
- Retain the closure of the upper decks (8 & 9).
- Close further decks.

- Reduce the numbers of cars on the deck.
- Remove or reduce the pedestrian loading from the plaza or determine a safe access route based on a structural assessment.
- Ongoing regular inspections of the structure including of the crack locations. The cracks should be monitored for thickness and length in all locations to determine cause and ongoing risk. Trigger points for crack growth should be provided to inform load reduction/closure.
- Opportunities for a wider cathodic protection system to reduce risk of ongoing corrosion to reinforcement and extend residual life of structure after strengthening could be explored.

Figure 5.1 Shear collar for columns

APPENDIX A - CERTIFICATE OF ASSESSMENT

Project Details

Name of Project:	Moorfield Car Park
Name of Structure:	Moorfield Car Park
Structure Ref No:	N/A

Section 1

We certify that reasonable professional skill and care has been used in the preparation of the assessment of the Moorfield car park with a view to securing that:

a. It has been assessed in accordance with the Basis for Structural Assessment document, 004STR-MMD-XX-XX-T-S-0002 Rev P04.1.
b. The assessment capacity of the structure is as follows:
   The current assessment has found that the structure has:
   - Utilisations <1.0 in Permanent Loading only
   - Utilisations >1.0 under ULS and SLS loading in punching shear and flexural capacity.

Signed	[redacted]
	Assessment Team Leader
Name	[redacted]
Engineering Qualifications	MEng MICE
Name of Organisation	Mott MacDonald Ltd
Date	16/05/2024

Signed	[redacted]
Name	[redacted]
Position Held	Associate Structures, South West Team Lead
Engineering Qualifications	CEng MIStructE
Name of Organisation	Mott MacDonald Ltd
Date	16/05/2024

Section 2

The results of the assessment and the signed certificate by the assessment team leader are in accordance with the criteria outlined in Section 1 are agreed.

The certificate is accepted by the TAA.

Signed ____________________

Name ____________________

Position Held ____________________

Engineering Qualifications ____________________

TAA ____________________

Date ____________________

APPENDIX B - RESULTS TABLES

Column Forces

Location	Case	Axial (P)				M_{yy}				M_{zz}			
		G_k	Q_k	SLS	ULS	G_k	Q_k	SLS	ULS	G_k	Q_k	**SLS**	ULS
Deck 1	3. Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	893	430	1323	1850	23	1.3	21.3	28.6	11	7	18	25
						0	0	0	0	0	0	0	0
Deck 1	4. Car Park_Lower Story (Max Moment Comb)- FULL LOAD	893	193	1086	1495	23	26.4	49	70	10	10	20	28
						0	0	0	0	0	0	0	0
Deck 3	1. Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	892	429	1322	1848	26	3	23	31	11	7	18	25
						0	0	0	0	0	0	0	0
Deck 6	2. Car Park_Upper Story (Max Bending Comb) – FULL LOAD	255	61	316	435	61	89	150	216	21	22	43	62
						-43	-59	-102	-147	-19	-20	-39	-55
Deck 6	7. Car Park_Upper Story (Max Bending Comb) – 2/3 bays loaded	256	73	330	457	57	83	140	202	21	22	43	62
						-44	-58	-101	-145	-19	-20	-39	-55
Deck 6	8. Car Park_Upper Story (Max Bending Comb) No cars top floor	258	0	258	347	57	6	63	86	22	6	28	39
						-44	-35	-79	-111	-19	-12	-31	-44
Plaza (Deck 1)	5. Plaza (Max Axial Load Comb)- FULL LOAD	358	310	668	948	96	83	180	255	11	7	18	29
						0	0	0	0	0	0	0	0
Plaza (Deck 1)	6. Plaza (Max Moment Comb)- FULL LOAD	155	158	313	446	96	158	255	368	10	10	20	28
						0	0	0	0	0	0	0	0
Plaza (Deck 1)	9. Plaza (Max Axial Load Comb) - SDL = 0.2 kPa	285	310	595	849	77	83	160	228	15	7	22	29
						0	0	0	0	0	0	0	0
Plaza (Deck 1)	10. Plaza (Max Moment Comb) - SDL = 0.2 kPa	123	158	281	404	77	158	235	341	10	10	20	28
						0	0	0	0	0	0	0	0
Plaza (Deck 1)	11. Plaza (Max Moment Comb) - IL = 5 kPa	155	79	234	327	96	79	175	249	10	10	20	28
						0	0	0	0	0	0	0	0

*For each location values are given at top (upper row) and bottom (lower row) of the column

Column utilisation

Location	Case	Applied moment		Rebar	Section capacity		Utilisation	Result
		$M_{yy,Ed}$	$M_{zz,Ed}$		$M_{yy,Rd}$	$M_{zz,Rd}$		
Deck 1	Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	70	104	4H25	282	124	0.93	Pass
Deck 1	Car Park_Lower Story (Max Moment Comb)- FULL LOAD	89	101	4H25	324	140	0.84	Pass
Deck 3	Car Park_Lower Story (Max Axial Load Comb)- FULL LOAD	43	62	4H25	282	125	0.45	Pass
Deck 6	Car Park_Upper Story (Max Bending Comb) – FULL LOAD	219	65	4H25	288	122	1.27	Fail
Deck 6	Car Park_Upper Story (Max Bending Comb) – 2/3 bays loaded	205	68	4H25	280	119	1.29	Fail
Deck 6	Car Park_Upper Story (Max Bending Comb) No cars 8&9 floor	84	42	4H25	257	109	0.71	Pass
Plaza (Deck 1)	Plaza (Max Axial Load Comb)- FULL LOAD	266	114	4H25	344	151	1.46	Fail
Plaza (Deck 1)	Plaza (Max Moment Comb)- FULL LOAD	375	105	4H25	277	118	2.26	Fail
Plaza (Deck 1)	9. Plaza (Max Axial Load Comb)- SDL = 0.2 kPa	238	63	4H25	336	147	1.06	Fail
Plaza (Deck 1)	10. Plaza (Max Moment Comb)- SDL = 0.2 kPa	346	33	4H25	269	114	1.28	Fail
Plaza (Deck 1)	11. Plaza (Max Moment Comb)- IL = 2.5 kPa	346	33	4H25	252	107	1.30	Fail

*M_{Ed} values include second order effects as calculated within Tekla Tedds, these are therefore larger than the values in the table above.

Case 1

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	282	43	0.15	PASS
Moment capacity (z)	kNm	125	62	0.50	PASS
Biaxial bending				0.45	PASS
Shear capacity (y)	kN	146	11	0.08	PASS
Shear capacity (z)	kN	146	103	0.70	PASS

Case 2

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	288	219	0.76	PASS
Moment capacity (z)	kNm	122	65	0.53	PASS
Biaxial bending				1.27	FAIL
Shear capacity (y)	kN	145	45	0.31	PASS
Shear capacity (z)	kN	145	140	0.97	PASS

Case 3

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	282	70	0.25	PASS
Moment capacity (z)	kNm	124	104	0.84	PASS
Biaxial bending				0.93	PASS
Shear capacity (y)	kN	146	10	0.07	PASS
Shear capacity (z)	kN	146	10	0.07	PASS

Case 4

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	324	89	0.27	PASS
Moment capacity (z)	kNm	140	101	0.72	PASS
Biaxial bending				0.84	PASS
Shear capacity (y)	kN	146	10	0.07	PASS
Shear capacity (z)	kN	146	15	0.10	PASS

Case 5

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	344	266	0.77	PASS
Moment capacity (z)	kNm	151	114	0.75	PASS
Biaxial bending				1.46	FAIL
Shear capacity (y)	kN	146	50	0.34	PASS
Shear capacity (z)	kN	146	55	0.38	PASS

Case 6

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	277	375	1.35	FAIL
Moment capacity (z)	kNm	118	105	0.89	PASS
Biaxial bending				2.26	FAIL
Shear capacity (y)	kN	138	50	0.36	PASS
Shear capacity (z)	kN	137	80	0.58	PASS

Case 7

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	280	205	0.73	PASS
Moment capacity (z)	kNm	119	68	0.57	PASS
Biaxial bending				1.29	FAIL
Shear capacity (y)	kN	140	50	0.36	PASS
Shear capacity (z)	kN	139	135	0.97	PASS

Case 8

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	257	84	0.33	PASS
Moment capacity (z)	kNm	109	42	0.39	PASS
Biaxial bending				0.71	PASS
Shear capacity (y)	kN	127	30	0.24	PASS
Shear capacity (z)	kN	124	62	0.50	PASS

Case 9

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	336	238	0.71	PASS
Moment capacity (z)	kNm	147	63	0.43	PASS
Biaxial bending				1.06	FAIL
Shear capacity (y)	kN	146	50	0.34	PASS
Shear capacity (z)	kN	146	55	0.38	PASS

Case 10

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	269	346	1.28	FAIL
Moment capacity (z)	kNm	114	33	0.29	PASS
Biaxial bending capacity check is not required					
Shear capacity (y)	kN	133	50	0.37	PASS
Shear capacity (z)	kN	132	55	0.42	PASS

Case 11

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	252	253	1.00	FAIL
Moment capacity (z)	kNm	107	32	0.30	PASS
Biaxial bending				1.30	FAIL
Shear capacity (y)	kN	124	50	0.40	PASS
Shear capacity (z)	kN	121	55	0.45	PASS

Slab utilisation – Full vehicle loading

Midspan sagging		Gk My (kNm)	Qk My (kNm)	Total My (kNm)	My/width (kNm)	Width (mm)	Utilisation				ULS	SLS
Typical end span	Transverse	108.54	73.82	257.04	28.6	0.8	0.55	(2bars)	1.08	(1bar)	FAIL	FAIL
Typical internal span	Transverse	58.19	48.96	152	16.9	0.8	0.33	(2bars)	0.64	(1bar)	PASS	PASS
Typical internal span	Longitudinal	80.93	47	179.58	20.0	0.8	0.39	(2bars)	0.76	(1bar)	PASS	PASS
Plaza midspan	Transverse	189.53	190.08	534.31	59.4	0.8	1.15	(2bars)	2.25	(1bar)	FAIL	FAIL
Hogging @ end of beam (column face) 0.3/0.15m												
Cantilever	Transverse	265.86	92.16	499.05	69.3	1.0	0.71	(4 bars)			PASS	FAIL
Typical internal	Transverse	115.29	61.39	249.22	34.6	1.0	0.35	(4 bars)			PASS	PASS
Typical internal	Longitudinal	132.64	69.6	279.6	38.8	1.0	0.57	(3 bars)			PASS	PASS
Plaza	Transverse	289.78	250.31	768.25	106.7	1.0	1.09	(4 bars)			FAIL	PASS
Hogging in waffle slap @ 0.9m from column centre – transition zone												
Typical internal	Transverse	56.89	30.07	121.94	13.5	0.8	0.32	(2 bars)			PASS	PASS
Cantilever	Transverse	190.14	60.84	349.83	38.9	0.8	0.92	(2 bars)			PASS	PASS
Typ internal	Longitudinal	52.69	28.58	118.62	13.2	0.8	0.35	(2 bars)			PASS	PASS
Plaza	Transverse	177.36	153.2	473.36	52.6	0.8	1.25	(2 bars)			FAIL	PASS

Slab utilisation – Reduced vehicle loading (2/3 spaces)

Midspan sagging		Gk My (kNm)	Qk My (kNm)	Total My (kNm)	My/ width (kNm)	Width (mm)	Utilisation				ULS	SLS
Typical end span	Transverse	108.54	72.8	255.51	28.4	0.8	0.55	(2bars)	1.08	(1bar)	FAIL	FAIL
Typical internal span	Transverse	58.19	32.64	127.49	14.2	0.8	0.27	(2bars)	0.54	(1bar)	PASS	PASS
Typical internal span	Longitudinal	80.93	48.3	181.65	20.0	0.8	0.39	(2bars)	0.76	(1bar)	PASS	PASS
Plaza midspan	Transverse	189.53	190.08	534.31	59.4	0.8	1.15	(2bars)	2.25	(1bar)	FAIL	FAIL
Hogging @ end of beam (column face) 0.3/0.15m												
Cantilever	Transverse	265.86	61.44	452.77	62.9	1.0	0.64	(4 bars)			PASS	FAIL
Typical internal	Transverse	115.29	46.32	226.44	31.5	1.0	0.32	(4 bars)			PASS	PASS
Typical internal	Longitudinal	132.64	64.4	275.63	38.3	1.0	0.56	(3 bars)			PASS	PASS
Plaza	Transverse	289.78	250.31	768.25	106.7	1.0	1.09	(4 bars)			FAIL	PASS

Hogging in waffle slap @ 0.9m from column centre – transition zone										
Typical internal	Transverse	56.89	25.44	114.43	12.7	0.8	0.30	(2 bars)	PASS	PASS
Cantilever	Transverse	190.14	40.56	316.62	35.2	0.8	0.84	(2 bars)	PASS	PASS
Typ internal	Longitudinal	52.69	26.61	111.04	12.3	0.8	0.33	(2 bars)	PASS	PASS
Plaza	Transverse	177.36	153.2	473.36	52.6	0.8	1.25	(2 bars)	FAIL	PASS

Plaza utilisation – alternative cases

Midspan sagging	Gk My (kNm)	Qk My (kNm)	Total My (kNm)	My/ width (kNm)	Width (mm)	Utilisation				ULS
0.2kN/m² SDL 5kN/m² imp.	150.88	190.08	482.81	53.65	0.8	1.03	(2bars)	2.06	(1bar)	FAIL
0.2kN/m² SDL 2.5kN/m² imp.	150.88	95.04	342.16	38.02	0.8	0.73	(2bars)	1.46	(1bar)	PASS*
1.35kN/m² SDL 2.5kN/m² imp.	189.53	95.04	393.97	43.77	0.8	0.84	(2bars)	1.68	(1bar)	PASS*
Hogging @ end of beam (column face) 0.3/0.15m										
0.2kN/m² SDL 5kN/m² imp.	231.15	250.83	688.30	95.60	1.0	0.98	(4 bars)			PASS
0.2kN/m² SDL 2.5kN/m² imp.	231.15	125.42	500.18	69.47	1.0	0.71	(4 bars)			PASS
1.35kN/m² SDL 2.5kN/m² imp.	289.78	125.42	579.31	80.46	1.0	0.82	(4 bars)			PASS
Hogging in waffle slap @ 0.9m from column centre – transition zone										
0.2kN/m² SDL 5kN/m² imp.	141.18	153.20	420.40	46.71	0.8	1.11	(2 bars)			FAIL
0.2kN/m² SDL 2.5kN/m² imp.	141.18	76.60	305.49	33.94	0.8	0.81	(2 bars)			PASS
1.35kN/m² SDL 2.5kN/m² imp.	177.36	76.60	354.34	39.37	0.8	0.94	(2 bars)			PASS

* assuming 2 bars

Punching Shear

Load case	Ved (kN)	VRd (kN)	Utilisation	Result
Permanent Load	404	429	0.95	PASS
Full Load	618	429	1.44	FAIL
Reduced Load (2/3)	575	429	1.34	FAIL