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Structural Characterisation of the Failed Manafwa Historic Bridge in Uganda

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Abstract: ***A visual inspection was carried out on thetwo-spanManafwa River Bridge along Mbale - Tororo Road to assess the extent of damage caused by traffic to the bridge superstructure. The superstructure slab developed a hole between two longitudinal steel beams located adjacent to the carriageway centreline on the Mbale bound lane on the Tororo span. At the time of inspection, the maximum length of opening parallel to traffic flow direction was 2530mm and was 1350mm in the normal direction. There is noticeable cracking of the deck surfacing at acorresponding location on the Mbale span. This report has a detailed analysis of the superstructure and the proposed research undertaken entailed the structural characterisation survey of the Manafwa Bridge which has recently shown signs of failure and for structural integrity comparisons evaluated using a finite element analysis method, ANSYS. In summary, the existing slab needs top reinforcement to resist hogging moments over longitudinal beams and at cantilever footwalk edges. The existing bottom reinforcement is adequate for sagging moments and punching shear requirements. The shear capacity of the longitudinal steel beams is satisfactory and can sustain loads greater than thecombination of HA and HB37.5.***

Keywords: Structural Engineering, Concrete Structures, Nondestructive Testing and Evaluation, Behavior of Structures, Numerical Modelling of Structures

1. Introduction

Uganda is bestowed with beautiful vegetation and wildlife, forests, water bodies and the sunshine that keep the environs and atmosphere cool all the year round. On the other hand, natural limitations to mobility and suppleness occur owing to Uganda's endowments. This ispreciselycorrect for all the lakes and rivers that oblige and give need to expensive bridges to enable manoeuvres and movements.

This chapterprovides the background to this proposal and defines theprimary research objectives, thescope of work and the limitations.

1.1 Background to the study

In Uganda majority of the bridges on the Uganda National Roads Authority (UNRA) network were constructed by the British during the colonial periods in the middle 20th century (from early 1930 to1970). UNRA, the body, charged with the national road network and bridges was established by an act of parliament in 2006 and is mandated with developing, expanding and maintaining the national roads network. From its establishment, many other bridges have been constructed in recent years especially on the district road network bridges and the national road network owing to the expansion of the road network. The bridges were designed to carry verylittle traffic loads, yet they continue carrying loads that were not initially or originally designed.

The majority of the bridges are considered historically critical and significant. Nevertheless they are continually decreasing and deteriorating in performance with many collapsing and hence reducing in number because of their degrading and unserviceable condition. Maintaining and conserving these bridges is quite a difficult task as many amongst them have materials and structural forms that are not related to the21st-centurybridges and that the current breed of engineers is unfamiliar with the technologies used. An improved and better understanding of the materials utilized for the historic bridges and the structural behaviour and responses in comparison to those innew bridges constructed is very critical to the evaluation and preservation of these bridges.

As such, the proposed research to be undertaken willentail the structuralcharacterisation survey and performance evaluation of the Manafwa bridge (a historic bridge constructed in the early 1950's) located in the Eastern part of Uganda which has recently shown signs of failure and for structural.

1.2 Statement of the Research Problem

With an overall road network encompassing of over 20,000km of national highways, 13,000km of district roads, 2,800km of urban roads and 30,000km of community access roads, Uganda's motorway linkage has significantly increased and expanded. At the time of independence in 1962, the motorway road network comprised of about 400km. Uganda lays claim to having over 400 bridges, majority of which were constructed using obsolete and antediluvian technology in the colonial periods when the country comprised of only about 5,000 vehicles (the biggest type being salon cars). The railway network then was the principal means of transport for the heavy goods and raw materials used in the local industry for production. As such the bridges were mostly designed for accessibility and mobility, and thus there is aneed to assess the structural capacity and performance.

The bridges discussed above were built using mainly concrete, with some even using timber. With this in mind, it is possible that the structural capacity of these bridge may have been compromised. In a press conference in 2012, the Minister of Works and Transport responsible for the Bridges in Uganda showed concern that there is a build-up of over 120 bridges that urgently require replacement and repair to make them able to cope with the current traffic growth and axle load (Ogwang, 2012).The majority of the steel type bridges transferred to Uganda from the United Kingdom in the colonial era for use in the Second World War are collapsing or showing signs of failure attributed to fatigue-correlated damage. For instance, the Mitaano Bridge on the Rukungiri-Mitaano-Kanungu highway that connects Uganda to the Democratic Republic of Congo collapsed in 2012 under the weight of a trailer overloaded with 1,000 bags of cement[1].

This case of thecollapse of a bridge is not isolated one as many other such bridges have similarly broken down in Aswa, Manafwa, Malaba, Kanungu, and elsewhere. Unfortunately, the roads they service are the preferred and most accessible means for both cargo and passengers. In a similar fashion, the Aswa Bridge on the Lira-Pader border collapsed showing further the wrong state of Bridges countrywide. The old bridge 79 years old was constructed in 1936 at Puranga in Northern Uganda by the British colonialists. Sources from the leading government newspaper company in Uganda, New Vision confirmed that all the post-independence governments had not taken an effort in giving due attention to the bridge, carrying out any maintenance works or upgrading of the out-of-date bridges (Aswa Bridge collapses, 2015, October 21). The bridge collapsed because of a heavily-loaded trailer that was carrying over 600 bags of cement heading to Kitgum. The trailer truck broke midway through the bridge and plunged into the river.

Another bridge that collapsed in the recent years is the Manafwa Bridge constructed 60 years back, and that is located in Busiu Sub County located in Mbale District. A part of the bridge located on the Mbale – Tororo Highway curved in leaving a colossal hole in the middle of the bridge resulting inparalysis of traffic along the road that links the Northern part of Uganda and Southern Sudan to the Central part of Uganda. Because of the collapse, vehicles on the road was disturbed with only motorcycles, bicycle riders and pedestrians able to use the small portion of the road remaining intact. A much longer diversion created through the Shikoye Lukhonge to Busiu route is an impediment to traffic and transport of goods and raw materials hence impacting on the economy significantly. To make matters worse, the bridge on the diversion route at Namwalye also curved in and was subsequently frail just three days later indicating the extent to which the evaluation of Bridges in Uganda is paramount and must be undertaken as soon as possible. The research is carrying out will aim to look for how to attain the 100-year design life for themajority of the Bridges constructed in Uganda and look at how the new bridges built can be preserved to achieve a longer design life.

Figure 1: Cracks in the Manafa bridge (The slab may not have been connected to the steel girder using studs)

1.3 Research Objectives

1.3.1 Main Objectives

The overall purpose of this study is to investigatethe structural characterization and performance evaluation of the failed Manafwa historic bridge in Uganda.(2)(3)

1.3.2 Specific Objectives

The specific aims of the proposal are as follows:

1) To determine the material and physical characteristics of the failed historical bridge
2) To numerically determine the initial and residual structural performance of the historic bridge
3) To experimentally determine the structural failure mechanism of the bridge

2. Experimental Results

2.1 Bridge Superstructure Inspections

I travelled to bridge site and carried out visual inspections of the bridge superstructure and substructures and the temporary by-pass bridge / road that was already in use by traffic from which the traffic inventory was carried out. The methodology adopted is as in the previous chapter. The findings of the visual inspection of the failed bridge and the by-pass bridge are summarized below:

2.1.1 Bridge Substructure

Generally, the substructure including the abutments, piers and the foundation footing are in good visual condition and have structural integrity as they are still intact visually as can be seen in the pictures below.

2.1.2 Bridge Superstructure Slab

An examination of the opening or the pothole in the bridge superstructure slab shown in the picture below revealed that there is no hogging reinforcement in the slab, the concrete looks generally weak with a higher percentage of coarse materials and that the longitudinal steel beams appear to have no shear connectors. Therefore, from the visual inspection it was concluded that thereis a need to examine the provided shear connectors in theexisting slab in order to reach a firm conclusion on whether thebeams were designed as composite sections or non-composite.The concrete strength of the existing slab should also be examined.

Figure 2: No hogging reinforcement in the slab, the concrete looks generally weak with a higher percentage of coarse materials

Figure 3: The longitudinal steel beams appear to have no shear connectors

The client through a consultancy firm had proposed the provision of a Bailey Bridge on top of the existing superstructure as indicated by the responsible parties upon seeking for their advice. The implications of adapting that methodology would result into raising of the road as it approaches the bridge from either sides by almost 500mm or more up to about one metre. The costs incurred in such a process would be very high and also would lead to narrowing of the final road layer hence becoming an impediment to the flow of traffic on the relatively very busy road. Furthermore, it would be a safety hazard.

2.2 Bridge Superstructure Geometry

2.2.1 Overall composite beam-slab system

The 20m long superstructure has a 200mm thick solid concrete slab cast on 6 steel I-section beams. Concrete parapet posts are cast monolithic with the slab and two galvanised circular hollow section handrails are provided between the posts. The figure belowshows the bridge cross section.

Expansion joints were not noticed on the bridge deck soffit. However transverse cracks were noticed at the three support positions in the surfacing. It cannot be verified from visual assessment whether the superstructure was designed as continuous for live loading or not.

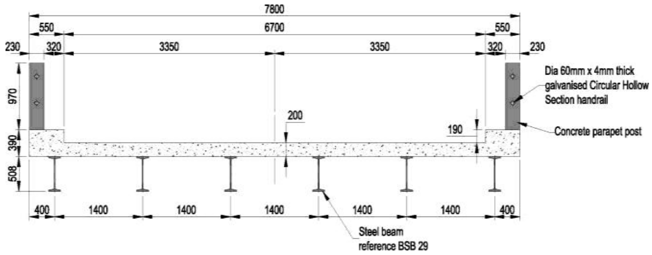

Figure 4: Manafwa River Bridge superstructure cross section

2.2.2 Steel I-Beam Section

The longitudinal steel beams are fixed in concrete at each abutment end. Three of the six beams are continuous over the pier and three are disjointed over the pier with a small non-uniform gap between the beam ends which could be a result of welding failure.

Figure 5: Bridge underside showing steel beams and pier.

The geometry including mainly the dimensions and cross-section measured during the detailed evaluation of the steel beams is as shown in the figure below;

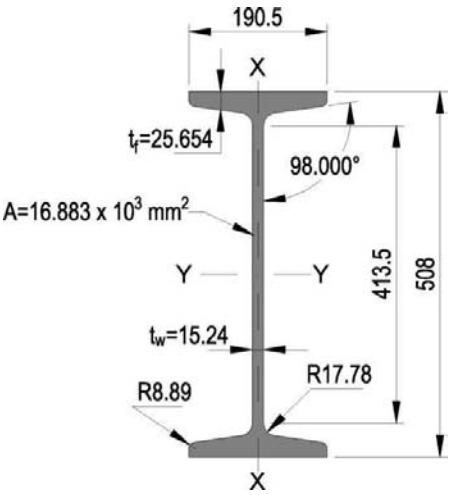

Figure 6: Steel beam cross section

2.2.3 Visual Inspection photography

The photos below show the state of the bridge superstructure at the time of inspections.

Figure 7: Manafwa River Bridge superstructure cross section immediately after the failure and before the visual inspection on the bridge(L-R).

Figure 8: Transverse crack in deck surfacing and concrete patching. There is no transverse joint in the concrete slab on the underside

Figure 9: Opening in superstructure slab. Maximum length of opening parallel to traffic flow direction is 2530mm and it is 1350mm in the normal direction

Figure 10: Opening in superstructure slab as seen from underneath the Bridge

Figure 11: Opening in superstructure slab showing longitudinal steel beam, bottom reinforcement, disintegrated concrete layer and gravel layer under surfacing. No hogging reinforcement over longitudinal beams

Figure 12: Bottom reinforcement at failed slab location. The reinforcement is diameter 16mm round bars at 100mm centres

2.3 Structural and Material Characterization of the Bridge

2.3.1 Introduction (Non-Destructive Testing)

The results of the non-destructive testing as clearly elaborated in the procedures in the previous chapter are shown below as per the different method employed. The results will be used for the finite element analysis.

2.3.2 Schmidt's Rebound Hammer Test Results

The chart below was used to convert the in-situ test results of the rebound number to the average compressive strength of different elements of the Manafwa bridge. As indicated in the methodology, there were rebound numbers less than 20. In order to determine the compressive strength, use was made of extrapolation as can be seen by the lower left hand corner of the graph which isn't part of the chart used.

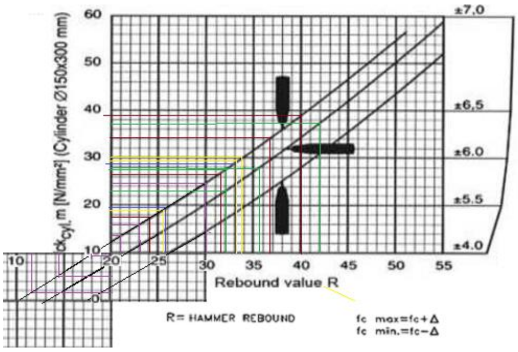

Figure 13: Conversion curve for the average compressive strength measured on cylinders (diameter 150 and thickness of 300mm)

The summary of the in-situ concrete strength test results from the Schmidt rebound hammer for the major elements are shown below, and the detailed results follow:

Table 2-1: Summary In-Situ Concrete Strength Test Results From The Rebound Hammer

Element	Average Compressive Strength (N/mm^2)	Rebar	Average Spacing (mm)
Deck Slab	13	R16	
		R12	110
Parapet wall	28		
Abutment	30		
Piers	25		
Footing	26		

2.3.3 Core Test Results

The compressive strength values of the bridge deck obtained from the testing of the three core samples obtained from the field are summarised as in the table 4-2 and 4-5 below. The calculated ultimate compressive strength values are then corrected using the correction factors given in Table 2-2. The actual correction factors arecalculated by interpolation and extrapolation of the length to depth ration values of each core in comparison to the respective correction factor.

The mean compressive strength of the Deck Slab is very low (9.6 N/mm^2) coinciding with that obtained from the non-destructive testing (13 N/mm^2). This is further supported by the appearance of the concrete as discussed in the visual inspection results above.

Table 2-3: Crushing Strength OF Cylindrical Cores obtained from the Bridge

STRUCTURAL CHARACTERISATION AND PERFORMANCE EVALUATION OF THE FAILED MANAFWA HISTORIC BRIDGE IN UGANDA

Road Name: Tororo - Mbale Road

Sample Reference : | Date : 30th March, 2016

Location : Manafwa Bridge | Technician: Iga Dan (Mr. Kanamwanje)

CRUSHING STRENGTH OF Cylindrical Cores obtained from the Bridge

Diameter = 145 | Area = 0.01708929 m^2

Sample No.	Location of the Core	Structural element	Side	Length	L/D	Remarks	Dimmension LXBXH (mm)	Weight (kg)	Density (kg/m^3)	Crushing Load (KN)	Ultimate Compressive Strength (N/mm^2)	Correction Factor	Corrected compressive strength	Mean Strength
A	Km 0+0014 offset 4.40m from road edge	Deck Slab	RHS	160	1.1	Cores had a top layer of oil and Asphalt which was removed to expose the concrete	Ø145*160	6.310	2308	140	8.2	0.89	7.3	9.6
B	Km 0+0083 offset 5.20m		Centre	100	0.7		Ø145*100	4.050	2370	160	9.4	0.80	7.4	
C	Km 0+0064 offset 6.3 m		LHS	175	1.2		Ø145*175	6.820	2320	190	11.1	0.92	10.2	

Test by: Iga Dan | Checked by: Ntuume William | Approved by: Dr. Timothy Nyomboi

2.4 Slab and the Shear Connection for Composite Behaviour

As from the visual inspection, it was found necessary to examine the provided shear connectors in the existing slab in order to reach a firm conclusion on whether the beams were designed as composite sections or non-composite. As such, two longitudinal beams were exposed for shear connector examination. It was observed that the beams lacked any form of shear connection apart from diameter 12mm round mild steel L-shaped bars welded to top flanges and spaced more than 2.5m. The superstructure slab was observed to be plastered on the underside so as to conceal the exposed reinforcement but the plaster in many sections especially towards the longitudinal beams was observed to be falling off.

Figure 14: Falling off of the plastered underside of the superstructure slab aimed at concealing the exposed reinforcement bars

Large isolated areas on the underside of the superstructure slab showed significant and deep honeycombing. From this observation, it seems there were a lot of fine particle lost during the process of concreting of the slab. As a result, the slab seemed to have a porous structure hence the brittle failure mechanism exhibited at the opened section.

Part of surfacing and concrete along crack in superstructure over pier was removed to examine extent of cracking and whether the deck was continuous over pier. The cracking continues to the bottom of the slab and there is no hogging reinforcement. The two span deck is not continuous over the pier. No proper joint was provided at construction stage.

Figure 15: Transverse crack in the slab running to the bottom of the layer

2.4.1 Steel I beams

A close examination of superstructure longitudinal steel beams revealed that they are all discontinuous over the pier contrary to the earlier inspection during the research proposal which stated that three of the six beams were continuous over the pier

2.4.2 Pier and Footing

There is scouring on the upstream side of the pier footing to a depth of about 260mm below footing soffit. The footing concrete vertical faces protrude beyond the mass concrete vertical faces by 150mm at upstream and at downstream faces and by 400mm at faces parallel to abutments.

Dimensions of the pier footing, columns and column capping beam were recorded including steel beam spacing's, depth and flange thicknesses and concrete slab thicknesses.

3. Analysis (Structural Evaluation and Assessment)

3.1 Superstructure Slab Shear Capacity Under Concentrated Loads

3.1.1 Load at middle of slab

Reference is made to BS 5400, Part 4, 1990 Clause 5,4,4,2. Let the subscript L denote a direction parallel to the longitudinal direction of the slab (traffic flow direction from Tororo to Mbale) and t a direction perpendicular to it (transverse direction). The superstructure slab has bottom reinforcement of R16 at 100mm centres in the transverse direction at bottom most level and R12 at 175mm centres in longitudinal direction on top of R16 bars. There is no top slab reinforcement. The concrete cover to bottom most steel is 20mm and the slab thickness is 200mm.

d_t	=	200 - 20 - 16/2
	=	172mm,
$A_{tsteel\ prov}$	=	R16 at 100
	=	2010mm^2/m and
d_L	=	200 - 20 - 16 - 12/2
	=	158mm,
$A_{Lsteel\ prov}$	=	R12 at 175
	=	665mm^2/m

Further Reference is made to BS 5400, Part 2, 1978, Clause 6,3,2 as shown in the figure attached below, assume no dispersion of HA single wheel load to the surface of the slab. A square contact area 300mm x 300mm will be assumed at the middle of the slab.The critical perimeter of the deck slab is shown in the figure 5-2below:

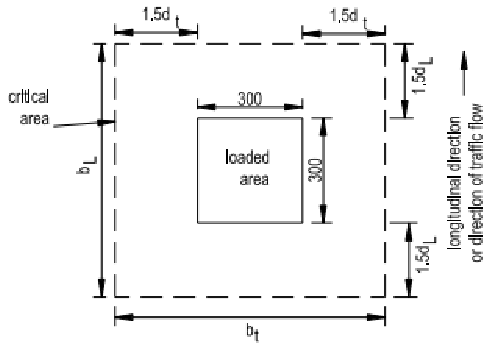

Figure 16 Shear Perimeter

b_t	=	2 x 1.5 x 172 + 300
	=	816mm
b_L	=	2 x 1.5 x 158 + 300
	=	774mm

Assuming HB37.5-wheel load was considered for design as observed for most bridges in Uganda, then the following will occur:

- The loaded area will be 291mm x 291mm (which is approximated to 300mm x 300mm in the analysis and in the shear perimeter drawing above).
- The ULS load will be 134.063 kN
- The closest spacing of HB wheels is 1000mm and
- The closest distance between the shear perimeters is 1000 - bt = 193mm > 0.

Therefore, the shear perimeters of adjacent HB37.5 wheels will not overlap.

The ULS load from the respective computations due to HA single wheel load is obtained as **165.000kN.** As such the ULS load due to HA single wheel load is greater than ULS due to HB37.5, that is to say165.000kN> 134.063kN for HB37.5. Use of HA single wheel load is critical for single loads.

3.1.1.1 In transverse direction

Reference is made to BS 5400, Part 4, 1990, Clause 5,3,3
Ultimate Shear Stress

$$v_c = \frac{0.27}{\gamma_m}\left(\frac{100A_s}{b_w d}\right)^{1/3}(f_{cu})^{1/3}$$

$$v_{cl} = \frac{0.27}{1.25}\times\left(\frac{100\ \times 2010}{1000\ \times 175}\right)^{\frac{1}{3}}\times f_{cu}{}^{\frac{1}{3}} = 0.228 f_{cu}{}^{\frac{1}{3}} N/mm^2$$

$$\varepsilon_{st} = \left(\frac{500}{175}\right)^{\frac{1}{4}} \qquad = 1.306 > 0.70$$

Take $\varepsilon_{sl} = 1.306$

As such,

$$\varepsilon_{sl} V_{cl} = 1.306 \times 0.228 f_{cu}^{\frac{1}{3}} = 0.298 f_{cu}^{\frac{1}{3}}\ N/mm^2$$

Hence

$$V_{cL} = \sum \varepsilon_{sl} V_{cl}\, b_t d_L = 2 \times 0.298 f_{cu}^{\frac{1}{3}} \times 774 \times 175 \times 10^{-3}$$

3.1.1.2 In longitudinal direction

Ultimate Shear Stress

$$v_{cl} = \frac{0.27}{1.25} \times \left(\frac{100 \times 665}{1000 \times 158}\right)^{\frac{1}{3}} \times f_{cu}^{\frac{1}{3}} = 0.162 f_{cu}^{\frac{1}{3}} N/mm^2$$

$$\varepsilon_{sl} = \left(\frac{500}{158}\right)^{\frac{1}{4}} \qquad = 1.334 > 0.70$$

Take $\varepsilon_{sl} = 1.334$

As such,

$$\varepsilon_{sl} V_{cl} = 1.334 \times 0.162 f_{cu}^{\frac{1}{3}} = 0.216 f_{cu}^{\frac{1}{3}}\ N/mm^2$$

Hence

$$V_{cL} = \sum \varepsilon_{sl} V_{cl}\, b_t d_L = 2 \times 0.216 f_{cu}^{\frac{1}{3}} \times 816 \times 158 \times 10^{-3}$$

$$V_{cL} = 55.697 f_{cu}^{\frac{1}{3}} kN$$

Shear resistance at the critical slab section

$$\boldsymbol{V_c = V_{ct} + V_{cl} = 135.041 f_{cu}^{\frac{1}{3}} kN}$$

Ultimate limit state shear load (ULS) exerted by the HA single wheel load is:

ULS_{HA} = 100 kN x 1.5 x 1.1
= 165kN

ULS dead load of slab within the shear perimeter

$ULS_{Dead\ Load}$ = 0.816m x 0.774m x 0.2m x 25kN/m^3 x 1.32
= 4.168 kN

ULS superimposed dead load due to the 100mm thick asphalt concrete surfacing

$ULS_{Asphalt\ surfacing}$ = 0.816m x 0.774m x 0.100m x 25kN/m^3 x 1.925
= 3.039 kN

Shear force acting on the critical slab section

Shear force, V= 165kN + 4.168kN + 3.039kN
= 172.207 kN

Maximum shear stress allowed in the slab, τ, is lesser of $0.75\sqrt{f_{cu}}$ or 4.75 N/mm^2

Maximum Allowable Shear,

V_a = τ ($2b_t d_L + 2b_L d_t$)
= 524.112 τ kN>V_c> V

The condition V_a>V should be satisfied

And No shear reinforcement is required when

V_c > V

Table 3-1: Punching shear capacity of existing superstructure slab for loading applied at middle point of the slab

Concrete strength, f_{cu}, (N/mm^2)	Maximum allowable shear, V_a, (kN)	Shear capacity, V_c, (kN)	Applied shear, V, (kN)	Comment
5	878.963	230.917	172.207	No reinforcement required
10	1243.041	290.937	172.207	No reinforcement required
15	1522.408	333.040	172.207	No reinforcement required
20	1757.925	366.558	172.207	No reinforcement required
25	1965.420	394.862	172.207	No reinforcement required
30	2153.010	419.604	172.207	No reinforcement required
35	2325.516	441.728	172.207	No reinforcement required
40	2486.082	461.834	172.207	No reinforcement required

The above results in the Table 5-1 show that the existing slab is adequate in punching shear for the different range of concrete strengths (including the current strength obtained from the non-destructive testing carried out) considered as such could not have failed due to punching shear.

3.1.2 Load at edge of slab

The carriageway portion of the superstructure slabis bounded by longitudinal beams and transverse abutment concrete wall into which the superstructure longitudinal beams are encased at ends hence there is no need to check for shear due to concentrated loads.

3.2 Superstructure Slab Moment Capacity

3.2.1 Moment capacity of Existing slab

For given amount of tension steel, A_s, of design characteristic strength, f_y, and effective depth, d, the ultimate limit state, ULS, moment capacity of a reinforced rectangular concrete section with no compression reinforcement, is obtained from the formula below obtained from a typical Bridge design example (derived from BS 8110):

$$M_c = \frac{M_u}{0.694}\left\{1 - \left(1 - \sqrt{1 - \frac{d f_y A_s}{3.313 M_u}}\right)^2\right\} \text{Where}$$

$$M_u = 0.156 f_{cu} b d^2$$

Let the subscript L denote a direction parallel to the longitudinal direction of the slab (traffic flow direction) and t a direction perpendicular to it (transverse direction) as described in the previous section:

The superstructure slab has bottom reinforcement of R16 at 100mm centres in the transverse direction at bottom most level and R12 at 175mm centres in longitudinal direction on top of R16 bars. There is no top slab reinforcement. The concrete cover to bottom most steel is 20mm and the slab thickness is 200mm.

d_t = 200 - 20 - 16/2
= 172mm,
$A_{tsteel\ prov}$ = R16 at 100
= 2010 mm^2/m and
dL = 200 - 20 - 16 - 12/2 = 158mm,
$A_{Lsteel\ prov}$ = R12 at 175
= $665mm^2/m$
f_y = 250 N/mm^2

The Table 6.2 belowsummarises the moment capacity of superstructure slab as obtained using the moment capacity formula above.

Table 3-2: Ultimate Limit State Sagging moment capacity of Existing slab

Concrete strength, f_{cu}, (N/mm^2)	Moment Capacity in transverse direction, M_{ct}, (kNm/m)	Moment Capacity in longitudinal direction, M_{cL}, (kNm/m)
5	32.683	18.197
7.5	43.308	19.360
10	53.933	20.523
15	61.016	21.298
20	64.557	21.686
25	66.682	21.919
30	68.099	22.074
35	69.111	22.184
40	69.870	22.268

3.2.2 Superstructure slab applied moments

The applied moments are obtainedusing the equations in the table 6-3 below which is extracted from BS 6110 part 2 1997 cl. 3.5.2.4 Table 13.2 for the ultimate bending moments and shear forces.

Table 3-3: Ultimate bending moment and shear forces in one-way spanning slabs

	End support/slab connection				At first interior support	Middle Interior spans	Interior supports
	Simple		Continuous				
	At outer support	Near middle of end span	At outer support	Near middle of end span			
Moment	**0**	**0.086Fl**	**-0.04Fl**	**0.075Fl**	**- 0.086F**	**0.063Fl**	**-0.063Fl**
Shear	**0.4F**		**0.46F**		**0.6F**		**0.5F**
NOTE: *F* **is the total design ultimate load (1.4Gk + 1.6 Q_k);** *l* **is the effective span.**							

Source BS 8110 pt2 1997 cl 3.5.2.4 Table 3.12

The applied moments are therefore summarised in Table 6-4 below

Table 3-4: Superstructure slab applied moments

Load	Transverse moments (kNm/m)		Longitudinal moments (kNm/m)	
	Sagging	Hogging	Sagging	Hogging
Permanent	1.4	2.2	0.9	0.4
Permanent and HA single wheel load	4.0	36.0	19.3	9.6
Permanent and HA	10.2	6.8	5.3	0.8
Permanent, HA and HB20	20.3	22.2	11.3	8.0
Permanent, HA and HB25	20.2	22.1	13.8	9.7
Permanent, HA and HB37.5	24.9	21.8	19.9	13.7

4. Conclusion and Recommendation

4.1 Summary and Analysis of the Results

The applied transverse sagging moments are all less than the slab moment capacities for the concrete strengths considered. The applied longitudinal sagging moments are all less than the slab moment capacities for the concretestrengthsconsidered except for 5 MPaand 7.5 MPaconcrete.This implies that there was significant deterioration of the concrete which as a result could not withstand the applied moments hence the collapse. No hogging reinforcement was provided in the existing slab.(2)(3)

4.2 Recommendations

Top steel should be provided which will imply that the overall thickness of the existing slab shouldbe increased so as to accommodate the reinforcement as a way to resist the moments hence ultimately accommodating much more traffic.

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