Civil and Environmental Research
ISSN 2224-5790 (Paper) ISSN 2225-0514 (Online)
Vol.3, No.13, 2013
www.iiste.org

Geoforensic Analysis of Agba Bridge Failure, Abakaliki, Ebonyi State

T.K.S. Abam[+] and T. Oba*

+Geotechnics Division, Institute of Geosciences and Space Technology,
Rivers State University of Science and Technology,
P.M.B 5080, PortHarcourt, Nigeria.

*Groundscan Services Nigeria Limited
No. 1, Brown Abiye Avenue, Off Dr. Peter Odili Road,
Trans-Amadi, P.O.Box 9528, PortHarcourt, Nigeria

Abstract

A geoforensic analysis of Agba bridge, in Abakaliki, Ebonyi State was carried out using a combination of geotechnical, geophysical and hydrological techniques. The results indicate a highly geotechnically variable, partially weathered meta-sediments as foundation. The weathered profile as confirmed by VES indicated a much thicker weathered soft soil to the south-west. An initial shallow foundation for the South-West Abutment was thus inadequate, leading subsequently to sliding movement, which displaced the reinforced concrete piers. Hydrological analysis of the Agba river revealed critical flow velocities capable of generating local scour of up to 3.5m deep, that equally threaten stability of shallow pier foundation. Accordingly, bored piles extended to 15m have been recommended for the second half of the bridge to the south-west as a remedial measure to guaranty the stability of the structure.

Keywords: Geoforensic analysis, bridge failure, scour, geo-electrical sounding, geotechnical

Introduction

A bridge across a major drainage river in Agba, near Abakaliki was 80% completed, with most of the piers in place, when some movement, was observed in one of the piers. This movement resulted in horizontal displacement of the pier. Photographs of sections of the bridge are shown in Plate-1.


Plate-1: Photographs of sections of bridge

Unsure of the cause, a Geoforensic analysis of the failure was commissioned, with a view to identifying the cause(s) and recommending corrective action to progress the construction to a satisfactory conclusion.

The existing bridge design spans roughly 84m, while the river depth is roughly 7.5m at the deepest reach. Geotechnical investigations of the underlying soils was considered the best approach to providing an insight into the reasons for movement and aid the formulation of appropriate corrective action.

The investigations involved borehole drilling at each Abutment and at the centre of the river as illustrated in Figure.1. This paper attempts to explain how and why the observed failure occurred by exploring the properties and behaviour of the foundation soils in combination with the hydrological conditions and using these to provide an account of deficiencies that led to failure of the structure.

Site Description and Geology

The geologic history of Abakaliki basin is characterized by compressional tectonic stresses. The associated stresses caused metamorphism and fracturing of older marine and volcanic rocks. The underlying rock in the area is the Abakaliki shale which lies within the Asu River Group of mid Albian age in the Southeastern Nigeria. The Abakaliki shale are poorly bedded, occasionally sandy, and consists of Splintery metamorphosed mudstones. Lenses of sandstone and sandy limestone are highly jointed and fractured.

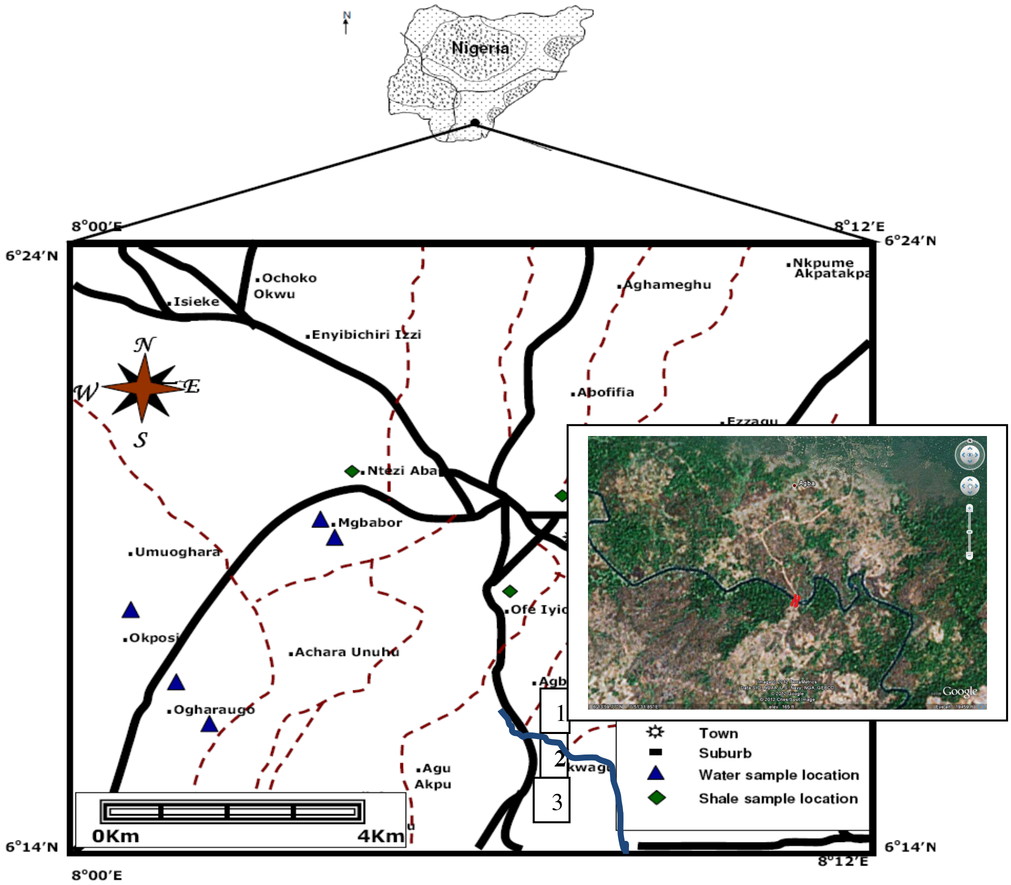

Figure.1: Map of the Agba area, showing river outline, general morphology with insert of imagery and borehole locations (adapted from Aghamelu, et al 2009)

Studies by Benkhelil et al., 1989, Murat, 1972; Nwachukwu, 1972; Ofoegbu and Amajor, 1987; Tijani et al., 1996 indicate that the group has average thickness of about 2000 m and rests unconformably on the Precambrian Basement, while the Abakaliki shale formation has an average thickness of about 500 m and is dominantly shale, dark grey in colour, blocky, and non-micaceous in most locations (Figure. 2). Although it is deeply weathered, there are isolated boulders in relatively unweathered state, which interfered with drilling or piling operations. The predominant shale has favoured the low erodability of the lithology, resulting in absence or near absence of deep cut valleys and erosion channels. Stunted trees and pockets of derelict woodland exist where the lithology has undergone high degree of laterization.

Figure.2: Exposed section of geologic formation underlying the Abakaliki area

Methods of Investigation

The site investigation was in three parts; namely, Reconnaissance, Vertical Electrical Sounding (VES), Cone Penetration Sounding and Geotechnical drilling. The VES techniques utilized the Schlumberger electrode configuration as described and interpreted by Reynolds (1998). The layout of test points are shown in Figure. 3 while GPS co-ordinates are shown in Table 1.

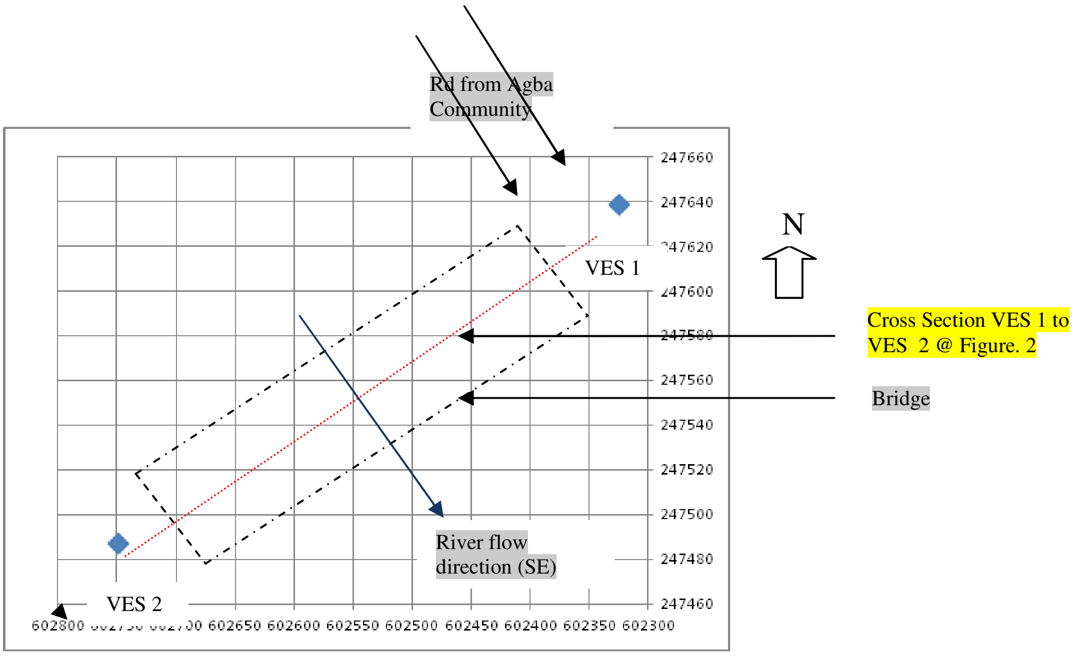

Figure.3. VES Points Layout in the Project Site

Table 1. GPS Co ordinates of the VES Points

VES NO	GPS CO ORDINATES			APPROX. ELEVATION	AZIMUTH
	EASTINGS		NORTHINGS		
1	0602324		0247639	47m	120^{0}N
	E 007^{0}51'36.1"		N 06^{0}13'47.5"		
2	0602749		0247487	45m	120^{0}N
	E 007^{0}51'33.8"		N 06^{0}13'42.8"		

A 5 Ton capacity static cone penetrometer was also used in the cone soundings using a discontinuous sounding procedure. The Cone Penetration Tests were driven to refusal prematurely at a depth of about 10m because of the unweathered boulders embedded in the sediments. The borehole drilling enabled *in-situ* SPT soundings and recovered samples that aided determination of the stratigraphy of the superficial deposit underlying the site, the relevant engineering characteristics of the deposits and provided data for computation of Axial Pile Capacity for various pile sizes.

Results and Discussion

The results of VES are presented in Tables 2 and 3 and Figures. 4 - 8. The apparent resistivity calculated for each subsurface layer are shown in table with corresponding depth. A geo-electric profile (pseudo sections, Figure. 4 – 8) ground model summarizes the probable subsurface geo-electric layers of the Abutment areas.

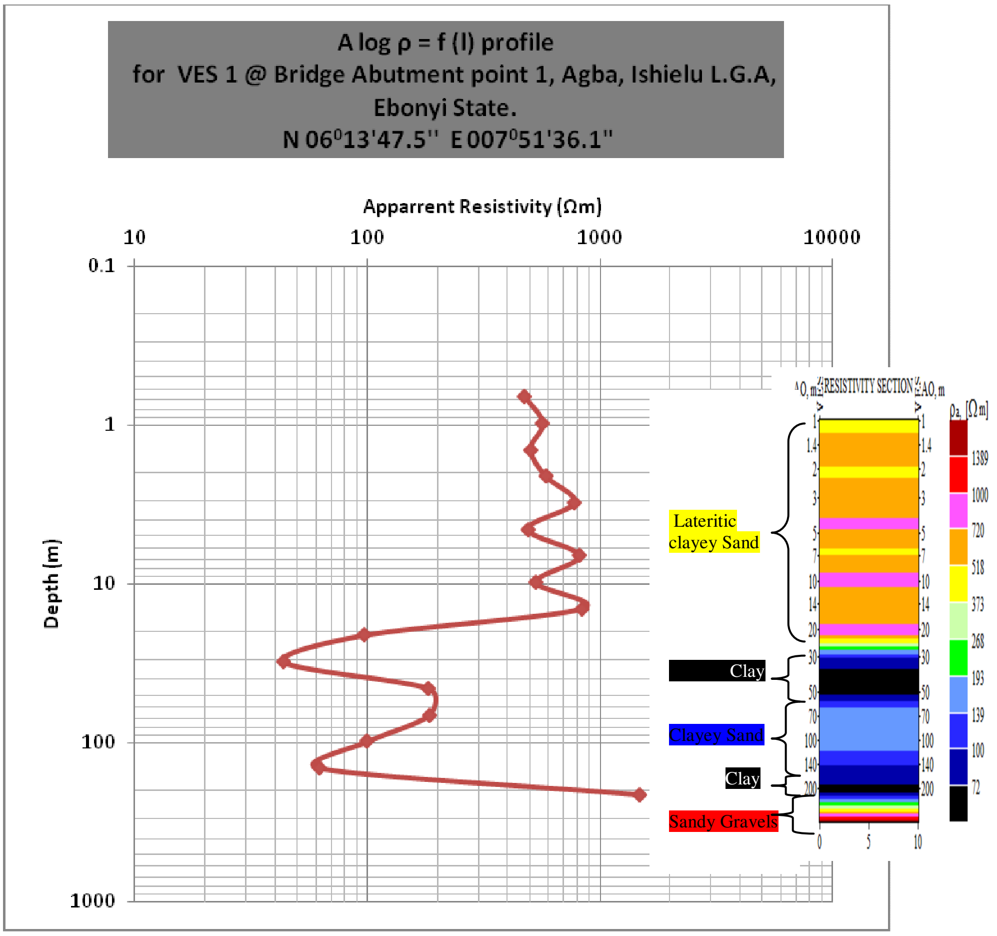

Figure 4. Resistivity log for VES 1

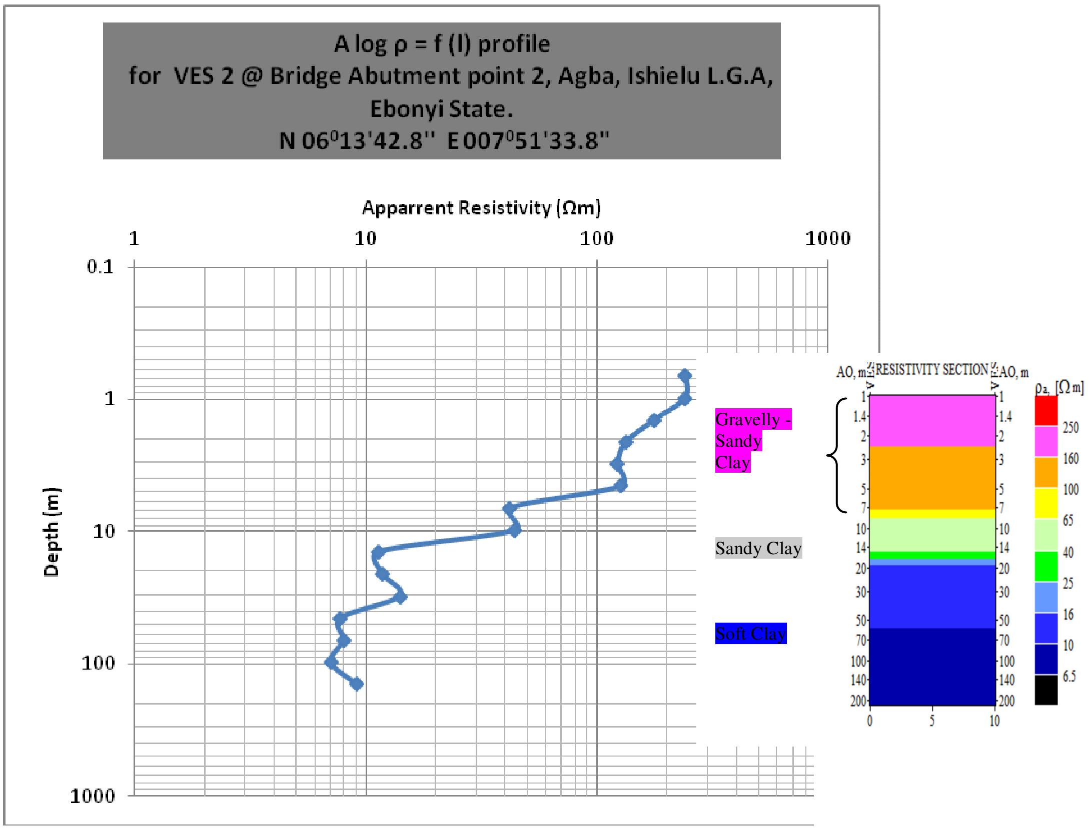

Figure 5. Resistivity log for VES 2

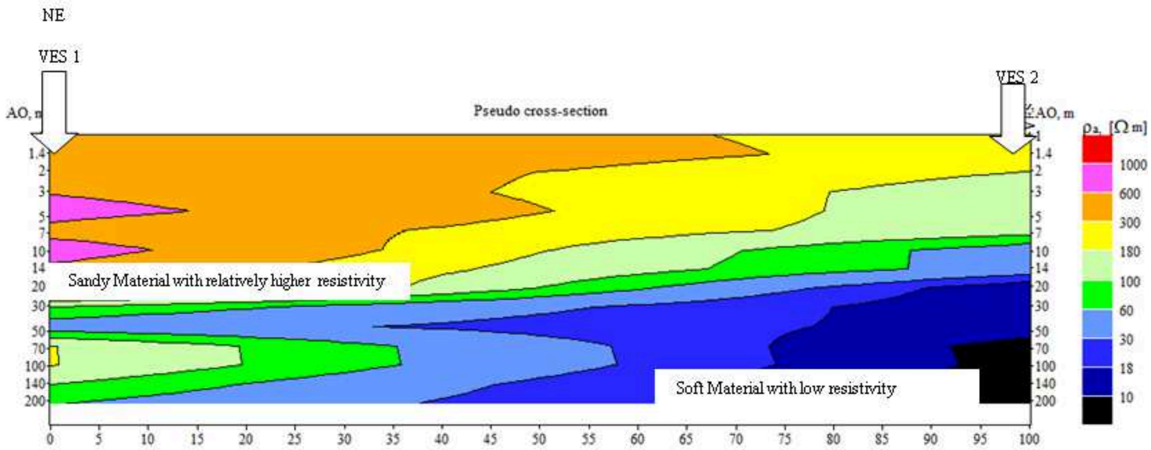

Figure 6 Pseudo Cross Section : (VES 8, 9 & 10) NE - SW

Soil Stratigraphy

The composite cross-section of the lithstratigraphy deduced from the combination of boring and VES is shown in Figure. (9). At the Abuttment-1, the stratigraphy comprises a firm top silty sandy clay layer of thickness 3m at the north Abuttment. This is underlain by a succession of soft silty/sandy clay layers occasionally interspersed by very stony layers. These stony layer presented substantial resistance to penetration during boring, eventually forcing an early termination of both the CPT and percussion drilling processes.

Two inferences were made based on the stratigraphy, namely;

(i) The two Abutments have dissimilar soil stratigraphy within the depths of engineering significance. The approach from Agba has a relatively dry lateritic soil interspersed with stones upto 15m from by relatively soft and saturated shaley formation. Whereas the thickness of the dry top soil across the river is much smaller with the soft shaley material located relatively closer to the ground surface.

(ii) The occurrence of soft saturated materials at relatively shallow depths, creating the possibility of piling the bridge foundation and at the same time creating the danger of soil failure by sliding movements.

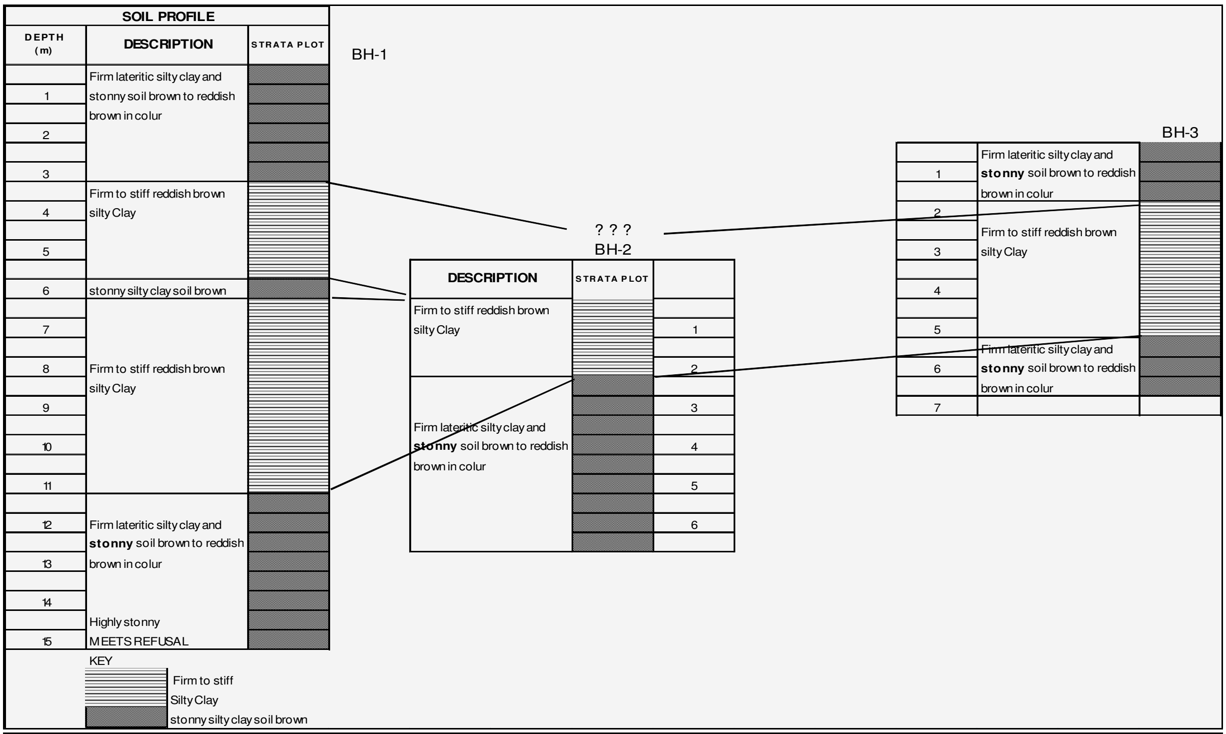

Figure. 9: Composite Stratigraphy across bridge site

Engineering Properties of the Soils

Classification, strength and compressibility characteristics of the soils were determined from the laboratory and in-situ tests. The relevant index and engineering parameters of the soils are summarized in Table 4.

Table 4: Summary of Soil Properties

BH. No.	Depth (m)	Moisture Content (%)	Bulk Unit (kN/m^3)	Cohesion (kN/m^2)	Angle of Int. Friction (Dec)	Liquid Limit (Ll)%	Plastic Limit (Pl)%	Plasticity Index (PI)%
1	1.5	36	21	45	15	42	20	22
	3	15.1	22	76	11	37	20	16
	7.5	25	20	85	17	26	17	9
	10.5	30.4	22	65	28	44	21	23
2	1.5	18.4	20	56	12	25	16	9
	3	16	22.3	78	21	29	14	15
	4.5	22	22	59	18	23	16	7
	6	23	21	61	23	33	17	16
3	3	39	18	45	10	56	31	25

As shown in the table, the Abakaliki shale recorded values of Liquid Limit (LL) which ranged from 26 to 56, Plastic Limit (PL) 14 to 31. The difference between LL and PL, termed the plasticity index (PI), represents the range in the water contents through which the soil is in the plastic state. The PI was moderate (ranged from 9 to 25). On the basis of LL and PI values, the Abakaliki shale could be classified as medium plasticity soil. Results indicated that the Abakaliki shale has mean Gs value of 2.32; a value less than that of a potentially durable construction aggregate with Gs value of 2.625 (Reidenouer (1970). Using slake durability tests, Ezeribe (1994) rates shale samples collected from two neighbouring locations on the Asu River Group (Ishiagu and Amenu Okposi areas) as very low durability shales.

Shear strength of a material denotes the ability of such material to resist shearing deformational stresses. It is expressed in terms of two parameters, angle of shearing resistance (f) and cohesion (Cu). Laboratory analysis showed that for Abakaliki shale ranged from 10 to 28°, while Cu ranged from 45 to 85 kN/m^2. The computed bearing capacity values (Table 3) suggests adequate strengths and capacity to withstand significant shear stresses. However, because of the highly jointed nature of the formation, allowing significantly high moisture influx, it is considered that the shales would deteriorate its constituent minerals, especially clays, resulting in strength reduction and perhaps, bearing capacity loss, during the engineering life of such project.

Table (3) Ultimate and Allowable Bearing Capacities for Shallow Foundation

Foundation Depth (m)	Width (m)	Allowable Bearing Pressure (KN/m^2) for Abutment-1			Allowable Bearing Pressure (KN/m^2) for Abutment-2			Allowable Bearing Pressure (KN/m^2) for River Center		
		L/B =1	L/B= 1.5	L/B = 5	L/B=1	L/B=1.5	L/B=5	L/B=1	L/B=1.5	L/B=5
1	1	208.7	209.0	209.3	212.0	212.3	212.7	208.7	209.0	209.3
	1.5	210.5	210.9	211.5	213.8	214.3	214.8	210.5	210.9	211.5
	2	212.3	212.9	213.7	215.6	216.2	217.0	212.3	212.9	213.7
	2.5	214.1	214.8	215.8	217.4	218.2	219.2	214.1	214.8	215.8
	5	223.1	224.6	226.6	226.4	227.9	230.0	223.1	224.6	226.6
	10	241.1	244.1	248.2	244.4	247.4	251.6	241.1	244.1	248.2
1.5	1	284.4	284.6	285.0	277.7	278.0	278.3	231.0	231.3	231.7
	1.5	286.2	286.6	287.2	279.5	279.9	280.5	232.8	233.3	233.8
	2	288.0	288.5	289.3	281.3	281.9	282.7	234.6	235.2	236.0
	2.5	289.8	290.5	291.5	283.1	283.8	284.8	236.4	237.2	238.2
	5	298.8	300.2	302.3	292.1	293.6	295.6	245.4	246.9	249.0
	10	316.8	319.7	323.9	310.1	313.1	317.2	263.4	266.4	270.6
2	1	323.4	323.6	324.0	240.0	240.3	240.7	253.4	253.6	254.0
	1.5	325.2	325.6	326.2	241.8	242.3	242.8	255.2	255.6	256.2
	2	327.0	327.5	328.3	243.6	244.2	245.0	257.0	257.5	258.3
	2.5	328.8	329.5	330.5	245.4	246.2	247.2	258.8	259.5	260.5
	5	337.8	339.2	341.3	254.4	255.9	258.0	267.8	269.2	271.3
	10	355.8	358.7	362.9	272.4	275.4	279.6	285.8	288.7	292.9

Settlement

Computation of Foundation Settlement was carried out for an assumed net foundation load of $\Delta\sigma$ = **200**kPa. Prediction of the settlement was made from the summation of the vertical strains caused by the anticipated load.

The soil beneath the foundation is divided into layers and the coefficient of volume compressibility m_v obtained for each layer. The result of the vertical and incremental Settlement distribution is shown in Figure. (10) indicating unevenness in the layer contributions.

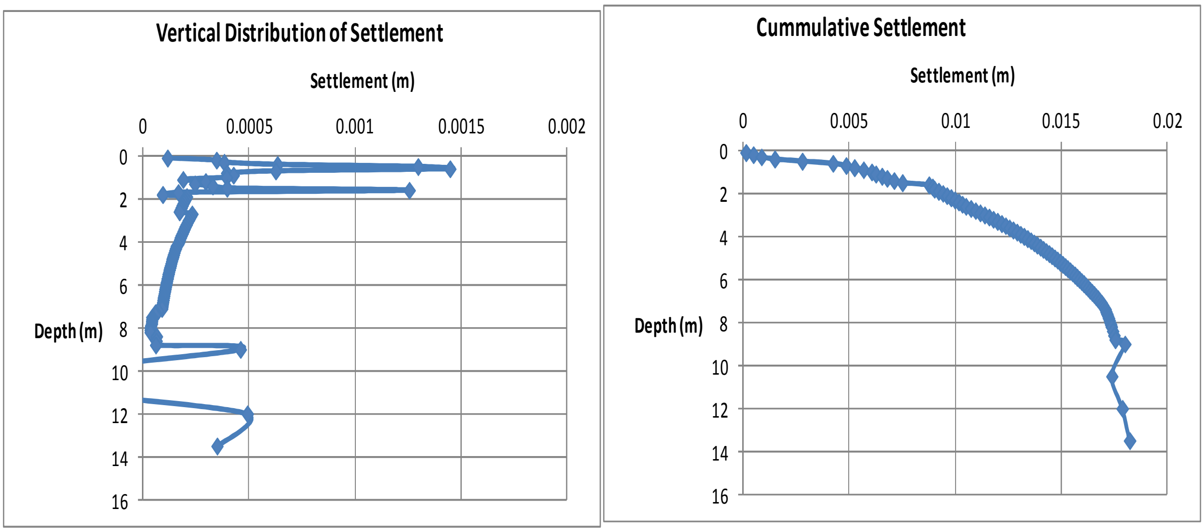

Figure. 10: Incremental and vertical Distribution of Settlement

Piled Foundation

Geotechnical Bored Pile Capacities were also computed using the relationships by Peck, et al; (1973): Computed Safe End Pile Capacities (KN) For various Pile Diameters are shown in Table (4) for both Abutments and River Bed.

Table 4) Safe End Pile Capacities (KN) For various Pile Diameters

SAFE END PILE CAPACITIES (kN) FOR VARIOUS PILE DIAMETERS						
BH1	DRIVEN PILES			BORED PILE		
Depth (m)	406mm	600mm	750mm	750mm	900mm	1200mm
1.5	19	40	62	59	85	151
4.5	76	156	236	209	300	534
6	92	183	275	231	333	592
9	446	920	1402	1259	1812	3222
12	476	953	1434	1209	1741	3096
13.5	1030	2108	3201	2829	4074	7242
15	1392	2840	4305	3779	5442	9674

SAFE END PILE CAPACITIES (kN) FOR VARIOUS PILE DIAMETERS

BH2	DRIVEN PILES			BORED PILE		
Depth (m)	406mm	600mm	750mm	750mm	900mm	1200mm
1.5	23	49	76	72	104	185
3	57	120	184	171	246	437
4.5	77	159	242	215	310	551
6	85	169	253	208	299	531
6.5	501	1059	1631	1538	2215	3938

SAFE END PILE CAPACITIES (kN) FOR VARIOUS PILE DIAMETERS

BH3	DRIVEN PILES			BORED PILE		
Depth (m)	406mm	600mm	750mm	750mm	900mm	1200mm
1.5	26	55	85	81	117	208
3	62	131	201	188	271	482
4.5	326	699	1082	1045	1504	2674
6.0	658	1408	2181	2104	3029	5386
6.5	841	1798	2783	2680	3859	6860

Scour Prediction

The role of scour in this bridge failure was also assessed. At the time of fieldwork in mid September 2012, flow velocity across the bridge averaged 0.9m/s. This is expected to increase to about 1.3m/s at the peak of the flood season in October. The turbidity of the water indicated the movement of substantial suspended load in the flow. Following the projection of the abutment into the river and the narrowing of the bridge span, the river cross-section at the bridge crossing was substantially reduced, leading to a constriction of the flow. Under these flow conditions, there is every reason to expect the development of scour around the piers and abutments. Bridge scour problems are thus relevant to the existing bridge.

To determine if the flow upstream of the bridge is transporting bed material, the critical velocity for initiation of motion Vc of the D_{50} size of the bed material was calculated (Table 5) using the equation (Barbhuiya and Dey 2004) below and compared with the mean velocity V of the flow in the main channel.

$$V_c=K_u y^{1/6} D^{1/3}$$

where:
V_c = Critical velocity above which bed material of size D and smaller will be transported, m/s
y = Average depth of flow upstream of the bridge, m
D = Particle size for V_c, m
D_{50} = Particle size in a mixture of which 50 percent are smaller, m
Ku = 6.19 SI units

The D_{50} was taken as average of the bed material size in the reach of the stream upstream of the bridge, as this is the characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 0.3 m of the stream bed.

Since the critical velocity of the bed material is larger than the mean velocity (Vc > V), then clear-water contraction scour will exist.

Table 5: Computation of Critical Velocity for different flow depths

Month	Flow Depth	Mean particle size			
	h(m)	D_{50}m	h(1/6)	D50(1/3)	Critical Velocity m/sec (Vc)
January	1	0.0001	1	0.046415888	0.287314349
	2	0.0001	1.122462	0.046415888	0.322499452
	3	0.0001	1.200937	0.046415888	0.345046419
	4	0.0001	1.259921	0.046415888	0.361993396
	5	0.0001	1.30766	0.046415888	0.375709621
	6	0.0001	1.348006	0.046415888	0.38730151
	7	0.0001	1.383088	0.046415888	0.3973809
	8	0.0001	1.414214	0.046415888	0.406323849
October	9	0.0001	1.44225	0.046415888	0.414378996

Prediction of Local Scour Depth

To determine the extent of scour, scour depth was predicted based on Breusers et al. (1977) using the equation:

$$y_{se} = 1.35Ki\ b^{0.7}{y_0}^{0.3}$$

Ki = 1.0 for circular piers
b = pier width
y_0 = flow depth

Summary of various scour depth at different flow depths corresponding to water levels at various times of the year is presented in Table 6:

Table 6: Computation of Maximum Scour Depths under different flow conditions

Month	Flow Depth	Scour Depth (m)
	y(m)	Yse
January	1	1.793
	2	2.208
	3	2.493
	4	2.718
	5	2.906
	6	3.069
	7	3.215
	8	3.346
October	9	3.466

As indicated in Table (6), the depth of scour is highly dependent on time.

Conclusions

This study on the Abakaliki shale formation has provided insights into the probable causes of failure of Bridge Abutment. This study associated the bridge failure to the placing of structural foundations on weak, scour susceptible soil deposits and the narrowing of the width of the natural stream channel by projecting the abutment into the natural channel.

Field and laboratory investigations indicate that the site is underlain predominantly by silty and sandy shaley soil with reasonably high bearing capacity but unquestionably susceptible to slaking and scour. Due to a combination of high flow velocity, bed material characteristics and the projected abutment, significant scour appear to have taken place around the piers. This will normally result in undermining the foundation of the piers, causing it to be unstable and leading ultimately to its failure. Such pier failures are more probable when the depth of embedment of the pier is smaller or equal to the maximum scour depth.

This finding highlights the inadequacy of geotechnical design for the approach embankments or abutments and also a lack of understanding of the subsoil condition and awareness on the possible problems/failure that could happen during construction. Furthermore, it underscores the inadequacy or complete absence of construction control and site supervision.

A Pile embedment depth of 15m is recommended. Driven Piles are likely to encounter pre-mature termination because of the geology. Accordingly, Bored piles extended to at least 15m would not only possess adequate pile capacities but also place foundations below the expected maximum depth of scour.

References

1. Barbhuiya, A. K. and Dey, S., (2004). “Local scour at abutments: a review.” *Flow Meas. Instrum.,* 29(5), 449-476.
2. Chiew, Y. M. and Melville, B. M. (1987). “Local scour around bridge piers.” *J. Hydraul. Res*., 25(1), 15-26.
3. Coleman, S. E., Lauchlan, C. S. and Melville B. W., (2003). “Clear-water scour development at bridge abutments.” *J. Hydraul. Res*., 41(5), 521–531.
4. Lai, J. S., Chang, W. Y., and Yen, Y. L. (2009). “Maximum local scour depth at bridge piers under unsteady flow.” *J. Hydraul. Engrg*., 135(7), 609-614.
5. Nwachukwu SO (1972). The tectonic evolution of the southern portion of the Benue Trough, Nigeria. Geol. Mag., 109: 411-419.
6. Okagbue CO (1986). The physical and mechanical properties of laterite gravels from southeastern Nigeria relative to their engineering performance. J. Afr. Earth Sci., 5(6): 659-664.
7. Okogbue CO, Aghamelu OP (2010a). The Impact of the Geotechnical Properties of the Abakaliki Shale on the Incessant Road Failures in the Abakaliki Area, southeastern Nigeria. Paper presented at the 1st International Workshop on Landslides and Other Natural Disasters,

8. Peck, R.B; Hanson W.E and Thornburn T.H (1973) Foundation Engineering 2nd Edition John Wiley and Sons 514pgs.
9. Reyment RA (1965). Aspects of Geology of Nigeria. Ibadan University, Ibadan, pp 145.
10. Whiteman AJ (eds) Proceedings of the Conference on African Geology held at Ibadan, Nigeria, pp. 251- 266.
11. SKEMPTON, A.W. (1951). The Bearing Capacity of Clays. Building Research Congress, London. 1:180-189.
12. Aghamelu O. P., Nnabo P. N. and Ezeh*H. N. (2011) Geotechnical and environmental problems related to shales in the Abakaliki area, Southeastern Nigeria, African Journal of Environmental Science and Technology Vol. 5(2), pp. 80-88, February, 2011