Global Journal of Researches in Engineering: E
Civil And Structural Engineering
Volume 18 Issue 1 Version 1.0 Year 2018
Type: Double Blind Peer Reviewed International Research Journal
Publisher: Global Journals Inc. (USA)
Online ISSN: 2249-4596 & Print ISSN: 0975-5861

Design of a Pedestrian-Steel Bridge Crossing Auchi-Benin Expressway

By Oyati, E. N
Auchi Polytechnic, Auchi

Abstract- This research study was to design a pedestrian steel bridge at Auchi Polytechnic Hostel Gate across Auchi-Benin Expressway so as to provide a safer and easy route for the users, especially students and also to reduce accident rate. The work involved the feasibility study of the chosen sections such as soil analysis, design of the structural components of the bridge,(beams, floorplate, column and foundation) which were designed to British Standard (BS 5400, BS 5950, BS 8110). Soil allowable bearing capacity of 233KN/m2 was established. This was used for the design of the pad footings for the steel stanchions whose dimensions were 1300 mm * 1300 mm * 450 mm and also the specification for plate was 80 mm *2 mm, staircase beam; 254 *102 * 28UB beam for bracing; 127*76*46UB, walkway beam; 356*171*57UB, landing; 254*102*28UB, column; 203*203* 46UC and foundation reinforcements were found to be 6Y20mm@300c/c (As=1050mm2) in each direction.

Keywords: pedestrian, beams, design, foundation, structural components, column.

GJRE-E Classification: FOR Code: 090599

DESIGNOFAPEDESTRIAN-STEELBRIDGECROSSINGAUCHI-BENINEXPRESSWAY

Strictly as per the compliance and regulations of:

Design of a Pedestrian-Steel Bridge Crossing Auchi-Benin Expressway

Oyati, E. N

Abstract- This research study was to design a pedestrian steel bridge at Auchi Polytechnic Hostel Gate across Auchi-Benin Expressway so as to provide a safer and easy route for the users, especially students and also to reduce accident rate. The work involved the feasibility study of the chosen sections such as soil analysis, design of the structural components of the bridge,(beams, floorplate, column and foundation) which were designed to British Standard (BS 5400, BS 5950, BS 8110). Soil allowable bearing capacity of 233KN/m^2 was established. This was used for the design of the pad footings for the steel stanchions whose dimensions were 1300 mm * 1300 mm * 450 mm and also the specification for plate was 80 mm *2 mm, staircase beam; 254 *102 * 28UB beam for bracing; 127*76*46UB, walkway beam; 356*171*57UB, landing; 254*102*28UB, column; 203*203* 46UC and foundation reinforcements were found to be 6Y20mm@300c/c (As=1050mm^2) in each direction.

Keywords: pedestrian, beams, design, foundation, structural components, column.

I. Introduction

The world faces today the big challenge of traffic accidents that harvest annually millions of human lives (Muhammad, 2013). The consequences of these traffic accidents do not only affect the victims or their families, but extend to the impact the community and its progress (Muhammad, 2013). Pedestrian bridges are structures made for allowing pedestrians to cross a street/road/highway without being exposed to the risks of car accidents. A pedestrian bridge is any structure that removes pedestrians from vehicle roadway (Muhammad, 2013).

The first pedestrian bridge in Nigeria was a steel structure erected at Idumota cenotaph on Lagos Island (The Guardian, 2015). However, according to the Guardian newspaper, two such concrete bridges were also constructed: one in Iddo railway terminals across the road and the second was from Oyingbo to Otto near the old Leventis mainland hotel. The two bridges were planned towards the 1960 independence celebration. The construction work was carried out by Taylor Woodrow Construction Company (The Guardian, 2015). It was a major event on its own in those days especially considering the swampy terrain that the bridges were required to cross through. With the advent of the third National Development Plan (1975-1980), reinforced concrete bridges on piles and prefab deck were constructed over Apapa-Oshodi expressway and the Agege Motor Way at Ikeja. A bridge is a structure that provides passage over an obstacle such as valley, rough terrain or body of water by spanning those with natural or manmade materials (Newman, 2003; Mosley and Bungey,1999; Jeswald, 2005).

According to Mugu (2004) a footbridge or pedestrian bridge is principally designed for pedestrians and in some cases cyclists, animal traffic and horse riders rather than vehicular traffic. Recently the Lagos State Government erected a multi-functional pedestrian structure at Oshodi (The Guardian, 2015). The current governor of Lagos State, Akinwumi Ambode, has approved the construction of pedestrian bridge at Berger area of the State to give room for easy crossing by pedestrian of the ever busy Lagos- Ibadan expressway (P M News, 2015). In Benin City, Edo State of Nigeria, there was a pedestrian steel bridge constructed at close proximity to the University of Benin main gate but was dismantled because of the dualisation of the road by the Edo State Government. Types of pedestrian bridge include: simple suspension, clapper, moon, step-stone and zig-zag bridge

Increasing rate of accident at the hostels' gate of Auchi Polytechnic is worrisome. This involves either two or more vehicles or at times two or more motor bikes. The fatal ones always attract the attention of the Federal Road Safety Corp (FRSC) who needs to evacuate the vehicles and the injured in order to allow the free flow of traffic. The main victims of hit and run by vehicles and bike riders especially at night have been the students living on and off campus of Auchi Polytechnic, Auchi. Thus this development necessitates the design and construction of a pedestrian bridge across the Auchi-Benin highway.

II. Materials and Methods

a) Study Area

This study focuses on the design of pedestrian bridge across the Auchi-Abuja Highway in front of Auchi Polytechnic, Auchi main entrance gate.

b) Design consideration, calculations and analysis

i. *Soil Test*

The following geotechnical parameters were determined as follows:

Author: Department of Civil Engineering Technology, Auchi Polytechnic, Auchi, Nigeria. e-mail: edithoyai@gmail.com

Percentage soil sample retained (%) = $\frac{massretained}{totalmass} \text{ x } \frac{100}{1}$ [1]

Percentage passing (%) = $\frac{totalpercentageretained}{\%\, retainedofeachsieves}$ [2]

Coefficient of uniformity (cu) = $D30/_{D60} \text{ x D10}$ [3]

Coefficient of uniformity (cu) = $D60/_{D10}$ [4]

ii. *Specific Gravity Test (Gs)*

Table 1: Specific Gravity Result at 0.5m & 1m: Data Sheet for Specific Gravity by Density Bottle

S/N	Observation and Calculations	Determination	No
		1	2
1	Density	4267.0	4331.0
2	Weighty of Empty Bottle (M_1) (g)	26.6	26.6
3	Weighty of Bottle +Sample (M_2) (g)	56.6	56.6
4	Weighty of Bottle + Sample H_2O (M_3) (g)	94.0	94.1
5	Weighty of Bottle + Sample + H_2O (M_4) (g)	77.6	76.6
6	M_2-M_1 (g)	30.0	30.0
7	M_4-M_3 (g)	13.6	13.5
8	Gs	2.2	2.2
Result	Density Gs = 5	2.2	2.2

Field data, 2016

Given:

Gs = $(M_2-M_1)/ (M_2-M_1)-(M_4-M_3)$

iii. *Density Test*

Table 2: Calculation and Results: Data Sheet for Dry Density by Core Cutter Method

S/N	Observation and Calculations	Determination	No
		1	2
1	Core Cutter No	501.0	502.0
2	Internal Diameter (cm)	10.0	10.0
3	Internal Height (cm)	13.0	13.0
4	Mass Of Empty Core Cutter (M_1)g	900.0	900.0
5	Mass Of Core Cutter With Soil (M_2)g	2900.0	2600.0
6	Mass Of Wet Soil M= M_2-M_1	2000.0	1700.0
7	Volume Of Cutter (V) (cm^3)	1021.0	1021.0
8	Moisture Content	0.1	0.2
9	Bulk Density = Wt of Soil / Vol of Cutter (g/cm^3)	1.96	1.67
10	Dry Density (g/cm^3)	1.75	1.45

Field data, 2016

Volume of cutter (V), mass of wet soil (M), Bulk density and dry density were computed as follows:

$$V = \pi r^2 H \quad [5]$$

$$M = M_2 - M_1 \quad [6]$$

Global Journal of Researches in Engineering (E) Volume XVIII Issue I Version I Year 2018

Bulk density (Bd):

$$Bd = \frac{M_2 - M_1}{\pi r^2 H} \quad [7]$$

Dry density $= \dfrac{\text{Bulk density}}{1+\%moisture}$ [8]

iv. *Shear Strength of Soil*

First Test Run 10kg = 66 Div.

Second Test Run 20kg = 95 Div.

Third Test Run 30kg = 115 Div.

Shear Stress at Failure

Test Run 1 (10kg)

Shear at Failure = 0.577 x 66 = 38KN/m^2

Test Run 2 (20kg)

Shear Stress at Failure = 0.577 x 97= 56KN/m^2

Test Run 3 (30kg)

Shear Stress at Failure = 0.577 x 115 = 66 KN/m^2

Normal Stress

Test Run 1 (10kg)

Normal Stress = 21.8 + 10 x 2.75= 49 KN/m^2

Test Run 2 (20kg)

Normal Stress = 21.8 + 20 x 2.75 = 77 KN/m^2

Test Run 3 (30kg)

Normal Stress = 21.8 + 30 x 2.75 = 104.3 KN/m^2

Table 3: Shear Strength Failure Result

Test No.	Load	Shear Stress at Failure (KN/M2)	Normal Stress (KN/M2)
1	10	38.0	49.0
2	20	56.0	77.0
3	30	66.0	104.3

Field data, 2016

Table 4: Shear Strength at Depth 1m

Time (Seconds)	Mass 1 10 kg	Maxx 2 20 kg	Mass 3 30 kg
5			
10			1
15			1
20			1
30			1
60			1
90	1		13
120	2	29	56
150	4	46	75
180	16	54	87
210	30	61	93
240	40	71	93
270	47	76	106
300	50	82	105
330	52	86	109
360	53	87	112
390	57	92	115
420	61	97	
450	62		
480	65		
510	65		
540	66		
570			
600			
630			
660			
690			
720			

Field data, 2016

v. *Bearing Capacity Computation*

Length = 1800mm; Breadth = 1200mm; Depth = 1000mm

Qu = 1.3CNC + rDfNq + 0.4rBNr [9]

Qu = 1.3 x 8 x 37.2 + 9.8 x 1 x 22.5 + 0.4 x 9.8 x 1.2 x 19.7

Qu = 386.88 + 220.5 + 92.67

Qu = 700KN/m^2

Where factor of safety = 3

Allowable bearing capacity = 700/3 = 233KN/m^2

Net allowable load = 233 x 1.8 x 1.2= 503KN.

vi. *Consolidation Test*

For 2kg Load

$$\text{Stress } \tau = \frac{forcexbeamratio}{area} \quad [10]$$

Beam ratio = 10.00

Diameter of sample = 5cm =0.05m

$$\text{Area } = \frac{\pi d^2}{4} = \frac{3142x0.05^2}{4} = 1.96x10^{-3}m^2$$

$$\text{Stress } \tau_1 = \frac{2x9.81x10x10^{-3}}{1.96x10^{-3}} = 100.102\text{KN/M}^2$$

For 4kg Load

$$\text{Stress } \tau_2 = \frac{4x9.81x10x10^{-3}}{1.96x10^{-3}} = 200.204\text{KN/M}^2$$

For 6kg Load

$$\text{Stress } \tau_3 = \frac{6x9.81x10x10^{-3}}{1.96x10^{-3}} = 300.306\text{KN/M}^2$$

For 8kg Load

$$\text{Stress } \tau_4 = \frac{8x9.81x10x10^{-3}}{1.96x10^{-3}} = 400.408\text{KN/M}^2$$

For 10kg Load

$$\text{Stress } \tau_4 = \frac{10x9.81x10x10^{-3}}{1.96x10^{-3}} = 500.510\text{KN/M}^2$$

Calculation for Coefficient of Consolidation under Stress

Ao =1.594; Af = 1.678; T50 = 0.018; A50 = 1.640

Note: These values were read off from the consolidation graph.

$$A_s = \frac{A_f - A_o}{2} \quad [11]$$

Determination of Cv

$$\text{Cv} = \frac{0.20H^2}{t50} \quad [12]$$

where H = ½ (H1 + H2)

$$\text{Cv} = 0.20\frac{(0.0175)^2}{(0.018)^2} = 0.19m^2/mins$$

Determination of Co-efficient of Compressibility (Av)

$$\text{Av} = \frac{\Delta e}{\Delta p} \quad [13]$$

$$= \frac{0.0565}{200.204-100.102}$$

$$= 56 \text{ x } 10e^{-4}m^2/KN$$

Determination of Mv

$$\text{Mv} = \frac{av}{1+ef} \quad [14]$$

$$= \frac{5.6\,x10e^{-4}}{1+ef} = 3.7 \text{ x } 10e^{-4}m^2/KN$$

Initial Saturated Density (Psat)

$$\text{Psat} = \frac{Gs+e}{1+e}x\,pw \quad [15]$$

$$= \frac{2.59+0.129}{1+0.129}x1000$$

$$= 2408 \text{ kg / } m^3$$

Determination of K

$$\text{K} = \frac{Mv.cv.9.81}{1408\ x\ 62} \quad [16]$$

Table 5: Consolidation Result

ΔH	Δe = 0.0565 ΔH	e = e - Δe	h = Ho - ΔH	Effective Stress
0	0.00000	0.513	20.00	0.0000
0.86	0.04859	0.464	19.14	100.102
1.16	0.06554	0.448	18.84	200.204
1.43	0.08079	0.432	18.57	300.306
1.59	0.08984	0.423	18.41	400.408
1.69	0.09549	0.448	18.31	500.501

Field data, 2016

c) *Design of Structural Elements*

i. *Live Load for Footbridge*

For loaded length in excess of 30m

Live load, qk = k x 5.0KN/m^2 [17]

Where,

K= $\frac{\text{nominal HA UDL for appropriate loaded length (in KN/m)}}{\text{30 KN/m}}$ [18]

HA value for loaded length (32.4m) = 29.1 KN/m

Therefore,

K= $\frac{29.1\text{ KN/m}}{30\text{ KN/m}}$ = 0.97

But qk = k x 5.0 KNIm^2

Therefore, qk = 0.97 x 5.0 KNIm^2= 4.85 KN/m^2

ii. *Steel Plate for Treads*

Assume 300mm x 2400mm size

Plate Loading

Dead load from plate

=25.55kg/m^2x0.3mx2.4m+25.55kg/m^2x0.147mx2.4m

=27.41kgx10=274.1Nx10^{-3}=0.274KN

Characteristic impose load =2.4m x0.3m x4.85kN/m^2=3.49KN

Impose load on riser/tread=3.49x22=76.78KN

Design load, n=1.4gk+1.6qk

=1.4x0.274+1.6x76.78=123.23KN

Bending moment

BM_{max}= WL/8 [19]

=123.23x0.3/8 =4.62KNM

For short span girder

Span/depth=12

Span/depth=15

2390mm/depth=15

Depth=2390mm/15=159mm≈160mm

Since d=160mm>150mm; take p_y=225N/mm^2

Area of flanges, Af=M/d x p_y [20]

=4.62x106/160 x 225

4620000/36000=128.33mm^2

Width of the flanges is within the range of 0.5d

=0.5x160mm=80mm

128.33/80=1.604; assume 2mm

Assume a plate size 80mmx2mm

Area=80x2=160mm^2>128.33mm^2

The section chosen for the plate girder flanges as OK, the plate can be used.

Adopt the section 80mmx2mm

For the Web

T ≥ d/20 [21]

T≥ 160/20=1.333

Since T is a little bit small use the dimension of the flanges for the web 80mmx2mm

Section Classification

Flanges

T=160mm; p_y=225N/mm^2

But

Ɛ=(275/py)½ [22]

Ɛ=(275/225)½=1.11

b=$\frac{80-2}{2}$ =39

T=160mm

b/T=39/160=0.243

For welded section

b/T=13Ɛ

b/T=13x1.11 = 14.43>0.24

Therefore, the flanges are semi compact

Serviceability Deflection under Imposed Load

$$\delta = \frac{050}{384} \times \frac{wl^4}{EI}$$ [23]

W = 1.0gk + 1.0qk

= 13.814+38.39=52.204KN

Where, E=205x10^9N/mm^2

Global Journal of Researches in Engineering (E) Volume XVIII Issue I Version I Year 2018

$I=4010cm^4=4010x10^4mm^4$

$\delta = \frac{5}{384} x \frac{52.204x103Nx(7670)4mm4}{7670mmx205x109N/mm2x4010x104mm4}$

$= \frac{9.0334x1020}{2.4212x1025}mm$

$= 3.7309x10^{-5}mm$

Butδ < span/360

$= 3.7309x10^{-5}mm < 7670mm/360$

$= 3.7309x10^{-5}mm < 21.30mm$: $Mc>BM_{max}$: Ok

Moment Capacity

$$Mc=pySx \qquad [24]$$

$Sx=1010cm^3=1010x10^3mm^3$

$Mc=275N/mm2x1010x10^3mm^3$

$= \frac{277750000Nmm}{106}$

$=277.75KNm>253.76Knm$

Serviceability deflection under imposed load

$W=1.0gk+1.0qk$

$=25.23+75.60$

$=100.83KN$

$\delta = \frac{5wl4}{384EI}$ (from eq. 23)

Where $E=205X10^9N/mm^2$

$Ix=16000cm^4=16000x10^4mm^4$

$\delta = \frac{5 x 100.83 x 10^3N x (12990) 4mm^4}{384x12.99x103mmx205N/mm2x16000x104mm4}$

$= \frac{1.4355x1022mm}{1.6361x1026}$

$=8.77x10\text{-}5mm$: OK

But

$\delta<L/360$

$8.77x10\text{-}5mm<12990mm/360$

$8.77x10\text{-}5mm<36.08mm$: OK

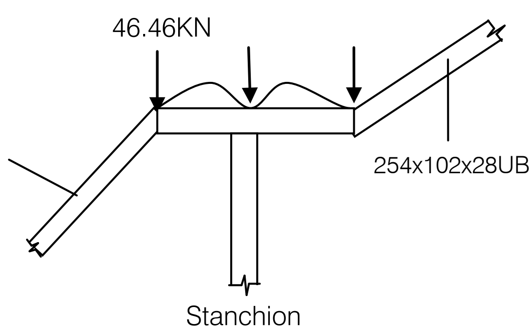


Fig. 1: Moment on the Stanchion

Bending moment

$BMmax=46.46x1.2+12.22x1.2/2$

$=63.08KNm$

Adopt 254x102x28UB since 77.43KNm>63.08KNm

Column

Dead load, gk =

$46.46+\frac{6.64}{2}+\frac{12.22}{2}+\frac{13kg/mx10x10\text{-}3}{2}+1.78$

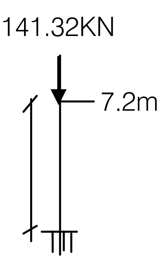
$= 57.74KN$

Live load, $qk = \frac{12.99}{2}x1.2x4.48$

$= 37.80KN$

Design load, $n = 1.4(57.74)+1.6(37.80)$

$= 141.32KN$



Effective length

$$Le = 1.0L \qquad [25]$$

Where, L =7200mm

Therefore, Le = 1.0 * 7200=7200mm

Assume section 203*203*46UC

T=11.0mm; area of section, $Ag = 58.7cm^2$

Radius of gyration, Vy=5.13cm

b/T =9.25; d/t=22.3

Since $T\leq 16mm$ $Py=275N/mm^2$

For outstand b/T=15Ɛ semi compact

For web d/t≤40Ɛ

Ɛ=(275/Py)½=(275/275)½ =1

b/T = 9.25≤15Ɛ

= 9.25≤15x1

= 9.25≤15

d/t = 22.3≤40Ɛ

= 22.3≤40x1

= 22.3≤40

$$PC= Agpc \qquad [26]$$

ʎ=LE/Vy=7200/(5.13x10)=140

Using curve C; assuming it buckle along y-y with S275

$Pc=76N/mm^2$

But $Ag =58.7cm^2=(58.7x10^2)mm^2$

PC= Agpc (from eq.26)

$= (58.7x10^2)\ mm^2 \times 76N/mm^2$

$=446120Nx10^{-3}$

=446.12KN>141.32KN: OK

Foundation

Axial Load = 141.32KN

Stanchion load from 203*203*46UC

$=46.1kg/m *10* 0^{-3}=0.461KN$

Allowable bearing capacity = $233KN/m^2$

Assume footing weight = 50KN

$Fcu=25Nmm^2$; $Fy =460N/mm^2$

Concrete cover =50mm

Live load

=4.85(2.4x7.67)+(1.2x7.67/2) 4.85

=111.59KN

Dead load = 141.32+0.461+50=191.78KN

Serviceability limit state

=1.0gk+1.0qk

=191.78+111.59=303.37KN

Required base area

Area of footing = $\frac{\text{Service load}}{\text{Bearing capacity}}$

$=\frac{303.37KN}{233KN/m2}$

=1.302m2

This provides 1300 mm x 1300 mm x 450 mm footing

Ultimate Limit State

n = 1.4gk+1.6qk

= 1.4x141.78+1.6x111.59

= 377.036KN

Earth pressure $=\frac{377.036KN}{1.69m^2}$

$= 223.09KN/m^2$

Assume 450mm Thick Footing

Concrete cover 50mm

Assume 20mm ø bar in both direction

Then

d=h-c- ø /2=450-50-20/2=390mm

Punching Shear

Critical perimeter =col perimeter+4∏d [27]

=4x203+4x3.142x390

= 5712mm

Area within Perimeter

$= (203+4d)^2-(4-∏)(2d)^2$

$= (203+4x390)^2-(4-3.142)(2x390)^2$

$= 2.58x10^6mm^2$

Punching shear force

VED=456(1.692-2.58)=125.9KN

Punching shear force=VED/col perimeter x d

=125.9x103/5712x390

$= 0.056N/mm^2$

Bending Reinforcement

$M=223.09x1.3x\frac{0.525^2}{2}$

= 39.97KNm

= 0.00808≤0.156

For the Concrete

$Mu=0.156fcubd^2$

$= 0.156x25x1300x390^2$

= 771.15KNm>39.97KNm

$K = M/ fcubd^2$

$= \frac{39.97x10^6}{25x1300x390^2}$

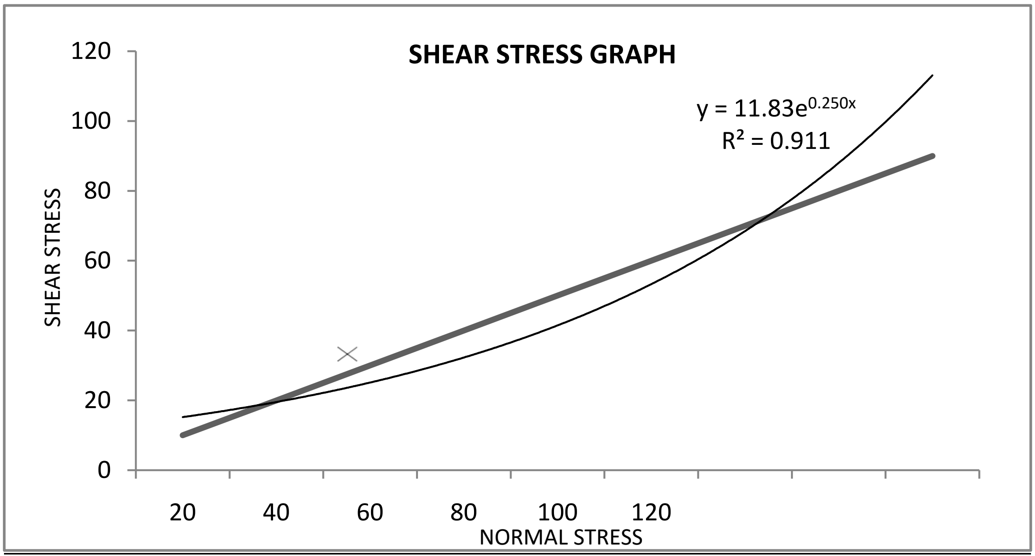
III. Conclusion

The outputs of the design analysis indicate that the chosen sections for all the structural members of the footbridge are adequate in term of ultimate and serviceability considerations. The soil analysis shows that it would be able to withstand the load from the columns and vibrations from vehicular movement.

References Références Referencias

1. Jeswald, P. (2005): How to Build Paths, Steps and Footbridges.Storey Publishing, North Adams, Massachusett.
2. Mosley, W. H., and J. H. Bungey, (1999): Reinforced concrete design. Palgrave, England.
3. Mugu, J. (1978): BS 5400 Part 2 Specification for Loads. Licensed Copy. University of Bath, England.
4. Muhammed, A., (2013): Evaluation of Pedestrian Bridges and Pedestrian Safety in Jordan. Published by IISE.
5. Newman, P. (2003): Archaeology of Dartmoor". Dartmoor National Part Authority, England. Pg 27.
6. PM News Nigeria (2015): Moves to Decongest Lagos Traffic". http://www.pmnewsnigeria.com (Accessed:18th June, 2015).
7. The Guardian (2015): On Pedestrian Bridge. http://ngrguardiannews.com (Accessed: 17/7/, 2015).

Global Journal of Researches in Engineering (E) Volume XVIII Issue I Version I Year 2018



CONSOLIDATION GRAPH