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Rehabilitation of Damaged Pier-Pile Joints in Tooz Bridge Using RC Confined By Steel Casing

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Abstract

Over the past years, Tooz bridge has been subjected to severely torrents led to erosion and collapse of soil at pile heads during execution, causing some damages to the heads of piles (joint between the pier and the pile) and its aspects. Technical reports indicated that the piles are implemented in diameter of (150 cm) with depths ranging from (12m) to (15m) and these ranges are more than what is required if we take into account soil capacity in this depths is over (20 ton/m^2), and the maximum capacity is within limits that are designed for. Test results indicated that the concrete of piles have a compressive strength of (31 MPa) to (59 MPa) with ultimate pile capacity ranged from (600 tons) to (720 tons) with deflection not exceed (5mm). The site surveys records deviations in location and coordinates of some piles (in varying degrees) in the longitudinal and transverse directions. After technical meeting between the constructing company and the consultant team, two methods for treatment have proposed, the first method is construct of pile cap around the damaged joints, while, the second method is using RC confined by steel casing. To save the time and cost, and to minimize the dead loads on piles, the second proposal is adopted and the structural analysis indicated that the used method is saved.

Keywords: Rehabilitation, Pier, Pile, Joint, Tooz, Bridge, Concrete, Steel Casing.

1-Introduction

Over the past years, Tooz bridge (Iraqi city situated in North-East Iraq and follow the Salah Al-Din Governorate) has been subjected to severely torrents led to erosion and collapse of soil at pile heads during execution, causing some damages to the heads of piles (joint between the pier and the pile) and its aspects as shown in Fig.(1).

Figure 1: Damages at the Heads of Piles (Joint) and at Aspects

After a site visit by the consultants team to the site of the bridge and in order to

Detail the damages and choose best treatment methods, the situation have been studied thoroughly based on available data to analyze and give the necessary recommendations to ensure the safety and durability of the piles for required design life according to the adopted standards and structural calculations for occurred deviations.

2-Bridge Description[1]

The Tooz Bridge (second way) consists of (20 span), the length of each span is (24m). Each span consists of eight pre-cast, prestressed girders, supported on transverse beam (cross-head beam) with dimensions of (11m), (2m) and (1.2m) for length, width and depth respectively. Each cross-head beam is support on three piers (columns) with diameter of (1.2 m) and each pier rest on

(1.5m) diameter bored-pile. The deck of bridge consist of (8m) passage for cars (carriageway) and two lanes for pedestrians (sidewalks), one with (1m) width and the other with (2m) width. It may be noted that the accomplished parts include all piles, some piers and pier cap (cross head beam), see Appendix-A.

3-Soil Investigation Report and Piles Test

3-1-Soil Investigation Report for the Site of Bridge (2)

Drilling records indicate that soil profile consists of a very dense layer of gravel mixed with sand till (4m) and sits on a layer of sand mixed with silty-clay. The water level varies between (9.4-10m) and the number of standard penetration is more than (100 strike) which indicates that the soil bearing capacity is very high. Table (1) shows the bearing capacity of soil at different depths.

Table 1: Bearing Capacity of Soil at Different Depths

Depth (m)	Bearing Capacity (KPa)
(1.5 – 2.5)	(140 – 160)
(2.5 – 3.0)	(160 – 180)
(3.0 – 4.0)	(180 – 200)

3-2-Piles Integrity Test (Low Strain Dynamic test) (3)

The technical report of Piles Integrity Test (PIT), which conducted on (27) piles, have been selected randomly, indicated that the piles are implemented in diameter of (150cm) with depths ranging from (12m) to (15m). Test results indicated that the concrete of piles have a compressive strength of (31MPa) to (59MPa), as well as the analysis of the results of the curves shows that implementing lengths ranging from (12m) to (15m) and is more than what is required if we take into account soil capacity in this depth of over (20 tons/m^2) and the maximum capacity is within limits that are designed for to transfer loads to the soil.

3-3-Dynamic load testing (or Dynamic loading) (3)

The technical report of Dynamic load testing, which conducted on (8) piles, have been selected randomly, indicated that the ultimate pile capacity ranged from (600 tons) to (720 tons) with deflection does not exceed (5mm). This indicating that the actual capacity (with safety factor=2) is greater than the required and the implemented lengths of piles is more than the lengths required taking into account the soil-bearing recorded in the site investigation report.

4- Deviations of piles (4)

The surveys submitted by the constructing company (Hamorabi State Constructing Contracts Company) records deviations in location and coordinates of some piles (in varying degrees) in the longitudinal and transverse directions. The biggest deviation in piles locations is reported and presented in Table (2) and Appendix- A.

Table 2: Deviations in Location and Coordinates of Piles*

Pile	Location	Deviations (cm)	Direction
First	P1,P2,P3,P4,P7	-7.5	Longitudinal -Downward
First	P11,P19	-5	Longitudinal -Downward
First	P8	+5	Longitudinal -Forward

*Appendix- A

The standard specifications for roads and bridges[5] in gloss (B5-03) page (B5-1) referred that the maximum allowable deviations in piles locations not greater than (15cm), so the deviations in the piles location are within the allowable limits.

5-Rehablitation of Damaged joints

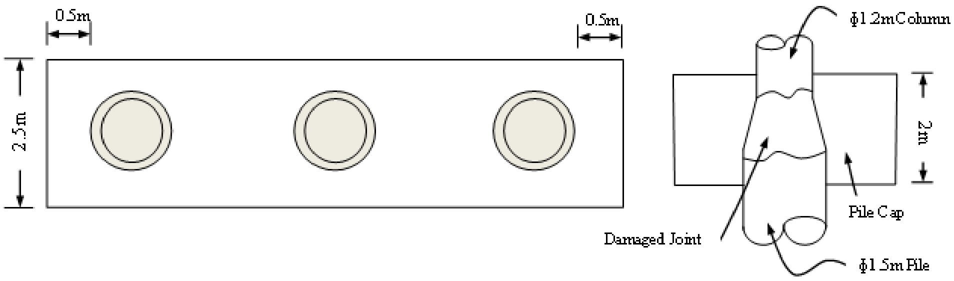
As mentioned before, because the implemented piles as well as part of the piers and the fact that the severity and speed of the water torrents led to erosion and collapse of soil at pile heads with poor implementation by the contractor causing damage. It's possible to increase the efficiency of concrete piers; and the joints between the piles and piers are preserved for longer operational life without any damage to the bridge in the future by rehabilitation using the following methods:-

1-Using concrete with layers of reinforcing bars or welded wire fabric (called locally BRC).
2- Using sheets or steel plates.
3-Using fiber reinforced polymers FRP (FRP Wrapping), such as GFRP or CFRP.

After a technical meetings between the constructing company (Hamorabi State Constructing Contracts Company) and the consultant team, two methods for treatment have proposed, as follows:-

First Proposal, which consists construct and poured of pile cap (around the joints) in dimensions of (2x2.5x10.5m) for depth, width and length respectively. The pile cap extend (0.5m) from the face of the pile (to the outside) and to be reinforced based on a certain engineering calculations and according to the available data and the results of structural analysis, Figure (2).

The construction of pile cap lead to increase the dead loads that reached to each pile (due to the weight of cap), Table (3), which can be saved and replaced by a local treatment to the damaged part of the joint and this will accelerate the treatment process and reduce the costs. Also, the construction of pile cap will change the system of implementation and transfer the loads from (Pile Bent System) to (Pile Cap System) and this requires a longer time of implementation and additional cost due to the increase in the quantities of concrete in all its items. It is possible to use this method, only, for the pier that have a big deviations (after ensuring that the piles have been implemented well and have very large carrying capacity).



Top view **Section**

Figure 2: First Suggestion, Pile Cap around Damaged Joints

Structural Analysis (Appendix-B), refer to that the axial load due to the weight of piles cap (if this proposal is adopted) on each pile is as shown in Table (3).

Table 3: Axial load due to Weight of Pile Cap*

Pile Location	Axial Load (kN)
Middle	538
Edge	362

* From Structural Analysis (Appendix-B)

Second Proposal, which consists of construction of concrete layers reinforced by steel bars or welded wire fabric (WWF) at the joint, Figure (3). To use this method, the joints must be totally or partially open from all sides with the need for simple machines and simple tools and the work can be completed in a short period of time (the state of our research). The advantages of this method are:-

1- Maintain the piers (which will be strengthened) from any damage, as well as maintain its esthetics shape, because this method require to remove of weak and vulnerable layers of piles only (local treatment for the damaged part) to reach to the sound concrete.

2- Strengthening of piers laterally through the use of a layer of reinforcing bars collars or stirrups with diameters and distances of certain distribution in vertical and transverse directions with using of external steel casing, attached around the treated area and extended vertically towards the top and the bottom with appropriate distances.

3- Strengthening of piers vertically through the use of a layer of reinforcing bars with certain diameters and distances extends in longitudinal direction and fixed perpendicular to stirrups.

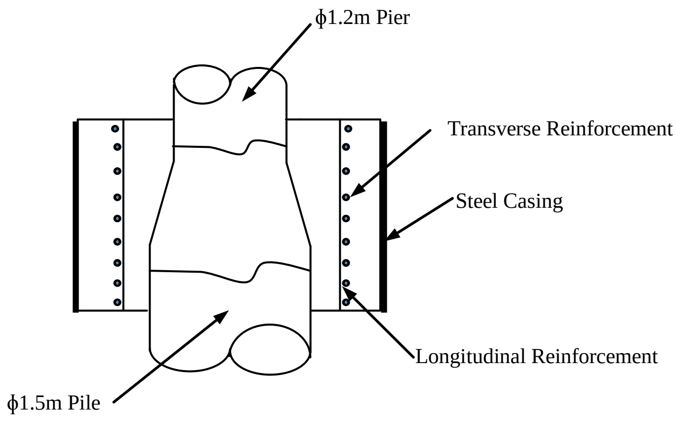


Figure 3: Second Suggestion, RC Confined By Steel Casing

The implementation of the above method leads to increase the efficiency of the joint through:-

1-Increase the flexural strength due to presence of both additional longitudinal reinforcement in outer face and steel casing.

2-Increase the shear strength due to presence of old longitudinal reinforcement (inside), additional reinforcement (outside) and steel casing (outside).

3- Increase the ductility due to the replacement of joint material from weak concrete (heads of piles) to reinforced concrete with large quantities of steel (the old, the additional longitudinal and transverse) and steel casing .

I believed that the local treatment is the best solution and has the benefits referred to above. To use this proposal, the transformed loads should be concentric (or with deviation within allowable limits). To ensure the transfer of loads from the superstructure, through the cross head beam then to piers and to piles (after the treatment of the joint, and after confirm that the piles have been implemented well and have large capacity) makes the proposal to be easy for application and adequate.

6-Treatment Using RC Confined By Steel Casing

For treatment of damaged piles, concrete layers with reinforcing bars covered by steel casing at the location of damaged joints are used, which performs two functions:

A- Perform as an external mold for new concrete of damaged joint.

B- Perform as a structural member from the joint and this lead to increases the flexural strength, shear strength and ductility as mentioned earlier.

In longitudinal direction, (2m) length (20ϕ25 mm) and (ϕ16mm @150mm) stirrups covered by (3mm) steel plate casing with length of (2m) are used. Concrete with compressive strength of (33MPa) is poured inside the mold (steel casing) after removing of weak layers and install of longitudinal and transverse reinforcement as shown in Figures (4) and (5).

Figure 4: Installation of Steel Mesh and Steel Casing (Before Concrete Poured)

Figure 5: Treatment of damaged joint by RC Confined by Steel Casing (After Concrete Poured)

7-Stability of Prestressed-Precast Concrete Girders

As a result of deviations in piles and piers, some differences in the lengths of the spans (increase or decrease) may be take place. To address this problem, we have proposed the following:

1- Use or manufacture of girders with unequal length (manufacture of girders based on actual length which may be increased or decreases due to deviation of cross beam).

2- Increase the seating distance (Enlarge seating Length) for Prestressed-Precast Concrete Girders) on the Cross Beam, for distance equal to deviation distance on the longitudinal direction and on both sides of the cross head beam.

8-Structural Analysis

8-1- Shear Effect (Shear in Piers)

The magnitude of the shear effect in reducing column strength is proportional to the amount of deformation that can be attributed to shear. According to the SSRC Guide [6], shear in columns is caused by:-

1-Lateral load, resulting from wind, earthquake, or other cause.

2-Slope, with respect to the line of thrust, due both to unintentional initial curvature and added curvature developed during the buckling process.

3-End eccentricity of load, introduced by the end connections or fabrication imperfections (for steel structures).

8-1 Shear Strength of Columns

The nominal or theoretical shear strength of a member (V_n) is provided by the concrete, the shear reinforcement and by the steel plate (casing).

$$V_n = V_c + V_s + V_{sp} \quad \dots (1)$$

The design shear strength of a member (V_u) can be expressed as follows:-

$$V_u = \varphi V_c + \varphi V_s + \varphi V_{sp} \quad \dots (2)$$

Due to small thickness of the steel plate, the contribution of steel plate (casing) to resist shear stress cab be ignored and Eq. (2) becomes:-

$$V_u = \varphi V_c + \varphi V_s \quad \dots (3)$$

ACI 318-05 Code[7] (Section 11.3.1.2) provides the following equation for determining the shearing force that can be carried by the concrete for a member subjected simultaneously to axial compression and shearing forces:-

$$V_c = (1 + \frac{N_u}{14Ag}) \frac{\sqrt{f_c^{'}}}{6} b_w d \quad \dots (4)$$

If the effect of the axial compression forces ignored, Eq. (4) becomes:-

$$V_c = \frac{\sqrt{f_c^{'}}}{6} b_w d \quad \dots (5)$$

According to the Commentary of the ACI 318-05Code in there section (11.3.3), the entire cross section in circular piers is effective in resisting shearing forces. The shear area, (b_w.d) in Eq. (5) then would be equal to the gross area of the pier. However, to provide for compatibility with other calculations requiring an effective depth, ACI requires that the shear area be computed as an equivalent rectangular area in which [8]:

$$b_w = D \quad \dots (6)$$

$$d = 0.8D \quad \dots (7)$$

The nominal shear strength of the transverse reinforcement (stirrups), V_s can be calculated from the following expression [8]:-

$$V_s = \min (A_v . f_y .d/s \, , 4 V_c) \quad \dots (8)$$

By using previous equations, the shearing force that can be carried by concrete is V_c=987 kN

8-2-Shear Stress on Columns

According to Iraqi Standard Specifications for Road Bridges Loading [9], the longitudinal forces (horizontal forces), due to military loading, equals to (30%) of the heaviest single military loading on the structure under consideration. Therefore, the longitudinal force is:-

$$L.F = 0.3x\,(HSML) \quad \dots (9)$$

Where

L.F= longitudinal force and HSML= 1060 kN

By using equation (9), L.F =0.3x1060=318 kN

Number of supports (spans) resisting longitudinal force=3

Shear force/ support=318/3=106 kN

Shear force/ column=106/3=36 kN

Assume load factor for shear =1.6

So, the ultimate shear stress/ column=V_u=1.6x36=58 kN

Since, V_c=987 kN > V_u= 58 kN the section is safe against shear stress

8-2-Flexural and Axial Capacity

Structural Analysis (Appendix-B) indicated that the axial and bending moment on the head of each pile is as shown in Table (4).

Table 4: Axial and Bending Moment on the Head of Pile*

Pile Location	Axial Load (kN)	Bending Moment (kN.m)
Middle	2824	810
Edge	2274	768

* From Structural Analysis (Appendix-B)

By using DT Column software [10] (circular column analysis), the interaction diagram (P-M diagram) is drawn and presented in Figure (6). The diagram refers to that the cross-section of pile and reinforcement is safe and adequate to resist the applied axial and bending moments.

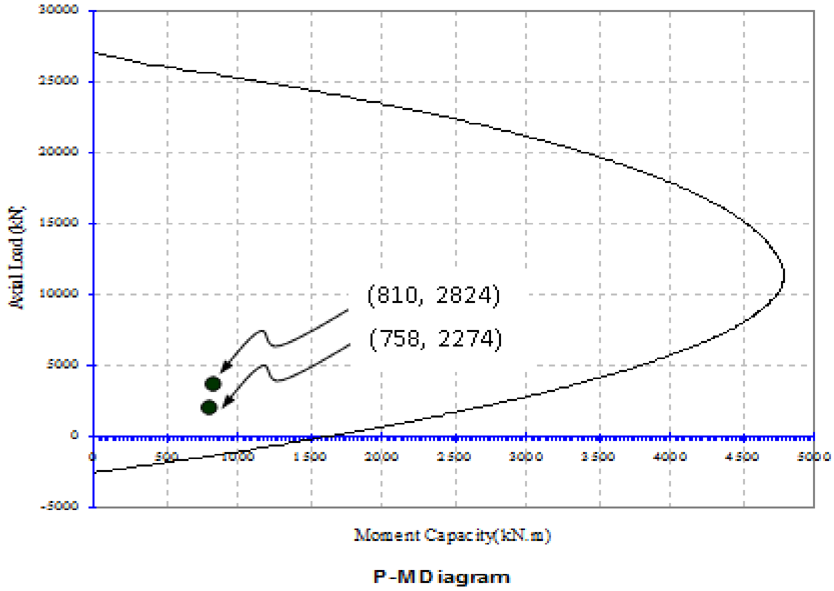


Figure 6: Interaction diagram (P-M diagram)

9-Conclusions

1- Investigations reports and structural analysis shows that the implemented piles are adequate to support and carry the applied loads (based on the assumed design age of designs and design) and the damage include the heads of some piles only.

2-Two methods for treatment have been proposed, construct of pile cap around the damaged joints and using of RC confined by steel casing around the damaged joints. To save the time and cost, and to minimize the dead loads on piles, the second proposal is adopted and the structural analysis indicated that the used method is saved.

10-References

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[3] Al-Tariq Engineering Bureau, "***Pile Dynamic Test Report for Tooz Khormato Bridge***", Report No. 47-D-2010, Baghdad, Iraq, November 2011.

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[6] Structural Stability Research Council (SSRC) Guide, "***SSRC Centrally Loaded Column Research Inventory***", USA, February 2003.

[7] ACI Committee 318, *"**Building Code Requirements for Structural Concrete**"*, (ACI 318-05) and Commentary (ACI 318R-05), American Concrete Institute, Farmington Hills, MI, USA, 2005, 430pp.

[8] McCormac, J. C. and Nelson J. K., "***Design of Reinforced Concrete- ACI 318-05 Code Edition***", 7th Edition, John Wiley & Sons. Inc., USA, 2006, 721pp.

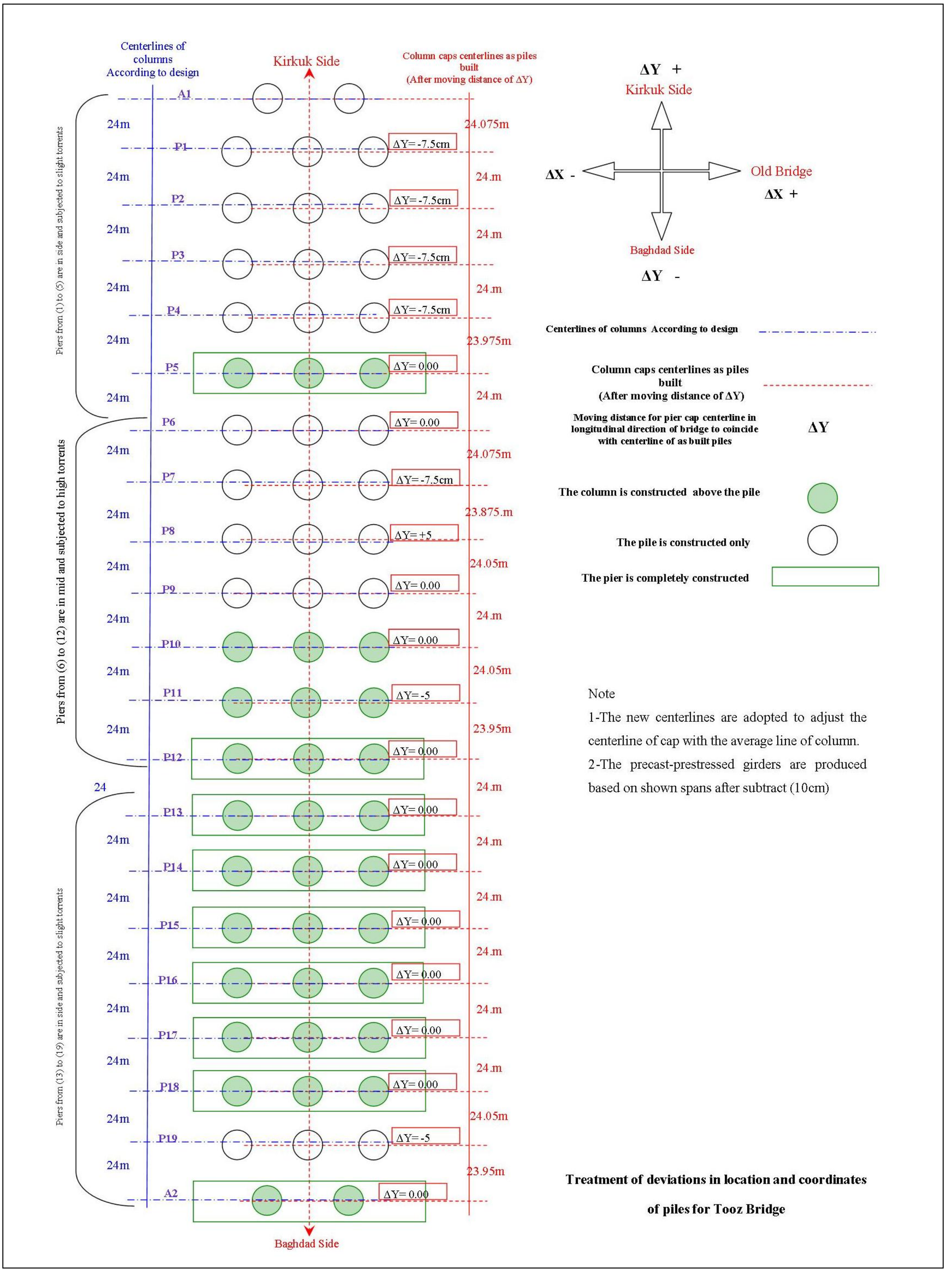
[9] Ministry of Housing and Construction, State Organization of Roads and Bridges, "***Iraqi Standard Specifications for Road Bridges- Loading- Revision 1978***", Baghdad, Iraq.

[10] DT Column software (http://www.dtware.com)

11-Notation

A_g	Gross Section Area
A_v	Area of Vertical Reinforcement (Stirrups legs)
b_w	Width of Section
d	Effective Depth
D	Diameter
f_c'	Cylinder Compressive Strength
f_y	Yield Tensile Strength of Steel Reinforcement
N_u	Axial Compressive Force
P	Axial Force
M	Bending Moment
V_c	Shear strength of concrete.
V_n	Nominal or theoretical shear strength of a member.
V_s	Shear strength of Steel Reinforcement.
V_{sp}	Shear strength of Steel Plate Casing.
φ	Reduction Factor for Shear

Appendix-A-Eccentricity of Piles in (Y) and (X) Directions [4]



Appendix-B- Structural Analysis

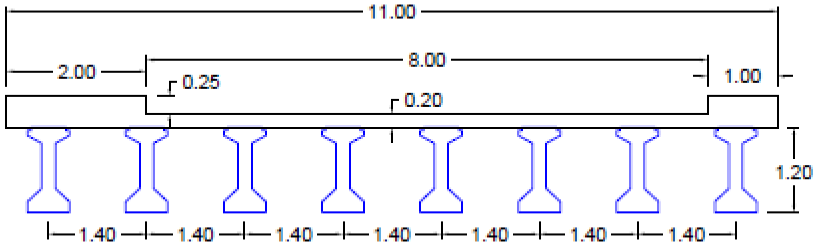


Figure (Appendix-B-1) Bridge Cross Section

Loads on bridge

1-Super Structure Dead Load

Deck slab weight=0.2x11x25x24=1320 kN (Assume γ_c= 25 kN/m^3)

Side walk weight=0.25x(1+2)x25x24=450 kN

Asphalt pavement (Surfacing)=0.06x8x22x24=254 kN (Assume $\gamma_{Asphalt}$= 22 kN/m^3)

Diaphragms* (assumed)=3x(0.3x1.2x1.2)x25x7=227 kN

*(Three diaphragms per span with seven segments of dimensions of (0.3x1.2x1.2m))

Railing and serviceability utilities=0.5x2x24=24 kN

Pre-stressed precast girders =8*242.7=1942 kN

Σ D.L=4217 kN (Per Span)

2-Live Load (Iraqi Standard)

2-1- UDL+ KEL (Civilian)

Carriageway= 8m →No. of Lanes=3

Lane width=8/3=2.67m

UDL=32.2 kN/m-lane → Modified UDL=32.2x2.67/3.05=28.15 kN/m-lane

KEL=122.4 kN → Modified KEL=122.4x2.67/3.05=107.2 kN

Based on Iraqi Standard, the UDL+ KEL are distributed throughout the Carriageway (lanes) as follows:-

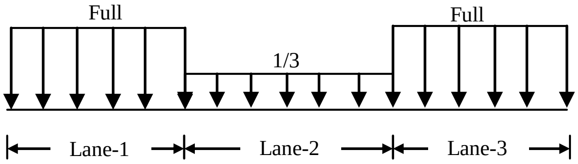


Figure (Appendix-B-2) Distribution of UDL+ KEL

Σ UDL=2X28.15x24+1x1/3x28.15x24=1576.4 kN

Σ KEL=2x107.2+1x1/3x107.2=250.32 kN

Side walk live load*=4x(1+2)x24=288 kN

* live load=4 kN/m^2

Σ L.L=2115 kN (Per Span) ← Governed

2-2- One Tracked (Military Loading)

Weight of 1- Tracked =1x900=900kN (Impact Factor for 24m Span=0.304)

=(1+0.304/2)x900=1037 kN

Side walk live load*=4x(1+2)x24=288 kN

* live load=4 kN/m^2

Σ L.L=1325 kN (Per Span)

3-Cross Head Beam Dead Load

Cross Head Beam Weight=11x1.2x2x25=634 kN

4-Load on Piers

Total Load= $DL+LL+DL_{X\text{-}Beam}$=4217+2115+634=6966 kN

Load/m=6966/11=633 kN/m

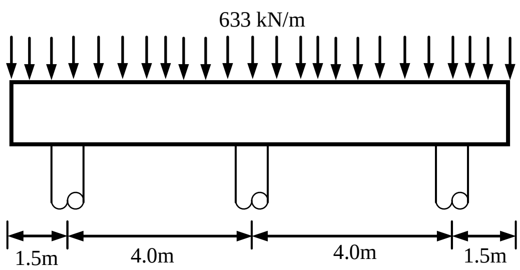
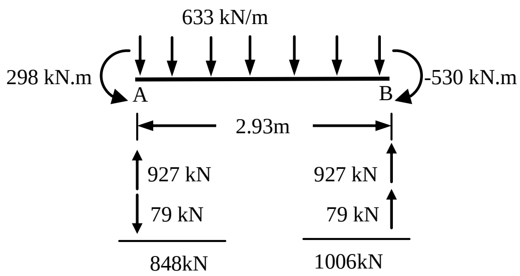


Figure (Appendix-B-3) Load on Cross Head Beam



Clear span=4-0.89x1.2=2.93m

Cantilever=1.5-0.89x0.6=0.97m

$M_A=(633x(0.97)^2)/2=298$ kN.m

$M_B=633/4x(-(2.93)^2/2+(0.97)^2)=-530$ kN.m

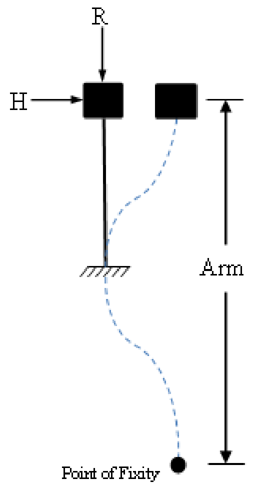
Load transformed to Middle Pier=2x1006+0.89x1.2x633=2688 kN

Load transformed to Edge Pier=848+ 0.89x1.2x633+0.97x633=2138 kN

Weight of columns=5*π/4x(1.2)^{2}x25=136 kN (Assume column height=5m)

Total Load on the Middle Pier (R) =2688+136=2824 kN

Total Load on the Edge Piers (R) =2138+136=2274 kN



5-Bending Moment on Piles

Arm= deck slab thickness+ girder height+cross beam depth+

Scour+2xColumn height

=0.2+1.2+1.2+4+2x5=16.6m

Longitudinal force = 0.3x 1060=318 kN (see Eq. (9))

Number of supports resisting longitudinal force=3

Shear force/ support=318/3=106 kN

Lateral load (H) =Shear force/ column=106/3=36 kN

Bending Moment on Piers= 36x16.6=598 kN.m

Figure (Appendix-B-4) Moment Arm

Additional Moment due to Edge Pier Deviation =2272*0.075=170 kN.m

Additional Moment due to Middle Pier Deviation =2824*0.075=212 kN.m

Total moment on Edge Pier=170+598=768 kN.m

Total moment on Middle Pier=212+598=810 kN.m

Check the capacity of pile by using interaction diagram (See Figure (6))

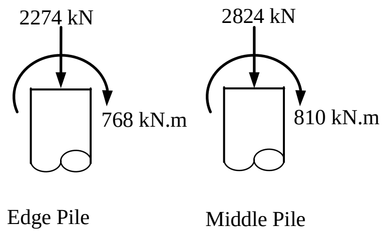


Figure (Appendix-B-5) Load on piles

6-Weight of Pile Cap (if used)

Dimension of pile cap=2x2.5x10.5m

Weight of pile cap=24x2x2.5x10.5=120 kN/m

Clear span=4-0.89x1.5=2.67m

Cantilever=1.25-0.89x0.75=0.583m

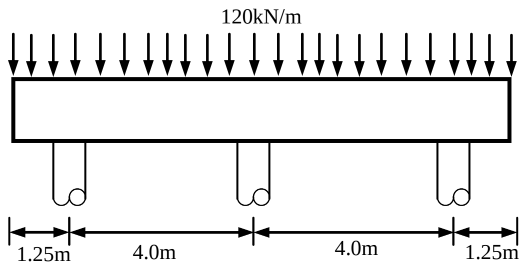
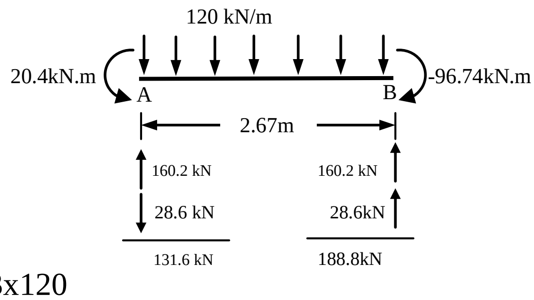


Figure (Appendix-B-6) Load on Pile Cap

$M_A=(120x(0.583)^2)/2=20.4$ kN

$M_B=120/4x(-(2.67)^2/2+(0.583)^2)=-96.74$ kN



Load transformed to Middle Pile=2x188.8+0.89x1.5x120
=538 kN

Load transformed to Edge Pier=131.6+ 0.89x1.5x120+0.583x120
=362 kN

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اعادة تاهيل المفاصل المتضرره بين الدعامات والركائز في جسر الطوز باستخدام الخرسانه المسلحه المقيده بغطاء حديدي

د.علي حميد عزيز
الجامعه المستنصريه- كلية الهندسه- قسم الهندسه المدنيه

الخلاصة:

خلال السنوات الماضية تعرضت جسور الطوز الى شدة من السيول ادت الى تعرية وانهيار التربة عند رؤوس الركائز اثناء التنفيذ مما تسبب في بعض الاضرار الانشائية لرؤوس الركائز (المفصل الرابط بين العمود و الركيزة) وتاكل جوانبها. اشارت التقارير الفنيه الى ان الركائز الخرسانية نفذت بقطر (150سم) وباعماق تراوحت بين (12 مترا) الى (15 مترا) وهو اكثر مما مطلوب لو اخذنا بنظر الاعتبار تحمل التربة بهذا العمق الذي يزيد على (20 طن لكل متر مربع) وان تحملها الاقصى ضمن الحدود التي صممت لاجلها. اوضحت نتائج الفحص ان خرسانة الركائز ذات مقاومة انضغاط تصل بين (31 ميكاباسكال) الى (59ميكاباسكال) وان تحمل الركائزالاقصى قد تراوح بحدود من (600 طن) الى (720 طن) مع هطول لا يتجاوز(5ملم). المسوحات الموقعيه سجلت وجود انحرافات في مواقع واحداثيات بعض الركائز (بدرجات متفاوته) و بالاتجاهين الطولي والعرضي. بعد عقد لقاء فني بين الشركه المنفذه و الفريق الاستشاري، تم اقتراح طريقتين للمعالجه، تضمنت الطريقه الاولى انشاء قبعة ركائز حول المفاصل المتضرره ،اما الطريقه الثانيه فتتضمن استخدام الخرسانه المسلحه المقيده بغطاء حديدي حول المفصل. لتوفير الوقت والكلفه، ولتقليل الاحمال الميته الواصله للركائز، تم تبني المقترح الثاني وقد اشار التحليل الانشائي بان الطريقه المقترحه امنه.