Rehabilitation of Damaged Columns-Piles Joints in Quri Chi Bridge Using Composite Section of RC Confined By Steel Casing

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Abstract

Over the past years, Quri Chi Bridge has been subjected to severe torrents led to erosion and collapse of soil at pile heads, causing some damages to the heads of piles (joint between the pier and the pile) and its aspects (edges). Technical reports indicated that the piles are implemented in diameter of (1.2m) with depths ranging from (11.5m) to (12m) and these ranges are more than what is required if we take into account soil bearing capacity at this depths is over (200kN/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 (28.5MPa) to (33.5MPa) with allowable (factored) pile capacity of (6500kN) without any deviation. Two methods for treatment have been proposed; The first method is to construct a pile cap around the damaged joints, while, the second method is using RC jacket confined by steel casing. Structural analysis, based on ACI-318 empirical equations and finite element analysis, by using ANSYS software is performed. The second proposal is adopted and the structural analysis indicated that the used method is safe.

Keywords: Rehabilitation, Pile, Joint, Bridge, Composite Section, Steel Casing, ACI-318, ANSYS.

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1-Introduction

Over the past years, Quri Chi bridge (Iraqi town situated in North-East Iraq and it is part of Kirkuk Governorate) has been subjected to severe torrents led to erosion and collapse of soil at pile heads, causing some damages to the heads and upper sides of piles (joint between the column and the pile) as shown in Figure (1).

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

Site visit was carried out by the consultant's team to the site of the bridge in order to inspect the damages and choose best treatment methods. The situation was studied thoroughly based on available data to analyze and give the necessary recommendations to ensure the safety and durability of the piles for the required design life according to the adopted standards and structural calculations.

2-Bridge Description[1]

The Quri Chi Bridge consists of (5 simply supported spans) of (20m) each. Each span consists of thirteen pre-cast, prestressed I-girders of (850mm) depth of AASHTO standards, supported on transverse beam (cross-head) with dimensions of (11m), (2m) and (1.2m) for length, width and depth respectively. Each cross-head is supported on three columns (extended columns) with diameter of (1.2 m) and each column rests on (1.2m) diameter bored-pile. The deck of bridge consists of (8m) carriageway and two sidewalks, one with (1m) width and the other with (2m) width. The abutments consist of cross-head beam with wing walls, supported on two (1.5m) diameter bored-piles. It may be noted that, all components of the bridge are completely executed.

3-Soil Investigation Report and Piles Capacity

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 (20m) and rests on a layer of sand mixed with silty-clay. The water table level is at (9.5m) depth and the number of standard penetration is (50-100) strikes, 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)	**(110 – 120)**
(2.5 – 3.0)	**(120 – 130)**
(3.0 – 4.0)	**(130 – 140)**

* Reference (3)

3-2-Piers (columns) Piles Capacity[(3)]

Site implementation technical reports indicated that the piles were implemented in diameter of (1.2m) with depths ranging from (11.5m) to (12m) and ultimate (factored) capacity of (6500 kN). Test results indicated that the concrete of piles have a compressive strength of (28.5MPa) to (33.5MPa) and it is more than what is required if we take into account soil capacity and friction at this depth of over (200kN/m^2) and the maximum capacity is within limits that are designed for to transfer loads to the soil.

4-Deviations of piles[(3)]

Based on surveys submitted by the constructing company (Hamorabi State Constructing Contracts Company), no deviations in location and coordinates of piles were recorded. Nevertheless, the standard specifications for roads and bridges[(4)] in clause (B5-03) page (B5-1) referred that the maximum allowable deviations in piles locations should not be greater than (15cm).

5-Rehabilitation 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 have led to erosion and collapse of soil at pile heads, with poor implementation by the contractor, causing damages. It's possible to increase the efficiency of concrete columns; and the joints between the piles and columns 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 gauge.

2- Using sheets or steel plates.

3- Using carbon fiber reinforced polymers CFRP (CFRP Wrapping).

After technical meeting between the constructing company (Hamorabi State Constructing Contracts Company) and the consultants team, two methods for treatment were proposed, as follows:-

The first proposal consists of constructing and pouring of pile cap (around the joints) in dimensions of (2x2.5x10.2m) for depth, width and length respectively. The pile cap extends (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 carried by 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 column that have a big deviations (after ensuring that the piles have been implemented well and have very large carrying capacity).

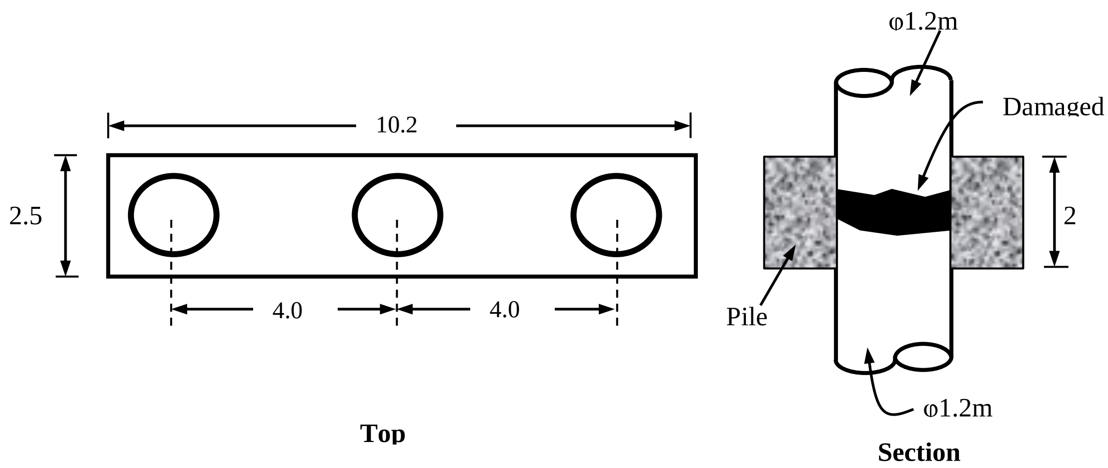

Figure (2) First Proposal, Pile Cap around Damaged Joints

Structural Analysis shows that the additional axial loads due to the weight of pile 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

The second proposal consists of construction of concrete jacket 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 columns (which will be strengthened) from any damage, as well as maintain its esthetics shape, because this method requires to remove the weak and damaged parts of piles only (local treatment for the damaged part) to reach to the sound concrete.

2- Strengthening of columns 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 and using external steel casing, around the treated area and extending vertically towards the top and the bottom with appropriate distances.

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

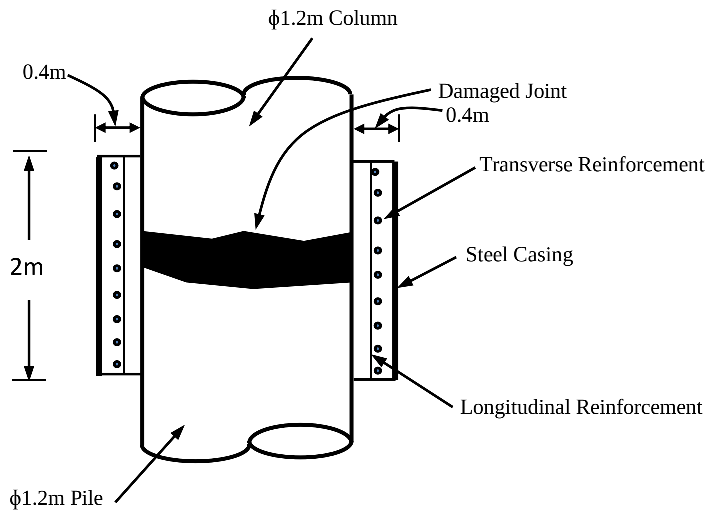

Figure (3) Second Proposed, RC Confined By Steel Casing

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 .

It is 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 then to columns and to piles makes the proposal to be easy for application.

6-Treatment Using RC Confined By Steel Casing

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

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

2- 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 (24ϕ25 mm) and (ϕ16mm @150mm) in horizontal direction covered by (3mm) thickness steel plate casing with length of (2m) and (2m) diameter are used. Concrete with compressive strength of (33MPa) is poured inside the mold (steel casing) after removing of weak layers and placing of longitudinal and transverse reinforcement as shown in Figures (4) to (7). It may be noted that the origin longitudinal reinforcement of piles was (24ϕ25 mm).

Figure (4) Preparation of Joints (Removing of weak Layers)

Figure (5) Placing of Steel Mesh and Steel Casing (Before Concrete Poured)

Figure (6) Steel Casing (Molds) and Concrete Poured (Concrete Pumping)

Figure (7) Treatment of damaged joints by RC Confined by Steel Casing

(After Concrete Poured)

7-Structural Analysis

7-1-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 [(5)], shear in columns is caused by:-

1- Lateral load, resulting from wind, earthquake, braking force 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).

7-1-1-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).

$$V_n = V_c + V_s + V_{sp} \quad \text{……………….Eq.(1)}$$

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

$$V_u = \varphi\, V_c + \varphi\, V_s + \varphi\, V_{sp} \quad \text{…………..Eq.(2)}$$

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:-

$$V_u = \varphi\, V_c + \varphi\, V_s \quad \text{………….……..Eq.(3)}$$

ACI 318-05 Code[6] (Clause 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:-

$$V_c = \left(1 + \frac{N_u}{14Ag}\right)\frac{\sqrt{f_c'}}{6} b_w d \quad \text{………..Eq.(4)}$$

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

$$V_c = \frac{\sqrt{f_c'}}{6} b_w d \quad \text{…………………..Eq.(5)}$$

According to the Commentary of the ACI 318-05 Code in there Clause (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[7]:

$$b_w = D \quad \text{……………..…………..Eq.(6)}$$

$$d = 0.8D \quad \text{……………. ………...Eq.(7)}$$

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

$$V_s = \min\,(A_v.\, f_y.d/s\ ,\ 4\, V_c) \quad \text{…......Eq.(8)}$$

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

7-1-2-Shear Stress on Columns

According to Iraqi Standard Specifications for Road Bridges Loading[8], 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) ……..Eq.(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 and Shear force/ column=106/3=36 kN
Assume load factor for shear =1.6
So, the ultimate shear stress/ column=Vu=1.6x36=58 kN
Since, Vc=987 kN > Vu= 58 kN the section is safe against shear stress.

7-2-Flexural and Axial Capacity

Structural analysis 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

By using DT Column software[9] (circular column analysis), the interaction diagram (P-M diagram) is drawn and presented in Figure (8). The diagram refers to that the cross-section of pile and reinforcement is safe and adequate to resist the applied axial and bending moments.

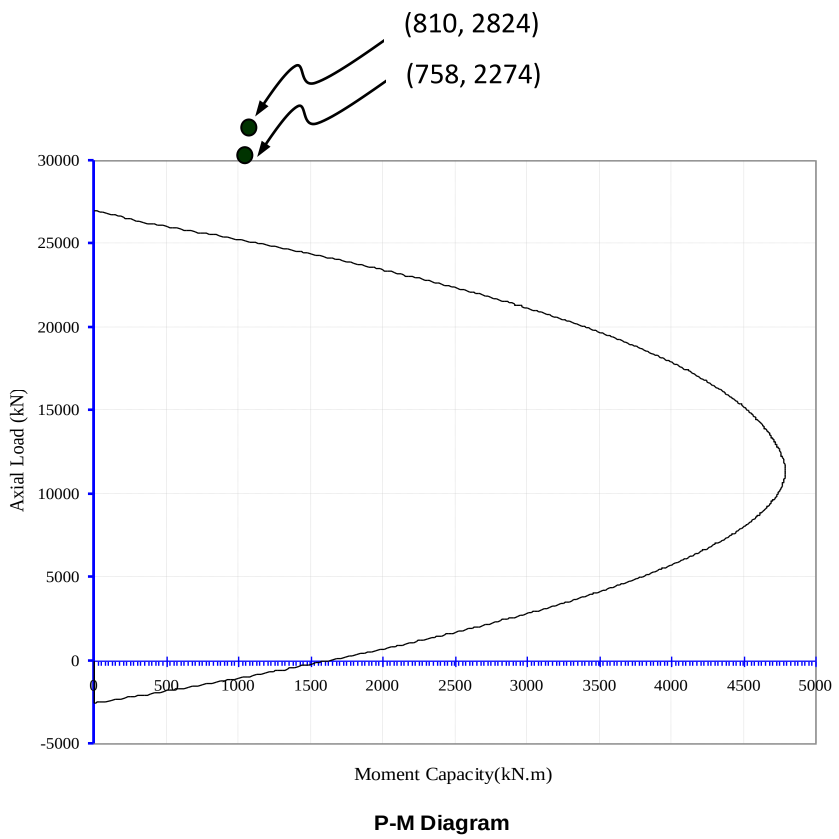

Figure (8) Interaction diagram (P-M diagram)

8-Finite Element Analysis

Three dimensional non-linear finite element analyses, using the ANSYS Software (Version-11), are carried out to study the behavior of the treated columns.

8-1-Geometry

The details of the column are shown in Figure (9). Analysis were carried out on a circular cross-section column with (1.2m) diameter and reinforced longitudinally by equally spaced (24 ф25mm) bars with transverse reinforcement of (ф16 mm) ties spread in (150mm). The characteristic strength of concrete was (25N/mm^2) and (33N/mm^2) for concrete at the damaged joint and other members respectively. Due to confinement of the damaged joint, additional reinforcement and additional concrete by the steel casing, perfect bond between all of these elements was assumed (no needs to contact elements). To provide the perfect bond, the link elements for the reinforcing bars (longitudinal and transverse) and shell elements for steel casing were connected between nodes of each adjacent concrete solid element, so the materials shared the same nodes.

8-2-Element Types[10]

Three types of elements are employed to model the columns. An eight-node solid element, SOLID-65, was used to model the concrete (old, damaged and new concrete). The solid element has eight nodes with three degrees of freedom at each node, translation in the nodal x, y, and z directions. The used element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. A LINK-8 element capable of plastic deformation was used to model the reinforcement bars; two nodes are required for this element. Each node has three degrees of freedom, translation in the nodal x, y, and z directions. SHELL181 was used to model the steel casing. The element SHELL-181 is suitable for analyzing thin to moderately-thick shell structures. It is a four-node element with six degrees of freedom at each node: translations in the x, y, and z directions, and rotations about the x, y, and z-axes.

8-3-Properties of Materials
8-3-1-Concretes

For finite element modeling, concrete constitutive stress-strain curve in compression can be described by isotropically, multi-linear stress-strain relationship. Constitutive model (surface of failure) in ansys can be specifying only by two constants (tensile strength (f_t) and compressive strength of concrete ($f_c^{'}$)) as given by criterion of Willam, and Warnke[11].

In this paper, transfer coefficients of shear for opened cracks (β_o) and closed cracks (β_c) are assumed to be (0.2) and (0.25) respectively. These values are selected to avoid convergence problems during iteration. The stress-strain curve for concrete in tension is assumed to be linear-elastic up to the maximum tensile strength. Smeared crack approach is used to model the concrete cracking. Poisson's ratio, for finite element modeling of concrete is assumed to be (0.2). In order to modeling the finite element of concrete, empirical equations of ACI-318 Code are used to determine the young's modulus (E_c) and tensile strength (f_t), as listed in Table (5).

8-3-2-Steel Plate and Reinforcement

Elastic modulus and yield stress for the steel plate and reinforcement used in FEM follow the design material properties. The steel for the finite element models is assumed to be an elastic-perfectly plastic material and identical in tension and compression. Von-Mises failure criterion is adopted for modeling. Young's modulus of (200GPa) and Poisson's ratio of (0.3) are utilized in FEM for steel plates and reinforcement. The input data for the concrete, steel reinforcement and steel casing are shown in Table (5).

Table (5) Material Properties and Input data

Properties	Materials					
	Concrete			Reinforcement		Steel Casing#
	Old	Damaged	New	Longitudinal	Transverse	
f_c' (MPa)	33	25	33	-	-	-
f_y (MPa)	-	-	-	420	420	420
E(MPa)	27000*	23500*	27000*	200×10^3	200×10^3	200×10^3
υ	0.2	0.18	0.2	0.3	0.3	0.3
β_c	0.25	0.25	0.25	-	-	-
β_o	0.2	0.2	0.2	-	-	-
f_t	4.0^+	3.5^+	4.0^+	-	-	-

*** $E_c=4700\sqrt{f_c'}$** **+ $f_t=0.7\sqrt{f_c'}$** ***# 3mm Thickness***

8-4-Finite Element Modeling and Meshing

Due to simple geometry of the treated columns, entire (full) column is used for modeling. In the beginning, the columns and steel plates (casing) are modeled as lines, then areas, and finally as volumes (3D Modeling). After creating of volumes, meshing of the finite element model is needed. In this stage, the FEM model is divided into a number of small elements. When the model problem is solving, the stresses and deformations (strains) are estimated at the Gaussian points of these small elements. Best results can be achieved by dived (meshing) the model into square (or rectangular) elements, Figures (9) and (10).

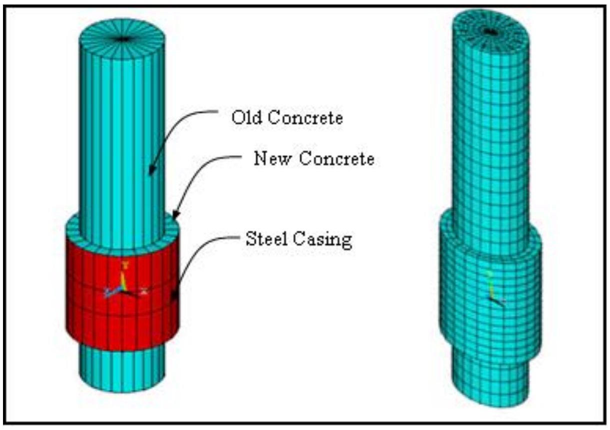

Figure (9) Column Modeling and Meshing

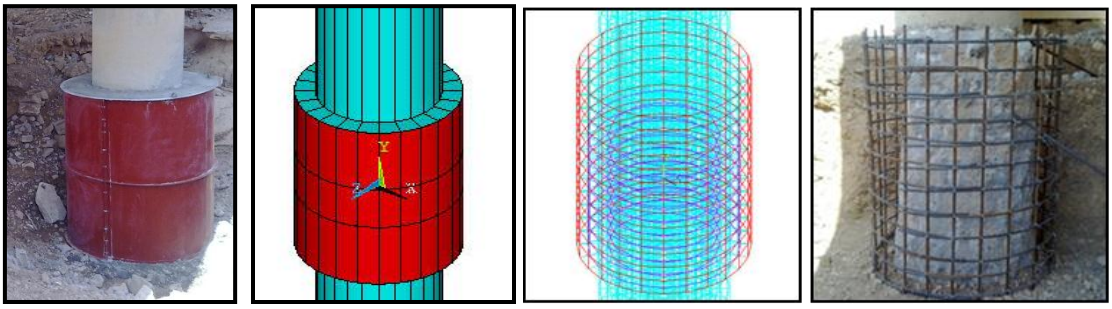

Figure (10) Actual and Modeling of Steel Reinforcement and Casing in Damaged Joints

8-5-Load Application and Boundary Conditions

To ensure that modeled columns are behaved as repaired columns, boundary conditions (displacement constrains) at the ends of column should be satisfied. Thereafter, the bottom end is assumed to be fixed (fully constrained), while, the top end is assumed to be free. Since the external load was applied directly on the columns and as a basic of FEM, the applied load is represented by an equivalent nodal force on the top nodes of the column (assuming equally distributed of applied load). The applied load is divided into load steps and done incrementally up to failure (based on Newton-Raphson technique). At a certain stages in the analysis, load step size is varied from large (at points of linearity in the response) to small (when cracking and steel yielding occurred). The failure is assumed to be occurred when the solution, for a minimum load is diverging and the models have a large deflection (rigid body motion). It may be noted that, the applied load and moment is same as indicated in Table (4).

8-6-FEM Results

The validity of the proposed method to repair the damaged joints is checked through comparisons between analytical (FEM) results and site implementation technical reports of RC columns.

8-6-1-Stresses in Concrete

FEM results indicated that the ultimate compressive stress in concrete (old, damaged and new concrete) does not exceed (7N/mm^2) which represents (28%) and (21%) of the ultimate compressive strength of the damaged and new concrete respectively; Figure (11) shows the stresses in concrete.

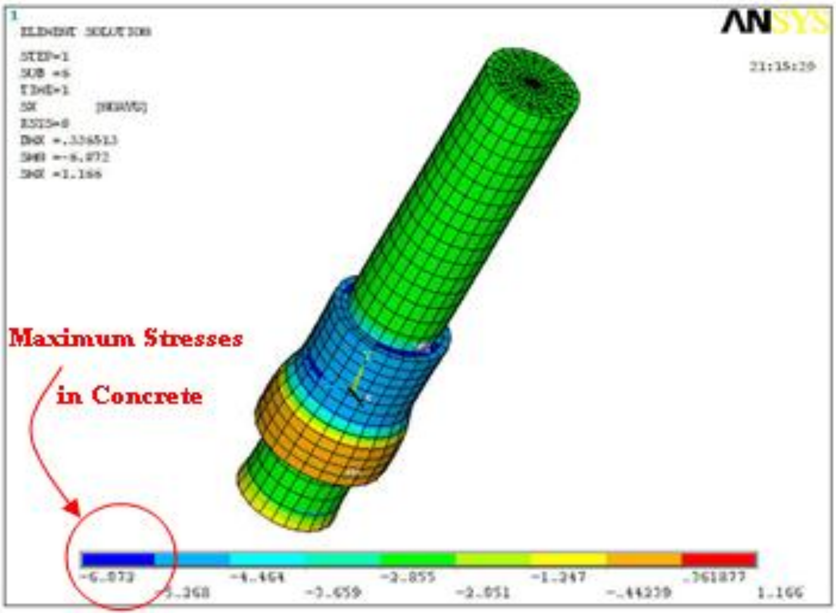

Figure (11) Stresses in Concrete

8-6-2-Stresses in Steel Casing

FEM results indicated that the maximum tensile stress in steel casing does not exceed (32N/mm^2) which represents (7.6%) of the yield tensile strength of steel plate (casing); Figure (12) shows the stresses in steel casing.

The theoretical results from Finite Element Analysis show higher capacity to carry the applied axial forces and moments than the recorded values (in technical reports).

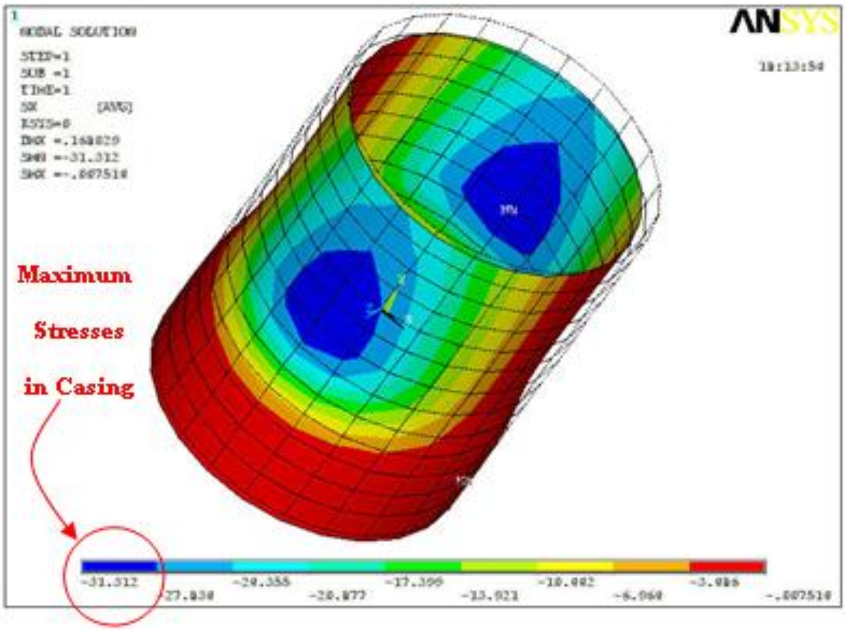

Figure (12) Stresses in Steel Casing

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.

3-Based on FEM analysis (using ANSYS program), it can be concluded that the computational finite element models adopted in the current study are useful and able to simulate the behavior of the treated columns. The analytical results indicated that the using of RC confined by steel casing around the damaged part is safe.

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Acknowledgment

This paper is part of a consulting contract in favour of Hamorabi State Constructing Contracts Company. The Author presents thanks and appreciation to the Engineering staff of Hamorabi State Constructing Contracts Company and State Commission of Roads and Bridges for their efforts in completing this work

اعادة تاهيل المفاصل المتضرره بين الاعمدة والركائز في جسر قورى- جاي باستخدام مقطع مركب من الخرسانه المسلحه المقيده بغطاء حديدي

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المستخلص

خلال السنوات الماضية تعرض جسر قورى جاي الى شدة من السيول ادت الى تعرية وانهيار التربة عند رؤوس الركائز 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 (1.2م) وباعماق تراوحت بين (11.5مترا) الى (12 مترا) وهو اكثر مما مطلوب لو اخذنا بنظر الاعتبار تحمل التربة بهذا العمق الذي يزيد على (200 كيلونيوتن لكل متر مربع) وان تحملها الاقصى ضمن الحدود التي صممت لاجلها. اوضحت نتائج الفحص ان خرسانة الركائز ذات مقاومة انضغاط تصل بين (28.5 ميكاباسكال) الى (33.5ميكاباسكال) وان تحمل الركائزالاقصى كان بحدود (6500 كيلونيوتن) وبدون اي انحراف. 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. تم اجراء تحليل انشائي باسستخدام المعادلات التجريبية لمدونة الخرسانة الاميركية (ACI-318) وباستخدام طريقة العناصر المحددة (باستخدلم البرنامج ANSYS). تم تبني المقترح الثاني وقد اشارت نتائج التحليل الانشائي بان الطريقه المقترحه امنه

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