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CASE STUDY: INVESTIGATING THE FAILURE OF KHANI POST-TENSIONED CONCRETE BOX GIRDER BRIDGE IN ZAKHO – IRAQI KURDISTAN

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ABSTRACT: Khani bridge in Zakho / Iraqi Kurdistan is under construction on Khabour River. The bridge has one simply-supported span of 63m and designed to have a non-prismatic post-tensioned concrete single-cell box girder section. This bridge, which consists of two box girders (Girder 1 for left side and Girder 2 for right side) encountered an excessive deflection during construction after removing the shores and caused a uniform deflection along the span with a maximum deflection of more than 47cm in the middle for Girder 1 bridge and 67 cm for Girder 2.

This paper investigates the cause of this failure and provides the evidence to be a lessen-learned for the future of post-tensioned concrete industry in the area and improve the local bridge construction practice and management.

KEYWORDS: Prestressed, Concrete, Bridge, Box Girder, Deflection.

1 SYNOPSIS AND HISTORICAL RECURRANCE

Khani Bridge, which failed during construction because of excessive deflection on both of the bridge girders, was a box girder, post-tensioned concrete bridge located in Zakho City in Kurdistan Region of Iraq built to help people cross Khabour River. This town, which is located a few kilometers from the Iraqi-Turkish border, is famous for the landmark historical Dalal Bridge. Zakho was known by the ancient Greeks and was described by Xenophon where ten thousand Greek mercenary units crossed Dalal Bridge during the Cyrus the Younger attempt to wrest the throne of the Persian Empire from his brother, Artaxerxes II in 401 to 399 BC [1]. Dalal Bridge is still standing after more than two millenniums and every year thousands of tourists visit it and cross it.

Bridges fail from different reasons but the majority of them fail because of floods and scouring and very few of them fail for design and construction errors. According to a study by Multidisciplinary Center for Earthquake Engineering Research (MCEER) of the State University of New York at Buffalo, only 6 percent of the bridges failed, failed during construction and only one percent have distress type of failure [2]. However, this data is not necessary applicable for a third-world region like Kurdistan where regulations' enforcement is not as strong. If compared to all the bridge types, girder bridges failed the most as shown in Figure 2.

Figure 1. Historic Dala Bridge in Zakho

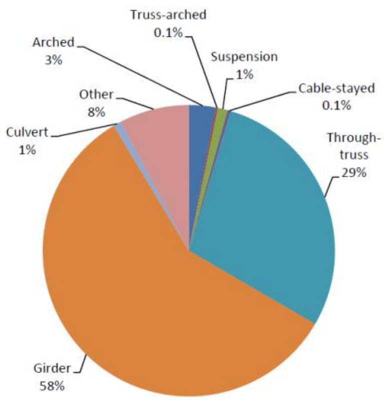

Figure 2. MCEER bridge failure survey [3]

From 1980 to 2012, 1061 bridges failed in US. This number looks big at a glance because it appears that an average of approximately 48 bridges failed annually across US. However, compared to the total number of bridges in US, which is 611,845 bridges, this is only 0.173%. [4]. Another interesting of MCEER study that was performed on 538 bridges that failed shows that 247 of them totally collapsed, 284 of them partially collapsed, and only 9 of them failed by distress as shown in Table 1.

Table 1. Causes of failure per MCEER [2]

Cause of Failure	Failure Types		
	Total Collapse	**Partial Collapse**	**Distress**
Design Error	38% (8)	52% (11)	10% (2)
Lack of Maintenance	67% (2)	33% (1)	0% (0)
Deficiency in Construction	32% (10)	65% (20)	3% (1)
Material Defect	23% (3)	46% (6)	31% (4)
Earthquake	38% (6)	63% (10)	0% (0)
Scour	50% (61)	50% (60)	0% (0)
Flood	75% (83)	25% (27)	0% (0)
Collision	39% (44)	60% (68)	1% (1)
Environmental Degradation	29% (12)	69% (29)	2% (1)
Overload	76% (71)	24% (23)	0% (0)
Fire	50% (12)	50% (12)	0% (0)
Wind	78% (35)	22% (10)	0% (0)

Another study published by Ohio State University for the bridges collapsed during the years between 2000 and 2012 also shows the number of bridge failures per different causes of failure as shown in Figure 3. It can be noticed that the results of this graph are consistent with that of Table 1.

The figure shows that the dominant causes are flood, scour, overload, and collision while design error is a part of the miscellaneous cases, which form a small portion among the causes.

Bridges may fail and collapse if they are not designed, constructed, and maintained properly. Bridge collapses and failures are not something new and they happened many times in the history of human being. Because of the nature of bridges in currying pedestrians and vehicles driven by humans, their collapse may be fatal and catastrophic. The earliest bridge collapse known to human being is the collapse of Stirling Bridge in Scotland in September 1297 which collapsed from an overload by the attractors who were watching Stirling Battle. Solders changed the style of marching on bridges after the catastrophic collapse of Angers Bridge in 1850 [5].

Bridges are vulnerable during construction especially if the contractor is not experienced with mega constructions. In July of 1996, a part of a section of the new freeway bridge in Halawa Valley in Hawaii that consisted of four 35-meter-long, 60-ton girders supporting an unfinished section of freeway bridge collapsed and injured some workers. A bridge collapsed during construction in Viet Nam in September of 2007 killing 52 workers and injuring 97 [5].

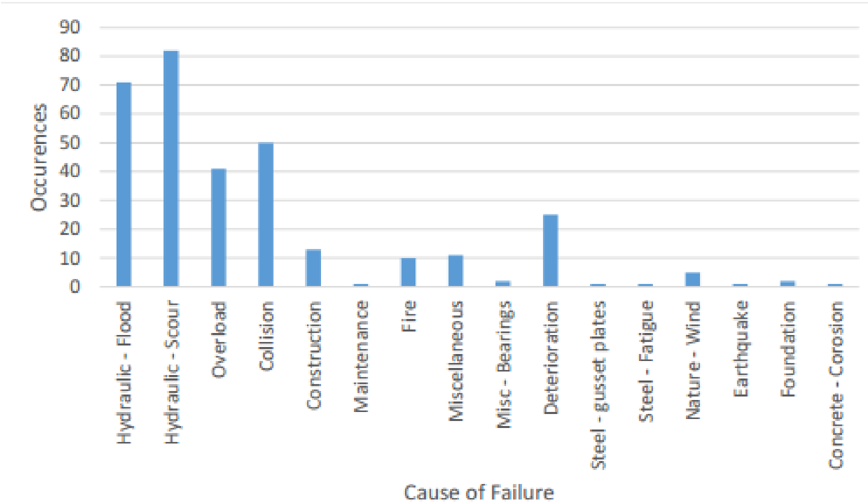

Figure 3. Number of bridge failures per causees of failure [4]

A similar bridge to Khani Bridge that was also failed is the original Koror-Babeldaob Bridge. They both had post-tensioned single cell concrete box girder non-prismatic sections. Koror-Babeldaob Bridge was a balanced cantilever prestressed concrete box girder bridge with a main span of 240.8 meters and a total length of 385.6 meters built in the Republic of Palau in 1978 by Socio Construction Company [6].

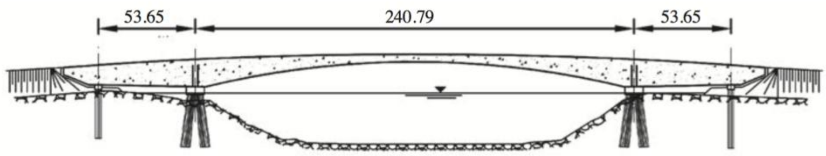

Figure 4. Kornor-Babeldaob Bridge [6]

On the evening of September 26, 1996 and after 18 years of service collapsed abruptly and catastrophically killing two people and injuring four. The collapse came few years after the two independent teams of international bridge experts, Louis Berger International and the Japan International Co-Operation Agency (JICA) had evaluated the walkway and declared it safe.

2 PROBLEM STATEMENT

Only few hundred meters away from the historical Dalal Bridge, the new Khani Bridge failed during construction with an excessive deflection after removing the shoring forms. Khani Bridge was designed as a post-tensioned box girder bridge in late 2012 and the design was implemented in 2014 and 2015. The construction started on March 3rd, 2014 to replace the old steel truss bridge shown in Figure 5. The new bridge consists of two portions (one for each side of the road). To reference them in this report, the north side portion is called Girder 1 and the south side called Girder 2. This bridge encountered an excessive deflection during construction after removing the shores and caused a uniform deflection along the span with a maximum deflection of more than 47 cm in the midspan for Girder 1 and 67cm for the Girder 2. Immediately after this form removing and failure the work stopped and the construction halted and an intensive investigation started from the owner of the bridge that is the Municipality of Zakho that is a subsidiary of the Governorate of Duhok.

3 RESEARCH SIGNIFICANCE

This research paper is written to show the results of a study done to investigate the causes of the failure and present the determinations. It points out the reasons as to why this bridge deformed immediately and uniformly along the major axis just by removing the shoring forms. The aim of this paper is to manifest the root causes and indicate the lessons learned for future benefit of bridge construction industry in the area. For this purpose, the following aspects of the bridge design and construction were studied:

- Construction items including material test procedure and results to assure that the quality of the materials meet the requirements of the applicable standards and codes.
- Tendon elongation amount and procedure to determine whether the tendons were stressed as required.
- Review the bridge design plans and calculation to check whether there is any design errors that contributed to the failure. This design review and check included the two tendon profiles used in the original design that was done in 2012 and the revised one that changed the tendon profile in 2014.

4 INVESTIGATIONS

4.1 Construction

The new Khani Bridge that is a post-tensioned concrete box girder system was planned to replace the old steel truss bridge shown in Figure 5. The old bridge was two spans that were supported on concrete abutments on the ends and a hammerhead wall pier at the center.

Figure 5. Old Khani Bridge

The new bridge is designed to have a non-prismatic section as shown in Figure 7. The bridge has a massy and fleshy section as it can be noticed that is 4m high at the ends and gradually reduces to 2m at the center. The bottom edge curve is closer to a straight line than an arch (see Figure 7 and Figure 8).

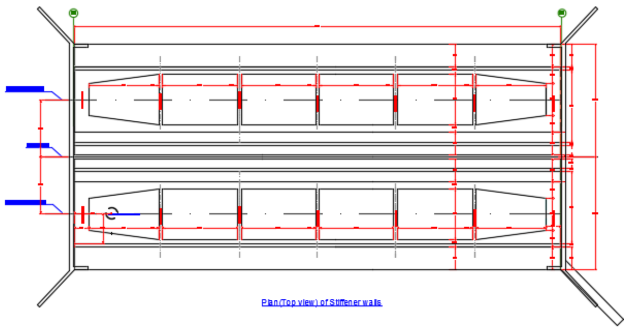

Figure 6. The new Khani Bridge plan

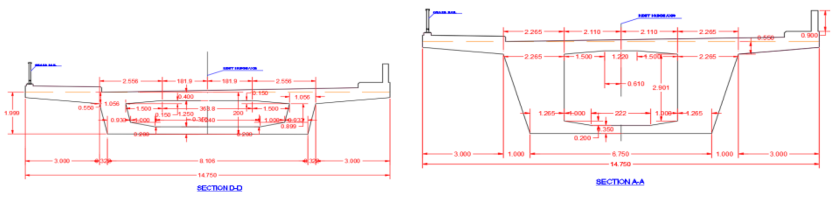

Figure 7. Center and end sections of the new bridge

Figure 8. The new post-tensioned concrete box girder bridge

On May 29, 2015, the scaffoldings to shore the superstructure were installed and the conventional reinforcing started on June, 7, 2015 and casting concrete started on July 4, 2015.

Figure 9. Installation of conventional reinforcement and PT ducts

The post-tensioning ducts and anchorages were installed on July 23, 2015. On August 18 and 19, 2015, the top flange slabs of the left side and right side were casted concrete. The integrated concrete sidewalks of Girder 1 were casted concrete on August 27 and 29. The top flange slab of Girder 2 was also casted concrete in two stages on September 3rd and 5th while the integrated sidewalks were casted on September 9th and 10th of the same year. The strands were prepared and pulled started on September 13 and ended on September 16, 2015. This means that the tendons were pulled only three days after the top slab was cast.

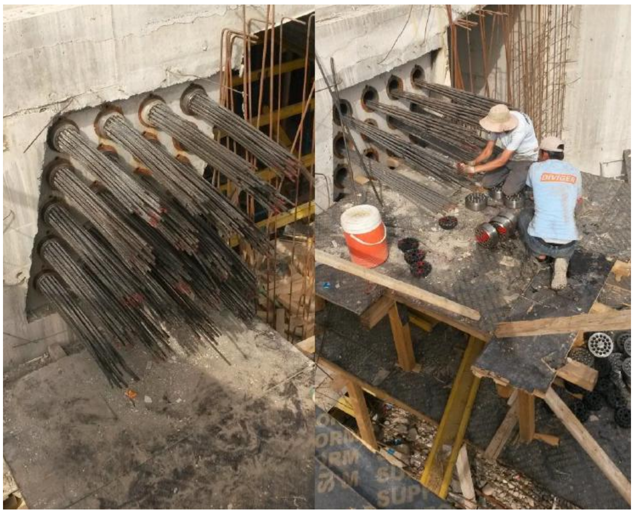

Figure 10. Tendon installations

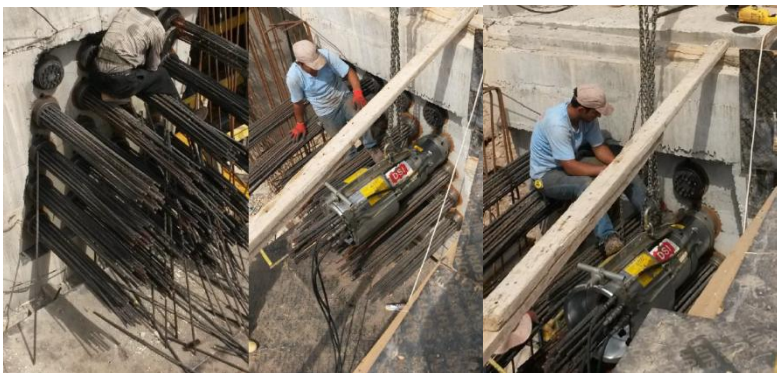

Figure 11. Tendon stressing process and grouting

Another problem is that the tendon stressing started from the top tendons and came down to the bottom ones rather than starting from the bottom first (Figure 11). The ducts were grouted between September 16 and 19. Standard specifications and common practice were not followed for the grouting procedure. The shoring forms were stripped on October 13, 2015.

Figure 12. Shore removing and form stripping

Immediately after the shoring removed, both girders of the bridge started to deflect downwards with an average speed of more than 3cm a day in the first 10 days and then started to slow down but did not stop until Girder 1 stopped finally at 47 cm and Girder 2 stopped at 67 cm. This deflection took approximately 90 days.

4.2 Tendon profile

The first thing caught attention was the design of the bridge. People asked whether the design was done by an experienced engineer in designing PTC box girder bridges. The first revision of the design drawings were issued for construction and submitted in May 2012 shows that the tendon profile was designed to have a concaved up shape (smily face) (Figure 13). In 2014, during the construction of the bridge and prior to tendon installations, another revision of the drawings was submitted.

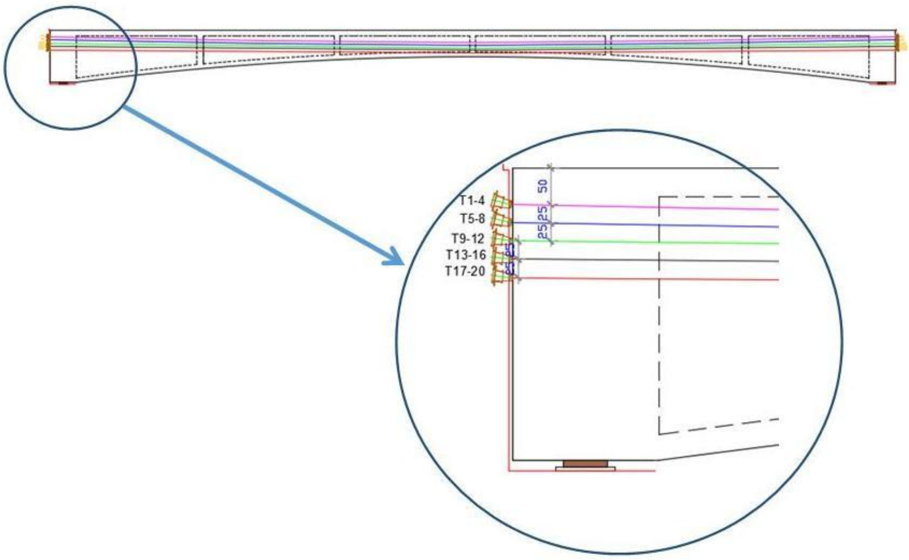

Figure 13. Tendon profile of the original 2012 design

The most significant change in this revision was that of the tendon profile by lowering the centroid of the tendon group at the ends to match the box section centroid and eliminating the eccentricity at the girder ends with an intention to distribute the stress at the ends and reduce the applied moment (Figure 13). This change of the tendon profile was done together with adding two strands per each tendon during a revision process to improve the design. These changes raised skepticism immediately after the bridge failure.

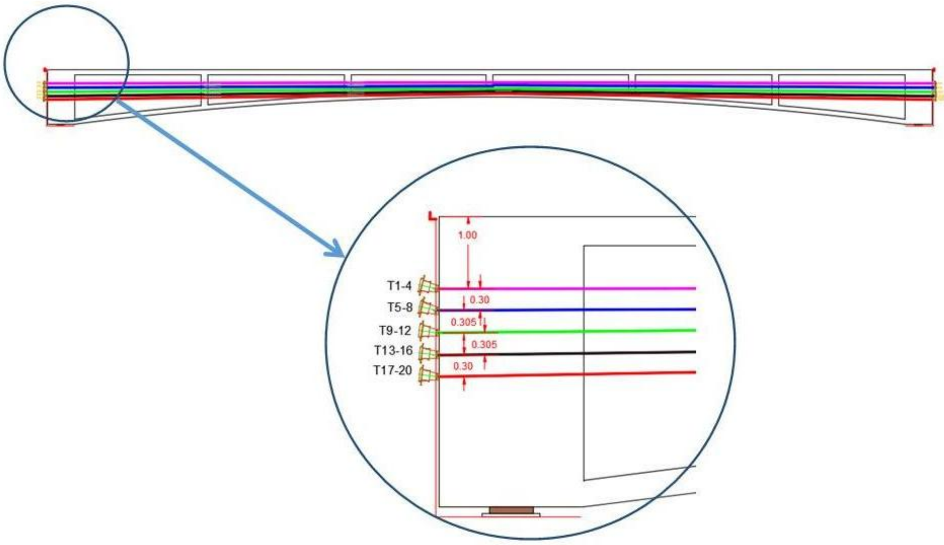

Figure 14. Revised 2014 tendon profile shape

As such, the construction supervision team started to look through the documents and raised a point of the change orders as is usual in any structure failure to question the change orders. After the claim went to the court as a lawsuit, the court requested an intensive investigation and forensic analysis to be performed by prestressed concrete bridge experts in Kurdistan Region. This paper is the result of this intensive investigation performed by the authors.

4.3 Interface horizontal shear

A third party conducted an investigation based on Duhok Governorate request and they submitted a report concluding that horizontal shear failure at the interface between the top deck slab and the walls is to be blamed for the excessive deflection. They supported their claim by some pictures taken of the cracks occurred at the interface for a good distance from both of the ends (see Figure 15 and Figure 16). This third party also concluded that the total deflection of this bridge according to the current design must not exceed 13cm.

Figure 15. Cracks at interface from both ends

Figure 16. Cracks and concrete scalping at interface (photo courtesy of Mega)

The same report claimed that the cracks at the interface between the top deck and the walls reduced the section and as a result, leads to the excessive deflection. This claim does not have a strong ground because these cracks that mentioned in Mega report did not appear until the deflection reached 180mm according to the site engineers. In fact, the authors of this paper visited the site on the first week that the failure happened and took pictures of the same areas that showing no cracks. The construction joints shown in Figure 17 are those that submitted as part of the design drawings.

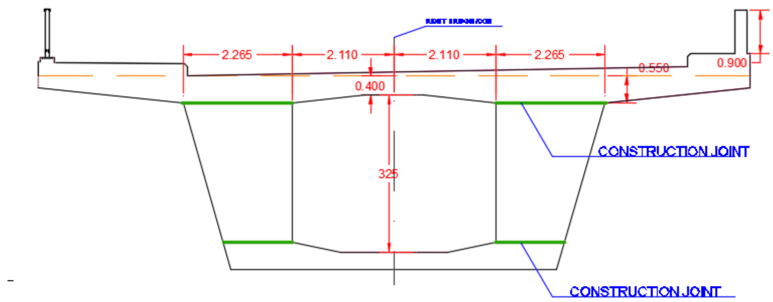

Figure 17. Construction joints in the bridge body

The horizontal shear resistance was calculated and showed that the resistance to the horizontal shear is sufficient to hold the section together as a composite section. This analysis is based on a fact that the Contractor did not follow the design notes and drawings in roughning the top surface of the walls but rather left it as is (see Figure 18). As such, the cohesion and friction factors were lowered for unroughened surface. Despite that, this analysis showed that the interface shear resistance is sufficient to hold the section together.

Cohesion Factor: $c := 0.52\text{MPa}$ Friction Factor: $\mu := 0.6$

Area of interface concrete surface for two webs: $A_{cv} := 2 \times 2.27\text{m} \times 1\text{m} = 4.54\,\text{m}^2$

ϕ 20 is used as interface horizontal shear reinforcement: $d_{20} := 20\text{mm}$ $f_y := 420\text{MPa}$ $S := 150\text{mm}$

$$A_{20} := \pi \times \left(\frac{d_{20}}{2}\right)^2 = 314.16 \times \text{mm}^2$$

Area of reinforcing steel for two intrface surface webs: $A_{vf} := 4 \times \frac{A_{20}}{S} \times 1\text{m} = 8377.58 \times \text{mm}^2$

Permanent net compression force is zero: $P_c := 0\text{kN}$

Nominal interface shear resistance: $V_{ni} := c \times A_{cv} + \mu \times \left(A_{vf} \times f_y + P_c\right)$

$V_{ni} = 4471.95 \times \text{kN}$

Resistance factor: $\phi := 0.9$

$\phi V_{ni} := \phi \times V_{ni} = 4024.76 \times \text{kN}$

Moment of inertia: $Q := 9.47\text{m}^3$ $I := 35.79\text{m}^3$

Ultimate horizontal shear: $V_{hu} = \frac{V \times Q}{I}$

$V_{hu} := \frac{1.25 \times 10653\text{kN} \times Q}{I}$ $\boxed{V_{hu} = 3523.47 \times \text{kN}}$ $\boxed{\phi V_{ni} = 4024.76 \times \text{kN}}$

Horizontal_Shear := if$\left(\phi V_{ni} \ge V_{hu}, \text{"OK"}, \text{"NG"}\right)$

Horizontal_Shear = "OK"

Figure 18. Top surface of the walls left as is without roughening

4.4 Strand specification and required elongation

The original cables were tested in Turkey in May 2015 and the results are shown in Table 2.

Table 2. Original test results of the strand cables prior to bridge failure

Specimen No.	E_p From Test (MPa)	f_{py} From Test (MPa)	f_{py} AASHTO (MPa)	f_{pu} (MPa)	Elongation (%)
1	116280	1150	1674	1979	4
2	111188	1100	1674	1979	4
3	114586	1150	1674	1973	4
4	112274	1125	1674	1919	3.5
5	114259	1150	1674	1971	3.5
6	115259	1150	1674	1971	3.5
Average	**113974**	**1138**	**1674**	**1965**	**3.75**

Like any other structural failure, the test results of the materials used in the construction were skeptical. Duhok Governorate ordered a retest of the strand cables after they realized that the modulus of elasticity shown in Table 2 is extremely low. Therefore, the strand cables sent to the lab again to double check and see whether the results match that was used in the design. The cable tests came back with higher quality than what was expected as shown in Table 3.

Table 3. Test results of the strand cables after bridge failure

Specimen No.	Breaking Load (kN)	Yield at 1% Strain (kN)	f_{py} (Mpa)	f_{py} (Mpa)	A_{ps} (mm^2)	Elongation (%)
1	301.2	263.4	1751	2003	150.4	8.9
2	297.1	258.3	1719	1977	150.3	9.6
3	299.3	257.7	1716	1993	150.2	10.3
Average			**1729**	**1991**	**150.3**	**9.6**

Just by comparing the data in Table 2 and Table 3, it can be noticed that the elongation percentage changed dramatically from 3.75% originally as shown in Table 2 to 9.6% in Table 3. Apparently, the original test results were accepted by the project managers without objecting the results. The authors of this paper carried out an analysis and changed the design input with the numbers of Table 3. The results of the analysis showed that the main cause for Khani Bridge failure is lack of stress in the tendons because they were not been elongated enough to reach the required stress. In another word, the concrete did not have enough compressive strength when the shores were removed. Another issue that contributed to this failure was the timing of the tendon pulling. They were pulled only three days after casting the top slab without assuring that the concrete strength reached the required number. Also, the project managers did not have any test results of the other materials used such as the wedges, plates, the grouting materials, and the elastomeric pads and there are no records of machine calibration tests. These problems were the results of mismanagement of the project during the construction.

The strands were pulled by a sub-contractor until they elongated for the average distance of 37.5 cm (see Figure 19). A simple calculation shows that the minimum elongation to the tendons must be no less than 59.4 cm based on the tests conducted on the strands after the fingers were pointed at the strand modulus of elasticity.

Ultimate strength: $F_u := 1991 \cdot MPa$

Applied Stress on strands at transfer: $F_{pi} := 0.75 \cdot F_u$ [AASHTO Table 5.9.3-1]

$F_{pi} = 1493.25\,MPa$

22 strands per tendon 20 tendons per side:	$A_p := 153\text{mm}^2 \cdot 22 \cdot 20 = 67320\,\text{mm}^2$	$A_p = 0.0673\,\text{m}^2$
Design Applied Stress on strands at transfer:	$F_{pdi} := 0.75 \cdot 1860\text{MPa} = 1395\,\text{MPa}$	
Strand Elastic Modulus:	$E := 200 \cdot \text{GPa}$	
Strain:	$\varepsilon = \frac{\sigma}{E}$	$\varepsilon := \frac{F_{pi}}{E} = 0.0075$
Strand Total Length (including the portion inside the jack and the strand curvature):	$L := 64.8\text{m} + 0.5\text{m} + 1.0\text{m} = 66.3\,\text{m}$	
Total Elongation Required:	$\Delta := \varepsilon \cdot L$	$\Delta = 49.5012\,\text{cm}$
To account for the strand loose parts inside the tendon ducts, add 20% to the theoritical elongation:	$\Delta_T := 1.2 \cdot \Delta = 59.4\,\text{cm}$	
This actual strands were pulled only 37.5cm:	$\Delta_{actual} := 37.5\text{cm}$	
So the strands were pulled only a percentage of the theoritical elongations:	$\frac{\Delta_{actual}}{\Delta_T} = 63.1297\,\%$	
Total strand elongations missing (we have 440 strands per side):	$P_i := \frac{\Delta_{actual}}{\Delta_T} \cdot F_{pi} \cdot A_p$	$P_i = 63461.54\,\text{kN}$

After this calculation, the determined total strand force of $P_i = 63{,}641.54\ kN$ was inserted to replace the $P_i\ of\ 92{,}400\ kN$, which represents the post-tensioning force of all the 20 tendons of each side (each tendon contained 22 strands of size 0.6-in), into the CSI Bridge model. The results match the expectations and the bridge girder cannot maintain its structural integrity with this low post-tensioning force by itself without shoring. These results are further clarified with graphs in Design Check section.

السايد الايسر
الجهة اليسرى من الرافد

	قبل السحب	بعد السحب	الفرق ما بين قبل السحب وبعد السحب
1	164.5	203.5	39
2	156	194	38
3	158.5	195.5	37
4	155.5	192	36.5
5	154.5	191.5	37
6	145	183	38
7	160	197	37
8	161	198	37
9	163	199.5	36.5
10	165	202	37
11	154	191	37
12	162.5	199.5	37
13	153	190.5	37.5
14	158.5	196	37.5
15	160	196.5	36.5
16	155	192.5	37.5
17	161	198	37
18	150	187.5	37.5
19	151	188	37
20	153	190	37

السايد الايسر
الجهة اليمنى من الرافد

1	163.5	203.5	40
2	158	195	37
3	155.5	192.5	37
4	159	197	38
5	150	187	37
6	151	188	37
7	155	192	37
8	150	187	37
9	166	203	37
10	159	197.5	38.5
11	170	207	37
12	160	197	37
13	150	187	37
14	164	201	37
15	155	193	38
16	157	194	37
17	160	197.5	37.5
18	150	187.5	37.5
19	148	185	37
20	162	199	37

السايد الايمن / الجهة اليسرى

	قبل السحب	بعد السحب	الفرق ما بين قبل السحب وبعد السحب
1	150	187	37
2	152	189	37
3	153	190.5	37.5
4	155	192.5	
5	148	185.5	37.5
6	150	187.5	37.5
7	160	197	37
8	150.5	187.5	37
9	154	190.5	36.5
10	164	201	37
11	157	194	37
12	150.5	187	36.5
13	148.5	185.5	37
14	155	192	37
15	160	197	37
16	153.5	190.5	37
17	170	207	37
18	150.5	187.5	37
19	170.5	207.5	37
20	148	185	37

السايد الايمن / الجهة اليمنى

1	155	193	38
2	160	198	38
3	153	190	37
4	148	185.5	37.5
5	150	187	37
6	155.5	192.5	37
7	170	207	37
8	150	187	37
9	157	194	37
10	148	185.5	37.5
11	150.5	187.5	37
12	160	198	38
13	170	207	37
14	148.5	185.5	37
15	150.5	187.5	37
16	155.5	193.5	38
17	150	187	37
18	160	198	38
19	155	193.5	38.5
20	150.5	187.5	37

Figure 19. Actual strand elongations recorded onsite

4.5 Design Check

The designer submitted three proposals in January 4th, 2012, which were of a cable-stayed bridge, steel arch bridge, and a two-span prestress pretention I girder slab-beam bridge. Among these scenarios, the designer indicated the latter option as his preference. However, Zakho Municipality preferred a single span bridge for aesthetic purposes because the bridge is located at the center of the town. For that, a single span post-tensioned concrete box girder was chosen. The new bridge was designed to have a non-prismatic section that is narrow in the midspan to account for floods (see Figure 8).

As mentioned above, the tendon profile was revised by the designer by lowering the tendon-group centroid at the ends of the girder from 1.0m distance from the top fiber to 1.60m to match the cross section centroid of the girder. Therefore, and as part of the investigation to determine the cause of the failure, the authors of this paper performed an analysis to recheck both of the original design of 2012 and the revised one of 2014 to determine whether revising the tendon profile contributed to the failure or not.

AASHTO LRFD Bridge Design Specification was (elastomeric pad) followed in this design and as such, the limit states of Service I for compression and Service III for tension and Strength I for flexure were checked against the allowable stresses listed in AASHTO LRFD. Moving load type HL- 93K is also used for live load. As such, the two design scenarios were compared for the following aspects:

- Deflection: This includes deflection due to DL, prestress force, Superimposed DL, and the LL.
- Top fiber stress due to Service I load combination against the maximum compressive stress allowed by AASHTO.
- Bottom fiber stress due to Service III load combination against the maximum tensile stress allowed by AASHTO.
- Maximum flexure moment due to Strength I load combination against the allowable flexure moment of AASHTO.

4.5.1 Comparison between the original 2012 design and the revised 2014 design

4.5.1.1 Deflection

1. Deflection induced by the bridge self-weight was 25.6 cm as shown in Figure 20.

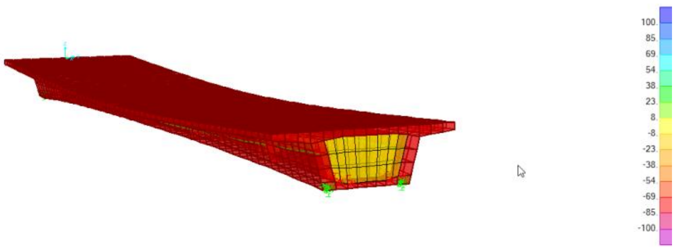

Figure 20. Deflection due to self-weight

2. Deflection induced by the tendon cable forces was 11.60 cm camber for the original 2012 design and 14.90 cm as camber for the revised design of 2014 (see Figure 21).

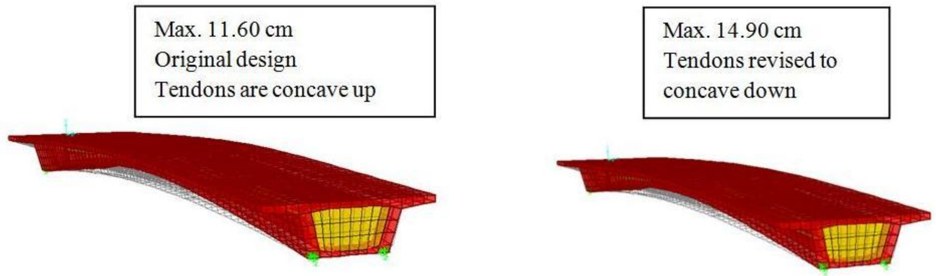

Figure 21. Deflection due to prestress load for the original 2012 design (left) and the revised design (right)

3. Deflection induced by the asphalt and other Superimposed DL was 0.70 cm (see Figure 22).

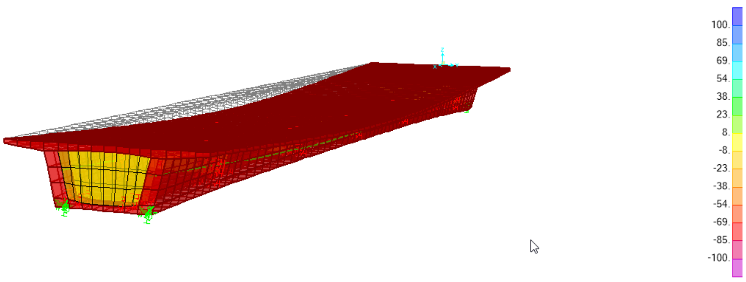

Figure 22. Deflection due to SID

4. Deflection induced by the vehicle live load was 1.39 cm (see Figure 23).

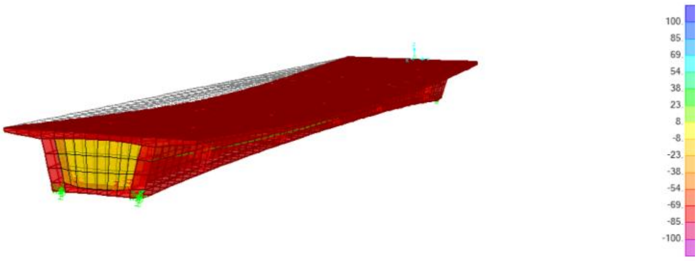

Figure 23. Deflection due to HL-93K LL

The maximum total deflection of the bridge girder at the midspan was 16.09 cm prior to the design revision and after the revision this deflection was reduced to 12.79 cm. This is a first indication that the design was improved by revising the tendon profile. On the other hand, if these numbers are compared to the actual deflection of 47 cm of the Girder 1 and 67 cm of Girder 2, it will be clear that this bridge was not failed because of a design error but rather because of a design implementation error. Also, the 20 cm difference between the deflection of Girder 1 and that of Girder 2 is unexplainable if both of the design and construction are correct and if the same design is implemented for both of the girders, they would have been deflected the same amount.

4.5.1.2 Stresses

1. Compressive stress from all the loads at top fiber at service was 20.83 MPa for the original design of 2012 versus 21.02 MPa for the revised design of 2014. Both of the compressive stresses are within the allowable pressure of AASHTO LRFD 2012 Compression Limit Stress, which is 24 MPa.

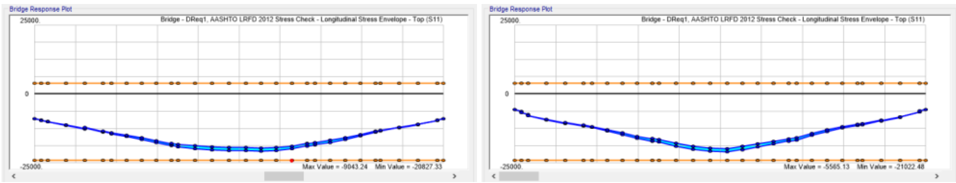

Figure 24. Top fiber compressive stress of Service I load combination of the original design 2012 (left) and the revised design of 2014 (right)

2. Tensile pressure at the bottom fiber is checked at Service III for all the loads and resulted in 6.70 MPa for the original 2012 design and 4.35 MPa for the revised 2014 design. Both of these stresses violate the AASHTO allowable limit of 3.984 MPa. However, the violation is rather small and if both of the graphs of Figure 25 were compared, it will be clear that the violation zone is much wider in the original design. By engineering judgement, both of these designs are acceptable for this limit state since the violation is rather small.

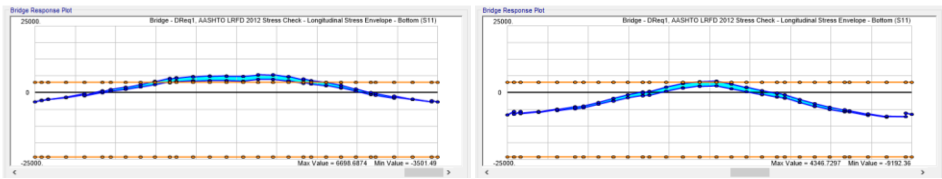

Figure 25. Bottom fiber tensile stress of Service III load combination of the original design 2012 (left) and the revised design of 2014 (right)

3. Applied bending moment according to Strength I load combination is 226,978 kN-m for the original design of 2012 and 232661 kN-m for the revised design of 2014. In both of the scenarios, the flexure moment violates the Resistance Moment at midspan according to AASHTO LRFD, which is 222902 kN-m (see Figure 26). Again, the violation is very slight and therefore, it's acceptable.

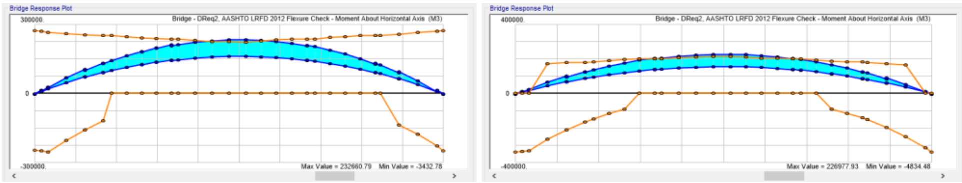

Figure 26. Flexure moment diagram due to Strength I load combination for the original design of 2012 (left) and the revised design of 2014 (right)

It should be mentioned that the acceptance of these small violations of the allowable AASHTO limit is based on a fact that this study is aimed for the determination of the cause of failure of Khani Bridge during construction. Since this bridge is failed during construction it means that it failed even prior to applying vehicular live load. The analysis shown above is conservative because it considered all AASHTO LRFD load combinations including SID and LL load cases.

4.5.2 Comparison between the revised 2014 design and analysis with the test data of Table 3

In this part, the bridge girder is analyzed with inputting the data from Table 3. As analyzed above, the total tendon force of all the 20 tendons of each wall is 63,462 kN instead of 92,400 kN of the design. The ultimate stress $F_{u=}1991\ MPa$ and the yield stress $F_y = 1729MPa$ are also used in accordance with the test data in *Table* ***3***. The results show that the deflection camber due to prestress is reduced to 9.56 cm.

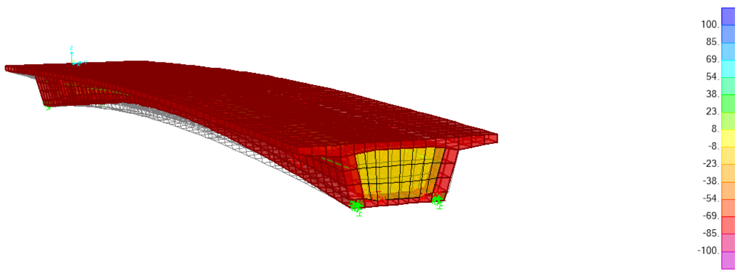

Figure 27. Deflection due to prestress load of tendon force using the data from test after the failure

This decrease in the camber due to prestress increases the total deflection to 18.82 cm. This large deflection causes the stresses to cross the limits specified by AASHTO by large amount.

The compressive stress from all the loads at top fiber at service was 21.08 MPa. This limit stresses is within the allowable pressure of AASHTO LRFD 2012 Compression Limit Stress, which is 24 MPa. However, the tensile stress at the bottom fiber largely violates the AASHTO LRFD limit with 14,163 MPa against the allowable, which is only 3,940 MPa (see Figure 28).

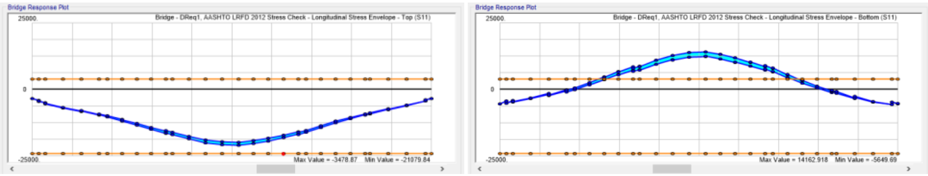

Figure 28. Top fiber compressive stress due to Service I (left) and bottom fiber tensile stress due to Service III (right) with the retest data input from Table 3

Tensile stress about four times the allowable. This means that the section is cracked and the cracked moment of inertia is approximately 40% of the gross moment of inertia (i.e. the dead load deflection increased 2.5 times). A hand calculation was performed to determine the total deflection based on the cracked moment of inertia and showed that the otal deflection is approximately 47 cm. This is in line with the actual deflection of Girder 1.

It can also be noticed that even the minimum stress violates the tensile stress limit largely with 12,386 MPa. This is a strong indication that the bridge failed for lack of compressive stress on the concrete that was induced by the tendon post-tensioning.

4.6 Checking the analysis

The two analyses of the original 2012 and 2014 design were checked using Midas software. Staged construction loading was used in the Midas model. The results of these two methods were consistent with that of the CSI Bridge.

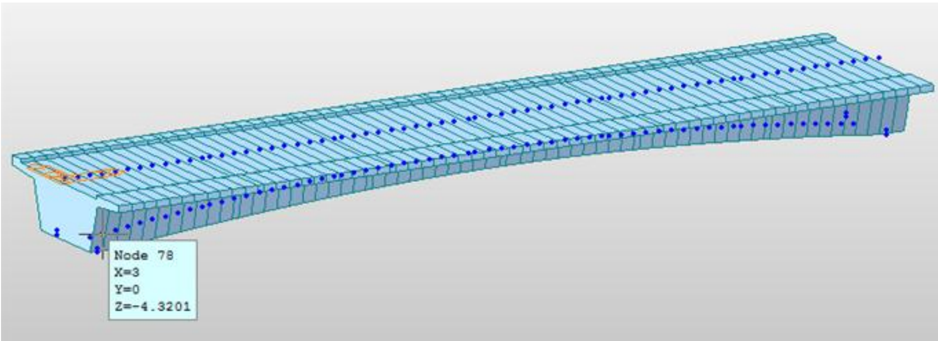

Figure 29. Khani Bridge modeled in Midas

Deflection due to the tendon forces is crucial to be compared between the two designs since deflection due to the other loads are the same. Figure 30 shows the deflection due to the post-tensioning tendon forces. The camber deflection caused by the tendon profile of 2012 is 10 cm while that of 2014 is 13.15 cm.

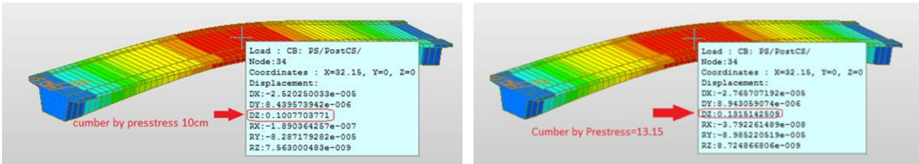

Figure 30. Deflection due to tendon forces of the original 2012 design (left) and revised 2014 design (right)

Figure 31 shows that the maximum top fiber compressive stress due to Service I load combination is 22.99 MPa for the original 2012 design and 21.69 MPa for the revised 2014 design. Both of the values are within the allowable as indicated by the dashed line.

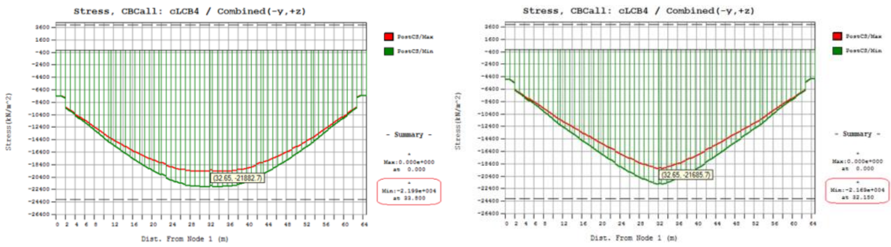

Figure 31. Top fiber compressive stress due to Service I load combination of original 2012 design (left) and revised 2014 (right)

Figure 32 shows that the maximum bottom fiber tensile stress due to Service III load combination is 5.21 MPa for the original 2012 design and 2.67 MPa for the revised 2014 design. It can be noticed that this tensile stress crosses the limit specified by AASHTO LRFD and shown as a dashed line in the original 2012 design; while it stays within the limit for the revised 2014 design. This is another indication that the design was improved by the revision of 2014.

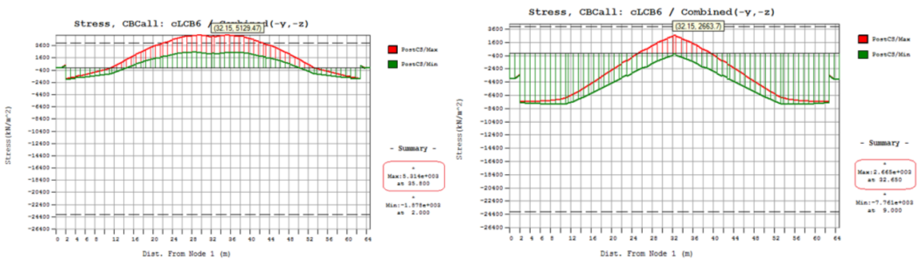

Figure 32. Bottom fiber tensile stress due to Service III load combination of original 2012 design (left) and revised 2014 (right)

5 CONCLUSIONS

This research study on Khani Bridge failure causes has led to the following conclusions:

1. Tendon elongation was not sufficient. Only 63% of the required stress was applied to each of the 880 cable strands. This lack of strand cable pulling amount lead to lack of compressive stress in concrete and by turn, the 65 m span girder could not support its own weight when the shores were removed.
2. The cable strands were checked twice; once during the construction and the second time after the bridge failure. These two test results do not match. While tendon the first test result shows that they can be elongated to approximately 3.7%, the second test shows 9.4%, which is more than 250% increase. This large difference between the two tests puts the testing credibility on the line.
3. The timing for tendon pulling was not appropriate. The tendons were pulled only three days after casting the top slab of the girders. The top slab did not reach its required compressive strength when the strands were pulled and applied post-tensioning stress on concrete. This means that the whole box section did not behave the same during the post-tensioning and acted more like a composite section instead of one piece box.
4. Most of the materials used in the bridge were not tested and they do not have manufacturing certificates. Among these materials are the wedges, plates, elastomeric pads, etc. also there is no record of the machine calibrations for the strand pulling machine.
5. Some of the concrete compressive strength tests failed especially for the walls and the top slab.
6. The project was managed poorly. In fact, the project did not have any consulting engineers neither from the contractor side, nor from the owner. Therefore, the materials were used without tests or with failed results.
7. The design cross section may not be the optimal choice but did not contribute to the failure.
8. The design was improved when the tendon ends were moved downwards such that the centroid of tendon group matched the neutral axis of the box section. In other words, the revised 2014 design gives better structural integrity to the bridge than the original 2012 design. The total deflection of the revised design of 2014 is 12.79 cm while the original design with the tendon ends above the neutral axis would have led to 16.09 cm deflection.
9. According to the hand calculation and the CSI Bridge model analysis, all the stress and flexure limit states are within the allowable AASHTO LRFD limits or just slightly passed them with all the applicable load combinations included.

The required theoretical tendon elongations were not submitted and approved prior to the strand pulling process. Also, the submitted actual pulling data for

880 strands was not detailed and all included on one sheet of paper.

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