Bridge Site – Hydraulic Study
At
Sulaymaniya Governorate

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Introduction

Why study bridge hydraulics?

Many people, indeed many engineers, who are not familiar with the subject, imagine that constructing a bridge across a river is entirely a problem in structural engineering. They assume that the bridge opening can be made so large that it will completely span the river at such a height that floodwater will never rise anywhere near the deck. If this was always true there would be little need to study bridge hydraulics, but in reality things are rarely this simple.

Economics often dictate the length of span and therefore how many piers have to be located in the river. Similarly economics, the geography of the site or the nature of the crossing may impose some restriction on the maximum permissible elevation of the deck. Consequently flood levels may rise to deck height or above. What initially appeared to be an elementary problem turns out to be quite complicated.

So why study bridge hydraulics? Four answers quickly spring to mind.

• Nobody can be allowed to build a new bridge that has piers and/or abutments in a river without first being able to prove by calculation or modeling that the resulting backwater will not cause, or significantly exacerbate, flooding of land and property upstream. This is becoming increasingly important as the demand for building land leads to construction on river floodplains that, by definition, are already prone to flooding.

• At locations where there is an existing bridge and significant flooding, an analysis may be required to determine how much of the flooding is caused by the bridge and how much by other factors—such as simply too much floodwater to be carried within the river channel. If the analysis shows the bridge to be at fault, then this may be sufficient justification to construct a new structure.

• If it is known that a bridge provides a significant obstacle to flow and is responsible for much of the flooding that occurs, with knowledge of bridge hydraulics it may be possible to design improvement works that will help to alleviate the problem.

• In addition to the nature and geometry of the river channel, the shape, spacing and orientation of the bridge piers and abutments will affect the flow through a bridge and the likelihood of scouring of the bed. Well designed bridges are not immune to this problem while bridges that are badly designed hydraulically are even more likely to fail and collapse. This study is to collection data available for Zharawa river to built a bridge on this river near the old bridge. The data were taken from UNOPS, Dokan dam administration, UNDP, field study and data measuring. The importance of the hydrulic study is to get a required information , charactristics, floods, bed material, velocity, discharge, water sources, river path, and etc. in order to safe and economical structure may be constructed or have a contact with Zharawa river from an probable events in order to have information for design, construction as well as for maintain.

Data Available

Information in this report is based upon the following bulletins and publication:

- Travel to the site and data collection.
- UNDP Investigation Progress Report 2C Sulaimany Governorate May 2002
- UNOPS map 5062III
- Dokan dam directorate irrigation
- Bridge manual 1999, hydraulics part
- Conversation with skill viligian pepole.

Site Description

Zharawa Bridge located on the Zharawa River (Chawmi Gafrain River) along the Zharawa-Qaladiza Road (Ranya-Qaladiza Road) in Sulaymaniyah Province, Iraq. The grid coordinates for the existing bridge are N 36° 12.916' E 045° 04.613'. Its about 7 Km from Qaladiza city. The area of drainage basin (Catchment area) upstream of the bridge is shown on UNOPS map 5062III.

Dutchwoman river originated in Qandeel mountains flowing eastern to Lesser Zab .The river in the lower reaches called Zharawa river.The catchments area of the river has a typical mediterranean climate modified by the influence of the high mountain ranges of Qandeel mountain.

The size of the Zharawa river Drainage basin at the proposed site is approximately 68.9 Km^2.

Both upstream and down stream reaches of the river are well defined with all normal flows being confined within a definite floodway. The adjacent upstream and down stream river banks are some concrete block factories.

The private residences are on the right side of the river is Zharawa city. There are sandstone outcroppings in the surrounding vicinity and the stream bed material consists of sand, gravel and boulders. The normal flow channel width is generally uniform in the vicinity of the project and measures approximately 625 meters.

Based upon field measurements, the water surface slope of the channel is approximately 0.073%.

The bridge located over the Zharawa river .The 10.25 m concrete girder structure built in 1961 has 12 spans in addition 10 x 3m box culverts with an overall length of 175 m. Also there are two box culverts at Zharawa side approach.

Zharawa damaged bridge

Magnitude and Frequency of Flood

In the vicinity of the project site, the Zharawa river is regulated. A flood frequency discharge analysis was made to estimate the design discharge for the 100 year flood (Design Event). The analysis method consisted of examination of flood estimates basing on the Manning Formula. A 100-year Design Discharge of 1206 m^3/s was elected because of using probably theory and consistency. All of the flow passes through the bridge opening in low flow with no road overflow. The design flow utilizes an area of 462 m^2 and has an average velocity of 2.6 m/s.

Hydrologic calculation were previously submitted on November14 ,2007 .The Design Discharge of 1206 m3/s .

There are no staff gauges in the site to observe the water level in the river.

The existing 100-year design high-water is 2.7 m with an associated backwater of 0.5 m.

Analysis of Hydraulic Characteristics

The hydraulic Characteristics of the floodplain for both existing and proposed conditions were calculated in metric units using the manning formula and probability theory.

Manning roughness coefficients were determined from field observations and estimated 0.04

There are no reports of road overflow or ice/debris problems at the site.

The mean annual precipitation over the drainage area approximates 1200 millimeters. Precipitation on the high peaks during the colder months is generally in form of snow.

The peak discharge at Zharawa bridge during 2006 flood was approximately (623 m3/s). The maximum high-water level is 2.7 m with an associated backwater of 0.5 m, during previous century.

Data are not available for the recent flood but calculated by using Manning formula depending on eye judgment of the wetted perimeter.

For purposes of bridge design which must be adequate to protect the bridge throughout its lifetime perhaps several hundred years a more conservative approach must be taking. The flood for which the bridge is designed should the maximum flood which could be reasonably expected ever to occur at the site.

Existing Structure and Approaches

The existing 12-span concrete girder bridge built in 1961 has clear spans of 9.25 m, in addition of 10 x 3m box culverts with overall length of the bridge equal to 175m. The abutments are vertical face with wing walls. The piers are tapered solid shafts.

The roadway width between curbs is 8 m. There are two sidewalks on both sides of the existing structure that is 0.75 m clear (total width = 9.5 m). Both the superstructure and substructure are in poor condition. In order to the road between Ranya and Qaladiza didn't cutoff, Sulaimanyah directory of road and bridge prepared a temporary steel bridge with 33m length and 4.5 m width placed on the exist damaged piers and its clear that don't stay long, also its not safe for use.

Failure reasons of the existing bridge

Late last year, the Zharawa Bridge was greatly damaged by flood. The bridge was subjected to foundation settlement, movement and displacement which resulted in cracks and concrete failure to several beams. In order to keep the road open and to reach other disaster areas, a temporary iron bridge was installed and riprap backfill was placed allowing traffic to cross the Zharawa River.

There are no sufficient maintains for the bridge during its life, however approximately at 1996 the strengthen of its foundation was done, but the maintains was not continuously, therefore at 2006 the bridge was failed by flood.

Examination of the counter mapping of the existing bridge indicates that there are existing local scour around the piers.

Therefore we know that the reason of this failure is local scouring of the foundation due to the flood.

Scour can be defined simply as the excavation and removal of material from the bed and banks of streams as a result of the erosive action of flowing water.

It is sometimes assumed that scour will be a problem only when the bed material consists of fine cohesionless material. This is not true: ultimately the scour depth in cohesive or cemented soils can be just as large, it merely takes longer for the scour hole to develop. For example, under constant flow conditions, scour will reach maximum depth in sand and gravel in a matter of hours (perhaps during one flood); in cohesive materials it will take days; in glacial tills, sandstones and shales it will take months; in limestone years; and in dense granite centuries. However, the biggest and most frequently encountered scour-related problems usually concern loose sediments that are easily eroded.

Scour is a very serious problem. Floods that result in scour are the principal cause of bridge failure. In 1973 in the USA a national study of 383 bridge failures caused by catastrophic floods showed that around 25% involved pier damage and 72% abutment damage (Chang, 1973). In 1985, some 73 bridges were destroyed by floods in Pennsylvania, Virginia and West Virginia, while during the spring floods of 1987 17 bridges in New York and New England were damaged or destroyed by scour. With about 485000 bridges spanning rivers in the US National Bridge Inventory, it is likely that hundreds of these structures will encounter a 1 in 100 year flood during any twelve month period, and that some will be damaged or destroyed. On a worldwide scale the problem is even larger. Many countries have programs that are designed to identify the bridges that are at risk from scour (i.e. scour critical) with the dual aim of ensuring the safety of users and preserving the affected structures. It is not just old structures, such as nineteenth century rail bridges, that are at risk; new structures can also be susceptible if not properly designed with scour in mind. Of course, it is easy to say that the foundations of all new structures should be made so deep as to eliminate any potential problems relating to scour, just as it can be said that the bridge opening should be made large enough to pass any flood that occurs, but in reality things are not this simple, and economic factors must also be considered. If unnecessary expense is incurred by making all of the foundations significantly deeper than the probable scour depth, the cumulative cost will be very substantial because a large number of bridges are involved. The cost of this enclosure varies with the plan area and the depth of excavation, so deliberately designing very deep foundations will complicate construction and add significantly to the cost. On the other hand, it should be remembered that the total cost of a failure may be of the order of two to ten times that of the original structure, allowing for disruption to transport and commerce. Thus it is necessary to strike a balance, setting the foundations deep enough

to resist the scour that can reasonably be expected at the site without going so deep as to incur additional unnecessary expense.
Unfortunately, when deciding just how deep is deep enough the equations that are available to predict the depth of scour are very numerous and contradictory. In 1987, Copp and Johnson reported that 35 different formulae for scour estimation at piers had been proposed since 1949, almost one per year! The famous of these equations were of the form $dsp/bp = K (Y/ bp) n$ ……….(1)

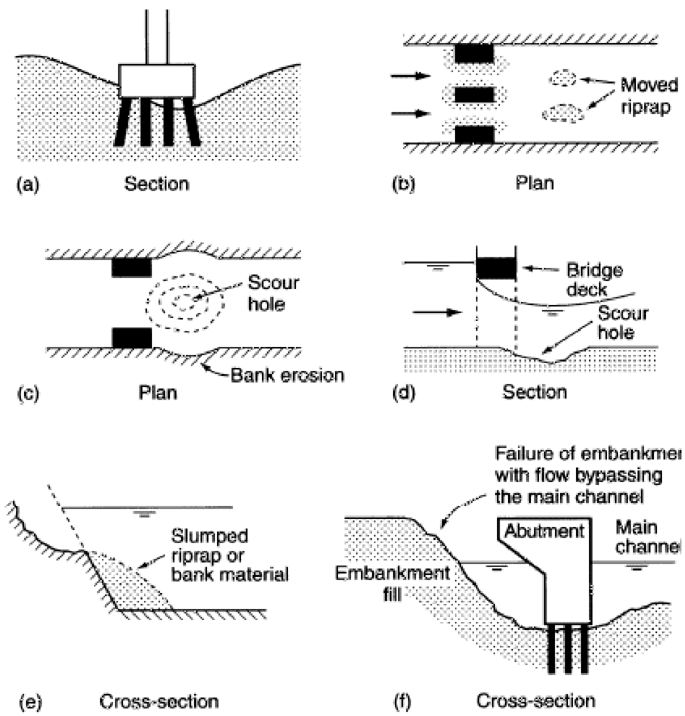

Fig. 1 Some observable effects of scour: (a) pier piles and pile cap exposed; (b) pier and abutment riprap moved downstream; (c) downstream scour hole and bank erosion; (d) downstream scour hole arising from submergence of the opening (pressure flow); (e) slumped material at the toe arising from failure of the riprap or bank; (f) erosion (mass wasting) and failure of the highway embankment with flow on both sides of the abutment.

Where dsp (m) is the predicted pier scour depth, bp (m) is the width of the pier, K is a dimensionless factor that allows for pier geometry and orientation to the flow, Y (m) is the depth of the approach flow, and n is a factor reflecting the erosive characteristic of the streambed.
Many of the equations for scour were derived from laboratory studies, for which the range of validity is unknown; some were

verified using very limited field data, which itself may be of doubtful accuracy. In the field, the scour hole that develops on the rising stage of a flood, or at the peak, may be filled in again on the falling stage so that the maximum depth cannot be assessed easily after the event.

Measurement or observation during flood using divers is not safe or practical, but it is sometimes possible to detect the maximum scour depth afterwards. For instance, if a cohesive material is scoured and then subsequently the pit is filled with an incohesive material, by probing it should be possible to detect the change in the strata. Similarly, with cohesionless material it may be possible to detect changes between the fill and the underlying bed material.

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Local scour

Local scour is the removal of material from around a pier abutment, spur dike, or the embankment. It is caused by an acceleration of the flow and/or resulting vortices induced by obstructions to flow.

Local scour arises from the increased velocities and associated vortices as water accelerates around the corners of abutments, piers and spur dykes.

The flow pattern around a cylindrical pier is shown in Fig. 2 the approaching flow decelerates as it nears the cylinder, coming to rest at the centre of the pier. The resulting stagnation pressure is highest near the water surface where the approach velocity is greatest, and smaller lowers down. The downward pressure gradient at the pier face directs the flow downwards. Local pier scour begins when the down flow velocity near the stagnation point is strong enough to overcome the resistance to motion of the bed particles.

Without a scour hole the maximum downward velocity is about 40% of the mean approach velocity (V), the maximum strength of the down-flow being recorded just below bed level. When scour occurs the maximum down-flow velocity is about 80% of V (Copp and

Johnson; Melville). The impact of the down-flow on the bed is the principal factor leading to the creation of a scour hole. As the hole grows the flow dives down and around the pier producing a horseshoe vortex, which carries the scoured bed material downstream.

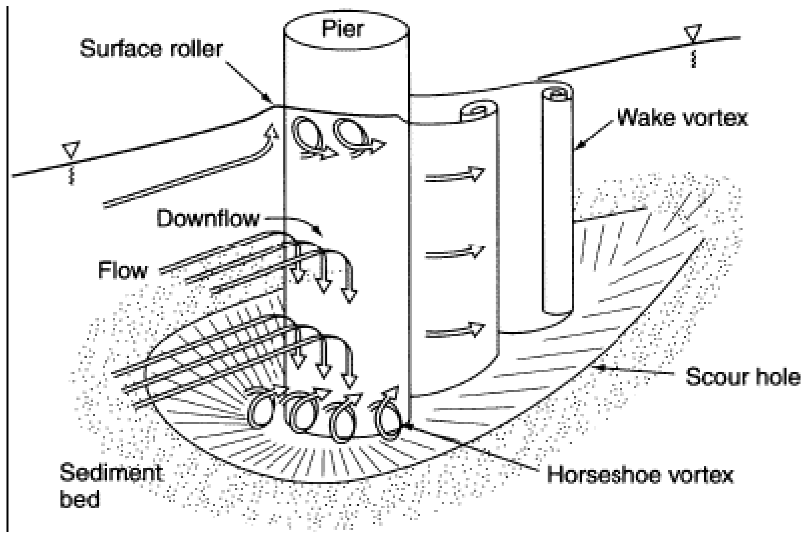

Fig. 2 The flow pattern and scour hole at a cylindrical pier. The down-flow, horseshoe vortex and wake vortex are the principal cause of local bed erosion

The combination of the down-flow with the horseshoe vortex is the dominant scour mechanism. As the scour hole becomes progressively deeper the down-flow near the bottom of the scour hole decreases until at some point in time equilibrium is reached and the depth remains constant.

At the sides of the pier flow separation occurs, resulting in a wake vortex whose whirlpool action sucks up sediment from the bed. As the vortices diminish and velocities reduce, the scoured material is deposited some distance downstream of the pier.

For piers that are essentially rectangular in plan and aligned to the flow the basic scour mechanism is similar to that just described, albeit rather more severe because of the square corners. However, as the angle of attack to a rectangular pier increases so does its effective width, so the scour depth increases and the point of maximum scour moves downstream of the nose to a point on the exposed side. With a large degree of skew the maximum scour may occur at the downstream end of the pier. If the flow direction is likely to change there is merit in using cylindrical piers to avoid these complications.

The scour mechanism at a bridge abutment is similar to that at a pier, although the boundary layer at the abutment or channel wall may result in an additional deceleration of the flow compared with a central pier. The approach flow can be considered as separating into an upper layer, which forms an up-flow surface roller on hitting the abutment, and a lower layer, which becomes the bottom or principal vortex (Fig. 3). Viewed in plan, the upper layer divides or separates, with part of the flow accelerating around the upstream corner of the abutment into the bridge waterway while the remainder slowly rotates in an almost stationary pool trapped against the face of the abutment and the river bank. In the bottom layer, the flow near the bank forms an almost vertical down-flow, while that nearer to the end of the abutment accelerates down and into the waterway, forming the principal vortex. Usually scouring starts in this region of accelerating flow and grows along the faces of the abutment. Downstream of the abutment wake vortices form.

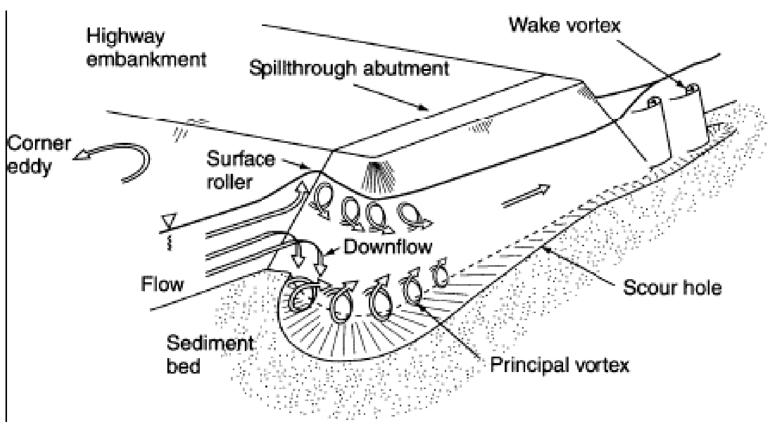

Fig. 3 The flow pattern at a spill through abutment. The down-flow and Principal vortex is the main causes of local bed erosion.

The recommended equation for determination of pier scour in the bridge manual was used for determining the pier scoured depth

$Ys/a = 2\ K1.\ K2.\ K3.\ K4\ \{y1/a\}^{0.35}\ Fr_1^{0.43}$

Where:

ys = Scour depth, ft, m
y1 = Flow depth directly upstream of the pier, ft, m
a = Pier width, ft, m
Fr1 = Froude number directly upstream of the pier = V1/ (gy1)
V1 = Mea Velocity of flow directly upstream of the pier, ft/s, m/s
g = Acceleration of gravity, 32.2 ft/s2, 9.81 m/s2
K1 = Correction Factor for pier nose shape
K2 = Correction Factor for angle of attack of flow
K3 = Correction Factor for bed condition
K4 = Correction Factor for armoring by bed material 0.7 - 1.0

The values of K1, K2, K3, and K4 are listed in tables 2, 3, and 4 page 39 Bridge Manual 1999.
The above study indicates the critical situation of the existing damaged Zharawa bridge and probability of the majar failure is very ocour.

Failure of Zharawa bridge (local scour)

Hydrodynamic forces on the bridge

1) Hydrodynamic forces on piers

These are the drag force (F_D kN) in the direction of flow and the lift force (F_L kN) perpendicular to it. According to Apelt and Isaacs :

Drag force, $F_D = C_D \rho V^2 YL / 2000$ ……………… (3)

Lift force, $F_L = = C_L \rho V^2 YL / 2000$ ……………… (4)

where C_D and C_L are the dimensionless coefficients of drag and lift respectively, ρ is the density of the water (kg/m^3), V is the approach flow velocity (m/s), Y is the depth upstream of the pier (m), and L is either the length of the pier in the direction of flow or the diameter of a single cylindrical pier (m).

The hydrodynamic forces on piers are usually small (e.g. compared with ship impact), which is often convenient because C_D and C_L depend upon factors such as the shape and spacing of the piers, the angle of attack, and the Reynolds number of the flow (Apelt and Isaacs; Farraday and Charlton). Note that (unlike coefficients of discharge) C_D and C_L can have values above 1.0. Very approximately, C_D values around 0.2–0.5 may be typical for some pier shapes pointing into the flow, but rise with the angle of attack to somewhere around 1.0–2.0. Since equations 3 and 4 are the same apart from the coefficients, a blunt pier, which would be expected to experience a larger drag than lift force, would have a higher C_D value than C_L, and vice versa for aerofoil shapes.

2) **Hydrodynamic force on submerged superstructures**

Hydraulic Engineering Circular 20 of the US Federal Highways Authority (FHWA, 1991) gave the drag force per meter length (F_D kN/m) of a submerged or partially submerged bridge deck as

$F_D = C_d \rho V^2 H / 2000$ ……………… (5)

Where C_d is the dimensionless coefficient of drag, which has a suggested value of between 2.0 and 2.2, H is the depth of submergence (m), and the other variables are as above.

In order to eliminate this effect on the proposed super structure, minimum 0.8 m freeboard necessary over 2.7m measured high water level recommended.

3) Ice forces

Neill suggested that piers with semicircular noses in plan and slightly inclined inwards to the vertical are effective in discouraging ice accumulations. The worst-case scenario may be large sheets of hard ice hitting the piers. Unfortunately the forces generated depend upon the type and strength of the ice, and how the ice fails (e.g. crushing, splitting, shearing, and bending). Farraday and Charlton, the American Association of State and Transport Officials (AASHTO,) and the UK's Highways Agency presented an equation for the horizontal force (F_H kN) on a pier having 'substantial mass and dimensions':

$$F_H = C_n S_i t_i b_p (C_p) \quad \ldots\ldots\ldots\ldots (6)$$

where C_n is a coefficient for the inward inclination of the nose (0–15°=1.0; 15–30°=0.75; 30– 45°=0.5), s, is the strength of the ice (between 700 and 2800 kN/m^2: the low value represents disintegrating melting ice and the high value major ice flows with freezing temperatures), t_i is the thickness of the ice in contact (m), and b_p is the width of the pier (m). The value of C_n may be modified according to pier width or pile diameter and ice thickness by multiplying by another coefficient (C_P), which ranges in value from 1.8 to 0.8 for b_P/t_i=0.5 and ≥4.0 respectively (Canadian Standards Association).

According the hydraulic study there is no reports records the ice/debris problems or the rolling of big rocks is not noticed, because the slope gradient is very low, causing decrease in water velocity and the site of the bridge is far from the mountains; therefore any protection of the structure in this case will be eliminated.

4) Debris forces

Australian specifications recommend allowing for a 2 ton log traveling at the normal stream velocity being arrested within 150mm and 75mm for column type and solid type concrete piers respectively. However, according to the UK's Highways Agency (1994), 3 ton logs traveling at almost 4.5 m/s have been observed. The average collision force (*F* kN) on the pier is

$$F = M\,V^2 / (2d) \quad \ldots\ldots\ldots\ldots\ldots\ (7)$$

Where *M* is the mass of the moving body (ton), *V* is its velocity (m/s), and *d* is the distance before it comes to rest (m). According to Farraday and Charlton some UK engineers assume a 10 ton mass being arrested in 75mm.

Forces can also be generated on the pier as a result of the flow impacting on debris trapped against the pier or across the waterway opening. Australian guidelines suggest calculating the hydrodynamic force on a minimum depth of 1.2m of debris over a width of half the sum of the spans adjacent to the pier up to a maximum of 21 m. The following equation gives the force (*F* kN) due to trapped debris of area *A* (m^2) being hit by a flow with an approach velocity, *V* m/s:

$$F = 0.517\,V^2 A \quad \ldots\ldots\ldots\ldots\ldots\ (8)$$

According the hydraulic study there is no reports records the ice/debris problems or the rolling of big rocks is not noticed, because the slope gradient is very low, causing decrease in water velocity and the site of the bridge is far from the mountains; therefore any protection of the structure in this case will be eliminated.

Discussion of proposed Structure Sizing

The potential for damage to many adjacent private properties due to flooding on the Zharawa river at this site is quite high. The proposed bridge structure must be design to resist and withstand the potential of scour.

The purpose tangent horizontal alignment across the bridge will be maintained while the proposed vertical alignment will be raised to facilitate the use of deeper super structure. The minimum length of the structure must be not less than 175m.

The height of structure not less than 3.5 m above the natural bed of the river, this height should be safe because maximum water level estimated 2.7 and 0.8 m freeboard.

The structure must not be skewed and the foundation must be deep to avoid scouring. Also depending on geotechnical study the most suitable foundation used for this bridge is board piles, with deeps range from (10 – 12 m), under the ground surface.

Also according the hydraulic study there is no reports records the ice/debris problems or the rolling of big rocks is not noticed, because the slope gradient is very low, causing decrease in water velocity and the site of the bridge is far from the mountains; therefore any protection of the structure in this case will be eliminated.

All satisfy there is no chance to use the Zharawa river for navigation and must pitching the bottom of the bridge with stones or rocks to minimize the erosion.

Summary of Hydraulic Characteristics

The hydraulic data for the existing and proposed bridge is summarized as follows:

Drainage Area	68.9 Km2
Design 100-year flow	1206 m3/s
Water Area	462 m2
Velocity through the bridge	2.7 m/s
Maximum design high water	2.7 m
Drag force on the peir	32 kN
Lift force	130 kN
Hydrodynamic force on submerged superstructure	(not allowed)
Ice forces	zero
Debris forces	zero
Over flow frequency	>100-year

Appendix (A)

UNOPS Map 5062 III

Appendix (B)

DOKAN DAM INFLOW DATA

DOKAN DAM

Inflow Records(m^3 / Sec)

Month	10	11	12	1	2	3	4	5	6	7	8	9	
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Avg
1958	53	75	92	184	220	261	203	133	85	48	34	29	118
1959	31	37	119	72	93	331	450	236	104	69	51	52	137
1960	39	80	67	106	118	134	193	216	63	39	57	48	97
1961	61	107	65	167	161	151	308	169	60	72	48	48	118
1962	21	64	158	198	296	264	271	201	114	85	67	63	150
1963	46	44	134	280	338	385	633	536	226	132	165	82	250
1964	65	123	174	184	407	759	567	366	200	122	76	99	262
1965	57	76	84	167	282	300	478	176	216	104	85	49	173
1966	72	110	92	110	137	370	283	184	98	53	78	91	140
1967	95	84	94	150	247	387	362	364	152	78	90	86	182
1968	90	114	222	135	188	369	577	380	75	78	90	80	200
1969	73	115	476	400	471	1380	1347	656	270	209	187	155	478
1970	112	165	148	244	195	230	266	158	162	73	88	69	164
1971	56	57	69	66	126	196	665	211	98	71	68	70	146
1972	67	84	141	136	214	531	760	708	220	101	74	132	264
1973	104	173	114	107	443	287	355	232	109	89	84	62	180
1974	74	80	117	116	154	1482	926	280	229	241	86	87	323
1975	70	92	110	146	390	319	294	242	113	70	71	57	165
1976	56	78	164	188	354	352	717	409	195	81	70	81	229
1977	88	105	119	131	281	335	423	233	100	66	87	116	173
1978	99	105	151	238	328	392	257	171	113	211	71	83	185
1979	87	42	301	236	289	211	248	170	74	41	51	52	150
1980	53	49	109	125	200	409	527	212	99	61	54	61	163
1981	60	108	57	218	280	515	398	294	146	89	77	67	192
1982	79	90	90	217	253	502	611	373	135	89	101	77	218
1983	117	188	133	177	238	412	286	320	133	81	66	70	185
1984	60	65	99	67	100	166	256	147	71	41	51	34	96
1985	52	156	168	313	675	588	630	311	169	115	67	82	277
1986	82	91	148	142	342	178	287	275	112	67	46	44	151
1987	55	153	167	144	268	517	437	270	132	65	57	52	193
1988	73	112	618	400	554	1510	814	515	256	162	84	91	432
1989	93	91	178	104	107	337	216	112	58	39	42	48	119
1990	53	105	231	218	241	351	355	210	109	49	31	40	166
1991	58	66	67	57	170	526	473	173	77	48	41	63	152
1992	68	89	382	278	650	583	1058	797	362	158	81	70	381
1993	74	172	323	289	337	427	686	579	247	116	73	62	282
1994	72	344	273	505	443	506	456	275	113	70	61	54	264
1995	65	328	314	389	460	370	742	402	192	84	57	50	288
1996	53	79	80	131	238	356	487	217	96	53	44	28	155
1997	37	46	81	117	147	434	520	294	143	66	42	35	164
1998	55	131	168	229	438	750	673	286	114	46	34	32	246
1999	41	61	65	85	194	100	108	68	31	17	11	13	66
2000	16	23	39	108	125	146	162	88	31	16	12	13	65
2001	14	22	57	45	109	201	132	66	27	17	12	12	59.5
2002	14	25	125	358	273	299	491	234	112	54	27	29	170
2003	29	54	271	277	375	655	573	294	152	59	35	33	234
2004	41	92	159	436	431	414	319	339	126	61	39	42	208
2005	42	133	95	151	420	586	281	178	89	53	39	34	175
2006	33	37	51	119	734	244	261	265	82	43	35	30	161
2007	55	197	52	80	283	232	410	247	92	41	34	28	183
Avg	61.2	102.34	156.2	190.8	296.34	370.81	433.938	285.44	131.64	79.86	62.62	59.7	
Max	117	344	618	505	675	1510	1347	797	362	241	187	155	
Min	14	22	39	45	93	100	108	68	31	16	11	13	

Appendix (C)

HYDROLOGY DATA

Hydraulic data of the Zharawa River

Serial	**River name**	**Location**	**Date**	**Discharge m^3/sec**	**Catchments Area km^2**	**Q min m^3/s**	**AV. Probable discharge m^3/sec**	**Av. Discharge for 100 year m^3/sec**	**Runoff**
1	Greater Zab.	Bekhma Damsite	10-20-2001	50.36	16780.4	50.36	1050	1874	95
2	Sirwan	Near distok & Kawte village	11-20-2001	12.063	13856.1	12.063	891	1743	28
3	Zimkan	Upstream of confluence	11-20-2001	1.829	2524	1.829	926	1442	22
4	Lesser Zab.	Derbandi-Rania	10-20-2001	11.62	9266.7	11.62	886	1514	41
5	Lesser Zab.	Delga	10-20-2001	7.477	7870.4	7.477	905	1562	32
6	Duchoman	Zharawa (road to Qaladiza)	10-20-2001	0	68.9	0	651	1206	0
7	Qaladiza	Qaladiza Town	10-20-2001	0	36.8	0	592	1035	0