D. Assessment 2002 - 2004

Content

D.1 General

In order to determine if the axle load 330 kN (BV-2000) could be used for the bridge, a preliminary assessment was made during 2002. This assessment was reviewed by the consulting company Reinertsen, see Kolster (2002). The programs Strip Step (1971) [see further Bengtsson & Wolf (1969, 1970) and Lundin (1971)] and Solvia (2000) were used and it was recommended that a more complete finite element model (FEM) should be applied. Banverket (The Swedish Rail Administration) then asked the consulting firm Tyréns to carry out such an improved analysis and an early version of the FEM program Brigade (2012) was used, Leander-Fredriksson (2003, 2004)

D.2 Assumptions

The calculations by Leander-Fredriksson (2003) were performed in accordance with BV Bärighet (2000) considering updates until 2002-12-19. Only the main arch, the top slab carried by the top beams were studied whereas the side spans were left out.

The finite element program Brigade Plus was used with 3D beam elements. The top slab was modelled as one cross section. The forces in the top slab and in in cross beams were calculated with simplified methods. Originally three models were tested with different beam elements, B31 (size 0,1 m), B32 (size 0,2 m) and B33 (size 0,3 m). Eigenfrequencies were calculated, see Table D2.1

Table D2.1 Eigenfrequencies (Hz) calculated with Brigade Plus for three beam element types, Leander-Fredriksson (2993), p 3:9.

mode	Modell 1	Modell 2	Modell 3
1	2.2124	2.2124	2.2124
2	3.2730	3.2729	3.2729
3	4.9308	4.9308	4.9308
4	6.1234	6.1234	6.1234
5	7.0304	7.0302	7.0302

In the following calculations the model with element B32 with a size of ca 0,2 m was used,

Section forces were studied for the arch, the top slab, the top beams with cross beams where columns are present.

All loads except the train loads are applied directly to the model. The effects of traffic loads are determined from influence lines in chosen sections. The influence lines are determined by moving a point load in Brigade Plus along the railway line.

Second order effects are considered by a factor which is multiplied with the first order bending moments. The factor is determined as M_2/M_1, where M_1, is the first order bending moment and M_2 is second order moment calculated considering initial deflections and nonlinear function.

The capacities have been calculated for reinforcement characteristic yield stresses f_{yk} = 390 MPa (Ks40) and 720 MPa (Ss 70A) and a concrete compression strength f_{cck} = 28,5 MPa (K400). The characteristic modulus of elasticities has been assumed to E_{sk} = 200 GPa and E_{ck} = 32 GPa. The following **design values** have been used, Leander- Fredriksson (2003), p. 4:1:

$f_{st} = f_{yk}/1,15\gamma_n$ with $\gamma_n = 1,2$ gives f_{st} =282,609 MPa for Ks 40 and f_{st} = 521,739 MPa for Ss 70

$f_{cc} = f_{cck,j}/1,5\gamma_n$ with $\gamma_n = 1,2$ and $f_{cck,j} = 1,15\, f_{cck} - 2 = 1,15\cdot 28,5 - 2 = 30,775$ MPa, which gives

f_{cc} = 17,097 MPa for Ks 40 and $f_{ct} = f_{ctk}/1,5\gamma_n = 1,95/(1,5\cdot 1,2) = 1,083$ MPa

$E_s = E_{sk}/1,05\gamma_n = 200/(1,05\cdot 1,2) = 158,73$ GPa; $E_c = E_{ck}/1,2\gamma_n = 32/(1,2\cdot 1,2) = 22,222$ GPa

The tested concrete samples in Chapter 3 gives a higher value f_{cck} = 56,5 MPa. The results of the assessments presented here can thus be somewhat conservative.

The fatigue capacity is calculated for a typical stress collective. The speed is 100 km/hour.

Several **train loads** were tested. The different train loads are given in TRV Tåglaster (2010). The most severe one was tested first. If the section could not stand this load a lesser load was tried.

BV2000 has an axle load of 330 kN, a line load of 110 kN/m, and an axle distance for bogies and buffers of 1,6m

UIC 71 has an axle load of 250 kN, a line load of 80 kN/m; and an axle distance for bogies and buffers of 1,6m

BV-3 has an axle load of 250 kN, a line load of 80 kN/m, an axle distance for bogies of 1,8 m and for buffers of 3,0m

C3 has an axle load of 200 kN, a line load of 72 kN/m, an axle distance for bogies of 1,8 m and for buffers of 3,0m.

For the train load BV2000 they used a load distribution as in Figure D2.1 as the depth of the ballast on the bridge was larger than 0,6 m.

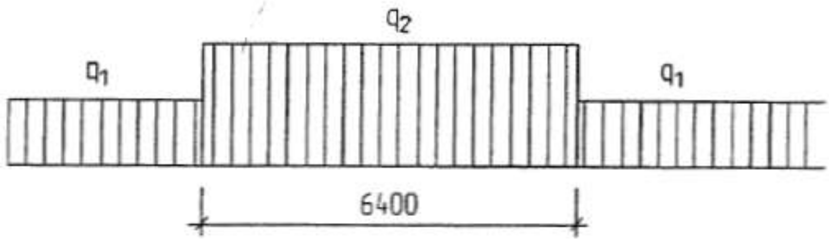

Figure D2.1. Load BV2000 as used by Leander-Fredriksson (2003), p 5:4, with q_1 = 116 kN/m and q_2 = 217 kN/m

A **deformation** of 14 mm at midspan for the live load was calculated (p.14:2) to be compared to the allowable deflection of L/800 = 89,5/800 m = 112 mm, with L = span length.

The following **sign conventions** are used, see Figure D2.2. Observe that the bending moment SM1 is positive when it gives tension in the top of the structure

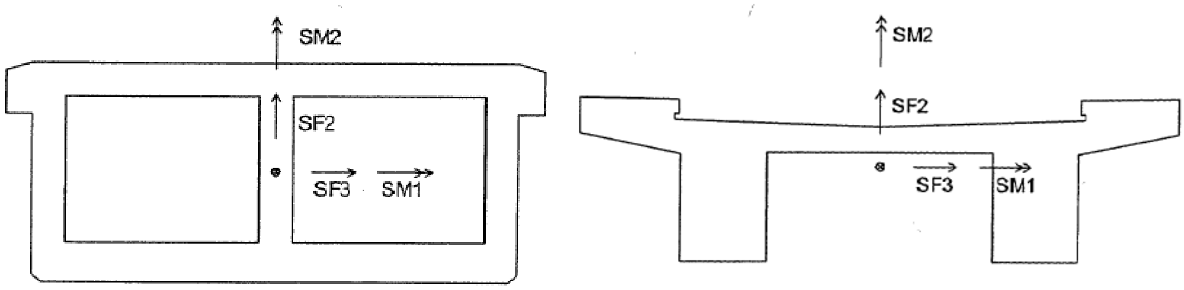

Figure D2.2 Sign conventions used for section forces for the arch (left) and the top slab and beams (right) used by Leander-Fredriksson (2003), p 8.2

D.3 Arch capacity

Five sections A, B, C, D, E and F were controlled by Leander-Fredriksson (2003), see Figure D3.1 (top).

In Table D3.1 some of the calculated bending moments (**E**ffects) $\mathbf{M}_E$ from different loads are given. Also, other section forces are given in Leander-Fredriksson (2003). For section A the ultimate capacities (**R**esistances) are calculated to M_R = 89,54 MNm and -111,11 MNm, giving M_R/M_E = 4,43 and 1,34 respectively, (p.9.17-18). Moments are here positive when giving tension in the bottom (contrary to what is assumed by Leander-Fredriksson.

In Table D3.2 some results are given. M_{max} and $M_{\min}$ denotes the safety factor for the applied maximum and minimum moments respectively ($M_{Resistance}$/M_{Effect}). The utility ratio η is the inverse of the smallest safety (e.g. 1/0,92 = 1,087) and should be ≤ 1.

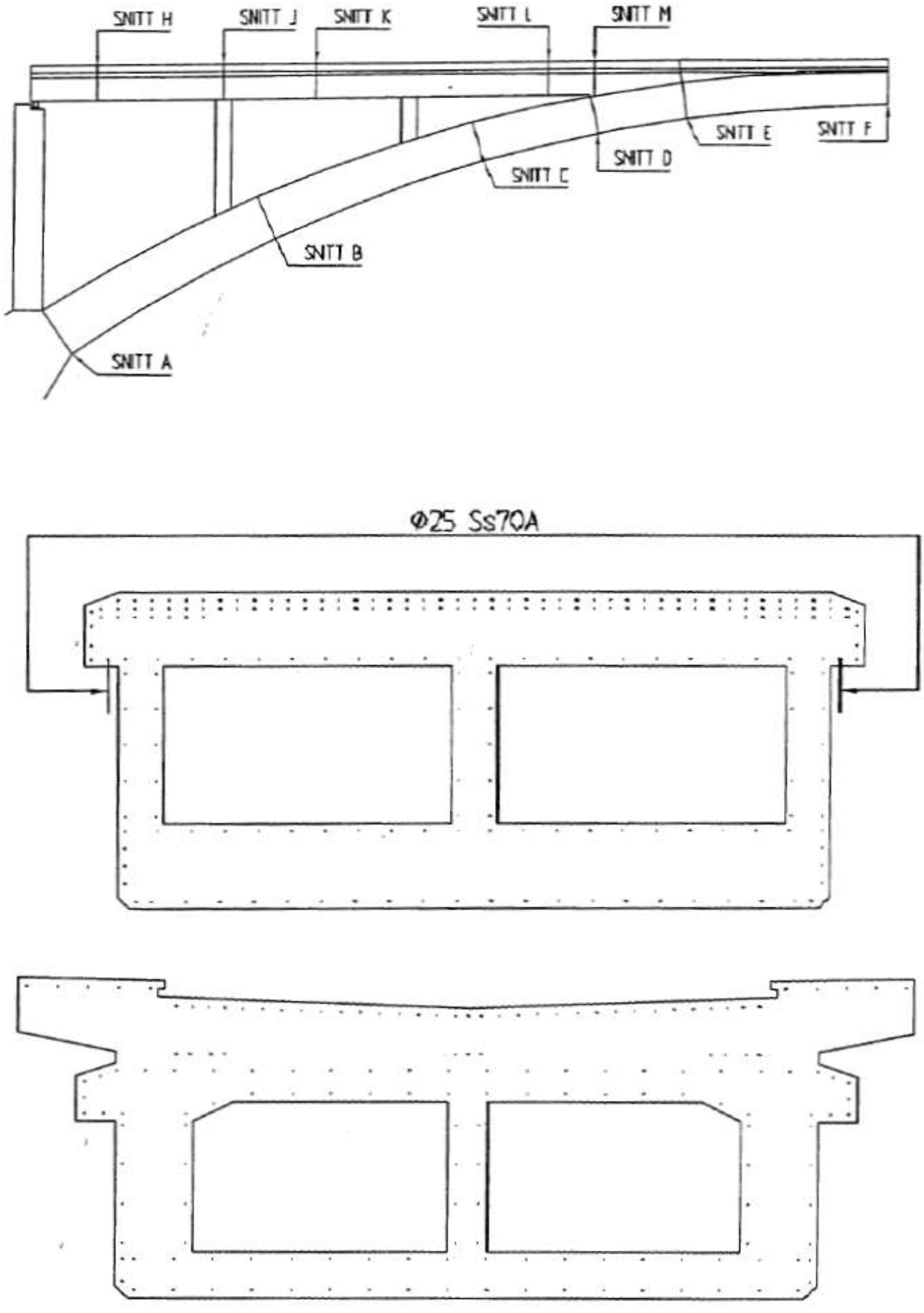

Figure D3.1 Midspan sections controlled by Leander-Fredriksson (2003). Top: Elevation, p 8:1; Middle: Section A, 112 ϕ25, Ss70A (f_{yk} = 720 MPa), 28 ϕ 25 and 74 ϕ16 Ks 40 (f_{yk} = 390 MPa) p.9:3; and Bottom: Section F, 68 ϕ19 and 98 ϕ16, Ks 40, p 9:6.

Table D3.1 Bending Moments caused by different loads from Leander-Fredriksson (2003), p. 8:2, 8:8. Moments giving tension in the top are denoted as positive, see Figure D2.2.

Bending moments, MNm	**Section A - Arch start**		**Section F - Arch top**	
	Normal	Exceptional	Normal	Exceptional
Dead Load	13,867	14,699	-4,073	-4,318
Ballast	1,241	1,315	-1,519	-1,610
Break 1	4,736	5,020		
Break 2		-5,020		
Temperature 1	-12,164	-19,341	5,783	9,125
Temperature 2		48,997		-23,294
BV2000-1	20,882	22,600	3,578	3,872
BV2000-2	-16,765	-18,144	-8,749	-9,469
BV2000-3	-16,049	-17,369	-3,321	-3,594
BV2000-4	19,999	21,644	-3,321	-3,594

Table D3.2 Safety factors and utility ratios η for arch sections, from Leander-Fredriksson (2003), p 9:7. M_{max} and $M_{\min}$ denotes the safety factor for the applied maximum and minimum moments ($M_{Resistance}/M_{Effect}$). The utility ratio η is the inverse of the smallest safety (e.g. 1/0,61 = 1,639) and should be ≤ 1. The positive moments here give tension in the top as in Figure D2.2

snitt	Tåglast	M_{max}	M_{min}	η	
A	BV-2000	1.34	4.43	0.746	
B	BV-2000	2.59	3.99	0.386	
C	BV-2000	2.66	3.28	0.376	
D	BV-2000	3.61	1.27	0.787	
E	BV-2000	3.67	2.39	0.418	
F	BV-2000	4.69	0.61	1.639	
F	-	5.73	0.74	1.351	utan inverkan av tåglast
F	BV-2000	4.90	1.23	0.813	ψ_γ = 0.6 på lasten temperaturändring

For section F, at the top of the arch, the train load BV-2000 gives a utility ratio of $\eta = 1{,}639 > 1$ with train load and $\eta = 1{,}351 > 1$ without train load, indicating too low a capacity for tension in the bottom at low temperatures. But when a lower load factor for temperature ($\psi_\gamma = 0{,}6$) is chosen $\eta = 0{,}813 < 1$, so the arch is OK.

According to Table D3.2, the arch has the highest safety for tension in the bottom in Section A (M_{min} =4,43, in Table D3.2), where the arch starts, and the highest safety for tension in the top in Section F (M_{max} = 4,69, in Table D3.2), at the top of the arch. This is probably caused by the fact that the moments of the opposite directions are the critical ones and that the arch has been designed for them and when the train load now is increased the safety is reduced. This would indicate that it is the reinforcement in the bottom of section F that may be critical for the arch at low temperatures.

D.4 Top beam longitudinal capacity

Five sections H, J, K, Land M, were controlled by Leander-Fredriksson (2003), see Figures D.3.1 and D.4.1

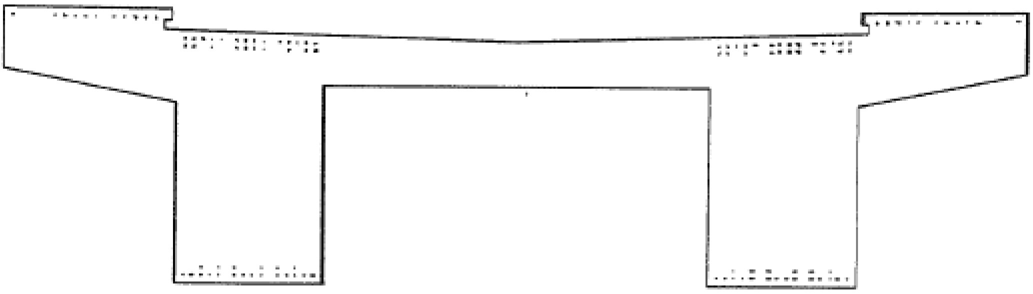

Figure D.4.1. Section M with reinforcement consisting of 110 bars ϕ 19 mm, page 10:5 in Leander-Fredriksson (2003),

The same train loads were tested as for the arch.

In Table D4.1 some results are given. For section M, where the top slab is joined to the arch, the train load BV-2000 gives a utility ratio of $\eta = 1{,}087 > 1$ for $M_{min} = 0{,}92$ indicating that the bottom reinforcement is slightly too small. Then the next lower train load UIC-71 was tested. It gives a utility factor of $\eta = 1{,}035 > 1$. But when BV-3 is tried a sufficiently low utility factor is achieved, $\eta = 1$.

Table D.4.1. Safety factors and utility ratios η for top slab beams, from by Leander-Fredriksson (2003), p 10:6. M_{max} and $M_{\min}$ denotes the safety factor for the applied maximum and minimum moments ($M_{Resistance}/M_{Effect}$). The utility ratio η is the inverse of the smallest safety (e.g. 1/0,92 = 1,087) and should be ≤ 1. See Figure D2.2 for sign convention.

snitt	Tåglast	M_{max}	M_{min}	η
H	BV-2000	3.41	1.23	0.813
J	BV-2000	1.33	9.61	0.752
K	BV-2000	2.83	1.35	0.741
L	BV-2000	4.60	1.19	0.840
M	BV-2000	4.08	0.92	1.087
M	UIC 71	7.15	0.97	1.035
M	BV-3	10.10	1.00	1.000

D.5 Top slab capacity

The slab is originally supposed to be **simply supported** on the longitudinal beams and the torsional capacity of the beams is neglected.

Four sections P, Q, R and S, were controlled by Leander-Fredriksson (2003), see Figures D.5.1

The same train loads were tested as for the arch.

In Table D5.1 some results are given for the bending moment. There are no problems for sections R and S to carry the bending moments of BV-2000. However, for sections P and Q the capacity is too low. It is unclear whether it is the top or bottom reinforcement that is too small. Section Q can manage UIC 71, but section P can only manage BV-3 with a reduced speed of 40 km/h or for C3 with the speed 100 km/h.

There are no problems with the shear capacity.

In order to study the influence of improved boundary conditions, i.e. that the slab is **built into the beams**, a new assessment was done by Leander-Fredriksson (2004). The slab was now modeled in Brigade Standard and the model is built of 3D shell- and beam elements.

In Table D5.2 some results are given for the bending moment. There are no problems for sections P, Q and S to carry the bending moments of BV-2000. However, for section R the capacity is too low for the top reinforcement and the section can only manage UIC 71

There are no problems with the shear capacity.

In fatigue, the bending stress range for the reinforcement $\Delta\sigma_s$. is maximally 183 MPa in Section S, 171,6 MPa in Section R, 161,9 MPa in Section Q and 157,7 MPa in section P, Leander-Fredriksson (2003), p 8:2

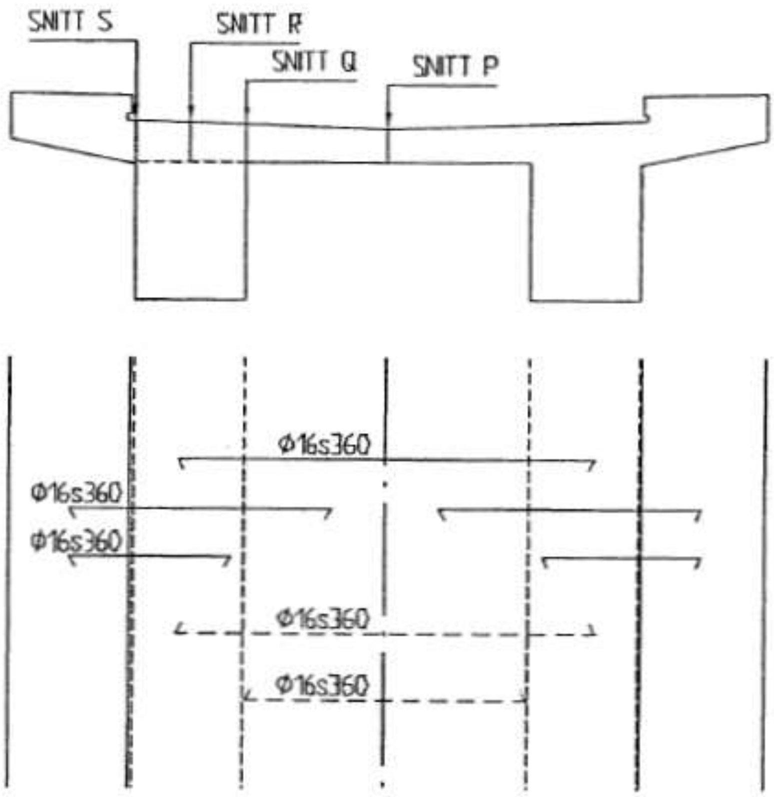

Figure D5.1. Sections of top slab controlled by Leander-Fredriksson (2003), page 11:8.

Table D.5.1 Safety factors and utility ratios η for top slab, from by Leander-Fredriksson (2003), p 11:8. M_{max} and M_{min} denotes the safety factor for the applied maximum and minimum moments ($M_{Resistance}/M_{Effect}$). The utility ratio η is the inverse of the smallest safety. See Figure D2.2 for sign convention

snitt	Tåglast	M_{max}	M_{min}	η	
P	BV-2000	0.50	56.37	**2.008**	
Q	BV-2000	0.96	22.65	**1.042**	
R	BV-2000	5.43	3.08	0.325	
S	BV-2000	12.75	12.58	0.079	
P	UIC 71	0.62	56.37	**1.608**	
Q	UIC 71	1.24	22.65	0.806	
P	BV-3	0.88	56.37	**1.135**	
P	BV-3 rh	1.01	56.37	0.990	med hastigheten v = 40 km/h
P	C 3	1.02	56.37	0.980	

Table D.5.2 Safety factors and utility ratios η for top slab, from by Leander-Fredriksson (2004), p 7:5. M_{max} and $M_{\min}$ denotes the safety factor for the applied maximum and minimum moments ($M_{Resistance}$/M_{Effect}). The utility ratio η is the inverse of the smallest safety. See Figure D2.2. for sign convention.

snitt	Tåglast	M_{max}	M_{min}	η
P	BV-2000	1.07	15.67	0.935
Q	BV-2000	1.59	1.11	0.901
R	BV-2000		0.90	1.109
S	BV-2000	2.86	1.04	0.962
R	UIC 71		1.14	0.877

D.6 Cross Beams

The cross beams were studied according to Figure D.6.1, Leander Fredrikasson (2003), p. 12:2. For bending moment and shear the mid-section U had a utility factor η = 0,817.

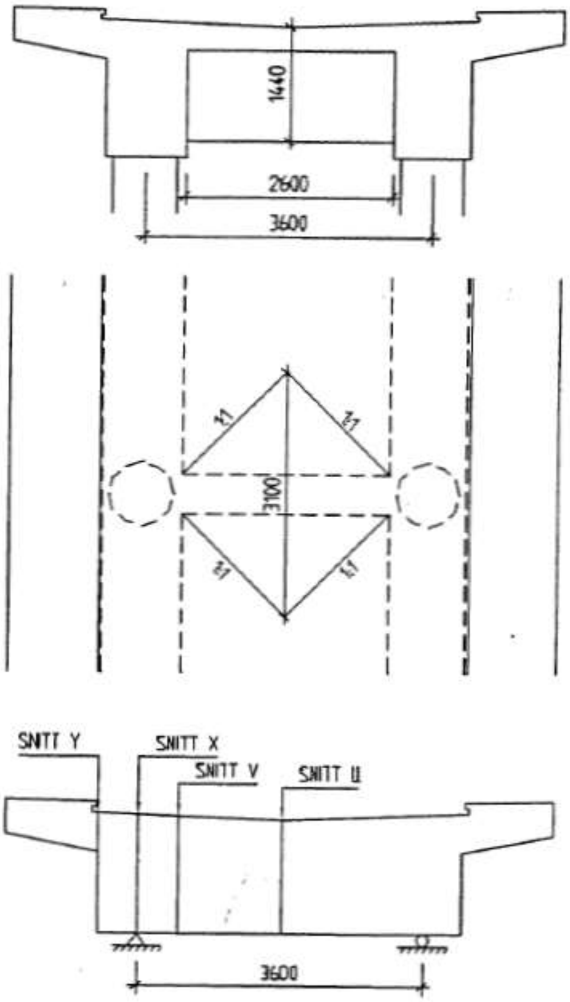

Figure D6.1. Cross beam with Sections U, V, X and Y., Leander-Fredriksson (2003), p 12:2.

D.7 Summary

The utility factors η are summarized in Table D.7.1. The capacity of the arch is OK for BV-2000 (axle load 330 kN) when temperature is not considered as a main loading case. The most critical part of the bridge is where the longitudinal beams join the arch, Section M in Figure D4.1. Here the reinforcement in the bottom of the beams only give a maximum allowable load of BV-3 (250 kN axle load).

Table D7.1 Maximum utility ratios (load/capacity) for different parts of the bridge and different load classes, Leander-Fredriksson, (2003), p.6, and (2004), p. 5.

Structural Part	**Section**	**Line Class according to BV Tåglaster (2009)**			
Train load class		**BV 2000**	**UIC71**	**BV-3**	**C3**
Axle load (stax), kN		330	250	250	200
Line load, kN/m		110	80	80	72
Distance between bogies/buffers, m/m		1,6/1,6	1,6/1,6	1,8/3,0	1,8/3,0
Arch	M_{min} in mid-section	**0,92** (1,64*)	-	-	-
Top Beams	M_{min} where the slab is connected to the arch	1,087	1,035	**1,000**	-
Slab, built in	M_{max} in mid-slab	1,109	**0,877**	-	-
(Slab, simply supported)	M_{max} in mid-slab	(2,008)	(1,608)	(1,135)	(0,980)
Cross beam	M and V	**0,817**			

*The number in parenthesis is for the case when a full temperature load is acting,