CASE STUDIES OF THE INNOVATIVE USE OF PARTIAL PRESTRESSING IN SWISS BRIDGES AND ITS AFFECT ON AESTHETICS

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ABSTRACT

Switzerland has led the way in developing partially prestressed bridges by fully integrating partial prestressing into their design code and making it the official design practice since 1968. Partial prestressing plays an important role in the design and development of Christian Menn's Felsenau and Reichenau bridges that allow the structures to be so elegant. The large cantilever overhangs of the Felsenau box girder provide a transparent and slender bridge supporting a six-lane roadway. The partial prestressing allows for the large overhang, but does not create the excessive upward deflection resulting from full prestressing for the high live loads. The problem of large moment reversals in the continuous deck of the Reichenau deck-stiffened arch is analyzed along with its use of partial prestressing as an innovative solution. The prestressing further stiffens the deck and allows for the wide spacing between crosswalls. In each case, calculations are made illustrating the creative use of partial prestressing, and the slender forms resulting from the use of partial prestressing are assessed for their aesthetic merits.

Keywords: Partial Prestressing, Partially Prestressed Concrete, Prestressed Concrete, Reinforced Concrete, Non-prestressed Reinforcement, Bridges, Aesthetics, Case Studies, Innovative Bridges, Design.

INTRODUCTION

Switzerland has led the way in developing partially prestressed bridges by fully integrating partial prestressing into their design code and making it the official design practice since 1968, with full prestressing only used in exceptional cases. Over 300 bridges have been constructed using partial prestressing with no documented problems directly related to it. This paper examines the innovative use and aesthetic result of partial prestressing through case studies of two bridges in Switzerland designed by Christian Menn.

CHRISTIAN MENN AND PARTIAL PRESTRESSING

Drawing from the influence of many works of Robert Maillart including his three-hinged box arches and deck-stiffened arches, Christian Menn has created many significant and aesthetically pleasing bridges in Switzerland and most recently in Boston. Two of Menn's works, the Felsenau Bridge and the Reichenau Bridge (Figures 1 and 2), have been shown throughout the world as elegant examples of structures that were not only economical and efficient, but enriched their surroundings visually. What many may not notice is the how partial prestressing plays such an important role in their design and development allowing the structures to be so elegant and innovative.

Figure 1 – Felsenau Bridge
(Photo by D. P. Billington)

Figure 2 – Reichenau Bridge
(Photo by Christian Menn)

Menn defines prestressing as "a special state of stress and deformations which is induced to improve structural behavior."[1] Notice that the reduction or elimination of tensile stresses is not a goal of prestressing according to Menn. "Limited" and "full" prestressing satisfy these two conditions respectively, whereas "partial" prestressing does not restrict concrete tensile stresses under service loads and thus encompasses the entire range between conventionally reinforced and fully prestressed concrete. The designer is then completely free to arrange the tendons and choose the desired amount of prestressing to achieve appropriate safety, serviceability, economy, and elegance. In order to determine an appropriate prestress for a structure, Menn refers to the importance of establishing a prestressing concept, the role of prestressing in terms of desired performance, at the early stages of design. Furthermore, the prestressing concept should overshadow any "degree of prestress", because a quantification of prestress does not indicate the quality, or performance, of the structure.[2]

The prestressing concept is established to address the issues of serviceability, economy, and construction. To satisfy economic considerations, the total amount of reinforcement provided should not exceed that required for safety at ultimate load, hence full prestressing will rarely be the prestressing concept. A common prestressing concept includes restricting cracking due to dead load plus prestress because moments due to permanent loads greater than the cracking moment will result in a large number of permanent cracks.[3] This concept usually results in preventing tensile stresses under dead load plus prestressing. The ability to control deformations may also influence the prestressing concept. One common case is to dimension the prestressing such that no vertical deflections result under dead load plus prestress (ie. load balancing). Menn refers to this type of prestress as "form-true" prestress.[4]

Menn's design approach[5] for common bridge types such as simply supported and continuous girders as well as segmental cantilever girders includes:

1. Determination of normal reinforcement required for an acceptable crack pattern.
2. Dimensioning of prestressing reinforcement to guarantee safety against collapse.
3. In exceptional cases, addition of normal reinforcement at extreme moment peaks to supplement prestressing steel.
4. Control of steel stresses and deflections.

For special bridge types such as arches and "form-true" prestressed members, Menn uses the following approach:

1. Determination of an economical, appropriate, or "form-true" prestress.
2. Dimensioning of normal reinforcement to guarantee safety against collapse.
3. Control of steel stresses (and deflections if necessary).

THE FELSENAU BRIDGE

Christian Menn proposed the Felsenau Bridge for a 1970 design competition of a six-lane viaduct north of Bern, Switzerland, over the Aare River at Felsenau. The cast-in-place bridge was constructed by cantilevering out on itself and tied back to the previously cast sections with prestressed tendons. Completed in 1974, three unique aspects of the bridge include the curved roadway plan with two-wall pier system, the profile (Figure 3) through a wide wooded to suburban valley, and the wide single-box cross-section.[6]

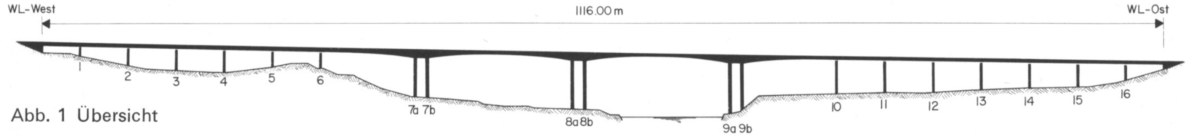

Figure 3 – Profile of Felsenau Bridge[7]

The curved roadway plan with curved box girder (Figure 4) and two-wall pier system (Figure 5) allows the structure to be continuous from end to end by limiting the affects of temperature and provides an efficient solution to the large longitudinal moments at the piers. The curved plan allows expansion outward, while the two-wall pier provides a stiff support with less material capable of resisting large moments. The two-wall pier also decreases the material needed for the long 156 m spans by decreasing the effective span to 144 m and provides a large section over the piers from which to erect the first cantilever section. Further, the two-wall pier and curved plan provide a visually interesting structure. The two-wall piers are thin sections that allow a more transparent view through the columns, especially compared to two thick columns of a twin structure.

Figure 4 – Curved Box Girder
(Photo by D. P. Billington)

Figure 5 – Two-wall Pier
(Photo by D. P. Billington)

Through the wooded to suburban valley, the haunched girders give the profile an arched appearance. The hauching is also efficient in the respect of using less material, however the haunching is more costly to construct, illustrating a possible compromise between economy and aesthetics to give a more visually interesting structure.

Finally, the wide roadway cross-section (Figure 6) is particularly important to emphasize the use of partial prestressing. The single box for a six-lane roadway is made possible by large cantilever overhangs that are partially prestressed. The single box with wide overhangs requires less material and less labor to build than a two-box structure or building twin structures. Time in construction is also saved by building the box girder in three sections (Figure 7), first the bottom slab and walls, then the deck slab between the walls, and finally the cantilever overhangs. The narrow box provides an integrated smooth transition from the piers to the box, as the box not only gets deeper but also gets narrower at the supports, and then from the box to the cantilevers with the sloping walls leading to a wide overhang. The wide overhang also creates interesting shadows on the much narrower girder and columns.

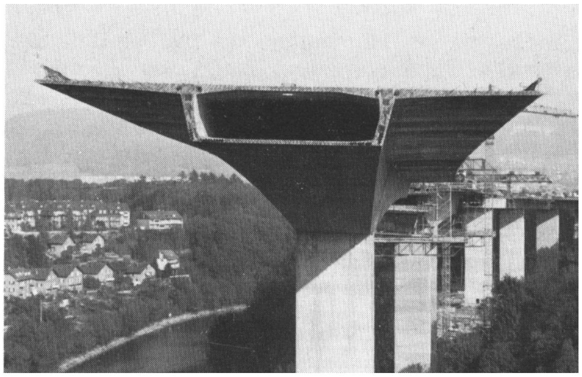

Figure 6 – Wide Overhang Cross-section[8]

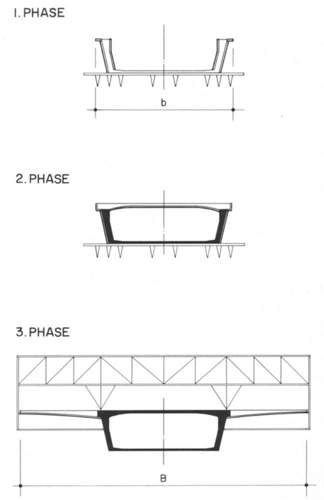

Figure 7 – Construction of Box Girder[9]

In design of the cantilever-constructed girder, Menn followed his first design approach by determining the normal reinforcement for an acceptable crack pattern and then dimensioning the prestressing reinforcement to meet the bending moment throughout.[10] This results in significantly different degrees of prestress throughout the girder while providing the necessary safety against collapse at every point. A single degree of prestress means essentially nothing for this design. At points of small moment, the necessary steel for developing an acceptable crack pattern provides a considerable portion of ultimate resistance, while in large moment regions the same steel provides significantly less of the overall capacity.

The large overhangs that cantilever out 7.5 m (24.6 ft) from the girder webs have a prestress for the cantilever dead load with normal reinforcement providing for ultimate load strength[11] (ie. design approach for "form-true" prestressed structures). Under dead load and prestress only, the prestress acts only axially through the slab, thus no deflections result. The construction sequence of forming the girder first and the cantilever slabs afterward, by simple scaffolding supported by the girder, along with the result of no deflections due to prestress immediately after forming allowed the scaffolding for the cantilever girder to be removed after only 36 hours and used in the next stage. The partial prestressing does not create the excessive upward deflection that would occur if the cantilever were fully prestressed to account for the high but transient live loads.

The following calculations verify the design of the Felsenau's cantilever deck overhang through a simple analysis with dimensions and resulting stresses matching published results in *Felsenaubrücke*.[12] All calculations were performed in metric units and where appropriate have been converted to English units for easy understanding.

DEAD LOAD

Figure 8 gives the general dimensions of the cantilever overhang section. The analysis uses a 1 m wide section, concrete strength of 3500 T/m^2 (~5000 psi), and density of concrete of 2.5 T/m^3 (156 pcf). The dead load moment at the cantilever support due to the concrete slab, barrier rail, and wearing surface were calculated to be 19.8, 3.3, and 6.1 mT/m section respectively. For form-true prestressing, the prestressing should be provided to result in no deflection under the dead load of the slab. The published results give a moment of 23.1 mT for this dead load, which matches that due to the combination of slab and barrier rail. Combining the entire dead load produces a moment of 29.2 mT at the support, slightly less than the 31.4 mT moment provided in the published results. This difference could easily be accounted for by an allowance for a slightly thicker wearing surface (a 13.5 cm wearing surface as opposed to a 10 cm wearing surface produces 31.4 mT) or a heavier wearing surface. The published results of 23.1 mT for prestressed dead load moment and 31.4 mT for total dead load moment will be used for further calculations. These moments are shown in Figure 9.

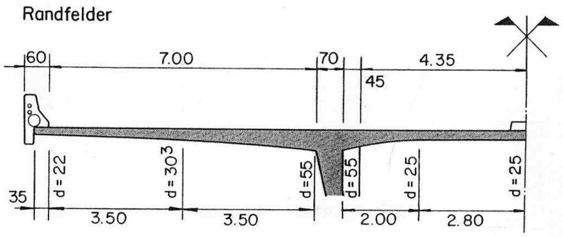

Figure 8 – Dimensions of Overhang[13]

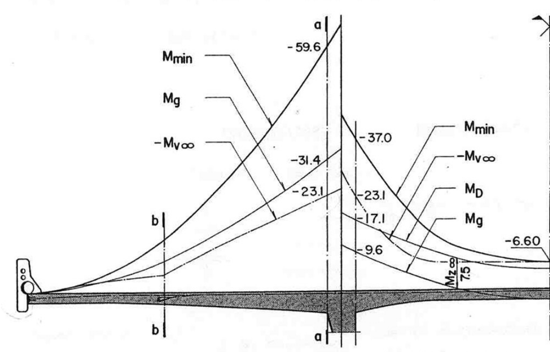

Figure 9 – Moments in Deck Slab[14]

LIVE LOAD

Live load moments in the cantilever at the girder support were verified assuming AASHTO truck live and impact loadings reduced to the corresponding 16 metric ton truck given in the published results. Five 8.32 T wheel loads factored for impact result on the wide cantilever and were positioned as shown in Figure 10, starting 1 foot from the barrier and spacing toward the girder support. The wheel loads are distributed according to AASHTO and produce a live load bending moment in the cantilever at the girder support of 33.5 mT/m section. The published live load moment was 28.2 mT, thus using AASHTO truck loadings produces a slightly larger moment at the support. Differences are obviously a result of the live load scenario used in the Swiss code differing from this AASHTO live load case. Again, the published live load moment of 28.2 mT will be used for further calculations to produce accurate design replication.

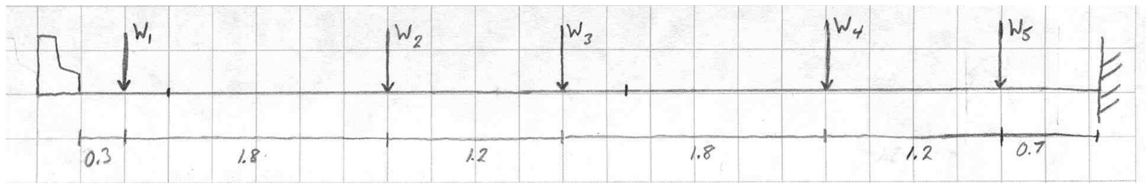

Figure 10 – Position of Live Loads on Cantilever Deck

PRESTRESSING FOR DEAD LOAD

Given the dead load moment of 23.1 mT to balance with prestressing and a maximum eccentricity at the girder support of 21 cm, allowing for 6.5 cm of cover, the required prestressing force is

$$F_{ps} = M_{ps} /e = 110 \text{ T/m section.}$$

To find the necessary area of prestressing steel for form-true prestress the prestressing force is divided by $0.6f_{pu}$, the practical prestressing steel stress after considering losses, where f_{pu} = 19 T/cm^2 (270 ksi) such that

$$A_{ps} = F_{ps} /(0.6f_{pu}) = 9.65 \text{ cm}^2.$$

Using ½” diameter 19 T/cm^2 (270 ksi) strands with an area of 0.987 cm^2, 9.777 strands/m section are required. Thus groups of 3 – ½” diameter strands every 0.3 m (10 strands/m section) as shown in Figure 11 from the published report was provided for the prestressing. This results in a prestressing force after losses of 112.1 T and taking 10% losses as assumed in the published results confirms an initial prestressing force of 124.6 T/m section. Notice in Figure 11 that every other prestressing cable ends at approximately half the cantilever span since the required prestress decreases along the cantilever.

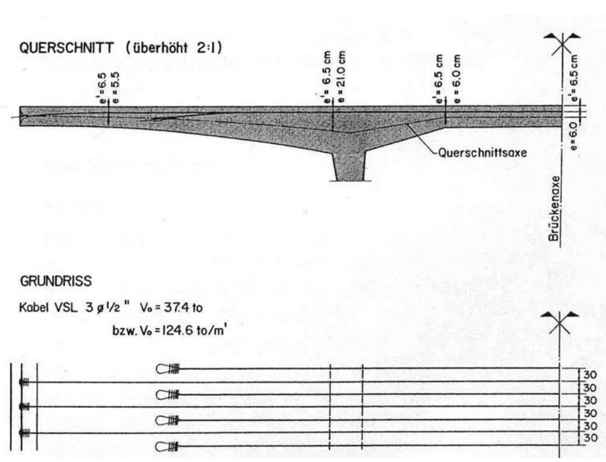

Figure 11 – Prestressing Cable Arrangement[15]

STRESSES IN UNCRACKED CONCRETE SECTIONS

The area and section modulus of the cantilever section at the support are 0.55 m^2 and 0.0504 m^3 per m section respectively. Thus the stresses in the section at the support are determined by adding the stresses due to applied moment

$$f = \pm M/S$$

to the stresses resulting from prestress

$$f_{ps} = \pm M_{ps} /S - F_{ps} /A.$$

Figure 12 shows the stresses in the concrete at the cantilever support for the prestressed dead load, total dead load, and full service load.

The form-true prestress for prestressed dead load results in load balancing such that only axial force exists in the section, hence no deflections (the slight difference is due to rounding up to whole cables). Notice that for total dead load the entire section is still in compression, thus tension and therefore cracks will not result. A compression stress of -20 T/m^2 was reported in the published paper. Given that this value results from the difference between two large stresses (623 – 671 = -48 T/m^2), the error is small. The section goes into tension at a decompression moment of 33.8 mT. Finally, at full service load the applied moment is equal to the total dead load moment plus the live load moment, which equals 59.6 mT. The resulting tensile stress at the tension face is 512 T/m^2 (727 psi), which is significantly greater than the limiting tensile stress of $6\sqrt{f_c'}$ = 424 psi used in the United States and the cracking stress, $7.5\sqrt{f_c'}$ = 530 psi, for 5000 psi concrete. Thus the top face of the cantilever slab has cracked under full service load and the section must be investigated for stresses in the tensile steel for a cracked section.

		Moment (mT)		Force (T)	Stresses (T/m^2)*		
Load Case	Face	Applied	P/S	P/S	$M_{applied}$	P/S	Total
P/S DL	Top	23.1	23.55	112.1	458	-671	-213
	Bottom				-458	263	-195
DL	Top	31.4	23.55	112.1	623	-671	-48
	Bottom				-623	263	-360
DL + LL	Top	59.6	23.55	112.1	1183	-671	512
	Bottom				-1183	263	-920

* Compression is negative

Figure 12 – Stresses in Uncracked Concrete Sections

CALCULATIONS FOR ULTIMATE

Before calculating the stresses in the steel for a cracked section, ultimate moment strength must be provided for the section by dimensioning the normal reinforcement to guarantee safety against collapse. Assuming the regular reinforcement is at the same depth from the compression face, d, as the prestressing steel which had a 6.5 cm cover, then d = 55 – 6.5 = 48.5 cm. The depth of the concrete stress block, a, is then determined by solving the equation

$$M_u = 0.85f_c\text{'}ba\ (d - a/2)$$

for a, where M_u is the ultimate moment determined in the Swiss code as $1.8(M_{DL} + M_{LL})$ = 107.28 mT, b is the width, 1 m, and f_c' = 3500 T/m^2. Using a = 0.081 m, the proper area of reinforcing steel, A_s, is dimensioned using the equation

$$M_u = (A_{ps}f_{py} + A_sf_{sy})(d\text{-}a/2),$$

where M_u, A_{ps}, d, and a have already been determined, f_{sy} is the yield stress of regular reinforcement, 60 ksi or 4.225 T/m^2, and f_{py} is the yield stress of the prestressed reinforcement. A yield stress of 16.8 T/m^2 is used, approximately 90% of ultimate as suggested in the AASHTO Code.[16] This leads to a required area of normal reinforcing steel equal to 17.9 cm^2 and 18 cm^2 was provided.

CALCULATIONS FOR CRACKED SECTION AT SERVICE LOAD

At service load, the tensile stress in the concrete assuming an uncracked section exceeds the rupture of the concrete, thus the concrete has cracked. Cracking in concrete has been linked to the spacing of normal reinforcing steel and stresses in the normal reinforcing steel as well as increased stress in the prestressing steel. Using the closest possible spacing consistent with placing concrete, cracks will be small and well distributed. The steel stresses in the normal reinforcement and change of stress in the prestressed steel at service load are determined from a cracked cross-section assuming the stress-strain distribution is still linear. Limiting the stresses and therefore strains in steel along with proper spacing of steel will limit the size of cracks occurring in the section. The same limitations also confine the extent of fatigue. Swiss bridge experience has found that by limiting the stresses in reinforcing steel and stress increases in prestressed steel to 150 MPa (1530 kg/cm^2 = 21.3 ksi) acceptable crack widths and results for fatigue loading are achieved.[17]

The following procedure for calculating stresses in the steel at a cracked cross-section has proven effective in Swiss design and tables based on this method have also been implemented to aid in design. The Swiss use the prestressing force after losses, neglecting the time-dependent effects of additional nonprestressed steel in the flexural tension zone, as a simplification to determining the force in the prestressing steel at decompression because the error involved is small.[18] The neutral axis of the cross-section after cracking is constantly changing with increasing moment. To find the neutral axis for the applied moment at service load, an axial load equal to the prestressing force, F_{ps}, with eccentricity, $e' = M_{SL} / F_{ps}$, from the prestressing steel is applied as shown in Figure 13. Using a transformed section, moments are summed about the eccentric force to obtain a cubic equation for determining the neutral axis,

$$\Sigma M_{Fps} = -f_c \, (n_s A_s + n_{ps} A_{ps}) \, (d - y) / y + \tfrac{1}{2} f_c \, (b \cdot y) \, (y/3 + e' - d) = 0.$$

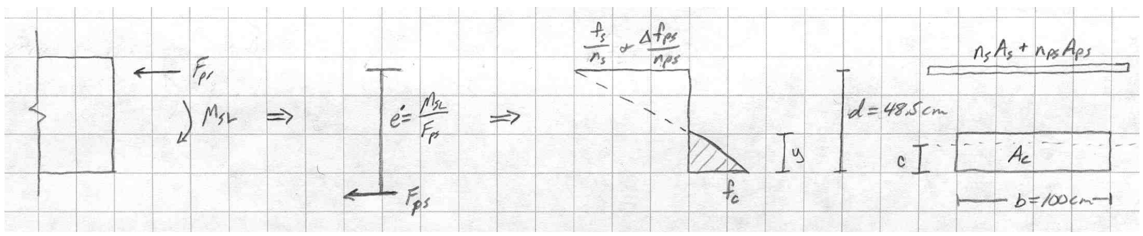

Figure 13 – Analysis for Cracked Transformed Section

$E_c = 283500$ kg/cm^2	$E_s = 2040000$ kg/cm^2	$E_{ps} = 1900000$ kg/cm^2
$d = 48.5$ cm	$n_s = E_s/E_c = 7.2$	$n_{ps} = E_{ps}/E_c = 6.7$
$b = 100$ cm	$F_{ps} = 112.1$ T	$M_{SL} = 59.6$ mT
$e' = 53.2$ cm	$y = 21.65$ cm	$c = 13.95$ cm
$A_{cr} = 2361$ cm^2	$I_{cr} = 339350$ cm^4	

Figure 14 – Parameters and Calculated Values for Cracked Section at Service Load

After solving for y, the area, centroid, and moment of inertia of the cracked section are easily found. Parameters used and values calculated are shown in Figure 14. Stresses in the concrete and reinforcing steel and the stress increase in the prestressing steel are calculated using the following formulas:

$$f_c = -F_{ps}/A_{cr} - F_{ps} \, (e' - d + c) \, c / I_{cr} = -133 \text{ kg/cm}^2$$

$$f_s = -n_s f_c \, (d - y) / y = 1190 \text{ kg/cm}^2$$

$$\Delta f_{ps} = -n_{ps} f_c \, (d - y) / y = 1107 \text{ kg/cm}^2.$$

The published report shows values of $f_c = -133$ kg/cm^2, $f_s = 1530$ kg/cm^2, and $\Delta f_{ps} = 1420$ kg/cm^2. The difference in the values appears to be due to a smaller area of normal reinforcing steel, possibly the area needed to satisfy the cracked section such that it remains less than the suggested limit, 1530 kg/cm^2, and higher, more conservative values of modular ratios. The ratio between n_s and n_{ps} matches the results. For instance, using a smaller area of normal reinforcing steel, $A_s = 12.75$ cm^2, and modular ratio $n_s = 10$ results in values approximating those in the published report: $f_c = -139$ kg/cm^2, $f_s = 1514$ kg/cm^2, and $\Delta f_{ps} = 1408$ kg/cm^2. Regardless, the results calculated above show that the regular reinforcing steel provided, A_s = 18 cm^2, to satisfy ultimate moment is enough to keep stresses in the normal reinforcement and stress increases in the prestressing steel below the suggested limit of 1530 kg/cm^2. Thus additional normal reinforcement is not needed to control cracks and limit fatigue and the section performs adequately under full service loads.

PARTIALLY PRESTRESSED VS. FULLY PRESTRESSED OVERHANG

Analysis of the partially prestressed section has been shown above, but as a comparison calculations were made for fully prestressing the section such that no tensile stresses result under the full service load moment of 59.6 mT. Deflection values were also calculated, using a simple SAP2000 analysis for the original prestressed load and full dead load, where assuming elastic behavior, absence of cracks, is valid.

The reason for prestressing for only a part of the dead load (P/S DL) rather than the entire dead load was to balance loads such that no deflections resulted and the forms could be removed and used for the next section after only 36 hours. For the fully prestressed section under the P/S DL load case a large compression stress (-725 T/m^2) results in the top of the slab and a small tensile stress (6 T/m^2) in the bottom. Thus a large moment results and removal of the formwork after only 36 hours would result in deflections before creep and shrinkage of 1.7 cm at the cantilever tip. Obviously as creep and shrinkage occur, the slab would have a tendency to deflect further upward under this prestressed loading condition. Recall that the P/S DL load case included the barrier. If the barrier is not included as a point load on the cantilever tip and only the slab exists upon removal of the forms, then further upward deflection would result under full prestressing. The resulting deflection and stresses are also taking into consideration the prestressed force after losses, such that under the jacking force, before losses have occurred, stresses of -857 T/m^2 and 58 T/m^2 result at the top and bottom of the slab respectively. Given the larger upward moment and resulting stresses, even larger deflections would occur.

Under partial prestressing for the P/S DL load case the stresses are relatively small, compressive, and almost equal across the entire section resulting in approximately zero deflection. Under full dead load, the most frequently occurring load on the bridge after completion, both sections are in complete compression, however a net upward deflection of 0.76 cm and downward deflection of 0.93 cm result for the fully prestressed section and partially prestressed section respectively. This is assuming that excessive upward deflection has not already resulted from full prestressing before the full dead load is added. Both cases have a very similar overall stress gradient, difference between top and bottom stresses at full dead load, thus similar long-term deflections would occur, except in opposite directions. Deflections under full service load would be zero for the fully prestressed section, while deflection calculations become more complicated for the partially prestressed section. After cracking has occurred, deflection calculations must be based similar to those for reinforced concrete sections considering bending and axial loads, however long-term deflections under permanent load are the main concern in which case the partially prestressed section is still completely in compression.

Further, form-true prestressing for dead load cannot be accomplished with full prestressing in most cases, especially for slabs where the decompression moment is approximately 1.45 M_{DL}.[19] This has been verified above for the original prestressed load because M_{Dec} = 33.8 mT < M_{SL} = 59.6 mT. Even if the slab was prestressed to balance full dead load, M_{DL} = 31.4 mT, where M_{ps} = 31.4 mT and F_{ps} = 149.5 T after losses, the decompression moment, M_{Dec}, is only 45.1 mT < 59.6 mT.

FULL UTILIZATION OF REINFORCEMENT

Full prestressing results in the use of excessive amounts of steel in most cases and as shown below the full utilization of reinforcement is only achieved with partial prestressing. It has already been shown above that for the partially prestressed section the ordinary reinforcement was dimensioned to meet ultimate load exactly. However the fully prestressed section is prestressed for the service load moment, 59.6 mT, such that the tensile stress due to service load is completely balanced by the compressive prestressing stress in the top of the slab:

$$f_{SLtop} = M_{SL}/S = 1183 \text{ T/m}^2,$$

$$f_{ps} = 1183 \text{ T/m}^2 = -M_{ps}/S - F_{ps}/A \text{ where } F_{ps} = M_{ps}/e, \text{ then}$$

$$M_{ps} = 41.5 \text{ mT and } F_{ps} = 197.7 \text{ T (after losses).}$$

This results in an area of prestressing steel

$$A_{ps} = F_{ps}/(0.6f_{pu}) = 17.34 \text{ cm}^2,$$

approximately 75% more prestressing steel than used in the partially prestressed section. The ultimate capacity provided by the prestressing steel is thus

$$M_u = A_{ps}f_{ps}(d-a/2) = 129.5 \text{ mT},$$

where f_{ps}, d, and a are the same values used for the partially prestressed section. M_u of 129.5 mT is approximately 20% greater than the required M_u of 107.28 mT. In many instances, some ordinary reinforcement is required in fully prestressed sections due to tensile stresses from temperature, shrinkage, support settlements, overloading, or variations in moment in indeterminate structures from those calculated, which would only increase the provided moment. Thus significant economic advantages are possible with partial prestressing by fully utilizing steel to meet ultimate load conditions.

FELSENAU CONCLUSIONS

Although only the critical section at the cantilever support was analyzed here, other sections could easily be analyzed in the same manner. The previous analysis has shown some of the many advantages of partial prestressing through the design of a statically determinate cantilever overhang with simple guidelines and easily applicable formulas. It also shows the advancement in Swiss design beyond that applied in the United States and a large portion of the world.

THE REICHENAU BRIDGE

The Reichenau Bridge over the Rhine at Tamins near Reichenau completed in 1964 is a deck-stiffened arch that draws its influence from Robert Maillart's deck-stiffened Valtschielbach (43.2 m) and Schwandbach (37.4 m) arch bridges, however with a significantly longer main span (100 m) several variations and innovations were necessary. The increasing labor costs in Europe following World War II made closely spaced vertical crosswalls uneconomical leading to the wider spacings made possible by advances in prestressing. The prestressing made deck stiffening possible for the widely spaced girder supports. The wide overhang of the prestressed single-cell box also casts a shadow on the vertical walls of the box leading to a more slender deck appearance. When combined with the widely spaced crosswalls, a "more open – transparent – and simpler appearance" is produced resulting from changes in the construction industry that produced a more economical, easier to build structure.[20]

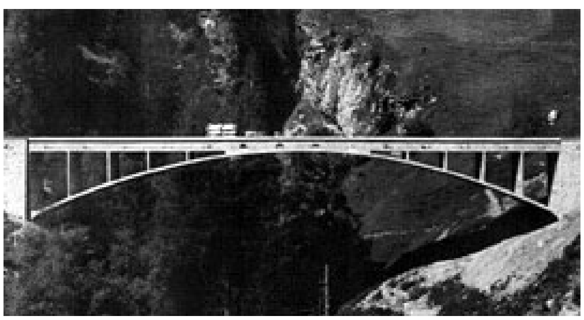

Figure 15 – Maillart's Valtschielbach
(Photo courtesy of M.-C. Blumer-Maillart)

Figure 16 – Menn's Reichenau
(Photo by Christian Menn)

Figure 17 – Wide Spacing Between Crosswalls
(Photo by D. P. Billington)

Figure 18 – Overhang of Deck
(Photo by D. P. Billington)

Full prestressing for large service loads in all directions creates a stiff structure with high initial stresses. These structures are uneconomical due to the large amount of prestressing steel and larger sections necessary for the prestress force, are less resistant to impact forces, and will likely be over-reinforced leading to brittle failures and a lack of safety at ultimate load.

Given the nature of the continuous deck connected by the crosswalls to the arch, large moment reversals are created in the spans under various live loading situations and thus full prestressing to account for both large positive and negative moments becomes uneconomical. To solve this problem, Menn prestressed the deck for the dead load moment of the girder, deriving from a continuous girder over rigid supports, and also implemented a centric prestress with tendons in the top and bottom of the deck for a portion of the live load. The remainder of the live load is taken by normal reinforcement. Again this provides the exact amount of steel necessary for ultimate moment capacity. The prestressing of the deck further increases the bending stiffness of the slender arch and deck system reducing the effects of second-order moments due to decreased moment-of-inertia after cracking and increased deflections.[21]

The following analysis investigates these ideas for the design of the Reichenau Bridge. Original design drawings and specific details of the design were not available for a detailed study, however the general concepts have been traced through Menn's paper, "Partial Prestressing from the Designer's Point of View" and his book, *Prestressed Concrete Bridges*. Dimensions such as the arch length, 100 m, arch rise, 20.9 m, deck width, 8 m, and deck depth, 1 m, were known, however other dimensions used were scaled from pictures and figures in Menn's book. These figures were not necessarily the Reichenau Bridge, but the chapter on arch bridges follows directly from his experience designing these deck-stiffened arch bridges.

DEAD LOAD

The assumed cross-sections of the deck girder, arch, and crosswall are shown in Figure 19 along with their corresponding areas, neutral axes, and moments of inertia. The crosswall spacings were scaled from figures in Menn's book where the spacings become smaller closer to the center of the arch for visual continuity and are shown in Figure 20 along with the support conditions. The polygonal geometry of the arch was determined by locating the base of each crosswall as close as possible to the pressure line due to dead load using just the above cross-sections with a density of concrete of 2.5 T/m^3 and assuming a three-hinged arch. Thus the slope of the arch breaks at each crosswall and the arch was modeled as with straight-line segments between the crosswalls. In the actual bridge, the arch segments between crosswalls are slightly curved to account for the dead load of the arch, but definite angle breaks are made at the crosswalls to show the true visual form of the arch.

With the arch assuming the polygonal geometry derived from the dead load pressure line of the bridge, very little bending moment is observed under dead load in the arch and the bending moments corresponding to a continuous girder on rigid supports appear in the deck as shown in Figure 21. The negative moment region over the center of the arch is affected by modeling the small vertical crosswalls and the simplified sections, however a similar moment diagram result is shown in *Christian Menn – Brückenbauer*, a book on Menn's bridges.[22]

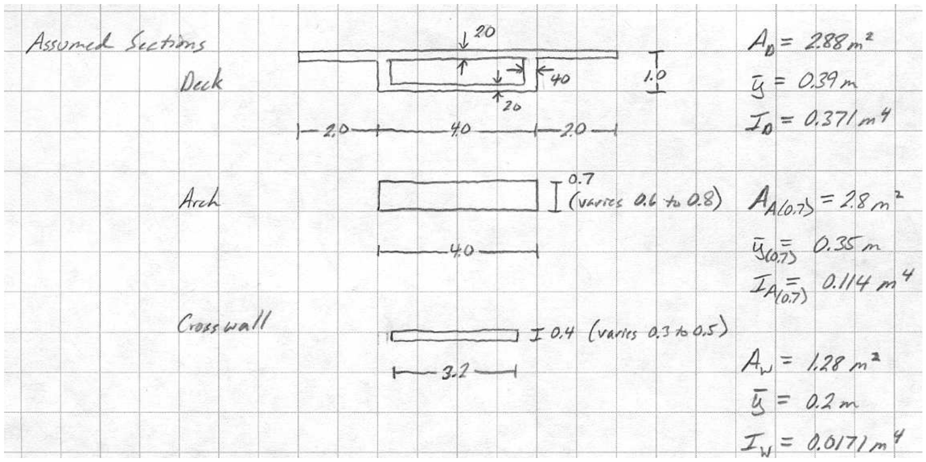

Figure 19 – Assumed Cross-sections

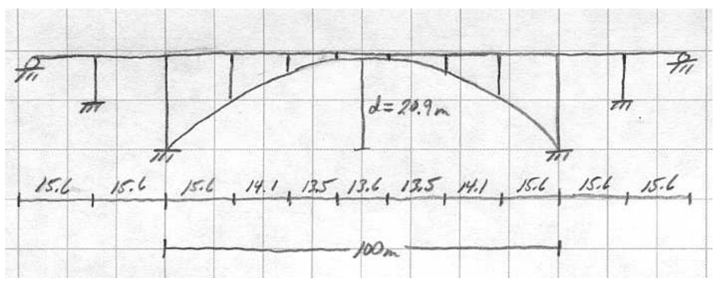

Figure 20 – Crosswall Spacing

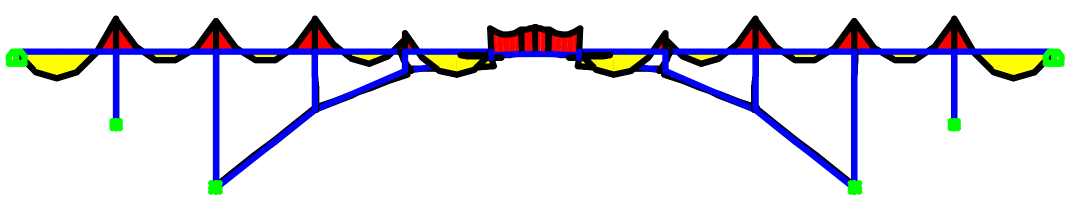

Figure 21 – SAP2000 Analysis Moments from Dead Load

LIVE LOAD

Due to the connection of the arch and deck through the crosswalls, the arch and deck combine to resist live load moments and will deform together. The moments of the combined arch/deck interaction are thus shared such that,

$$M_G = M * I_G/(I_G + I_{A,C}) \text{ and } M_A = M * I_{A,C}/(I_G + I_{A,C}),$$

where M is the moment obtained from either the a purely deck-stiffened arch or a purely stiff arch. M_G and M_A are the moments in the girder and arch, while I_G is the moment of inertia of the girder and $I_{A,C}$ is the moment of inertia of the arch at the crown.

Two main partial live load cases create maximum positive and negative moments in the deck/arch interaction. The first case involves loading only one half of the arch and the second case involves loading the center third of the arch. More detailed load cases should be used to determine final maximum moments, however these two load cases show the general trend resulting in large positive moments that can be completely reversed under the opposite loading condition. These moments are then proportioned via the formulas shown above to the deck and arch. Based on the moments of inertia of the arch at the crown and deck, the arch should take approximately 16% of the moment and the deck 84% of the moment.

The live loading scenarios described above were analyzed with the SAP2000 model for a scaled down AASHTO HS live loading to coincide with a 16 metric ton truck, as used for design of the Felsenau Bridge, with two lanes resulting in a distributed load of 1.68 T/m and point load of 14.4 T. The moment diagrams for the two cases described above along with an overall moment envelope for the maximum positive and negative moments are shown in Figures 22 – 24. The smallest maximum moments are used to dimension the concentric prestress in the deck. Maximum positive moments in the deck and arch at the quarter point for the half live load case over that quarter point result are 165.8 mT and 39.2 mT respectively. This corresponds to the deck taking approximately 81% and the arch taking 19%.

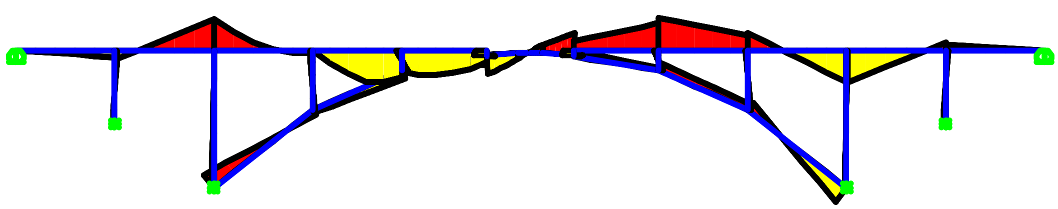

Figure 22 – SAP2000 Analysis Moments from Half Live Load on Left Side

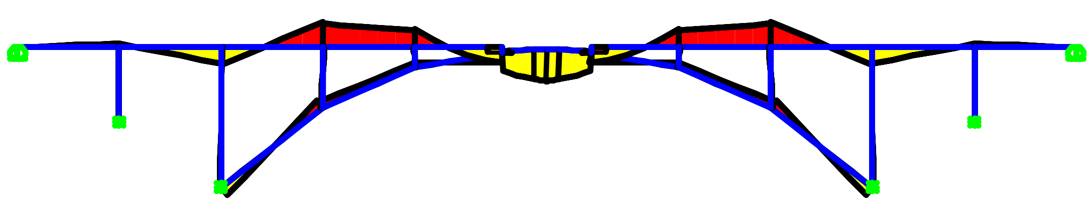

Figure 23 – SAP2000 Analysis Moments from Center Live Load

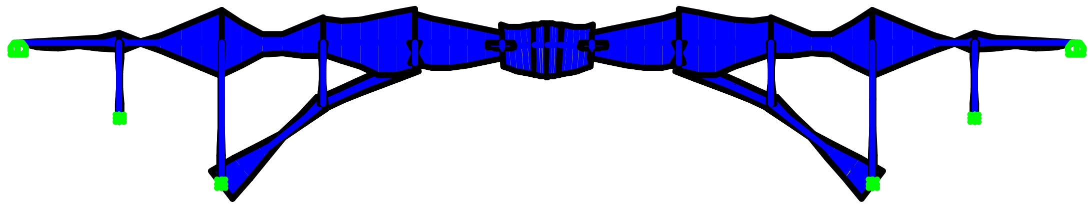

Figure 24 – SAP2000 Analysis Moments from Live Load Envelope

PRESTRESSING THE DECK GIRDER

Dead load and live load moments in the deck girder follow a completely different pattern, thus efficient prestressing for the two conditions must consist of two parts. Menn uses a form-true prestress to counteract the dead load moment such that dead loads are balanced and no deflection results in the girder. Prestressing is provided for the smallest maximum live load moment in the girder in a concentric manner such that both the top and bottom slab are prestressed the same amount to account for positive and negative moments that occur at that section. The remaining moment in sections with higher moment is resisted by normal reinforcing steel proportioned to meet ultimate load requirements. This arrangement, similar to the proportioning in the Felsenau example, assures an economical use of steel. If the girder were concentrically prestressed for the maximum moment, significantly more prestressing steel would be needed than is necessary at the sections with small maximum moment. Further, prestressing for such a large moment would require significantly greater prestressing force and lead to a larger deck cross-section to handle this force.

Figure 25 shows a detail of the tendon arrangement in the prestressed deck girder. Group II are the dead load prestressing tendons moving up and down in the webs to counteract the dead load bending moment of the continuous girder over the crosswall supports. Groups I and III are the concentric prestressing cables for live load that run horizontally in the top and bottom slabs. Notice that Groups II and III stop before the arch crown, while Group I in the deck slab continues all the way through the girder. The larger section at the crown resists live load moments over the crown of the arch. The prestressing in the top slab helps control cracks in the deck slab and based on the SAP2000 dead load moments would help counteract the negative dead load moment over the center of the arch. Figures 26 and 27 show details of the tendon arrangements within the sections at the points circled in Figure 25.

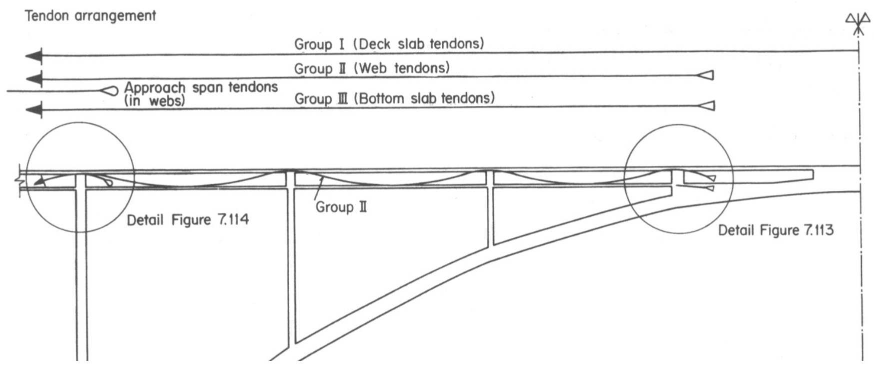

Figure 25 – Prestressing Tendon Arrangement in the Deck Girder[23]

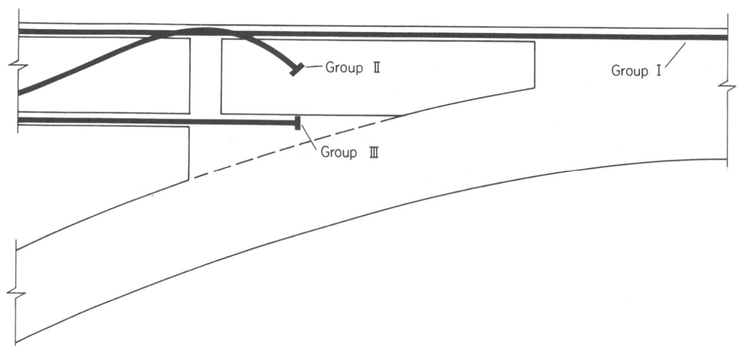

Figure 26 – Prestressing Tendon Arrangement near Crown[24]

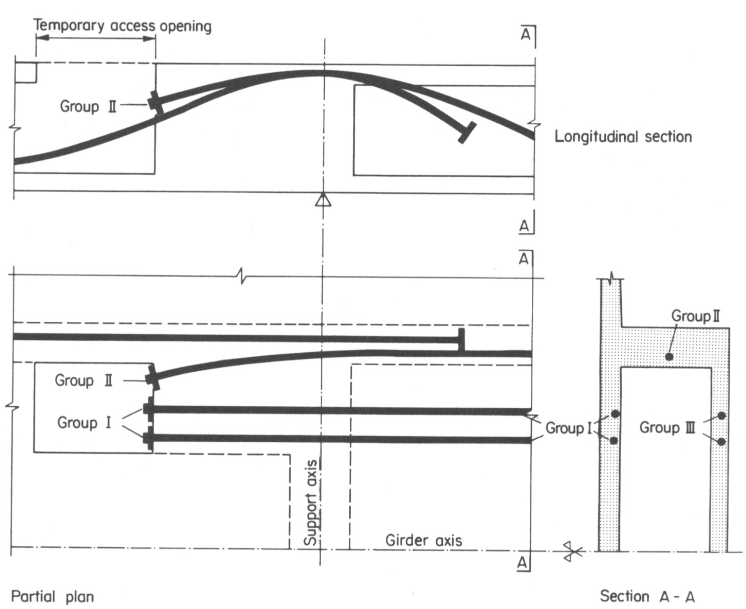

Figure 27 – Prestressing Tendon Details[25]

CONCLUSIONS

The two design examples have illustrated the innovative use of partial prestressing by Christian Menn in two bridges designed in the 1960's and 70's that was made possible by the greater realm of partial prestressing encouraged by the Swiss code. This paper also described the aesthetic advantages brought forth by partial prestressing. The optimum amount of prestressing and regular reinforcement has been guided by economic and design factors leading to various prestressing degrees throughout a structure. It is important to note that the prestressing degree had no influence on the design. Rather, conditions of ultimate load capacity and service load behavior were met independent of any optimum prestressing degree. Limitation of stresses in regular reinforcing steel and stress increases in prestressing steel after decompression as well as proper distribution of steel along the tensile face provide adequate behavior at service loads due to cracking and fatigue related problems. Partial prestressing also allows the full utilization of reinforcement to create an economical design.

The area of prestressing steel necessary to satisfy ultimate conditions for a slab and in many other situations is less than the area required to satisfy working load conditions for full prestressing, thus partial prestressing is advantageous in saving steel. As shown in the two examples, disadvantages of the use of full prestressing with only prestressing steel include the inability to "form-true" prestress and effectively deal with moment reversal. Finally, the use of simplified concepts of partial prestressing and structural concrete design with careful attention to design and construction led to the high quality, efficient, economic, and elegant bridges analyzed above.

ACKNOWLEDGEMENTS

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