PRINCIPAL RENOVATIONS FOR THE OLYMPIC ROWING STADIUM IN HELSINKI, FINLAND 1996 – 1998 (Kari Avellan, KAREG Consulting Engineers)

Kari Avellan piti esitelmän PRINCIPAL RENOVATIONS FOR THE OLYMPIC ROWING STADIUM IN HELSINKI, FINLAND 1996 – 1998 konferensissa, Icomos Joint International Scientific Committee Meeting, 21.6. – 23.6.2007 Chicagon Teknillisessä korkeakoulussa (Illinois Institute of Technology) ja toimi kutsuttuna panelistina

Abstract

Principal renovations for the Olympic Rowing Stadium in Helsinki were realized during the years 1996 to 1998. The consulting engineering firm KAREG was responsible for all structural renovation design: structural studies of corrosion effects, carbonation and hardness tests, strengthening and repairing plans of the structures. KAREG also supervised the work at site.

Olympic Rowing Stadium

The Olympic Rowing Stadium of Helsinki was built on Taivallahti cape at the western side of the city-center during the years 1939 - 1940. The building was designed by the architect Hilding Ekelund (1893 - 1984) and structural engineer Alpo Lippa. It must be mentioned that the project's structural drawings and calculations were produced under exceptional circumstances, and with a fair degree of optimism. At the time he was designing the project, the engineer Mr. Lippa was stationed at the eastern front, a few months prior to the start of what would become known as the Finnish Winter War. The Second World War had begun in 1939 and the 1940 Helsinki Olympics were cancelled. The facilities only welcomed canoeing teams at the rescheduled Helsinki Olympic games of 1952. Built to seat approximately 2000 spectators, the features of interest of this elegant work are its long spanning canopy (overhanging roof) as well as the over-all thin concrete structures.

Photograph 1. Helsinki Olympic Rowing Stadium.

Initial steps

In 1996 the concrete testing department of the Technical Research Center of Finland (VTT) took 50 cylinder core samples throughout the roof beams. They made their condition analysis according to the compressive strength of these cores. Based on their findings two recommendations were made; either the roof structure as a whole be demolished for not being safe enough according to today`s code standards, or further studies should be implemented on how to upgrade it. Since this work of significant historical importance was already protected, the second recommendation was pursued.

After a call for biddings by the Sports Department of the City of Helsinki, KAREG was successful in securing the work mandate for the structural renovation design of the Stadium. To begin with, we proceeded with a simple analysis based on load history of the roof and on bending deformities (according to the Anastylosis principle).

- The officially imposed snowload in the revised building code was now 50 % higher than it was 1939 (from 1.20 kN/m^2 [25lb/ft^2] to 1.80 kN/m ^{2} [38lb/ft^2])

- From 1939 to 1995 the snowloading condition of 1.80 kN/m^2 was met between 5 and 10 times
- It was verified that the overhangs of the single span beams had no plastic deformation (no vertical deflection)

Scope of work

The principal repairs of the Rowing Stadium were carried out between 1996 and 1998. Our assignment consisted in providing all structural repair design including structural studies of corrosion effects, carbonation and hardness tests as well as all strengthening and repairing plans of the structures. We also had the responsibility of supervising the work at site. The most important and difficult work was to strengthen the roof overhangs. To this effect, the roof beams, especially the overhangs had to be shored against deformation. New concrete with reinforcement was poured on the old roofslab in line with the beams underneath. To get a good bond between the old and new concrete the old concrete face was machine-grinded. The whole roof was also insulated to prevent deferential movements between the old and new concrete structures. Below the stands many of the structures were strengthened with additional bars and shotcrete (gunite).

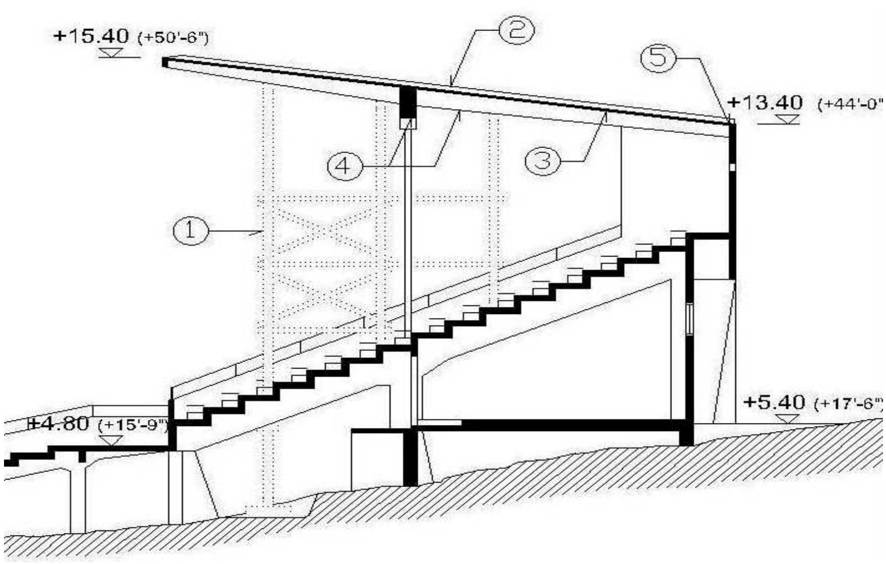


Figure 1. First phase renovation stages.

Renovation

The renovation work was realized in two phases; the first renovation phase consisted in strengthening the roof beams.

The principal steps of the first work phase were as follows (Figure. 1)

1-A temporary structure consisting of heavy timber shoring and bracing was built (Figure 2)

2- The Strengthening of the beams proceeded as follows: (Photographs 2, 3 and 4)
-Machine-grinding at a depth of 5 mm (3/16”) of the surface of the roof slab, in an area 500 mm wide (1’-7 ½”) in line with the beams underneath that are 160 mm wide (6 ¼”).
-Cleaning and preparing the bonding surface
- Reinforcement for the strengthening of the beams (Photograph 2)

- Height of Formwork 100 mm (4”) and width of 420 mm (1’-4 ½”) narrowing to 240 mm (9½”) at both ends (Photograph 3)

-Concrete pouring: The mix consisted of non-shrinking concrete EMACO S88 + 20% with natural gravel 5 to 10 mm (3/16” – 3/8”) (Photograph 4)

3- Canopy soffit
-Sand blasting
-Wet cleaning
-Shotcrete d =20 mm (3/4”) + mesh AISI 304 + ϕ 3.4 mm (1/8”) # 100 mm (4”) grid
-Repairing plaster max size 1.2 mm (3/64”), d = 5 mm (1/4”)
-Painting

4- Beams (same as in 3)

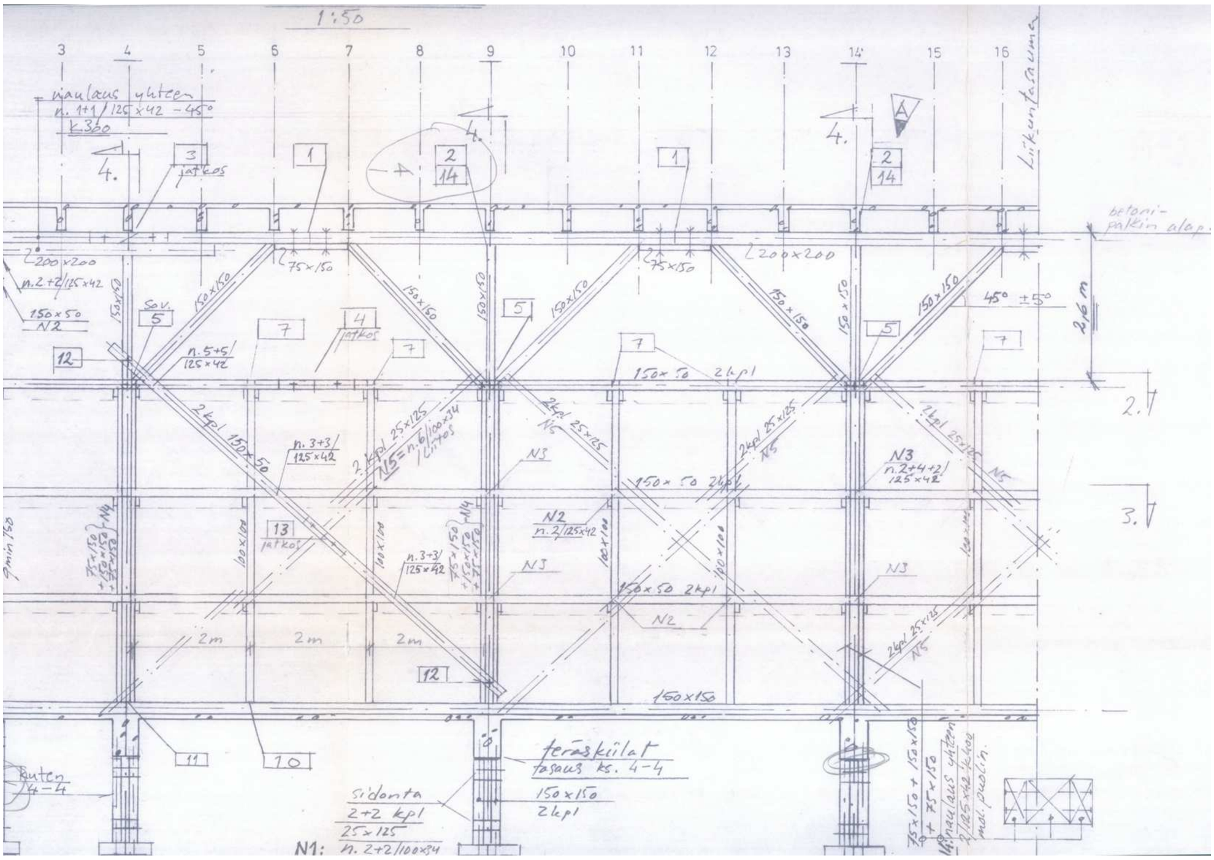
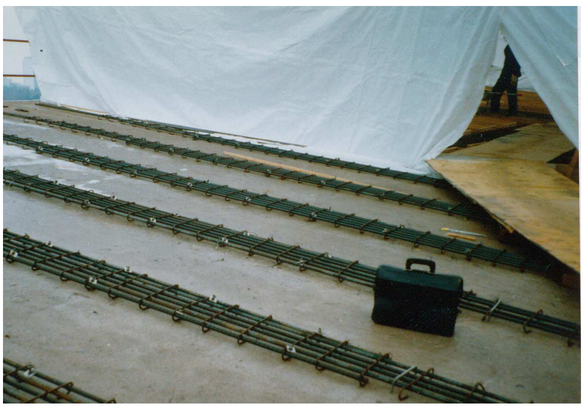


Figure 2. Heavy timber shoring of roof beams.


Photograph 2. Reinforcement ready for the upper parts of the beams and part of winter hall.

Photograph 4. Concrete pouring of the upper parts of the beams.

Photograph 3. Formwork and reinforcement of upper parts of beams. Mr. Kokko of the contractor firm Kokkoma Ltd and the writer.

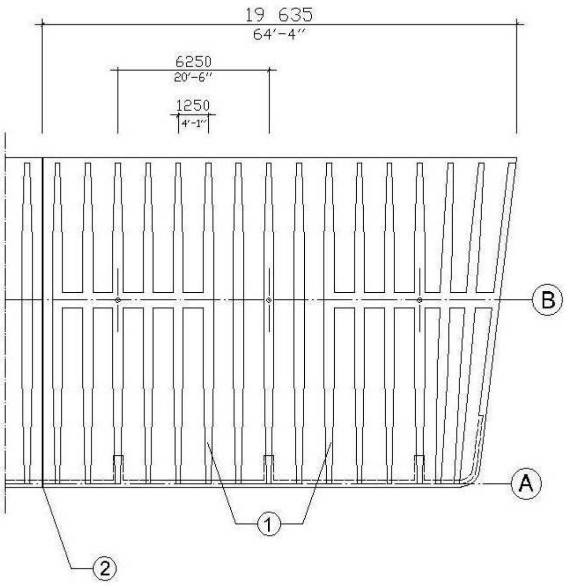


Figure 3. Partial roof plan with strengthened beams.
1) Typical strengthened beams
2) Expansion joint

Quality control for the canopy structure

Before temporary supports of the canopy could be removed, it was primordial to verify that the old and new structures were working as a monolith. For this purpose we proceeded with a sufficient number of bond strength tests. These tests were carried out by drilling free cores in the new concrete to a depth slightly exceeding the surface of the old concrete. These cores were then pulled-out and the strength measured, as well as the nature of the failure noted. Totally 40 tests were done throughout the surface of the canopy at points that the supervising engineer determined. Precise measurements done earlier allowed the boring to proceed without any damage to the reinforcement bars (Figure 5). The allowable strength of bonding was determined to be 0.8 Mpa (0.116 kip/in^2) and the minimum acceptable value 0.5 Mpa (0.0725 kip/in^2). If the result of a test was less than the minimum acceptable, the contractor proceeded with and additional tests at his own cost; this occurred only at three points. The mean value (fctb) was 1.15 Mpa (0.167 kip/in^2) and the standard deviation was +/- 0.26 Mpa (0.0377 kip/in^2) and the grading strength was 0.89 Mpa (0.129 kip/in^2).

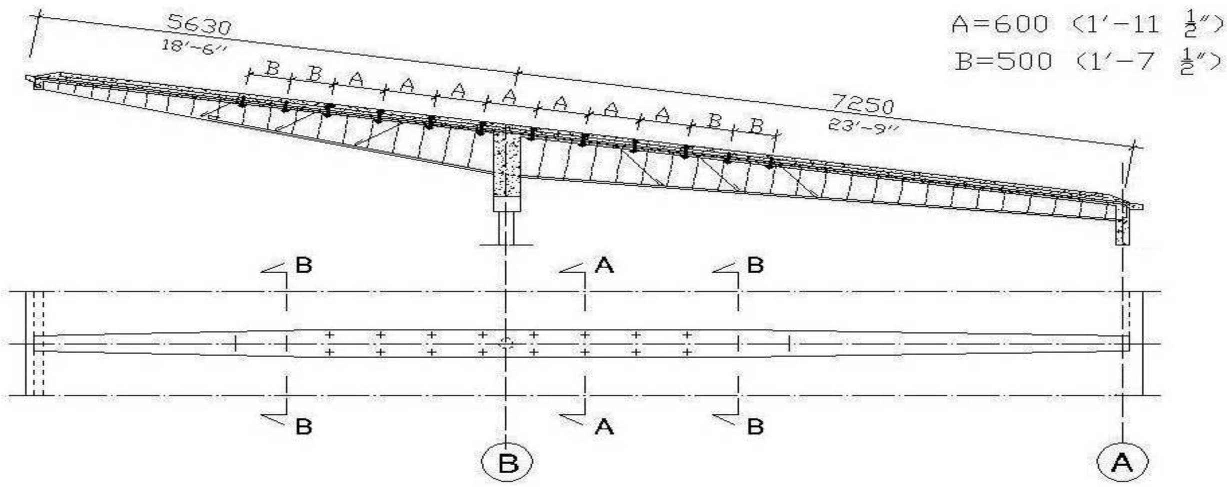


Figure 4. Typical beam with strengthening.

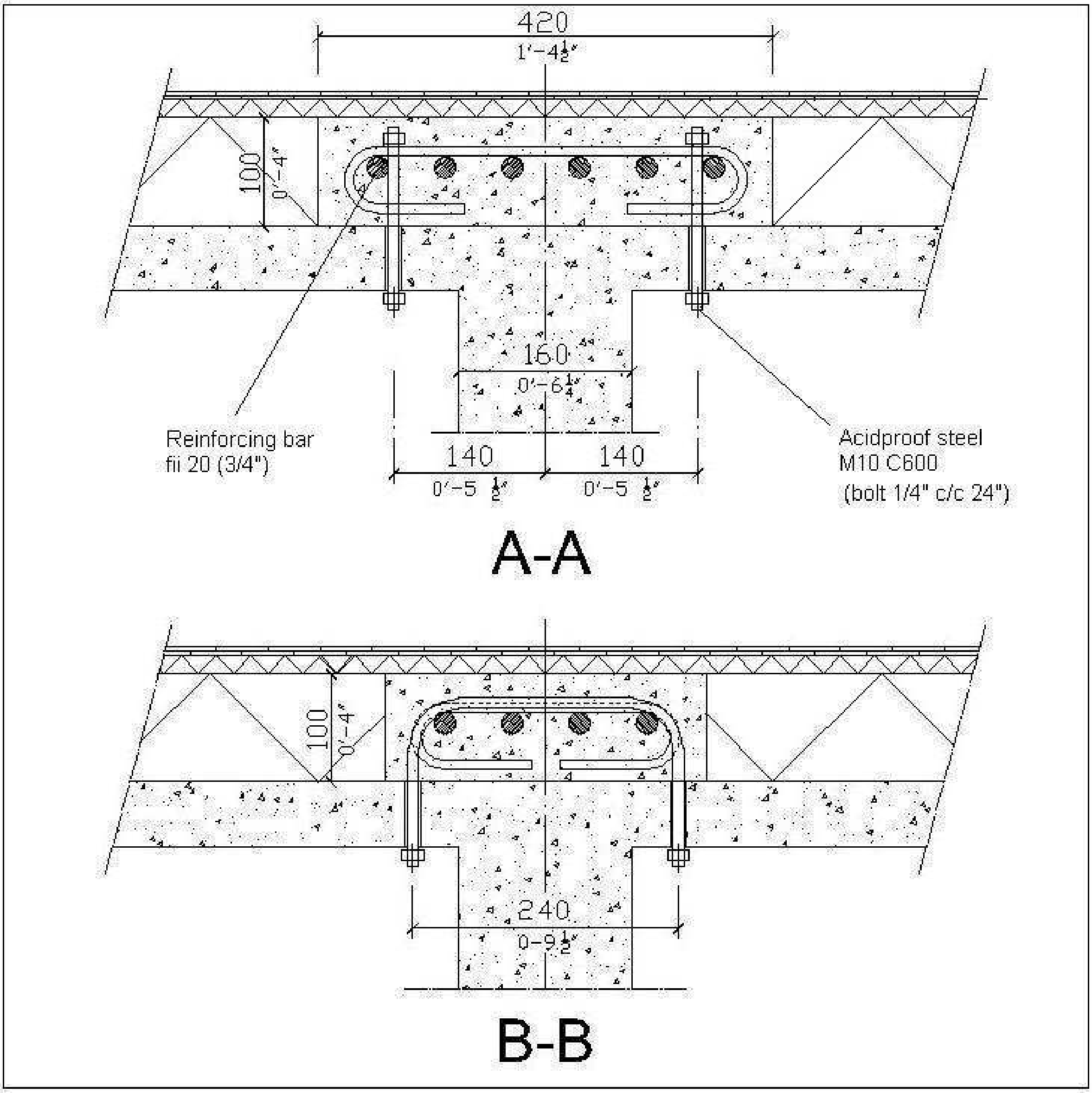


Figure 5. Detail sections of reinforcement.

Durability design for the canopy (carbonation estimation for exposed concrete)

As mentioned, the Technical Research Center of Finland (VTT) tested the strength of the canopy's concrete. The mean strength was 14.5 Mpa (2.10 kip/in^2) and the carbonation depth varied from 20 to 30 mm (3/4"to 1 1/8") depending on the strength of the old concrete. According to structural drawings made by the engineer Mr. Lippa, the concrete mix was in volumes 1 : 2 ¾ : 2 ¾ = cement : sand : coarse gravel. At that time nothing was noted about the water / cement ratio.

Since the top part of the canopy was to be insulated and protected, only the exposed concrete of the soffit had to be taken into account for the estimation of the carbonation process.

The well know formulas for carbonation estimation are:

	where	
$x = k\sqrt{t}$ and	x =	carbonation depth
$k = x/\sqrt{t}$	k =	coefficient of carbonation per year
$t = (x/k)^2$	t =	time (a)

The mean strength of the VTT test cores was measured at C 14.5 Mpa (2.10 kip/in^2). The time t was ~60 years and the water / cement ratio measured at w~0.85. Relative humidity varied between RH~65%…85%. Making estimations retrospectively, based on the above mentioned values, and using shotcrete (gunite) w equal or smaller than 0.50 (the k coefficient for shotcrete being about 2.5-3 mm, 1/8”per year), we then obtain with 20 mm (3/4”) shotcrete a carbonation time of approximately 45 years. Taking into account the repairing plaster (Φ1.2 mm [3/64”], d=5 mm [3/16”]) which had according to the manufacturer the k-value of 1.2 mm (3/64”) per year we get a carbonation time of about 15 years. Shotcrete and repairing plaster together give carbonation times of more than 60 years.

Photograph 5. Rand beam of the roof. Old bars visible.

Concrete repair

After sandblasting and in some cases mechanical hammering, the reinforcement bars of the beams became visible. The bars were first treated with special rust control mortar and then with extra shotcrete (20 mm–30 mm [3/4”-1 1/8”]) to obtain a suitable surface for standard shotcrete and repairing plaster (Photograph 5).

Conclusion

KAREG Consulting Engineers completed its mandate for the renovation of the Helsinki Olympic Rowing Stadium in 1998. The contractor, Kokkoma Ltd, who was responsible for this project was one of a few finnish special firms able to realize this demanding work.

This project offered a number of challenges which were met successfully. The Stadium's canopy now meets today’s more stringent code requirements in terms of load bearing and extended serviceability and therefore was saved from demolition. Generally, interventions were realized in the spirit of the original building with use of matching materials and techniques. The visual integrity of the Stadium’s design was preserved.