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Prolonged life for the Ullevi stadium's cable suspended roof

Alexander SEHLSTRÖM*, Lars SCHAGERSTRÖM, and Daniel EKSTRÖM

*WSP
Fabrikstorget 1, 412 50, Göteborg, SWEDEN
alexander.sehlstrom@wsp.com

Abstract

Ullevi is an outdoor stadium built for the football World Cup hosted by Sweden in 1958. A roof suspended in 28 cables hung in two concrete pylons covers part of the grandstand. Following a Bruce Springsteen concert in the late 1980s so vivid the jumping audience managed to set the arena in self-sway, the roof was strengthened to alter its eigenfrequency. After 65 years in service, some cables showed signs of decay. This paper discusses a project started in 2022 aiming to investigate the remaining cable service life. Based on original as well as 1980s drawings, a parametric model of the roof has been developed and analysed using FEM to assess the cable load effects and utilisation based on their original strength. Some cables had a high utilisation ratio while showing signs of thread breakage and corrosion, and physical testing was decided upon. In late 2023, two cables were replaced by new ones, requiring bespoke details and advanced temporary propping of the structurally non-redundant roof. Destructive testing of demounted cables and sockets showed cables are in better condition than expected. However, load test till failure led to unsatisfactory low resistance, with preliminary results indicating cable slippage due to poor socket design.

Keywords: Cable suspended roof, Stadium, Maintenance, Sustainability.

1. Introduction

Ullevi (fig. 1) was designed following an architectural competition in 1954 for the 1958 football World Cup in Sweden. Architects Jaenecke & Samuelson stood for the design, and construction works began in May 1957. The stadium was inaugurated in the spring of 1958 with a capacity of 53,000 spectators, of which 11,000 were under the roof. Following expansions completed in 1995 and 2012, Ullevi now has a concert capacity of 75,000 spectators and is Scandinavia's largest stadium for music, football, and athletic events [7].

About a fourth of the roof is suspended by 28 cables from two concrete pylons, whereas the rest of the roof consoles form the concrete frame at the back of the grandstand. After 65 years in service, some cables show signs of decay. A visual inspection in 2021 noted durability issues, e.g., peeling paint and corrosion on cables and sockets. Suggestions to perform further investigations and to apply a new paint system within two years were made [5]. The inspection also suggested an assessment of the load-bearing capacity of the roof, clarifying the possible effects of a cable rupture. Finally, the remaining service life of the cables was estimated to be 20 years, i.e., they should be replaced by 2041. Inspections in 2022 using magneto-inductive testing indicated defects on all tested cables, recommending dismantling a cable for further inspection [14]. This paper describes the load-bearing capacity assessment of the cables and the preparatory work enabling destructive testing on two cables. WSP was appointed to perform the assessment beginning in 2022 with calculations before the design of replacement cables and temporary roof supports took place in 2023. Following successful cable replacements in December 2023,

Figure 1: Ullevi as seen from the east with the pylons at the far end of the photo and the 1995 grandstand extension in the front. Photo [9].

destructive tests were performed during spring 2024, including deformation-controlled axial loading till failure.

1.1. Structural action of the cable suspended roof

When Ullevi was designed, a large set of drawings and reports [1, 2, 8] were produced. One of these were Rune Bergkvit's account for the structural action of the cable suspended roof [4]. Bergkvist begins:

> The cable-suspended roof consists of two symmetrical halves separated from each other and from the cantilevered roof by means of expansion joints. Each roof half is suspended by cables stretching from the primary beams to the top of each pylon (approx. 50 m high). All but two primary beams hang in two cables per beam, the other two in one cable per beam. In addition, the primary beams connect via a hinge to each concrete frame. The forces in the cables, which thus form different angles with the primary beams, are taken up by the primary beams as normal forces and by special compression tubes roughly perpendicular to the primary beams. To the extent the forces in the compression tubes from both sides of the pylon do not cancel each other out, tension ties from the junction between the compression tubes and the primary beam in front of the pylon to the concrete frames lying closest to the pylon resolve the imbalance.

The suspended roof consist of the following components (see elements in fig. 2 for the colour references):

- **Primary beams** (red) of rolled profiles (Dimel 38–50), c/c 12.5–14.5 m with an inclination of 6–14° to the horizontal plane.
- **Secondary beams** (blue) of rolled profiles (Dimel 22–28), c/c 5 m, supporting 15 cm thick lightweight concrete panels. The top surface of the panels sits at or just above the top flange of the primary beams. Primary and secondary beams and the lightweight concrete panels form a special plane, above which the compression tubes, tension ties, and cables lie.
- **Compression tubes** (yellow) made of tubes approx. Ø52 cm in diameter, designed roughly like pendulum columns with cast steel bearings at the ends. They connect to each other via a pin in which the primary beams are hung via a plate. At the ends of the tubes, there are welded plates to attach the cables to such that the extension of the cables and the centre lines of two meeting tubes intersect in the middle of the primary beam.
- **Tension ties** (magenta) for absorption of uneven forces in the compression tubes consist of steel plates 40x400 mm. The tension ties can be post-tensioned with the help of wedges placed in some

of the joints.

- **Cables** (black) of open spiral strand cables Ø36–57 mm with length 28–65 m. The cables are wedged at the ends at special sleeves. At the lower end, the sleeve attaches to 2 m long threaded rods, which proceed from the aforementioned plates from the compression tubes. On these rods, the cable tension can be adjusted using jacks. Into the top of the pylon, flat steel plates are cast, which are double-bent, so that a cable on either side of the pylon can be attached to each. To the extent that the forces from the two ropes to each flat plate are not equal, this can be done with welded-on U-beams as stops. Cables are designed to have at least 3,5-fold safety towards failure, compared to the 1958 characteristic loading.

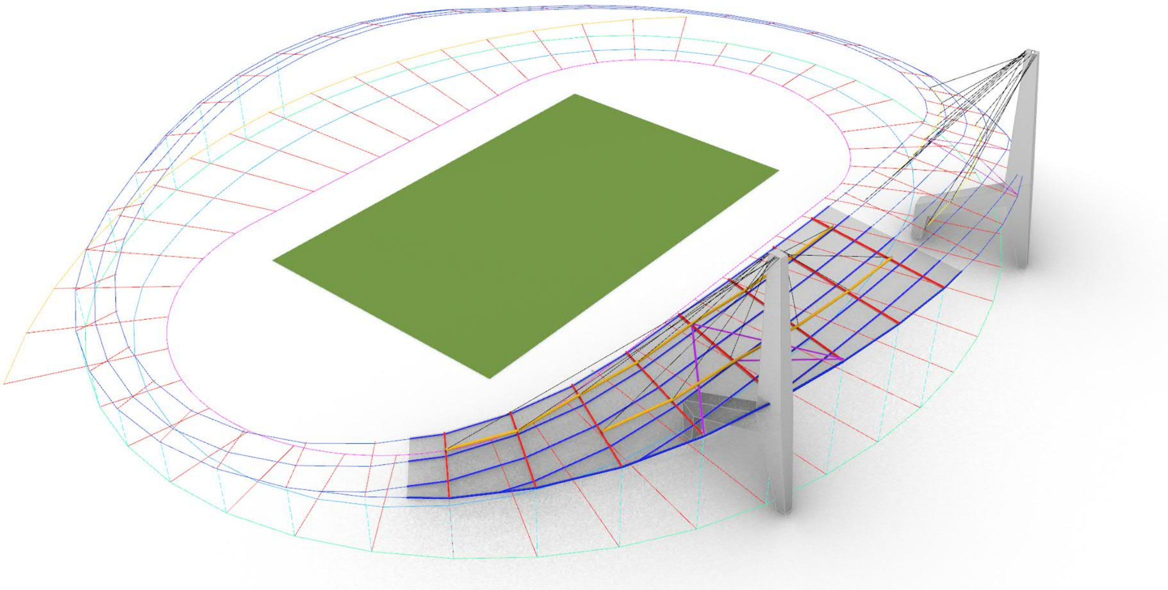


Figure 2: Parametrically generated 3D model of Ullevi with the considered suspended roof-half rendered.

1.2. Interventions following the 1985 Springsteen concert

In 1985, Bruce Springsteen held a concert at Ullevi. The atmosphere was intense, causing the audience to jump in sync with the music in such a way they managed to excite the self-sway eigenfrequency of the arena. The event put the cable-suspended roof at an immediate risk of partial or full collapse. Luckily, that never happened. However, some plates from which the primary beams hang were plastically deformed sideways due to the movement and replaced immediately. Investigations showed that the soft clay under the pitch was set in movement by the audience, which in turn caused the piled pylons to sway sideways, causing the suspended roof to move substantially. Several long-term measures were taken around the end of the 1980s to avoid similar problems in the future, such as:

- The construction of an underground garage under the pitch. The garage sits on excavated piles cast on the bedrock, reducing the possibility of setting the intermediate clay in motion by a synchronised jumping audience on the pitch.
- The installation of *counter beams* under the primary and secondary beams of the cable-suspended roof. A viscous bitumen compound sandwiched between the original beams and the counter beams damps out oscillating movements; the upper surface of the compound elongates while the bottom surface shortens during a downward beam movement and vice versa during an upward movement.
- The installation of vertical tie rods mounted to the primary beams just under each cable anchor. In the other end, the rods attaches to a damper placed on the concrete grandstand. The system

dampens longitudinal oscillations of the inclined cables between primary beams and pylon.

2. Assessment calculations

In the 1950s, hand calculations were used to verify the arena design. Most calculations are done in a 2D plane containing the element of interest, and trigonometry is used to figure out the out-of-plane force components taken care of by connecting elements. The exception was a fifteen degrees-of-freedom eigenfrequency analysis of the pylon and its fourteen connected cables, which was solved using Chalmers University of Technology's first 'electric automatic calculator,' i.e., computer, indicating an oscillation period of 2.4s [12, 2].

The original calculations have been instrumental in our understanding of the arena's structural action and load-bearing capacity. However, updating the hand calculations with new design loads and checks according to today's design codes [6] was not feasible. Instead, we decided to establish a 3D FE model for the computation of cable load effects, accounting for the non-linear geometric behaviour of the roof.

2.1. From 2D hand drawings to parametric 3D FE model

By listing relevant data from the 1950s and 1980s 2D drawings in tables and using Rhino [3], Grasshopper, C# code snippets, and additional plugins, we efficiently established a centre-line model representing the main elements (primary and secondary beams, compression tubes, tension ties, cables, and pylon) of the arena roof. A forgotten set-out drawing found during archival searches defining xy-coordinates for 83 FIX points used as references for the arena geometry was the key to succeeding with this work. One of the plugins was Strusoft's FEM-Design 21.7.0 Grasshopper API [13], enabling us to parametrically define loads, load cases and combinations, boundary conditions, cross-sections, material properties, fictitious link elements, etc., needed to establish and run our FE analysis model in FEM-Design 3D Structures 21.

2.2. Non-linear analysis and utilisation

Result accuracy of the FE model was verified using sensitivity analysis, altering, for example, assumptions on boundary conditions, cable stiffness, initial imperfections, and initial deflections. Computing vibration modes and eigenfrequencies allowed for easy identification of modelling mistakes and verification of the model behaviour.

We computed the maximum load effect on each cable using non-linear analysis accounting for second-order geometric effects. Load effects were determined according to today's code [6], stipulating, for example, twice as much snow load as the 1958 code did and accounting for the increased roof self-weight following the Springsteen interventions.

In parallel, the original—undamaged—cable capacity was determined according to Eurocode, enabling the calculation of utilisation ratios of the cables in the ultimate limit state to be within the range 58–92%, depending on the cable. The calculation accounts for cables being *key elements*, with demand for a design capacity at least 1.3 times the design load effect [6].

Thus, slightly simplified, loss of cable cross-section area of more than 8% would risk leading to insufficient load-bearing capacity.

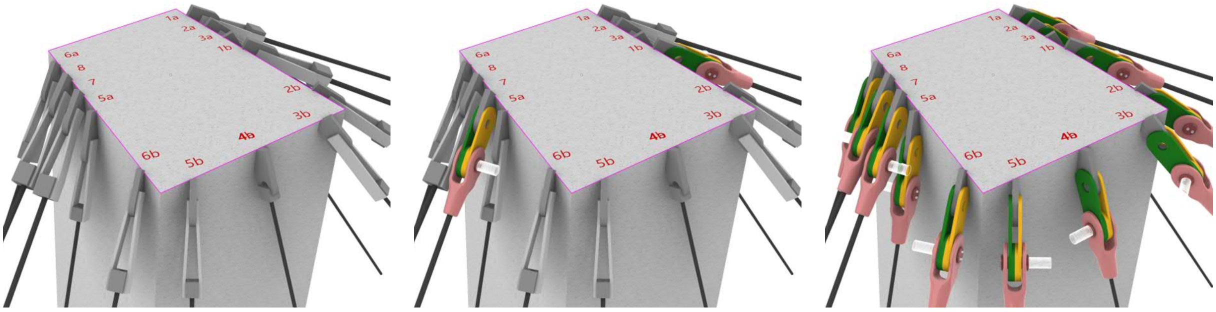
3. Replacement cables

Dismantling of two cables and destructive testing of these requires the installation of new cables in their place. As such, both new cables and sockets were needed as well as temporary props to support the roof while replacement took place.

3.1. New cables and sockets

Ullevi's original cable sockets were custom-made. To replicate these was neither seen as practically possible nor economically feasible. Instead, we chose sockets from the cable suppliers' catalogue: a fixed fork socket and an adjustable bridge socket for the top and bottom cable ends. Using full-lock coil cables, which may not be twisted around their axis, required installing the sockets at an angle to one another.

The top socket is attached using a pin to two new bespoke parallel link plates, which, in turn, are pinned to the original steel plate anchored in the pylon. Determining the link plate geometry required careful studies of the space needed to install the pins, considering both the current replacement and future replacements (fig. 3).



(a) Original top sockets. (b) Cables 1b and 7 replaced. (c) All cables replaced.

Figure 3: Top sockets fixed to the pylon at level +64 m. In (b) and (c), the red sockets are mounted at the end of the cables by the cable supplier, and the white cylinders represent the needed pin installation space. The bespoke green and yellow link plates enable the 'folding' of the details during installation, ensuring clash-free pin installation.

The bottom socket required a bespoke steel detail consisting of a solid steel block and two plates accommodating the non-compatible geometry of the new socket and the existing connection detail on the compression tubes. Welded-on shims strengthen existing holes. Figure 4 compares the original bottom socket with the new one.

(a) Original socket and threaded bars. (b) New socket, threaded bars, and steel block with forks.

Figure 4: The bottom cable ends are pinned to the compression tubes from which the primary beams hang.

3.2. Temporary props for the roof

As the suspended roof has no redundant load paths and sufficient capacity to work without a cable, cable replacement necessitates temporary roof supports. The maximum vertical design load to temporary support is about 720 kN. Two alternative approaches were considered: using (1) temporary cables between roof and pylon and (2) a prop under the roof placed on the grandstand.

The first approach would have been fairly easy to assemble, only needing a mobile crane and a few days of work. A temporary fixture for the cable around the compression tubes was investigated. Such a solution would move the point where the roof suspends from the plate extending from the primary beam to the compression tubes, which showed to have insufficient bending capacity.

The second approach took inspiration from the props used during the erection of the roof, with temporary masts consisting of cable-stiffened steel H-beams pin-jointed to the grandstand and the primary beams. Similar masts were considered. However, no suitable way to lift them from the pith up to the grandstand was found, primarily due to the limits the underground garage introduces on mobile crane weights.

Instead, we opted for scaffolding towers. To handle the vertical load, the base dimensions of the tower needed to be 4.8×4.8 m. As the grandstand consists of only 60 mm thick concrete elements with insufficient capacity to carry the tower, a load-transferring structure was placed underneath it, bringing the loads directly onto the 600–800 mm wide concrete frames under the primary roof beams. Additional diagonal studs and ties connecting the tower to the concrete frame it stands on and the two adjacent frames some 15 m away were used to handle non-vertical loads due to wind and imperfections and to reduce the tower's buckling length.

To change the load-bearing behaviour of the primary beam as little as possible, the primary beam needed to be supported by the tower below or near the intersection of the action lines of the primary beam and

Figure 5: The roof prop adaptor (unpainted) and transfer beams (red) temporarily supports the roof primary beam (gray), exerting a load of 720 kN on the scaffolding tower.

the cable. The vertical tie rods connected to the dampers mounted in 1987 were also attached at these intersection points and were temporarily detached. As the counter beams are clamped to the primary beams, their interface risk to form a hinge if put under compression. To avoid this, we developed an adaptor placed on a load-transferring system at the top of the scaffolding tower. The adaptor (fig. 5) consists of horizontal HEA beams from which two vertical HEA consoles upwards, one on either side of the primary beam. A plate extending from the top of the consoles provides via shelves welded to the web of the primary beams support for the roof.

3.3. Construction works

Temporary props were installed September–October 2023. Cable replacement was planned for November, but due to bad weather it took until late December before the works were completed. Figure 6 shows a few photos from when original cables were being released and dismantled.

Figure 6: Workers unload cable 1b using jacks (left) and a winch (right), laying the roof on the scaffolding tower underneath.

4. Cable testing

Both non-destructive and destructive testing has been performed.

After the cable disassembly, DEKRA repeated the magneto-inductive (MI) testing from 2022 [14] in 2024. MI gives spike diagrams where each spike indicates the position of a defect. For the calibration of the spike diagrams, threads of cable segments with defects according to MI diagrams have been unwinded and examined. No or insignificant corrosion inside the cable could noted. By comparing the 2022, 2024, and future MI diagrams, the idea was to establish and monitor a defect growth rate. However, comparison of spike diagrams and destructive examination have shown poor reliability, making it unsuitable for future monitoring.

The use of deformation-controlled load tests of dismounted cables and their sockets has enabled us to determine their ultimate limit capacity and failure mode (cable/socket failure) and verify their modulus of elasticity. Approximately 3.5-meter cable segments, including the original top and bottom socket, were chosen, resulting in 2×2 samples. All four samples failed due to socket slippage at approximately 70% of the original 3.5-safety factor load level. Preliminary results indicate failure due to poor socket design, with geometry deviating significantly from current design recommendations [11], possibly in combination with cable thread corrosion at the socket interface. Based on the results, the owner recently decided to start planning to replace the remaining 26 cables before summer 2027.

5. Reflections

Although the suspended roof is elegant with its strict structural logic and hierarchy, its lack of redundancy imposes several challenges that probably would make the structure impossible to construct today. Even if treating the cables and their connections as *key elements*, with demand for increased capacity, the loss of one cable would most likely lead to the failure of the suspended primary beam, causing a large roof area to collapse. If such a collapse damages the pin-bearing connection between the compression tubes so they can not transfer loads, the entire suspended roof half loses its equilibrium. That would put the entire suspended roof at risk. Thus, another structural system is needed to ensure redundant load paths and sufficient redistribution capacity. A possible solution would be to replace each suspension cable with a pair such that each cable can support the roof self-weight alone, whereas both must share the load due to downward wind and snow load. Such a configuration would allow cable replacements without needing expensive temporary props while fulfilling clauses 2.3.6 (1)–(2) in EN 1993-1-11 [10], suggesting replacement and accidental loss of a cable is accounted for during design.

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References

[1] Sven Olof Asplund. *Utlåtande beträffande det linupphängda taket: Nya Ullevi, Göteborg*. 1958.
[2] Sven Olof Asplund and Kjell Magnusson. "Svängningsberäkningar av taket över Nya Ullevi". In: *Väg- och vattenbyggaren* (5 1959), pp. 131–134.
[3] Robert McNeel & Associates. *Rhino 7*. https://www.rhino3d.com/7/. Accessed: 2024-03-11. 2020.
[4] Rune Bergkvist. *Nya Ullevi: Linhängda taket: Kortfattad redogörelse*. 1958.
[5] S. Birgersson and E. Sundbom. *Linprovning Nya Ullevi, Göteborg: Inspektion av bärlinor och infästningar*. Tech. rep. Göteborg, 2021.
[6] Boverket. *Boverket's Application of the European construction standards*. https://www.boverket.se/en/start/building-in-sweden/swedish-market/laws-and-regulations/national-regulations/construction-standards/. Accessed: 2024-03-11. 2023.
[7] Got Event. *Ullevi: Scandinavia's largest stadium is a knock-out*. https://gotevent.se/en/our-arenas/ullevi/. Accessed: 2024-03-11. 2023.
[8] Bernt Gerde. "Aktuellt bygge: Nya Ullevi". In: *Väg- och vattenbyggaren* (3 1958), pp. 64–68.
[9] Martin Olsson. Own work, CC BY-SA 3.0, https://commons.wikimedia.org/w/index.php?curid=773285. 2006.
[10] SIS. *Eurocode 3: Design of steel structures — Part 1-11: Design of structures with tension components*. Standard SS-EN 1993-1-11. Stockholm: SIS, 2006.
[11] SIS. *Terminations for steel wire ropes - Safety - Part 4: Metal and resin socketing*. Standard SS-EN 13411-4:2022. Stockholm: SIS, 2022.
[12] "STF i september". In: *Teknisk tidskrift* 37 (1958), pp. 975–976. URL: https://runeberg.org/tektid/1958/1001.html.
[13] Strusoft. *API for FEM-Design*. https://strusoft.com/software/3d-structural-analysis-software-fem-design/api/. Accessed: 2024-03-11. 2024.
[14] V. Svensson, J. Stam, and A. Laurila. *Linprovning Nya Ullevi, Göteborg*. Tech. rep. 2022.