THE TRETTEN BRIDGE COLLAPSE. How could it happen?

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1. Introduction

On the morning of 15 August 2022, Tretten Bridge in Norway collapsed and fell into the river Gudbrandsdalslågen and onto the E6 highway.

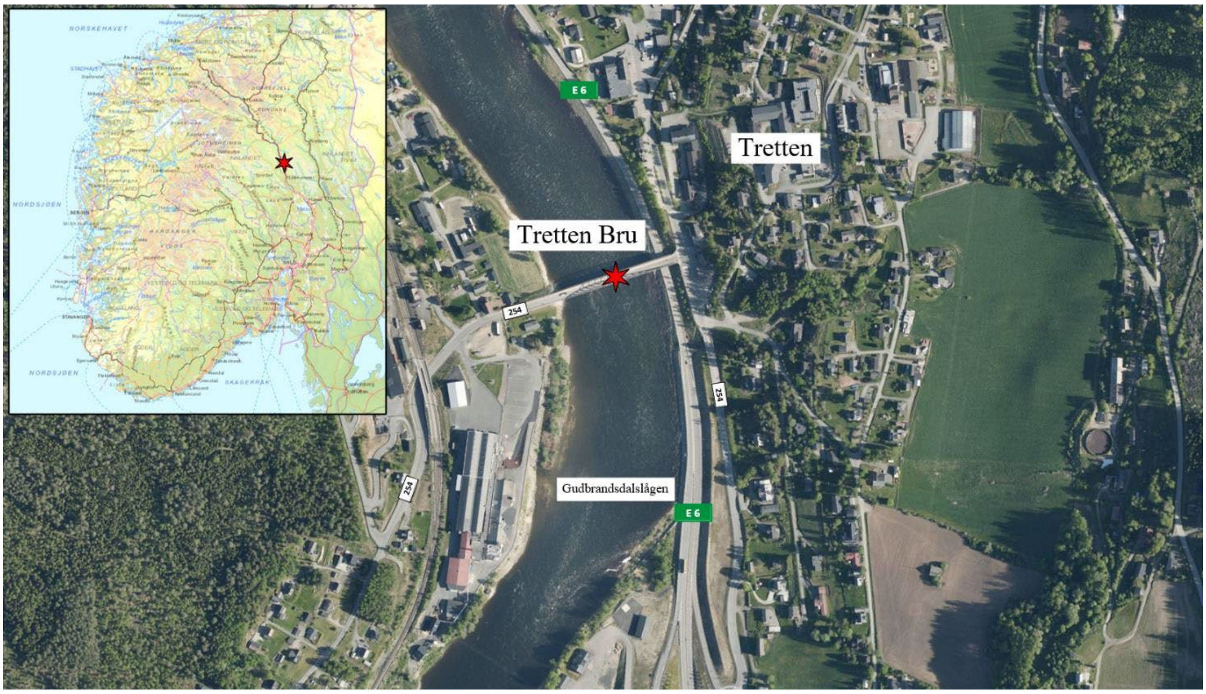

Figure 1: Location of Tretten Bridge in Norway

Not only one of the main river crossings was lost, also the European highway (E6) which is the main route to north and north-western parts of Norway, was blocked and the traffic had to be redirected through narrow roads in residential areas at Tretten. The bridge collapsed without warning, for traffic loads much smaller than the design load. Luckily no-one was seriously injured, both drivers on the bridge at the collapse were safely rescued.

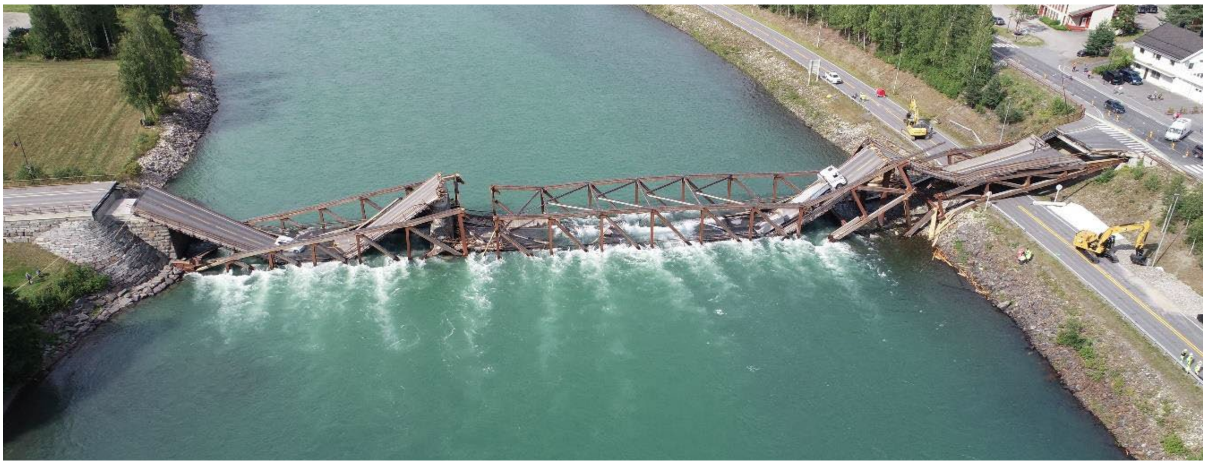

Figure 2: Bridge after collapse

The work for this new bridge at Tretten, replacing an old single lane steel truss bridge, started in 2004. The new Tretten bridge, a two-lane combined steel and timber truss bridge, was finalized in 2012.

The bridge concept may briefly be explained as U-shaped moment rigid frames in the transversal direction formed by the verticals and the crossbeams in steel. All diagonals and upper- and lower chords are in wood (glulam), except the noses for landing the truss-work at the abutments, which are in steel. The deck, resting on the crossbeams, is a stress-laminated continuous timber deck with asphalt topping. The two bridge trusses were manufactured in 6 subframes each and connected by installation of the dowel in the joints on site. Both the upper and lower chords are continuous, except at the 5 splicing locations. More details about the bridge is present in a conference paper, see ref. [1].

Figure 3: Finished bridge in 2012.

The Norwegian Safety Investigation Authority (NSIA), together with an external expert group, have carried out the technical investigations with respect to the collapse. The investigation is documented by the report [2]. The present author was a member of the external expert group.

2. Investigation

2.1. Securing evidence

Shortly after the collapse, the local police closed the actual parts of the connected roads and started to explore the situation. The traffic situation on highway E6 required removal of the bridge parts blocking the highway as soon as possible. This work was challenging due to the large dimensions of the bridge components, rough stream in the river and a difficult situation with respect to the workers HSE conditions. Based on videos of the collapsed bridge, it was decided to put the emphasis on the zones marked with circles in Figure 4. The majority of the parts within the circles were carefully collected and made available for inspections.

2.2. Disintegration at axes 3

A drawing of the U-frame in steel is shown on Figure 5. The vertical forces are transferred directly through brackets, while moments engage the bolts in the connections.

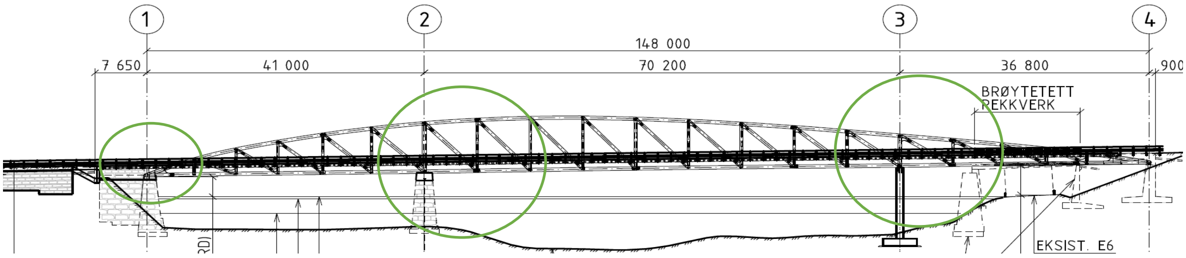

Figure 4: bridge and bridge zones with top priority (marked with circles)

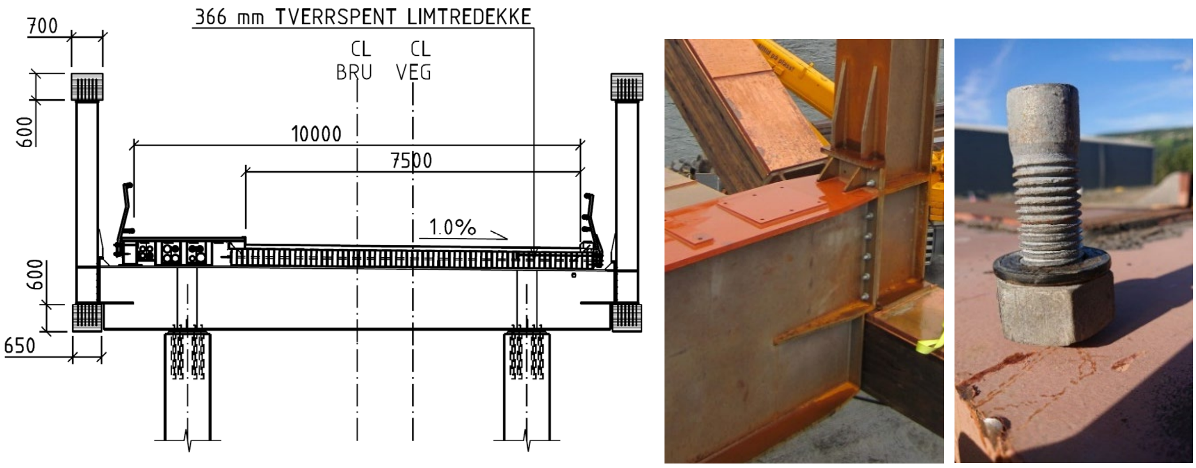

Figure 5: Left: U-frame in steel at axis 3 Right: Moment rigid connection in U-frame.Clipped bolt

As it may be observed from photo on Figure 4, the U-frame at axis 3 has lost its integrity and the connected trusses have fallen off the supporting crossbeam. This possible failure scenario requires that the bolts securing the vertical steel member to the steel crossbeam fail. An initial failure in this connection requires hence an axial force in the bolts, either causing a tensile failure in the bolts or fatigue failure due to cyclic moments. All bolts had fallen off and therefore the first rescue operation at the bridge site was to use large electro-magnetic devices to collect all possible metal parts which had ended in the river due to the collapse. In this way almost half of the bolts were found and based on the fracture zones, it was clear than all bolts were clipped off, and no trace of axial or fatigue failure was seen. It was therefore concluded that the support in axes 3 was not the initial failure location.

2.3. Structural modelling for collapse sequence scenarios

The bridge was modelled with different levels of detailing using several 3D programs in addition to a 2D program. As the behaviour of the bridge is almost symmetrical about the longitudinal centreline of the bridge, and the collapse mode also appears symmetrical, the results from 3D and 2D programs were very close. Figure 6 presents the basic modelling approach together with the axial force distribution due to the lorry on the bridge at the collapse in critical position. The lorry model was lorry no 3 in fatigue load model 4 (FLM4) in EN 1991-2, as it was quite similar in weight and load distribution. The resulting axial force distribution indicates that the most utilized zones are close to axes 2 and 3.

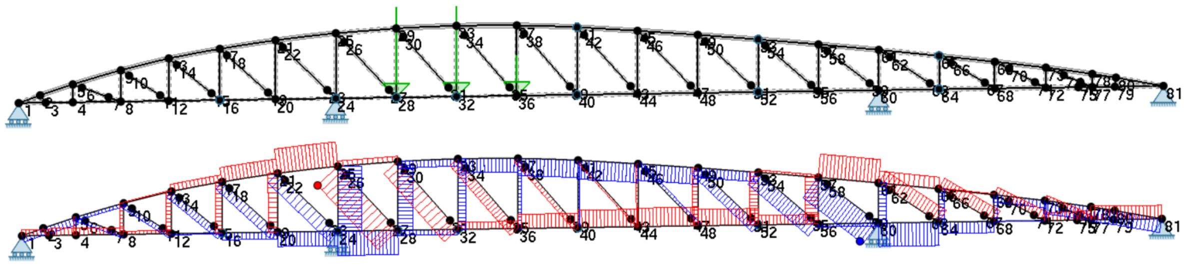

Figure 6: Numerical simulation of the loading immediately prior to collapse Upper: Vehicle on critical location for zone around axes 2 Lower: Axial force distribution, resulting critical zones close to axes 2 and 3

It was discovered that the deck structure (continuous stress laminated timber deck) and the support columns at axis 3 had a significant role in the collapse, so the structural FE models were altered to accommodate for a more realistic behaviour when large deformations occur in the simulations. The left part of the truss between axes 1 and 2 behaves like a rigid body during the collapse (see photo on Figure 4) and the force distribution within this part becomes insignificant. The lower chord here was replaced by the deck structure, and the deck was allowed to detach from the line of the lower chord. The roller support at axis 3 was replaced by a steel column fixed to the joint in axis 3. The self-weight and traffic load on the structure were kept constant. The modified static FE model and the resulting force distribution in undamaged state are shown in Figure 7.

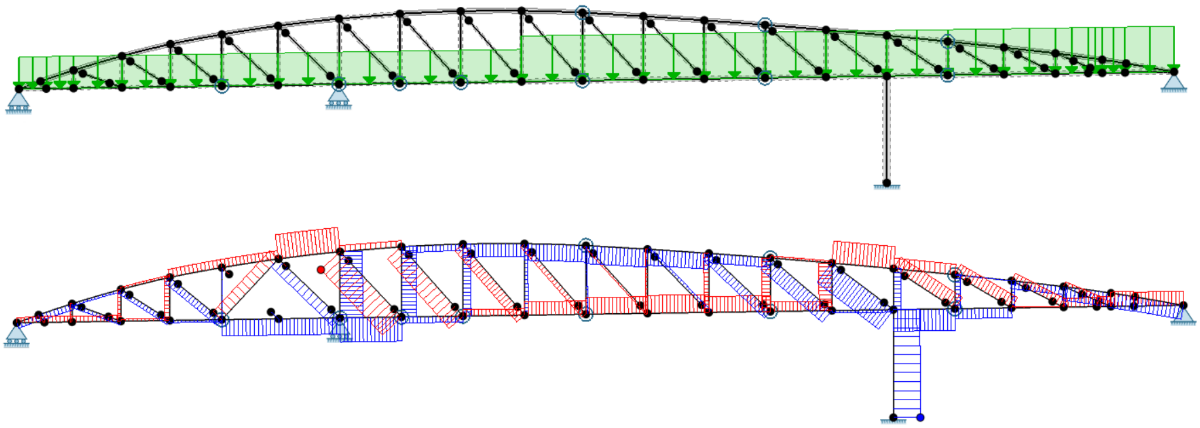

Figure 7: Modified FE model for progressive collapse

These modifications did not alter the force distribution in the elastic small deformation regime, but they allowed to follow the collapse using large deformation computations. The 2d static analyses program (Fap2D) used in this paper takes into account large geometrical deformations, but no material nonlinearities, so the degradation in strength was handled by lowering the stiffness moduli and removal of elements according to the strength estimations.

Several initial locations were explored, but it turned out that only failure initiation in diagonals 6 or 7 were consistent with the observed damages from the collapse, and had the potential to propagate the failure into a full collapse for the given load scenario with a single lorry of 490 kN on the bridge. However, an initial failure in diagonal 7 did not give immediate failure propagation, but merely additional damage to the neighbouring elements like diagonal 6. When a failure in diagonal 6 occurs, the failure will propagate. The potential elements for failure initiation are visualized on Figure 8.

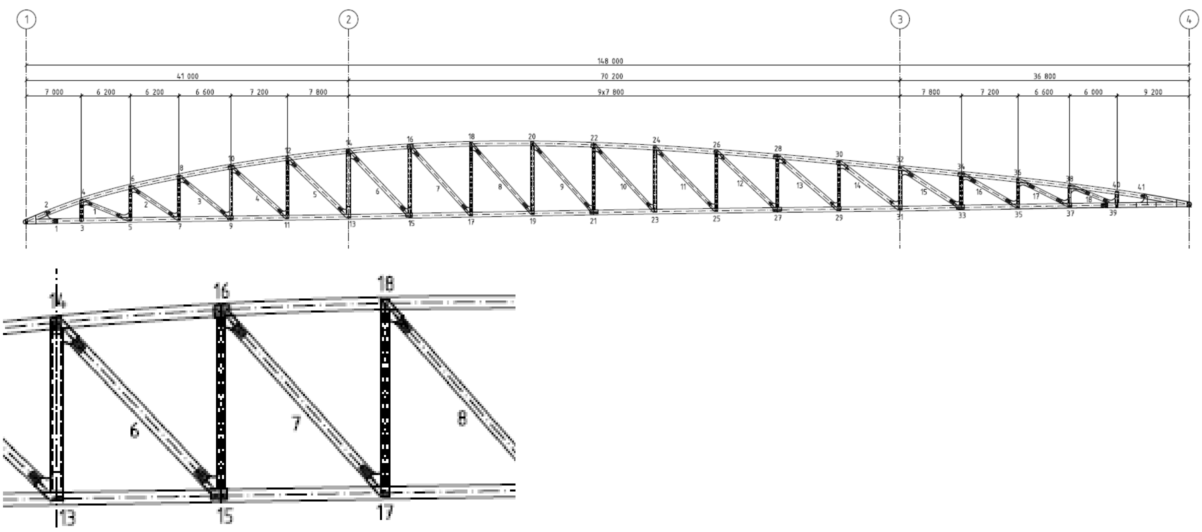

Figure 8: Potential elements for initial failure, diagonal 6 and 7

The resulting deformations from the numerical simulations of the collapse propagation are shown on Figure 9.

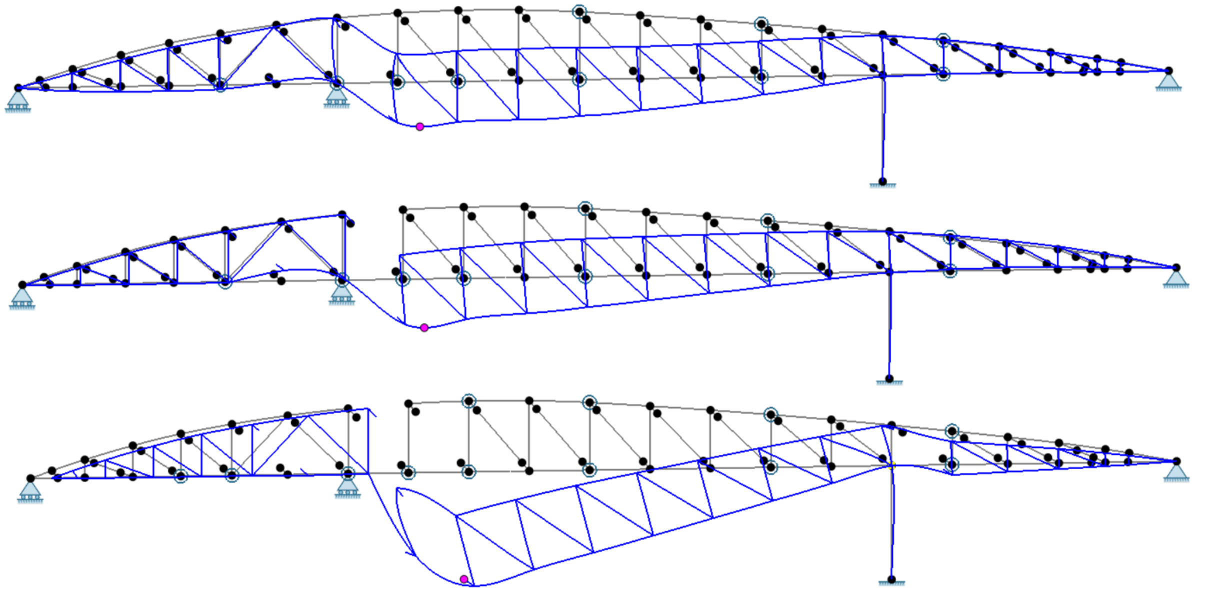

Figure 9: Progressive collapse after initiation in diagonals 6 or 7

The failure sequence is summarised as follows and visualized on Figure 10.

- The failure is initiated in diagonals 6 or 7.
- Left part axes 1-2 moves like a rigid body, the deck acts as a membran and pulls off the cross beams from the supports at axes 1 and 2.
- The splice and diagonals to the right of axis 3 become overloaded.
- The bolts at the crossbeam in the U-frame in axis 3 are clipped by the rotational movement of the center part of the bridge and trusses fall.

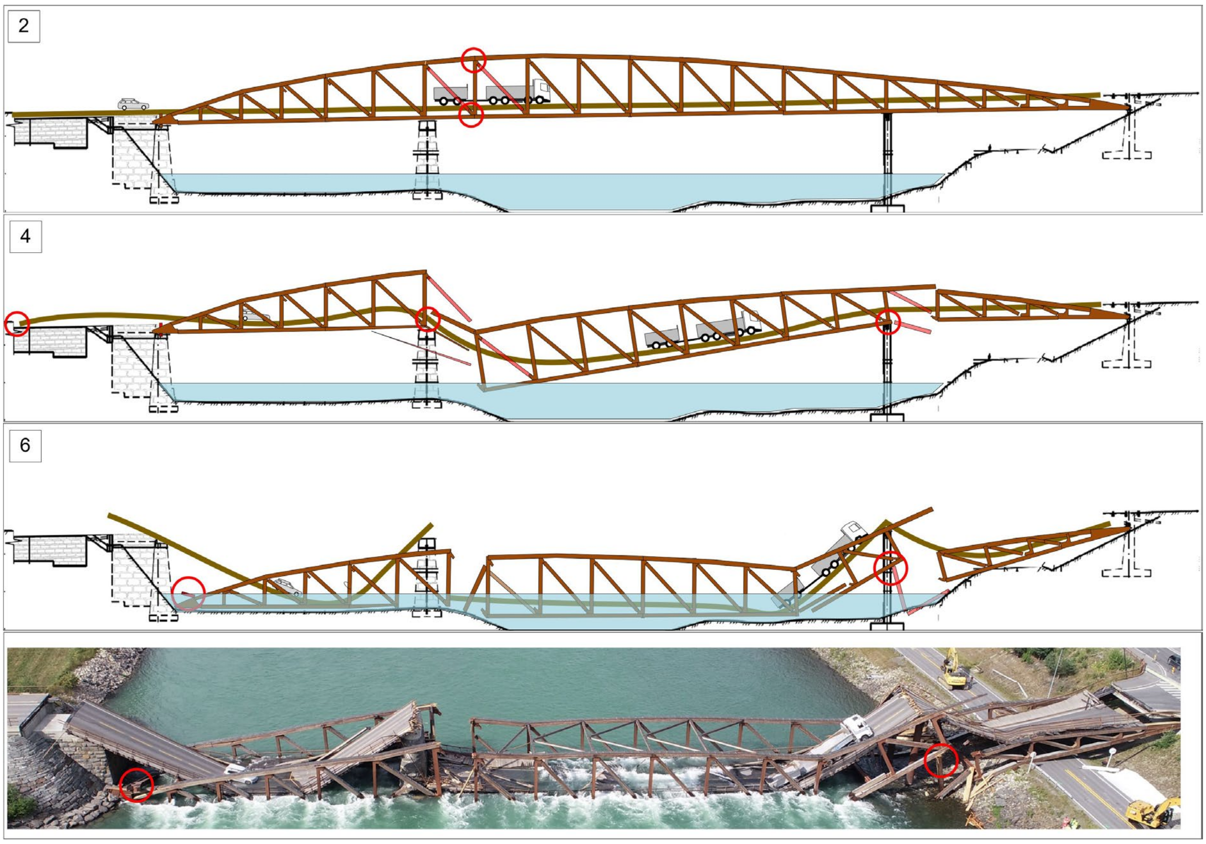

Figure 10: Visualization of the stages of progressive collapse

3. Failure mode and strength

The diagonals 6A, 6B, 7A and 7B all had large tensile forces (A = upstream truss, B=downstream truss). The moments at the joints of the trusses were not large but might have effect on the strength of the connections. During the progressive collapse the moments in some joints became large and led to moment failure. Most of the failed joints in diagonals 6A, 6B, 7A and 7B showed variants of block (shear) pull-out failure, consider Figure 11.

Figure 11: Block (shear) pull-out failure joint 14B in diagonal 6B

The material in the diagonals was GL30c, i.e. the inner lamellas (T14.5) had lower strength than the outer lamellas (T22). The dowel connections were located in the inner lamellas and therefore T14.5 strength was the actual strength for verification and calculations.

Diagonal 13B had minor damages and was rescued, and from this diagonal several tests on the material and connection strength were made. The objectives of the testing were to evaluate the mean tensile strength as well as to determine the connection strength of the diagonals 6 and 7.

Figure 12: Connection test leading to block pull-out failure

The tests confirmed that block (shear) pull-out was the lowest failure mode, and that the small head tensile area was critical together with the tensile strength. The evaluation of the mean tensile strength gave the following:

- Uniaxial tension tests: >19.6 Mpa (shear failure at fixture)
- Combination of compression and 4-points bending:~32 Mpa
- Block pullout tests and net head tensile area:~33 Mpa
- From testing of sorting procedures for T14.5:~29 Mpa

Based on the experimental results and observations of failure parts, no indications of wood material decay or lack of fulfilment of the specified material properties were found. As the statistical basis for the results from sorting procedures was much broader, it was decided to use 29 Mpa as representative mean tensile strength for T14.5 lamellas in the computations to evaluate the cause of the collapse. The strength of the connections was then determined to be:

- Diagonal 6: 2630 kN
- Diagonal 7: 1890 kN

These strengths are consistent with those in modern timber regulations like [4, 5], but with mean values and all factors related to statistical distribution and safety removed.

4. Cause of failure

The sequence of the collapse from the physical evidence and the numerical simulations showed no contradictions and hence the failure probably started in one of the diagonals 6A, 6B, 7A and 7B. The mean tensile strength of the material of these diagonals was estimated by tests and calculations, and based on this knowledge, the loading scenarios which might lead to the failure were investigated. In short there are only two possibilities, either overloading by too many or too heavy vehicles on the bridge simultaneously, or damage accumulated over time leading to reduced strength.

4.1. Extreme loading

The bridge collapsed with an ordinary truck passing the bridge in direction axes 1 to 4 (load scenario 1). The weight of the truck was about 50 ton or 490 kN. This loading is far less than the design load. The following loading scenarios for overloading were computed by the numerical FE models, both in 2D and 3D and the corresponding forces in the diagonals were compared to the mean strength.

1. Weight of truck at collapse: 50 ton (500 kN)
   Force diagonal 6: 1776 kN (Capacity =2630 kN)
   Force diagonal 7: 1400 kN (Capacity =1890 kN)
2. Largest special vehicle in 2015: 97 ton (970 kN)
   Force diagonal 6: 1969 kN (Capacity =2630 kN)
   Force diagonal 7: 1516 kN (Capacity =1890 kN)
3. Two 50 tons trucks are meeting on worst locations, or two 50 tons trucks are in a row in the same lane, or whole midspan filled with 25 kN/m in one lane (timber transport 60t per vehicle in a row without separation) give maximum forces:
   Force diagonal 6: 2215 kN (Capacity =2630 kN)
   Force diagonal 7: 1561 kN (Capacity =1890 kN)

From the above loading scenarios on the bridge it is not likely that the bridge becomes overloaded by too much simultaneous traffic. From comparison of the special transport in 2015 (load scenario 2) to the load on the bridge at collapse (load scenario 1), it is obvious that the damage was not created by the truck on the bridge at collapse, but it must have been present in advance. No damage was reported after load scenario 2 in 2015, and no serious damage has been reported after the regular inspections of the bridge.
There is a traffic station which register the traffic passing the bridge, and only a regular traffic pattern has been recorded in the time before the collapse. There is therefore no trace indicating that overloading was the direct cause of the bridge collapse.

4.2. Fatigue loading

An overview of the heavy traffic on the bridge was put together based on the traffic registration and on the regular traffic to and from the dairy plant of Tine. Tine is not included in the traffic registration due to their entrance being located just after the bridge abutment in ax1s 1, which is just before the traffic registration location. The overview is an estimation of the heavy traffic which has passed the bridge during its lifetime of ten years, and is given in Table 1.

Table 1: Heavy traffic during 10 years

Source	Length [m]	Load [ton]	Passes per day	Comment on load	Total no of passes
Tine	12.5 - 16	50	20	(6 d pr week) full load	62400

Tine	12.5 - 16	20	20	(6 d pr week) empty	62400
Traffic registr	7.6 – 12.5	30	35	full load	127750
Traffic registr	7.6 – 12.5	15	35	Empty	127750
Traffic registr	12.5 - 16	50	5	Full	18250
Traffic registr	12.5 - 16	20	5	Empty	18250
Traffic registr	16 – 24	60	7.5	Full	27375
Traffic registr	16 – 24	25	7.5	Empty	27375
Specialtransp		80		specified	8

It was judged that lorry number 3 specified in fatigue load model 4 (FML4) in EN 1991-2 [3] was quite representative with respect to load and load distribution of much of the traffic, so FML4-lorry 3 was used throughout the fatigue computations. The loading was then sorted after weight in load cases presented in Table 2, where load train 1 is in the lane giving most stresses in the diagonals 6B and 7B (eccentric lanes).

Table 2: Load cases for fatigue evaluation

Load case i	**Weight (ton)**	**No of passes**	**Scale factor Load train 1**	**Scale factor Load train 2**
1 (Selfweight)		1	0	0
2	15	127750	0.3061	0
3	20	244900	0.4082	0
4	25	273375	0.5102	0
5	30	127750	0.6122	0
6	50	80650	1.0204	0
7	60	27375	1.2245	0
8 (special)	80	8	1.6327	0
9 (meeting)	50 + 30	2357	1.0204	0.6122

The fatigue computations considered all failure modes in the connections and most of the connections showed no vulnerability to fatigue damage. However, some of the connections were extremely exposed to fatigue damage, especially the connection in diagonal 7, which has passed its lifetime to an extreme degree with a Miner-Palmgren sum of more than 200. The next connection is the one in diagonal 6 with Miner-Palmgren sum more than 9, hence this diagonal was also well above its expected lifetime. Also, connections in diagonal 16 close to axis 3 showed too much fatigue damage. The fatigue damage were all due to tensile block (shear) pull-out failure mode.

Table 3: Fatigue results for damaged parts in Tretten Bridge

Load case-> Component	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**Sum all load cases**
Diagonal 6			0.07	0.14	3.54	4.29	0.010	1.16	9.21
Diagonal 7	0.01	0.35	2.27	3.97	82.94	104	0.29	30.62	224
Diagonal 8					0.12	0.13		0.03	0.28
Diagonal 9					0.02	0.02			0.05
Diagonal 15					0.16	0.20		0.06	0.43
Diagonal 16			0.02	0.03	0.68	0.94		0.28	1.95
Joint 21 right					0.03	0.03		0.01	0.08
Joint 27 left					0.05	0.06		0.02	0.13

5. Conclusion

Tretten bridge had reduced strength prior to the collapse. There was no trace of traffic overload and the most likely cause of the collapse is damage accumulated over time i.e. fatigue. The failure mode was block (shear) pull-out failure. The bridge was designed according to regulations [7] from the national road authorities and the Norwegian national standard for timber structures at that time (NS 3470-1) [8], which did not have any regulations regarding block (shear) pull-out failure. Diagonal 7 was much more exposed to fatigue than diagonal 6, and most probably failure initiation took place in diagonal 7. However, also a failure in diagonal 6 was necessary for propagation. Wood has large variability in strength, and by assuming that diagonal 7 is 20% stronger than diagonal 6, the order of failure initiation may be switched between diagonals 6 and 7. The fatigue is caused by too high stress concentrations in the head tensile area resulting from narrow dowel groups.

6. References

[1] Hårstad-Evjen, J.A. et al: Tretten Bridge – Timber and Steel in Harmony. International Conference on Timber Bridges 2013- Las Vegas, Nevada USA, https://intrans.iastate.edu/app/uploads/sites/12/2019/03/ID_100_Harstad-Evjen.pdf.

[2] Statens havarikommisjon: (NSIA):Delrapport 1 Kollapsen av Tretten bru i Øyer kommune 15. august 2022, RAPPORT VEI 2023/03, August 2023, havarikommisjonen.no.

[3] CEN European Committee for Standardization, EN 1991-2 (2010), Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges.

[4] CEN European Committee for Standardization, EN 1995-1-1 (2004), Eurocode 5 Design of timber structures – Part 1-1: General – Common rules and common rules for buildings.

[5] CEN European Committee for Standardization, prEN 1995-1-1 (2023), v2023-04-19 N1729. Eurocode 5 Design of timber structures – Part 1-1: General – Common rules and common rules for buildings.

[6] CEN European Committee for Standardization (2004) Eurocode 5 – Design of timber structures – Part 2: Bridges, Annex A, EN 1995-2: 2004.

[7] Statens vegvesen, Bruprosjektering. Håndbok 185. Oktober 2009. Norwegian Public Road Authorities.

[8] NS 3470-1, Prosjektering av trekonstruksjoner. 5. utgave juli 1999, Norwegian Standards.