1. Introduction, failure background, aims of the investigation

On November 25th/26th, 2005 strong south-west wind with storm force (8 Bft. $\approx$ 18 m/s) and heavy snowfall at a temperature of approx. 0 °C were predominant in the region Münsterland, Germany. Due to these weather conditions wet snow rolls formed around the conductors of several overhead transmission lines. This led to excessive line loads of approx. 5 kg/m, Fig. 1. Some of the covered conductors were sagging to the ground. Eighty-two tension and suspension towers of five different, 110 kV overhead electrical lines collapsed mostly by buckling, e.g. failed catastrophically, Fig. 2. As a consequence nearly

Fig. 1. Wet snow roll with clear ice cover, diameter approx. 13 cm, line load approx. 5 kg/m. Source RWE.

Fig. 2. Collapsed deviation tension tower M65 and suspension tower M66 (appearing small in background, right). Source RWE.

250.000 people have been cut off from electrical power supply for several days. The BAM Federal Institute for Materials Research and Testing was assigned by the German Federal Network Agency (BNetzA) for the conduction of a forensic failure analysis and an assessment of possible failure causes. The task was to find out if there were other causes besides the weather conditions. Following topics had to be investigated in detail:

- Materials conditions of the specimens taken from the towers.
- Possible ageing of the transmission towers' material.
- Description of failure mechanism(s).
- Code fulfilment at the time of erection or failure.
- Sufficiency of load assumptions, in particular the assigned ice loading zone, given in former or present (time of failure) codes.

Putting it differently the aims of the investigation described in this paper were to identify the loading situation present during the disaster, which of the towers collapsed first, and which substructure in these towers failed first and if embrittlement of the steel used was one of the failure causes. Furthermore, one crucial task was to give recommendations for the prevention of similar disasters. Based on these recommendations the authority BNetzA issued new requirements [1]. Contrary to other publications such as [2] or [3], the aim of this investigation was not to predict the failure under given loads by numerical calculations. The intention was a proof (combined analytical–experimental) of an evident failure by derivation of the corresponding loading (from picture information) and determination of the reduced load carrying capacity of structural parts.

Brittle fractures are not characteristic for this type of mild structural steel. Pohlmann [4] described brittle fractures of diagonal members and corner rods he found analysing in service failures of transmission towers in Europe. Helms et al. [5] investigated the failure of a transmission tower of the german railways and found out, that embrittlement is localised around the stamped holes of steel profiles made of Thomas steel, compare Sections 2.3 and 2.4.

Latest publications regarding failure analyses of transmission towers describe the simulation techniques used for structural failure prediction in order to avoid expensive full scale tests of transmission lattice towers [2,3] or transmission poles [6]. Moon et al. [7] carried out sub-assemblage test of a half-scaled transmission tower to estimate its performance against wind loads and compared the experimental results with those of numerical analysis. Forensic analyses like those on failures of wind turbine towers [8,9] have not been published for transmission tower failures until now.

2. Failure investigation and results

2.1. Failure site inspection

During a visit of the failure site in the western region of Münsterland eleven of the failed electricity towers were inspected on December 20th in 2005, Fig. 3. Signs of surface corrosion were not found. A corrosion induced weakening of the structures could be excluded. Fifteen samples were taken from four of the failed transmission towers: broken diagonal members, corner rod connections and more components. The collapsed transmission tower named M65, Fig. 4, was in focus of the failure investigation based on its function as deviation tension tower, the brittle fracture of a component found there, its condition and accessibility. It was assumed, that the collapse of M65 as deviation tension tower is responsible for the failure of the supporting towers M66...M73. Moreover, an extensive photographic documentation shortly after failure has been available

Fig. 3. Failure site inspection, transmission tower M65 layed down for inspection. Searching for ruptures with brittle fracture surface.

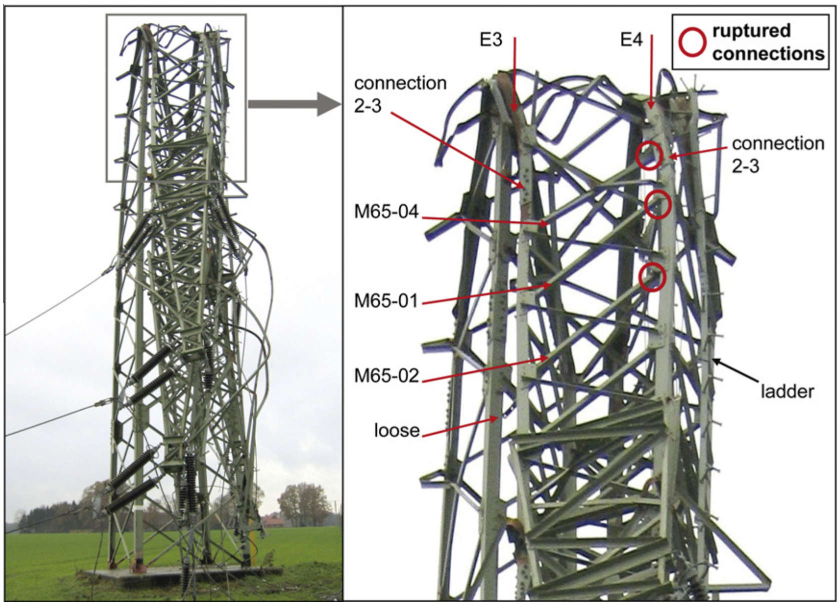

Fig. 4. Failure site inspection, transmission tower M65 in as failed position (left), Source RWE. Corner rods E3 and E4. Diagonal members M65-..., ruptures are marked (right).

for this tower, its neighbours and some more towers of this transmission line BL1503. These photos showed snow/ice rolls on some of the cables, Fig. 1, and the ice load situation at towers that did not fail. Later, more components were taken from tower M65 to be used for comparative component testing. Those were selected to be an original component, unbent and without visible damage.

2.2. Materials investigations, fractography

Materials investigations revealed that the corner rods and diagonal members of towers, that were built in the 1960's or earlier, were made of Thomas steel. The corresponding microstructure of the steel profiles close to the stamped holes showed signs of ageing in form of very small plate-like nitrides of type Fe_8N to some extent, Fig. 5. These nitrides can be etched applying Fry's reagent to the microsections. The nitrides can only be viewed in full detail using a SEM at high magnifications.

A fractographical analysis of the fracture surfaces showed that all fractures were forced fractures. Overall, no signs for fatigue failure or for corrosion induced cracking were found. The diagonal members mainly failed in a predominant brittle manner. Figs. 6 and 7 show the fracture surface of M65-4 before and after cleaning, respectively. For removing the rust warm citric acid was used at low concentration. After cleaning, the details of the fracture surfaces were analysed in the SEM. The SEM image from the centre region of the M65-4 fracture surface shows intergranular fracture, Fig. 8, whereas the image from the edges shows ductile fracture, Fig. 9. The cleaning process has generated etching pits all over the fracture surface which is the price for removing most of the rust.

2.3. Thomas steel

For a better understanding of the relations between Thomas steel and ageing as a feature of physical metallurgy some brief basic explanations are given in the following paragraphs. A recent systematic review of terms, metallurgical

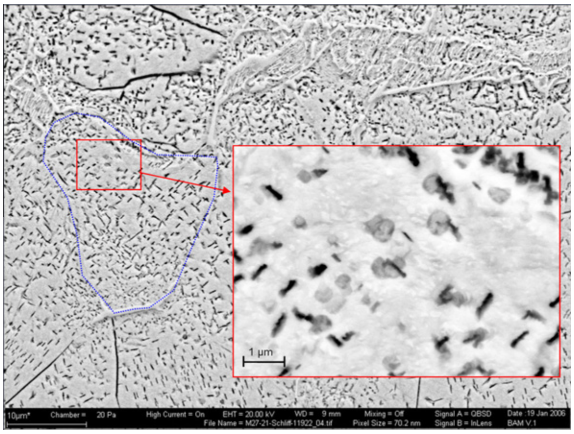

Fig. 5. Microsection of specimen M27-21, etched (Fry's reagent), SEM-micrograph: ferrite grains with nitride-precipitates. The dotted line marks a ferrite grain whose (001)-plane is perpendicular to the section plane.

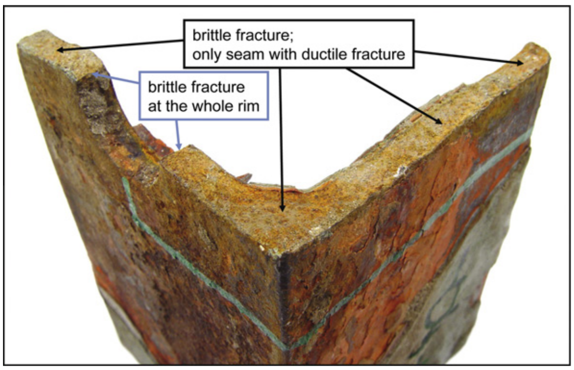

Fig. 6. Original fracture surface of diagonal member M65-04, L-profile $60 \times 60 \times 6$ mm^3, material: plain carbon steel St 37.12 (1960), Thomas steel, mainly brittle fracture with only a few ductile areas.

Fig. 7. Original fracture surface M65-04 after cleaning. Most of the rust was removed.

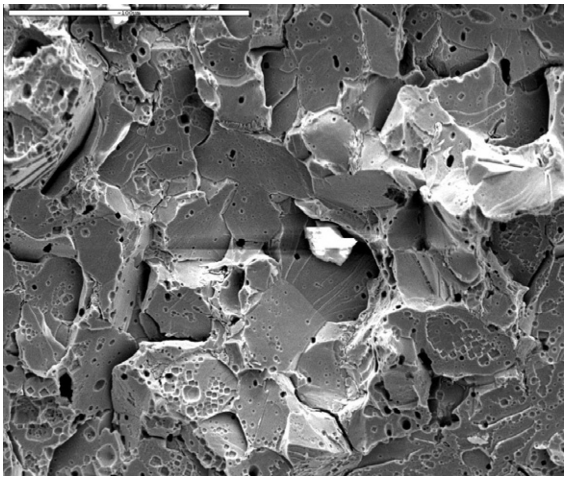

Fig. 8. Magnification of fracture surface M65-04, cp. Fig. 7, intergranular fracture with etching pits. SEM, SE mode.

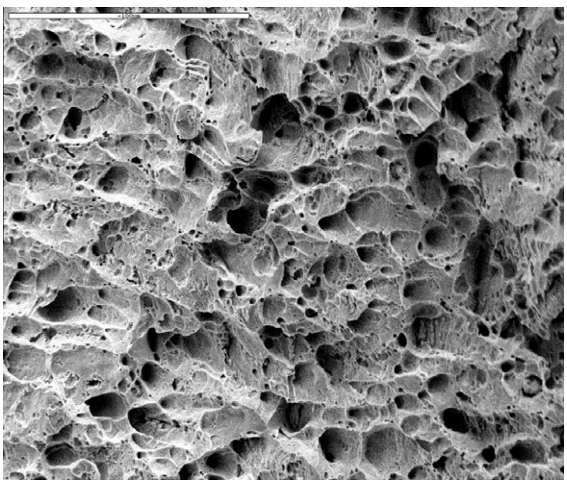

Fig. 9. Fracture surface M65-04, ductile fracture with etching pits at the seams of the fracture surface. SEM, SE mode.

background and mechanisms of ageing as well as its impact on mechanical and technological properties of Thomas steel is given in [10].

In 1878 Sidney G. Thomas and Percy G. Gilchrist introduced a method for steel making which is suitable to use iron ores rich in Phosphorus. The method is based on a basic lining of the Bessemer converter and air blasting from the bottom of the converter. Later on the method was called Thomas process and all steels produced by it Thomas steels. Over the decades, a number of technological modifications of the Thomas process had been successfully invented to improve the properties of Thomas steels. It is important to note that Thomas steels had been used very successfully and gained a remarkable economical importance worldwide. In Germany, where solely until 1943 about 300 million tons of Thomas steels had been molten [11], the production had remained over a period of about 100 years until the late 1970s. At that time the steel making by Thomas process was ceased due to technical and economical aspects.

From a metallurgical point of view, Thomas steels are typically characterized by elevated contents of Phosphorus (due to the iron ore) and Nitrogen (due to air blasting) and they are comparably low in carbon, Table 1. The nitrogen content >0.01 wt.% makes Thomas steels susceptible to ageing.

2.4. Ageing of Thomas steel

Ageing is a time dependent alteration of mechanical and technological steel properties whereas two basic mechanisms are distinguished. Age hardening (quench-ageing, natural ageing) depicts the blockage of dislocations by interstitial atoms and the precipitation of embrittling nitrides and carbides in a supersaturated-iron solid solution. Ageing after a mechanical deformation is denoted strain ageing. Within strain ageing the migration of interstitial atoms to dislocations and their blockage is the predominant mechanism due to the high number of dislocations present. The blockage of dislocations by interstitial atoms as well as the complication of dislocation movement by precipitations reduce ductility and toughness of the material. Basically, both interstitial elements, nitrogen as well as carbon, take part in diffusion processes within ageing. But the major role can be attributed to nitrogen due to its higher solubility in the iron lattice and its higher diffusion rate [12–16].

Embrittlement as result of ageing may become a prime issue with respect to the assessment of the loading behaviour and safety of Thomas steel components. Outstanding examples are cut edges and stamped holes in sheet metal, plates and profiles of screwed or riveted steel structures. These material regions are exposed to significant plastic deformation which acts as prerequisite of strain ageing. In combination with a steel grade susceptible to ageing embrittlement (like Thomas steels) as well as structural stress concentration, higher stress triaxiality due to component thickness and loading features like low temperature and/or higher loading rate brittle fractures can be caused in service. Details of a corresponding failure analysis on overhead transmission line towers of Deutsche Bundesbahn are reported in [5].

2.5. Mechanical testing

In order to find out the type of steel used, standard specimens were prepared from components from tower M65, see Section 2.1, and mainly used for tensile and Charpy tests. Tensile tests were performed in accordance to DIN EN 10002-1 [17]. Tensile strength and elongation of all tested standard specimens ($R_m \sim 400$ MPa, $A \sim 37\%$) were within the tolerances of materials standards for mild steel (present and year of erection). Standard Charpy specimens ISO-V made of a diagonal member of M65 fulfilled the 27 J-criterion at 20 °C with a value of 47 J but failed at 0 °C with a value of 21 J and at −20 °C with a value of 6 J. Thus Charpy testing revealed low temperature embrittlement which still was in agreement with old material standards (DIN 17100:1957). Mechanical testing of standard specimens did not reveal any effect of embrittlement at all. This may be explained by the small volume of strain aged material around stamped holes in components, see Section 2.4, which usually is not part of the material volume of standard specimens.

For estimating the effect of embrittlement on fracture forces, component type specimens containing the original riveted or boltet connection between corner rod and diagonal member were worked out of original parts of deviation tower M65 and tested, Fig. 10. The tests were performed at ambient temperature and at 0 °C and resulted in both cases in more or less brittle fractures, Fig. 11. The results were in perfect agreement with previous investigations [4].

The tensile tests of the structural parts, see Fig. 11 and Table 2, revealed fracture forces which partly achieved only 60% of the design capacity, the estimated minimum fracture forces based on the corresponding codes [18]. This means that standard specimens without holes or notches are not suitable for the identification of Thomas steel embrittlement of components, because they did not undergo plastic deformation and ageing. Today steel for use as a construction material for transmission towers has to comply with Sxxx-JR requirements, e.g. a transition temperature $\leqslant -70$ °C.

Table 1
Chemical composition of original members of tower M65 (built 1960). All reported values are in wt.%.

	C	Si	Mn	P	S	N
Corner rod (M65)	0.1690	<0.006	0.3680	0.0338	0.0267	0.0120
Diagonal member (M65)	0.0480	<0.006	0.4680	0.1040	0.0666	0.0160

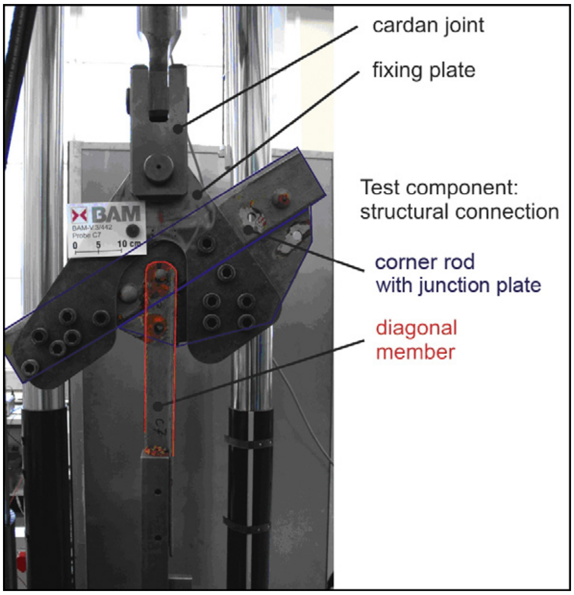

Fig. 10. Test rig for tensile testing of components (corner rod/diagonal member connection).

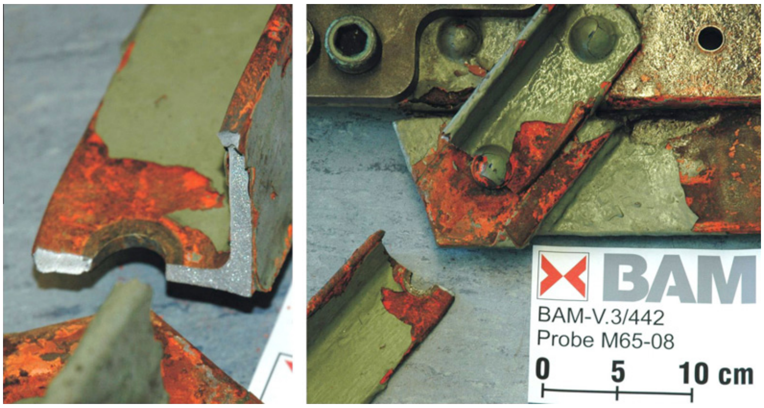

Fig. 11. Tensile testing of structural parts (corner rod/diagonal member connection). Brittle fracture of netto cross-section.

Table 2
Results of the tensile tests of structural parts of tower M65.

Tested component	Net cross section (mm^2)	Calc. max. force (kN)	Test temp. (°C)	Max. force during test (kN)	Deviation[1] %
B3	444	151	23	188	25
B5	444	151	23	177	17
B4	445	151	0	185	22
A5	444	151	0	182	21
C2	464	158	23	137	n.a.[2]
C1	464	227	23	155	n.a.[2]
M65-07B	464	158	23	131	−17
M65-07A	464	158	23	143	n.a.[2]
C3	464	158	0	158	0
C7	464	158	0	164	4
M65-01	464	158	0	170	n.a.[2]
M65-08A	464	158	0	125	−21
M65-09A	464	158	0	94	−40

[1] Deviation between calculated maximum force and maximum force during test in %.
[2] Failure of bolt or rivet.

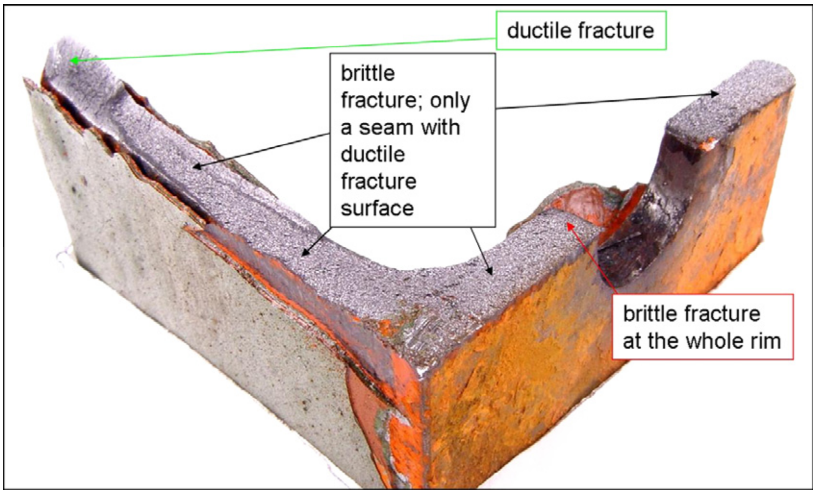

Fig. 12. Fracture surface M65-09 after mechanical tests in the laboratory. Similar fractographic appearance as the original fracture surface, cp. Figs. 6 and 7.

The fracture surfaces of diagonal members tested in BAM, Fig. 12, appeared almost identical to the original fracture surface concerning areas of brittle and ductile fracture, Fig. 6. Therefore it was concluded, that the fracture forces of the diagonal members during failure of the tower were about the value of those tested in the laboratory (94…125 kN), cp. Table 2. Consequently, these fracture forces were used in further static analyses.

2.6. Influences, failure load case

Weather conditions (snowfall, strong southwest wind) led to preferred deposition of snow on conductors of overhead electrical lines (northwest–southeast) perpendicular to dominant wind direction (southwest). Wet snow rolls with 13 cm and more in diameter formed, Fig. 1, that exceeded the line loads given in former [18] and current [19] design codes. At direction changes of overhead electrical lines specially designed deviation tension towers are used which are able to carry higher horizontal loads. During the severe weather conditions a change in line direction led to a deposition of snow rolls on the conductors, Fig. 1, perpendicular to the wind direction while in the direction parallel to the wind there were nearly no snow rolls. This applies for nearly all aerial lines that were pulled down during that night. Starting from the lines affected, it was investigated why the towers failed. Extensive photographic documentation provided by the operator (references in [20]) showed that snow rolls of the critical size were only existant at one field of deviation towers but not on the other field more parallel to wind direction. Moreover, photographs showed fields of this line BL1503 where conductors on the right side were covered with ice while they were not on the left side. This could be explained by different electrical currents of left and right electric system during snow/ice accretion. This meant for the further investigated deviation tension tower M65 that only the conductors on the right side (viewing direction towards increasing tower numbers) were covered with snow rolls while the conductors on the left side were not. This led to an asymmetric load distribution. The incoming conductors had only little snow coverage. This onesided and field-dependent (unequal) loading with lineloads of approx. 5 kg/m was identified as load case before failure of tower M65 and will be referred to as "failure load case", Fig. 13.

2.7. Load analysis

One deviation tower (M65) and one suspension tower (M66) were modelled for linear structural analysis. According to the current design code for overhead electrical lines (EN 50341-1:2001[19]) and also according to all the earlier codes (e.g. VDE0210:1958 [18]) linear analysis is sufficient. It could be confirmed that the transmission towers M65 and M66 fulfilled the code requirements (VDE0210:1958, [18]) applicable at the time of erection. The calculations clearly showed that for the forensic failure analysis further investigation should be focused on the deviation tower M65 as the suspension tower M66 is much less susceptible to increasing vertical loads respectively asymmetrical loading. From the failure site inspection (see Section 2.1) and fracture surfaces analysis (see Section 2.2) it was obvious, that rupture under tensile loads must be considered.

A "failure load case" could be derived on basis of the available information, Section 2.6, which was used to calculate the forces acting on the components prior to failure, Fig. 13. However, on site rupture was not observed for components of tower M65 showing the highest calculated tensile forces, e.g. highest utilisation factor. By comparison with experimental fracture forces of components (diagonal members) from tower M65, Section 2.5, the component showing primary failure could be identified: primary failure occurred at the joint of a diagonal member under calculated tensile force $F_{S,calc}$ of about

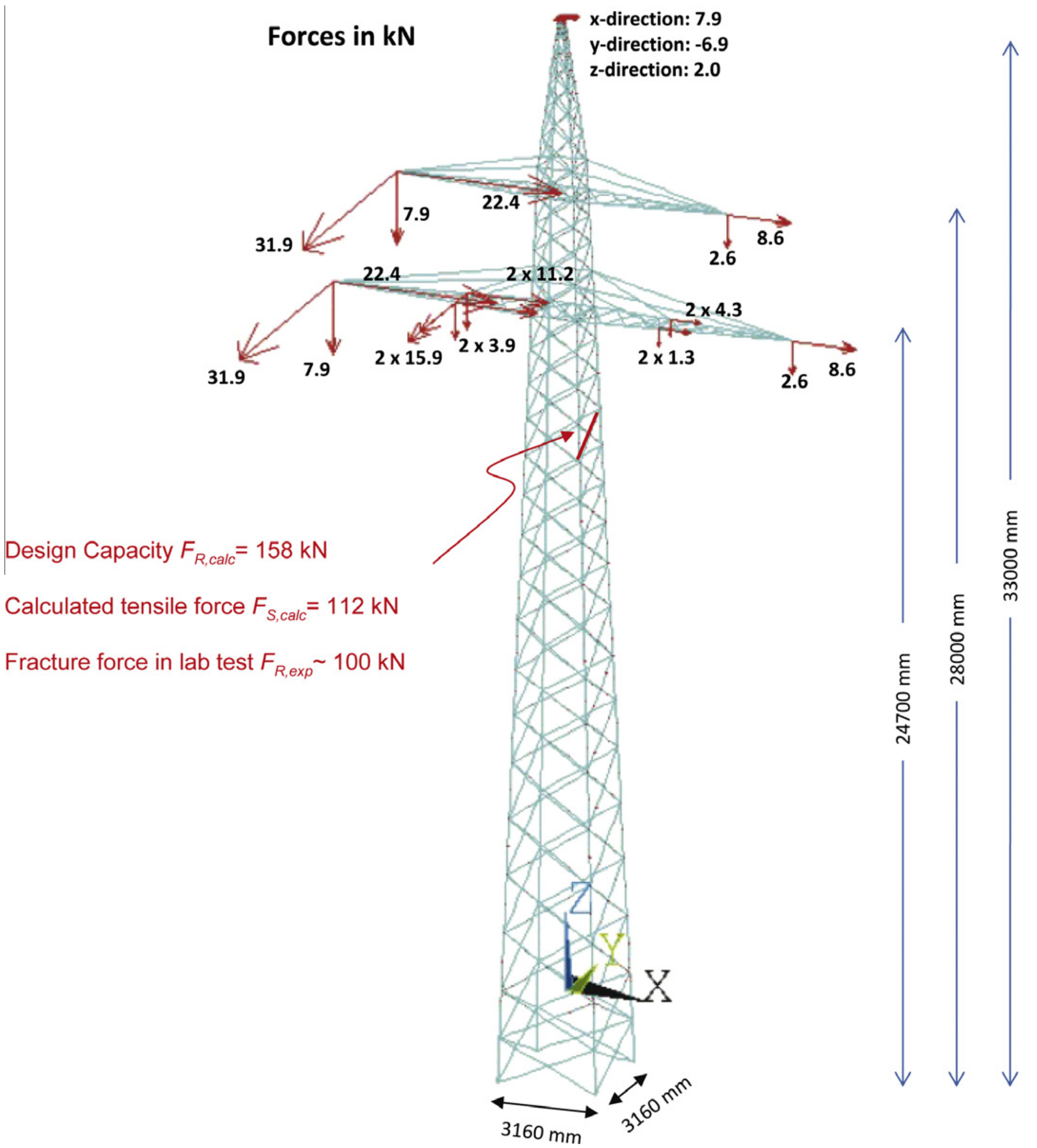

Fig. 13. FE-model of transmission tower M65, main tower dimensions, reconstructed "failure load case" with applied forces in kN and resulting loads on the diagonal member that failed initially.

112 kN, see diagonal member M65-04 in Fig. 4. This diagonal member with a L profile Section 60 mm × 60 mm × 6 mm originally had a design capacity $F_{R,calc}$ = 158 kN. Its failure reason is the weakening of this member (made of Thomas steel) by embrittlement down to a fracture force $F_{R,exp}$ of about 100 kN (94–125 kN), see Section 2.5. The brittle fracture of diagonal member M65-04 occurred due to ageing and embrittlement around the stamped holes (highly stretched zone) for the rivets of the joint connecting the member and the main leg, Figs. 6 and 7. As a consequence of the rupture of the diagonal member M65-04, the neighbouring diagonals M65-01 and M65-02 in Fig. 4 ruptured in a similar way due to a subsequent sudden increase of their load. This caused an inevitably large deformation of the tower beneath the cross arms and the tower failed eventually due to lack of bracings and buckling of the main legs.

Other failure mechanisms such as stability failure by buckling of single highly stressed diagonal members, Fig. 4, were under consideration but discarded as primary failure causes since they were contrary to visual observations [20].

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