

2017-2020

PROGRESS

PROVISIONS FOR GREATER REUSE OF STEEL STRUCTURES

Research Fund for Coal and Steel
Grant agreement No: 747847



Deliverable D7.1

Demonstration of the deconstruction and verification principles

The deliverable D7.1 “Demonstration of the deconstruction and verification principles” consist of the following reports:

- **Report on deconstruction of a single storey hall for reuse (D7.1a)** covers pre-demolition audit (part A of the Task 7.1) and deconstruction process (part C of the Task 7.1) including monitoring of small particles, UAV photogrammetry and testing of new deconstruction technique. Because the suitable deconstruction project was only available in spring 2020, the material tests were demonstrated on components removed earlier from different buildings and reported separately in D7.1b.
- **Report on testing of materials and products (D7.1b)** presents two innovative methods for the verification of material properties before deconstruction (part B of the Task 7.1), tests of mechanical and chemical properties of load-bearing structure (parts D and E of the Task 7.1) and the verification of the quality of the envelope (part F of the Task 7.1).

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Report on deconstruction of a single storey hall for reuse

May 2020



Deliverable: D7.1a Report on deconstruction of a single storey hall for reuse

Confidentiality: Public

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Executive summary

This report is part of Deliverable D7.1 of PROGRESS project. It covers pre-deconstruction audit, steelwork deconstruction and demolition of the remaining parts of a single storey steel building (SSB).

A suitable SSB to become a test case to demonstrate the recommendations from Work Package 2 was found in Cologne. The hall was built in the beginning of 2000. The project contained steel components and concrete based parts. The hall itself was a steel structure while remaining parts on and below the ground were made of concrete.

The structure was surveyed by UAV device. The SSB is built of 7 different parts. From up to down the hall consists of the roof insulation build out of styrofoam with a foil, the roof itself made of trapezoidal sheets, a secondary steel structure connected to the primary frames. The steel construction is founded on a reinforced concrete plate. The plate and abutments are located approx. 1.0 m above the ground. The space in between is filled with sand or any other material suitable at the original building time.

Costs and working hour information were collected to get results on how much extra costs are derived due to deconstruction for reuse.

The project was selected as an example to demonstrate that deconstruction for reuse is possible even for projects primary announced as demolition projects. The deliverable shows clearly, that the costs for deconstruction are acceptable and reuse is an additional option making the extra effort worth for all relevant parties.

The possibilities to perform deconstruction with the same machines as the regular demolition were also tested in this study.

The pre-deconstruction audit report based on the Deliverable D2.1 of the deconstructed SSB is attached to this report as Appendix.

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1 Introduction

The goal of this study is the verification of the theoretical research in WP2 in a real-life case. The possible object needs to fit the earlier description of a modern single storey hall, should not be older than 25 years and the cladding should be made of sandwich elements.

Typically, single storey halls are not built as temporary objects. Although costs for building are lower compared to concrete structures, today's buildings are not typically intended for reuse. Nevertheless a perfect single-storey steel building (SSB) was found in Cologne. The owner had to remove a single storey hall from the area of the great market in Cologne. His intention was just the clearing of the area as he was committed by contracts with the operator of the market area.

For the practical reasons, a compromise needed to be found between traditional demolition with the advantages such as little costs and high performance and new technologies which intend the reuse of the whole building.

Additional goal was to explore a technology based on machines which are typical for demolition work and to perform a deconstruction with them. The use of the standard demolition machines is helpful because it opens the possibility to switch from deconstruction to demolition extremely fast. Even for projects not intended for deconstruction during planning this might open the possibility to optimize the project for reuse on a later stage.

1.1 Overview of the project



Figure 1 Area of the great market of Cologne with the object to deconstruct for reuse

The great market of Cologne is an area, where different sellers of food and fruits store imported goods for reselling to companies, restaurants, and general food stores (Fig. 1). The area consists of many buildings and halls from all decades. The great market itself is not operating as seller itself. It generates the framework for other companies, which are selling the goods. The idea is to have all the big players of imported fruits on one place. Most of the larger cities in Germany have such markets for almost a century. Nevertheless, today there is little need for the idea of such centralized market. Thus, the market area in Cologne is closing during the next years and first companies are leaving the area already today.



Figure 2 View through the market area with the single storey hall to be deconstructed on the right.

Especially the bigger companies were running out of space somewhere in the past and built additional storage halls to extend their capabilities. Unfortunately, when leaving the area they are contracted to remove “own” buildings.

The result is the need to remove even relatively new buildings which are perfectly reusable.

2 The hall in Cologne

2.1 Surveying the structure

In most cases for objects to be demolished there is a lack of information. The original building plans do not exist anymore after some decades and the change of owners. In most cases, there is no information about materials used, possible hazardous waste (which in general was unknown to be hazardous in older times) and no drawings of the formerly planned building.

Survey of materials and even chemical tests of mineral waste is common and necessary due to the risk to contaminate the environment or risk health of people. To create drawings of buildings to demolish is usually not worth the effort. Nevertheless, exact geometry knowledge could be very helpful for the planning of a demolition project.

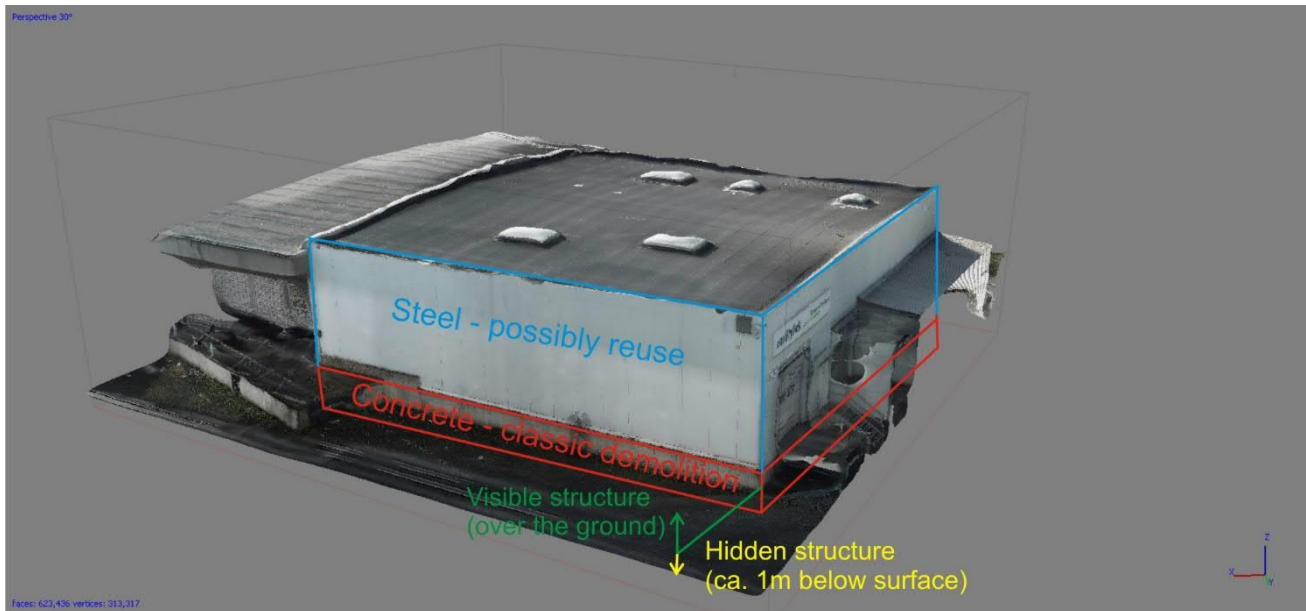


Figure 3; Overview of the SSB on a UAV model: The hall consist of a visible over the ground structure which is made of steel with attached cladding (blue), a concrete structure which ends ca. 1m below the surface (red).

During the planning of the work, the steel structure was identified as a possible reusable item while the concrete founding are to be demolished. For the crushed concrete, there is a general possibility to downcycling and reuse the material as recycling-gravel. This is obviously the lowest form of recycling.

The concrete structure is mainly unknown. From general experience it can be assumed, that floor plate and walls were made of 20cm reinforced concrete. The space below the floor plate is assumed to be filled with sand or a similar filling. For calculation, a box with no holes and no empty spaces is assumed.

The steel-work is easier to survey. While the roof is made of trapezoidal sheets with insulation, the primary structure consists of 4 frames made of 2 parts which are connected in the middle axis. The frames are made of non-standard steel beams. The secondary structure is made of small beams. It was planned to deconstruct the primary and secondary structure. The trapezoidal sheets and the

cladding are not recommended for reuse possibilities since a deconstruction of the light-weight parts is very time intensive.



Figure 4: Steel structure: The primary structure is made of 4 steel frames. There is no visible connection plate on the ground. Obviously, the columns are grouted with the concrete.

Table 1: Calculated masses and parts

1	Roof	531	m ²	Recycling, disposal
2	Wall (steel)	414	m ²	Recycling, disposal
3	Ground plate	106,2	m ³	Recycling off-site
4	Foundations / walls (concrete)	55,2	m ³	Recycling off-site
5	Filling	630	m ³	Based on chemical quality
6	Reusable structure	20	t	Deconstruction reuse



Figure 5: Between each of the frames smaller cold-formed steel beams as a secondary structure are visible. The connections are screwed.

Table 1 lists the main parts of the objects from the roof down to the foundations. In general, the concrete and filling are about 700 m³ / 1.500 t. The steelwork with cladding is approximately 50 t only. Even if the structure is going to be reused, most materials of the building are still becoming waste.



Figure 6: Inside there are 2 rooms. The walls between those rooms were made of cladding walls.

The height of the hall is below 10 m. That means, that the machines used for demolition may be used for deconstruction as well. This is important to switch during the work process between parts to deconstruct for reuse and concrete demolition work without changing the machines used. As a test case this opens up the possibility as well for projects not initially intended for reuse to deconstruct parts if it suits.

2.2 Additional object information

Besides the exploration of a structure all known information are collected. Especially on industrial sites it is urgently important to collect data about bombings due to World War II. Close to the building a probably not-exploded bomb was detected. Also, exactly at the position of today's hall, a German gun emplacement was located. The bomb itself is not affecting the demolition of the hall – the distance will not become closer as 30 m. Since the hall was built around the year 2000 the former gun emplacement was already known at times of building the hall. During earthwork at that time possible remains were already removed.

The following illustrations Fig. 8 and Fig. 9 show the original results from the German authorities on possible leftovers from World War II for the area of the demolition project.

The result of the analysis of leftovers is, that there is no danger due to the found unexploded bomb as well as there is no need for further local investigations. Nevertheless, work below the ground-level should be done with care.

While the finding of a World War II leftover doesn't affect the deconstruction / demolition project itself, the possible bomb will be investigated by authorities soon.

2.3 Boundary conditions: Sewer pipes

Any building is connected to the sewage system in the ground as well as to electricity and other services. It is evident that a building to remove needs to be cut from all the service systems before the demolition. The connection to the sewer system needs to be sealed to prevent pollution of the sewer system with ground material causing blockage of the sewage pipe.

The owner of the sewage system postulates a partly removal of the sewer system as well as sealing of the connections with concrete. The sewer system of the area is well documented (Fig. 7)

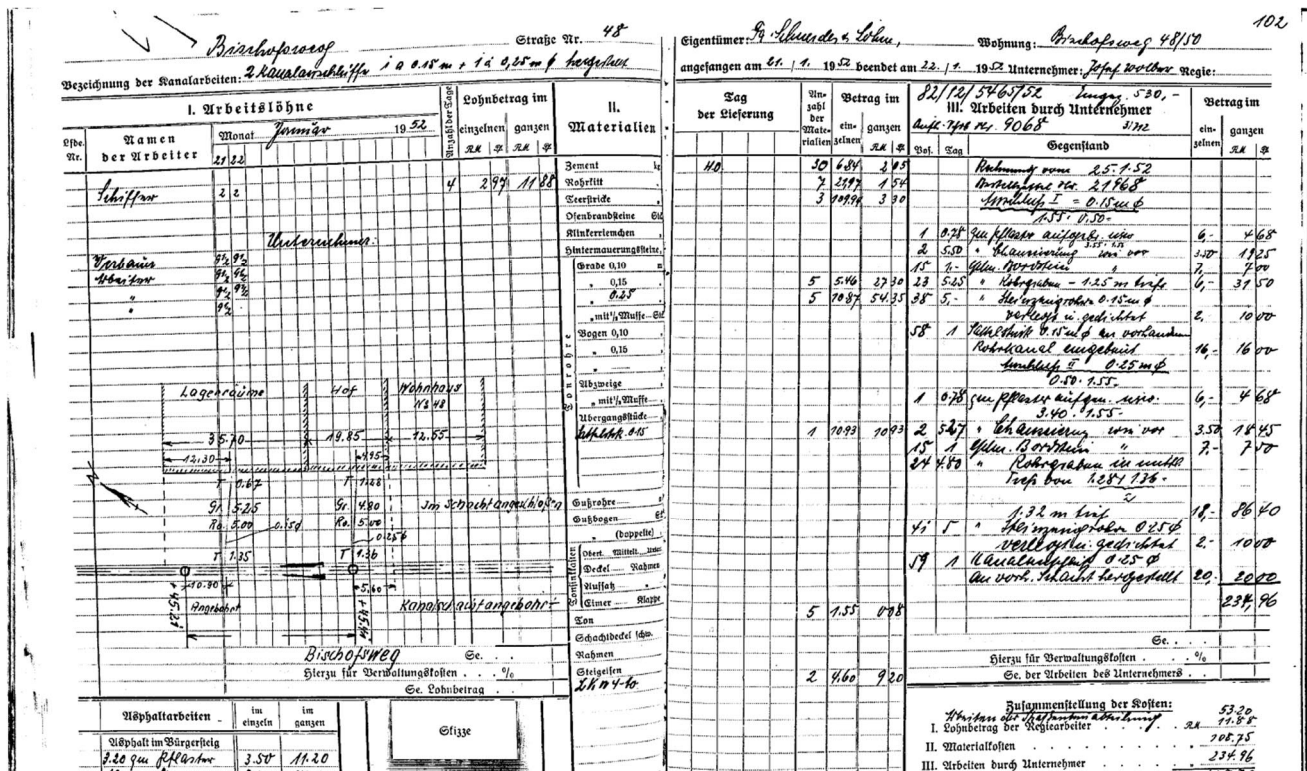


Figure 7: documentation of sewer system from 1952.

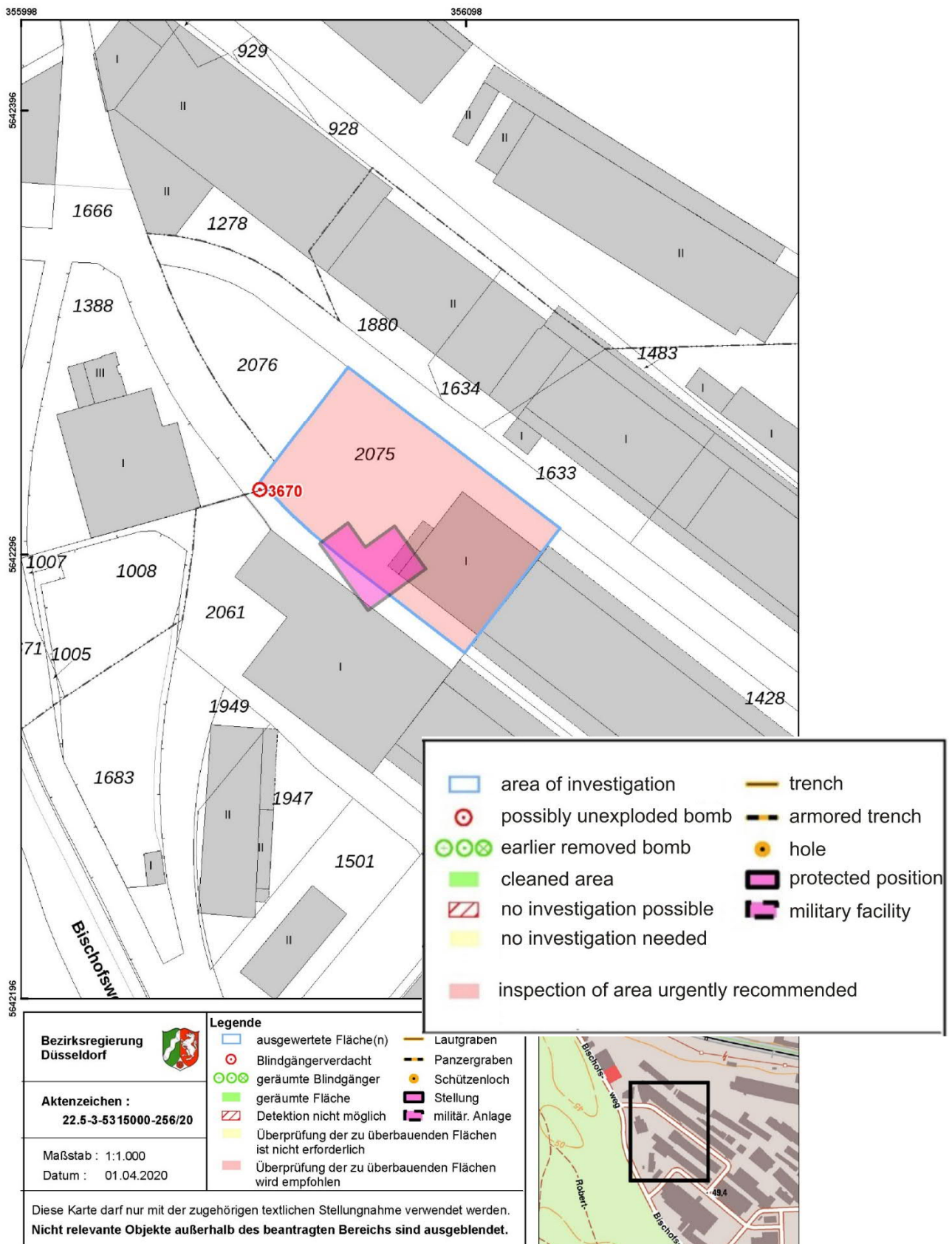


Figure 8: Result of investigation about World War II leftovers such as bombs which were not exploded.



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Bearbeiter Dunker

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Kommune Köln

Projekt Bischofsweg 48a in Köln Rondorf (Gmkg. Rondorf, Flr. 53,
Flrst. 2075)

Angaben zum Verdachtspunkt

Punktnummer 3670

Rechtswert 356051,33

Hochwert 5642310,13



Figure 9: Exact coordinates for a possible bomb from the times of world war II.

2.4 Calculation of waste

Any demolition project needs detailed calculation of the possible waste and information about the possible disposals. If possible, recycling plants are preferred.

Table 2: Calculated potential waste of the SSB

waste, not dangerous starting from 1 to / 1m³				
AVV	name		mass	type
170101	Concrete	☒	600	to
170102	Brick	☐		to
170103	Tiles, ceramics, brick	☐		to
170107	Mixture of above	☐		to
170201	wood (not painted All)	☐		to
170203	plastics	☐		to
170302	Bitumen	☐		to
170405	metals, steel	☒	60	to
170504	earth, natural stones	☒	600	to
170604	insulation	☒	10	to
170802	Gypsum	☐		to
170904	other waste	☒	40	to

Waste, dangerous				
AVV	name		mass	type
170106	Mixtures of concrete, bricks, ceramics with dangerous contaminations	☐		to
170204	Wood, plastics and glas with dangerous contaminations	☐		to
170301	bitumen polluted with tar	☐		to
170303	products containing tar	☐		to
170503	Earth, natural stones contaminated	☐		to
170601	insulation containing asbestos	☐		to
170603	dangerous insulation	☐		to
170605	asbestos	☐		to
170801	contaminated gipsum	☐		to
170903	other waste contaminated	☐		to
170902	waste polluted by pcb	☐		to

In summary, the object of a SSB in Cologne can be divided into 5 different waste types. The total steel was calculated to be up to 60 t including the sheets from the sandwich elements used as cladding. The total constructional elements would be about the half of the total used steel. The main waste will be produced from the concrete parts of the hall and the filling below the ground plate. Insulation and general waste (for example the sealing of the roof) are calculated to be up to 50 t and therefore more by weight than the total steel.

The list of possible waste is not giving information about the reuse potential, because it is not part of the German protocol for demolition. Thus, for the planning all potential kinds of waste have to be declared.

3 Deconstruction of the hall by excavators

3.1 Idea of excavator-based deconstruction

Generally, demolition is excavator based while deconstruction is based on lifting parts up. In easier words: Demolition is a technique to destroy a building or hall controlled and sort the masses into classes of materials. Deconstruction needs to survey the former elements. While during demolition connections of steel beams can be cutted with the help of the excavator (e.g. steel shear devices), a deconstruction project needs the connections to be unscrewed. Thus, an excavator seems not to be the right instrument for deconstruction.

On the other hand, after deconstruction of the steel parts an excavator is needed to remove the concrete basement parts. That means, that even a deconstruction of all reusable parts ends in traditional demolition of concrete based parts. Today, the removal of halls or other buildings is mostly entitled as a demolition. Thus, the contract of the client only describes the total removal of a specific object but does not describe the technique. The contractor owes the removal of the object in a given time span. Mostly, the contract will not specify that the former object has to be destroyed and become waste. Thus, it is up to the contractor how he plans to remove the object. Deconstruction is only an alternative, if the deconstruction can be performed fast enough to follow the given time span for the demolition project. To minimize costs of the extra work for deconstruction it would be a good to use the same machines as for the demolition. Then, a deconstruction could be performed as an alternative with the later chance to sell the reusable parts.

The less additional machines are used, the better the deconstruction can perform against the demolition.

For the Cologne SSB the deconstruction by excavators was tested. As a first step, the different parts of the SSB were analyzed if deconstruction would be an alternative to demolition.

3.2 Analysis of parts to be deconstructed

The SSB is built of 7 different parts. From up to down the hall consists of the roof insulation build out of styrofoam with a foil, the roof itself made of trapezoidal sheets, a secondary steel structure connected to the primary frames. The steel-based construction is founded on a reinforced concrete plate. The plate and fundaments are located ca. 1,0 m above the ground. The space in between is filled with sand or any other material suitable.

The deconstruction causes higher costs. Thus, it is important that parts are suitable for reuse after deconstruction. If they fail to be reused, those parts would be handled as waste / recycling materials only with the same effect as generally demolished parts. That means, that only those parts shall be deconstructed, which are not predisposed for deformation during the deconstruction process. Trapezoidal sheets are not deconstructable without deformation as well as sandwich elements could easily be damaged. Thus, it was chosen to deconstruct the primary and secondary structure while demolish the light-weight parts such as the cladding and the trapezoidal sheets.

Table 3 lists the parts and the decision about the handling for the test case. Figure 10 shows the deconstruction plan.

Deconstruction of a SSH

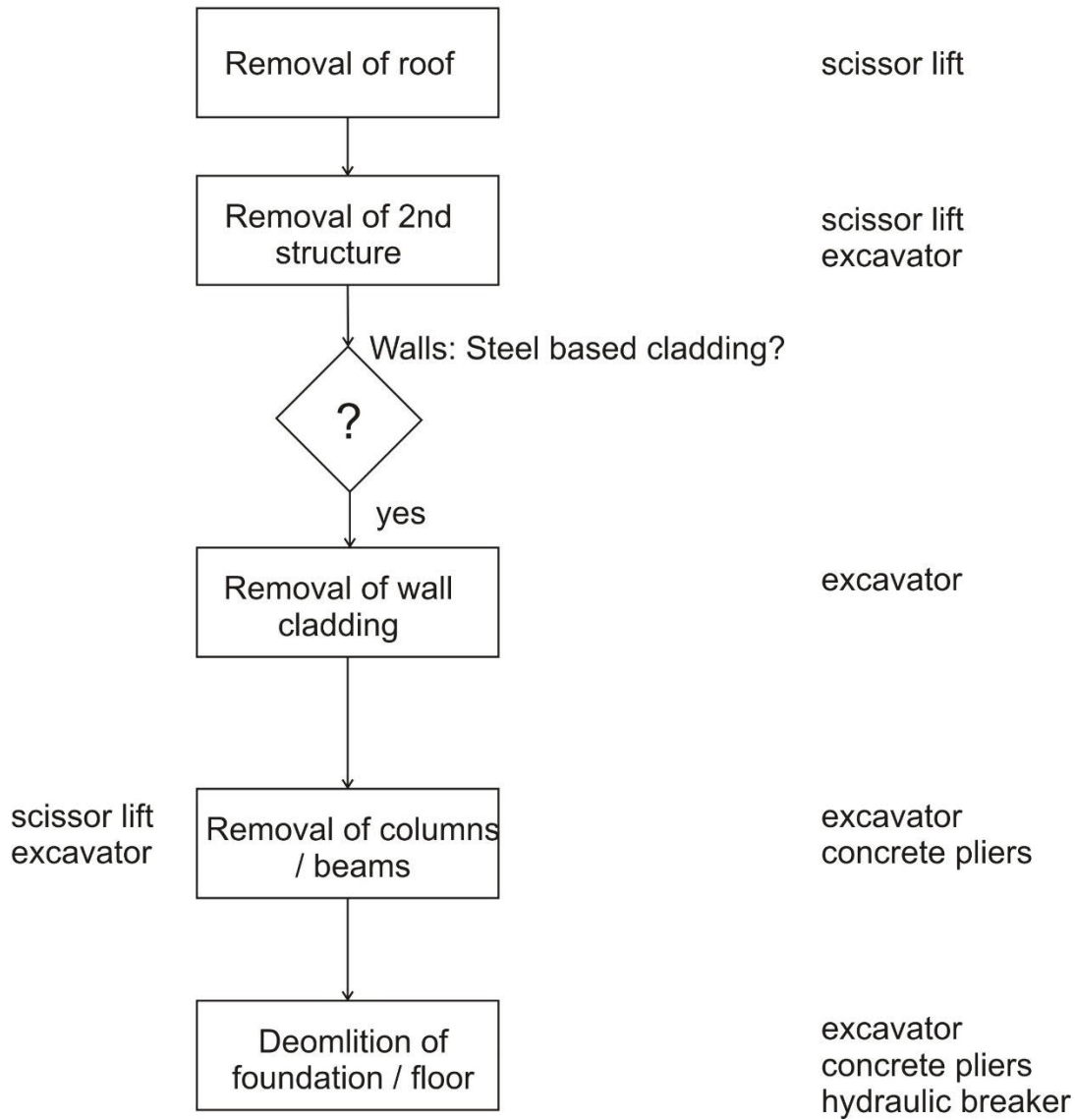


Figure 10: Deconstruction of the Cologne SSB by excavators. The primary and secondary strucutre is deconstructed for reuse. Cladding and light-weight materials are chosen for recycling only. Foundation is handled as regular demolition.

Table 3: List of parts of the SSB and decision of deconstruction vs. demolition

Part	Description	Demolition	Deconstruction	Comment
Roof insulation	Foil with insulation of styrofoam		(X)	Manual removal. Possible reuse but no market for used insulation parts
Roof	Trapezoidal sheets	X		Tends to warp. Potential of deconstructed parts to fail for reusing.
Secondary structure	Steel beams, screwed		X	parts can be unscrewed. Lifting by excavator
Primary structure	Steel frames, screwed but grouted into concrete		X	Uncovering old connection plates
Side walls	Sandwich elements	X		Tends to warp. Potential of deconstructed parts to fail for reusing.
ground plate	reinforced concrete	X		Monolithic part. Only available technique.
Filling	Sand, earth or equal	(X)		No structure. Only waste-management needed

3.3 Dismantling of structure by excavator

For general demolition of an SSB the safe technique for demolition of the structure is given in fig. 11. The technique is based on flame cutting the adjacent girders and pulling the structure down. If the primary structure is poured in concrete, there exist cutting techniques to create a hinge that allows the needed movement.

Further in the project, it will be checked, if the connection can be uncovered so that flame cutting will become evident.

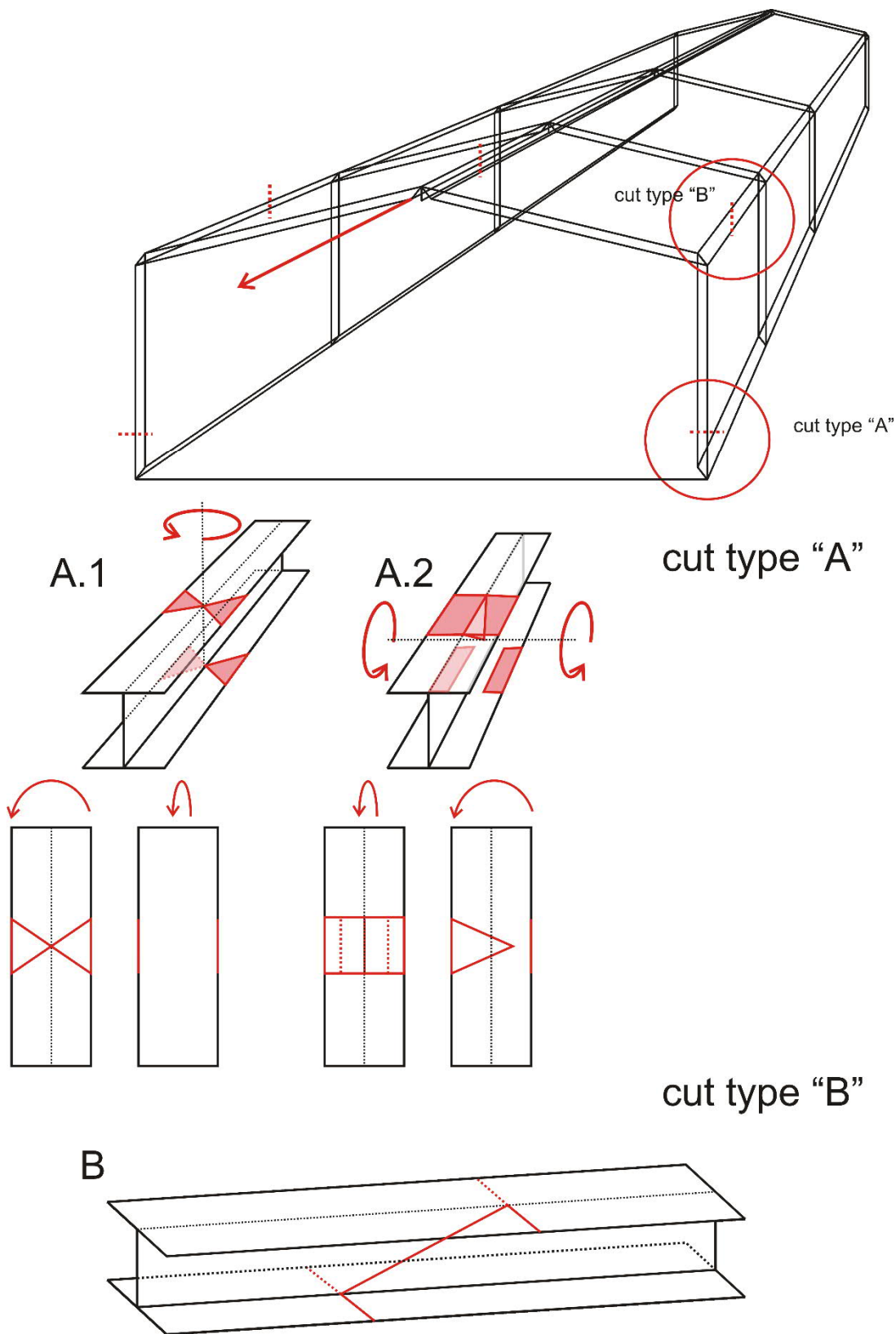


Figure 11: Combined technique to dismantle the primary / secondary structure for demolition.

In fact, the technique is the same for deconstruction, except that no flame cutting is used. Instead the joints need to be unscrewed. The poured parts have to be dismantled by small excavators. It is evident

not to release joints of the secondary structure before the anchoring in the ground. As well, deformation may become an issue. Thus, it is recommended to use two excavators for laying the frames on the ground.

3.4 Documentation of deconstruction / demolition

Any deconstruction or demolition follows the top-down principle. Halls in general are removed from the outer parts to the bearing structure. That means, after the deconstruction of the roof and the walls, the secondary and primary structure is dismantled. After the total removal of the steel based hall the demolition works of concrete parts follows. In the end, the filling will be transported away. Preliminary, the filling will need to undergo a chemical analysis.

3.4.1 Roof



Figure 12: Removal of roof insulation and sealing; Left picture before removal

Modern roof structures are insulated by a 30cm or more thick insulation of styrofoam. Removal by machines causes parts of the styrofoam to be destroyed and being blown away. To avoid the waste to be blown away it is recommended to remove the insulation by manual work. After the insulation removal, above the trapezoidal sheets all non-steel materials are deconstructed.

3.4.2 Removal of cladding

After the insulation of the roof the side walls were dismantled. During the dismantling by excavators with an attached gripper it was tested, whether the sandwich elements can be dismantled without damage. Thus, it was tested, if those elements were usable for reuse. In fact, the gluing between the metal sheet and the insulation was weathered so much, that the connection was not stable anymore. Some parts were transported to RWTH Aachen to undergo more technical tests. At least, the metal sheets were separated from the insulation. This separation helped to lower the total mass of general waste and increase the steel, which could be recycled.



Figure 13: SSB with removed roof insulation and removed walls of sandwich elements. On the right, an excavator is starting to separate the metal sheets from the inner insulation of the sandwich elements.

For separation the following technique was invented:

- 1) A Single element is laid down the ground.
- 2) The excavator grips one corner and pulls the metal sheet up. While pulling the sheet up the weight of the element is used to separate the metal sheet.
- 3) Two workers remove parts of insulation which adhere on the sheet.
- 4) The separated materials are sorted into metals and general waste.

Figure 14 shows the dismantling by excavator without deforming the sandwich elements. It was determined that separation of the different materials is best function, if the elements are not badly brought up but mostly in shape. Figure 15 illustrated the process of separation.

Thus, although in this example the sheets could not be reused, in general a dismantling technique open for reuse is recommended for the sheets. Even if reuse seems not practicable separating the materials will be easier if the elements are not deformed yet.



Figure 14: Dismantling of sandwich sheets. Testing of reusability. Technically, safe dismantling is possible by excavators. Reuse might not be recommended since the glue between metal and insulation is weathered. Besides, the metal sheets have many holes e.g. from attached lamps, signs and other inventory attached to the sheets.

After removal steps 1 and 2 the hall is left as a steel structure without any walls and the roof by trapezoidal sheets. The further dismantling has to consider the special geometry. The steel-based structure is connected to a concrete slab, which is located on metre above the ground for easier access to trucks. For the deconstruction of the structure the excavators are moving forward and shall remove only the trapezoidal sheets from the next axis to dismantle. Before removing the primary structure, the secondary structure needs to deconstruct. After deconstruction of each axis, the platform of concrete needs to be demolished.



Figure 15: Separation of metal sheets by help of excavator and two workers

3.4.3 Preparation for deconstruction of structure

The main advantage of excavators in opposite to dismantling structures by hand is the better efficiency. Trapezoidal sheets are very light weight compared to the secondary or primary structure. If the sheets are not needed anymore the use of an excavator avoids unnecessary unscrewing of the sheets. In fact, screws will burst if the excavator grabs a sheet. Because the sheets are much thinner than any structure, no parts of the secondary or primary structure are affected. In fact, it seems that the excavator just pulls off the sheets without any resistance.

At this stage, most materials from the object became waste. All metals became part of the recycling process and are put back into the cycle for re-melting. All insulation was used for energy recovery – in other words the styrofoam was burned and only used as energy source.

But, at this stage no parts were put back on the market as reusable parts. The reasons, why no parts were suiting the idea of reusing were:

- 1) Thickness of flat parts. Parts to be reused should be able to be handled without too much care. If parts tend to deform the chance is high that a certain percentage does not fit the claim of later reusing. If the parts are sold later for reusing but are not reusable due to deformation, they might be returned. The effort to deconstruct parts with zero deformation would be higher than the costs for generally new parts.
- 2) Especially the cladding on walls and roof are used during the first life for connecting certain building equipment parts to it. That causes changes in original colour, holes from connections and scratches to the sheets. While the bearing structure is mostly as on the first day of usage, the metal sheets look used. Reusing means that the whole SSB never will fit the definition of a new building.

- 3) It remains an open question, who will be responsible for deformations. If the former SSB was sold as one piece of reusable hall, each deformed part can cause claims by the buyer for a additional part which is free of deformation. Unfortunately, the risk is high to be responsible for the costs of those additional parts.
- 4) Even if just dismantled parts are sold, which may fit or not fit the needs for reusing, a price above the price of general scrap metal is only possible, if the percentage of not-deformed parts is at least 90%. To reach such a high percentage of intact parts, the efforts is becoming very high which may result in prices not in line with the market.



Figure 16: Dismantling old trapezoidal sheets just before deconstruction of structure.

After the removal of the trapezoidal sheets the structure remains. All parts up to this stage were sorted and were planned to recycle. The structure now will be the part which is carefully dismantled and not become waste but shall be reused. Mainly, in this stage there are 2 kinds of waste: Steel coming from the sheets and the cladding and styrofoam as filling of the wall cladding and as part of the roof insulation (see fig. 17)



Figure 17: Sorted waste for recycling. Mixture of trapezoidal sheets and sheets from sandwich elements.

3.4.4 Dismantling secondary and primary structure elements

The secondary structure is made of small beams with a diameter of 8 cm. Dismantling them by excavator can cause deformations while any manual deconstruction by workers is safe. Thus, the dismantling was done by scissor lifts and workers. The excavator was used as a crane only to lift up the parts. Besides, the excavator prevents parts from falling down.

To obtain stability, only the secondary structure between two main frames is dismantled. After the removal of a main frame, the secondary structure between the next fields is removed.

Figure 18 illustrates the dismantling procedure: The excavator is used as a crane with chains to prevent unwanted falling of the part. Workers release the connections of the beams to the primary frames. After releasing all connections, the beam is lifted and removed.



Figure 18: Dismantling of secondary structure. These parts are still too thin, that handling with an excavator only would result deformation. On the other hand, these parts are not tending to deform as long as dismantled manually. In opposite to the cladding, only five elements between each section are to dismantle.



Figure 19: The hall during the process of dismantling just before deconstruction of the structure. The trapezoidal sheets are being removed by 50%. Only the structure is remaining. After the removal of the trapezoidal sheets all remaining steel parts are considered for reuse.

Generally, the deconstruction of parts is mainly done by cranes. For the test case it was chosen to use excavators. On the one hand, excavators are the typical machines for demolition and are multifunctional. Excavators can be used not only to lift things up but also with attached tools to remove concrete, to load materials and to separate waste.

If excavators are planned to be used, they are limited by height. If the object is larger than the height of the excavator, a deconstruction by excavator work will fail. The possible height of excavators already reaches 50m and more. Beside those special machines standard excavators reach ca. 15m in height. That matched most SSB.

Nevertheless, the higher the distance between the structure and the excavator, the little horizontal forces are (see. Fig. 20). Especially if the frame is made of connected parts additional forces should be avoided. It was chosen to pull the frames down by two parallel working excavators.

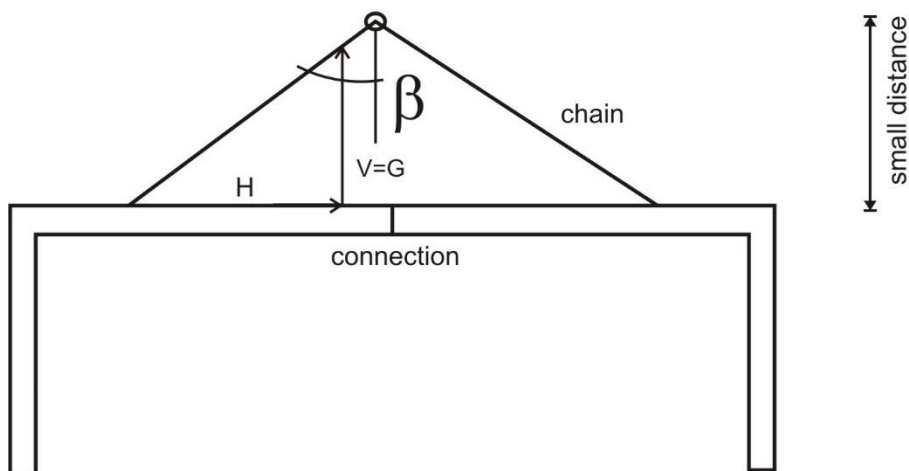
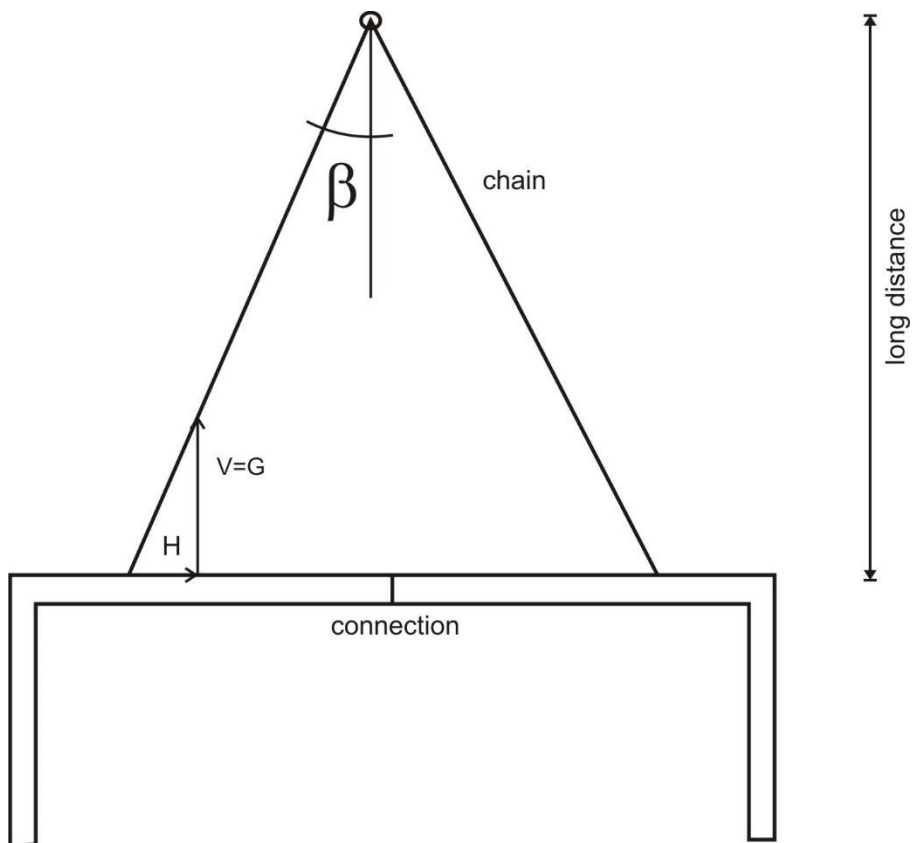


Figure 20: Forces in dependence to the vertical distance of the crane. The smaller the distance becomes, the more horizontal forces arise. It may affect the connections so that other techniques are needed instead of classical up lifting by excavators.



Figure 21: To avoid instabilities, two excavators carefully pull down the structure to the ground.

Figure 21 shows the solution: Lifting the whole frame up by one excavator would affect the connection point in the middle of the frame and is not recommended. Instead two excavators were used with grabbers to slightly pull down the frame. On the ground, all connections were released without the danger of collapse of parts of structure.

Before pulling the frame down, it was important to explore the connection of the steel-based frame to the concrete floor. From first explorations it was known that the steel frames were not having plates on the concrete plate but were shed into the concrete.

Figure 22 and 23 demonstrate the anchoring of the steel columns. After creating the fundaments, the construction of the steel frames was set up. When the primary / secondary construction was finished, the floor plate was made. The anchoring points were hidden underneath the concrete. During the planning it was discussed to cut the steel just above the concrete slab. That would cause that the reusable hall will be some smaller than before. After the exploration of the connection all anchoring points were uncovered so that the parts kept their original length.

The uncovering of the anchoring points resulted in temporary instability of the hall if no device was preventing tipping. Thus, while at least one excavator was stabilizing a frame, the anchoring points were uncovered only for the frame to pull down.



Figure 22: Removing concrete at jointing of steel frames. The steel connections was shed below the floor plate directly on the fundaments. Afterwards the connections was shed into the concrete of the floor plate. To release this connection, the concrete was removed partly around the anchoring.



Figure 23: View from the downside of the connection. The plates were connected to the fundamentals and shed into the concrete of the ground floor plate.

3.4.5 Demolition of mineral based parts

After the careful deconstruction of the structure only the parts not made of steel were left. The complete removal of the hall includes the demolition of those parts as well. The increase of the ground plate compared to the general ground was filled with sand.

Unfortunately, the sand was contaminated by PAK (tar) and reused from somewhere else instead of being a clean material.

Below the sand filling there exists still the old pavement from the times before the hall was created. This pavement was not removed, so that today you will find a pavement as 25 years ago again.

The figures 24 and 25 give an expression of the demolition of the ground parts. In the end any traces of the former SSB are gone.



Figure 24: Demolition of concrete slab and removal of sand filling. Below the old pavement was recovered.



Figure 25: Area of the SSB afterwards. The site is cleaned up.

3.5 Calculation of extra costs for dismantling structure

3.5.1 General costs of removal the SSB

The removal of modern SSB is not expensive. In fact, an SSB consists of little materials only, is mostly light-weight and mostly there are no contaminations like asbestos etc. Thus, for the given example the costs were 62.174€. About half of the costs were loans for workers, truck drivers and other people involved into the removal (32.979€). The second half of the costs included the waste management – the costs for concrete, general waste and disposal of the sand which was used for the filling underneath the concrete slab.

Table 4: Costs of the removal of the SSB

Removal costs	
Workers, machines	32.979,50€
Waste management	29.194,66€
Total costs	62.174,16€

The costs of work are separated in demolition work, the additional deconstruction work and the costs for separating the sandwich sheets and cleaning. Demolition and Waste management are quite the same with 13.664€ for demolition and 12.275€ for waste management. The additional costs for deconstructing the structure by scissor lifts, unscrewing on the ground, making the parts etc. were 7.039€. The additional costs were little because the machines planned for demolition were used and no additional “deconstruction only” machines were involved. The goal was to perform deconstruction by minimized costs and with the help of excavators to use this technique as alternative. In fact, the result is very nice if the additional costs are only half of the general demolition and / or waste management costs. The additional costs for deconstruction are about 1/5 of the total work-costs and little more than 10% of the total costs. Thus, deconstruction of structure can compete market conditions.

Table 5: List of work costs for Demolition, additional deconstruction and waste

Costs of work	
Demolition	13.664,86 €
Deconstruction	7.039,47 €
Waste management	12.275,17 €

32.979,50 €

The view on table 6 shows the costs of waste management including costs for disposal or recycling plants. Since the filling sand was slightly contaminated the main costs were for disposal of this sand. General (burnable) waste is of high costs always and mainly shows the immense costs of styrofoam which is not recyclable yet. Steel does not produce real costs except some leftover of transportation while concrete still is one of the lowest costs per ton.

Table 6: Costs of Waste management

Costs of Waste management			
Concrete	520,18	to	4.768,58 €
Earthwork / sand	849,24	to	14.935,15 €
General waste	37,53	to	8.686,85 €
steel	38	to	804,08 €
29.194,66 €			

The removal of the SSB took 688 hours “work” including general workers, truck drivers, excavator drivers. Comparable to the costs about 20% were the additional time for deconstruction instead of total demolition. Demolition took slightly more than waste management but still both actions are comparable by time.

Table 7: Working hours

Working hours		
Demolition	304,70	h
Deconstruction	156,97	h
Waste management	226,83	h
688,50 h		

3.5.2 Reuse of cladding

As written above the wall cladding was removed as if these parts were planned for reusing. The reason was just the separation of metal sheets. Nevertheless, due to the careful dismantling of the cladding it was possible to calculate costs comparing two cases: Deconstruction of cladding and reuse against demolition of cladding and buying new cladding for the hall to be reused.

If all the sandwich elements would be recovered without any transformations to the sheets, reuse would be cheaper than demolition and buying new cladding, even although the costs for deconstruction are much higher than demolition (see Table 8 deconstruction 15,84€ / demolition 3,5€; but: total costs 19,34€ for deconstruction / 23,40€ demolition).

Starting with a loss of 20% of the deconstructed elements the idea of reusing would produce higher costs than demolition with new cladding. That means, if during a deconstruction project including the

sandwich elements costs are cheaper as long as more than 80% of the deconstructed sheets would be in a condition to reuse those elements again. That means, mostly the reuse would be the cheaper possibility.

But, the aging effects of sandwich elements have to be taken into account. For the Cologne SSB the aging effects did not allow reuse of the sheets.

Table 8: Comparison of costs for reuse and renew sandwich elements for a hall to be reused.

costs of reuse cladding

Deconstruction				
Deconstruction of wall cladding		1 [m ²]		15,84 €
Transport, Warehouse		1 [m ²]		3,50 €
Total costs		1 [m ²]		19,34 €
Total w Loss / new 10%		0,1 [m ²]	2,59 €	21,93 €
Total w Loss / new 20%		0,2 [m ²]	5,19 €	24,53 €
Worker Deconstruction	16 [h]	160 [m ²]	42,00 € [€/h]	672,00 €
Scissorlift	1 [pc]	1 [d]		300,00 €
Crane, small	1 [pc]	1 [d]		640,00 €
Worker Forklift / Packaging	16 [h]		42,00 € [€/h]	672,00 €
Forklift	1 [pc]	1 [d]		250,00 €
				2.534,00 €
Demolition / New Cladding				
Demolition		1 [m ²]		3,50 €
Costs of new cladding		1 [m ²]		19,90 €
Total costs		1 [m ²]		23,40 €
Grabbing by excavator	4 [h]	600 [m ²]	144,00 € [€/h]	576,00 €
Separating	8 [h]	600 [m ²]	144,00 € [€/h]	1.152,00 €
Helper	16 [h]		42,00 € [€/h]	672,00 €
			-150,00	
Income steel		4,464 [to]	€ [€/to]	-669,60 €
Costs of Waste		2,976 [to]	125,00 € [€/to]	372,00 €
				2.102,40 €

3.6 Quality-check of parts

All parts identified for reuse needed to be checked afterwards. In general, all parts suited the quality needed. No parts were deformed. Some screw holes were damaged during flame cutting. Flame cutting was used, if unscrewing was not possible due to rust, mechanical errors, or impaired supply of the threaded rods.



Figure 26: deconstructed parts of primary structure ready to be reused.



Figure 27: Detail. Connection of frames



Figure 28: Parts before transportation to the warehouse



Figure 29: Detail: Small damage due to flame cutting.

3.7 Measurements of dust

Solid particle counters are used to measure dry particles for various industrial applications. One such application could be for the detection of particle size coming from a rock crusher within a mining quarry. Sieves are the standard instruments used to measure dry particle size. Vision based systems are also used to measure dry particle size. With a vision based system quick and efficient particle sizing can be done with ease and tremendous accuracy.

There are several methods used for detecting and measuring particle size or size distribution — light blocking (obscuration), light scattering, Coulter principle and direct imaging.

The light blocking optical particle counter method is typically useful for detecting and sizing particles greater than 1 micrometer in size and is based upon the amount of light a particle blocks when passing through the detection area of the particle counter. This type of technique allows high resolution and reliable measurement (Fig. 30).

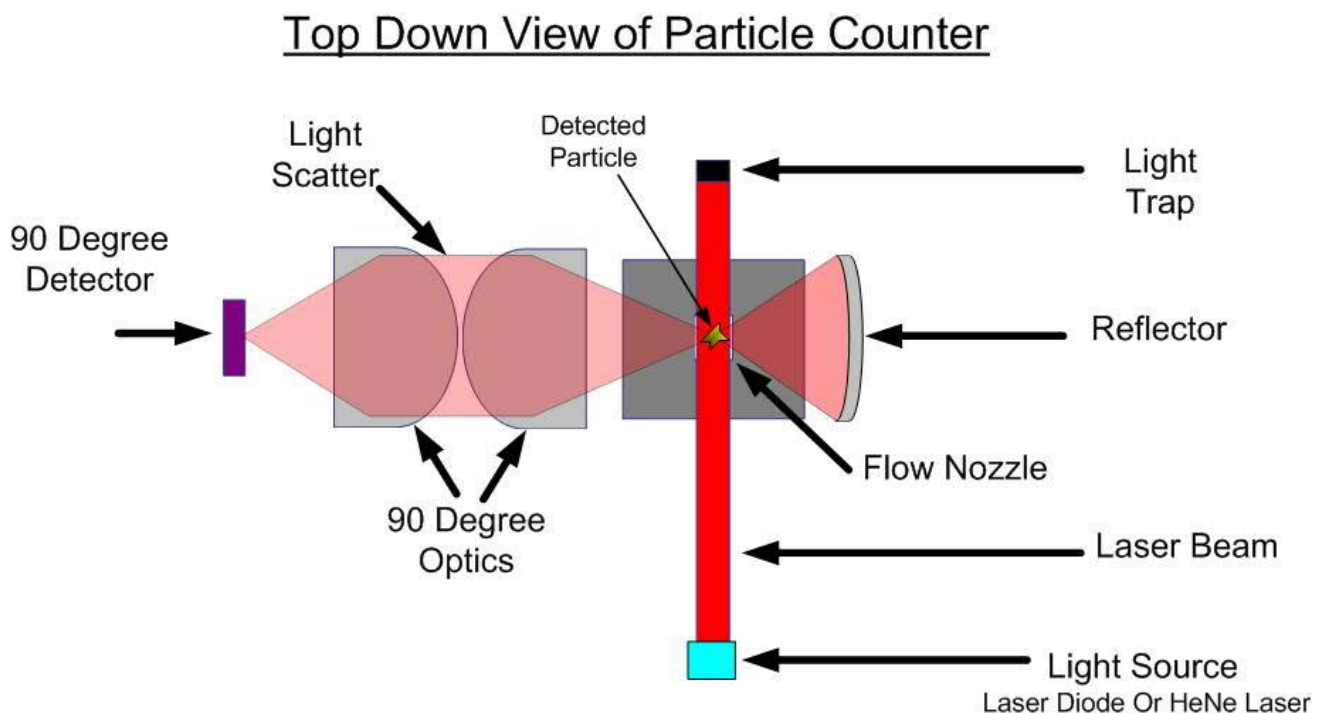


Figure 30: General scheme of a particle counter

The light scattering method is capable of detecting smaller-sized particles. This technique is based upon the amount of light that is deflected by a particle passing through the detection area of the particle counter. This deflection is called light scattering. Typical detection sensitivity of the light scattering method is 0.05 micrometre or larger. However, employment of the condensation nuclei counter (CNC) technique would allow a higher detection sensitivity in particle sizes down to nanometre range. A typical application is monitoring of ultrapure water in semiconductor fabrication facilities.

The light blocking method is specified for particle counters that are used for counting in hydraulic and lubricating fluids. Particle counters are used here to measure contamination of hydraulic oil, and therefore allow the user to maintain their hydraulic system, reduce breakdowns, schedule maintenance during no or slow work periods, monitor filter performance, etc. Particle counters used for this purpose typically use ISO Standard 4406:1999 as their reporting standard, and ISO 11171 as the calibration standard. Others also in use are NAS 1638 and its successor SAE AS4059D.

Direct imaging is a technique that uses the light emitted by a laser as a source to illuminate a cell where particles are passing through. The technique does not measure the light blocked by the particles, but rather measures the area of the particles functioning like an automated microscope. A pulsed laser diode freezes the particle motion. The light transmitted through the fluid is imaged onto an electronic camera with macro focusing optics. The particles in the sample will block the light, and the resulting silhouettes will be imaged onto the digital camera chip.

Especially for controlling conditions of clean rooms, small particle counters are on the market. These counters were tested now for indoor conditions of fiber contaminated rooms. In fact, the number of fibers / m³ are give an important information about the contamination. For asbestos a room is free of contamination if the number of fibers is below 500/m³. Exact measurement is expensive and not possible ion the air. Since fibers of asbestos are little a good possibility to control the conditions is a small particle counter. Since particle counters result in information of the number of particles of a specific size, this instrument can give an approximate feedback of the total number of particles in a room. The difference to regular measurements of asbestos or other fibers is, that particle counter do not differentiate between a fiber-particle and a general particle. It can be assumed, that particles smaller than 0,5µm could be fibers.

With particle counters originally created for controlling clean rooms (medicine reasons, electronical devices) there is a possibility to use results as approximate tests for contaminated rooms in buildings to demolish.



Figure 31: Particle counter with display.

First tests showed the following results:

1. The measured number of particles indoor is generally high and above 10.000 / m³.
2. A dusty room results in a measured number of particles > 1.000.000.
3. The use outside e.g. to control the dust produced due to demolishing a building is not given usable results.
4. A room cleaned up after the removal of asbestos and other fiber products shows particle numbers of < 2.500. This may be a sign that the work was following the guidelines.
5. Official measurement to declare a room as free of asbestos is still only possible by special measurement of asbestos.

Today, particle counters are helpful for self control reasons. Small numbers of particle are a sign of good work especially in contaminated areas. Measurement may become interesting for outdoor as well. Ongoing research is needed here.

4 Conclusion

A SSB suitable to become a test case was found in Cologne. The hall was built in the beginning of 2000's. The possibilities to perform deconstruction with the same machines as the regular demolition were tested.

The identification of parts which were deconstructed for reuse tried to estimate the risk of damage due to deformation. Thus, the primary and secondary structure was deconstructed while all light-weight parts were recycled only. During the deconstruction and demolition, it was realized, that the sandwich elements were aged so much over 20 years, that the glue failed on the connection of metal sheets and insulation was too bad for reuse. On the other hand, it was experimented, how separation of sheets could be possible on site.

Nevertheless, most waste masses are created by the demolition of concrete parts. The total masses only change little if reuse is applied.

On the other hand, if reuse is prepared with deconstruction techniques performed by excavators, the total costs are only little higher compared to general demolition. The additional time for deconstruction was only 20% higher than for a demolition without deconstructing parts. Thus, to increase the decision to deconstruct parts of modern SSB for later reuse, the later reuse has to be seen as an additional chance with low extra costs. This might help to start deconstruction as an idea performed even without having a special client or buyer. The risk of high costs is little. The chance to increase income of demolition of SSB due to selling reusable parts may justify more experiments.

From the practical view the deconstruction of light-weight parts seems the challenging thing in the future. Besides technical issues it is important to find a solution for the following question:

Who is responsible for parts which are not reusable after deconstruction due to deformation? What percentage of loss is acceptable? At the moment it seems difficult to put a hall on the market with only 75% of cladding. Additional, sandwich profiles undergo an evolutionary process and may not be the same some years in future as today. That could result in a higher percentage of parts which are needed to be renewed. On the other hand: Deconstruction of light-weight parts is even competing against new parts if about 20% loss is included. Besides the technical problems the reuse of light-weight parts is a legal question of acceptable contract conditions.

The main result is the possibility to perform deconstruction by excavators instead of cranes. Deconstruction for reuse becomes so something, any demolition contractor could do or at least try.

Nevertheless, planning becomes important. A view to temporary situations as well as a higher risk of collapse follows the deconstruction idea.

Annex A: sampling protocol of possible contaminated sand

Probennahmeprotokoll

Protokoll über die Entnahme einer
 Reststoff- / Abfallprobe

Paul
 Kamrath

Abbruchtechnik
 und Entsorgung
 Ingenieurrückbau GmbH

① Allgemeines:

Probenbezeichnung	Füllboden Köln Bischofsweg				
Baustelle	Köln, Bischofsweg 48a				
Zweck der Probenahme	Einordnung des RCL-Materials nach LAGA / DK0				
1	Probennahmestelle	Material aus Haufwerk			
2	Witterung	sonnig, trocken, ca. 16°C			
3	Datum	29.04.2020	Uhrzeit	12:15h	
4	Art der Probe	Mischprobe			
5	Entnahmegesetz	Schüppe			
6	Einzelprobe		Mischprobe	X	Zahl der Einzelproben

② Entnahmedaten:

7	Entnahmetiefe	40cm tief aus Haufwerk			
8	Farbe	Braun-gelb			
9	Geruch	Eigengeruch			
10	Probenmenge	ca. 3,5kg			
11	Probenbehälter	Eimer			
12	Probenkonservierung	Luftdichtes Verschließen			
13	Bemerkungen				
14	Foto	 probe: Site: Reason: Where: Whether: Type of sample: Tool: Nr. Of samples: One mixture Depth: Color: Mass: Container: Sand filling from Cologne Cologne, Bischofsweg 48a Analysis of material following the German LAGA / DK0 classification Middle of heap sunny, dry, 16° Mixture of sand shovel 40cm brown / yellow 3,5 kg bucket			
15	Unterschrift	Paul Kamrath Ingenieurückbau GmbH Hamburger Str. 27, 4432 Dortmund Tel. 0231-579751/52 Fax 0231-579753			

Annex B: sampling result. PAK (red marker) above limit



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Paul Kamrath Ingenieurrückbau GmbH

Hamburger Str. 97

44135 Dortmund

ISO 14001
ISO 45001
zertifiziert



Prüfbericht-Nr.: 2020P214401 / 1

Auftrags/Proben-Nr. 20204947 / 001

Probeneingang 29.04.2020

Probenehmer durch den Auftraggeber

Material Boden

Probenbez. Köln, Füllsand

Prüfbeginn / -ende 29.04.2020 - 07.05.2020

Parameter	Messwert	Einheit	Methode
Aussehen	sandig, steinig		organoleptisch 2
Farbe	braun		organoleptisch 2
Angelieferte Probenmenge	5,84	kg	
Probenvorbereitung	manuell + Backenbrecher		DIN 19747: 2009-07 ^a 2
Trockenrückstand	96,5	Masse-%	DIN ISO 11465: 1996-12 ^a 2
EOX	<1,0	mg/kg TM	DIN 38414-17: 2017-01 ^a 2
Kohlenwasserstoffe	<100	mg/kg TM	DIN EN ISO 16703: 2011-09 ^a i.V.m. LAGA KW/04: 2009-12 ^a 2
mobiler Anteil bis C22	<50	mg/kg TM	DIN EN ISO 16703: 2011-09 ^a i.V.m. LAGA KW/04: 2009-12 ^a 2
Cyanid ges.	<1,0	mg/kg TM	DIN ISO 17380: 2013-10 ^a 5
Summe BTEX	<1,0	mg/kg TM	DIN EN ISO 22155: 2016-07 ^a 2
Summe LHKW	<1,0	mg/kg TM	DIN EN ISO 22155: 2016-07 ^a 2
Naphthalin	<0,050	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Acenaphthylen	0,057	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Acenaphthen	0,059	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Fluoren	0,070	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Phenanthren	0,89	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Anthracen	0,27	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Fluoranthren	2,0	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Pyren	1,4	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Benz(a)anthracen	0,88	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Chrysen	1,1	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Benzo(b)+(k)fluoranthren	1,3	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Benzo(a)pyren	0,88	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2

Die Prüfergebnisse beziehen sich ausschließlich auf die genannten Prüfgegenstände. Es wird keine Verantwortung für die Richtigkeit der Probenahme übernommen, wenn die Proben nicht durch die GBA oder in ihrem Auftrag genommen wurden. In diesem Fall beziehen sich die Ergebnisse auf die Probe wie erhalten. Ohne schriftliche Genehmigung der GBA darf der Prüfbericht nicht auszugsweise vervielfältigt werden. Entscheidungsregeln der GBA sind in den AGBs einzusehen.

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Hamburg
Handelsregister:
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USt-Id.Nr. DE 118 554 138
St.-Nr. 47/72.3/00196

Geschäftsführer:
Ralf Murzen,
Dr. Roland Bernerth,
Kai Plinke,
Dr. Dominik Obeloer





GBAGROUP
 ENVIRONMENT

Parameter	Messwert	Einheit	Methode
Dibenz(ah)anthracen	0,20	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Indeno(1,2,3-cd)pyren	0,49	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
Benzo(g,h,i)perylene	0,52	mg/kg TM	DIN ISO 18287: 2006-05^a 2
Summe PAK (EPA)	10	mg/kg TM	DIN ISO 18287: 2006-05 ^a 2
PCB 28	<0,0010	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 52	<0,0010	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 101	0,0035	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 153	0,0093	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 138	0,0083	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 180	0,0067	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB 118	0,0014	mg/kg TM	DIN EN 15308: 2016-12 ^a 2
PCB Summe 6 Kongenere	0,028	mg/kg TM	DIN ISO 10382: 2003-05 ^a 2
PCB Summe 7 Kongenere	0,029	mg/kg TM	DIN EN 15308: 2016-12 ^a 2
Arsen	8,4	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Blei	89	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Cadmium	0,65	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Chrom ges.	17	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Kupfer	47	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Nickel	22	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Quecksilber	0,27	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Thallium	<0,30	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Zink	206	mg/kg TM	DIN EN 16171: 2017-01 ^a 5
Glührückstand (550°C)	97,4	Masse-% TM	DIN EN 15169: 2007-05 ^a 2
Glühverlust (550°C)	2,6	Masse-% TM	DIN EN 15169: 2007-05 ^a 2
TOC	0,70	Masse-% TM	DIN EN 13137: 2001-12 ^a 2
Lipophile Stoffe	0,041	Masse-% TM	LAGA KW/04: 2009-12 ^a 2
Eluat-Einwaage	104	g	DIN EN 12457-4: 2003-01 ^a 2
Eluivolumen	996	mL	DIN EN 12457-4: 2003-01 ^a 2
Filtratvolumen	990	mL	DIN EN 12457-4: 2003-01 ^a 2
pH-Wert	9,9		DIN EN ISO 10523: 2012-04 ^a 2
Leitfähigkeit	343	µS/cm	DIN EN 27888: 1993-11 ^a 2
Chlorid	4,2	mg/L	DIN EN ISO 10304-1: 2009-07 ^a 22
Sulfat	110	mg/L	DIN EN ISO 10304-1: 2009-07 ^a 22
Cyanid ges.	<0,0050	mg/L	DIN EN ISO 14403-2 (D3): 2012-10 ^a 5
Phenolindex	<0,0050	mg/L	DIN EN ISO 14402: 1999-12 ^a 5
Arsen	0,0043	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Blei	0,0015	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Cadmium	<0,00030	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Chrom ges.	0,0026	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Kupfer	0,0046	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Nickel	<0,0010	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Quecksilber	<0,00020	mg/L	DIN EN ISO 12846: 2012-08 ^a 5
Thallium	<0,0010	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Zink	<0,010	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
DOC	3,8	mg/L	DIN EN 1484: 1997-08 ^a 2



GBAGROUP
 ENVIRONMENT

Parameter	Messwert	Einheit	Methode
Cyanid I. freis. (CFA)	<0,010	mg/L	DIN EN ISO 14403-2 (D3): 2012-10 ^a 5
Fluorid	0,19	mg/L	DIN EN ISO 10304-1: 2009-07 ^a 22
Barium	0,014	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Molybdän	0,0052	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Antimon	0,0011	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Selen	<0,0020	mg/L	DIN EN ISO 17294-2: 2017-01 ^a 5
Abdampfdruckstand	228	mg/L	DIN 38409-2: 1987-03 ^a 2
Ges.-Gehalt an gel. Feststoffen	228	mg/L	DIN 38409-2: 1987-03 ^a 2
Wasserlöslicher Anteil	0,22	Masse-% TM	DIN 38409-1: 1987-01 ^a /DIN 38409-2 : 1987-03 ^a 2
Aussehen	klar		organoleptisch 2
Farbe	farblos		DIN EN ISO 7887: 2012-04 ^a 2

Die mit * gekennzeichneten Verfahren sind akkreditierte Verfahren. Die Bestimmungsgrenzen können matrixbedingt variieren.
 Untersuchungslabor: 2GBA Gelsenkirchen 5GBA Pinneberg 22GBA Herlen

Gelsenkirchen, 07.05.2020

i. A. J. Franzen

i. A. Jan-Niklas Franzen
 Projektbearbeitung

Annex C: List of waste transports

Paul Kamrath Abbruchtechnik & Entsorgung GmbH
 Hamburger Str. 97
 44135 Dortmund

Paul
Kamrath

Abbruchtechnik
und Entsorgung
GmbH

Abfuhr / Entsorgung

Seite 1 von 3

Datum	Uhrzeit	KFZ	Entsorger	Tonnage
Köln, Bischofsweg 48a, Großmark				
170101		BetonSchutt		
15.05.2020		DO-PK 1017	REMEX (Auffüllung)	19,36 to
15.05.2020		GI-AT 605	REMEX (Auffüllung)	21,60 to
14.05.2020		DO-PK 545	REMEX (Auffüllung)	25,00 to
14.05.2020		DO-PK 545	REMEX (Auffüllung)	25,00 to
12.05.2020		DO-PK 1017	REMEX	21,00 to
12.05.2020		DO-PK 545	REMEX	22,18 to
05.05.2020		GI-AT 605	REMEX	26,42 to
05.05.2020		GI-AT 605	REMEX	27,34 to
05.05.2020		GI-AT 605	REMEX	25,22 to
04.05.2020		ME-AM 1086	REMEX	25,14 to
04.05.2020		ME-AM 1086	REMEX	29,38 to
04.05.2020		ME-AM 1086	REMEX	29,90 to
04.05.2020		ME-AM 1086	REMEX	27,58 to
04.05.2020		GI-AT 605	REMEX	26,20 to
04.05.2020		GI-AT 605	REMEX	24,52 to
04.05.2020		GI-AT 605	REMEX	23,48 to
04.05.2020		GI-AT 605	REMEX	27,42 to
04.05.2020		GI-AT 605	REMEX	25,00 to
04.05.2020		ME-AM 1086	REMEX	24,40 to
28.04.2020		DO-PK 1017	Remex	25,00 to
28.04.2020		DO-PK 1017	Remex	25,00 to
Summe				526,14 to
170107		Schuttgemisch		
24.04.2020		DO-PK 1017	RRR	21,32 to
Summe				21,32 to
170405		Metalle, Eisen		
05.05.2020			Mösta	1,00 to
Summe				1,00 to

Dienstag, 19. Mai 2020

Datum	Uhrzeit	KFZ	Entsorger	Tonnage
170504		Boden & Steine		
15.05.2020		DO-PK 545	AMAND	25,30 to
15.05.2020		GI-AT 605	AMAND	30,90 to
15.05.2020		GI-AT 605	AMAND	29,05 to
15.05.2020		ME-AM 1086	AMAND	25,90 to
15.05.2020		DO-PK 1017	AMAND	23,65 to
15.05.2020		ME-AM 1086	AMAND	30,35 to
14.05.2020		GI-AT 605	AMAND	26,90 to
14.05.2020		GI-AT 605	AMAND	26,25 to
14.05.2020		ME-AM 1086	AMAND	32,15 to
14.05.2020		GI-AT 605	AMAND	29,20 to
14.05.2020		ME-AM 1086	AMAND	25,85 to
14.05.2020		ME-AM 1086	AMAND	25,85 to
14.05.2020		DO-PK 545	AMAND	24,70 to
12.05.2020		DO-PK 545	REMINERAL	29,34 to
12.05.2020		DO-PK 1017	REMINERAL	28,90 to
12.05.2020		DO-PK 545	REMINERAL	29,02 to
12.05.2020		DO-PK 1017	REMINERAL	30,46 to
07.05.2020		DO-PK 1017	Remineral	27,96 to
07.05.2020		HA-VB 3001	REMINERAL	25,62 to
07.05.2020		HA-VB 3001	REMINERAL	26,42 to
07.05.2020		DO-PK 1017	Remineral	26,88 to
07.05.2020		DO-PK 545	Remineral	29,94 to
07.05.2020		DO-PK 545	Remineral	29,70 to
06.05.2020		HA-VB 3002	REMINERAL	27,28 to
06.05.2020		HA-VB 3001	REMINERAL	29,18 to
06.05.2020		HA-VB 3002	REMINERAL	28,32 to
06.05.2020		HA-VB 3001	REMINERAL	27,64 to
06.05.2020		DO-PK 545	REMINERAL	28,96 to
06.05.2020		DO-PK 1017	remineral	32,82 to
06.05.2020		DO-PK 1017	REMINERAL	30,54 to
05.05.2020		ME-AM 1086	REMEX	30,06 to
Summe				875,09 to

170904		Mischabfälle		
15.05.2020		DO-PK 1974	AVG	2,10 to
05.05.2020		DO-PK 1974	AVG	10,33 to
30.04.2020		DO-PK 1974	AVG	4,75 to
28.04.2020		DO-PK 765	AVG	9,70 to
24.04.2020		DO-PK 1974	AVG	6,89 to
23.04.2020		DO-PK 1974	GWA	,88 to

Datum	Uhrzeit	KFZ	Entsorger	Tonnage
21.04.2020		DO-PK 1974	GWA	,58 to
20.04.2020		DO-PK 765	GWA	,54 to
20.04.2020		DO-PK 765	eco	1,76 to
Summe				37,53 to
Sand				
15.05.2020		DO-PK 545	REMEX	26,02 to
15.05.2020		GI-AT 605	REMEX	18,28 to
14.05.2020		GI-AT 605	REMEX	26,82 to
14.05.2020		ME-AM 1086	REMEX	25,46 to
14.05.2020		DO-PK 545	REMEX	26,58 to
Summe				123,16 to
170101 Stahlbeton				
27.04.2020		DO-PK 1017	Remitec	22,72 to
Summe				22,72 to
170405 Mischschrott				
05.05.2020		DO-PK 1974	Steil	4,00 to
Summe				4,00 to
170405 Schrott				
23.04.2020		DO-PK 1974	Steil	4,12 to
21.04.2020		DO-PK 1974	Steil	2,78 to
Summe				6,90 to
Gesamtsumme				1617,86 to

Annex D: Example of pre-deconstruction audit report

A General information

A1 Description of the building		
A1.1 Building description	Single-storey steel steel hall with welded-steel frames, fabricated by unknown sandwich panel wall envelope, unknown fabricator trapezoidal sheet roof, unknown fabricator	
A1.2 Address, site number	50969 Cologne, Bischofsweg 48a, Germany	
A1.3 Year of steelwork fabrication	2002	
A1.4 Floor area	523m ²	
A1.5 Main dimensions	span: 22,26m, length 23,60 m, height: ca. 8,5m	
A1.6 Envelope	sandwich panels: 578m ² , trapezoidal sheets: 530m ²	
A1.7 Crane (description, type, tonnage)	no	
A1.8 Steelwork is already deconstructed	no	
A2 Purpose of the deconstruction		
The rental agreement for the land on which the hall is located expires. The tenant must deconstruct the hall he has built himself.		
A3 Description of the future use of deconstructed components (if known)		
n/a		
A4 Building owner (name, address, e-mail, phone)		
Fruchtimport vanWylick.GmbH, Industriestraße 16, 50735 Cologne		
A5 Information about the auditor (name, address, e-mail, phone, certification(s))		
Paul Kamrath Ingenieurrückbau GmbH, Hamburger Str. 97, 44135 Dortmund		
A6 Information about the demolition/deconstruction company (if known)		
Paul Kamrath Ingenieurrückbau GmbH, Hamburger Str. 97, 44135 Dortmund		
A7 Building documentation and drawings		
A7.1 Design documentation	no	
A7.2 Fabrication documentation	no	
A7.3 Documentation of use and inspection reports	no	
A8 Certification		
A8.1 Mill certificates of delivered steel	no	
A8.2 CE marking of delivered steel products or steelwork	no	
A8.3 Other certification:	no	

B Description of the audit and recommendations

B1 Description of the available documentation

No design documents or certificates are available. Information of past WW2 bombing is given

B2 Description of the on-site visits (dates, persons, equipment)

1st visit:

Date January 2020

Persons: Paul Kamrath, Richard Stork, Responsible owner

Equipment: Standard measuring equipment, camera

Distances, spans and cross-sections were measured during the visit. Since no design documentation was available, some of the primary structure and secondary structure was checked in more detail. Photo documentation was obtained. Connection of main frame to ground was checked in detail. The frame structure ends underneath the concrete and is not visible.

After a visual inspection, it was concluded that the structure is not visibly damaged and its dimensions are within the acceptable tolerances.

2nd visit:

Date 4 March 2020

Persons: Paul Kamrath, Markus Kuhnhenne, Kevin Janczyk, Richard Stork

Equipment: Standard measuring equipment, cameras, drones

- Site inspection
- Discussion between RWTH and PKIR concerning dismantling and testing of envelope
- Drone flights, data recording for 3D-Model

B3 Description of the labelling system of reusable components

The main structure consists of four mainframes A to D.

B4 Description of the on-site measurements (equipment, results)

Distances and cross-sections were measured by help of 3D model:

Spans: Outer: 2 x 11,13m Distance between frames: 5,3m

No regular steel girders. Connections of mainframe and secondary structure are screwed.

B5 Description of sampling (methods, sample sizes, locations, numbers)

Samples of cladding were extracted

Samples of sand were taken for the contamination tests

B6 Evaluation tools and methods for off-site testing and condition evaluation

Samples of cladding were sent to RWTH for further investigation
 Results of sand testing are included in the Annex B

B7 Results of the evaluation

Structural frames A to D can be reused as they fulfil the admissibility conditions.
 Secondary structure was not evaluated before the deconstruction.

B8 Recommended reuse scenario(s) with optional evaluation of reusability (if not in A3).

Envelope will be recycled. Insulation is used for thermal energy. Only main structure and secondary structure are reused.

B9 Recommended treatment of materials and components

Roof skin:	waste for energy plant:	AVV 17 09 04
Roof insulation:	waste for energy plant:	AVV 17 09 04
Cladding:	recycling of metal sheets:	AVV 17 04 05
Insulation:		AVV 17 09 04
Trapezoidal sheets:	recycling of metal sheets:	AVV 17 04 05
Doors, gates:	recycling of aluminium:	AVV 17 04 02
Illuminants:	recycling ba lightcycle:	AVV 16 02 14
Concrete founding:	demolition and recycling	
Off-site		AVV 17 01 01
Filling beneath:	recycling off site	AVV 17 05 04
Frame structure:	Deconstruction and reuse	

Deconstructed elements for reuse will be transported to their new place.

C Inventory of reusable components

C1 General information

C1.1 Component	C1.2 Dimensions	C1.3 Quantity	C1.4 Location	C1.5 Year of fabrication	C1.6 Will be reused	C1.7 Reuse description
Frames	see the drawings	4	see the drawings	2002	no ¹	-
Secondary structure		m	see the drawings	2002	no ¹	-
Trapezoidal Sheets		m ²	roof	2002	no	-
Sandwich Panels	see the drawings	m ²	walls	2002	no	-

¹ It was decided about reuse after the audit was completed

C2 Material information

C2.1 Component	C2.2 Steel manufacturer	C2.3 Component manufacturer	C2.4 Certified steel grade	C2.5 Recommended steel grade	C2.6 Recommended material testing
Frames	unknown	unknown	n/a	S235 without testing	-
Trapezoidal Sheet	Unknown	tba	n/a	S235 without testing	-
Sandwich Panels	unknown	tba	n/a	-	PROGRESS procedure, testing according to EN 14509

C3 Recommended treatment

C3.1 Component	C3.2 Observed damage	C3.3 Recommended repairs	C3.4 Existing coating	C3.5 Recommended surface treatment
Frames with intumescent paint	n/a	n/a	n/a	-
Purlins	n/a	n/a	n/a	-

D Inventory of waste and hazardous substances

D1 Hazardous substances

D1.1 Substance	D1.2 Present	D1.3 Quantity/concentration	D1.4 Location	D1.5 Recommended treatment
Asbestos	no	n/a	n/a	n/a
PCB	no	n/a	n/a	n/a
Creosote	no	n/a	n/a	n/a
Other: no		n/a	n/a	n/a

D2 Inventory of waste¹⁹

D2.1 Material	D2.2 Code	D2.3 Hazardous	D2.3 Quantity	D2.4 Location	D2.5 Condition	D2.6 Recommended treatment
Iron and steel	170405	no	30 t	steelwork	painted, corroded	R4 (recycling of metals)
Concrete	170101	no	500 t	base slab	good for recycling	R5 (recycling of inorganic materials)
Mixed CDW	170703	no	60 t	base slab	n/a	D12 (permanent storage/backfilling)
Soil and stones	170504	no	800 t	excavation	n/a	D2 (land treatment)

E Drawings and photos







2017-2020

PROGRESS

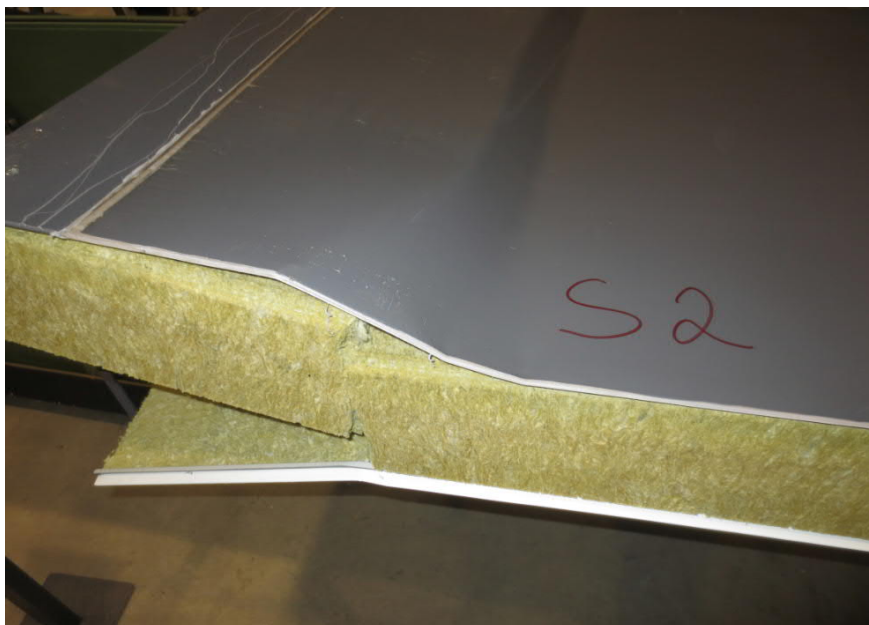
PROVISIONS FOR GREATER REUSE OF STEEL STRUCTURES

Research Fund for Coal and Steel
Grant agreement No: 747847



Report on testing of materials and products

February 2020



Deliverable: D7.1b Report on testing of materials and products

Confidentiality: Public

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Paul Kamrath
Paul Kamrath Ingenieurrückbau

Executive summary

This report is the deliverable D7.1b of the PROGRESS project (Provisions for Greater Reuse of Steel Structures) and covers the second part of Task 7.1 of the project Grant Agreement, namely the following topics:

- verification of the mechanical properties of the load-bearing structure (frames, secondary elements) after the deconstruction in Section 2.2,
- verification of the chemical properties of the load-bearing structure in Section 2.3, and
- verification of the quality of the envelope in Section 3.

The objective of the work reported in this document is to demonstrate the technical performance and efficiency of the testing methods and strategies for re-use of single-storey building components. The verification/testing methods are to be demonstrated on the materials and components removed from the existing buildings. The main aim is to review the processes needed if re-used components are going to be used in the new building in order to show to investors the supposed competitiveness of steel buildings due to their high technical reusability compared to different solutions.

The protocol for sampling and interpretation of material properties for the relevant certification is presented in PROGRESS deliverable D2.3 “Quality verification protocol including test results” [1] and the purpose of this document is to benchmark the experimental methods referred in [1]. Their complete overview is in Section 0 of this document and the complementary material is provided in the Annexes.

The materials and components for testing were obtained from different existing buildings of various age (12 to 70 years) and climate exposure (Romania, Germany, Finland) and tested in several laboratories (e.g. RWTH Aachen University, Universitatea Politehnica Timișoara, VTT Technical Research Centre of Finland).

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1 Introduction

The objective of the work reported in this document is to demonstrate the technical performance and efficiency of the testing methods and strategies for re-use of single-storey building components. The verification/testing methods are to be demonstrated on the materials and components removed from the existing buildings. The main aim is to review the processes needed if re-used components are going to be used in the new building in order to show to investors the supposed competitiveness of steel buildings due to their high technical reusability compared to different solutions.

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The materials and components for testing were obtained from different existing buildings of various age (12 to 70 years) and climate exposure (Romania, Germany, Finland) and tested in several laboratories (e.g. RWTH Aachen University, Universitatea Politehnica Timișoara, VTT Technical Research Centre of Finland).

1.1 Overview of the tests

The following text summarizes the number and types of experiments conducted on structural steelwork and envelopes recovered from single-storey steel buildings as well as the reference experiments on new materials. The specimens are labelled according to the component type and (if needed) the location of the extraction from the components.

Component types

- ST – Structural steel component
 - ST-B – steel bar (tie or stud) tested in SCIENCE project in 2013
 - ST-P – steel I-section removed from the steel hall in Romania in 2019
 - ST-P-I – web of the I-section
 - ST-P-T – flange of the I-section
 - ST-P-X – intermediate area between the web and the flange
 - ST-C – steel angle removed from the steel hall in Germany in 2018
- MW – Mineral wool sandwich panel
 - MW-N – panel removed from the north side of the building in Finland
 - MW-S – panel removed from the south side of the building in Finland
 - MW-Ref – reference (new) panel tested in 2007
- PIR – Polyurethane wool sandwich panel
 - PIR-S – panel removed from the south side of the Cube demo building in Aachen
 - PIR-W – panel removed from the west side of the Cube demo building in Aachen

The overview of mechanical, chemical and other tests reported in this document is presented in Table 1. It should be noted that some results for the reference materials (e.g. bars and MW panels) were obtained from the previous work by the project partners.

Table 1. Overview of the tests

Sample	Test type	Number of tests	Year	Standard(s)
ST-B1	tensile test	(5) ¹	2013	EN 10002-1
	small punch test	4	2019	prEN 15627
ST-B2	tensile test	(5) ¹	2013	EN 10002-1
	small punch test	4	2019	prEN 15627
ST-P1	tensile test	6	2019	EN ISO 6892-1
	small punch test	6	2020	prEN 15627
	instrumented indentation test	2	2020	-
	sub-sized impact toughness test	4	2020	-
	hardness test	25	2020	-
ST-P2	tensile test	4	2019	EN ISO 6892-1
ST-P3	tensile test	4	2019	EN ISO 6892-1
ST-C	tensile test	1	2018	EN ISO 6892-1, method B
	impact test at room temperature	1	2018	EN ISO 148-1
	chemical test	1	2018	-
MW-Ref	measurement of density	(6) ¹	2007	EN 14509
	compression test	(10) ¹	2007	EN 14509
	tensile test	(10) ¹	2007	EN 14509
	bending test	(5) ¹	2007	EN 14509
	shear test	(9) ¹	2007	EN 14509
MW-S1	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
	shear test	1	2018	EN 14509
	tensile test of fastener	5	2018	-
MW-S2	measurement of density	6	2018	EN 14509
	shear test	1	2018	EN 14509
MW-S3	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
	shear test of fastener	3	2018	-
MW-S6	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509

MW-S7	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
	shear test	1	2018	EN 14509
	thermal resistance test	2	2018	EN 12667, ISO 8301
MW-S8	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
MW-N4	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
	shear test of fastener	5	2018	-
	tensile test of fastener	5	2018	-
	thermal resistance test	1	2018	EN 12667, ISO 8301
MW-N2	bending test	1	2018	EN 14509
MW-N3	bending test	1	2018	EN 14509
MW-N5	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
MW-N6	measurement of density	6	2018	EN 14509
	compression test	5	2018	EN 14509
	tensile test	5	2018	EN 14509
	bending test	1	2018	EN 14509
PIR-S1	bending test	1	2020	EN 14509
PIR-S2	bending test	1	2020	EN 14509
PIR-S3	compression test	7	2020	EN 14509
	tensile test	7	2020	EN 14509
PIR-S6	shear test	7	2020	EN 14509
PIR-W1	bending test	1	2020	EN 14509
PIR-W2	bending test	1	2020	EN 14509
PIR-W3	compression test	7	2020	EN 14509
	tensile test	7	2020	EN 14509
PIR-W5	shear test	7	2020	EN 14509
Total number of tests		269 (+50)¹		

¹ Test results were obtained from the previous work

2 Verification of the quality of structural elements

2.1 Test specimens

The test programme for structural elements presented in this section covers structural steel elements from hot-rolled and cold-worked steel produced after 1970's that are supposed to be performing well according to the current standards, and a steel element from 1930's which was especially tested to conform to the strength, toughness and weldability criteria.

2.1.1 Steel bars

Two tensile test specimens made from structural steel grade S355 named *ST-B1* and *ST-B2* were delivered in the PROGRESS project for blind tests by Small Punch and nanoindentation test methods. The specimen designated *ST-B1* corresponds to the specimen 1 from 24 mm stud (Figure 1 left) and *ST-B2* corresponds to the specimen 2 made from 18 mm tie (Figure 1 right). Specimen *ST-B2* was cold worked to a higher strength as can be seen from the tensile test results in Section 2.2.1.



Figure 1 Tensile test specimens *ST-B1* (first on the left) and *ST-B2* (second on the right)

The tensile specimens were tested to rupture in SCIENCE project [2], and one grip end was given for Small Punch testing while the other end went to nanoindentation testing. The grip ends were cylindrical with diameter 20 mm in *ST-B1* and 18 mm in *ST-B2* and the tensile test was performed in hydraulic grips.

The purpose of this testing exercise was to demonstrate miniature-testing methods for validation of dismantled building frame beams. The benefit of miniature methods is that the required sample size is so small that it can be removed from the surface of the component by not compromising the integrity of the component.

2.1.2 Structural steel I-profiles

Four specimens from hot-rolled structural steel sections were prepared for the mechanical testing designated *ST-P1* to *ST-P4* (see Figure 2).



*Figure 2 Structural steel sections for the mechanical testing (from left to right)
ST-P2, ST-P4 (not tested), ST-P1 and ST-P3*

2.1.3 Structural steel angle

The last sample (ST-C) was tested separately as it was removed from the steel truss that was deconstructed for reuse. The purpose of the test was to check if the material's chemical composition allows the steel to be welded during the reuse process.



Figure 3 Structural steel section for the chemical testing ST-C

2.2 Testing of mechanical properties

This section summarizes the testing programme to determine mechanical properties of the steel extracted from the existing steelwork intended for reuse and the pilot tests made on steel stud and tie reused from the previous experimental programme. Two new minimal-invasive methods to determine tensile properties of steel were assessed and their results compared to standard coupon tests: small punch test and instrumented indentation tests.

2.2.1 Tensile tests

The basic properties of 24 mm structural stud *ST-B1* and 18 mm tie *ST-B2* were determined from coupon tests in accordance with EN 10002-1 'Metallic materials. Tensile testing. Part 1: Method of test (at ambient temperature)'. The results are presented in

Standard dog-bone coupons were extracted from the hot-rolled structural sections *ST-P1*, *ST-P2* and *ST-P3* and loaded with tensile displacement until the failure. The coupons extracted from the web are designated *ST-P1-I* to *ST-P3-I* and the coupons extracted from the flanges are designated *ST-P1-T* to *ST-P3-T*. The results of tensile tests of the structural steel hot-rolled sections are summarized in Table 4 and Table 5. The yield strength $R_{p0.2}$ and the ultimate tensile strength R_m are in Table 4, and the elongation at maximum load A_{gt} and the elongation at failure are presented in Table 5. The results of the coupon test extracted from the structural steel angle *ST-C* are presented in Table 3.

Table 2 Tensile test results of round specimen - strength

	ST-B1		ST-B2	
Sample	$R_{p0.2}$ [MPa]	R_m [MPa]	$R_{p0.2}$ [MPa]	R_m [MPa]
1	395*	541*	623	674
2	389	538	625*	673*
3	376	538	624	675
4	386	543	625	675
5	383	543	623	669
Average	386	541	624	673
St.dev	6.3	2.2	0.9	2.2

* specimen used for small punch and instrumented indentation testing

Table 3. Tensile test results of structural steel angle

	ST-C		
	$R_{p0.2}$ [MPa]	R_m [MPa]	A_t
Results of EN ISO 6892-1, method B	338	365	27.6%
USt 37-1 according to DIN EN 17100	> 240	370 -450	> 25%

Table 4 Tensile test results of hot-rolled sections webs (I) and flanges (T) strength [MPa]

	ST-P1-I		ST-P2-I		ST-P3-I	
Sample	$R_{p0.2}$ [MPa]	R_m [MPa]	$R_{p0.2}$ [MPa]	R_m [MPa]	$R_{p0.2}$ [MPa]	R_m [MPa]
1	377	506	355	470	370	481
2	376	504	355	475	364	480
3	367	507	-	-	-	-
Average	373	506	355	473	367	481
St.dev	4.5	1.2	0.0	2.5	3.0	0.5
	ST-P1-T		ST-P2-T		ST-P3-T	
Sample	$R_{p0.2}$	R_m	$R_{p0.2}$	R_m	$R_{p0.2}$	R_m
1	357	488	349	482	384	458
2	356	483	366	511	391	471
3	351	489	-	-	-	-
Average	355	487	358	497	388	465
St.dev	2.6	2.6	8.5	14.5	3.5	6.5

Table 5 Tensile test results of hot-rolled sections webs (I) and flanges (T) elongation

	ST-P1-I		ST-P2-I		ST-P3-I	
Sample	A_{gt}	A_t	A_{gt}	A_t	A_{gt}	A_t
1	20.8%	35.1%	21.1%	34.6%	14.2%	-
2	20.1%	36.0%	21.0%	35.0%	17.4%	34.0%
3	18.1%	-	-	-	-	-
Average	19.7%	35.5%	21.0%	34.8%	15.8%	34.0%
St.dev	1.13%	0.44%	0.05%	0.20%	1.60%	-
	ST-P1-T		ST-P2-T		ST-P3-T	
Sample	A_{gt}	A_t	A_{gt}	A_t	A_{gt}	A_t
1	17.0%	-	17.7%	31.0%	12.1%	25.6%
2	18.8%	35.2%	14.5%	-	11.1%	-
3	17.0%	-	-	-	-	-
Average	17.6%	35.2%	16.1%	31.0%	11.6%	25.6%
St.dev	0.87%	0.00%	1.63%	-	0.48%	-

2.2.2 Small punch tests

For Small Punch testing a sample can be removed either by drilling by a tube drill, which leaves a cylindrical sample of minimum 8 mm diameter, or by a larger tube drill applied in a small angle to the surface. Also in special cases, dedicated wire erosion methods can be applied. The Small Punch specimen size is just 8 mm in diameter with thickness of 0.5 mm. In practice, the sample should be at least 0.7 mm thick because it must be ground on both sides in order to remove any deformation produced by the sample removal process. The test specimens were prepared by a subcontractor by cutting 0.7 mm thick slices from which 8 mm diameter specimens were cut by wire erosion and ground on both sides to a thickness of 0.500 ± 0.005 mm. All the specimens fulfilled the thickness requirement.

Methods

Tensile Small Punch tests were carried out according to the prEN15627 [3] draft standard at room temperature at a puncher displacement rate of 0.3 mm/min. Load was measured by a calibrated load cell and the specimen deflection was measured from below by a digital displacement transducer. The schematic test arrangement is shown in Figure 4. From each test, a load-deflection curve is measured as shown in Figure 28. At room temperature, the deflection measurement rod (part D in Figure 4) does not have to have a thermocouple. That is necessary only in Small Punch creep testing at elevated temperatures where a direct measurement of the test specimen temperature is crucial.

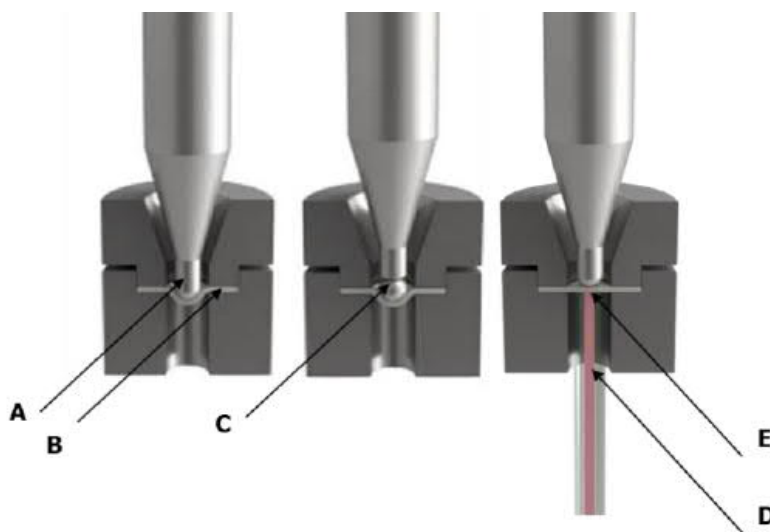


Figure 4. Small Punch test arrangement, where A is the punch; B is the specimen; C is the punch-ball; D is a ceramic rod for displacement measurement (and temperature E); E is the location where a thermocouple can be integrated into the ceramic displacement rod.

The calculation of yield strength is based on two lines between deflections 0 and 0.5 mm fitted by a dedicated bilinear method where the fitting error is minimised as shown in Figure 31. For calculation of the yield stress, the load at the intersection point is used (load F_e in Figure 31). The load-deflection curves are shown for ST-B1 in Figure 11 and for ST-B2 in Figure 12. In the fourth test for ST-B2, there was an operator error when the deflection measurement transducer was properly set up and therefore the deflection could not be recorded. Therefore the yield and ultimate tensile strength values for test no 4 are estimated on the basis of cross-head movement and comparison with test no 2 because for these two test the cross-head displacement curves were practically overlapping. The yield strength values were calculated according to the Equation (1) (D.1 in the prEN15627 [3] draft standard) with a coefficient value of 0.51:

$$R_{p0.2} = \beta_{Rp0.2} \cdot \frac{F_e}{h_0^2} \quad (1)$$

where F_e is the yield load measured as in Figure 31 and h_0 is the specimen thickness. The ultimate tensile strength values were calculated according to the Equations (2) and (3) (C.1 and C.2 in the prEN15627 [3] draft standard) Equation (2) with a coefficient value of 0.2784 and Equation (3) with a coefficient value of 0.192.

$$R_m = \beta_{Rm} \cdot \frac{F_m}{h_0 u_m} \quad (2)$$

where F_m is the maximum load and u_m is the deflection at F_m .

$$R_m = \beta_{Rm} \cdot \frac{F_i}{h_0^2} \quad (3)$$

where F_i is the load at deflection $u = 0.552$ mm. This equation is based on the idea that the active load carrying area is dependent on how far the ball has penetrated into the specimen.

Results

The yield and ultimate tensile strength values are tabulated in Table 6. For *ST-B2* the yield strength value is higher than the ultimate tensile strength values, which is not physically possible. The ultimate strength values calculated by Equation (2) are systematically higher for *ST-B1*, but lower for *ST-B2* compared to the values calculated by Equation (3). The reason for this is the higher yield stress of *ST-B2* and therefore the different shape of the load-deflection curve is different and the F_i values are higher for *ST-B2*. The different load curve shape is seen clearly in Figure 30. In the case of the second sample *ST-B1*, the values for test 4 are based on cross-head movement and comparison with test 3.

The load curves can also be plotted as a function of stress as shown in Figure 32 and Figure 33 for *ST-B1*. It can be seen that the shape of the stress curves in Figure 32 and Figure 33 are rather similar and there is not much scatter between individual curves, but the maximum stress values are much lower when using Equation (3). The load-deflection curves from *ST-B2* are rather close to each other as seen in Figure 29. In addition, the stress curves in Figure 34 and Figure 35 are similar in shape, but the maximum stress is higher when using Equation (3).

The standard deviation for the yield strength in Table 6 is typically higher than for the tensile strength, and this was the case for *ST-B1*, but not for *ST-B2*, which showed exceptionally little scatter in the yield strength analysis. For *ST-B2* the yield strength value is higher than the ultimate tensile strength values, which is not physically possible, which shows that the current analysis method for yield strength is not very suitable for materials, which a yield strength close to the tensile strength. Then, the *ST-P1* specimen was tested by Small Punch method. For the assessment of tensile strength, the Equation (3) was used. The results for the web and flange are shown in Table 7. The agreement with the traditional test method is reasonable. The standard deviation for the flange material in SP testing was surprisingly low.

Table 6. Small punch test results

	ST-B1			ST-B2		
Sample	$R_{p0.2}$ [MPa] Eq. (1)	R_m [MPa] Eq. (2)	R_m [MPa] Eq. (3)	$R_{p0.2}$ [MPa] Eq. (1)	R_m [MPa] Eq. (2)	R_m [MPa] Eq. (3)
1	351	584	490	679	637	656
2	377	585	502	680	624	643
3	377	577	495	691	614	639
4	379	573	492	677	626	645
Average	371	580	495	682	626	645
St.dev	13.4	5.7	5.3	6.3	9.5	7.9

Table 7. Small Punch test results for the web (ST-P1-I) and flange (ST-P1-W)

	ST-P1-I		ST-P1-T	
Sample	$R_{p0.2}$ [MPa] Eq. (1)	R_m [MPa] Eq. (2)	$R_{p0.2}$ [MPa] Eq. (1)	R_m [MPa] Eq. (2)
1	386	498	389	481
2	412	500	384	486
3	430	492	379	491
Average	409	497	384	486
St.dev	22.1	4.2	5.0	5.0

2.2.3 Instrumented indentation testing

Instrumented indentation measurements were carried out to obtain material data to eventually extract the yield strength for the material. Anton Paar Tritec's UNHT device was used with a spherical ($R=20\mu\text{m}$) indenter. The plan at start was to carry out similar measurements as described in [7]. However, this could not be carried out due to device limitations. The measurements were eventually carried out using several normal loads to obtain material information later on utilized in investigation of the yield strength of the material. A total of 10 repeated indentations were carried out per normal load using quadratic loading with loading and unloading time and hold time at the peak load of 15 seconds. The normal loads used varied between 1 and 75 mN. The measurements were carried out to ground and polished samples "2" (flange) (see Figure 5) and "4" (web) taken from a structural section *ST-P1*. The mean force-penetration graphs of indentations with normal loads 1-75 mN are shown in Figure 6 and Figure 7.

The data obtained from the nanoindentation includes e.g. elastic modulus (E), hardness (H) and stiffness (S). Oliver & Pharr method [8] was utilized in the calculations. The plan was to use the nanoindentation data to draw a stress-strain curve and eventually calculate the material yield strength.

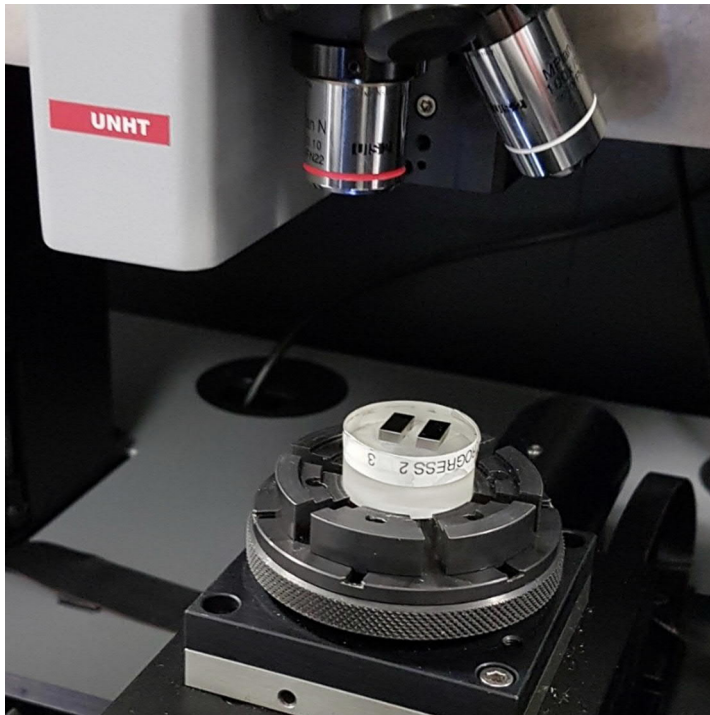


Figure 5. Sample "2" under the UNHT device.

Nanoindentation measurements were analyzed with the procedure described in [7]. An average value for the needed variables for each 10 repeated indentation loads were used to compute each stress and strain value. Young's modulus for the analysis EIT value was used. It takes into account the assumed Poisson's ratio (0.3 in this case) but it was found to be a bit low. E^* could have been more suitable as it was closer to the assumed value for these types of steels. The calculated young's moduli for the flange and web were 169 GPa and 168 GPa respectively. The effect of low young's modulus value to the stress-strain curve is assumed to be small. The plastic constrain factor was assumed to be 3 (as in [7]) which might be a bit low in our case since it has great impact on the calculated stress and a bit higher factor would have yielded more realistic values for the stress-strain curve. The stress-strain curves for both materials are presented in Figure 8 and Figure 9.

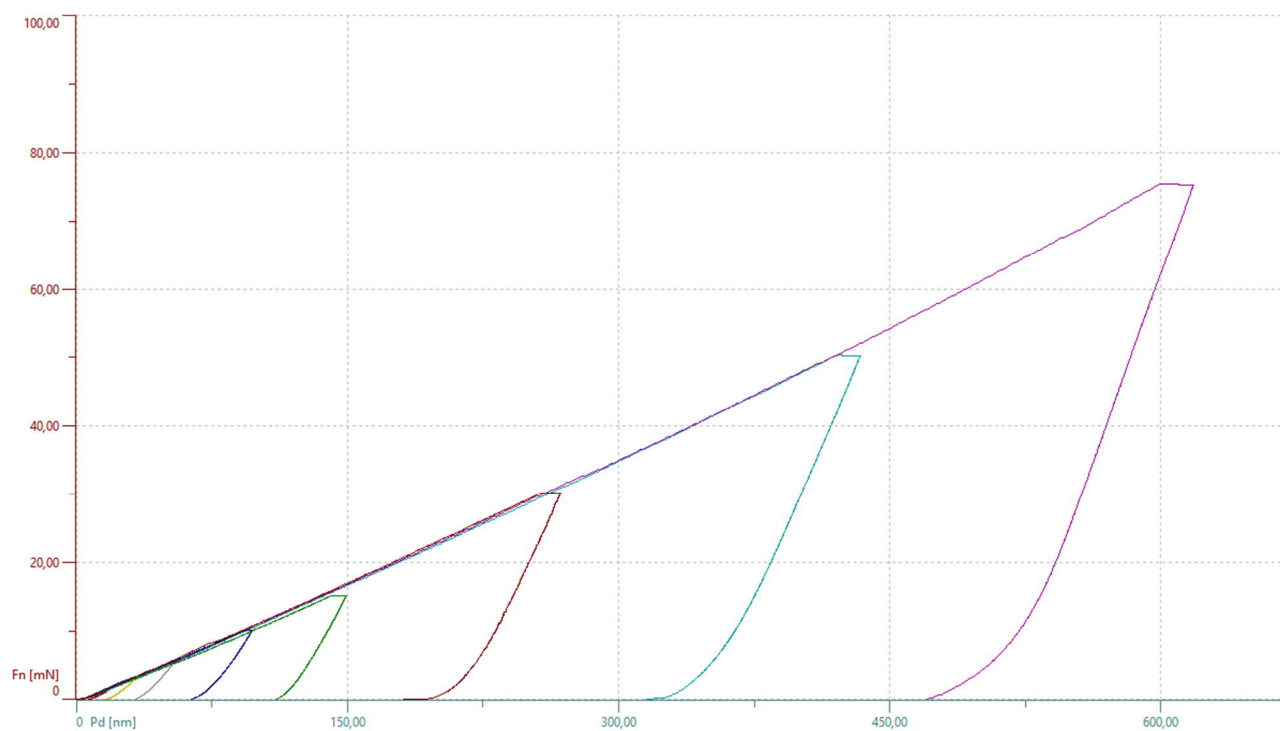


Figure 6. Mean force-penetration graphs of indentations with loads 1-75mN carried out on sample "2" (flange).

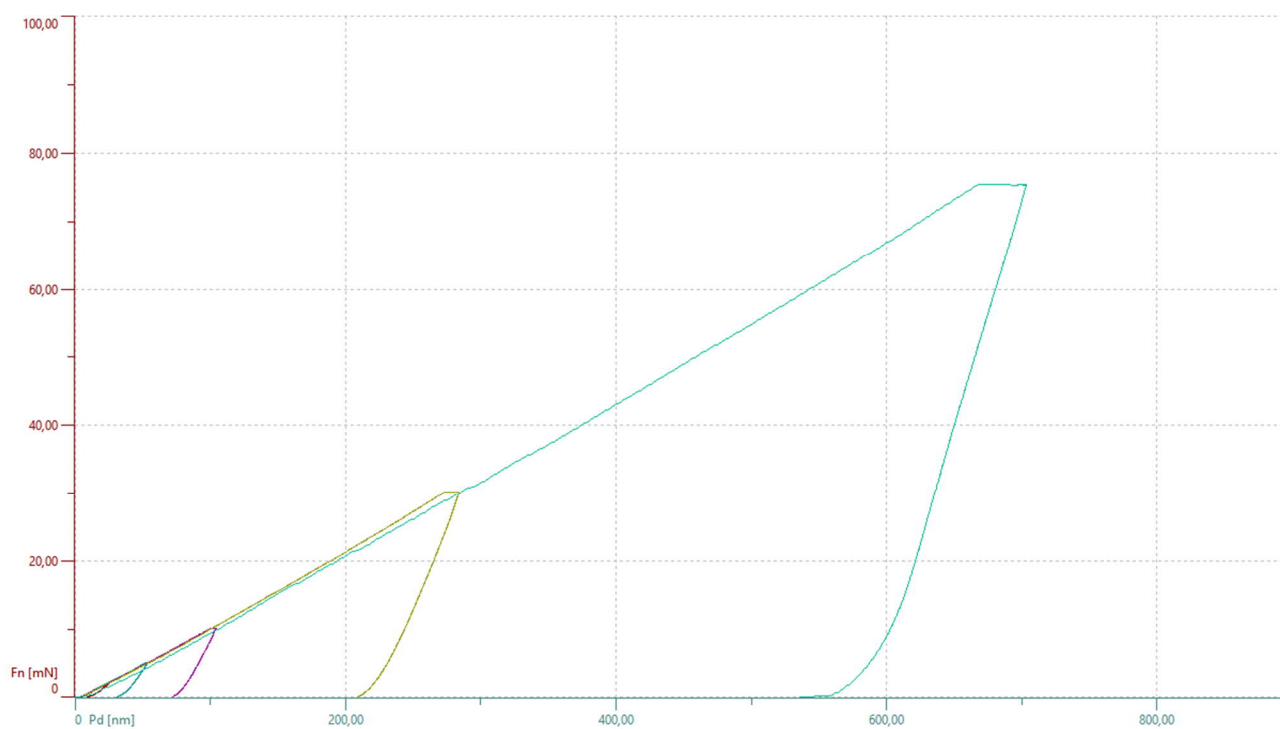


Figure 7. Mean force-penetration graphs of indentations with loads 2-75mN carried out on sample "4" (web).

The yield strength of the samples was found to be unrealistically high when the analysis procedure described in [7] was applied to analyze the stress-strain values on the basis of the individual indentations. The yield point when assumed to be the intersection point of the line with E^* as the slope and curve produced by the stress-strain point calculated from the indentations are found to be between 700 and 800 MPa in both cases when estimated from the Figure 8 and Figure 9. It seems that the procedure need deeper analysis to yield more realistic tensile strength values. Next step would be to analyze the indentation marks more deeply especially the area under the indentation whether there is any sign of sink-in or pile-up that have an impact to the area under the indentation and thus the stress values calculated from the depth-penetration curves.

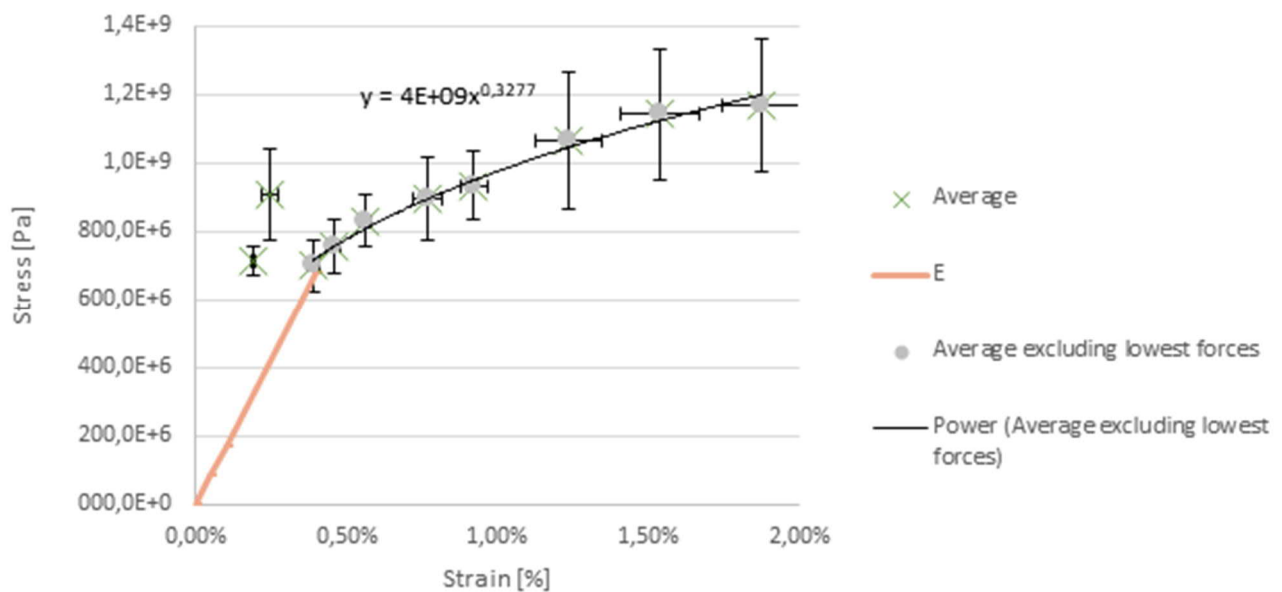


Figure 8. The approximated stress-strain curve for sample "2" (flange).

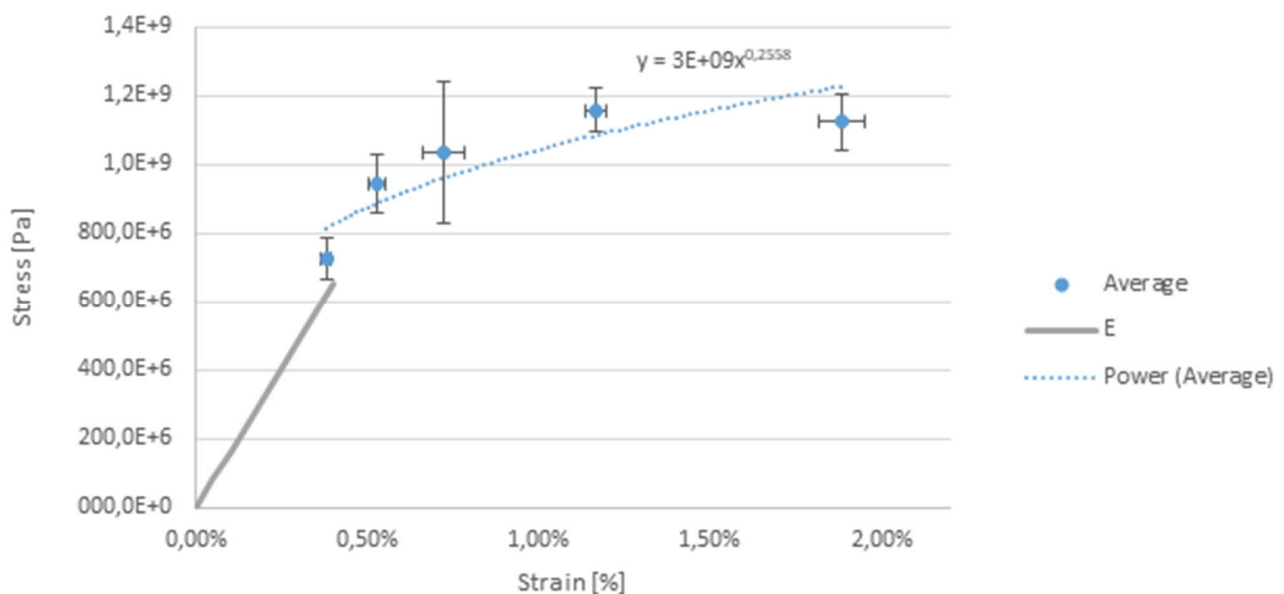


Figure 9. The approximated stress-strain curve for sample "4" (web).

2.2.4 Sub-sized impact toughness testing

Sub-size Charpy specimens 5 x 5 x 27 mm were extracted from the web and the flange of hot-rolled section *ST-P1*, two specimens from each. The impact energies at room temperature were then converted to full size (10 x 10 x 55 mm) Charpy impact energy equivalents according to Equations 7.26 and 7.27 from [4]. The results are summarised in Table 8

Table 8. Sub-size impact energies and their full size equivalents

	ST-P1-I		ST-P1-T	
Sample	Impact energy [J]	Full size equiv. [J]	Impact energy [J]	Full size equiv. [J]
1	18.47	129	14.02	91
2	17.45	121	13.29	84
Average	17.96	125	13.66	88
St.dev	0.510	4.0	0.365	3.5

2.2.5 Hardness testing

HV10 hardness measurements were conducted on the cross-section of the T-profile across the web and the flange and in the region of the intersection (see Figure 10). The hardness values were then converted to ultimate tensile strength equivalents according to EN ISO 18265 [5]. The results are summarised in Table 9.

Table 9. Hardness (HV10) test results and the ultimate tensile strength equivalents from cross-section web (ST-P1-I), flange (ST-P1-T) and their intersection (ST-P1-X).

	ST-P1-I		ST-P1-T		ST-P1-X	
Sample	HV10	R_m equiv. [MPa]	HV10	R_m equiv. [MPa]	HV10	R_m equiv. [MPa]
Average	179	572	154	492	157	499
St.dev	3.6		2.6		6.3	

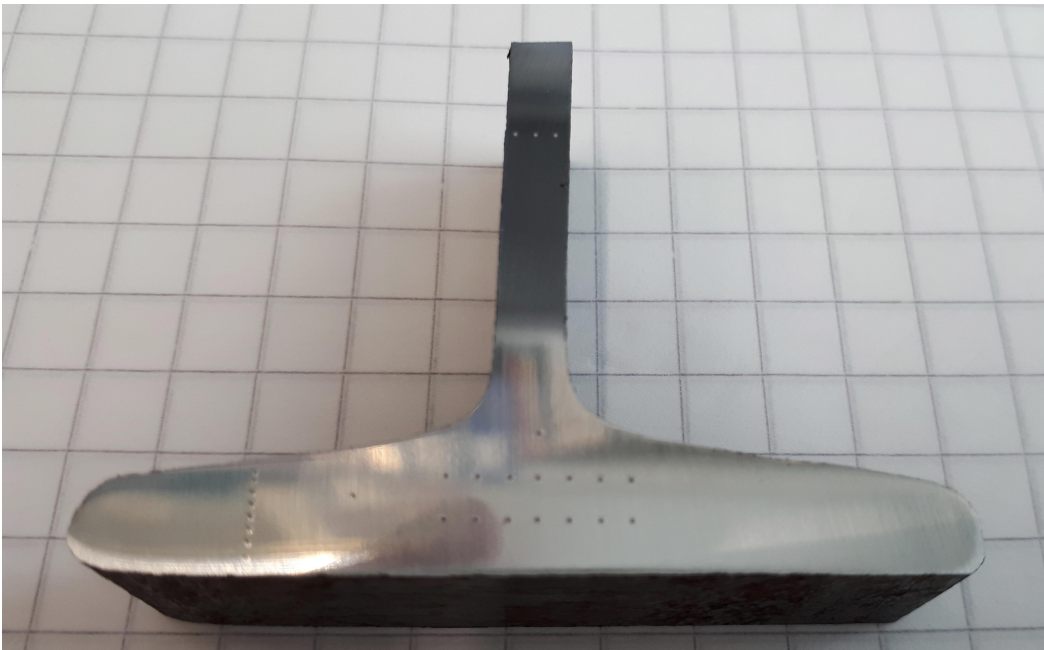


Figure 10 Points of hardness measurements

2.2.6 Summary of mechanical testing

The experiments conducted on the structural steel elements didn't indicate any significant degradation of the material properties in the extracted materials from ST-P1 to ST-P3 sections and the material quality seems to be the same as in the time of the fabrication of the steelwork that was tested. It can be concluded that structural steel section ST-P1 would qualify as S355JR grade. However, the tensile strength of the structural angle ST-C was slightly below the minimum value of USt 37-1 according to DIN EN 17100. Whether this can be accounted to the ageing of the material or some other effect could not be confirmed.

Two new methods for minimum-invasive experimental verification of steel tensile properties were tested. Small punch testing provided good prediction of the tensile strength and very small standard deviation of the results, but this method is not fully portable and needs specimens to be sent to the laboratory. It was more difficult to predict accurately the yield strength of work-hardened steel (ST-B2) On the other hand, prediction of the tensile strength by instrumented indentation was not very successful as this method needs more development for the conversion of measured results to the stress-strain relationship. The method can be potentially fully portable and more accurate than on-site hardness measurements.

The comparison of the prediction of yield and ultimate strength of the material is in Figure 11 and Figure 12 for bars ST-B1 and ST-B2 and in Figure 13 and Figure 14 for structural section ST-P1.

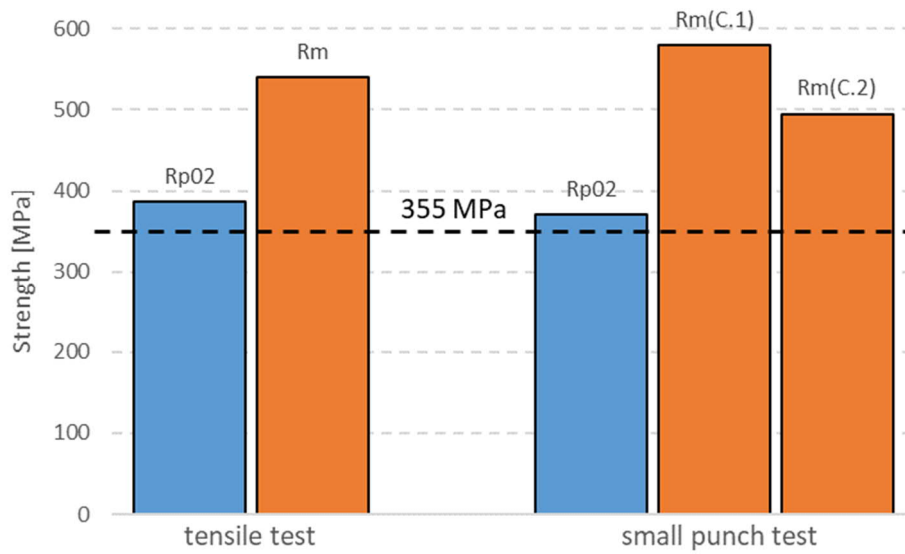


Figure 11 Comparison of mechanical test results of 24 mm stud ST-B1

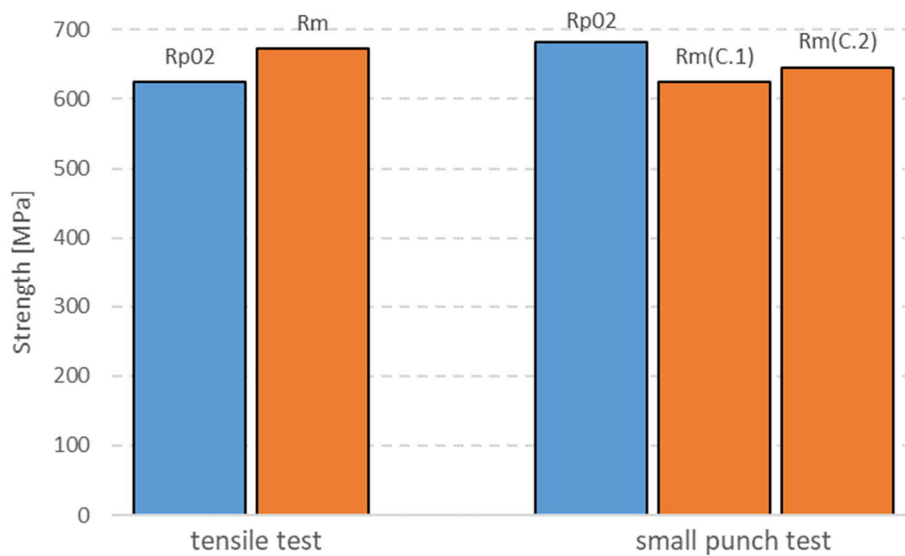


Figure 12 Comparison of the test results of 18 mm tie ST-B2

From the initial testing of Small Punch method on steel bars, it can be seen that the calculation of tensile strength according to method C.2 (Equation (3)) is more accurate, and therefore this method was used in the further tests on the structural profiles recovered from the single-storey steel hall.

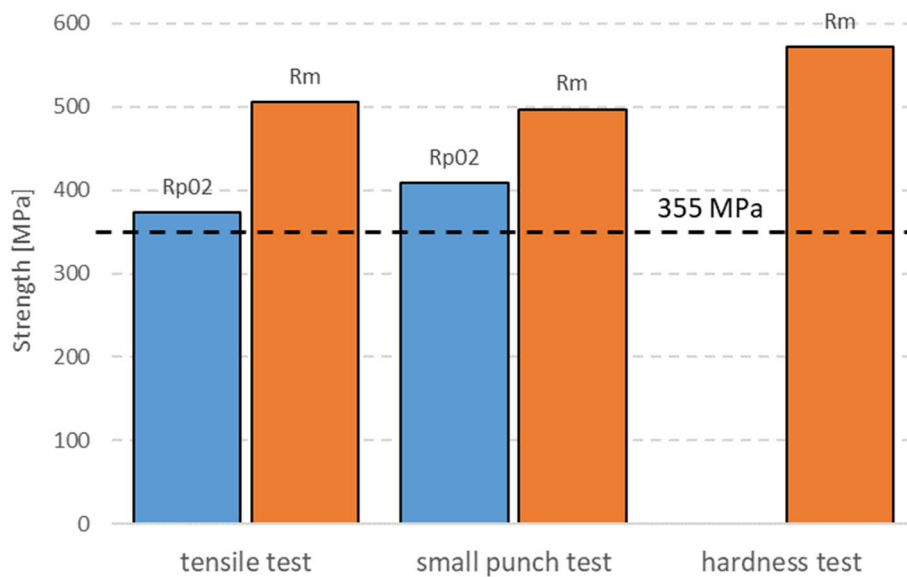


Figure 13 Comparison of the test results of I-section ST-P1-I (web)

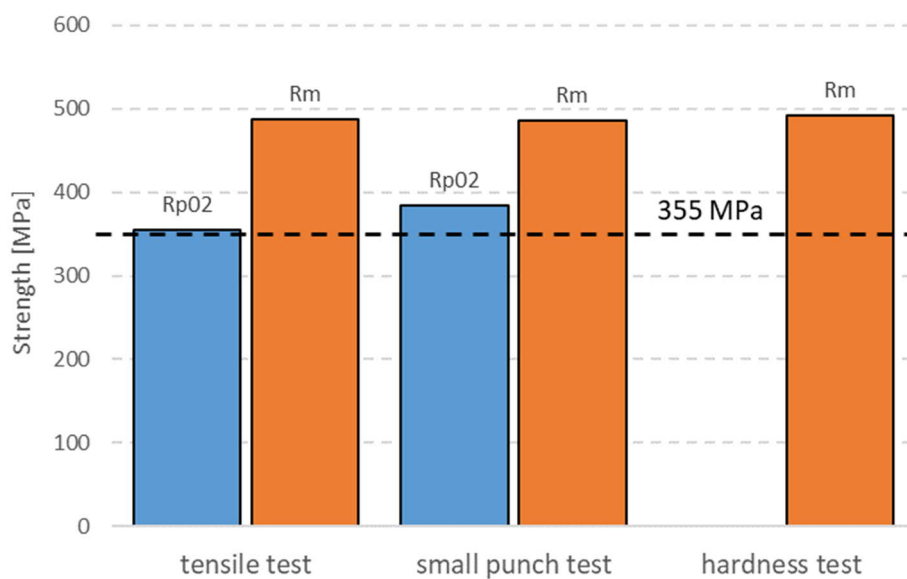


Figure 14 Comparison of the test results of I-section ST-P1-T (flange)

2.3 Testing of chemical properties

The chemical composition of the sample extracted from the structural angle ST-C was verified and the results expressed as element's mass fraction are presented in Table 10. Then the macrographic examination by sulphur print (Baumann method) was conducted on the cross-section (see Figure 15). The results were compared with the limits specified in DIN 17100 [6] for the steel U St37-1. St37 is a basic unalloyed low-carbon steel grade with the tensile strength 370 MPa.

Table 10. Chemical testing results of sample ST-C

Element	Mass fraction [%]	Specification of USt37-1 according to DIN 17100
C	0.034	max. 0.25
Si	<0.01	-
Mn	0.29	-
P	0.067 ¹	max. 0.090
S	0.070 ²	max. 0.063
Cr	<0.01	-
Mo	<0.01	-
Ni	0.03	-
Al	<0.001	-
Cu	0.04	-
Nb	<0.002	-
Ti	<0.002	-
V	<0.002	-
N	0.02143 ³	-

¹ Maximum value ~0.109%

² Maximum value ~0.102%

³ Maximum value ~0.0273%

As can be seen from Table 10, several elements were higher than the limits for the mild steel U St37-1 according to DIN 17100 [6], but the macrographic examination on Figure 15 did not reveal any defects, porosity, cracks or other irregularities in the cross-section. Such profile has to be excluded from welding, but if the mechanical properties are satisfactory, it might be able to carry the mechanical load. It would be impossible to obtain CE marking for this material, as there is no harmonized standard or ETA approval to which the performance can be declared.

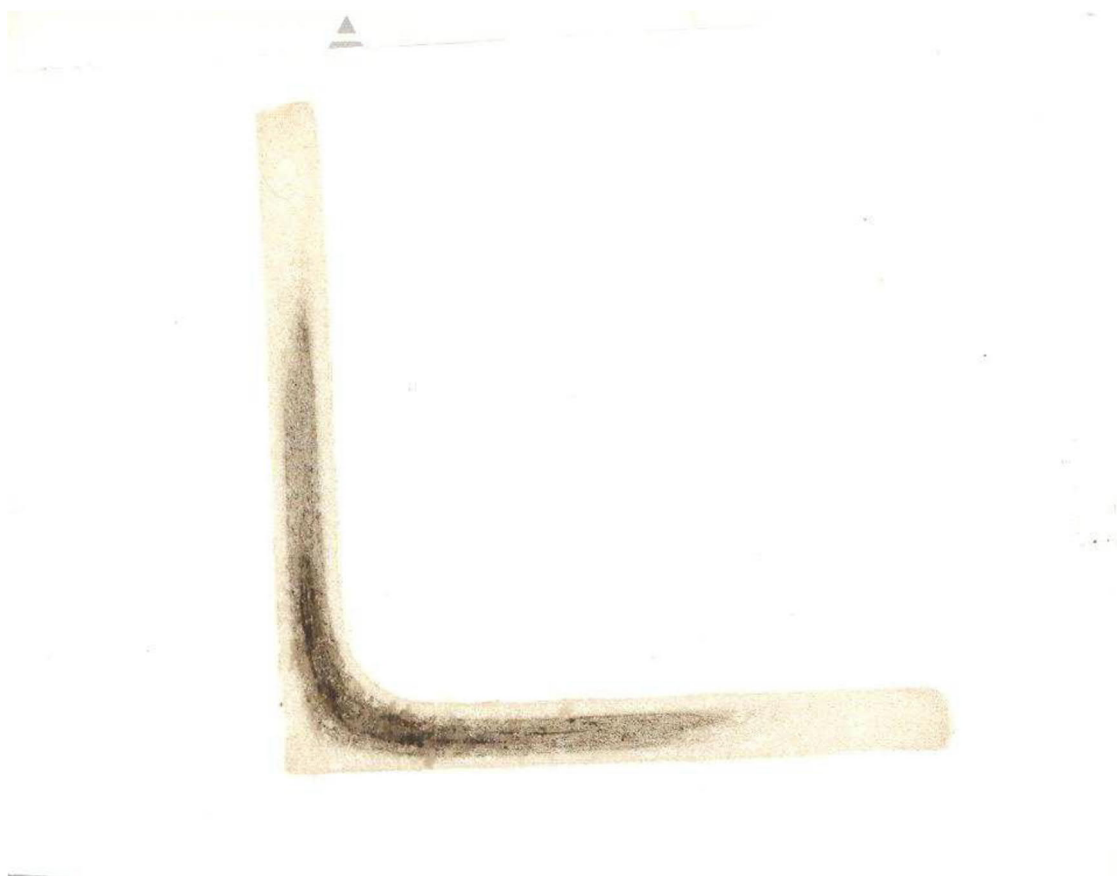


Figure 15 Sulphur print for the macrographic examination

3 Verification of the quality of the envelope

3.1 Test specimens

3.1.1 Mineral wool (MW) panels

Mineral-wool sandwich panel tests for used panels were carried out in Finland in 2018. The tested panels were dismantled from experimental building (See Figure 3-1) located in Hämeenlinna, Finland. The building was used for experimental research for long-term behaviour of sandwich panels during 2000-2007. After the research, the building was equipped with new vertical installed 150 mm thick and 1200 mm width mineral-wool sandwich panels in 2007. In that time also 0-series structural factory tests according to EN14509 standards were conducted to the same batch of panels as installed. Thus, there are available reference values to the 2018 tested values and the ageing behaviour of the panels was possible to evaluate. Further to structural testing, also lambda-tests for core material and coating tests for external steel sheet were made. Removed panels were in southern and northern façade. The lengths of the panels were 3,8 m and 5,1 m. Three whole width (1200 mm) panels were in both facades. Furthermore some longitudinally cut panels were also utilized for small sample tests. Figure 16 shows southern façade of the building where the panels were removed.



Figure 16 Test building from where sandwich panels were dismantled (South façade)

Dismounting process

The dismounting process started with marking of the panels, planning of dismounting order and preparing transporting packages. The dismounting was realised in 25th to 26th of June 2018. Temperature was about 18 to 24°C and weather was half-cloudy without raining during the dismounting. First, all flashings were removed carefully and panels were visually inspected. Figure 17 shows preparation of dismounting. The panels were one span panels screwed only in the ends with three screws per end. The panels were extracted from the building frame and lifted with crane with help of panel lifting device carefully onto a timber forklift pallet. The dismounting went smoothly without damages to the panels (see Figure 18). The packages were well protected for transportation and transported into the panel factory in Alajärvi, Finland. Panel marking and numbering is shown in Figure 19.



Figure 17 Preparing dismounting



Figure 18 Dismounting of panels with crane.

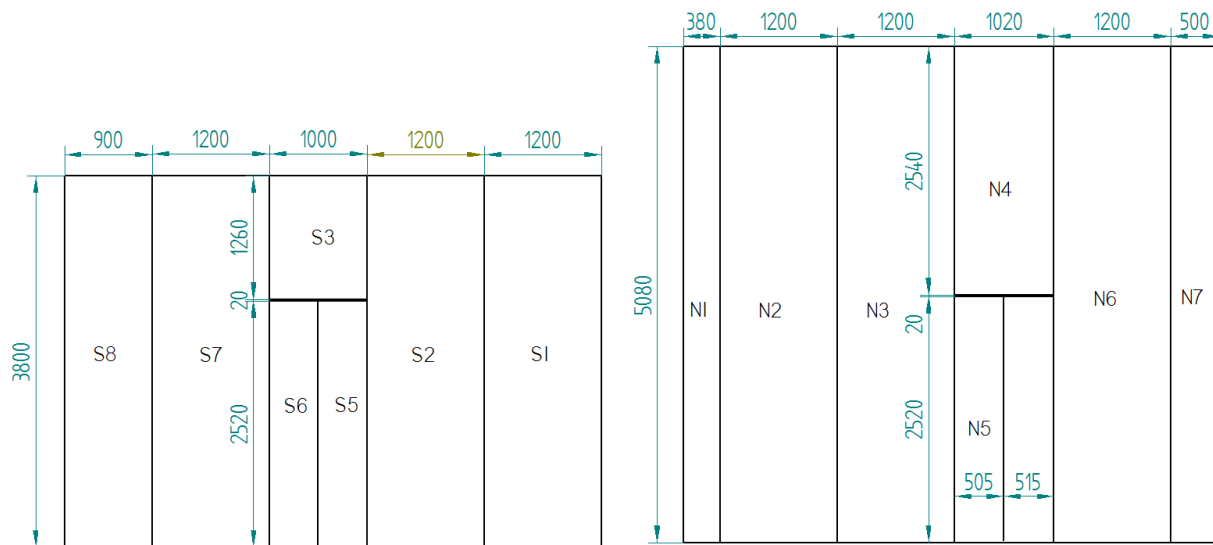


Figure 19 Panel marking from outside (S=South, N=North). Panel N4 is from different production patch.

3.1.2 Polyurethane (PIR) panels

In 2005 the research building Sandwich-Demo-House was built by the Institute of Steel and Lightweight Metal Construction of RWTH Aachen University in cooperation with the International Federation for Lightweight Metal Construction (IFBS) in Aachen. The aim of this project was to test various aspects of lightweight metal construction under real boundary conditions. Various building physics investigations were carried out in the sandwich demo house. Furthermore the use of different sandwich manufacturers and the different laying of the sandwich elements led to further empirical values in the construction method. Figure 20 shows the Sandwich-Demo-House.



Figure 20 Sandwich-Demo-House

The research building was built in lightweight metal construction. For each wall a different sandwich manufacturer was chosen as already described. The wall elements have a micro-profiled outer shell and a lined inner shell. The outer shells of all elements are 0.6 mm thick and the inner shells are 0.5 mm thick. The south and west walls were installed horizontally, and the north and east walls were installed vertically. Seven sandwich elements were used on each side. A different colour was chosen for each of the four walls to investigate their influence on the properties of the walls. The sandwich

elements on the north side are blue, those on the east side are green, the south side is yellow, and the west side is red.

As part of the project, the south and west walls were dismantled and then tested for their mechanical properties in accordance with EN 14509 [9]. This allows a comparison to be made with the target values of the WPK tests from the approvals and the evaluation of reuse. Thereby, the following values were determined:

- Tensile Strength- and Tensile Modulus
- Compressive Strength and Compressive Modulus
- Shear Strength and Shear Modulus
- Wrinkling Stress

Figure 20 shows the dismantling process of the south- and westside of the Sandwich-Demo-House.



Figure 21 Dismantling Process of Sandwich-Demo-House

The following illustration shows the designation and position of the panels that were used for the tests. For both walls, only the upper elements could be used for the bending tests, as only these had the necessary length and no window recesses.

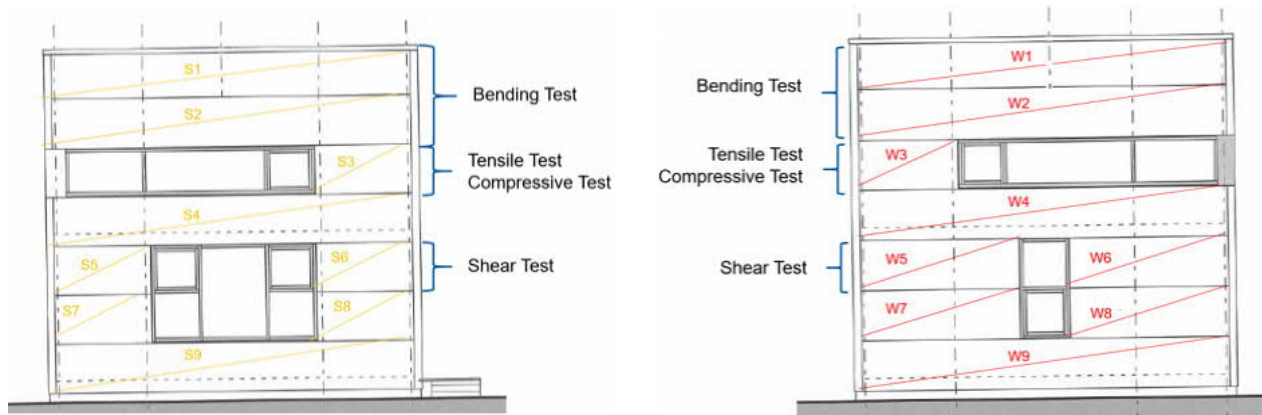


Figure 22 Designation and Position of the Panels used for testing

3.1.3 One-sided fasteners

The following fasteners illustrated in Figure 23 were used in the test series:

- SL2-T-A14-4.8x20 (self-drilling screw) (marked as 'SL2')
- SL3-H15-6.3x32 (self-drilling screw) (marked as 'SL3')
- SL3-H15-S16-6.3x32 (self-drilling screw) (marked as 'SL3-S16')

Self-drilling screws were manufactured by SFS Intec.

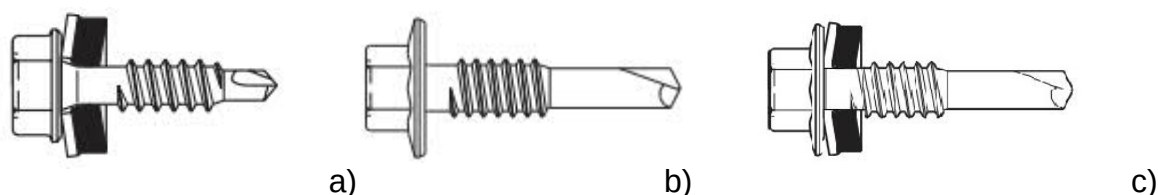


Figure 23 Fasteners studied in the tests: a) SL2-T-A14-4.8x20, b) SL3-H15-6.3x32, c) SL3-H15-S16-6.3x32

The following sheet steels were fixed with the studied fasteners to the exterior faces of sandwich panels:

- S280GD + Z275 PE; nominal thickness 0.6 mm (marked as "S280 t=0.6 mm")
- S350; nominal thickness 1.0 mm (marked as "S350 t=1.0 mm")

Sandwich panel 150R50 pieces were tested. The nominal thickness of the steel surface was 0,6 mm. Test samples were cut from different panels: 5 tensile and 5 shear test samples were cut from the sandwich panel which was facing north (marked as "N4", "N5") and same number of samples was cut from the sandwich panel which was facing south (marked as "S1", "S3"). In shear tests the nominal size of the test specimen was 400 mm × 200 mm × 150 mm and in the tensile tests 400 mm × 400 mm × 150 mm. Test combinations and number of tests are presented in Table 3-11 Tested panel side, fasteners, fixed sheet metals and number of tests according to the loading type studied in the test programme Table 11.

Table 11 Tested panel side, fasteners, fixed sheet metals and number of tests according to the loading type studied in the test programme

Test	Panel	Fastener	Sheet metal fixed to the face	Number
Shear	N4	SL3	S350 (t=1.0 mm)	5
	S3	SL3		3
Tensile	N5	SL2	S280 (t=0.6 mm)	1
	N4	SL3	S350 (t=1.0 mm)	1
				3
	S1	SL2	S280 (t=0.6 mm)	2
		SL3	S350 (t=1.0 mm)	3

3.2 Structural tests

Structural tests of MW panels were carried out at Alajärvi panel factory in Finland, PIR panels were tested in RWTH Aachen in Germany.

Structural testing included following measurements and tests according to EN 14509 [9]:

- Density of the core material
- Compressive strength and modulus of elasticity of core material
- Cross panel tensile test
- Vacuum chamber tests for whole panel (bending or shear strength)

Because the limited number of MW specimen and actual lengths, separate bending and shear tests were not able to carry out, but only vacuum chamber tests were made and failure mode was depended on the panel dimensions. Material properties for the panels as follows:

- External facing: S320GD+Z /0.6 mm, polyester or PVDF* coating, colour dark grey RR23
- Internal facing: S320GD+Z /0.6 mm, polyester coating, colour white RR20
- Core: constructive stone wool lamella-core; CES50CS100 (Paroc Oy Ab)

*Panel N:o N4

3.2.1 Density of the core material

Measured densities of mineral wool pieces are shown in Table 12. Both results from 2007 and 2018 measurements are presented. Nominal density of Paroc CES50CS100 MW-grade is 115 kg/m³. An average value for original panels was 124.8 kg/m³ whereas for demounted panels it was 115.8 kg/m³. Difference is about 7%. There is no remarkable difference between north and south façade panels.

Table 12 Measured densities of mineral wool [kg/m³]

Year	2007	2018									
Sample	MW Ref	MW S1	MW S2	MW S3	MW S6	MW S7	MW S8	MW N3	MW N4*	MW N5	MW N6
1	122.5	114.0	114.0	112.0	124.0	118.0	113.0	115.0	118.0	115.0	116.0
2	121.0	118.0	116.0	117.0	114.0	112.0	112.0	116.0	116.0	117.0	116.0
3	122.0	114.0	115.0	120.0	119.0	116.0	112.0	117.0	112.0	115.0	114.0
4	130.5	112.0	111.0	114.0	119.0	120.0	113.0	116.0	116.0	117.0	118.0
5	125.5	114.0	115.0	115.0	119.0	116.0	113.0	116.0	122.0	118.0	117.0
6	127.5	115.0	116.0	118.0	120.0	116.0	112.0	118.0	120.0	117.0	117.0
Average	124.8	114.5	114.5	116.0	119.2	116.3	112.5	116.3	117.3	116.5	116.3

*Not included to analyses

3.2.2 Compression tests

The compressive test is used to determine the compressive strength and modulus of the panel core. The compressive strength f_{Cc} and the modulus E_{Cc} are calculated according to equation (1).

$$f_{Cc} = \frac{F_{10\%}}{A} \text{ and } E_{Cc} = \frac{\Delta F d_c}{\Delta w A} \quad (1)$$

where $F_{10\%}$ is applied force at 10 % compression, A is cross section of the specimen, $\frac{\Delta F}{\Delta w}$ is the slope of the linear part of the force-compression curve, d_c is thickness of the panel.

The PIR panel specimens were placed between two stiff steel plates. The deflection rate was defined with 2 mm/min. The specimen was loaded up to a compression of 10%. A load-compression curve and the applied force $F_{10\%}$ were recorded. MW specimen dimensions are 124 x 124 mm. Measured values of compression strength are shown in Table 13 and in more detail in Annex B.

Table 13 Measured values for compression strength of mineral wool [MPa]

Year	2007	2018							2020	
Sample	MW Ref	MW S1	MW S3	MW S6	MW S7	MW S8	MW N4*	MW N5	PIR S3	PIR W3
1	0.117	0.116	0.104	0.112	0.124	0.101	0.124	0.098	0.125	0.165
2	0.125	0.108	0.111	0.119	0.127	0.121	0.123	0.103	0.117	0.159
3	0.152	0.105	0.107	0.101	0.120	0.124	0.118	0.127	0.131	0.163
4	0.144	0.103	0.116	0.131	0.123	0.116	0.111	0.097	0.129	0.164
5	0.101	0.113	0.114	0.100	0.111	0.120	0.112	0.087	0.140	0.162
6	0.102	-	-	-	-	-	-	-	-	-
7	0.100	-	-	-	-	-	-	-	-	-
8	0.098	-	-	-	-	-	-	-	-	-
9	0.118	-	-	-	-	-	-	-	-	-
10	0.123	-	-	-	-	-	-	-	-	-
Average	0.118	0.109	0.111	0.112	0.121	0.117	0.117	0.102	0.128	0.163
St.dev	0.019	0.005	0.005	0.013	0.006	0.009	0.006	0.015		

*Not included to analyses

Mean value of all MW tested values in 2018 was 0.112 N/mm² whereas standard deviation was 0.011 N/mm². It can be concluded that compression strength was only 5% lower than the original one from 2007. The declared 5% fractile of PIR panels compression strength 0.100 MPa is lower than the 5% fractile of the measured strength 0.109 and 0.157 MPa for south and west panels respectively.

Measured elastic moduli in compression tests are shown in Table 14 and in more detail in Annex B.

Table 14 Measured values for elastic moduli in compression tests [MPa]

Year	2007	2018							2020	
Sample	MW Ref	MW S1	MW S3	MW S6	MW S7	MW S8	MW N4*	MW N5	PIR S3	PIR W3
1	7.0	10.1	5.9	4.4	11.8	8.3	11.1	5.5	3.01	3.66
2	7.0	10.3	7.8	10.2	10.1	10.1	6.3	8.5	3.00	3.75
3	11.0	11.0	5.2	6.7	8.4	11.0	6.9	5.2	3.07	3.58
4	9.0	11.5	5.8	11.5	13.4	9.8	9.8	4.0	3.20	3.59
5	8.0	13.0	7.6	10.0	9.6	10.7	6.6	6.9	3.15	3.77
6	6.0	-	-	-	-	-	-	-	-	-
7	6.0	-	-	-	-	-	-	-	-	-
8	6.0	-	-	-	-	-	-	-	-	-
9	12.0	-	-	-	-	-	-	-	-	-
10	7.0	-	-	-	-	-	-	-	-	-
Average	7.9	11.2	6.5	8.5	10.6	10.0	8.1	6.0	3.09	3.67
St.dev	2.1	1.2	1.1	2.9	2.0	1.1	2.2	1.7		

*Not included to analyses

Average value of all MW tests from 2018 is 8.8 N/mm² that is bit higher than the 2007 result. Declared 5% fractile of modulus of elasticity of the west PIR panel 2.5 MPa is lower than the 5% fractile of the tests 3.46 MPa. However, the declared values for south PIR panels 2.93 MPa and 3.57 MPa for 5% fractile and average modulus of elasticity respectively are higher than the measurements.

3.2.3 Tensile tests

In this test series, the tensile strength f_{ct} and the tensile modulus E_{ct} will be determined. The tensile strength f_{ct} and the tensile modulus E_{ct} is calculated according to equation (2). Thereby, it is necessary to say, that the standard EN 14509 [9] allows a decrease of the tensile test f_{ct} of 50 % during the ageing process.

$$f_{ct} = \frac{F_u}{A} \text{ and } E_{ct} = \frac{\Delta F d_c}{\Delta w A} \quad (2)$$

Where F_u is maximum tensile load, A is cross section of the specimen, $\frac{\Delta F}{\Delta w}$ is the slope of the linear part of the force-displacement curve and d_c is thickness of the panel. It should be noted that the standard EN 14509 [9] allows a decrease of the tensile test f_{ct} of 50% during the ageing process.

Samples were taken from the sandwich panels and tested in a tensile testing machine. The PIR specimens have dimensions of 150 x 150 mm and are 100 mm thick, MW specimens have dimension 124 x 124 mm. Measured values of tensile strength are shown in Table 15, elastic modules in tensile tests are shown in Table 16 and in more detail in Annex B.

Table 15 Measured values for tensile strength [MPa]

Year	2007	2018							2020	
Sample	MW Ref	MW S1	MW S3	MW S6	MW S7	MW S8	MW N4*	MW N5	PIR S3	PIR W3
1	0.072	0.112	0.103	0.021	0.113	0.111	0.060	0.083	0.078	0.071
2	0.071	0.098	0.102	0.082	0.105	0.097	0.090	0.109	-	0.076
3	0.046	0.113	0.083	0.088	0.127	0.068	0.058	0.113	-	-
4	0.049	0.073	0.055	0.088	0.109	0.057	0.049	0.114	-	-
5	0.060	0.117	0.092	0.084	0.078	0.068	0.110	0.094	0.076	0.101
6	0.040	-	-	-	-	-	-	-	0.064	0.077
7	0.044	-	-	-	-	-	-	-	0.059	0.078
8	0.093	-	-	-	-	-	-	-	-	-
9	0.034	-	-	-	-	-	-	-	-	-
10	0.052	-	-	-	-	-	-	-	-	-
Average	0.056	0.103	0.087	0.073	0.106	0.080	0.074	0.102	0.069	0.080
St.dev	0.018	0.018	0.020	0.029	0.018	0.023	0.026	0.013		

*Not included to analyses

Table 16 Measured values for elastic moduli in tensile tests [MPa]

Year	2007	2018							2020	
Sample	MW Ref	MW S1	MW S3	MW S6	MW S7	MW S8	MW N4*	MW N5	PIR S3	PIR W3
1	13.0	8.3	15.5	11.7	11.0	5.4	12.4	23.1	3.19	3.97
2	17.0	7.5	18.1	19.9	4.4	7.3	18.1	22.5	2.93	3.79
3	12.0	9.8	19.8	19.3	3.9	9.4	14.0	20.2	2.64	-
4	12.0	10.0	14.5	23.5	6.6	4.2	11.9	25.1	-	-
5	14.0	4.3	15.4	24.3	6.3	6.3	20.7	24.9	3.05	3.96
6	16.0	-	-	-	-	-	-	-	-	3.21
7	10.0	-	-	-	-	-	-	-	2.51	3.12
8	15.0	-	-	-	-	-	-	-	-	-
9	12.0	-	-	-	-	-	-	-	-	-
10	13.0	-	-	-	-	-	-	-	-	-
Average	13.4	8.0	16.7	19.7	6.5	6.5	15.4	23.2	2.86	3.61
St.dev	2.1	2.3	2.2	5.0	2.8	2.0	3.8	2.0		

*Not included to analyses

Mean value of all 2018 tested values was 0.092 N/mm² whereas standard deviation was 0.023 N/mm². It should be noted that the tension strength of this panel patch seems to be quite low and would not meet current requirements. Tensile strength is not usually critical factor in panel design and thus this information does not have that importance. Anyway, it can be concluded that average tensile strength of 2018 measured values was 59% higher than the original one. Average modulus of elasticity of all 2018 test is 13.4 N/mm² that is same as in 2007 result, but on the other hand, variation is quite high.

3.2.4 Bending tests

The bending test is used to determine the bending loading capacity and the associated wrinkling strength with a 6-point-bending test. For this, the limit load F_u is considered, to which the dead weight of the sandwich panel and the load introduction must be added. The bending moment M_u and the wrinkling strength σ_w is calculated according to equation (3).

$$M_u = \frac{F_u L}{8} \text{ and } \sigma_w = \frac{M_u}{e A_{F1}} \quad (3)$$

Where F_u is the maximum load, L is span length, M_u is maximum bending moment, e is the distance between the centres of gravity of the surface layers and A_{F1} is cross section of the face in compression.

Bending tests of MW panels were carried out with vacuum box to simulate equal loading (EN14509 [9] A.5). Test results are shown in Table 17 and in more detail in Annex B.

Table 17 Measured values of wrinkling strength in bending tests [MPa]

Year	2007	2018	2020	
Sample	MW Ref	MW N2, N3, N6	PIR S1, S2	PIR W1, W2
1	150	-	136	153
2	147	144	185	167
3	150	140	-	-
4	166	-	-	-
5	152	-	-	-
6	-	127		
Average	153		161	160
St.dev	7.5			

Failure mode in all MW tests was wrinkling failure partly inwards (2007 tests) and partly outwards (2018 tests) as presented in figures in Annex B and their wrinkling strength was lower than the strength measured in 2007. The measured strength of PIR panels is, on the other hand, higher than its declared value 142 MPa and 131 MPa for south and west panels respectively.

3.2.5 Shear tests

To determine the shear strength and the shear modulus, the load is applied to generate pure shear stress in the test specimen. EN 14509 A3.3 recommends a small span of 1000 mm to achieve a typical shear failure. The width of the specimen was 100 mm. The shear strength f_{cv} and the shear modulus G_c is calculated by equation (4).

$$f_{cv} = \frac{F_u}{2Be} \text{ and } G_c = F_u \frac{L}{6A_c \Delta w_s} \quad (4)$$

Where F_u is the maximum load, B is width of specimen, e is the distance between the centres of gravity of the surface layers, Δw_s is the slope of the linear part of the load-deflection curve, A_c is cross section of the core and L is span length of the specimen.

Shear tests of MW panels in 2007 test series were carried out according to EN14509 [9] Clause A.4. The 2018 tests were realized with vacuum box according to EN14509 [9] Clause 4.5. Test results are shown in Table 18 and in more detail in Annex B.

Table 18 Measured values of shear strength and shear moduli in shear tests

	Shear strength [MPa]						Shear modulus [MPa]					
Year	2007	2018			2020		2007	2018			2020	
Sample	MW Ref	MW S1	MW S2	MW S7	PIR S6	PIR W5	MW Ref	MW S1	MW S2	MW S7	PIR S6	PIR W5
1	0.063	0.046	0.066	0.070	0.130	0.152	2.5	2.0	2.7	2.9	3.161	3.359
2	0.064	-	-	-	0.131	0.144	2.7	-	-	-	2.934	3.082
3	0.067	-	-	-	0.137	0.148	2.8	-	-	-	2.864	2.928
4	0.067	-	-	-	0.131	0.151	2.4	-	-	-	2.782	3.301
5	0.068	-	-	-	-	-	2.5	-	-	-	-	-
6	0.072	-	-	-	0.135	0.147	2.5	-	-	-	2.728	3.066
7	0.071	-	-	-	0.138	-	2.8	-	-	-	2.961	-
8	0.066	-	-	-	-	-	2.6	-	-	-	-	-
9	0.065	-	-	-	-	-	2.5	-	-	-	-	-
Average	0.067				0.134	0.148	2.6				2.905	3.141
St.dev	0.003											
5% fract.					0.126	0.141					2.568	2.731

Failure mode of all MW specimens was shear failure at lower face.

3.3 Tests on mechanical fasteners to one face of sandwich panel

The test programme was carried out to obtain information regarding the resistance of the fasteners used to fix additional elements to a face layer only of the sandwich panels. Test panels were old panels removed from the test building. Three types of self-drilling screws were studied in the test programme. The resistances with respect to tensile and shear loads were studied. The total number of individual tests was 18. Test programme was performed in the research hall of HAMK Tech Research Unit in Hämeenlinna between 8 and 14 May 2019. The tests were made by Mr. Silver Jänes and Mr. Lev Antimonik from HAMK University of Applied Sciences.

Test arrangement

The test specimens were tested by loading the fastening by a tensile or shear force. The loading was caused by the universal testing machine Zwick 50 kN. A special test jig was manufactured and fixed to the lower head of the testing machine. Test specimens were supported by the jig. The force applied to the specimen and the vertical displacement of the upper head of the testing machine was measured.

The arrangements are illustrated in Figure 24 in shear and in Figure 25 in tensile tests. In shear tests the width of the sheet metal strip, which was fixed to the panel by the studied fastener, was 30 mm. The free end of the strip was attached to the upper head of the testing machine. In tensile tests the upper head of the testing machine was attached to the tested screw using a special U-shaped fixing part. The studied fixture went through the bottom of the fixing part into the face of the panel. The head of the testing machine was connected to the fixing part using a M10 8.8 bolt.

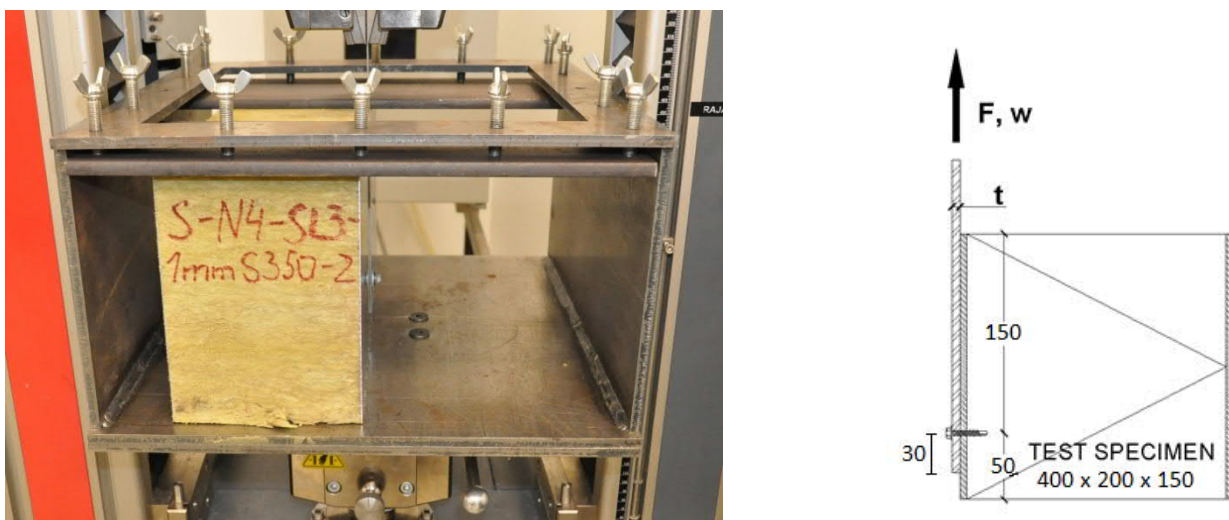


Figure 24 Test arrangements in shear tests

After tests on fasteners, 6 tensile test samples were extracted from exterior face layer material of the sandwich panel pieces: 3 from north and 3 from south side specimens. Samples were cut from panel pieces used in tensile tests. Face material is S280GD + Z275 PE. Purpose of these tests is to obtain information about material properties of surface material. Dimensions of test sample are presented in Figure 26.

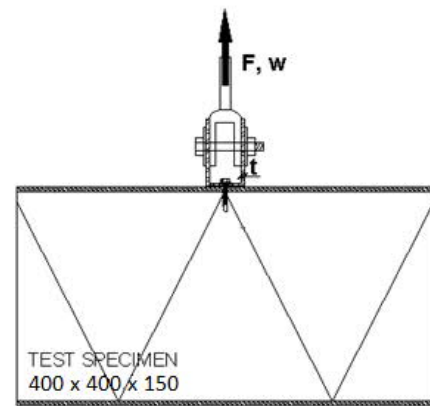
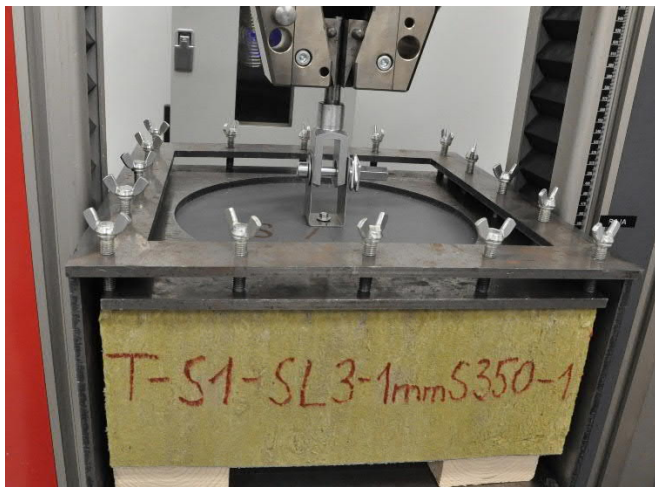


Figure 25 Test arrangement in tensile tests

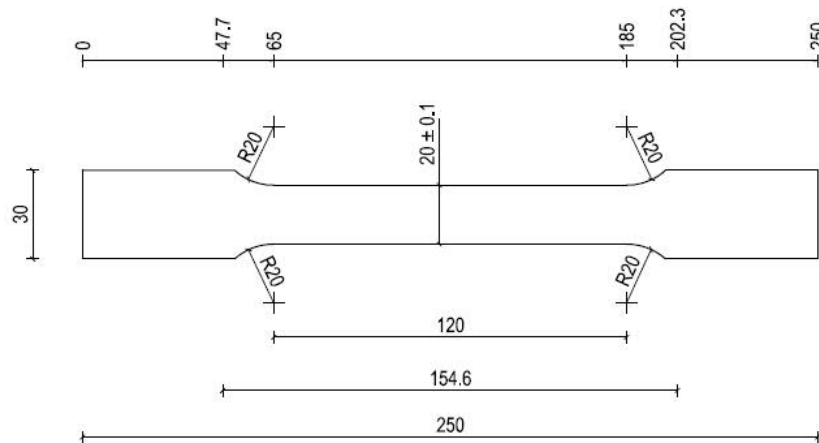


Figure 26 Tensile test sample for surface sheet material tests

Results

Table 19 gives the mean resistances of the tested connections. Load versus displacement graphs are given in Appendix 2 and photographs in Appendix 3.

In shear test the failure mode was the bearing failure of the face sheet, and finally, the screw came out (i.e. pull-out failure). In the similar cases the resistances in the case of SL3-screw were higher than the resistances in the case of SL2-screw. S-N4-SL3-1mmS350-1 test was done with different test arrangement with steel plate with round hole in the middle (see Figure 47). Due to this panel's face layer has bent in several places during the test, test arrangement was changed after that. Photographs of test arrangement and panel damages are shown in Appendix 3. Plate with hole was substituted with two $400 \times 100 \times 10$ mm plates at the sides of the panel. Image of new arrangement is presented in Figure 26.

In tensile tests the failure mode was pull-out failure in all cases. The resistances of the screws were at the same level and almost independent of the fixing plate type.

Tensile test results of surface material samples are presented in Table 20. Samples were extracted from T-N4-SL3 and T-S1-SL3 panel pieces. Stress versus strain graphs can be found in Appendix 3.

Table 19 Test results for shear and tensile tests in one face fixing tests

Sample	Test type	Panel	Fastener	Sheet metal fixing	Force [N]	Average [N]	St.dev [N]
S-N4-SL3-1mmS350-1	Shear	N4	SL3	S350 t=1.0 mm	3053.2	3139.98	136.57
S-N4-SL3-1mmS350-1-1					3157.3		
S-N4-SL3-1mmS350-2					3174.9		
S-N4-SL3-1mmS350-3					3337.5		
S-N4-SL3-1mmS350-4					2977.0		
S-S3-SL3-1mmS350-1		S3			3373.1	3054.49	310.54
S-S3-SL3-1mmS350-2					3037.7		
S-S3-SL3-1mmS350-3					2752.7		
T-N4-SL3-1mmS350-1	Tensile	N4	SL3	S350 t=1.0 mm	1151.8	1184.26	60.10
T-N4-SL3-1mmS350-2					1253.6		
T-N4-SL3-1mmS350-3					1147.4		
T-S1-SL3-1mmS350-1		S1			1149.4	1153.85	17.95
T-S1-SL3-1mmS350-2					1173.6		
T-S1-SL3-1mmS350-3					1138.6		
T-S1-SL2-0_6mmS280-1		SL2	S280 t=0.6 mm	1161.9	1124.60	52.80	
T-S1-SL2-0_6mmS280-2				1087.3			
T-N4-SL2-0_6mmS280-1				N4	1125.1	1105.54	27.64
T-N5-SL2-0_6mmS280-2				N5	1086.0		

Table 20 Tensile test results for outer surface steel sheet

Sample	Direction	a_0 [mm]	b_0 [mm]	S_0 [mm ²]	E [GPa]	$R_{p0.2}$ [MPa]	R_{eH} [MPa]	R_{eL} [MPa]	R_m [MPa]
1	North	0.56	19.65	11	201	426	429	419	488
2	North	0.56	19.64	11	182	414	414	411	487
3	North	0.56	19.67	11.02	213	422	421	413	487
4	South	0.553	19.58	10.83	192	350	-	-	449
5	South	0.56	19.57	10.96	199	351	352	350	448
6	South	0.553	19.59	10.83	190	350	350	348	448

Test results were compared also to the tests for new panels performed in years 2009 and 2010. Current shear test results were almost 70% better than the test results in previous tests. It should be noted that the core material was not exactly same in these test series. In tensile tests difference was lower. In the case of 0.6 mm cases current mean results were exactly same as in the previous tests and in the case of 1.0 mm cases current results were 11% weaker than in 2009 and 2010 series.

3.4 Thermal resistance/conductivity (λ) tests

Lambda values of totally three pieces (600 mm * 600 mm * 140 mm) of mineral wool insulation were tested at Eurofins Expert Services in Finland. The test pieces were removed from Panels N4 and S7. Pictures of the test specimen are shown in Figure 27.

The tests were performed according to the standard EN 12667 [10], "Building materials – Determination of thermal resistance by means of guarded hot plate and heat flow meter methods – Products of high and medium thermal resistance". The apparatus used was in accordance with the standard ISO 8301 [11].

During the test, the specimen was in horizontal position and the heat flow was vertical. The tests were performed at mean temperature of +10 °C. The test results are shown in Table 21.



Figure 27 Lamda test specimen

Table 21 The thermal conductivities for the studied specimen

Product	Thickness d (mm)	Dry weight m ₁ (g)	Weight after test m ₂ (g)	Dry density ρ kg/m ³	Mean temperature T _m °C	Temperature difference ΔT K	Density of heat flow q W/m ²	Thermal resistance R (m ² K)/W	Thermal conductivity λ ₁₀ W/(m · K)
S7-1	143,0	6118	6120	116,9	9,94	20,07	5,832	3,440	0,0416
S7-2	143,3	6151	6148	116,9	9,93	20,05	5,832	3,438	0,0417
N4	143,6	6155	6152	117,7	9,93	20,06	5,845	3,433	0,0418

Mean value of the thermal conductivity was $\lambda_{10} = 0.0417$ W/(m·K). All the measured values were well lower than the declared value $\lambda_D = 0.045$ W/(m·K) (based on $\lambda_{90/90}$ of the manufacturer for the Paroc CES 50 CS100 insulation provided in the production time 2007).

4 Summary and conclusions

The report summarizes results of 269 experiments on structural steel sections and sandwich panels removed from four buildings of different age. The results of structural steel tests showed a good agreement with the expected values with the exception of specimen produced in 1930s before the current steel quality standards were developed, where the main barrier for reuse was its poor weldability due to the unsuitable chemical composition. The tests of sandwich panels showed signs of degradation caused by ageing, but the main problem was typically large variation of the test results.

The second goal of this study was to demonstrate the full package of minimum invasive material tests that will not compromise the integrity of structural components and can be performed before deconstruction. Hardness, chemical and sub-sized impact tests are already well-established, but the methods to determine material yield and tensile strength and ductility are still being developed. In this report, two experimental approaches were used for this purpose: small punch test and instrumented indentation test. According to the literature review, both methods were already successfully tested for similar applications and both methods can be used on-site with portable testing apparatus. Moreover, the small punch test method is in the standardization process.

It can be concluded that small punch tests could determine yield and tensile strength with acceptable accuracy and the variation of the test results was surprisingly low. On the other hand, it was impossible to use the documented method for prediction of material strength from the indentation tests. Therefore, it is recommended to study the indentation method further. Both test methods provide load-displacement measurements that can be alternatively converted into stress-strain relationship using more sophisticated reverse Finite Element Modelling.

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- [11] International Organization for Standardization, ISO 8301:1991 Thermal Insulation – Determination of steady-state thermal resistance and related properties – Heat flow meter apparatus, Geneva, Switzerland, 1991.

Annex A Small punch tests results

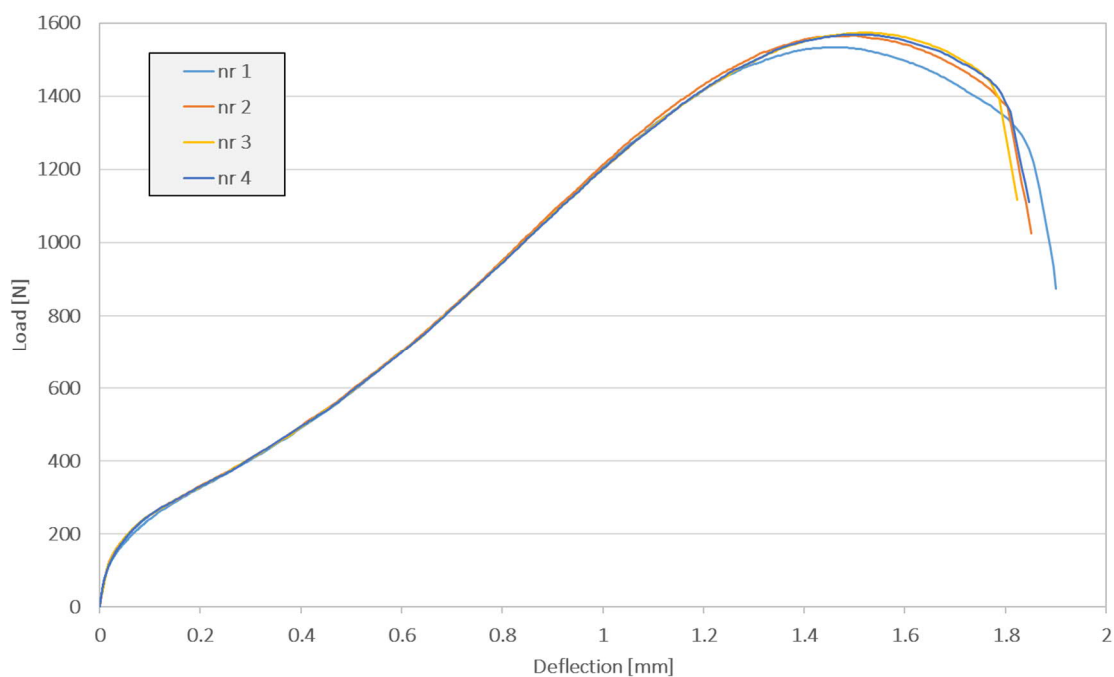


Figure 28. Load – deflection curves of tensile Sample 1.

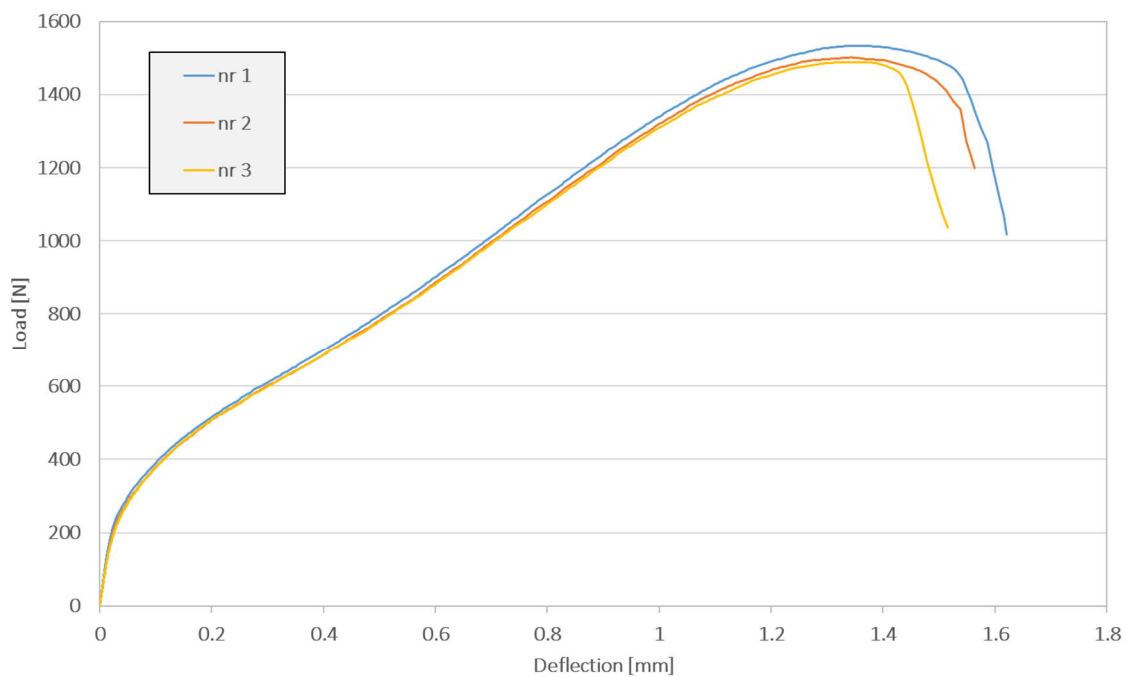


Figure 29. Load – deflection curves of tensile Sample 2.

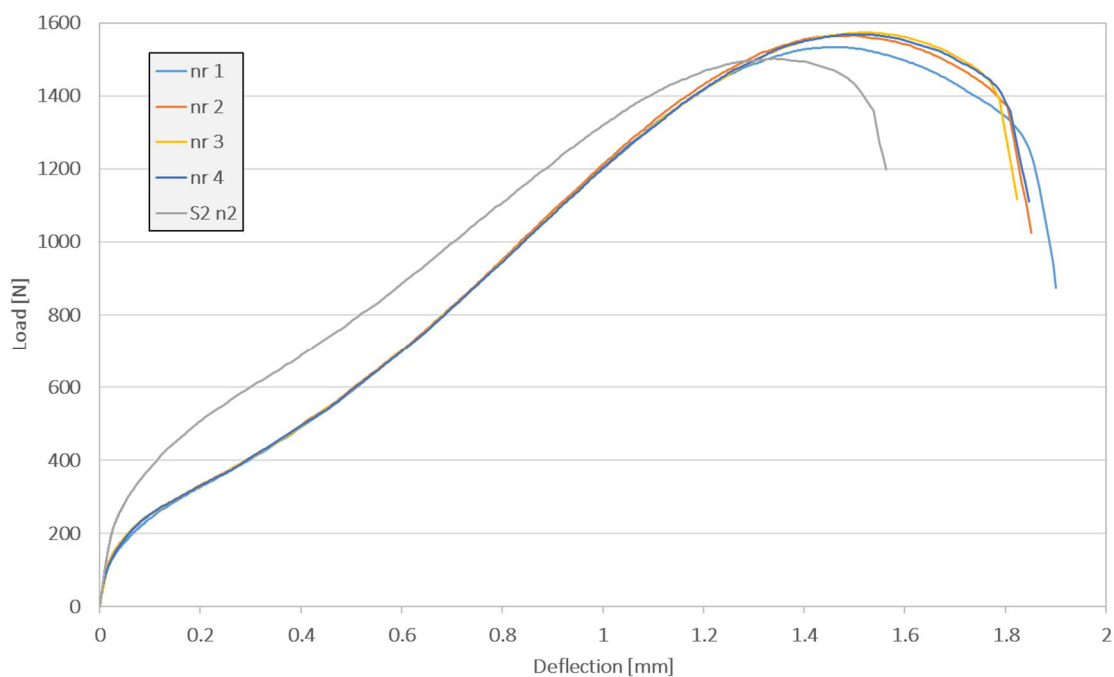


Figure 30. Comparison of Sample 2 specimen nr 2 load curve with the load curves from Sample 1.

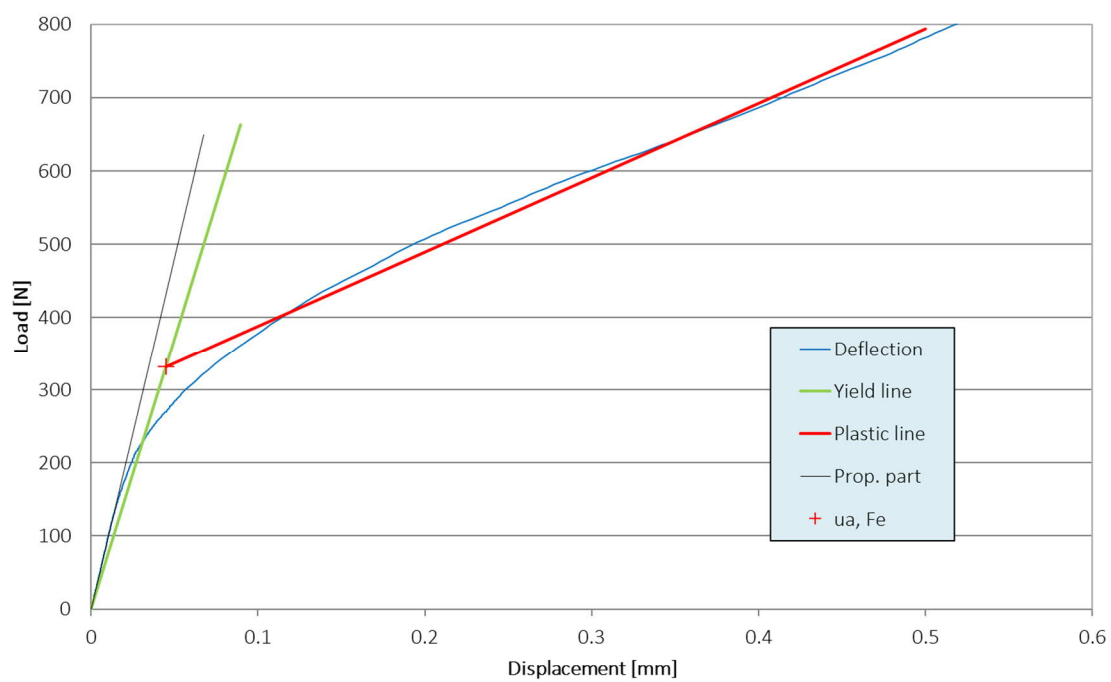


Figure 31. Example of definition of yield load determination from the tensile curve.

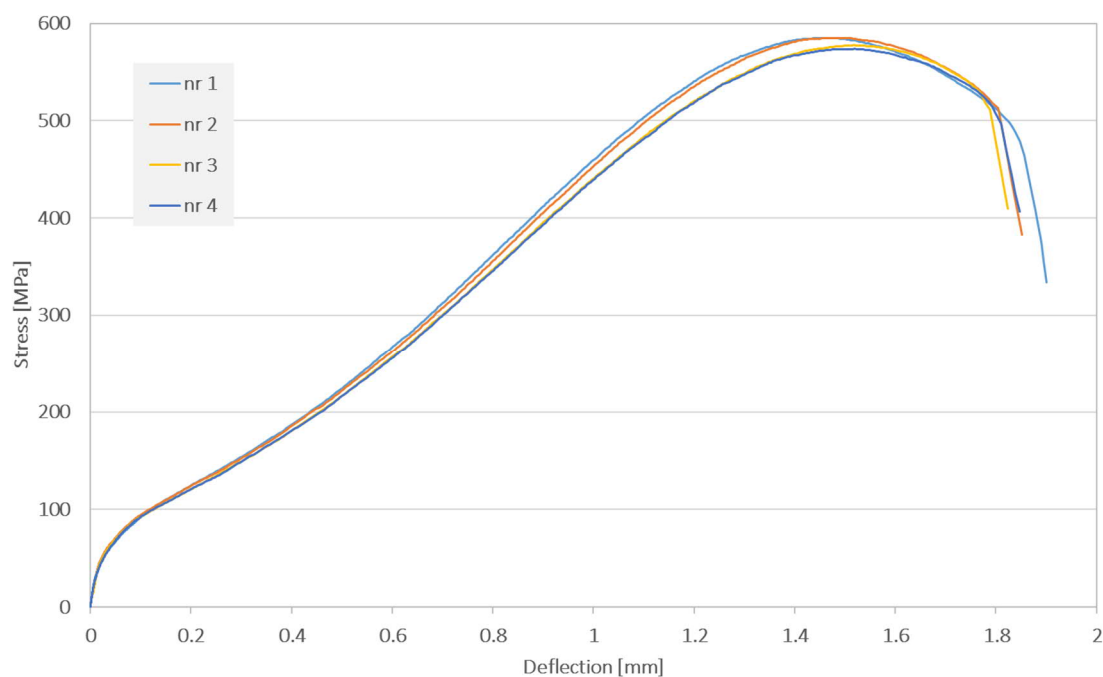


Figure 32. Stress – deflection curves of Sample 1 calculated by Equation C.1.

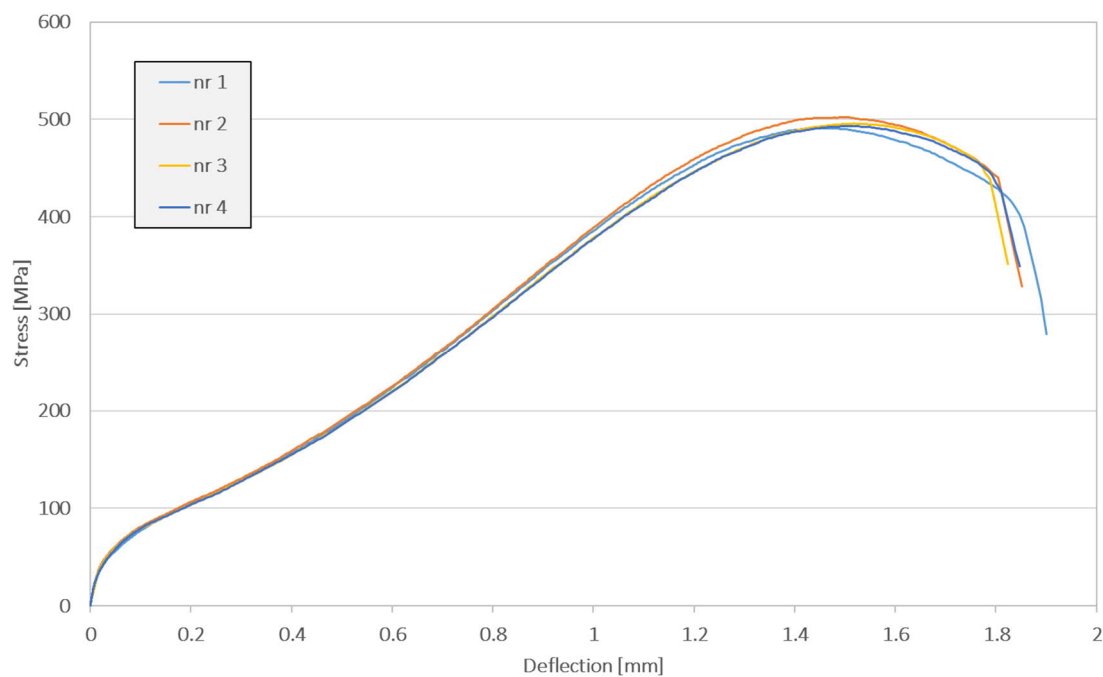


Figure 33. Stress – deflection curves of Sample 1 calculated by Equation C.2.

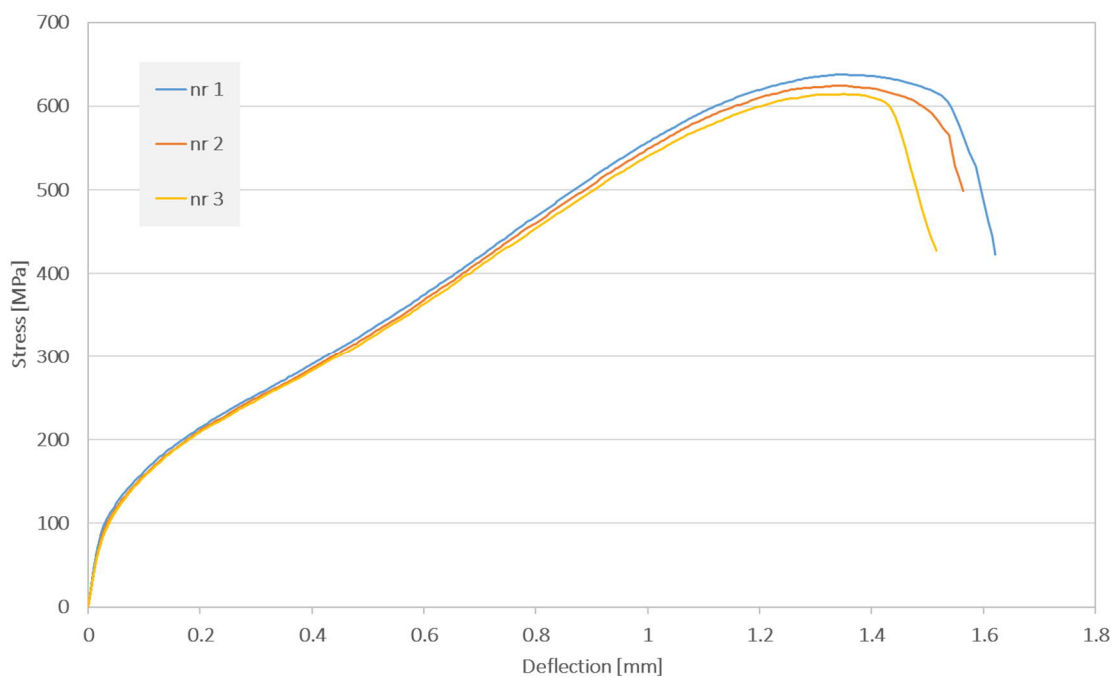


Figure 34. Stress – deflection curves of Sample 2 calculated by Equation C.1

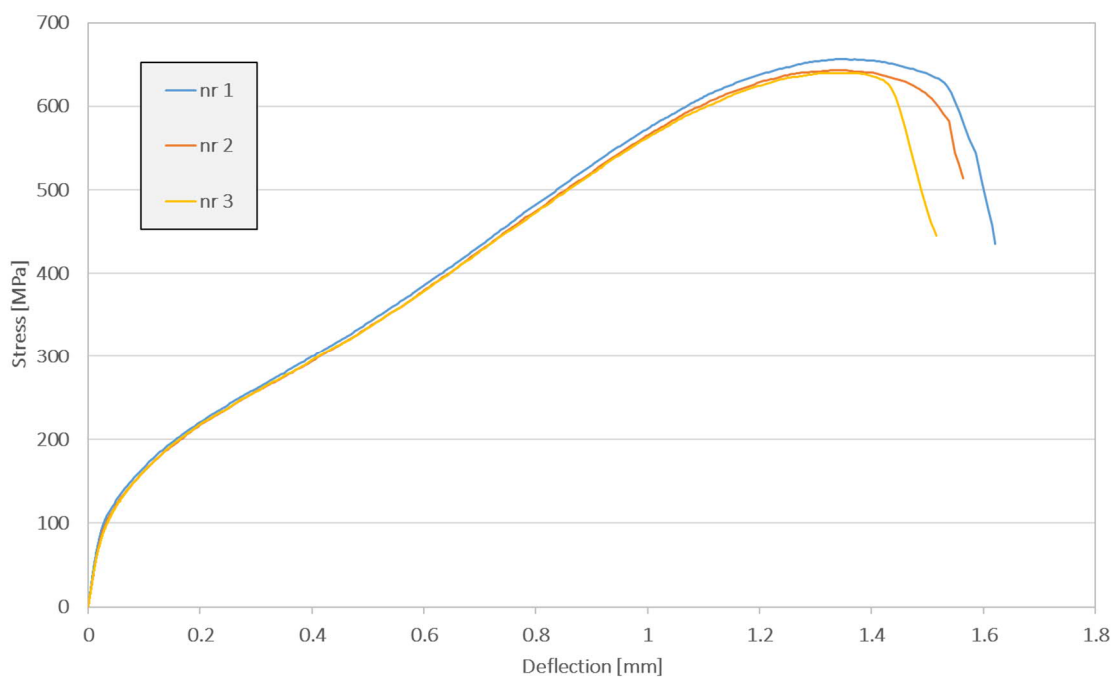


Figure 35. Stress – deflection curves of Sample 2 calculated by Equation C.2.

Annex B Sandwich panels mechanical testing results

Table 22 Tensile test results PIR panels south side (PIR-S3)

Specimen	Max. Load [N]	Elongation [%]	Tensile Strength [MPa]	Tensile Modulus [MPa]
1	1824.65	2.15	0.078	3.19
2	1127.19	1.63	-	2.93
3	1019.94	1.96	-	2.64
5	1736.22	2.78	0.076	3.05
6	1542.68	2.64	0.064	-
7	1329.11	2.42	0.059	2.51
Test results	Average		0.069	2.864
	5%-fractile		0.048	2.230
Declared	Average		-	3.57 ²⁾
	5%-fractile		0.08 (0.04 ¹⁾)	2.93 ²⁾

¹⁾ According to EN 14509 [9] a decrease of 50 % of tensile load is acceptable

²⁾ No values for tensile modulus declared; $E_c = 3,8 = 0,5 \cdot (E_{ct} + E_{cc})$

Table 23 Tensile test results PIR panels west side (PIR-W3)

Specimen	Max. Load [N]	Elongation [%]	Tensile Strength [MPa]	Tensile Modulus [MPa]
1	1630.22	1.93	0.071	3.97
2	1729.28	1.99	0.076	3.79
5	2275.94	2.45	0.101	3.96
6	1759.30	3.01	0.077	3.21
7	1763.72	2.96	0.078	3.12
Test results	Average		0.080	3.610
	5%-fractile		0.057	2.692
Declared	Average		-	3.8
	5%-fractile		0.08 (0.04 ¹⁾)	3.3

¹⁾ According to EN 14509 [9] a decrease of 50 % of tensile load is acceptable

Table 24 Compressive test results PIR panels south side (PIR-S3)

Specimen	Max. Load [kN]	Compressive Modulus [MPa]	Compressive Strength [MPa]
1	2.76	3.01	0.125
2	2.58	3.00	0.117
3	2.95	3.07	0.131
4	2.86	3.20	0.129
5	3.17	3.15	0.140
Test results	Average	3.086	0.128
	5%-fractile	2.881	0.109
Declared	Average	3.57	-
	5%-fractile	2.93	0.10

Table 25 Compressive test results PIR panels west side (PIR-W3)

Specimen	Max. Load [kN]	Compressive Modulus [MPa]	Compressive Strength [MPa]
1	3.66	3.66	0.165
2	3.51	3.75	0.159
3	3.59	3.58	0.163
4	3.61	3.59	0.164
5	3.57	3.77	0.162
Test results	Average	3.67	0.163
	5%-fractile	3.46	0.157
Declared	Average	-	-
	5%-fractile	2.50	0.10

Table 26 Results of MW panels bending test 2007

Sample	Panel length [mm]	Max. load [kN]	Max. deflection [mm]	Location of failure [mm]	Wrinkling strength at midspan [MPa]	Shear strength at support [MPa]	Shear modulus [MPa]
980260-3	5080	22.83	52	2460	150	0.063	2.9
980260-5	6500	17.36	57	3140	147	0.048	3.0
980260-6K	6500	17.68	51	2700	150	0.049	4.2
980260-7K	6500	19.55	56	2850	166	0.054	4.4
980260-8	6500	17.92	56	2600	152	0.050	3.3
Average					153		3.6
St.dev					7.5		

Table 27 Results of dismantled MW panels bending tests 2018

Sample	Panel length [mm]	Max. load [kN]	Max. deflection [mm]	Location of failure [mm]	Wrinkling strength at midspan [MPa]	Shear strength at support [MPa]	Shear modulus [MPa]
MW-N2	5070	24.3	57.6	1000	144.4		2.8
MW-N3	5070	23.5	44.1	1900	139.7		3.4
MW-N6	5070	21.4	39.3	2500	127.2	0.058	3.6
Average					137.1		3.2
St.dev					8.9		

Table 28 Results of dismantled PIR panels bending tests 2018

Specimen	Max. Load [kN]t	Displacement [mm]	Wrinkling Strength [MPa]	Wrinkling Strength Declared [MPa]
PIR-S3 01	10.1	121.3	135.8	142
PIR-S3 02	13.7	114.1	184.6	
PIR-W3 01	11.92	131.76	152.91	131
PIR-W3 02	13.01	98.87	166.90	

Table 29 Results of MW panels shear test 2007

Sample	Panel length [mm]	Max. load [kN]	Max. deflection [mm]	Location of failure [mm]	Wrinkling strength at midspan [MPa]	Shear strength at support [MPa]	Shear modulus [MPa]
980260-1	5080	23.05	63	100	145	0.063	2.5
980260-2	5080	23.69	60	100	150	0.064	2.7
980260-4K	5080	24.52	57	150	155	0.067	2.8
980260-9	3000	25.28	30	100	89	0.067	2.4
980260-10	3000	25.73	28	200	90	0.068	2.5
980260-11K	3000	27.12	31	100	95	0.072	2.5
980260-12K	3800	26.34	40	130	121	0.071	2.8
980260-13	3800	24.41	39	100	112	0.066	2.6
980260-14	3800	24.22	35	150	111	0.065	2.5
Avg.						0.067	2.6
St.dev.						0.003	

Table 30 Results of dismantled MW panels shear tests 2018

Sample	Panel length [mm]	Max. load [kN]	Max. deflection [mm]	Location of failure [mm]	Wrinkling strength at midspan [MPa]	Shear strength at support [MPa]	Shear modulus [MPa]
MW-S1	3800	16.8	33.3	150	75.9	0.046	2.0
MW-S2	3800	25.1	34.8	200	107.2	0.066	2.7
MW-S7	3800	26.6	36.3	250	113.6	0.070	2.9
Average						0.061	2.5
St.dev						0.013	

Table 31 Shear test results, PIR panels south side

Specimen	Max. Load [N]	Displacement [mm]	Shear Strength [MPa]	Shear Modulus [MPa]
S01	2473.03	17.06	0.130	3.161
S02	2491.93	20.28	0.131	2.934
S03	2622.38	25.46	0.137	2.864
S04	2486.28	21.09	0.131	2.782
S06	2552.99	22.36	0.135	2.728
S07	2611.80	20.73	0.138	2.961
Test results	Average		0.134	2.905
	5%-fractile		0.126	2.568
Declared	Average		-	3.500
	5%-fractile		0.100	3.070

Table 32 Shear test results PIR panels west side

Specimen	Max. Load [N]	Displacement [mm]	Shear Strength [MPa]	Shear Modulus [MPa]
W01	2895.09	22.29	0.152	3.359
W02	2768.75	20.71	0.144	3.082
W03	2835.43	20.97	0.148	2.928
W04	2904.42	22.35	0.151	3.301
W06	2810.68	21.27	0.147	3.066
Test results	Average		0.148	3.141
	5%-fractile		0.141	2.731
Declared	Average		-	3.800
	5%-fractile		0.130	3.40

Force versus displacement graphs for shear tests (S) and tensile tests (T)

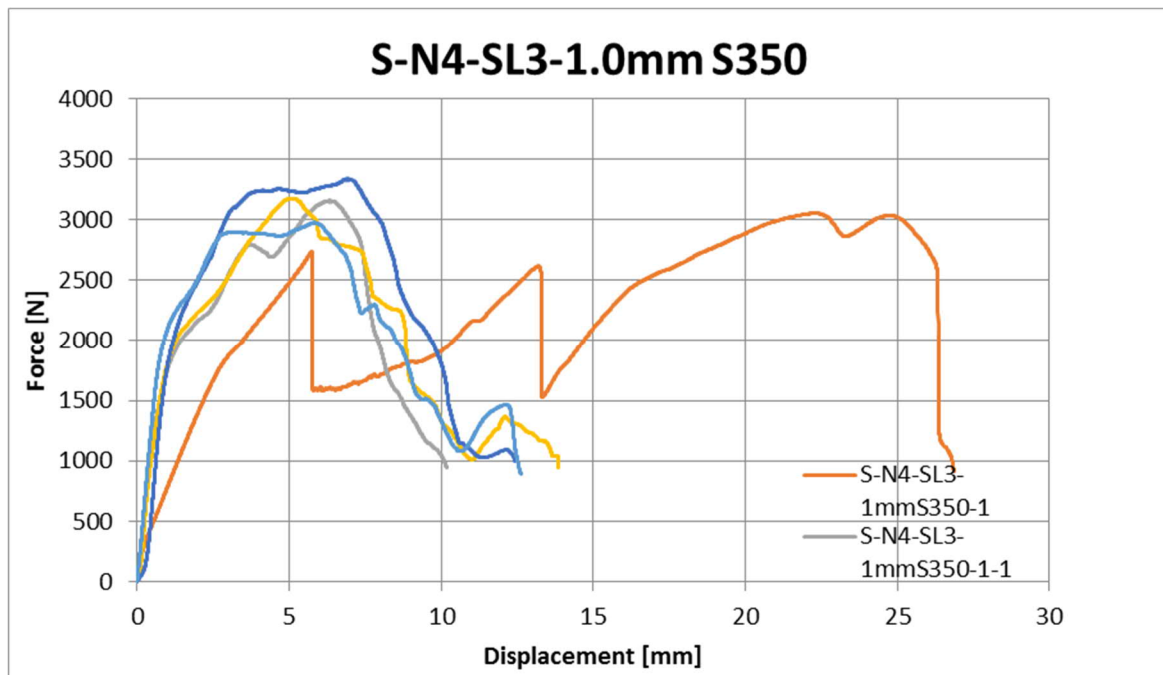


Figure 36 Load-displacement results from S-N4-SL3-1.0mm S350

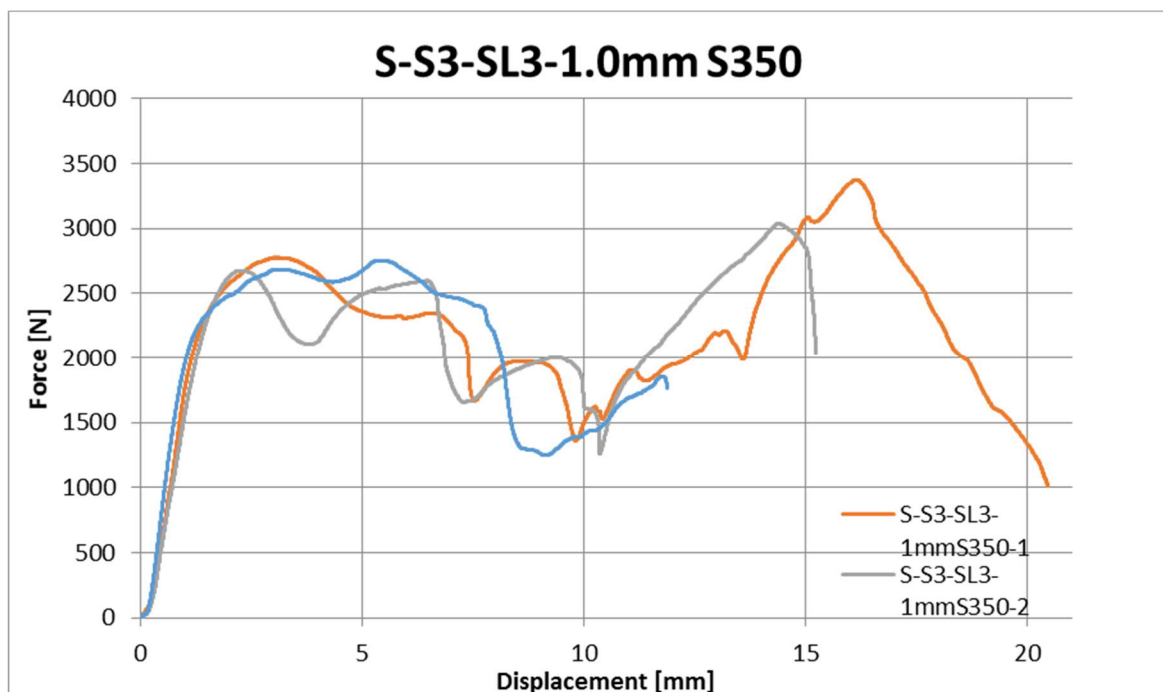


Figure 37 Load-displacement results from S-S3-SL3-1.0mm S350

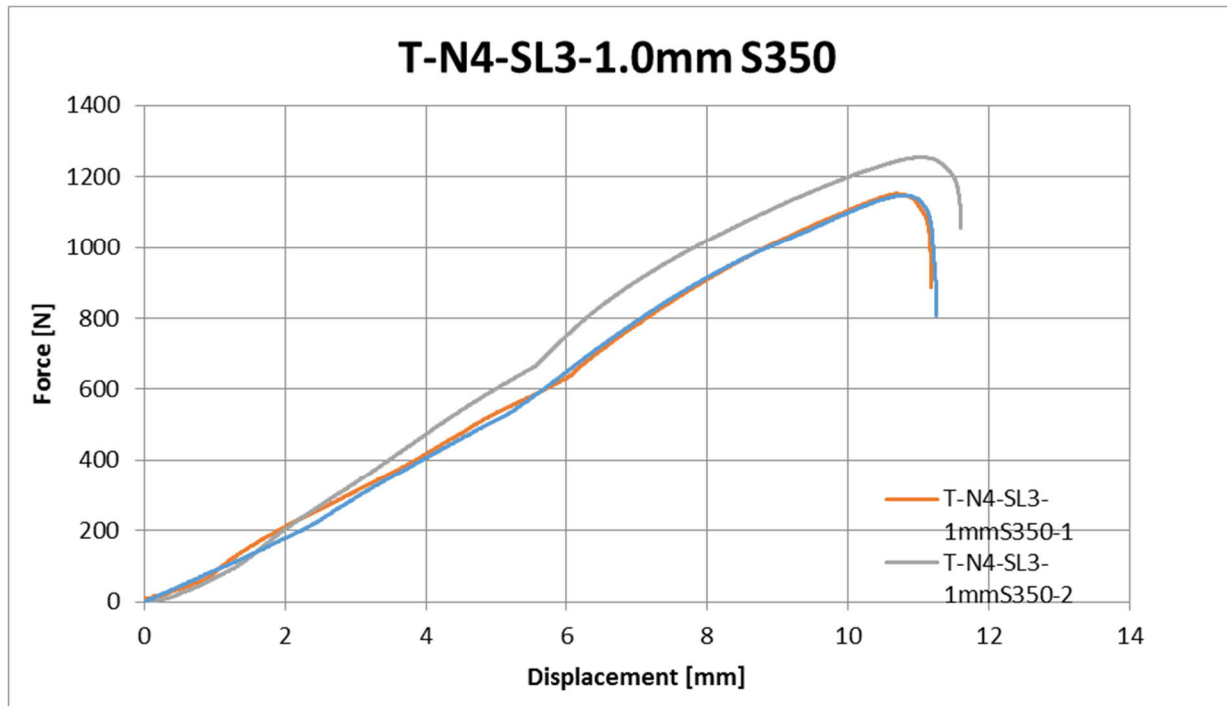


Figure 38 Load-displacement results from T-N4-SL3-1.0mm S350

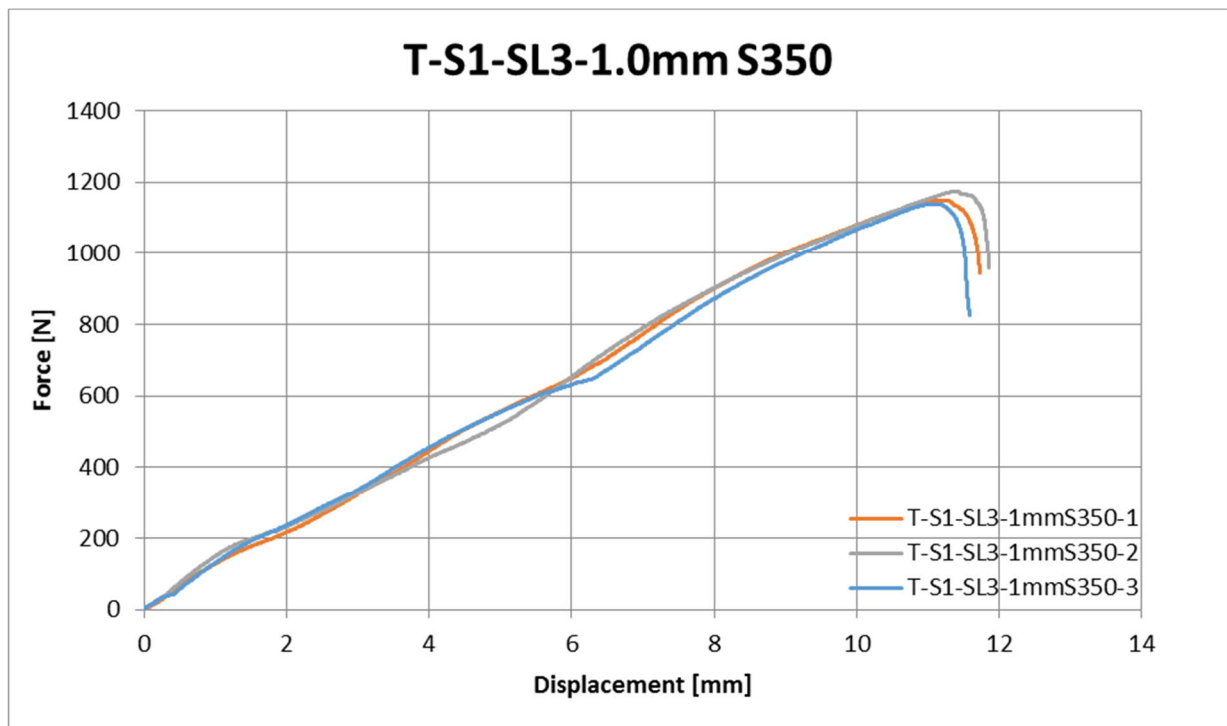


Figure 39 Load-displacement results from T-S1-SL3-1.0mm S350

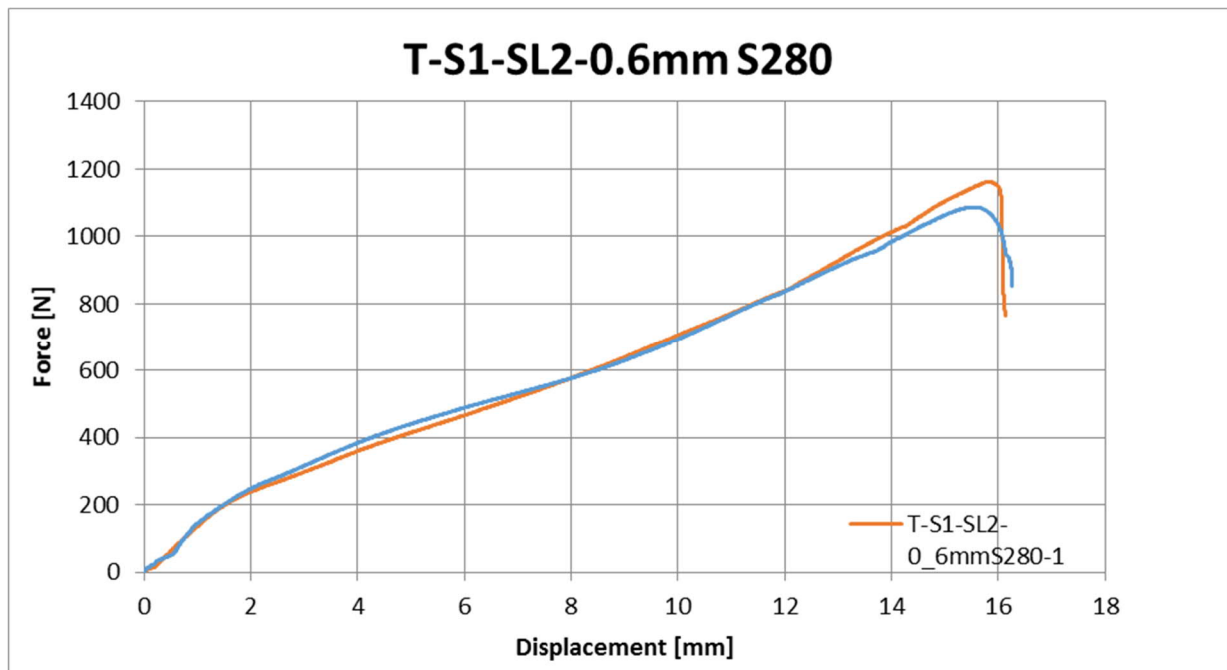


Figure 40 Load-displacement results from T-S1-SL2-0.6mm S280

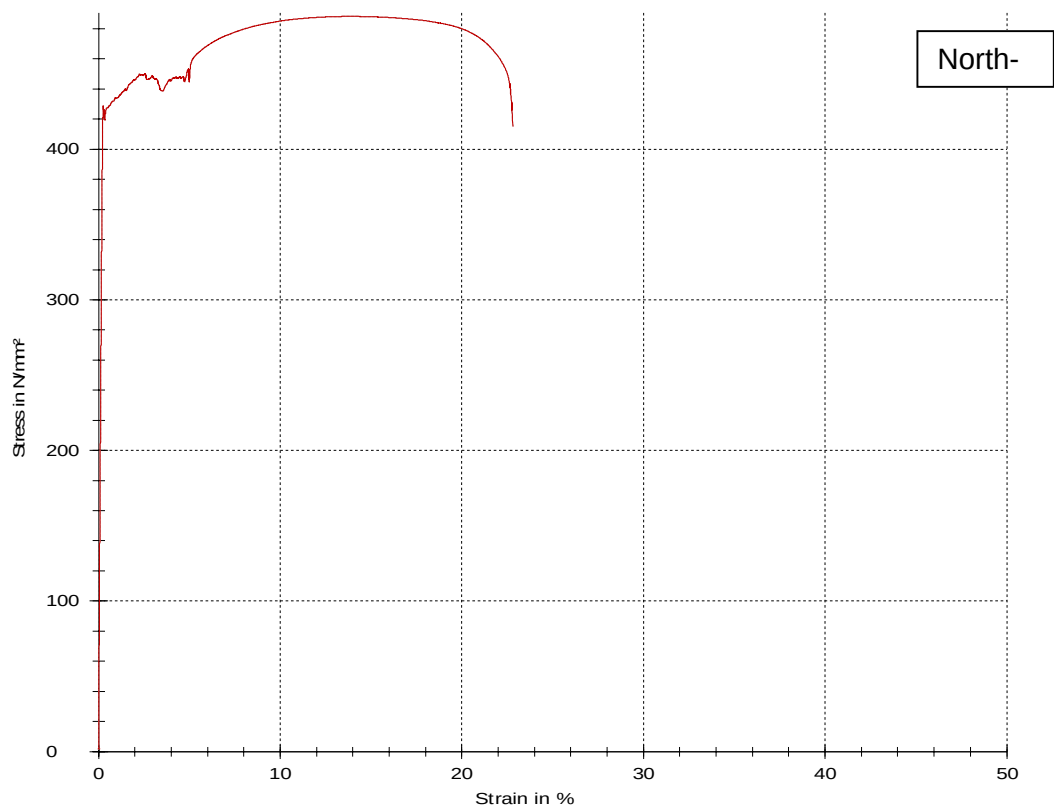


Figure 41 Stress-strain relationship

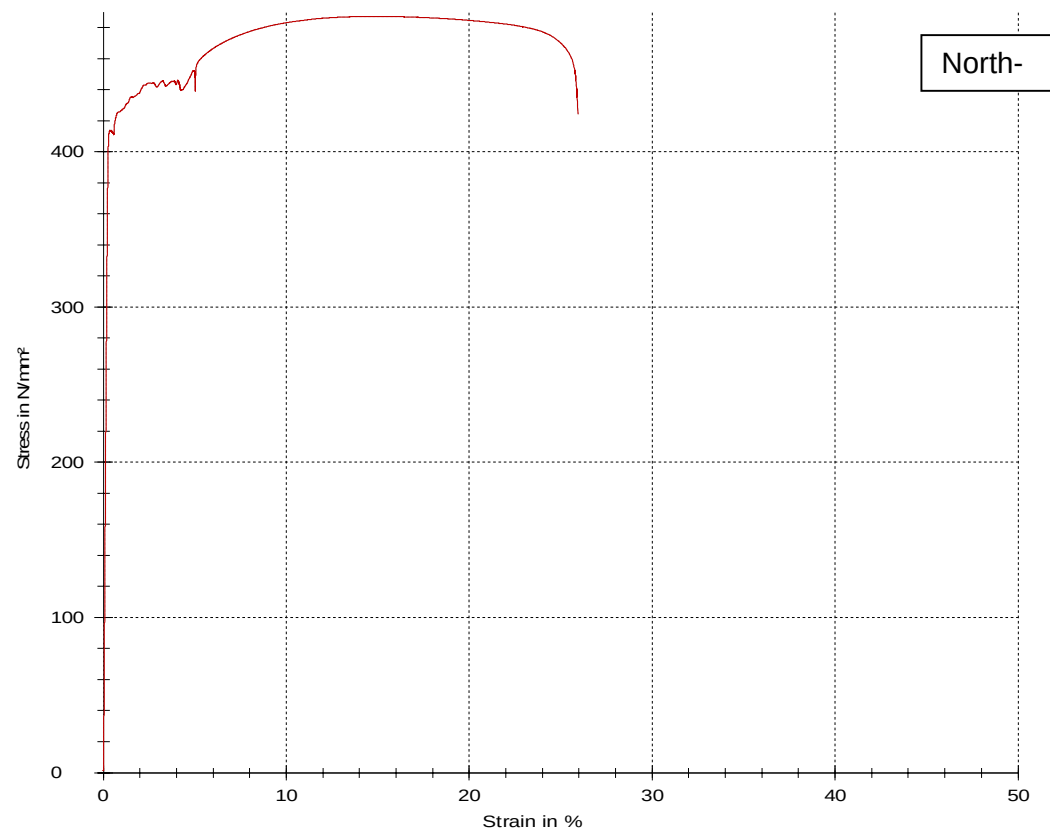


Figure 42 Stress-strain relationship

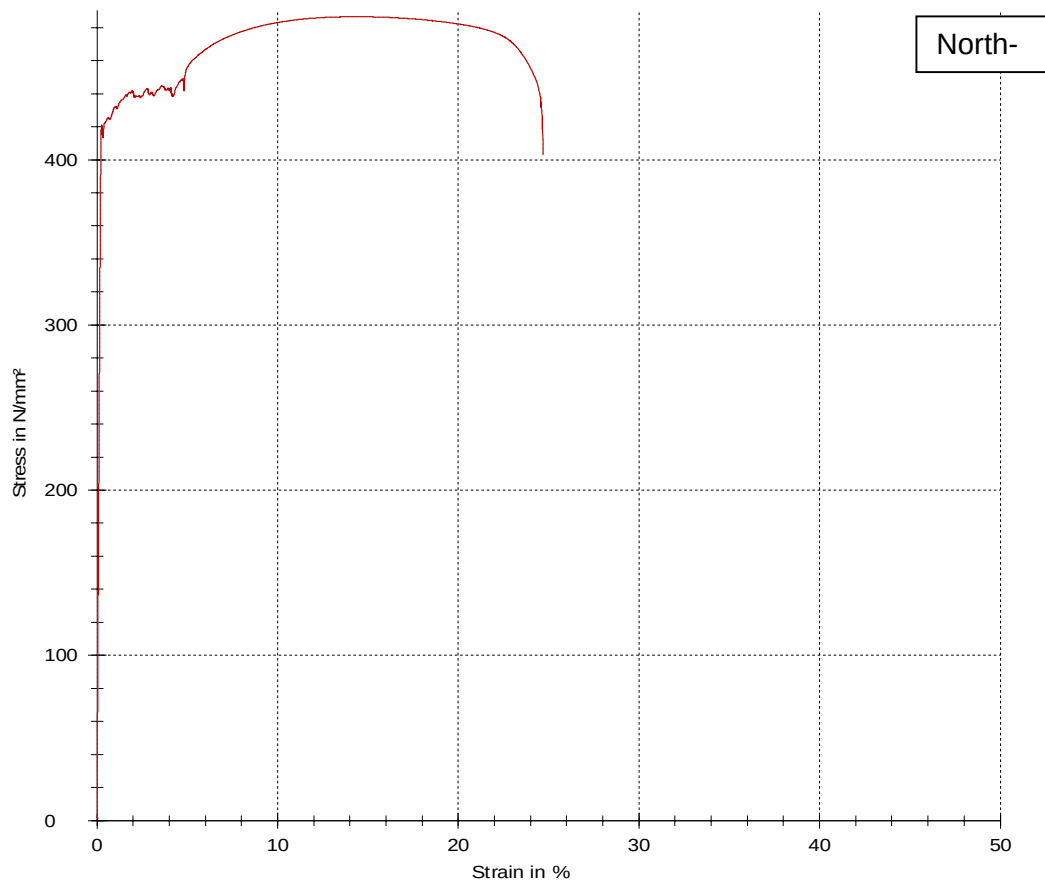


Figure 43 Stress-strain relationship

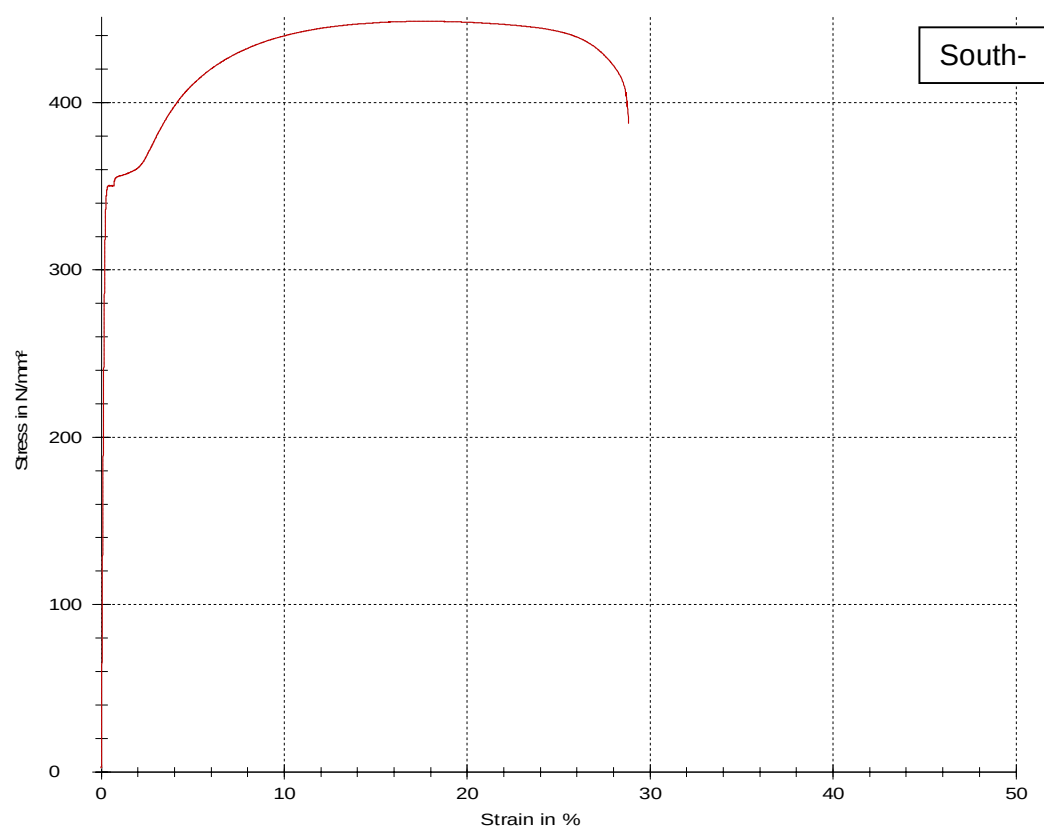


Figure 44 Stress-strain relationship

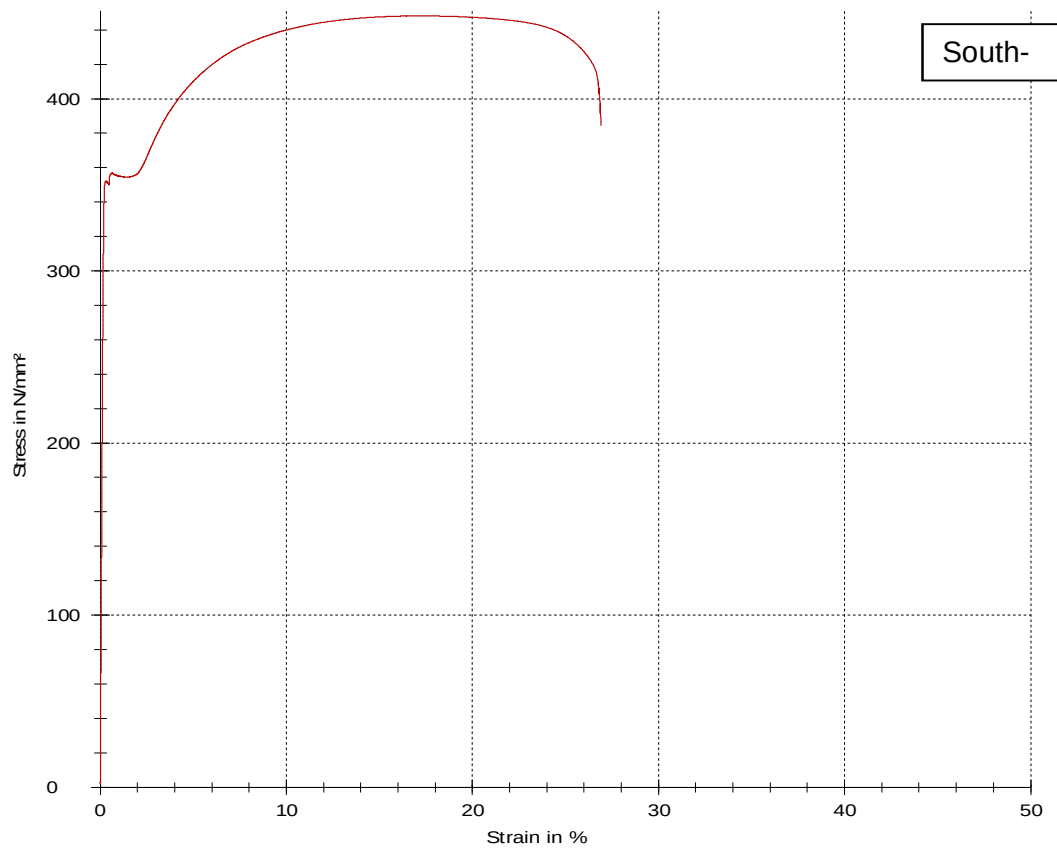


Figure 45 Stress-strain relationship

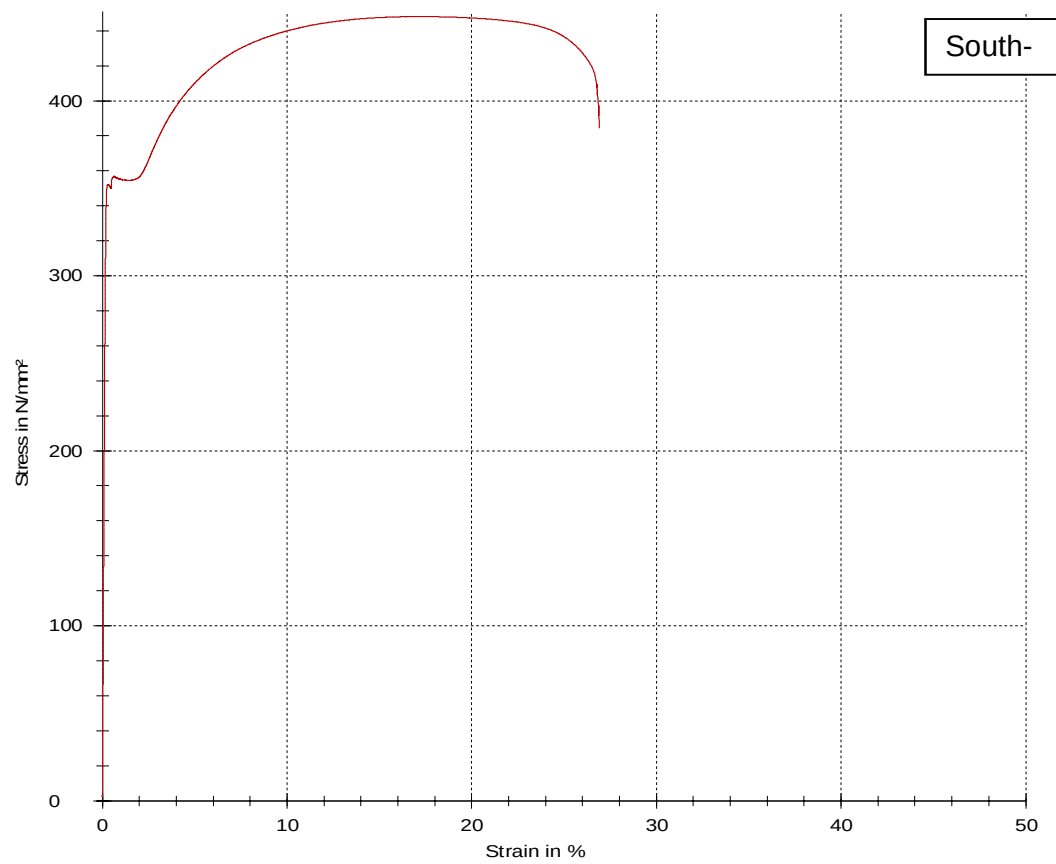


Figure 46 Stress-strain relationship

Annex C Analysis of structural testing of sandwich panels

The number of available tested specimen was quite low and thus not all test series were able to perform. Proposed criteria for strength values of reusing sandwich panels are summarized in Table 33 followed by an evaluation is for tensile and shear strength.

Table 33 Proposed criteria for reused panels

Mechanical strength	
Testing cross panel tensile strength 3 samples a minimum (EN 14509, A.1): Calculate characteristic result for tensile strength Testing one sample for shear strength (EN 14509, A.3 or A.4)	
Tensile strength $\geq 0,9^*$ Declared value of Tensile strength and Shear strength $\geq$ Declared value of Shear strength	If YES: No further testing. All declared values for mechanical strength can be used If NO: New declared values to be determined with a test program according to EN 14509 for -Tensile strength -Compression strength -Shear strength The wrinkling strength is reduced with same amount that shear strength is reduced.
Durability	
Tensile strength $\geq 0,9^*$ Declared value of Tensile strength	If YES: No further testing; panels are fit for use If NO: <u>MW panels:</u> The 7 days testing (see EN 14509 clause B.3.4) is to be done. The reduction in tensile strength after ageing shall not exceed 15 % of the mean value of the tensile strength in ambient temperature <u>For all other panel type</u> the procedure in EN 14509 Annex B.2 is followed so that the panels are tested 14 days in the temperature as described in B.2.4. The reduction in tensile strength after ageing shall not exceed 17% of the mean value of the tensile strength in ambient temperature
Tolerances; damages evaluated by visual inspection	If no serious damages or faults found; Panel can be reused If serious damages are found causing weakness in strength, in insulation behavior or tightness of joints then those panels are rejected
Moisture content	
If no notable wetness of core material found the panels can be reused	

Tensile strength f_{Ct}

Characteristic value of tested values is determined according to EN 14509 A [9] as 16.3 for 30 tensile tests from Table 15:

$$n = 30$$

$$\text{Mean value of test results: } 0.092 \text{ N/mm}^2$$

$$\bar{y} = \frac{1}{n} \sum_{i=1}^n L_n(x_i) = -2.431$$

$$\sigma_y = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (L_n(x_i) - \bar{y})^2} = 0.341$$

$$R_k = e^{\bar{y} - k \cdot \sigma_y}$$

$$k = 1.87$$

$$\underline{f_{Ct,Rk}} = 0.046 \text{ N/mm}^2$$

Minimum value for tensile strength f_{Ct} is according to type approval of that time 0.150 N/mm^2 , thus the tested value would not meet the criteria $f_{Ct,Rk} > 0.9 f_{Ct,Declared}$ and new declared values should be determined for the reused panel patch. On the other hand it should be noted that the original tensile test results were very low and 2018 tested values were almost 60% higher than those. Thus, it is difficult to draw conclusion on ageing behaviour based on these test series.

Shear strength f_{Cv}

Characteristic value of tested values is determined according to EN 14509 A [9] as 16.3 for 3 shear tests from

Table 29:

$$\begin{aligned}
 n &= 3 \\
 \text{Mean value of test results: } &0.061 \text{ N/mm}^2 \\
 \bar{y} &= \frac{1}{n} \sum_{i=1}^n L_n(x_i) = -2.819 \\
 \sigma_y &= \sqrt{\frac{1}{n-1} \sum_{i=1}^n (L_n(x_i) - \bar{y})^2} = 0.227 \\
 R_k &= e^{\bar{y} - k \cdot \sigma_y} \\
 k &= 3.15 \\
 \underline{f_{Cv,Rk}} &= 0.029 \text{ N/mm}^2
 \end{aligned}$$

Minimum value for shear strength f_{Cv} is according to type approval of that time 0.054 N/mm^2 , thus the tested value would not meet the criteria $f_{Cv,Rk} > f_{Cv,Declared}$. It should be noted, that number of test specimen was very low thus leading to very high k -value.

Annex D Photographs



Figure 47 S-N4-SL3-1mmS350-1 test arrangement



Figure 48 S-N4-SL3-1mmS350-1 panel condition during the test.

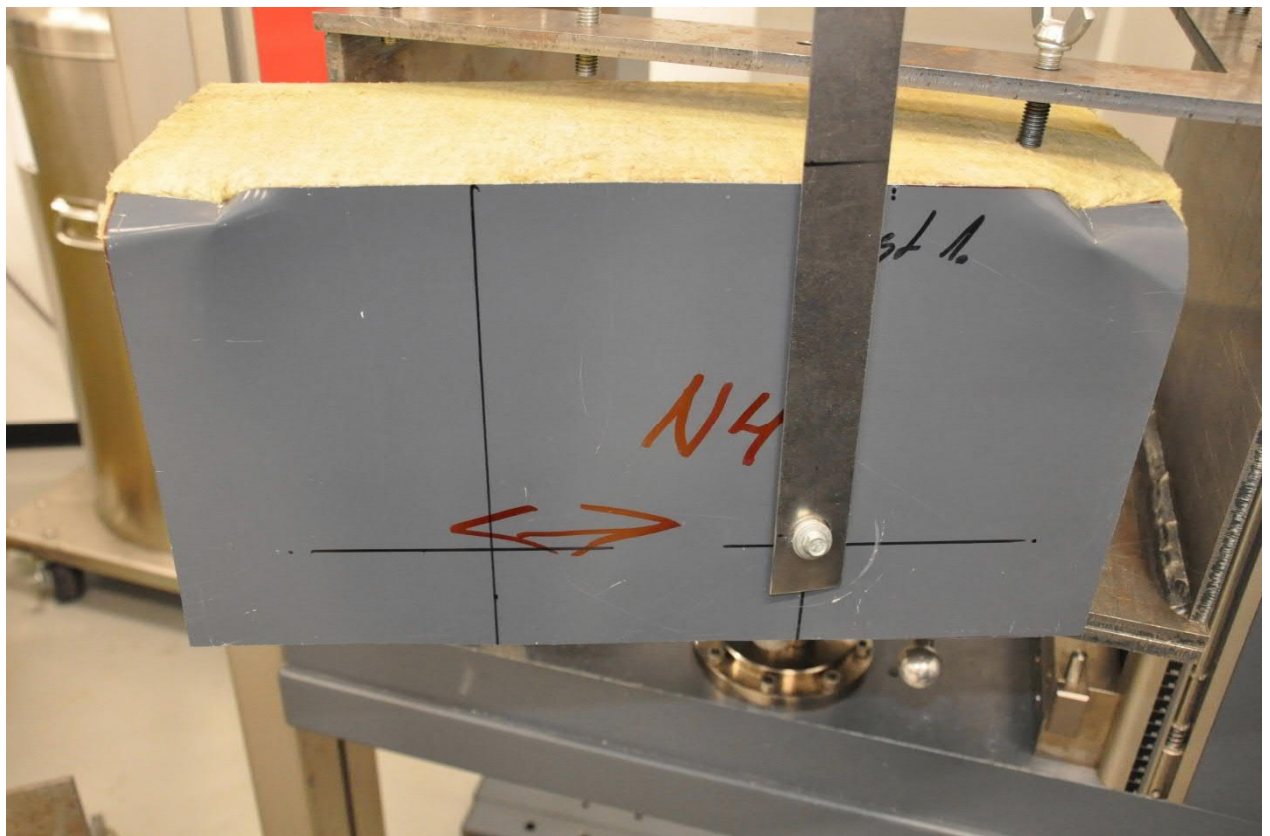


Figure 49 S-N4-SL3-1mmS350-1 panel condition after the test.

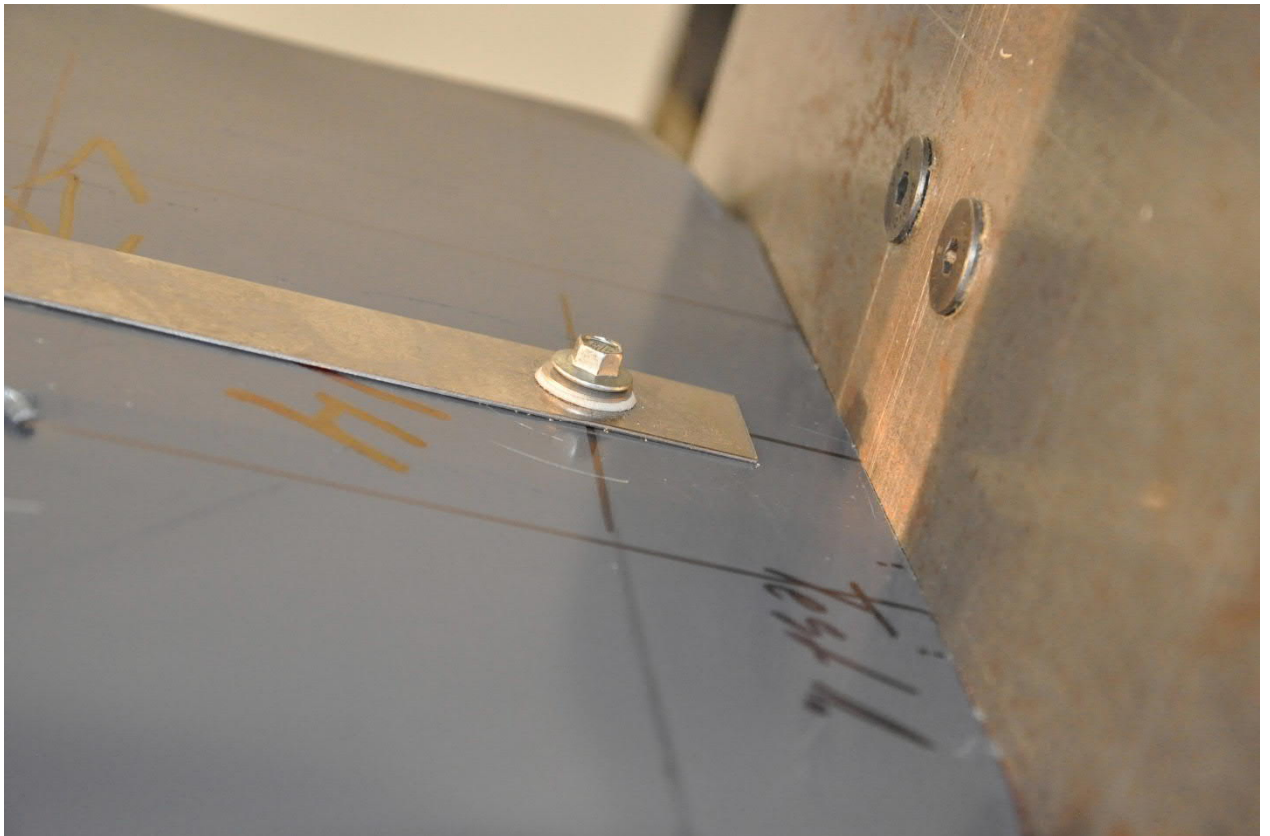


Figure 50 S-N4-SL3-1mmS350-1-1 before the test.



Figure 51 SL3 screw pull-out failure mode in shear tests.

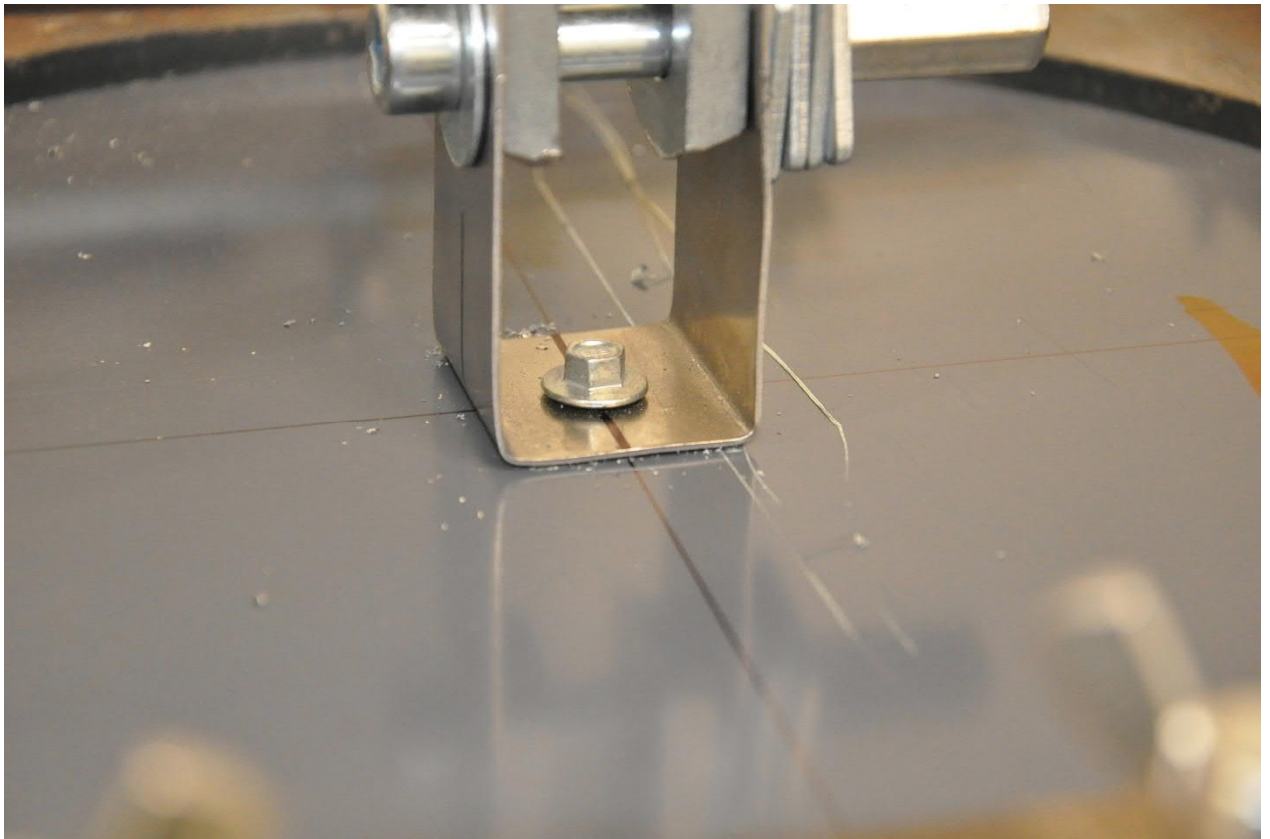


Figure 52 T-N4-SL3-1mmS350-1 before the test.

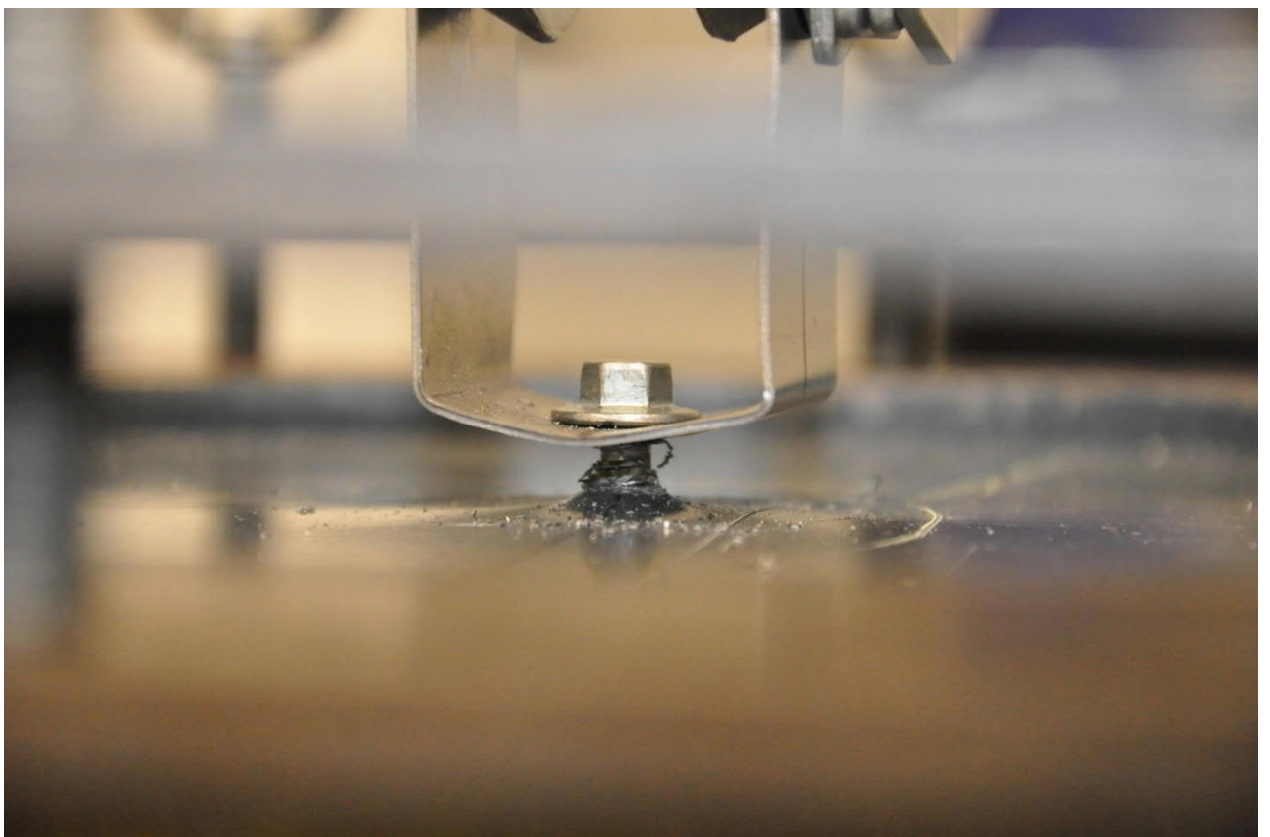


Figure 53 SL3 failure mode in tensile tests.

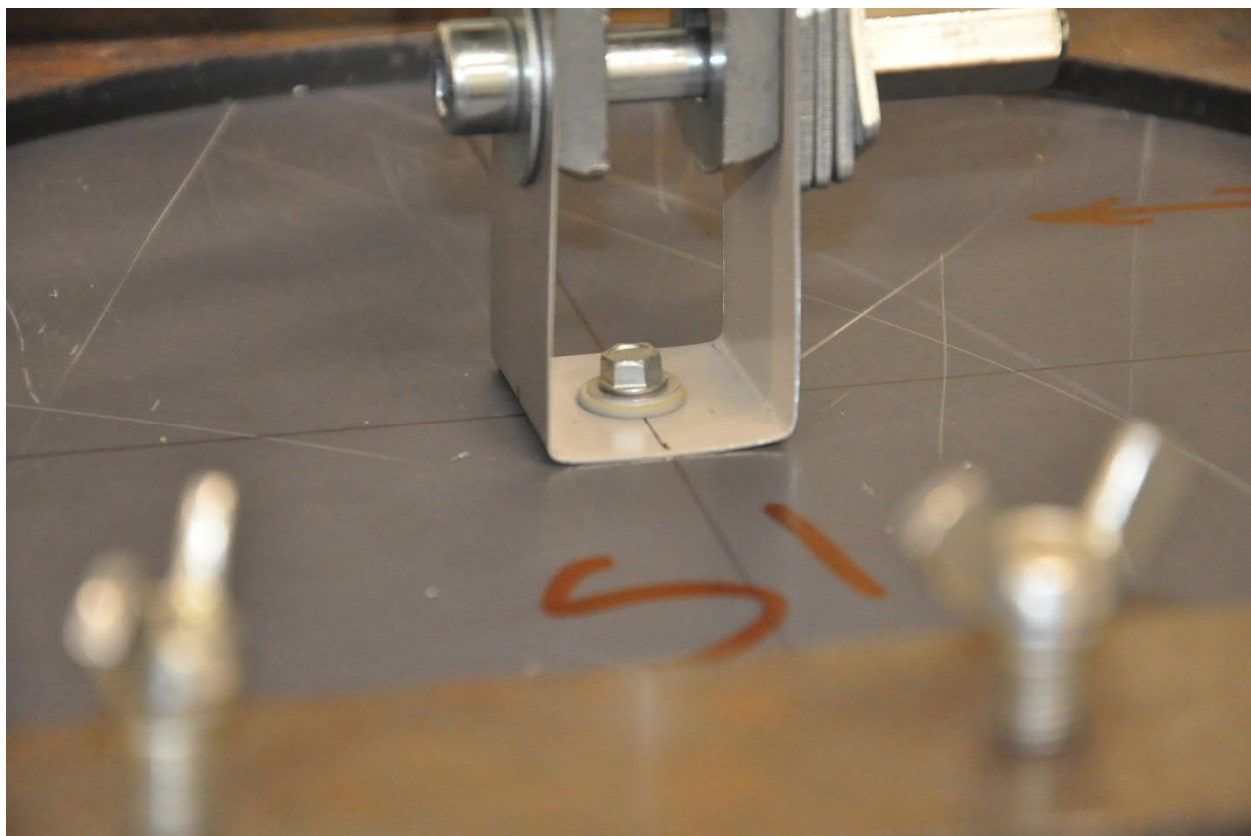


Figure 54 T-S1-SL2-0,6mmS280-1 before the test.

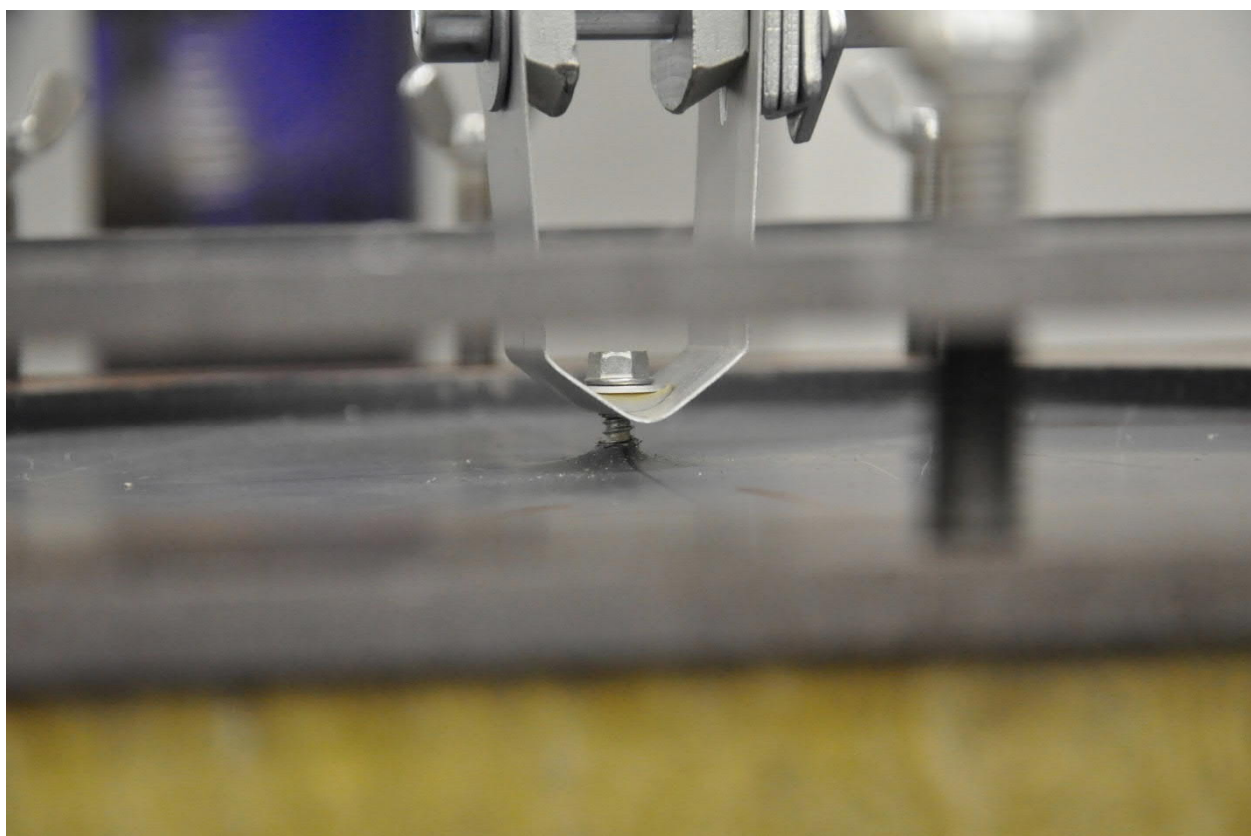


Figure 55 SL2 failure mode in tensile tests.



Figure 56 Screws for tests.