Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report
on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of

Ministerie van Economische Zaken - The Netherlands

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1 Objectives

The Ministerie van Economische Zaken (EZ) of the Netherlands has commissioned a systematic study considering all future safety aspects concerning the potential consequences of former hardcoal exploitation in South Limburg. The project is shortly named „Na-ijlende gevolgen steenkolenwinning Zuid-Limburg".

The consequences and potential hazards of former hardcoal exploitation were subdivided in 7 different effects or topics, resulting in 7 work packages. In the structure of the project „Na-ijlende gevolgen steenkolenwinning Zuid-Limburg" these 7 potential effects or topics have been investigated and assessed by different working groups with special expertise on the executed theme.

The potential hazards/risks caused by mine shafts or mining activities near to the surface or rather near to the top of the Carboniferous bedrock are comparable and therefore the work packages 5.2.2 (risks from mine shafts) and 5.2.3 (risks from near-surface mining) were executed by the same team.

Prior to this study there was an intense collection of basic data (data acquisition) done by TNO with IHS as subcontractor. The results of the investigations and the assessments that are described in this report start with the transfer and the compilation of this TNO-data, as far as the working groups were concerned.

The report in hand presents a summary of the investigations and assessments that have been performed by IHS/DMT on the topics of "risks from mine shafts" and "risks from near-surface mining".

2 Collection and compilation of mining documents

2.1 Preceded data-acquisition by TNO and IHS

The collection of the basic data for this study (data acquisition) has been done by TNO with IHS as subcontractor. The results of the investigations and the assessments that are described in this report start with the transfer and the compilation of this TNO-data, as far as the WG 5.2.2/5.2.3 were concerned. The approach of the data acquisition and the most relevant results are described as follows.

After abandonment of the mining activities in the South Limburg mining district the archive material from the mine companies and later on also from Staatstoezicht op de Mijnen (SodM) was transferred to public archives like:

- Nederlands Mijnmuseum, Heerlen
- Nationaal Archief incl. the "Winschoten-List", The Hague
- Regionaal Historisch Centrum Limburg, Maastricht (RHCL)
- Sociaal Historisch Centrum voor Limburg, Maastricht (SHCL)
- Rijckheyt - centrum voor regionale geschiedenis, Heerlen

In a first step the archives mentioned above were browsed carefully online as well as in hard copies. If an archive seems to contain relevant information for the workings groups the inventory lists were searched in detail and the findspots (inventory numbers) of relevant information were marked. To obtain the documents that were identified from the inventory lists the archives were visited several times and especially the RHCL-archive was visited regularly.

The main objective of the research project for the WG 5.2.2/5.2.3 was the evaluation of all available mining maps as well as other maps and plans to identify the locations of old mine shafts and to work out potential hazards from shafts and near-surface mining activities.

Some years ago TNO first started to digitise their own mining map archive. Within the context of the research project this data pool was substantially added by scanned maps from different archives. Furthermore, the mining maps were cataloguised and georeferenciated. The general workflow of this procedure is shown in the following diagram (Fig. 1).

Basically, two major types of documents were relevant to the working groups:

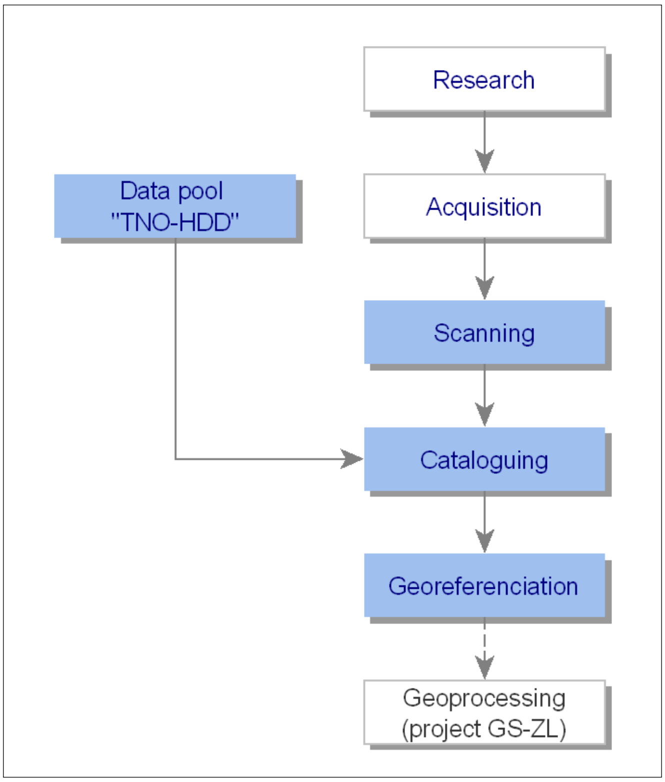

Fig. 1: Workflow data-acquisition of mining maps

- Ground plans
  - Working plans and plans of seams (“plattegrond” and “laagplannen”): horizontal projections of the excavations, their labels, extraction periods
  - Mine level sheets (“hoofdgrondplannen”): horizontal projection showing the galleries, etc. in each main floor
  - Surface plans (“bovengrondsch plannen”): plans showing the surface situation including subsidences (“verzakkingen”) and other mining induced damage at the surface (“drempels and scheuren”)
  - Subsurface plans: plans of the uncovered bedrock with altitude indication
- Vertical sections
  - Geological cross-sections
  - Vertical sections following galleries („steengangprofielen“)
  - Drill logs

All maps were georeferenciated to fit the official Dutch spatial reference system (RD-New). The workflow for georeferenciation is shown in Fig. 2. Primarily, the software WGEO® was used for georeferenciation since most mining maps contained coordinate grids. In case a map does not comprise coordinate specifications the georeferenciation was carried out in ArcGIS®. To assure the parallelism of the maps‘ coordinate systems the affine transformation method was used.

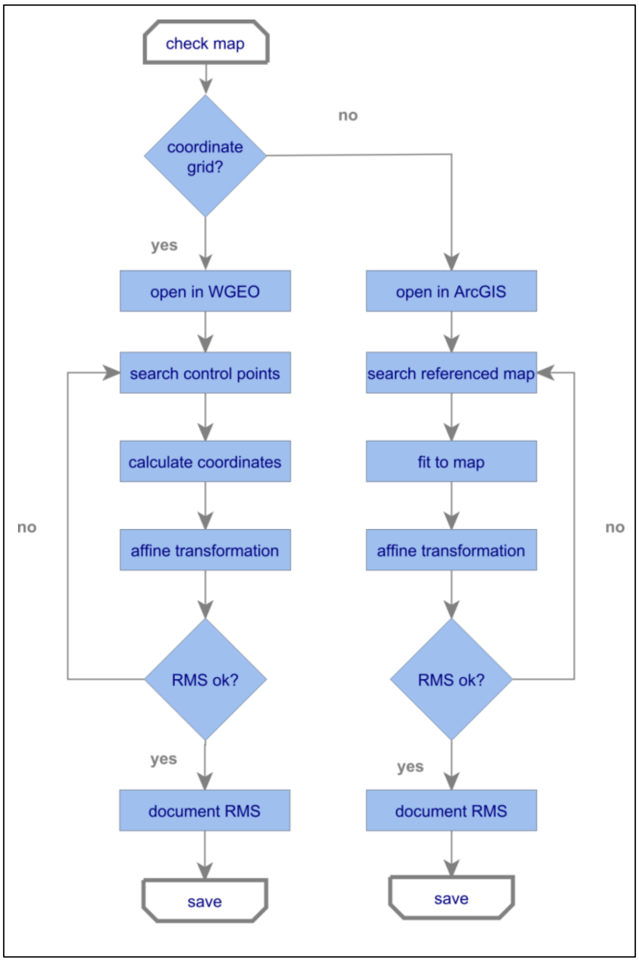

Fig. 2: Workflow of georeferenciation-procedure

2.2 Compilation of mining documents by IHS

As outcome of the data-acquisition about 7.676 mining maps and mining map related documents were available for evaluation. The data pool includes an EXCEL-spreadsheet that comprises a number of relevant metadata of the mining maps/documents. Fig. 3 gives an overview of the number of mining maps and documents per concession.

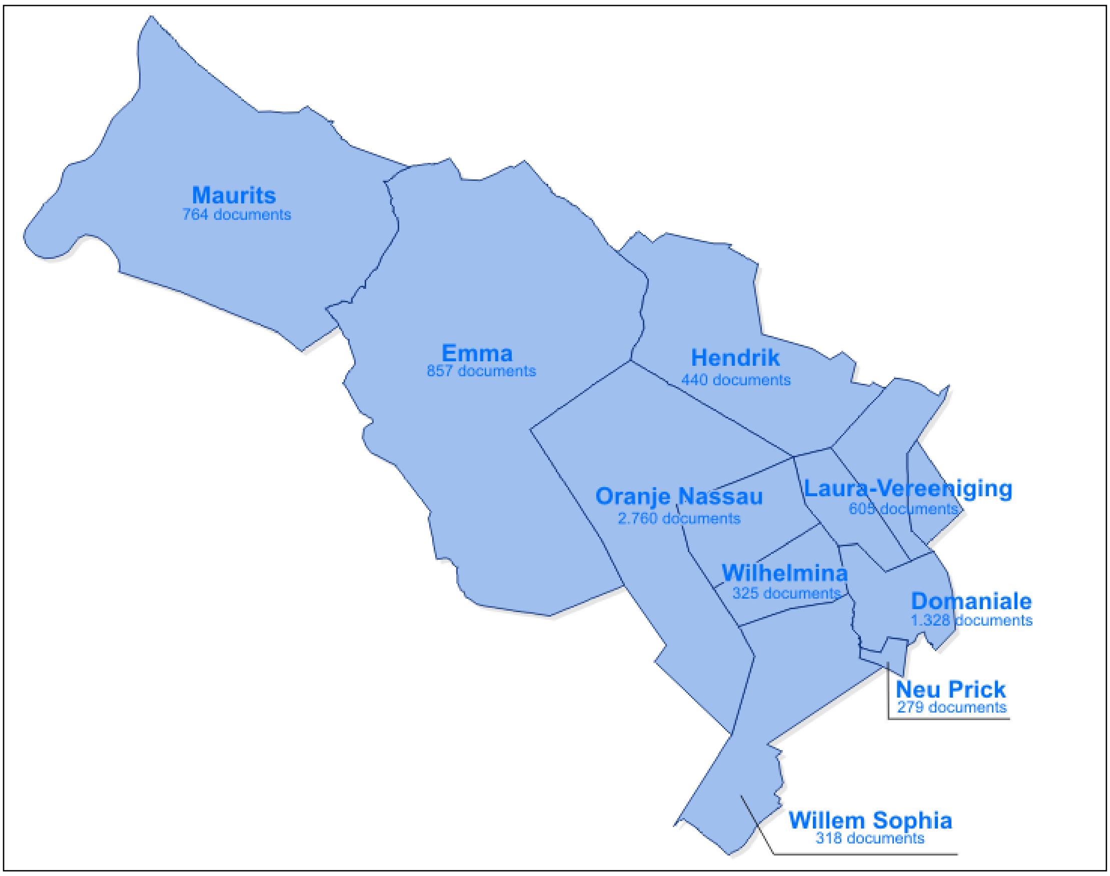

Fig. 3: Number of mining maps and documents per concession

Beside the maps and documents that could be matched to the concessions, the data pool also included 72 general maps and documents. This results in a total number of 7.748 mining maps/documents.

As shown in Fig. 4, geoprocessing the (already georeferenced) data was a basic step for the WG 5.2.2/5.2.3. With the help of the included spreadsheet the data was re-examined and mining maps for each concession were compiled hereafter. The general workflow is given by Fig. 4.

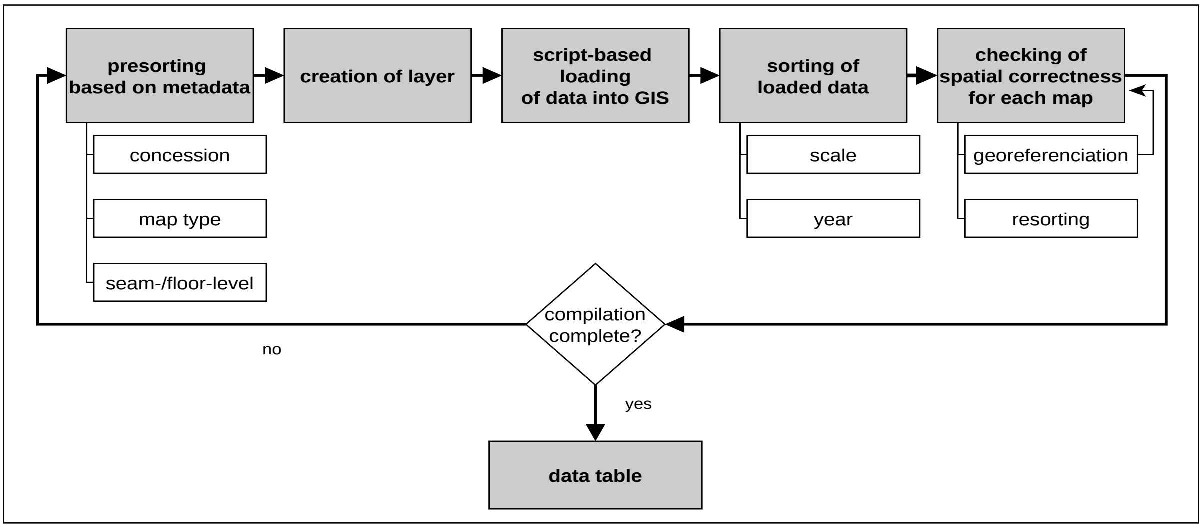

Fig. 4: General workflow of the basic geoprocessing

In general, the mining maps were geoprocessed in the Geographic Information System ESRI® ArcGIS™.

To structurise the data the mining maps/documents were **presorted** based on their metadata. In a three-stage process the maps were sorted by the concession and the map type. Since plans of seams and mine level sheets have the largest share of the data pool the data were also presorted by these features.

For each concession an ArcMap™-document was created. Within these documents **layers** and sublayers were created for the different map types and levels. Based on a script the mining maps were then loaded into the different ArcMap™-documents and matched the related layers and sublayers.

In some cases a further **sorting** of the mining maps was necessary. For these data distinctive features like the scale of a map or the editing status were adduced to create additional sublayers.

Subsequent to the sorting, the spatial correctness of each map was validated individually. The following guidelines were considered in validating the position of a map:

- Borderlines of the concessions
- Given point informations such as shaft sites
- Position of the maps among each other

If necessary, unmachting maps were georeferenced anew. Sometimes the maps had to be resorted.

To assure the completeness of the geoprocessed data the data stock was compared to the data listed in the spreadsheet.

The data pool comprises about 760 cross-sections. From these 481 sections were selected by their content, i.e. only sections that contain relevant information with regard to either the tectonic structure or the detailed stratigraphy in the South Limburg mining district were selected. These selected cross-sections were „georeferenced“ as well, i.e. the sections were digitally scaled to match the real-world geometries. For easy accessibility to the sections the corresponding profile lines were constructed in GIS and linked to the data.

2.3 Classification of project areas

Prior to the evaluation of the mining maps and other collected data, the study area was subdivided into three project areas. An outline map of the study area and the subdivision into the project areas 1 to 3 is given by Fig. 5.

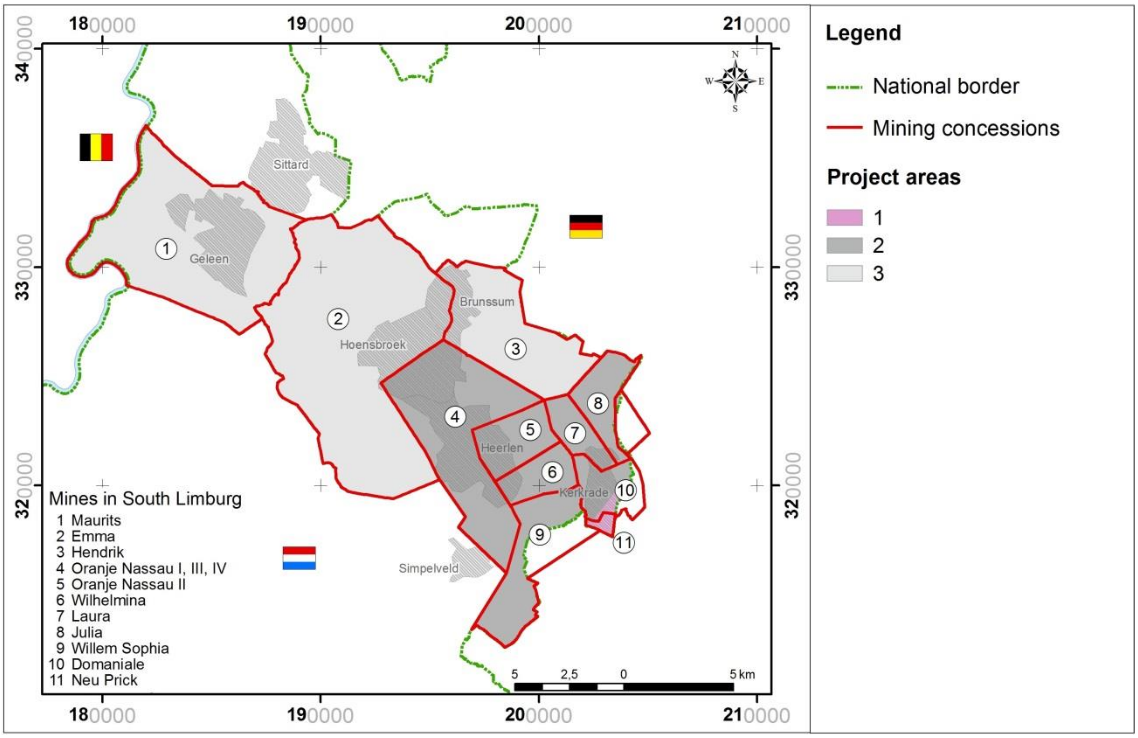

Fig. 5: Outline map of the study area and subdivision in project areas

Basically, different geologic-tectonical conditions and different mining activities were taken as distinctive feature to subdivide the three project areas. Tab. 1 gives a summary of the characteristics of the three distinguished project areas.

In the eastern part of South Limburg, close to the German border, the tectonic situation is characterised by intensive folding with large variations in the dip of the strata. The folding has led to numerous outcrop lines of the coal seams at the top of the Carboniferous bedrock. In parallel with this, the Tertiary and Quaternary sediment cover is no thicker than 40 m which is characteristic for old near-surface mining. This area in the Domaniale and Neu Prick concessions is defined as project area 1.

More to the northwest, the Variscan folding is less distinctive and most coal seams are dipping gently/flat. Instead of intensive folding a kind of undulation rules the tectonic situation. Simultaneously the thickness of the overburden is increasing up to approximately 100 m. This project area 2, consisting of the

Willem Sophia, Wilhelmina, Oranje Nassau, Laura, and Julia concessions, is characterised by modern industrial mining activities.

The project area 3, consisting of the Emma, Hendrik, and Maurits concessions, is characterised by modern industrial mining activities at large depths and below an overburden of large thickness (> 100 m).

Tab. 1: Different characteristics of the three distinguished project areas

Project area	Concession	Municipalities	Characteristics
1	Neu Prick Domaniale	Kerkrade	- historical mining - numerous variable outcrops of coal seams - thin overburden - intense tectonic folding
2	Willem Sophia Wilhelmina Laura Julia Oranje Nassau	Kerkrade Heerlen Simpelveld Landgraaf	- industrial mining - flat dipping coal seams - thick overburden
3	Hendrik Emma Maurits	Landgraaf Heerlen Brunssum Onderbanken Voerendaal Nuth Schinnen Sittard-Geleen Beek Stein	- industrial mining - flat dipping coal seams - very thick overburden

3 Results of WG 5.2.2 “Risks from mine shafts”

3.1 Shafts of historical mining in project area 1

3.1.1 Identification, inventory and digitisation

In a first step all the register shaft lists that have been compiled during the data acquisition period were examined and evaluated. These register shaft lists were existent as hard copy and contained the noted shafts with a consecutive numbering from DOM 1 up to DOM 277 (so-called DOM-lists). These different lists contained either all noted shafts, including those in the German part of the Domaniale-/Neu Prick concessions *(Oude schachten in het veld van de Domaniale Mijn en Buurmijnen (1969))*, or only those noted shafts in the Dutch part of the Domaniale-/Neu Prick concessions *(Oude schachten in het Nederlandse gedeelte van de Domaniale en Neu Prick Concessie (1993))*.

Furthermore, some Excel-files with information on old shafts were delivered by SodM. These different Excel-lists were of varying integrity containing partially only the shafts on the Dutch territory or, for example, containing only information about the modern industrial shafts.

According to the given age of the lists it could be assumed that the list from the year 1969 was the oldest one and that the younger ones (hard copy or digital) were based on this original compilation. This 1969-list was allocated by a general map with the location of the shafts.

In a first step the Ubachsberg-coordinates of the DOM-shafts from this 1969-list were transformed into the RD-New-Coordinate-System and imported into a GIS. Subsequently, all the younger lists were also transformed into the RD-New-Coordinate-System and imported into the GIS. Comparing the results some evident discrepancies in particular shafts were found. A detailed check of these discrepancies revealed only some typing errors/transposed digits resulting from the transcription from the basic list. Therefore, at the start of the investigations, the coordinates from the basic 1969-list were used.

All relevant data referring to the old shafts were integrated into one Excel-file; Appendix 1 contains the sampled data.

Based on this first compilation of old shafts, all the mining documents that already have been sampled, scanned and georeferenciated in the data acquisition period were checked in detail on the depiction of old shafts. For each single shaft that was depicted in one or several of the old mining maps special geoferenciation was performed using streets or older buildings for fitting the mining map in the neighbourhood around the old shaft. Therefore, as an intermediate result a scatter plot of various positions for each shaft was achieved. The final result was the definition of a "most probable shaft-coordinate" and a circle with an "accuracy of position" around this coordinate. This circle with the "accuracy of position" contains all singular scatter plots for the shaft and in most cases also the original shaft position according to the coordinates of the 1969-list.

The determined "accuracy of position" of each shaft depends on different parameters like original scale of the mining map, number and quality of pass points for georeferenciation. In general a grading in 5 m-steps from ±30 m to ±5 m was exercised.

At the beginning of the research project, altogether 55 shafts of historical mining were known in the South Limburg mining district. In the project progression, a further shaft, the Ham I shaft in the Willem Sophia concession close to project area 1 (see Plan 1), was assigned to the "historical shafts". In fact, this shaft was first referred to as an "industrial shaft" due to its depth (125 m), however, there is hardly any information about its abandonment. This fact, the lack of information about its actual conditions, makes this shaft a “historical shaft” in terms of risk assessment so that the known “historical” shafts added up to 56.

According to the available documents, the sinking of the Ham I shaft started in 1878. The shaft has a circular diameter of 7 m. Due to influx of water, the shaft was sunken to a final depth of 125 m in 1880 and served as ventilation shaft afterwards. Shaft fittings comprised buntons, guide rails, a ladder compartment and a piping. For the shaft, two insets are documented. There is no information about the abandonment or a backfill of the Ham I shaft. The available information about the Ham I shaft is summarised in Appendix 1.

The detailed check of the mining maps brought (only) 3 more shafts of historical mining to daylight; these were assigned with a provisional DOM-number (278, 279, 280). An overview of all historical shafts in project area 1 including their corresponding DOM-number is given by Plan 1.

3.1.2 Shaft-Protection-Zones

In North Rhine-Westphalia/Germany it is an obligation to assign a Shaft-Protection-Zone (“Schachtschutzzone”) around an old shaft. Inside this Shaft-Protection-Zone a hazard of subsidence or the formation of a sinkhole is latently existent due to a potential failure of the shafts casing or an insufficient filling of the old shaft with loose soil material. Furthermore, old shafts in general represent a zone where gas might find its way to the ground surface (see report of WG 5.2.6).

According to the criteria that have been applied in the area of historical mining in Herzogenrath/Germany each Shaft-Protection-Zone consists of the following components:

- Dimensions of the shaft (usually referred to as „diameter“)
- Safety margin
- Width resulting from impact of overburden
- Accuracy of position

Dimensions of the shaft

In few cases there was some information about the dimensions and the geometry of the old shafts in the hard copy lists or these could be achieved from the mining maps. If no data was available the dimensions were estimated taking into account the type of shaft. A general result of the analysis was that there was nearly no information about the dimensions, depth and former use of the historical shafts.

Safety margin

The safety margin incorporates potential disaggregation at the side walls of the shaft; usually the safety margin is 1,5 m.

Width resulting from impact of overburden

This component describes the influence of the overburden in case of an actual failure in the casing of a shaft wall resulting in a collapse of the shaft. In this case the soft rock (“soil”) from the Tertiary and Quaternary overburden might move or slip into the collapse structure of the old shaft and cause damage at the ground surface in a wider area around the collapsed shaft.

The area of influence of the overburden is usually delimited by the slope that moves up with an angle of 45° from the top of the Carboniferous bedrock to the ground surface (“angle of repose”). Therefore the width of the impact of the overburden at the ground surface is identical to the thickness of the overburden.

To meet the requirements in project area 1, the thickness of overburden was derived from the digitial elevation model (AHN2[1], 5 m resolution, tiles “69fn1” and “69fz1”) and from 42 drillings that have reached the top of the Carboniferous bedrock. The already available data on the overburden (e.g. REGIS 2.1 and REGIS 2.2) lack of a sufficient resolution and do not cover the whole area.

Accuracy of position

See the remarks in chap. 3.1.1

The delimited Shaft-Protection-Zones can be seen from Plan 1. It has to be noticed that also some industrial shafts are situated in this area; these shafts will be discussed in chap. 3.2.

3.1.3 Risk assessment

3.1.3.1 Bow-Tie-Analysis as general method for risk assessment

As defined in the project proposal, the so called Bow-Tie-Analysis is applied for risk assessment. Initially, the report in hand describes an individual Bow-Tie-Analysis for the technical risk assessment of mine shafts and near-surface mining.

[1] Rijkswaterstaat (2012) - Actueel Hoogtebestand Nederland, version 2; online available: http://www.rijkswaterstaat.nl/apps/geoservices/geodata/dmc/ahn2_5/geogegevens/raster/ (13.10.2015)

The individual analysis will be combined to an integrated Bow-Tie-Analysis for all working groups afterwards. The results of this integrated Bow-Tie-Analysis will be published separately.

The Bow-Tie-method is an effective risk assessment technique that assists the identification and management of risks. Furthermore, the comprehensive layout makes this method a suitable tool for communicating risks. In the following, an outline of the method is presented. A simplified Bow-Tie-diagram is given by Fig. 6.

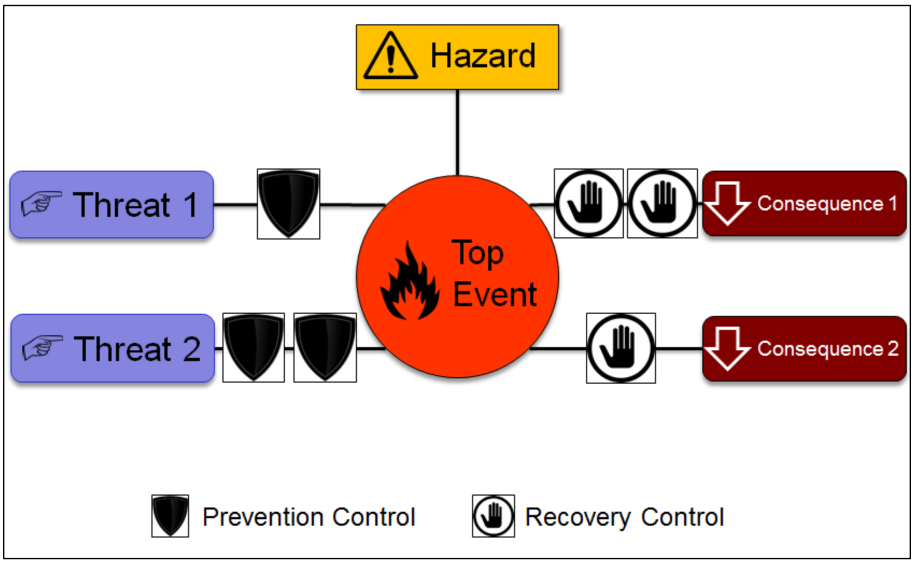

Fig. 6: Simplified Bow-Tie-diagram (Escalation Factors and Escalation Factor Controls not shown)

A Bow-Tie-model revolves around a certain Hazard. When released/activated, an undesired event (Top Event) may arise from this Hazard. Modelled after a chronology, triggers (Threats) that may release the Hazard, i.e. that may cause the Top Event, are placed on the left-hand side. Following the chronology, the Top Event may result in actual impacts (so called Consequences) that are placed on the right-hand side of the model. Threats, Top Event and Consequences are interconnected with lines with each line representing a different potential incident related to the Hazard.

In order to control the Top Event, i.e. both prevent the Top Event from occurring and stop the Top Event from occurring and limit the severity of a Top Event, respectively, the Bow-Tie-method includes so called Controls (Prevention Controls and Recovery Controls, respectively). In the model, the Controls are arranged between a Threat and the Top Event and between the Top Event and the Consequence, respectively. If there is more than one Control, the Controls usually are sequential.

The efficacy of Controls can be reduced by so called Escalation Factors. Escalation Factors themselves cannot cause a Top Event, but they can increase a risk by increasing the likelihood of a certain incident. To prevent these Factors the Bow-Tie might also include so called Escalation Controls.

3.1.3.2 Relative and absolute probabilities and risks

In the following remarks about risk assessment for mine shafts and near-surface mining areas, the ranking terms "high", "medium" and "low" are used to describe a **relative** probability of occurrence (POO) of an incident. In this context it is very important to notice that the POO has to be seen in the context of the **absolute** probability.

According to STRATHAM & TREHARNE (1991) the absolute probability (P) of subsidence occurring at any site within a coalfield can be estimated as follows:

$$P = \frac{N_i * A_i}{T * A_c} \qquad (1)$$

Where N_i is the number of recorded incidents, A_i is the area affected by an incident, T is the time period and A_c is the area of the whole coalfield.

Historical near-surface mining:

In the historical mining area of Herzogenrath/Germany (A_c approximately 12 km²), between 1950 and 2009 about 30 mining related incidents have been recorded officially. The average area affected by these incidents might be 10 m². This makes an assumed probability of approximately $4 \cdot 10^{-7}$ per year.

This absolute probability is slightly higher when it refers to the identified risk areas instead of the whole mining district. In this case in the historical mining area of Herzogenrath/Germany the identified risk areas ("Impact categories EK[1] 1 "red" and EK 2 "yellow") add up to 38 % of the historical mining area; this means that A_c is diminished down to approximately 4,6 km² and the assumed probability is approximately $1 \cdot 10^{-6}$ per year.

In simple words this estimation should show that the absolute probability for the occurrence of a subsidence or a sinkhole with an area of 10 m² under a single building of 100 m² might take place once in 100.000 years. Thus, it is important to see the relative ranking terms "high", "medium" and "low" in the context of this low absolute probability.

Additionally it has to be noticed that even the occurrence of a subsidence or a sinkhole does not obligatory mean that there is severe damage to buildings or even damage to persons.

The above described makes clear that it is not reasonable to approach the problem of historical near-surface mining with the recommendation to remediate

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[1]The abbreviation "EK" refers to the German term „**E**inwirkungs**k**lasse" that can be translated as "impact category"

all identified near-surface mining zones. **The main target should be to manage the existing risks and not to create new risks.** This point of view will be pursued in the following risk assessments in principle.

Shafts of historical mining:

Regarding the shafts of historical mining the situation however is quite different. As these shafts represent locally fixed hazard areas the approach of STRATHAM & TREHARNE (1991) is not feasible. In this case the comparison of the affected area A_i with the whole area A_c (or with the areas of the Shaft-Protection-Zones) is not constructive and would lead to a blurred result.

In the historical mining area of Herzogenrath/Germany about 600 old shafts without remediation/safety measures are registered. IHS has knowledge about the collapse of 3 of these shafts since about the year 1995 (in 20 years). From this data it can be conducted that on the average every 7th year such an incident might take place on one of these 600 shaft locations.

The other way round and transferred to the situation in project area 1 (Kerkrade) with 59 old shafts (10 %) this means that from the statistical point of view a certain of these shafts might collapse every 70th year and therefore with a probability of approximately $2 \cdot 10^{-4}$ per year.

Additionally it has to be noticed that every collapse on a shaft has to be regarded as a severe incident. As the old shafts are normally vertical structures, a collapse will certainly produce damage to nearby buildings and, in case that people are incidentally present, even injuries or fatalities can not be excluded.

The above described makes clear that for the shafts of historical mining it is quite reasonable to approach the problem by aiming at the complete remediation of all identified old shafts. Certainly this will be a long-term project. **Therefore the**

main target should be to eliminate the existing risks in a long-term-project and in the meantime avoid to create new risks. This point of view will be pursued in the following risk assessments in principle.

3.1.3.3 Bow-Tie-Analysis on shafts of historical mining (project area 1)

Shafts of historical mining are regarded to be a major problem in respect of the ground stability in affected areas. In general, the problems might arise from different characteristics of these shafts:

- Abandonment was not regulated in former times.
- Commonly, the shafts were closed by simple techniques; the shaft columns were backfilled with loose material or were even left open.
- The exact position of the shafts is commonly unknown; nowadays the shafts are commonly not visible in the field or the area is already developed.
- Documents on shafts of historical mining are hardly existent.

As discussed in chap. 3.1.2, there are in general two major hazards associated with abandoned mine shafts. These hazards pertain particularly to developed or infrastructural areas in densely populated regions, as they constitute a high risk for public safety and thus might involve restricted land use (AK 4.6, 2013).

- The first, major hazard is a <u>geotechnical hazard</u> that is linked to ground movements in the vicinity of shafts. The potential impact area that might be influenced from the geotechnical hazard is limited by the Shaft-Protection-Zones (see chap. 3.1.2). It is mainly determined by the stability of the shaft in general as well as by the subsoil conditions around a shaft.

- The second hazard arises from the emission of gas to the surface. The area that might be affected by gas emission from shafts is limited by the "Gas-emission-protection-zones" (see WG 5.2.6).

In the following, a Bow-Tie-Analysis is developed for the geotechnical hazard that arises from historical mine shafts; for the corresponding Bow-Tie-diagram see Appendix 2.1.

It should be noted that the Controls in Appendix 2.1 are arranged sequentially for reasons of clarity and comprehensibility. In reality, commonly one measure or a specific combination of different measures is applied. The most suitable measure or combination of measures has to be determined on a case-by-case basis.

The geotechnical hazard arising from historical mine shafts

In a Shaft-Protection-Zone, two types of ground movement are likely to occur. Both movements are determined by gravity, and thus are pointing downward. Dependent on the time behaviour and the spatial distribution one can differentiate between:

- Collapse/formation of a sinkhole: a discontinuous, often sudden downward movement of the surface.
- Subsidence: a more or less continuous downward movement over time and/or space.

In the following Bow-Tie-Analysis that revolves around the geotechnical hazards arising from historical mine shafts, collapse, the formation of a sinkhole and subsidence are defined to be the same Top Event since the Threats, Consequences and Controls are identical for these Top Events.

However, both types of ground movement are different with respect to the time period for the initiation of Recovery Controls and the severity of the incident. In general, collapse or the formation of a sinkhole is highly unpredictable and can cause large damage due to rapid ground movements. On the other hand, subsidences develop over a more or less long period of time that is accompanied with specific signs ("early warnings", cracks in the ground surface or in buildings, etc). These characteristics make subsidences easier to counter by means of Recovery Controls.

Threats releasing the geotechnical hazard arising from historical mine shafts

The general mechanisms that may cause subsidence or collapse of the ground surface related to relicts of (historical) mining are well known among the experts. All these mechanisms have been studied by several authors and have been published in a large number of papers. Among others, LECOMTE & MUÑOS NIHARRA (2013) as well as DIDIER et al. (2008) and MAINZ (2008) deliver comprehensive compilations of the state-of-the-art.

According to these authors, five general mechanisms can cause subsidence/collapse of the ground surface in a Shaft-Protection-Zone. For the Bow-Tie-Analysis these mechanisms are taken as Threats. In most cases, the Top Event will be a result of several Threats combined.

The influence of water is considered to have a key role in the interdependencies that may cause a Top Event.

- **Failure of shaft head:** After abandonment, shafts of historical mining were frequently closed by means of simple techniques (wooden platforms, both on- and near-surface) that warrant no long-time stability. In some cases the shaft column was even left unfilled/open. Afterwards, the closed shafts were mainly covered by soil material. Nowadays, the precise location of historical mine shafts is commonly unknown (see chap. 3.1.1). Thus, the instable shaft head might be overloaded unintentionally and the shaft head is caused to fail subsequently. Often, weathering or biological degradation can also result in a failure of the shaft head.

In this case, the range of the sinkhole is commonly confined to the open diameter of the shaft. However, in dependence of the filling level of the shaft, those sinkholes can be very deep, which is an additional source of risk. Furthermore, a damaged or even failed shaft head can facilitate the influx of water into the backfill column.

- **Failure of deep closure structures:** During the filling of old shafts, the shaft columns were often sealed up against the connected mine workings by means of stoppings or barricades. However, barricades were not always erected or simply were to weak to resist the subsequent pressures by the fill.

  In addition, (mine) water that is commonly aggressive to bricks and mortar can weaken the structures. As a consequence of water influx a non-competent backfill might become saturated; the overload might destroy the barricades and thus, the backfill might collapse or run out into the adjacent mine workings.

  In case a shaft was not covered and the shaft lining remains stable, subsidence is more or less confined to the open diameter of the shaft. If, on the other hand, a collapse or even the run-out of the backfill causes the shaft lining to fail, the whole Shaft-Protection-Zone might be affected.
- **Collapse of backfill material:** The influx of water into the backfilled shaft column can alter the stable conditions within the backfill material. In general, a slow degradation of the backfill material takes place and disrubs the equilibrium of forces within the backfill column. External factors like additional loads of water or tremors can “activate the dynamic mobilisation of the column” (LECOMTE & MUÑOS NIHARRA, 2013) and thus cause the backfill to collapse. In some cases, a collapse can result from an inappropriate installation of the backfill column, i.e. voids may have formed

during the dumping of the material by arching-effects.

The complete run-out of the backfill column is considered to be a special case of this Threat. In most cases, collapse or run-out of the shaft backfill material co-occurs with other Threats like the failure of deep closure structures or the failure of the shaft lining or the shaft head. The failure of deep closure structures is a common trigger for the collapse of the backfill material and can cause the run-out of the whole backfill column into the connected mine workings. Damaged shaft linings or shaft heads can facilitate the influx of water and thus can cause a run-out of the backfill column.

The collapse or run-out of the backfill column, in turn, can destabilise the shaft lining and even may cause the shaft lining to fail. In case a shaft was not covered and the shaft lining remains stable, subsidence is more or less confined to the open diameter of a shaft. If, on the other hand, a collapse or even the run-out of the backfill causes the shaft lining to fail, the whole Shaft-Protection-Zone might be affected by the Top Event.

- **Failure of shaft lining:** For historical mine shafts the failure of the shaft lining often is a direct consequence of the run-out of backfill material. Prior to failure, several factors can weaken the shaft lining. During the operational phase, the shaft lining could have already been damaged. Incautiously executed backfill measures might also have damaged the shaft lining. After the shaft has been abandonded, ageing/weathering is taking place; the degradation of the material can be accelerated by the influence of aggressive mine water. Insufficiently designed closure structures might also have damaged the shaft lining. Damaged shaft linings can facilitate the influx of water, which, in turn, can cause a run-out of the backfill column.

  After failure of the shaft lining, the non-competent overburden most likely collapses into the open shaft column which may result in the formation of a larger sinkhole that might affect the whole Shaft-Protection-Zone.

- **Failure due to water effect and/or particular geologic formation:** As described above, water flow in general can have destabilising effects on the shaft lining, the backfill column and the closure structures. A further effect of flowing water might be the solution of particular geological formations or the displacement of material. These effects can result in the creation of voids behind the shaft lining and can have destabilising effects on the shaft lining. In the relevant project area, no solvable geologic formations exist, but the possibility of material transport especially from the fine-grained silty sands of the Tongeren formation into an unfilled shaft is not implausible.

Consequences from the geotechnical hazard arising from historical mine shafts

In densely populated areas like the South Limburg mining district, relicts of historical mining can be a major threat. According to expections, technical structures such as buildings, infrastructure and supply lines are most likely affected by potential incidents related to the geotechnical hazard of mine shafts. Hence, the Consequences mainly focus on the impact on people and on small-scale impacts on technical structures. Other potential, rather large-scale Consequences like impacts on plants and animals as well as impacts on hydrology and on agriculture are not discussed.

- **Injury/loss of life:** Injury and loss of life can be both a direct Consequence and an indirect Consequence of the geotechnical hazard arising from mine shafts. This Consequence is considered to be the worst case but also the most unlikely one.

  Direct Consequences are most likely given when people fall into an open void that was created by the Top Event or get buried by debris from collapsed structures. Yet in most cases, people might be affected by the indirect Consequences of a Top Event. These Consequences highly depend on the specific damage event and cover a large spectrum (e.g. being hit by falling objects, being hit by an explosion).

- **Damage of buildings:** Differential ground movements, as they are typically related to subsidence or collapse of the ground surface, can have different damaging effects on buildings and their foundations, respectively. The main causes of surface damage arise from tilt, tensile stress and compressive stress. Tilt might be a special threat if high buildings are affected as it might induce collapse of these buildings. In general, a slight or moderate tilt is regarded to be tolerable if tilting is not accompanied by other patterns of damage. Buildings can withstand deformation forces to a certain degree. In more serious cases ground movements can impair the statics of buildings. If a building is directly affected by rapid ground movements (e.g. formation of a sinkhole) the building might collapse or be partially destroyed. Collapse of buildings is the exception; in most cases damage of buildings starts slowly and is commonly accompanied by "early warnings".
- **Damage of infrastructure:** Damage of infrastructure is also induced by differential ground movements. There are several patterns of damage such as fracturing that might lead to deterioration of foundations, corrugations on the running surface, damage and displacement of pavements as well as disruption of drainage. Differential ground movements can cause cracks or leaks in supply lines. These patterns of damage can result in malfunctioning of the system or lead to a loss of the conducted goods. Malfunctioning can also be introduced by tilt of supply lines. Besides a financial loss, the loss of conducted goods can also result in environmental pollution (e.g. leakage of sewage) or even might constitute a separate hazard (e.g. leakage of gas).
- **Social unrest:** As the Top Event might affect personal property and might as well impair the personal sense of protection the Top Event might lead to

social unrest. Social unrest might even get worse if no action is taken by the authorities.

Prevention Controls for the geotechnical hazard arising from historical mine shafts

In general, Prevention Controls for the geotechnical hazard arising from mine shafts can have two different approaches.

The first approach is based on the elimination of the basic triggering Threats for the Top Event. As mentioned above, influence of water (i.e. seepage water) is the most important trigger for the failure of shafts. Another important trigger is the presence of excessive loads in the direct vicinity of a shaft. It should be noted that the elimination of triggers is not sufficient to extinguish a hazard completely.

The second approach is the elimination of the hazard itself by means of “mine technical measures”. For the elimination of the hazard, two basic methods can be taken into consideration. Some comprehensive compilations on the treatment of abandoned mine shafts are given by AK 4.6 (2010) and LECOMTE & MUÑOS NIHARRA (2013).

Naturally, the application of most measures requires the knowledge of the exact position of mine shafts. Thus, the measures have to be based on the results of an on-site-investigation-programme.

- **Limitation of loads on shaft head:** As mentioned earlier in this report, excessive loads can result in a failure of the shaft head. If the location of a shaft is known and the area is not developed yet, it is common practice to blockade the area. As a matter of principle, the direct development of an area with an insufficient secured shaft head is prohibited.

- **Limitation of loads in the vicinity of shaft head:** Shaft failure can also be caused by excessive loads in the vicinity of a shaft. For this reason, land use in Shaft-Protection-Zones is often restricted.
- **Limitation of seepage water influx:** In general, the influx of seepage water is considered to have destabilising effects to the the shaft lining and the backfill column. However, there are different methods to limit seepage water influx into a shaft. The methods focus on sealing of the surface mainly. Some of the techniques are identically equal to techniques of Safeguarding.
- **Site inspections:** To be able to respond to a looming release of a Top Event as soon as possible the shaft sites might be inspected on a regular basis. The (visual) inspections should be performed by a mining expert. If necessary, appropriate action has to be taken.
- **Safeguarding:** The purpose of Safeguarding is the medium-term to long-term ensuring of public safety for years and centuries, as well as a safe, albeit mostly restricted land use (prohibition of development, barrier and signage of hazardous area). However, the hazard itself is not remediated by means of Safeguarding. A geotechnical and hydrogeological assessment is a requirement for the realisation. The measures themselves can include both constructive measures below the surface and constructive measures above the surface. Safeguarding always has to be accompanied by an adapted monitoring programme and periodic maintenance measures.
- **Remediation measures:** The purpose of Remediation measures is a sustainable hazard prevention and an elimination of damages related to mine shafts. The measures are based on the utilisation of a permanently stable Remediation Horizon and a Remediation Body. The source of hazard is fundamentally changed or even widely removed by means of Remediation measures. In principle, the development potential of the former hazard area can be archived after Remediation measures have been executed. Major

advantages of the measures are freedom from maintenance. The respective measures have to be adapted to meet the requirements of the recent or projected land use. As a matter of principle, extensive preinvestigations have to be performed before Remediation measures can be realised. The success of the executed measures has to be verified by means of suitable controls.

Recovery Controls and Escalation Controls for the geotechnical hazard arising from historical mine shafts

In general, Recovery Controls can pursue two different targets: reduction of the vulnerability by means of active prevention measures and/or retrofitting of affected structures by means of reactive measures.

In contrast to Prevention Controls, active prevention measures cannot prevent the Top Event from occurring, but they can minimise the severity of a Top Event; thus, they are Recovery Controls in the proper sense. However, active prevention measures have to be implemented prior to a Top Event to be effective in case of a Top Event.

According to AK 4.6 (2013) three different “scenarios” have to be considered in the land use of an area that is influenced by historical mining:

- First development of an area
- Damaging events impair the usage of already developed areas
- Extension of use or rezoning in already developed areas

- **Regional development planning:** As discussed earlier in this report, shafts of historical mining can be a major risk to both people and technical structures. By means of a proper regional development “risks can be averted before they emerge”. Among others, risk mitigation can be realised by certain prohibitions or building regulations such as adapted site investigation prior to a construction project or adapted construction (see below).

Thus, regional development planning in areas that are characterised by historical mining should always incorporate information about the areas that might be affected by the impacts of historical mining. In new development plannings, these information allow all stakeholders to adapt their plannings and give a certain planning security.

In Germany, regions of active and passive mining as well as mining relicts have to be delineated in land use plans and in development plans subordinated to these land use plans. The municipalities get the information about mining areas from the respective mining authorities (see AK 4.6, 2013).

As a matter of principle, in the historical mining area of Herzogen-rath/Germany, shafts of historical mining usually have to be treated by Safeguarding or Remediation measures prior to the realisation of a construction project in the Shaft-Protection-Zone.

- **Awareness-raising:** Raising the public awareness for the hazards arising from shafts of historical mining is considered to be an effective measure to reduce vulnerability and hence, reduce risk.

  Residents in regions that might be affected by the impacts of historical mine shafts should be informed about potential risks as well as about typical patterns of damages related to shafts, i.e. should be able to recognise the "early warnings". In case of an incident, this knowledge potentially allows the timely initiation of suitable measures for mitigation.

  Naturally, the realisation of this measure requires a certain administrative machinery that acts within a corresponding statutory framework. Among others, the administrative machinery should include a central information service and independent experts that are able to judge the patterns of damage and give advice.

In Germany, several administrative bodies and authorities (e.g. "Bezirks-regierung Arnsberg" as responsible authority for mining in North Rhine-Westphalia) inform the public about the hazards of historical mining in a general way. Stakeholders can consult so called "publicly appointed and sworn experts" that are exceptionally qualified in the assessment of mining related damage and are sworn to act independently and impartially.

- **Change of use:** Change of use is a common planning tool to reduce risk in already developed areas of Shaft-Protection-Zones. By means of this measure, risk is reduced by minimising the number of elements at risk (people in particular). Change of use can only be realised in accordance with the respective statutory framework.
- **Adapted site investigations:** As mentioned above, construction projects in Shaft-Protection-Zones of shafts with unknown position should only be realised after site investigations have been performed. Predominantly, site investigations shall reveal the exact position of a shaft. If the exact position of a shaft is known, the summand "Accuracy of position" becomes irrelevant in the calculation of the Shaft-Protection-Zone; i.e the Shaft-Protection-Zone can be reduced. At best, the construction project lies out of range of the recalculated Shaft-Protection-Zone. With regard to the construction project, no further actions are needed in this case. If, on the other hand, the Shaft-Protection-Zone still overlaps with the construction project usually Remediation measures are required to realise the project. The measures have to be adapted to meet the requirements of the future land use.
- **Adapted construction:** Buildings can withstand deformation forces to a certain degree. For construction projects in Shaft-Protection-Zones, in some special cases, some constructional methods can be realised to prevent future damage of the structures.

- **Quick response team:** Within the existing rescue services there should be a team that should be educated to take the right actions (see Immediate Measures) in the case a Top Event occurs.
- **Immediate Measures:** AK 4.6 (2010) lists several immediate measures ("Erstsicherung") that can be considered to be Recovery controls to limit the severity of a Top Event. First measures could be the evacuation, signage and the barrier of hazardous areas. These measures could also be considered as preventive measures. In dependence of the already occurred damage, immediate static-constructive measures like underpinning the fundament or the backfill of sinkholes with loose material can be necessary for mitigation. Due to the limited durability and stability, the measures have to be accompanied by short-periodic control-, maintenance- and monitoring-measures (e.g. levelling, monitoring of cracks, laser-based surveillance)
- **Constructional support work**: In contrast to the "mine technical measures" mentioned above, support work relates to the elements at risk, i.e. buildings, streets, supply lines and so on. For technical or economical reasons support work commonly is the only option to counter the hazards from mining relicts, e.g. if mining relicts are inaccessible (AK 4.6, 2013). AK 4.6 (2013) lists different approaches for constructional support work.

3.1.3.4 Prioritisation system

The scientific literature about risk management of historical shafts is mostly based on prioritisation systems which try to differentiate between the partial risks resulting from the shaft itself and the actual land use in the area of the Shaft-Protection-Zone. Especially the differentiation of the actual land use is in some prioritisation systems very detailed. This was feasible because these detailed prioritisation systems deal with younger shafts of industrial mining that are documented quite well.

From the inventory and digitisation/georeferenciation of the historical shafts in project area 1 it was quite obvious that there is not much data available about the shafts themselves. The mining documents delivered only some fragmentary information about depth and/or diameter about only a few of these historical shafts. Therefore it was not really constructive to create a detailed prioritisation system partly based on the available data about the shaft.

In addition the results of the georeferenciation showed that for most of the old shafts the accuracy of position was not very high and nearly all of the historical shafts were positioned in the urbanised area of the municipality of Kerkrade. Therefore it was difficult to define the actual land use for each individual shaft.

These problems were encountered first by performing an on-site-inspection of each potential shaft location collecting some information about the land use and the actual situation around the assumed shaft position. These on-site-inspections were performed in march 2015.

As a result of these evaluations, the 59 recorded old shafts in the historical mining area of Kerkrade were classified in three categories with decreasing potential for vulnerability on “goods deserving/requiring protection”:

- Category 1: Shafts in areas with “goods deserving/requiring **high** protection” (Under buildings or very close to buildings);
- Category 2: Shafts in areas with “goods deserving/requiring **medium** protection” (Near buildings, in gardens or streets, etc.);
- Category 3: Shafts in areas with “goods deserving/requiring **low** protection” (Forests, grassland, etc.).

An overview of the classified historical mine shafts is given by Tab. 2. The position of these shafts is shown in Plan 1.

Tab. 2: Overview of the classification of the historical mine shafts

Category	Shaft (DOM-Number)
1	9; 17; 20; 21; 22; 23; 24; 25; 26; 28; 29; 30; 33; 34; 35; 37; 42; 43; 44; 45; 46; 47; 48; 50; 52; 53; 55; 211; 216; 218; 279; 280
2	10; 11; 12; 13; 14; 15; 16; 18; 27; 32 ;36; 38; 39; 40; 41; 49; 51; 54; 56; 214; 215; 263; 264; 278
3	269; 277; HAM I

3.1.4 Conclusions and Recommendations

The performed analysis of the given situation concerning the shafts of historical mining leads to the following main conclusions:

- In the area of the municipality of Kerkrade 59 shafts of historical mining are expected.
- The Shaft-Protection-Zones of 6 shafts of historical mining, situated across the German border, extend into the area of Kerkrade. In these cases there should be a coordination with the German mining authority.
- There is nearly no further information available about the shafts, neither about dimensions and depth nor about an earlier treatment.
- The shafts are mostly situated in a densely populated and urbanised area.

As usually historical shafts are representing one of the major hazards of mining relicts which might evolve dangerous consequences for people as well as buildings; action is strongly recommended.

As obviously the treatment of 59 old shafts will be a long-term project it is recommended to establish first an **On-Site-Investigation-Programme** which should result in a graded **Remediation-Programme**. Furthermore the development of **Administrative Tools** is recommended.

On-Site-Investigation-Programme:

- The actual position of the historical shafts should be investigated by On-Site-Investigations (i.e. small scale hammer probing, seismic investigations, core drillings) in order to verify the actual risk situation and to reduce the Shaft-Protection-Zones.
- This programme should start with the shafts of category 1 and continue with those of categories 2 and 3 but also respect the actual local situation on-site. Depending on the results of the investigations the classification of some shafts might change.
- One main result of the programme should be an improved prioritisation system for the shafts of historical mining as basis for the Remediation-Programme.
- The second main result will be the reduction of Shaft-Protection-Zones because the term “accuracy of position” can be neglected.
- The On-Site-Investigation-Programme should cover a time span of about 5 years.

Remediation-Programme:

- Based on the results of the On-Site-Investigations it is strongly recommended to start with a Remediation-Programme which will perform Remediation measures on most of the shafts of historical mining.
- The Remediation-Programme should start as soon as possible.

- The actual measure for each shaft has to be fixed with attention to the local situation and the results of the investigation programme.
- Based on the assumption that about 40 of the historical shafts nowadays are accessible by technical measures and the experience this Remediation-Programme might cover a time span of about 10 years.

Administrative Tools:

The most important target that can be achieved by the implementation of administrative tools is to prevent the increase of risks by the construction of new buildings or other changes in land use. Therefore it is strongly recommended that any project (construction planning or other development planning) inside Shaft-Protection-Zones should be combined with safety measures. The actual approach has to be determined by experts with sufficient experience on these historic mining issues.

- Existing buildings and present land use inside of Shaft-Protection-Zones usually should have something like a “right for continuance”.
- Further administrative tools should be implemented with respect to the Dutch legislation. These should aim for example at general awareness-raising, general information of stakeholders, emergency plans etc.

3.2 Shafts of industrial mining in project areas 1, 2, and 3

3.2.1 Identification, inventory and digitisation

Analogue to the procedure concerning the shafts of historical mining, in a first step, all available documents on the shafts of modern industrial mining were evaluated.

In this case the most important source was a list of the Rijksgeologische Dienst, Bureau Heerlen, with the designation "*Lijsten met concessiegrenspunten door coördinaten vastgelegd en situatie-en overzichtskaarten van de mijnconcessies van het Zuid-Limburgs Mijngebied"*. The coordinates listed in this document were integrated into an Excel-file, transformed into the RD-New-Coordinate-System and afterwards imported into a GIS showing the corner points/borders of the different mining concessions as well as the position of the industrial shafts.

A comparison with more recent lists and documents revealed no severe differences between the data sets. Only for one of the industrial shafts a typing error was detected in a more recent list. Therefore, for all further work, the transformed coordinates from the above mentioned original list were used.

Information about the abandonment of deep mine shafts in project areas 2 and 3 and documents related to the final planning, respectively, were available from the „Nationaal Archief", The Hague. The relevant data were digitised and were made available in PDF. Further information about the abandonment of deep mine shafts was taken from SodM's annual reports („Jaarverslag").

3.2.2 Examination of reports on shaft remediation

Within the 1960s, 70s and 80s in the coal-mining area of South Limburg the shafts of deep mining (industrial mining) were closed and secured. These shafts were backfilled and covered up according to the guideline "Nadere regelen Mijnreglement vullen van schachten" (Stcrt. 1973, 10) of 05.01.1973. In 1994 the last abandoned shaft, the Beerenbosch II shaft, was secured.

Based on the existing documentation the shaft stabilisation (according to the implementation planning) will be assessed. Taking under consideration the rising mine water those shafts will be evaluated under safety measures and the Shaft-Protection-Zone will be defined.

The examination and assessment of the reports and documents about the remediation of the industrial shafts led to an extensive report of its own. For lucidity and readability this extensive part was divested of the main report and is annexed in Appendix 4.

In Appendix 5 a table with all 39 shafts and their securing concepts is listed.

3.2.2.1 Securing of abandoned industrial mine shafts in the coal mining area South Limburg

Between 1967 and 1983 altogether 38 of the existing industrial shafts were secured by the following methods.

- **Method I: „shaft barrier as abutment“**

  On the level of the topmost floor an abutment made out of concrete is embedded and rests with its bend lower edge upon the surrounding rock in the range of the shaft-landing.

  On the topmost floor an abutment of iron beams covered with a concrete board, which rests with its bend lower edge upon the surrounding rock, has to be installed. By this mean the pressure occurring from the load-bearing filling and the backfilled loose material is spread best. Above the barrier the shaft column is filled with backfill material.

  In Fig. 7 the securing concept I is shown.

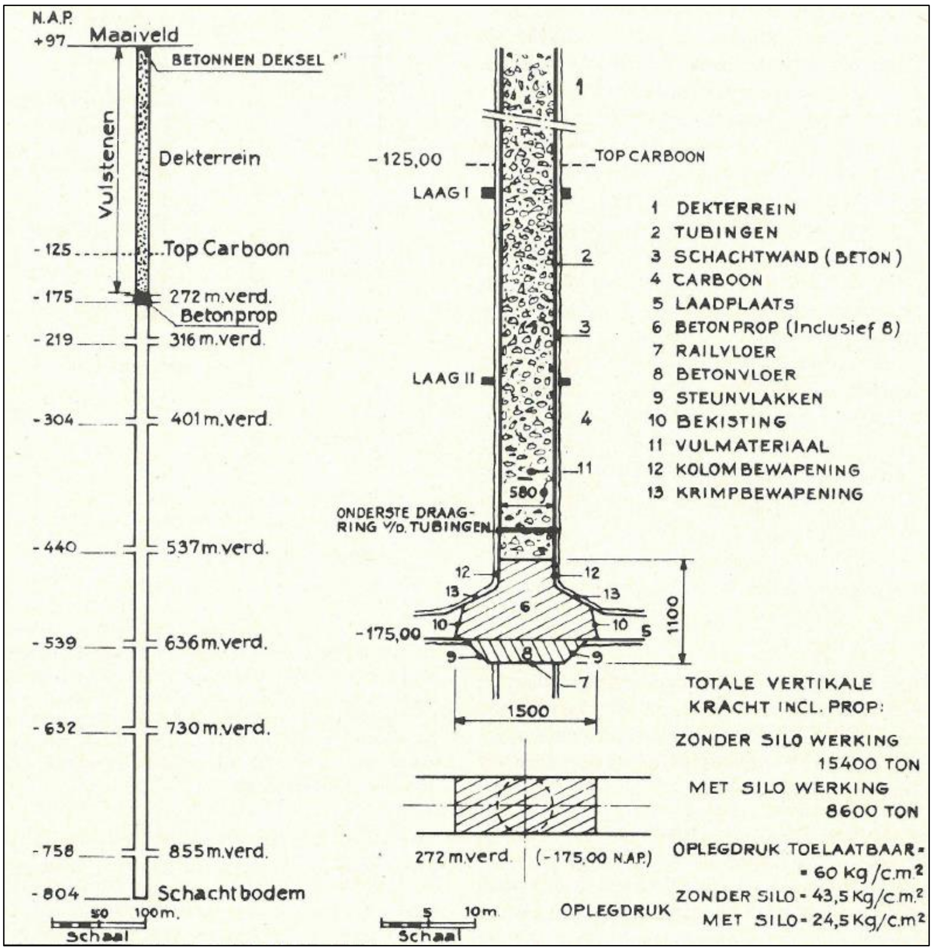

Fig. 7: Schematic sketch securing concept I, shaft barrier

Method II: „shaft barrier as load-bearing filling“

This method is divided into three variants.

Variant IIa: shaft is backfilled overall with concrete from the level of the topmost floor up to the ground surface (Fig. 8).

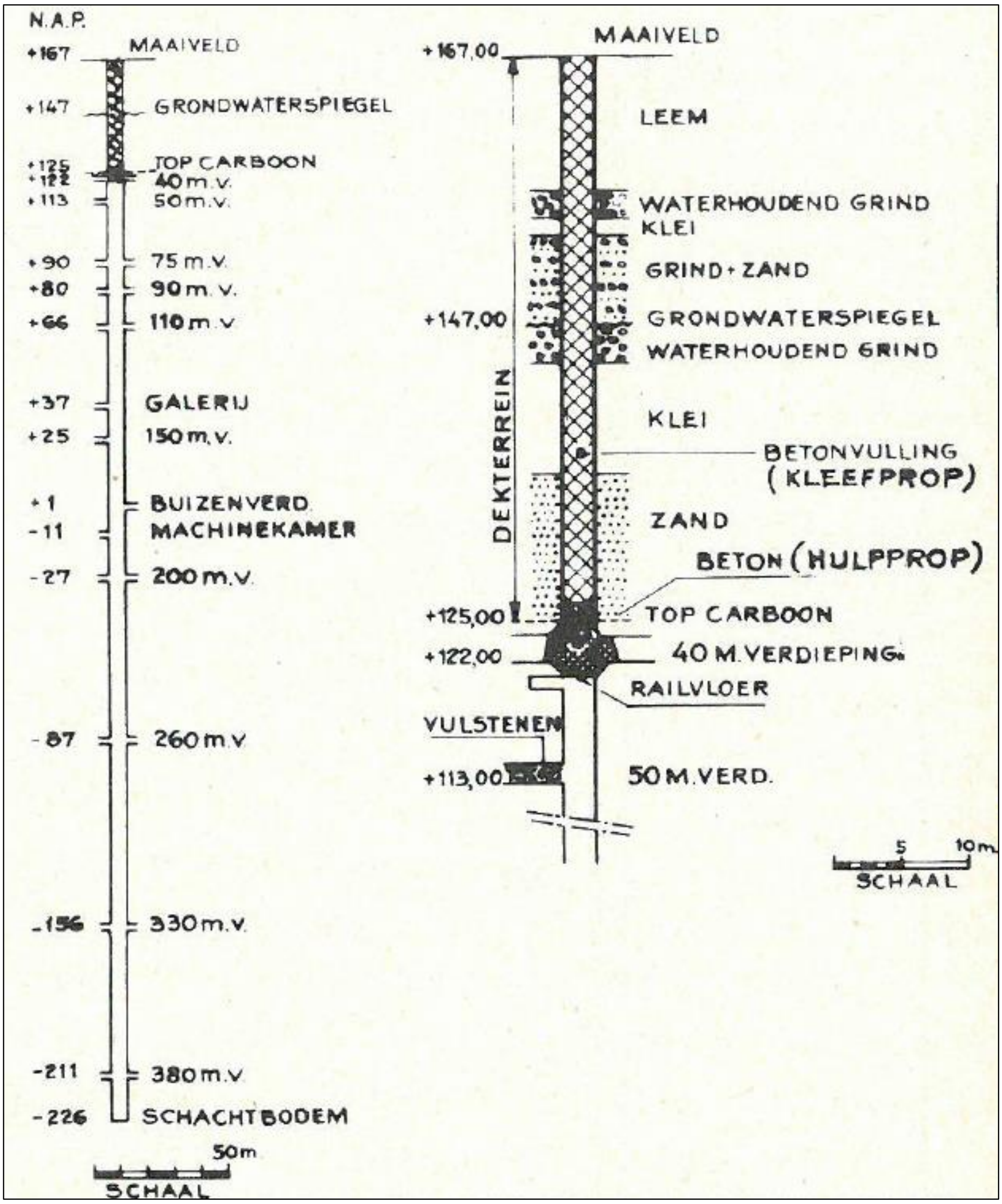

Fig. 8: Schematic sketch securing concept IIa, shaft barrier

Variant IIb: above the topmost floor an abutment is embedded. Furthermore above this barrier the shaft is backfilled with clastic material up to the ground surface. Finally the shaft head is provided with a shaft cover (Fig. 9).

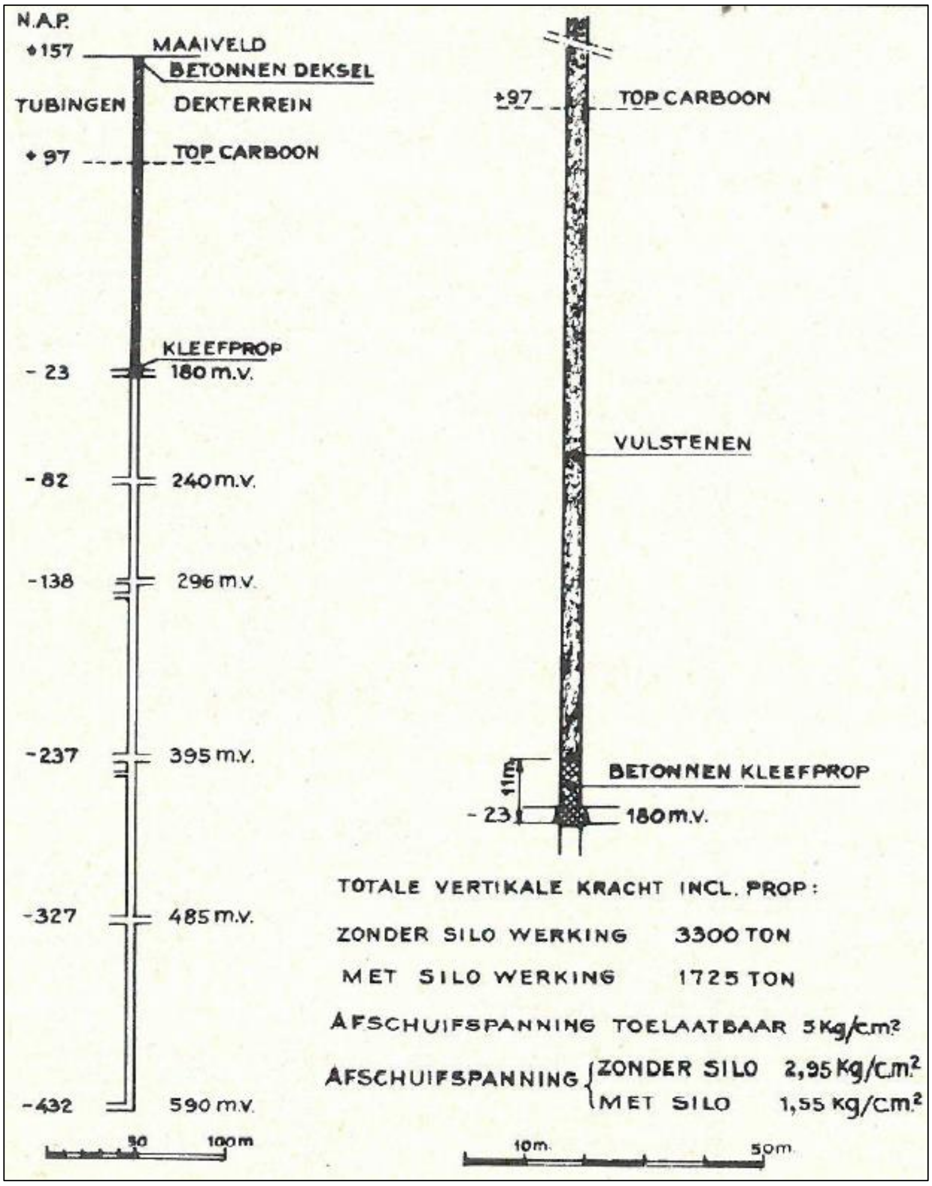

Fig. 9: Schematic sketch securing concept IIb, shaft barrier

Variant IIc: major parts of the shaft column are backfilled alternating with load-bearing fillings and clastic material. The fillings are located on the level of insets respectively above those. The topmost filling seals the topmost floor completely and reaches the overburden. Finally the shaft is provided with a shaft cover (Fig. 10).

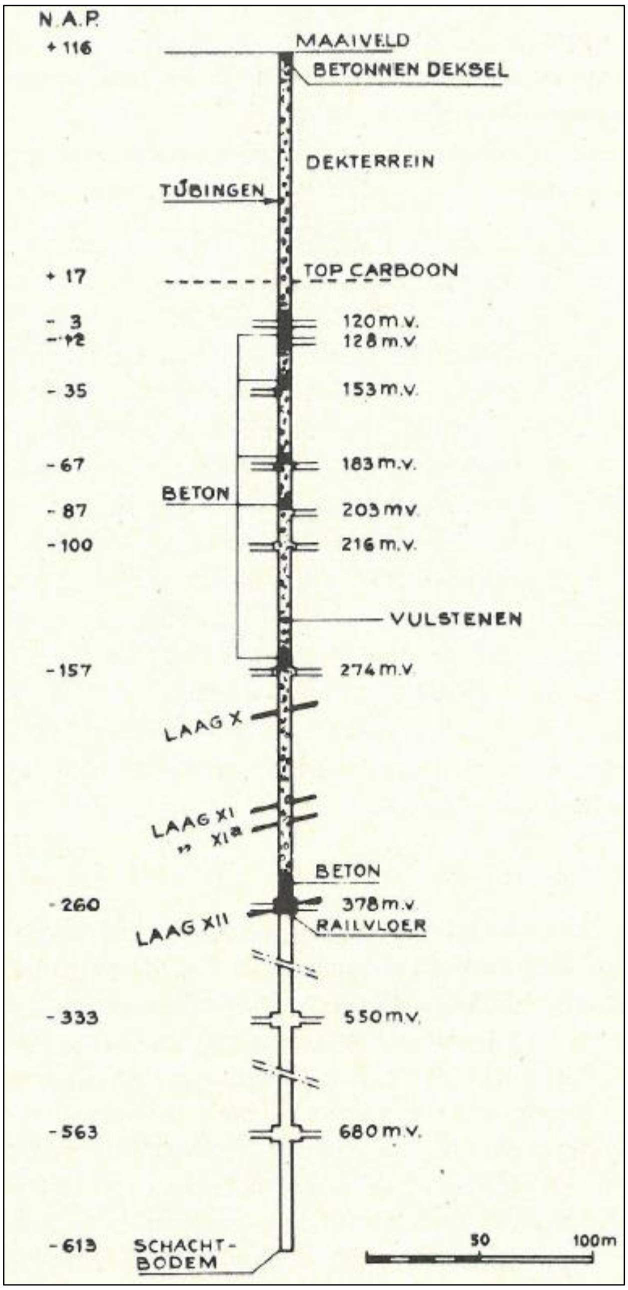

Fig. 10: Schematic sketch securing concept IIc, shaft barrier

The dimensioning of a shaft barrier is affected by its strength and the following aspects:

- The barriers qualities: the dimensions of the contact surfaces and/or the quality of the abutment between the barrier and the surrounding rocks has to prevent any leakage.
- The barriers shape: the load-bearing filling spreads to each sides beyond the shaft cross-section into the shaft-landings. The load-bearing filling always is embedded into the shaft diameter.
- Within the strength calculation the size of the barriers as well as the load-bearing capacity of the different types of fillings play an important role. The occurring load consists of the force exerted by the fill material upon the concrete barrier as well as the dead weight of the concrete barrier itself.
- The maximum mass (normal stress) of the abutment and the shear stress of the load-bearing filling are relevant for the load-bearing capacity.

Within the securing concepts I, IIb and IIc the shaft is backfilled with clastic material overall and provided with a shaft cover. The concrete covers have a permitted load factor of 10 t/m^2 (100 kN/m^2). The covers are provided with an opening for refilling.

The securing concept I was used under the following conditions:

- Heavy overburden
- Major shaft cross-section
- Even shaft wall

The securing concept II was used under the following conditions:

- minor overburden
- small interspace between the topmost floor and the top of the carbon layer
- minor shaft cross-section

3.2.2.2 Regulation „Nadere regelen Mijnreglement vullen van schachten“

According to the code „Mijnreglement 1964“, Paragraph 136 and 143 the regulation „Nadere regelen Mijnreglement vullen van schachten“ came into force on 05.01.1973. In this document the handling of abandoned mining is regulated.

This document essentially contains the following requirements:

- No open connections exist between the shaft to be backfilled or a part thereof to be filled, and an underground drift or another underground working.
- The filling must have positional stability by water-exposure (washout).
- Safe closure between overburden, shaft and mine workings.
- Exclusion of precarious earthwork at banking level. Sealing constructions (dam) must be designed for emerging surcharge and hydraulic pressure.
- A maximum load of 60 kg/cm^2 (6 MN/m^2) on the bed rock has to be estimated for the dimensioning of a seal.
- The maximum shearing stress between shaft lining and load-bearing parts of the filling constitutes 3 kg/cm^2 (300 kN/m^2).
- The fill in with loose material for the load determination is permitted.
- Remarks for construction: sounding during backfilling, installation technology (pipelines).
- Monitoring of the filling level, levelling and length measurement after backfilling.
- Shaft closure at the surface by means of a manhole cover (remark: in general 10 t/m^2 - 100 kN/m^2).

After entry into force of the regulation in 1973, eleven of 39 shafts were secured. All other shafts were secured before 1973.

3.2.3 Detailed analysis of deep mine shafts

According to the information at hand, 36 out of a total of 39 industrial mine shafts which have been part of this analysis have received a durable treatment by installment of a concrete plug on the topmost floor level (see Appendix 5). An exception to this is shaft Neuland.

The shafts have been backfilled mostly with loose material, partly also with concrete, above the plug and up to the surface. Generally, the material was loosely dumped into the shaft. The documentation does not contain any information about the shaft undergoing salvage work prior to its closure, i.e. removal of guide rails, scaffolds, transverse beams etc. After the shaft was backfilled it received a concrete cover with a manhole for monitoring and further backfilling.

Apart from the Louise and Laura II shafts, the shafts that have been secured with a plug have remained without a backfilling below the plug. Based on the depth in which the plug has been installed, the shaft diameter, the shaft's total depth and the number and size of insets each shaft has a potential void volume that can take up caved material in case of a failure. This potential void volume is also listed in Appendix 5 with the caveat that the number and size of insets have not been considered.

3.2.3.1 Assessment of the shaft lining in zones with unstable overburden

In zones with unstable overburden 26 shafts are lined with metal tubbings, a further 9 are lined with brickwork/masonry and another 4 are lined with concrete.

Based on the available information, an assessment of the state and condition of the shaft lining regarding its stability and impermeability is not possible for any of the 39 shafts. It has to be assumed that the shaft lining can fail, in the event that the backfilling fails or if there is no backfilling at all. The rising mine water affects the stability of the shaft lining in a positive way, as the hydraulic gradient between the mine water and the groundwater is reduced and in this way the forces acting on the outside of the shaft lining are also reduced. By the same token the rising mine water level reduces the risk of an influx of water or fluid-like loose material.

One special case is the **Melanie shaft**. According to the available documents the shaft has not been backfilled on top of the concrete plug. Instead it has been used as a water reservoir. Here, the integrity of the shaft lining is of fundamental importance to the stability of the surface and therefore needs a constant monitoring.

3.2.3.2 Assessment of the stability of the backfilling

Cement-based cohesive backfilling

The Willem I, Willem II, Buizenschacht, Beerenbosch I, and Nulland shafts (all of Domaniale) as well as the HAM II shaft (Willem Sophia) have been backfilled with concrete between the plug and the surface. The Beerenbosch II shaft has received a cohesive, partial backfilling. Because of the hydraulic-setting cement the stability of the backfilling is given under the condition that the backfilling was done according to proper form.

The backfilling of the Willem II shaft (Domaniale) was drilled through in 1980. The drill cores were put through testing of their compressive strength. The uniaxial compressive strength ranged between 6,9 MN/m² and 15,1 MN/m². This was only half of the specified value in the planning specifications. As for the stability of the backfilling, the measured compressive strength has to be considered as sufficient.

<u>Backfilling with loose materials</u>

A backfilling with loose materials was done in 29 out of 39 shafts.

The backfilling was applied either on top of a plug or by a complete backfilling of the entire shaft. The backfillings were mostly done with waste rock and washery tailings and either by loosely dumping the material into the shaft or by using pipes. Preceding salvage works are not documented so it is likely that fixtures like guide rails, scaffolds, transverse beams and pipes remained in the shaft. If these fixtures remained and the shaft was backfilled with loosely dumped materials it is possible that the fixtures took damage or tore off. This may have damaged the shaft lining. At the same time it is possible that torn off fixtures clogged the shaft so that a void-free backfilling could not be achieved. There is also the risk of voids building behind fixtures, if the backfill material cannot flow freely around these fixtures. These voids can result in a later settling of the backfilling.

The backfill could be monitored through manholes integrated into the shaft cover slabs. This was done for some time, but today the manholes are inaccessible because they have been covered in concrete. **Right now, in some cases the status of the backfill cannot be monitored.**

As long as the concrete plug remains intact, the loose material of the backfill cannot flow into the mine workings at the upmost inset. The possible mechanisms behind the failure of the backfilling stem from two basic scenarios. The first scenario involves the stability and integrity of the plug itself and is not directly influenced by the properties of the backfill. The second scenario involves movement that is based on properties of the backfill material.

In the cases where the plug fails the loose material of the backfill will relocate into the open insets as well as into the remaining parts of the shaft. This can occur as a sudden process but requires the sudden and complete failure of the plug. This scenario can be regarded as very unlikely.

In cases of a partial failure of the bedrock surrounding the plug loose material from the backfill can also be relocated into the unfilled shaft. The height of the backfill would subsequently decline over time. A vertical flow of water within the backfill can further promote the relocation of material into open mine workings. In this scenario, a rising mine water level has a positive, stabilising effect once the water level reaches the plug.

A relocation of material within the backfill can also lead to a declining backfill height and can result from water interacting with backfill material, especially claystones and shales that are components of washery tailings. This can lead to subsidence in the backfilling that corresponds to a 10 % loss of volume; these are results of a research project (SCHERBECK et al., 2012).

Based on these scenarios it has to be assumed that subsidence and settling of a loose material backfill can still occur in the long-term. This can negatively impact the inner bedding of the shaft lining. The height of the backfill should therefore be monitored so that the shaft can be refilled as soon as the need arises.

Special cases are the Baamstraat, Neuland, and Catharina shafts.

The **Baamstraat shaft** has a total depth of 21 m. It has been backfilled with loose material up to the top of the lowest inset. This inset had also been backfilled previously with loose rock material. As such, there is no increased probability of material relocating from the shaft backfill into the mine workings.

In the **Neuland shaft,** instead of a concrete plug, a 0,75 m thick arched concrete roofing was installed at a depth of 85 m. This is around 22 m below the upmost inset. It is unknown whether this inset was sealed off. The backfill is composed of rubble and includes the area of the inset. As such, it is entirely possible that the loose material relocates into open mine workings which adds to the effects that can cause a decline in height of the backfilling, as described above.

The **Catharina shaft** was completely backfilled with loose materials. Additionally, the backfilling was stabilised by injection grouting down to a depth of 90 m. The stability of the grouted backfilling is monitored with extensometers.

3.2.3.3 Assessment of the stability of the concrete plug

The concrete plug as a sealing element for the shaft comes in two different varieties. One type is constructed at an inset, i.e. it is supported by the floor level. The other type is a shear plug. An exception to this is the arching structure that was built in the Neuland shaft. The type of plug in each of the shafts is given in Appendix 5. As far as both types' stability is concerned the following predictions can be made based on the existing documentation:

Floor-supported plug

- Based on the plug's shape the load transmission into the surrounding bedrock can be considered as very good.
- The static dimensioning of the plug considers both the unladen weight of the plug itself as well as the additional load from the water-saturated backfilling. The effects described by the silo theory have also been considered. The design load, while comprehensible, does not include a safety margin.

- The maximum design load on the surrounding bedrock of 6 MN/m² can be considered as sufficiently conservative.
- The statical system is insensitive to a rising mine water level.
- The composition of the concrete is unknown and as such the resistivity against exposure to chemical agents is also unknown (e.g. chemical interaction with mine water).
- The actual construction work is not sufficiently documented.
- If build according to specification, the likelihood of a failure of the plug is very low.

Shear Plug

- The two most basic requirements for a sufficient load transmission into the surrounding bedrock are firstly a preferably large ratio between the length of the shear plug and the shaft diameter and secondly a proper bond between the rock and the shaft lining.
- A confirmation of a sufficient load transmission between the shaft lining and the surrounding bedrock is not part of the existing documentation. There is no information regarding a consolidation of the annular space.
- The static dimensioning of the plug considers both the unladen weight of the plug itself as well as the additional load from the water-saturated backfilling, the effects described by the silo theory have also been considered. The design load, while comprehensible, does not include a safety margin.
- The static dimensioning considers a maximum shear stress of 300 kN/m² between the plug and the shaft lining. This can be considered as sufficiently conservative even in the case of full submersion in groundwater.
- The composition of the concrete is unknown and as such the resistivity against exposure to chemical agents is also unknown (e.g. chemical interaction with mine water).

- The actual construction work is not sufficiently documented.
- If build according to specification, the likelihood of a failure of the plug is low. However, there is not much of a safety margin, because the tie-in length of the shear plug into the stable formation is often rather short as well as the ratio between plug length and shaft diameter is unfavourable.

The most important factors for the functionality of the plug is the shear and compressive strength of the surrounding bedrock. With the exception of the Buizenschacht, Beerenbosch I, Willem I, and Willem II shafts (Domaniale) all plugs have their foundation in the Carboniferous bedrock. Under normal conditions the Carboniferous bedrock is of sufficient strength for a proper transmission of loads from the plug into the rock, however in the presence of coal seams or near geological faults this is not necessarily the case.

The aforementioned Buizenschacht, Beerenbosch I, Willem I and Willem II shafts (Domaniale) have their plugs installed into the transition zone between Carboniferous bedrock and the overburden. The tie-in length of the plugs into the Carboniferous bedrock is around 4,5 m at the Buizenschacht, around 3,5 m at the Willem I shaft, around 7 m at the Willem II shaft and around 5 m at the Beerenbosch I shaft. The upper parts of these shafts have been filled with concrete. The overburden at these shafts consists of an alternating sequence of sand, silt and clay which are likely saturated and not entirely consolidated. Because of the short tie-in length it is possible that unconsolidated overburden migrates into the open shaft beneath the plug if the shaft lining and surrounding bedrock fails just under the plug. Based on a difference between mine water and groundwater levels, water currents can enhance this process. Another unknown factor is the level of weathering of the top of the bedrock. Empirically, there is a layer of about 1 m thickness of weathered rock so that the tie-in length into stable bedrock is further reduced. Furthermore the ratio between plug length in stable bedrock and shaft diameter is < 1 for the Willem I and Wilhelm II shafts (Domaniale), which is below a safe threshold.

Based on the position of the plugs at the transition between Carboniferous bedrock and overburden and the low tie-in lengths, the safety level at the 4 shafts Willem I and II, Beerenbosch I and Buizenschacht (Domaniale) is rated to be very low.

The Neuland shaft was treated with an arched concrete roofing of 0,75 m thickness at 22 m below the upmost inset in 1919. This cannot be considered as a permanent safety measure. The safety level of the Neuland shaft is hence rated to be very low.

3.2.3.4 Assessment of the stability of cover slabs

In general, the cover slabs were designed for a permissible load of 10 t/m² (100 kN/m²). The cover slabs were founded close to the ground surface on top of the shaft linings in place. Based on general experience, the permissible loads can be considered to be sufficiently designed, provided that the function of the slabs is not impaired. However, the introduction of additional loads, e.g. loads from buildings or additional cover with soil, is prohibited without further statical assessment of the slabs. The failure of a cover slab might cause damage at the ground surface if the underlying backfill column has moved from its initial position, e.g. due to sagging. In case of a failure, provided that the shaft lining remains stable, the stability-related impact at the ground surface is limited to the area directly above the slab. If the shaft lining does not remain stable an angle of break of 45° has to be considered.

3.2.4 Residual Shaft-Protection-Zones

In general the stability of a surface area where the overburden is affected by a nearby abandoned mine shaft is ensured under the conditions

- that the shaft cover and the shaft lining in zones with an unstable formation are stable with respect to all acting forces

and

- that the shaft lining or the backfill is impermeable to an influx of fluids or a fluid-like formation, both currently and in the future

or

- that the shaft is completely and permanently backfilled with a stable, erosion-resistant material in zones with an unstable formation (concrete, cohesive material).

If the above listed conditions are not met, subsidence or sinkholes may occur. This can cause physical injury and property damages in the affected area. The area of the overburden that can possibly be affected by a failure of the shaft lining or backfill is the so called Shaft-Protection-Zone (see Fig. 11). The Shaft-Protection-Zone for a vertical mine shaft is based on empirical values and geostatics as supported by the guidelines of North Rhine-Westphalia (BEZIRKSREGIERUNG ARNSBERG, 2007):

shaft diameter
+ 2 x thickness of shaft lining
+ 2 x 1,5 m safety margin
+ 2 x height difference between the surface and the stable bedrock
+ (i.e. thickness of unstable overburden)
= diameter of the Shaft-Protection-Zone

This formula is applicable for up to 100 m thickness of the unstable overburden. If the thickness of unstable overburden is more than 100 m, the Shaft-Protection-Zone is assumed to have a flat radius of 100 m. This practice is based on empirical data from the Ruhr-area in Germany.

The Shaft-Protection-Zones for the shafts that have been examined in this survey are listed in Appendix 5.

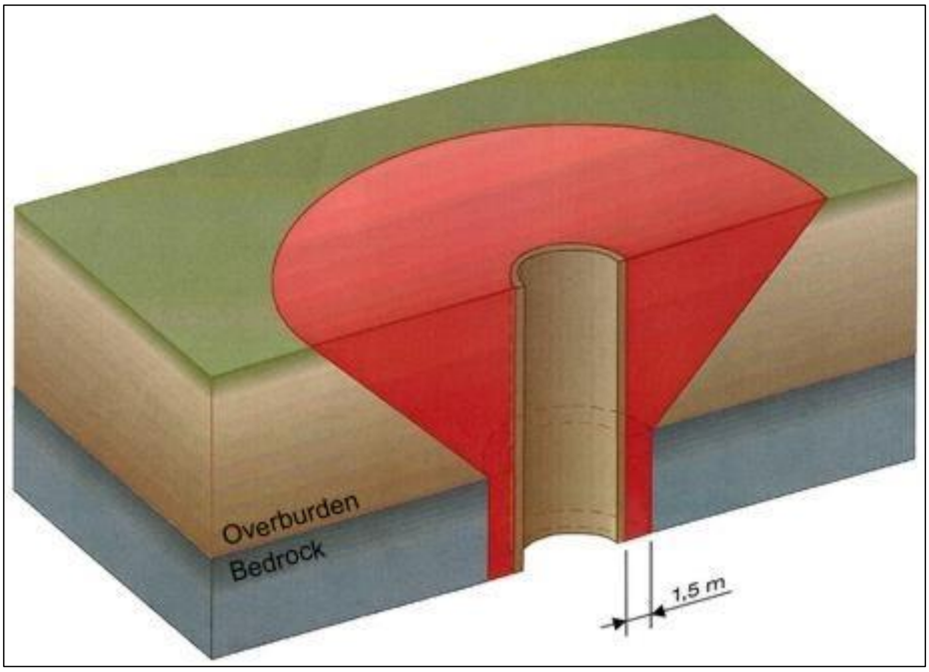

Fig. 11: Schematic profile of the Shaft-Protection-Zone of a vertical mine shaft

3.2.5 Bow-Tie-Analysis on shafts of industrial mining

There are no verifiable documents regarding the construction of the actually performed safety measures in the 39 industrial mine shafts. This concerns in particular the execution of the preparatory work (e.g. salvage work, shaping of the plug etc.) and audits on the execution (e.g. examination of concrete qualities).

A detailed analysis of the individual safety measures that have been applied to each shaft is therefore not possible.

In general, the Bow-Tie-Analysis that was designed for the geotechnical hazard that arises from historical mine shafts (see chap. 3.1.3.3) can be transferred one-to-one to the geotechnical hazards that arise from industrial mine shafts. However, there is a major difference between the historical mine shafts and industrial mine shafts in terms of their general risk level. Due to the fact that most industrial shafts were remediated in accordance with a guideline, the general hazard level of industrial shafts is regarded to be considerably lower in comparison to the general hazard level of historical mine shafts. Hence, there are some slight alterations in the Bow-Tie-diagram for the geotechnical hazards that arise from industrial shafts; the corresponding diagram is shown in Appendix 2.2.

Additional Threats:

- Failure of shaft lining in unstable strata: Industrial shafts, in general, feature considerable long shaft linings within the overburden strata (see Appendix 5). Hence, the sections that are situated within the overburden strata often also intersect larger layers of partially unstable strata. This fact gives the Threat a certain significance. The general mechanisms that are related to the failure of a shaft lining are described in chap. 3.1.3.3.
- Failure of shaft plugs: As decribed above as well as in Appendix 4 most industrial mine shafts were remediated using shaft plugs as sealing element. The plugs are regarded to be a special form of deep closure structures. The stability of these sealing elements mainly depends on the grip length of the plug. A failure is most likely given when flowable overburden material is able to pass the plug. This process requires a failure of the shaft lining in the respective section of the shaft.

Additional Prevention Controls:

- Monitoring industrial mine shafts: As described above, the run-off of the clastic backfill column is a common mechanism that might lead to the Top Event. In general, the industrial shafts were backfilled using clastic material. The length of the columns can reach up to approximately 380 m (see Appendix 5.2). Alterations in the backfill column will most likely reflect themselves at the surface of the backfill column. Because most industrial shaft heads are accessible there is a good option for a monitoring using sounding measurements.
- Remediation measures at 6 shafts: According to the performed assessment of the safety level of industrial shafts there are only 6 shafts that constitute a major hazard (see below). Additional remediation of these shafts is regarded to be a useful way to eliminate the hazards that arise from these shafts. Safeguarding is not required when Remediation measures were carried out.

So far, no surface damages have been documented in the area of the 39 industrial mine shafts. This fits to the fact that there is also no information about claims of damages outside the South Limburg coalfield, which are due to material failure of a plug. There are, however, examples of cave-ins at the surface that were triggered by a failure of the bedrock surrounding the plug. The probability of such a failure is increased if the plug is built into bedrock with unfavourable geotechnical conditions. This includes an insufficient embedment in stable strata. The safety level of these kinds of shafts has to be rated as very low.

Shafts treated with shear plug where the ratio between the plug length and the shaft diameter is low are considered to have a low safety level.

Shafts treated with a loose material backfill are considered to have at most a medium safety level, because the stability of the backfill cannot be verified.

A high safety level is reserved for shafts where the plug and its embedment into stable strata is of sufficient length and where the backfill is made of concrete in zones of unstable overburden.

Shafts where the safeguarding measures are state of the art regarding their longevity can be regarded as permanently safe.

Based on the available information and experience, the safety level of each shaft is assessed below. The relative classification is based on the assumption that the remediation measures were executed in accordance with the available documentation and that the backfillings from loose materials kept their functionality as a securing element.

- **not treated yet:**
  Melanie
- **very low safety level:**
  Buizenschacht, Willem I, Willem II, Neuland, Beerenbosch I (all Domaniale)
- **low safety level:**
  Willem I (Willem Sophia), Julia I, Julia II, Louise
- **medium safety level:**
  Willem II (Willem Sophia), Baamstraat, Sophia, Oranje Nassau (7 shafts), Wilhelmina I, Wilhelmina II, Emma I-IV, Hendrik I-IV, Maurits I-III, Catharina, Laura I, Laura II
- **high safety level:**
  Nulland, Ham II
- **permanently safe with state of the art treatment:**
  Beerenbosch II

The Shaft-Protection-Zone for each shaft has been defined as outlined before and is shown in Plan 6.

The colouring of the Shaft-Protection-Zones has been chosen based on the colour codes used for the impact categories of coal seams (see chap. 4.2.1.1); the outcomes of the assessment is outlined in Tab. 3.

Tab. 3: Outcomes of the assessment of the industrial mine shafts

Category (colour)	Shaft	Mine	Safety level	Suggested action
1 (red)	-		-	-
2 (yellow)	Buizenschacht, Willem I/II Beerenbosch I Neuland Melanie	Domaniale Willem Sophia	Very low or not yet treated	Investigation of current situation and remediation measures in the short-term
3 (blue)	Baamstraat Louise Catharina Willem I/II Sophia Laura I/II Julia I/II all 7 shafts Shafts I/II Shafts I - IV Shafts I - IV Shafts I - III	Domaniale Neu Prick Willem Sophia Laura-Julia Oranje Nassau Wilhelmina Emma Hendrik Maurits	Low and medium safety level	Periodic monitoring of the backfilling column based on the current surface use
4 (green)	Beerenbosch II Nulland HAM II	Domaniale Willem Sophia	Permanently safe or high safety level	Periodic monitoring of shafts Nulland and HAM II

The Shaft-Protection-Zones of shafts with a very low safety level are shown in yellow. The Shaft-Protection-Zone should not be used for sensible infrastructure. Building development should be avoided. Access by people should be minimised.

The following figures show the current use of the shafts.

Fig. 12: Shaft-Protection-Zones of the Buizenschacht and Willem I/II shafts

The shaft head/mouth of the Buizenschacht and Willem I/II shafts (Domaniale) is located nearby a green area used as a playground (see Fig. 12). Furthermore public traffic areas and buildings are located within the Shaft-Protection-Zones.

The shaft head/mouth of the Beerenbosch I shaft (Domaniale) is located in a green area right next to a cart-road (see Fig. 13). A radio mast is situated only a few meters from the shaft mouth. Within the Shaft-Protection-Zone the land use consists of agricultural and wooded land.

Fig. 13: Shaft-Protection-Zone of the Beerenbosch I shaft

Fig. 14: Shaft-Protection-Zone of the Neuland shaft

The shaft head/mouth of the Neuland shaft(Domaniale) is located in a non-public backyard of a residential building (see Fig. 14). Within the Shaft-Protection-Zone the land use consists of public traffic areas and buildings.

Fig. 15: Shaft-Protection-Zone of the Melanie shaft

The shaft head/mouth of the Melanie shaft (Willem Sophia) is located in wooded area (see Fig. 15). Furthermore a federal roadway and agricultural land are located within the Shaft-Protection-Zone.

At the locations marked with a blue Shaft-Protection-Zone (low and medium safety level) subsidence has to be considered during the construction of buildings and infrastructure. Construction of facilities with an increased vulnerability to the effects of subsidence (e.g. railways, sewage pipes) may require a special foundation. It is generally advisable to avoid a high-quality land use in these

Shaft-Protection-Zones. The shafts have to be accessible for inspection. Construction of buildings on top of the shafts should be avoided.

Land use of the green Shaft-Protection-Zones (high and permanent safety level) is not limited regarding aspects of surface stability. However, the construction of buildings on top of the shafts should still be avoided.

3.2.6 Conclusions and Recommendations

For the 39 industrial mine shafts in the South Limburg coalfield the following further actions are suggested:

- not yet treated:
  Investigation of the current situation, monitoring of shaft lining, fencing-off of the area (1 shaft)
- very low safety level:
  Investigation of the current situation and application of additional remediation measures in the short-term (5 shafts)
- low and medium safety levels:
  Monitoring of the backfill columns (30 shafts)
- high safety level:
  No immediate action necessary (2 shafts), periodic monitoring advisable
- permanent safety level:
  No immediate action necessary (1 shaft)

<u>Suggestions for the investigation of the current situation</u>

- **Buizenschacht, Willem I, Willem II and Beerenbosch I (Domaniale)**

  To determine the condition of the shafts below the plug it is suggested to drill through the backfill columns with core drillings. The properties of the plug can be checked with the core material. Subsequently, the open shaft below the plug can be inspected down to the mine water level. This allows to assess the condition of the shaft lining and the position and condition of fixtures. Another focus is the identification of influx points for groundwater. Possible inspection methods include borehole TV or laserscanning.

Depending on the test results further precautionary measures may be planned and executed. Based on currently available information it may be feasible to permanently secure the shafts by installation of a sufficiently long cohesive backfilling between the mine water level and the bottom of the plug.

- **Neuland**

Firstly, it is recommended to regularly monitor the height of the backfill. For a permanent treatment, three options can be considered based on the currently available information. Each of these options needs prior investigations of the subsoil and/or shaft conditions.

Option 1: Stabilisation of the loose material backfill by injection of a cement-based suspension (grouting)

Option 2: Excavation of the loose material down to a to-be-determined level and backfilling with concrete

Option 3: Construction of a closed outer ring of bored piles as a foundation for a cover slab

- **Melanie**

According to the available information, the Melanie shaft has not yet been secured. The current situation should be investigated, e.g. with a borehole TV inspection. Based on the results proper remediation measures can be taken. Based on current knowlegde it appears to be feasible to install a concrete backfilling on top of the existing plug. This would permanently secure the Melanie shaft.

- **Monitoring of the backfill column**

To ensure surface stability at the shafts with a low and medium safety level a regular monitoring of the backfill column is necessary. If subsidence of the column is observed, further backfilling is required. This means the shafts need a functional manhole in the cover slab to allow an observation of the backfill. If an opening is not available or has been sealed, this should be drilled. Changes in the height of the backfill column should be documented. Unusually large subsidences should prompt further investigation and precautionary measures on a case by case basis.

At shafts with a sensitive land use inside of the Shaft-Protection-Zone (buildings on the shaft head, roads going through the Shaft-Protection-Zone, etc.) it is suggested to install an electronic monitoring system for continuous observation of the backfill; this can be used for remote alert triggering.

The stability of the injected loose material backfill in the Catharina shaft can be monitored with the existing extensometer.

4 Results of WG 5.2.3 “Risks from near-surface mining”

As mentioned in chap. 2.3 the general mining situation varied between project area 1 and project area 2 and 3. Hence, one has also to distinguish between a “Historical near-surface mining” and an “Industrial near-surface mining”. In general, “Historical near-surface mining “ was limited to project area 1 whereas “Industrial near-surface mining” took place in project areas 2 and 3.

4.1 Digitisation of the different mining relicts

4.1.1 Near-surface mining areas in project area 1 (“Historical near-surface mining”)

In the area of historical mining (project area 1) the approach to inventory and digitise potential mining relicts had to be different from the approach in the areas of industrial mining (project areas 2 and 3). The historical mining is so old that documents on the mining activities were either not yet drawn or perhaps they got lost or were demolished. Therefore, it is quite obvious that by no means all of the near-surface mining activities of historical mining are documented.

Analogue to the approach in North Rhine-Westphalia/Germany for each coal seam that seems to be worth mining because of its thickness (“Main coal seams” and “mineable coal seams”) a near-surface mining activity is hypothetically presumed and this coal seam will be incorporated in the system of risk assessment.

In this area of historical mining the tectonic situation and especially the inclination of the coal seams as well as the outcrops of the various coal seams at the top of the Carboniferous bedrock usually is not shown in mining maps. Hence the tectonic structure had to be clarified by “geological tools” using the general scientific knowledge about tectonics, stratigraphy, sequence of the coal seams etc. and projecting the coal seams according to their inclination from well-known deeper levels upwards to the top of the Carboniferous bedrock.

The main target of this geologic work was to create a map of the project area 1 in which the intersection of all relevant coal seams with the top of the Carboniferous bedrock is shown. Along these intersection lines each coal seam would be "visible" if the Carboniferous bedrock would be without overburden. Furthermore this map should include the direction of dipping and the inclination of the coal seams as well as the main tectonic elements (axis of synclines and anticlines, faults).

To create this map all available information about the geologic-tectonic situation, about boreholes and about the deeper mining situation was evaluated and interpreted. Also the documented cross sections of the deeper underground were used as one basis for this construction. In regions where the information about the underground conditions was not sufficient enough the approach was supported by creating new cross sections and comparing them to the preceding interpretation. By this iterative way with creating altogether 14 cross sections a satisfactory result was achieved.

The result of this work is a map with the outcrop lines of 13 coal seams ("Main coal seams" and "mineable coal seams") in project area 1. In total the length of the constructed outcrop lines add up to 25,6 km in an area of 1,53 km^2.

Based on this map a first segmentation of the constructed outcrop lines was performed according to the following criteria:

- Alternation in dip of coal seam (≥ 36°/< 36°)
- Recurving of synclines or anticlines

- Tectonic faults cutting the strike of beds
- Special local knowledge from borings
- Special local knowledge about mining activities from documents

These segments of the coal seams were the input data for the risk assessment that is described in chap. 4.2.

4.1.2 Near-surface mining areas in project areas 2 and 3 ("Industrial near-surface mining")

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The editing was done in such a way that all the mining areas were recorded separately in the concessions by mines, coal seams and fields. Each local mining area has been digitised with maximum and minimum values for mining heights and mining periods (Tab. 4). After the digitisation the data was checked to eliminate duplicate registrations from the different mining maps wherever possible.

To identify the areas close to the top of the Carboniferous, the "Upward drillings" (see chap. 4.1.4) and "Downward drillings" (see chap. 4.1.5) were used for further analysis; i.e. information about the bedrock surface level were mainly derived from these drillings. Those mining areas who have a shorter distance than 20 m to the top of the Carboniferous (in accordance with the values of the boreholes) have been identified, cut out and attributed. The coordinate system used is the current Dutch system "RD-New".

Tab. 4: Definition of attributes recorded for mine workings in project areas 2 and 3

Field	Type		Description
concession	text	20	concession related to the mining maps
GB_no	text	10	name of the coal seam
coal_seam	text	20	name of the coal seam (local name)
annotation	text	254	remarks or additional information
min_lvl	numeric	Short	minimum height of mining
max_lvl	numeric	Short	maximum height of mining
start	numeric	Short	beginning of mining
end	numeric	Short	end of mining

4.1.3 Near-surface galleries

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The processing was carried out analogously to that shown in chap. 4.1.2 with the same preparation and base data. Galleries that have a distance less than 20 m to the top of the Carboniferous were attributed accordingly. The coordinate system used is the current Dutch system "RD-New".

Tab. 5: Definition of attributes recorded for near-surface galleries in project areas 2 and 3

Field	Type		Description
concession	text	20	concession related to the mining maps
GB_no	text	10	name of the coal seam
coal_seam	text	20	name of the coal seam (local name)
annotation	text	254	remarks or additional information
min_lvl	numeric	Float	minimum height of mining
max_lvl	numeric	Float	maximum height of mining

4.1.4 “Upward drillings”

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The mining maps were examined for information on examination boreholes from the area of the Carboniferous into the overburden. The data collection was carried out as data points with the attributes according to the following table. In case there was more than one information about the location and the heights the most probable value has been selected. The coordinate system used is the current Dutch system "RD-New".

Tab. 6: Definition of attributes recorded for the “Upward Drillings”

Field	Type		Description
Type	text	20	type of boring according to the mining maps
Number	text	20	name/number of boring according to the mining maps
carbon_lvl	numeric	Float	height at top of Carboniferous
annotation	text	254	remarks or additional sources
source	text	254	source of information

4.1.5 “Downward Drillings”

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The mining maps were examined for references to drillings from the surface down into the Carboniferous. The data was collected as data points with the attributes according to the following table. In case there was more than one information about the location and the heights the most probable value has been selected. In a few cases no former heights from the surface were present; in this case the values were taken out of the provided Shape:
„…\10_TNO_data\06_Limburg_surface_motion\7_historic_maps\TOPhoogteMD\TOPhoogteMD\geogegevens\shapefile\landsdekkend\tophoogte„
This source had values close to those from the times of the original drilling. The coordinate system used is the current Dutch system "RD-New".

Tab. 7: Definition of the attributes recorded for the “Downward Drillings”

Field	Type		Description
Type	text	20	type of boring according to the mining maps
number	text	20	name/number of boring according to the mining maps
ground_lvl	numeric	float	height at surface (source mining maps)
carbon_lvl	numeric	float	height at top of Carboniferous
annotation	text	254	remarks or additional sources
source	text	254	source of information (mining maps)

4.1.6 “Drempels and Scheuren”

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The mining maps were examined for references to "Drempels and Scheuren". The acquisition was aligned to the given attributes and geometry of the mining maps. When digitising the line items the digitised direction was additionally indicated to ease the representation in a GIS. If available, the date of the event has been added to the shape.

The coordinate system used is the current Dutch system "RD-New".

For the Willem Sophia concession there was no information available.

Tab. 8: Definition of attributes recorded for “Drempels and Scheuren”

Field	Type		Description
source	text	254	source of information (mining maps)
measurem	text	20	type of measurement of "drempels"
dip_direct	text	10	dip direction according to the direction of digitising
vert_throw	numeric	Float	vertical throw according to the mining maps in meters
Depth	numeric	Float	no information in mining maps available - for later use
area_surf	numeric	Float	calculated length of "drempels and scheuren" in meters
month	numeric	Short	month of occurrence of the event
Year	numeric	Short	year of occurrence of the event
annotation	text	254	remarks

4.1.7 “Verzakkingen”

Data basis for the analysis were the provided mining maps (see chap. 2). For the collection of the data the programme ArcGIS® was used. The mining maps were examined for references to "Verzakkingen". The acquisition was aligned to the given attributes and geometry of the mining maps. Digitisation was carried out as area information. If available, the date of the event has been added to the shape.

The coordinate system used is the current Dutch system "RD-New".

For the Willem Sophia concession there was no information available.

Tab. 9: Definition of attributes recorded for "Verzakkingen"

Field	Type		Description
source	text	254	source of information (mining maps)
measurem	text	20	type of measurement of "verzakking"
depth	numeric	float	depth of "verzakking" in meters
area_surf	numeric	float	calculated area of "verzakking" in meters
month	numeric	short	month of occurrence of the event
year	numeric	short	year of occurrence of the event
annotation	text	254	remarks

4.2 Risk assessment for the different mining relicts

The assessment of risks arising from mining relicts other than shafts is also performed using the Bow-Tie-method (see chap. 3.1.3.1). The assessment focusses on near-surface mining and mining close to the top level of the Carboniferous bedrock. Further, hazard-related investigations are performed for „Upward and Downward drillings". For dealing with former mining related damage pattern („Drempels and Scheuren" and „Verzakkingen") some recommendations are made.

4.2.1 Near-surface mining areas in project area 1 ("Historical near-surface mining")

Prior to the Industrial Revolution in the midth of the 19th century, mining focused on near-surface deposits. Coal was exploited using the pillar and chamber method; often, the pillars have also been mined afterwards. Hence, larger voids have to be expected on an areawide basis in the level of the coal seams. In the course of time, the former stopes have commonly fallen-in; however, residual voids have to be expected locally even today.

With regard to possible impacts to the ground surface arising from these near-surface stopes, subsidence or the formation of sinkholes have to be expected for an unlimited period. Often, the layers lack of sufficient thickness to establish a stable vault over a larger coverage.

The probability of incidents related to these mining relicts strongly depends on both the tectonical conditions and the mining conditions. Following the approach that was chosen in the adjacent historical mining area of Herzogenrath/Germany, different “impact categories” are defined for the outcrops of coal seams at the top of the Carboniferous bedrock. For relative and absolute probabilities compare the discussion in chap. 3.1.3.2.

To estimate the area that might be influenced by a possible incident, so called “potential impact areas” are defined for all outcropping coal seams; the corresponding impact categories are assigned to the impact areas.

Based on the specified impact areas, a Bow-Tie-diagram is developed to assess the hazards and risks related to near-surface mining in the historical mining area of Kerkrade.

4.2.1.1 Categories

Subsequent to the construction described in chap. 4.1.1, the segmented outcrop lines of coal seams were assigned to four different impact categories that are based on the German model. The impact categories are defined in Tab. 10.

Tab. 10: Overview of the impact categories for outcrops of coal seams in project area 1

Impact category	Classification Criteria	Estimated relative probability for future sinkholes and/or subsidence	Colour Code
EK 1	If dip ≥ 36°: - Documentation of sinkholes in the past - Evidence of near-surface mining in documents - Indication of mining activities above the uppermost gallery	High	Red
EK 2	If dip ≥ 36°: - Documentation of mining activity in “Mineable Coal Seams” on the level of the uppermost gallery - outcrop of “Main Coal Seam” at top of Carboniferous bedrock If dip < 36°: - “Main Coal Seams” or “Mineable Coal Seams” show evidence of near-surface mining in documents - “Main Coal Seams” or “Mineable Coal Seams” show indication of mining activities above the uppermost gallery	Medium	Yellow
EK 3	If dip ≥ 36°: - Outcrop of “Mineable Coal Seams” without documentation of near-surface mining but with likeliness of mining because of the general tectonic situation If dip < 36°: - Outcrop of “Main Coal Seams” at the top of the Carboniferous bedrock, even by uncertain documentation	Low	Blue
EK 4	Remediation measures have been done	None	Green
none	Coal seam can not be matched to the impact categories.	None	None

As can be seen from Tab. 10 the classification of impact categories is based on the differentiation of “Main Coal Seams” and “Mineable Coal Seams”. The attribution of coal seams is also based on the German model. A further important differentiator in the classification of impact categories is the dip of coal seams; in general steep dipping coal seams are considered to be more hazardous in recent times.

With regard to the assignment of impact categories, documented stopes such as near-surface mining above the uppermost gallery („Stollensohle“), goafs (“Alter Mann”) or stopes reached by drilling are of particular importance. In general, these segments were assigned to EK 1 or EK 2. Furthermore, steep dipping main coal seams of the historical mining area in Herzogenrath/Germany are always assigned to EK 2; flat dipping main coal seams are always assigned to EK 3. In general, steep dipping mineable coal seams are assigned to EK 3.

4.2.1.2 Bow-Tie-Analysis

The estimation of areas at ground surface level that might be affected by the impacts of near-surface mining (impact areas) provides the basis for the further risk assessment. It follows the same approach that was chosen in the adjacent historical mining region of Herzogenrath/Germany. This approach is based on the assumption that all mining-related incidents in impact areas are causally provoked by a failure of the underlying bedrock. The impact area is defined perpendicular to the outcrop line of a coal seam to both the tectonic hanging wall and the laying wall; it comprises four components:

- Outcrop width of the coal seam;
- Impact area at the top of the Carboniferous bedrock;
- Width resulting from impact of overburden;
- Accuracy of the system.

Outcrop width of the coal seam

The outcrop width of the coal seam is a function of the real thickness of the coal seam and its angle of dip. The average thickness of coal seams was taken from stratigrafic lists; often, the angle of dip is indicated in mining maps. Sometimes, the angle of dip had to be determined graphically-constructive, i.e. with the aid of cross-sections.

Impact area at the top of the Carboniferous bedrock

Failure of the solid rock roof is confined to a certain area, the so called impact area. The impact area at the top of the Carboniferous is defined according to the nomogram of HOLLMANN & NÜRENBERG (1972) (Fig. 16). Here, the width of the impact area at the top of the Carboniferous bedrock is a function of the dip of a coal seam in which the width generally decreases when the dip angle increases.

As can be seen from Fig. 16, four consequences can be distinguished. A danger for the formation of a sinkhole due to structural breakdown and structural disintegration is possible, in dependence of the dip of the coal seam, in the direct vicinity of a coal seam. According to the nomogram, at a greater distance to rather flat dipping seams, structural loosening and disintegration might occur. The potential for the occurrence of sinkholes is restricted to the fields in red and/or yellow colour (Fig. 16).

In the range between 0 and 62° only the tectonic hanging wall contributes to the impact area at the top of the Carboniferous bedrock. Starting from approximately 63° the tectonic laying wall also contributes to the impact area at the top of the Carboniferous bedrock.

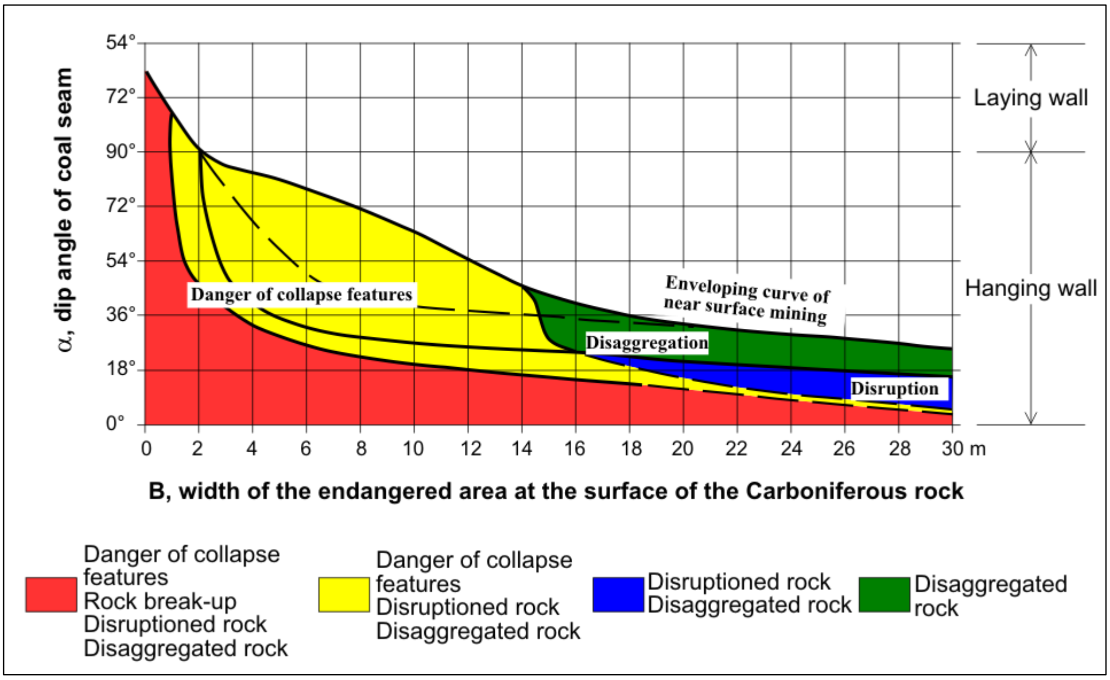

Fig. 16: Nomogram for the definition of potential impact areas at the top of the Carboniferous bedrock (adapted after HOLLMANN & NÜRENBERG, 1972)

Width resulting from impact of overburden

By analogy with the Shaft-Protection-Zones, the potentially affected area at ground surface level is delimited by the thickness of the overburden. Here, too, an angle of 45° is taken as angle of repose (see chap. 3.1.2). For the definition of impact areas resulting from the thickness of the overburden, the thickness is included in one meter steps.

Accuracy of the system

In this case, the accuracy of the system is related to the outcrop lines of the coal seams. As described in chap. 4.1.1, the uncovered geological plan of project area 1 was constructed based on more or less precise (historical) mining maps and cross-sections. To account for this, a system accuracy of 20 m was assigned to coal seams dipping ≤ 36°; 15 m were assigned to coal seams dipping > 36°.

A general plan of the potential impact areas at the ground surface level in project area 1 is given by Plan 2. In this plan, the impact categories (Tab. 10) have been assigned to the impact areas that were defined according to the approach described above.

As can be seen from Plan 2, the impact categories EK 2 and EK 3 are predominant in project area 1. One minor area assigned to impact category EK 1 can be found in the northwestern part of the project area; this originates from a coal seam in Germany. Only some smaller parts of the project area 1 are not covered by impact areas at all. In a greater part of the project area 1, the impact areas of two or more coal seams are overlapping.

As already mentioned, the major problem with near-surface mining in historical mining areas is the possible presence of stopes near the top of the Carboniferous bedrock, especially if they have not collapsed yet. Present-day collapse of these voids might migrate through the overburden and cause impacts on the ground surface down to the present day.

The area that potentially might be affected by these impacts is defined by the impact areas; the corresponding relative probability for the occurrence of an incident is given by the impact categories. Analogous to the Bow-Tie-Analysis of mine shafts in the historical mining area (chap. 3.1.3.3), this hazard is referred to as geotechnical hazard.

The geotechnical hazard arising from “Historical near-surface mining”

In general, the types of ground movement that are likely to occur in the impact areas of coal seams are the same as those that are likely to occur in Shaft-Protection-Zones, i.e. collapse/formation of a sinkhole and subsidence. However, in comparison to the formation of sinkholes, in these impact areas the occurrence of subsidence is more likely than sinkholes. Here, too, for the reasons discussed in chap. 3.1.3.3, both types of ground movement are defined to be the same Top Event.

In the following, a Bow-Tie-Analysis is developed for this geotechnical hazard; for the corresponding Bow-Tie-diagram see Appendix 3.

It should be noted that the Controls in Appendix 3 are arranged sequentially for reasons of clarity and comprehensibility. In reality, commonly one measure or a specific combination of different measures is applied. The most suitable measure or combination of measures has to be determined on a case-by-case basis.

<u>Threats for geotechnical hazard arising from "Historical near-surface mining"</u>

In general, three superordinated mechanisms are regarded to be able to cause a Top Event in an impact area of coal seams; these three mechanisms are defined to be the Threats in the Bow-Tie-Analysis. Here, too, (mine) water has an important role in these mechanisms.

Generally, a direct danger for the formation of a sinkhole is most likely given in connection with a failed rock roof due to structural breakdown or structural disintegration. However, the displacement of material might also lead to the formation of a sinkhole if certain geologic conditions are present. For the Threats corresponding to displacement of material, subsidence is regarded to be the most likely Top Event.

- **Failure of the rock roof:** The failure of the rock roof is considered to be the root cause for most of the (severer) Top Events. As can be seen from Fig. 16, failure is generally preceded by two processes: structural breakdown and/or structural disintegration. Two general failure mechanisms can be differentiated (see MAINZ, 2008). Failure of the crown pillar commonly occurs due to an insufficient thickness of the residual rock mass. In this case, the fall-in of the adjacent bedrock is very likely. The second failure mechanism is the caving-in of material from the hanging wall into the residual stopes. This is common in the area of disintegration. While, in this case, the crown pillar stays intact, the caving-in of material migrates upwards.

- **Displacement of material by erosion:** Fine, non-competent material from the overburden might be washed out by flowing seepage water or groundwater (suffosion). In case the underlying strata is disintegrated by former mining activities, displacement of material to the deeper underground may occur. In this context, upward drillings might also play a certain role (see chap. 4.2.4).

  In the historical mining area of Kerkrade, the Tongeren formation is overlying the Carboniferous bedrock on an areawide basis. This fine-grained sand is flowable, i.e. the material can be displaced downwards by water.

  The underlying rock roof is often loosened due to the impacts of mining, i.e. it includes cracks or fissures that are preferential pathways for flowing water. Commonly, residual voids of near-surface mining serve as reservoirs for the washed-out material. The volume deficite in the overburden is compensated by collapsing material which, in turn, can cause subsidence or, depending on the actual geologic conditions, can cause the formation of a sinkhole.

- **Displacement and weakening of material by mine water rise:** The influence of mine water can also cause displacement of material. In this case, mine water is considered to liquefy the backfill material and cause the erosion of material. Hence, mine water is considered to give rise to new, former backfilled voids. The loss of abutment, in turn, might weaken the overlying strata and thus, may cause failure of the rock roof.

  Furthermore, rising mine water is considered to alter the stress regime in both the Carboniferous bedrock and in the overburden.

  In general, the mine water has not yet reached the level of the historical near-surface mining area.

Consequences from the geotechnical hazard arising from "Historical near-surface mining"

The Consequences from the geotechnical hazard arising from "Historical near-surface mining" are assumed to be identical to the Consequences from historical mine shafts described in chap. 3.1.3.3, that are:

- **Injury/loss of life**
- **Damage of buildings**
- **Damage of infrastructure**
- **Social unrest**

As can be seen from Plan 2, the area that is potentially affected by the Consequences is considerably larger than the area that is potentially affected by the Consequences corresponding to historical mine shafts (see Plan 1). **However, experiences acquired in the historical mining area of Herzogen-rath/Germany have shown that the Consequences from the geotechnical hazard arising from near-surface mining are both less probable and less severe compared to the Consequences corresponding to historical mine shafts.**

Prevention Controls for the geotechnical hazard arising from "Historical near-surface mining"

Prevention Controls for the geotechnical hazard arising from "Historical near-surface mining" follow the same approach that has been discussed in chap. 3.1.3.3.

For the Top Event under discussion, only one Prevention Control is considered to be theoretically feasible and practical:

- **Stabilisation of underground mine voids and rock roof:** As discussed above, residual underground mine voids near the top of the Carboniferous bedrock are the underlying problem of near-surface mining in historical mining areas. The elimination of the hazard aims at the filling of these voids and/or the stabilisation of the rock roof. Usually, the filling of voids is performed by the utilisation of techniques known from foundation engineering such as grout injection.

  By default, voids in the subsurface are opened up by drillings that are sunken starting from ground surface level, i.e. the position of an underground mine void has to be sufficiently explored prior to the measure. These drill holes are used to grout a concrete slurry into the void, subsequently. When a certain grouting pressure is reached and the concrete slurry has hardened the former voids and the rock roof are considered to be sufficiently stabilised.

  By backfilling the underground mine voids, future failure of the rock roof can be precluded. As an additional consequence of this measure, a further displacement of material due to the influence of water is prevented.

<u>Recovery Controls and Escalation Controls for the geotechnical hazard arising from "Historical near-surface mining"</u>

Fundamentally, for the geotechnical hazards arising from "Historical near-surface mining", the same Recovery Controls can be applied that already have been discussed for the geotechnical hazard arising from historical mine shafts (see chap. 3.1.3.3). These are:

- **Regional development planning**
- **Awareness-raising**
- **Adapted site investigations**
- **Adapted construction**
- **Immediate Measures**

- **Constructional support work**

In addition, two further measures are regarded to be useful and are assigned to the Recovery Controls:

- **Pilot research Heerlen:** In the context of a Pilot project in Heerlen, the underground conditions are to be investigated by means of vertical drillings for a detailed examination of a potential hazardous zone in a highly frequented area. Although the examination is no Recovery Control in the proper sense, further insights that might be acquired from the research might improve the other Recovery Controls.

- **Development early warning system ground motion:** Early detection of looming Top Events is a key to conquer the hazards of near-surface mining. Since the hazard spreads across a larger area spatial monitoring of the ground surface (e.g. using InSAR) is regarded to be useful.

Due to the extension of the possible impacts on an areawide basis, active prevention measures (i.e. **regional development planning** and **awareness-raising**) are considered to be of particular significance.

With regard to the **adapted site investigation** for the hazard of “Historical near-surface mining”, there is an important difference to the measures described for shafts: the investigation programme for construction projects in impact areas of near-surface mining should be based on the corresponding impact category.

For the historical mining area of Herzogenrath/Germany, the following approach was defined:

- **EK 1/EK 2**: Prior to the realisation of construction projects (i.e. new buildings as well as certain construction projects subjected to approval such as substantial extension and/or reconstruction of existing buildings), a detailed investigation of the actual mining-geotechnical conditions in the underground has to be performed on behalf of the owner/builder/investor. Normally, the investigation programme contains 2-3 core drillings. If the investigations reveal unfavourable conditions, a stabilisation of the underground mine voids has to be perfomed before the realisation of the project.

- **EK 3**: Construction projects in EK 3 usually only require an inspection of the excavation pit with regard to indications of mining impacts on behalf of the owner/builder/investor. If necessary, the rating of the area has to be adjusted. The remaining risk has to be accepted by the owner/builder/investor.
- **EK 4**: No measures are required.

4.2.1.3 Conclusions and Recommendations

The outcome of the hazard mapping in project area 1 can be summarised as follows:

- The densely populated historical mining area of Kerkrade is extensively affected by possible impacts related to near-surface mining.
- A larger part of the delimited impact areas is attributable to the component “accuracy of the system”.
- Impact categories EK 2 and EK 3 are predominant; i.e. the relative probability for actual incidents is considered to be medium to low in a larger part of the historical mining area.
- Only one small region in the fringe area of project area 1 is characterised by a high (relative) probability for an actual incident (EK 1).

The stabilisation of underground mine voids using techniques of foundation engineering can be an effective measure for the elimination of the hazard. However, this measure requires a more or less detailed knowledge of the position and distribution of the underground mine voids. Usually, the voids have to be reached by drillings to enable a stabilisation, subsequently.

At this point, the benefit-cost ratio has to be taken into consideration. Measures for the minimisation of a risk are only reasonable if the benefit outweighs the costs (see ALARP-principle). Benefit-cost calculations performed for the historical mining area of Herzogenrath/Germany revealed that the costs are by far out of proportion to the risk. This is mainly to the fact that the absolute probability of occurrence is considered to be low (see chap. 3.1.3.1).

To handle the risks of near-surface mining effectively, the principle of urban development should be not to increase the risk. In essence, stabilising measures or constructional support work is only to be performed if:

- The risk is substantially increased due to construction projects, construction projects subject to approval or change of use
- Actual mining related damage emerges

Based on a combination of Prevention Controls and Recovery/Escalation Controls, in the following, a strategy is developed to counteract the geotechnical hazard arising from near-surface mining in the historical mining area of Kerkrade. A similar strategy has already archived good results in the comparable historical mining area of Herzogenrath/Germany.

- **Full integration of impact areas into regional development planning:** Future regional development should, in particular, consider the outcomes of this study, i.e. include the delimited impact areas of near-surface mining as well as information about “historical” drillings (see chap. 4.2.4 and chap. 4.2.5) as well as historical damage events (see chap. 4.2.6 and 4.2.7).
- **Awareness-raising:** The residents of the historical mining area of Kerkrade should be aware of the hazard to be able to act properly if any damage

emerges. For further information, a central information service should be established.

- **Statutorily regulated procedures for the development of new areas as well as for construction projects subject to approval:** Prior to construction projects the “non-existence of possible mining related hazards” has to be proven for the respective area by the owner/builder/investor. In impact categories EK 1 and EK 2 the actual mining-geotechnical conditions have to be verified by suitable methods (e.g. drillings). If necessary, underground mine voids have to be stabilised. Construction projects in EK 3 usually only require an inspection of the excavation pit with regard to indications of mining impacts. Adapted construction and constructional support work can also be taken into consideration for development or construction projects. All measures have to be supervised by experienced experts. For more vulnerable structures (e.g. public facilities such as schools or hospitals, plants etc.) an expert opinion should be obtained. If needed, further investigations and, where required, stabilising measures should be performed.
- **React to damage events:** In an event of damage, immediate measures shall be provided for mitigation. A root cause analysis that considers the outcomes of this study is to be performed by an experienced expert. If needed, the continuance of a hazard should be stopped by stabilising the underground mine voids. If stabilising is not possible, proper constructional support work has to be realised.

According to current knowledge, the component “accuracy of the system” constitutes larger parts of the estimated impact areas in project area 1. **Herein, targeted core drillings could be considered for a more precise delimitation of the impact areas.**

Furthermore it is strongly recommended to regularly adjust the hazard map to new results that have been achieved by core drillings. All new data should be sampled and incorporated in the Geoinformation system (GIS) for example every 3 years.

4.2.2 Near-surface mining areas in project areas 2 and 3 ("Industrial near-surface mining")

The project areas 2 and 3 are characterised by industrial deep mining. Here, the coal seams are mainly situated below a thicker overburden (see chap. 2.3). Due to mining regulations, mining activity in these project areas is better documented than in project area 1. In contrast to mine workings in project area 1, stopes close to the top of the Carboniferous bedrock were generally excavated under preservation of a thicker crown pillar.

However, since 1939, mining regulations allowed the mining companies to reduce the crown pillar heights from 50 m to 10 m or even 3 m (DE MAN, 1988). According to DE MAN (1988), for extraction under a reduced crown pillar, certain requirements had to be met, where safety of mineworkers had the highest priority; among others, only retreating longwall mining must be used.

The crown pillar reduction to a height of 3 m was permitted only if the overlying strata was investigated by means of upward drillings. By these upward drillings the presence of strongly water-bearing layers or the presence of quicksands ought to be verified. In case such strata was encountered, it was common practice to dewater the layers by means of upward drillings to enable safe conditions for subsequent exploitation. Locally, extraction even extended up to the overburden so that there was no crown pillar left (DE MAN, 1988).

The excavated areas were commonly not backfilled; collapse of the solid rock roof was supposed to backfill the voids to prevent influx of water-bearing sands at high velocities (DE MAN, 1988). The author points out that many of these underground mine voids shallow below the top level of the Carboniferous bedrock are assumed not to have collapsed so far. Especially if there is a rather thin overburden, the long-time persistence of underground mine voids is considered to be very likely.

In fact, the sinkhole at “Winkelcentrum ‘t Loon” in Heerlen that occurred in autumn of 2011 revealed that stopes under a reduced crown pillar height (approximately 8 m), albeit covered under a relatively thick overburden (approximately 90 m) can cause strong damage, even nowadays. However, to this day, the incident at “Winkelcentrum ‘t Loon” is the only damage event in the whole Aachen and South Limburg mining district that is clearly attributable to deeper mining. For a more detailed review of the damage event see KLÜNKER et al. (2013).

Based on the investigations of the sinkhole in Heerlen and on their findings, respectively, as well as being modelled on the impact areas and impact categories that were applied in the historical mining area of Kerkrade, a modified approach for the risk assessment of mine workings close to the top level of the Carboniferous bedrock was developed. Here, too, the major hazard is mainly given by not fallen-in stopes.

4.2.2.1 Categories

For the hazard mapping in project areas 2 and 3 an impact-relevant limit depth of 20 m, measured against the top-level of the Carboniferous bedrock, was defined. This means all stopes that are located in the range between 0 and 20 m below the top of the Carboniferous bedrock are assumed to be able to cause certain hazards to the ground surface.

This defined range considers both the rockmechanical properties and a certain data-related lack of clarity (i.e. accuracy and readability of historical mine maps as well as the accuracy of geological constructions that were derived therefrom).

According to the depth-related nomogram of HOLLMANN & NÜRENBERG (1972) the defined range corresponds to a dip-angle between 0 and 63° and therefore covers the spectrum of the tectonical setting in this area. The deeper stopes are considered not to cause damage at ground surface. The digital mapping of the stopes is described in chap. 4.1.2.

The definition of impact categories is based on the approach in Germany/NRW but especially takes into account the specific geologic-tectonical settings in South Limburg. Furthermore the investigations of the incident at “Winkelcentrum ‘t Loon” in Heerlen are taken into account. Modeled after the impact categories used in project area 1, three categories are distinguished. An outline of the chosen approach for the impact categories EK 1 and EK 2 is given by Fig. 17.

- **EK 1:** As discussed by KLÜNKER et al. (2013) the sinkhole at “Winkelcentrum ‘t Loon” occurred above a stope that is characterised by a tri-angle-shaped/acute-angled geometry. As known from civil engineering, a special type of stress distribution is prevailing under these conditions that enables a persistence of open voids (see “arching-effect”). Thus, the existence of not fallen-in voids down to the present day is assumed to be more likely if such acute-angled geometries are present. Implemented into the assessment of the geotechnical hazard, this fact is taken into account by assigning impact category EK 1 to these areas. For the determination of further stopes that are characterised by similar conditions, an angle up to 60° was taken as a basis.

The impact area EK 1 was delimited to the inner stope by 50 m measured from the peak of the triangle-shaped area in the dip-direction.

Furthermore from the evaluation of the mining maps it was well known that in

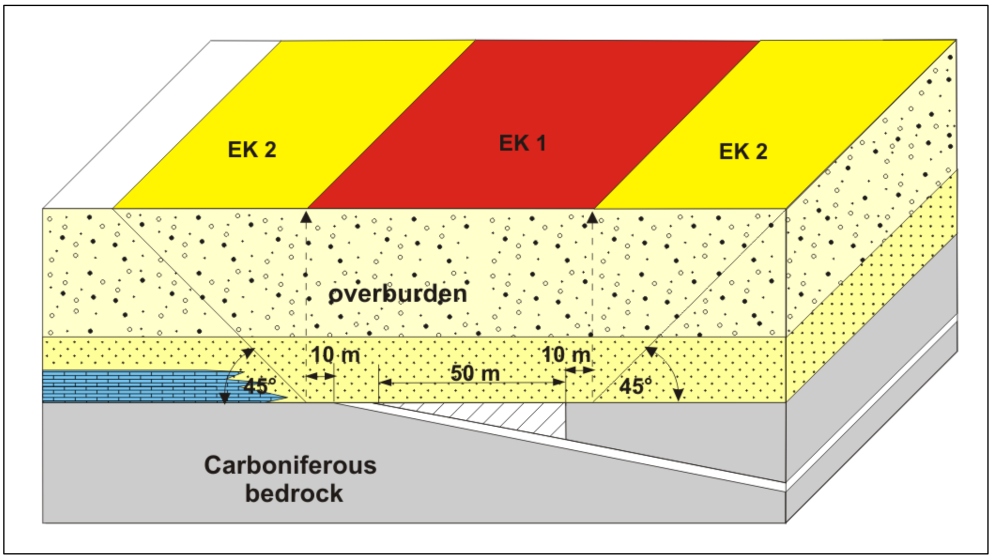

Fig. 17: Outline of the definition of impact categories EK 1 and EK 2 in project areas 2 and 3

the town of Kerkrade, especially in the area near the “Westelijke Sprong” some Room & Pillar Mining took place. Thus, also in these areas the existence of not fallen-in voids down to the present day is assumed to be more likely. Therefore all mining maps were evaluated with respect to Room & Pillar Mining between 0 and 20 m below the top of the Carboniferous bedrock. These areas were assigned to the impact category EK 1 also.

- **EK 2:** In areas of impact category EK 1, in case of displacement of material in the level of the top of the Carboniferous bedrock the voids might migrate through the overburden and cause impacts on the ground surface; the possibly affected area at ground surface in the surroundings of impact area EK 1 is defined to be impact category EK 2. It is delimited by the thickness of the overburden. An angle of 45° is taken as angle of repose (Fig. 17).

The thickness of overburden was derived from the Digital Terrain Model AHN2 and upward drillings that were documented in the mining maps (see chap. 4.1.4) instead of using the REGIS 2.1 or REGIS 2.2. This approach yielded better results due to the high data density (large number of drillings) in the pertinent regions. The thickness of overburden was included into the delineation of impact areas in 5 m-steps.

- **EK 3:** As mentioned above, stopes located in the range between 0 and 20 m below the top level of the Carboniferous bedrock are assumed to have also potential to be impact-relevant to the ground surface. Hence, implemented into the risk assessment, these stopes are assigned to the impact category EK 3. In defining the corresponding impact areas, the actual stopes were extended by 10 m to each side to incorporate a certain position accuracy and the thickness of the overburden is incorporated also, taking an angle of 45°.

A general map of the impact areas in project areas 2 and 3 is given by Plan 3. As can be seen from this plan a clustering of impact areas can be found in the southeastern part of the South Limburg mining district (the Domaniale, Willem Sophia, Wilhelmina, Oranje Nassau, Laura, and Julia concessions). In contrast, only some scattered impact areas can be found in the Maurits, Emma, and Hendrik concessions.

Furthermore, it can be clearly seen that impact area EK 1 is an exception; 24 “stope-fragments” and 2 “Room & Pillar-areas” of impact category EK 1 were identified in the South Limburg mining district. The impact areas EK 2 and EK 3, on the other hand, stand out clearly.

4.2.2.2 Bow-Tie-Analysis

In general, the Bow-Tie-Analysis that was developed for the geotechnical hazard of "Historical near-surface mining" can be transferred one-on-one to the geotechnical hazard that arises from "Industrial near-surface mining" in project areas 2 and 3. However, there is one important difference between "Historical near-surface mining" and "Industrial near-surface mining" in respect of the definition of hazard.

The sinkhole at "Winkelcentrum 't Loon" in Heerlen was the first documented damage event in the whole Aachen and South Limburg mining district that was clearly attributable to abandoned deeper mining. As root cause for the incident the concurrence of a failed solid rock roof and suffosion/influence of rising mine water is discussed (see KLÜNKER et al., 2013).

In the risk assessment, this single incident defines the parameters for the highest probability of occurrence. However, some similar underground mine voids close to the top of the Carboniferous bedrock have not been flooded yet. The ground stability above these underground mine voids depends on several parameters:

- Conditions and thickness of the solid rock roof.
- Thickness of the overburden: major influence on the persistence of underground mine voids; generally, the actual existence of voids is more likely if the overburden is thin.
- Composition of the overburden: are there flowable layers within the strata or do they even have a direct connection to the underlying bedrock?
- Hydrogeological conditions in the overburden: are the layers overlying the bedrock saturated or can they be saturated by rising mine water?

From a present-day perspective, the ground stability above the mine workings close to the top of the Carboniferous bedrock can not be predicted with certainty. As one could derive from the present knowledge, mine workings covered by a very thick overburden and that have already been flooded seem to have no impact to the ground surface.

On the other hand, some mine workings in the southeastern part of the South Limburg mining district have not been flooded yet. In addition, they are commonly covered by a thin overburden only. Most of these mine workings are overlain by the flowable Tongeren formation.

In comparison to the absolute probability of occurrence in the area of “Historical near-surface mining” the absolute probability of occurrence is considered to be significantly lower in the area of “Industrial near-surface mining”.

4.2.2.3 Conclusions and Recommendations

The risk assessment in project areas 2 and 3 was performed with particular respect to the incident at „Winkelcentrum ‘t Loon“ in Heerlen. For the geotechnical hazard arising from mining close to the top of the Carboniferous bedrock the impact areas were delimited following the approach that was chosen for the risk assessment in project area 1 as far as this was reasonable. The outcomes of this delimitation can be summarised as follows:

- A clustering of impact areas can be found in the southeastern part of the South Limburg mining district.
- In the northern and northwestern parts, only some scattered impact areas are present.
- 26 smaller areas are assigned to EK 1.

The risk management is based on the same Bow-Tie-diagram that was developed for the geotechnical hazard in project area 1 as the underlying scenarios are generally the same in all project areas.

- **Full integration of impact areas into regional development planning** (see chap. 4.2.1.3)
- **Awareness-raising**: Information about the general hazard and potential damage pattern
- **Development regulations in EK 1 and EK 2:** For development projects in EK 1 and EK 2 a detailed investigation of mining and geotechnical conditions by means of drillings is recommended. If necessary, stabilising measures should be carried out on behalf of the owner/builder/investor.
- **Development regulations in EK 3:** In general, there are no restrictions concerning the development potential in these impact areas. However, a more detailed testing of the subsoil stability prior to construction projects is recommended. If necessary, constructional support work should be carried out preventively. For more vulnerable structures (e.g. public facilities such as schools or hospitals, plants etc.) an expert opinion should be obtained. If needed, further investigations and, where required, stabilising measures should be performed.
- **React to damage events** (see chap. 4.2.1.3)

4.2.3 Near-surface galleries

Basically, near-surface galleries can be seen as underground mine voids. In contrast to stopes their spatial extension is line-like. Hence, the potential impact area of near-surface galleries is, in general, smaller compared to those resulting from stopes.

There are no impact-relevant surface galleries in project area 1. However, if necessary, the potential dewatering function of galleries has to be maintained. This fact has to be particularly considered when it comes to grout injection in the context of stabilising measures.

In project areas 2 and 3 all galleries in the range between 0 and 20 m below the top level of the Carboniferous bedrock were captured (see Plan 3). However, in terms of risk assessment, no differentiation is made between galleries and stopes located close to the top of the Carboniferous bedrock in project areas 2 and 3. Hence, at this point reference is made to chap. 4.2.2.

4.2.4 “Upward Drillings”

In the report in hand all drillings that started below the top level of the Carboniferous bedrock and, in addition, deliver level indications of the top of the bedrock are referred to as “Upward Drillings”. Naturally, upward drillings are links between the overburden and underground mine voids as they usually were carried out starting in galleries or stopes. The digitisation of these drillings is described in chap. 4.1.4.

The annexed Plan 4 shows the distribution of the digitised upward drillings in the investigated area; the total number of upward drillings amounts to about 7.250. For all points, an accuracy of position of 5 m is assumed and designed in the GIS.

According to DE MAN (1988) upward drillings were done not only to investigate the overlying strata (i.e. to verify whether there are water-bearing layers or quicksands above the bedrock), but also to dewater water-bearing layers.

Subsequent to completion, upward drillings were usually sealed using simple techniques such as wooden plugs (see DE MAN, 1988).

Experience has shown that upward drillings, under given conditions, can facilitate major ingress of water into the adjacent stopes. This is mainly due to the fact that upward drillings were commonly carried out following a narrow drilling grid.

DE MAN (1988) points out a possible hazard that might arise from these drillings. The author outlines a scenario in which displacement of overburden material occurs due to a renewed saturation of the former dewatered layers. In this scenario, provided that the (wooden) plugs fail, the upward drillings are of particular importance as they constitute preferential pathways for flowable material between the overburden and underground mine voids. The displacement of material, in turn, might cause subsidence at the ground surface (Top Event); the formation of a sudden sinkhole however is quite unlikely. Therefore subsidence is considered to be the Top Event for the Hazard “Upward Drillings”. As there is one Threat only (failure of the plug) and there are no feasible Prevention Controls, the Bow-Tie-Analysis is waived. For possible Recovery Controls see chap. 3.1.3.3.

The Technische Commissie Bodembeweging (TCBB) of the Netherlands has investigated one announcement of a damage at a building and evaluated this case as “mining induced” because of the existence of such upward drillings in the vicinity of this building.

Recommendations:

For the handling of the mining relicts "Upward Drillings" the following recommendations are made:

- The knowledge about the "Upward Drillings" should be given to the competent and responsible authorities at the municipal, provincial, and state levels.
- If damage events emerge or if damage is reported, especially that related to subsidence, the local situation with regard to these "Upward Drillings" should be checked.
- Based on the ALARP-principle no preventive remediation measures seem to be feasible at the moment.

4.2.5 "Downward drillings"

In terms of risk management, downward drillings can be seen as small-scale shafts as they potentially constitute a link between the ground surface level and the Carboniferous bedrock. However, in contrast to mine shafts, downward drillings are usually characterised by smaller drilling diameters. In addition, most downward drillings are not connected to underground mine voids. Minor subsidence at the ground surface level might potentially arise if there is compaction within the backfilled drilling column.

Hence, in general, backfilled drilling columns are not considered to be a serious source of hazard to the ground surface. For already-existing buildings there is no future impact to expect. But for new buildings, if the foundation, or particularly the piles, are unfortunately placed on or inside such a downward drilling, this might lead to significant problems for constructions although a risk for persons has not to be expected.

The annexed Plan 4 shows the distribution of the digitised downward drillings in the investigated area; the total number of downward drillings sums up to 274. For all points in the GIS-version an accuracy of position of 20 m is assumed and designed (in Plan 4 the dots are disproportional). These marked areas are considered to indicate "geotechnical zones of weakness".

Recommendations:

For the handling of the mining relicts "Downward Drillings" the following recommendations are made:

- The knowledge about "Downward Drillings" should be given to the competent and responsible authorities at the municipal, provincial, and state levels.
- If damage events emerge or if damage is reported, especially that related to subsidence, the local situation with regard to these "Downward Drillings" should be checked.
- The authorities should arrange a visual inspection of each excavation pit by a geotechnical expert and/or mining expert if such a "Downward Drilling" is documented in the affected property.

4.2.6 "Drempels and Scheuren"

"Drempels and Scheuren" (roughly translated as "discontinuities at the ground surface, cracks or fissures") are damage patterns that have been observed and recorded at the time of active mining. The digital mapping of these features is described in chap. 4.1.6.

These damage patterns usually develop at the outer edges of a subsidence trough that evolves parallel to mining activity in coal seams. In most cases of hard coal mining activities, an angle of approximately 60° between the outer border of the mined coal seam and the ground surface is used to delimit the outer borders of the subsidence trough.

As one result of the investigations about the development of the sinkhole at “Winkelcentrum ‘t Loon” in Heerlen it was noticed that shortly after the mining activity some “Drempels” occurred at the northeastern face of the mine workings. As these “Drempels” are indicators of a loosened/weakened overburden it was supposed that they enabled or reinforced some transport of soil material from upper horizons downward to the mine openings by means of seepage water originating from precipitation. This process was referred to as “suffosion”.

In terms of risk assessment one has to point out that “Drempels” as such do not constitute a hazard. The main cause for the sinkhole at “Winkelcentrum ‘t Loon” in Heerlen was the (late) collapse of a mine void near to the top of the Carboniferous bedrock. Although the “Drempels” might have enabled or reinforced a process of “suffosion”, the sinkhole occurred nearly vertical above the mining void and not in the area of the “Drempels”.

Therefore, the potential impact areas that are shown in Plan 3 and Plan 2 include the possible cumulative influence of the associated “Drempels”. Even if subsidence might take place not directly vertical above the mining voids, an angle of repose of 45° was chosen to delimit the potential impact area. This angle is sufficiently wider than 60° (see above).

One can summarise that former damage patterns like “Drempels and Scheuren” as such do not constitute a hazard for subsidence or sinkhole. However, the location and the distribution of these former damage patterns can contribute to a better understanding of recent damage patterns.

In addition, these former damage patterns might indicate "geotechnical zones of weakness" as the structure of the near-surface soil has been changed.

The annexed Plan 5 shows the distribution of the digitised "Drempels and Scheuren" in the investigated area. These areas are considered to indicate "geotechnical zones of weakness".

Recommendations:

For the handling of the mining relicts "Drempels and Scheuren" the following recommendations are made:

- The knowledge about the "Drempels and Scheuren" should be made available for the competent and responsible authorities at the municipal, provincial, and state levels.
- If damage events emerge or if damage is reported, both related to subsidence or ground heave, the local situation with regard to these "Drempels and Scheuren" should be checked.
- The "geotechnical zones of weakness" have to be considered by the planners of construction projects.
- Based on the ALARP-principle no preventive remediation measures seem to be feasible at the moment.

4.2.7 “Verzakkingen“

Basically “Verzakkingen“ are small-scale subsidences and sinkholes that emerged in the time of active mining. The digital mapping of these features is described in chap. 4.1.7.

These former damage patterns generally do not constitute a hazard. However „Verzakkingen“ are a clear indication for a weakend subsoil. There can be no presumption that the former subsidences have been sufficiently remediated. On the contrary, it has to be assumed that underlying underground mine voids can still be existent.

The annexed Plan 5 shows the distribution of the digitised “Verzakkingen” in the investigated area. These areas are considered to indicate “geotechnical zones of weakness”. In the original GIS-Version the “Verzakkingen” are digitised in their actual shape which in most cases is quite irregular. As these zones of “Verzakkingen” normally are very small, for reasons of visibility in Plan 5 all these “Verzakkingen” are designed by an enlarged violet dot.

Recommendations:

For the handling of the mining relicts “Verzakkingen” the following recommendations are made:

- The knowledge about the “Verzakkingen” should be given to the competent and responsible authorities at the municipal, the provincial, and the state levels.
- If damage events emerge or if damage is reported, especially that related to subsidence, the local situation with regard to these “Verzakkingen” should be checked.

- The authorities should arrange a visual inspection of each excavation pit by a geotechnical expert and/or mining expert if such a "Verzakking" is documented in the affected property.
- The "Verzakkingen" have to be considered by the planners of construction projects.

Aachen/Essen, 31. August 2016/Rev. a: 02. December 2016

DMT Dipl.-Ing. Peter Lux Dipl.-Ing. Tobias Friedrich

IHS Dr.-Ing. Michael Heitfeld Dr. Johannes Klünker

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Appendix 1

Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

Sampled data of historical shafts

by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of
Ministerie van Economische Zaken - The Netherlands

Aachen/Essen, 31. August 2016
(Rev. a: 02. December 2016)

DOM-Number	Mine shaft	Concession	Shaft location				Operation period		Geologic conditions				Shaft dimension	Category
			Easting (RD new) [m]	Northing (RD new) [m]	Position accuracy [m]	Ground surface level [mNAP]	Initiation date	Closing date	Overburden thickness [m]	Bedrock surface level [mNAP]	Shaft depth [m]	Sump [mNAP]	[m]	
9	Bure sur Steinknipp	Domaniale	202577	318788	±25	155	before 1828	n/s	38	117	150	10	n/s	1
10	Bure de la Paix/Friedensschacht/Fahrschacht/Bure d'air ou bure d'echelle	Domaniale	203121	318900	±10	164	1814	n/s	24	140	67 (176)	-12	1,6 x 2,7 (Riss)	2
11	Schacht op Senteweck	Domaniale	203363	318884	±15	165	before 1828	n/s	33	132	>33 (70)	95	n/s	2
12	Schacht op Athwerk/Verm. Oude Prickscht.	Domaniale	203463	318940	±15	165	before 1828	n/s	34	131	>33,5 (50)	115	n/s	2
13	Prickschacht/Verm. Oude Prickscht.	Domaniale	203538	318965	±20	165	n/s	n/s	39	126	>39 (55)	110	n/s	2
14	Bure sur Grauweck/Schacht op Grauweck	Domaniale	203394	319106	±15	163	n/s	n/s	32(40)	123	>32	n/s	n/s	2
15	Prickschacht/Vieux Bur Prick/Verm. Oude Prickscht.	Domaniale	203287	319116	±30	162	n/s	n/s	41	121	n/s	n/s	n/s	2
16	Oude Schacht	Domaniale	202984	319220	±20	159	n/s	n/s	46	113	n/s	n/s	n/s	2
17	Oude Schacht	Domaniale	202970	319248	±20	158	n/s	n/s	44	114	n/s	n/s	n/s	1
18	Oude Schacht Prick/Vieux Bur Prick/Verm. Oude Prickscht.	Domaniale	203255	319175	±30	162	n/s	n/s	41	121	n/s	n/s	n/s	2
20	Bonaparte daarna Wilhelm	Domaniale	203260	319422	±30	162	1814	n/s	46	116	n/s	n/s	n/s	1
21	St. Philippe	Domaniale	203297	319468	±30	162	n/s	n/s	46	116	n/s	n/s	n/s	1
22	Schacht no. 7 Guillaume actuel of Puits de Guillaume sur Athwerk/Bure Guillaume actuel/7/Bonaparte/Scht. 7	Domaniale	203396	319326	±15	162	1819	1828	34 (43)	119	72 (51)	n/s	1,3 x 2,5 (Riss)	1
23	Schacht no. 1 Succes of Bure comblé dit no. 1/Bure succes/Bure No 1/Puits d'Extraction dit No. 1 afsis fur la couche Grauwek/Scht. 1	Domaniale	203596	319462	±15	167 (162)	1819	before 1833	34 (42)	133 (120)	57 (51)	110,36	2,2 x 1,9 (1,5 x 2,8)	1
24	Schacht no. 2 of Bonne Esperance/Bur No 2	Domaniale	203403	319434	±30	162	1827	n/s	43	119	n/s	n/s	2,2 x 1,9	1
25	Schacht no. 6 de la nouvelle D'esperance/6/Scht. 6	Domaniale	203521	319296	±15	165	1819	1830	24 (41)	121	132 (55)	n/s	1,5 x 2,3 (Riss)	1
26	Schacht no. 5 D'esperance/Bure de L' Esperance/alter Förderschacht Hoffnung/5/Scht. 5	Domaniale	203567	319299	±15	165	1819	n/s	24 (41)	121	105	n/s	1,5 x 2,2 (Riss)	1
27	Schacht no. 8 Machine hydraulique à cheveaux/Kannaalschacht 6/alter Schacht/No 6/ Bure du Canal/Scht. 8	Domaniale	203590	319384	±15	167 (163)	n/s	1833	34 (41)	133 (121)	55	112,31	2,5 x 2,9 (Riss)	2
28	Schacht no. 2 de la machine D' Epuissement of Rosskunst/alter Schacht/Frühere Roßkunst/Machine hydraulic que a chevaux/Roßkunst/Puits de la Machine d' Epuissement afsis sur Athwerk/Scht. 3	Domaniale	203559	319396	±15	167 (163)	1819	n/s	32 (41)	135 (121)	54 (101)	112,87	1,7 x 2,7 (Riss)	1
29	Schacht no. 3 puits d'extraction of Bure aux pompen/Bure No 2/No 1/alter Schacht/Puits d'Extraction dit No.2 afsis fur la couche Grauwek/Scht. 2	Domaniale	203520	319421	±15	162	1814	n/s	34 (42)	128 (120)	133 (53)	112,44	1,6 x 2,5 (Riss)	1
30	Schacht no. 4 op Grauweck/Schacht op Grauweck/Bure de Grauweck/Oude Schacht	Domaniale	203674	319333	±15	165 (163)	1907	n/s	33 (40)	132 (123)	79	112,09	2,5 x 3,5 (Riss)	1
32	Alter Förderschacht/Alter Schacht/Alter Förderschacht	Neu Prick	202994	318572	±20	161	n/s	n/s	36	125	80	n/s	n/s	2
33	Dumont	Neu Prick	202839	318343	±20	161	1815	1822	34	127	n/s	n/s	n/s	1
34	Alter Schacht	Neu Prick	202861	318341	±20	161	n/s	n/s	34	127	n/s	n/s	2,0 x 3,8 (Riss)	1
35	Oude Prickschacht	Neu Prick	202800	318294	±30	160	n/s	n/s	32	128	n/s	n/s	n/s	1
36	Schiffer I	Neu Prick	202892	318275	±30	160	n/s	n/s	32	128	n/s	n/s	n/s	2
37	Schiffer II/Schifferschacht/Scht. Auf Großmühlenbach	Neu Prick	202928	318339	±20	162 (161)	n/s	n/s	34	127	120	42	2,4 x 3,5 (Riss)	1
38	Schacht op Mühlenbach	Neu Prick	202921	318165	±20	162	n/s	n/s	33	129	n/s	n/s	n/s	2
39	Backhausschacht/Alter Schacht	Neu Prick	202979	318158	±20	161 (162)	n/s	n/s	33	129	64	97	2,0 x 3,8 (Riss)	2
40	Oude Schacht	Neu Prick	203068	318012	±30	161	n/s	n/s	31	130	n/s	n/s	n/s	2
41	Valde Schacht/Alter Schacht/Valterschacht	Neu Prick	203041	318209	±20	161 (162)	n/s	n/s	33	129	48	113	2,5 x 3,8 (Riss)	2
42	Prick op Merl	Neu Prick	203200	318301	±30	162	n/s	n/s	33	129	n/s	n/s	n/s	1
43	Prick op Merl	Neu Prick	203048	318313	±30	162	n/s	n/s	34	128	n/s	n/s	n/s	1
44	Förderschacht/Scht. auf Merl	Neu Prick	203136	318362	±20	162	n/s	n/s	33	129	n/s	n/s	2,0 x 3,8 (Riss)	1
45	Alter Pumpenschacht 1/Alter Förderscht/Alter Schacht	Neu Prick	203222	318362	±20	164	n/s	n/s	33	131	n/s	n/s	2,2 x 3,8 (Riss)	1
46	Alter Pumpenschacht 2/Al. S.	Neu Prick	203283	318214	±20	164	n/s	n/s	33	131	n/s	n/s	2,8 x 4,4 (Riss)	1
47	Prickschacht/Förderschacht/Alter Fahrschacht	Neu Prick	203428	318391	±20	165	1889	n/s	34	131	n/s	n/s	2,2 x 4,0 (Riss)	1
48	Prickschacht/Bur Prick	Neu Prick	203454	318578	±30	164	n/s	n/s	34	130	n/s	n/s	n/s	1
49	Oude Schacht/Nicolausschacht/Nicolas Schacht	Neu Prick	203347	318009	±20	166	n/s	n/s	35	131	n/s	n/s	2,8 x 4,2 (Riss)	2
50	Oude Schacht	Neu Prick	203321	318100	±20	166	n/s	n/s	35	131	n/s	n/s	n/s	1
51	Schacht op Großmühlenbach	Neu Prick	203251	318170	±20	165	n/s	n/s	34	131	n/s	n/s	n/s	2
52	Feldgrubeschacht/alte Feldgrube	Neu Prick	203213	318084	±20	166	n/s	n/s	35	131	n/s	n/s	2,8 x 4,2 (Riss)	1
53	Couillet/Bur cuillet	Neu Prick	203302	317918	±30	166	n/s	n/s	34	132	n/s	n/s	n/s	1
54	Fetkoul/Bur Feldkoul	Neu Prick	203232	317832	±30	166	n/s	n/s	34	132	n/s	n/s	n/s	2
55	Fetkoul/Bur Feldkoul	Neu Prick	203161	317905	±30	163	n/s	n/s	32	131	n/s	n/s	n/s	1
56	Fetkoul/Bur Feldkoul	Neu Prick	203115	317961	±30	163	n/s	n/s	32	131	n/s	n/s	n/s	2
211	Prick Schacht/Pumpe/Alter Kunstschacht/Pumpen Scht.	Neu Prick	203385	317980	±20	166	n/s	n/s	34	132	n/s	n/s	2,8 x 4,4 (Riss)	1
214	Prick Schacht/Alter Schacht (TÖB 2504/5634/004)	Neu Prick	202923	317890	±20	156	n/s	n/s	25	131	n/s	n/s	n/s	2
215	Prick Schacht/Alter Schacht (TÖB 2504/5634/005)	Neu Prick	202926	317883	±20	156	n/s	n/s	25	131	n/s	n/s	n/s	2
216	- none -	Neu Prick	202953	317903	±20	153	n/s	n/s	24	129	n/s	n/s	n/s	1
218	Neu Prick	Neu Prick	203284	317706	±20	166	n/s	n/s	34	131	n/s	n/s	n/s	1
263	St. L (Stollenlichtloch)/Stollenschacht	Neu Prick	203428	318350	±20	165	n/s	n/s	34	131	n/s	n/s	n/s	2
264	Prick Schacht/Alter Schacht	Neu Prick	202976	318145	±20	162	n/s	n/s	33	129	n/s	n/s	n/s	2
269	Beerenbosch A	Domaniale	203670	320925	±20	140	n/s	n/s	162	-22	n/s	n/s	n/s	3
277	Beerenbosch B	Domaniale	203483	320589	±10	153 (152)	1905	1905	53	99	60	93	n/s	3
278	No. 8	Domaniale	203617	319387	±15	163	n/s	n/s	40	123	n/s	n/s	n/s	2
279	Bur Prick	Neu Prick	203434	318301	±30	166	n/s	n/s	34	132	n/s	n/s	n/s	1
280	- none -	Neu Prick	203339	318301	±20	165	n/s	n/s	34	131	n/s	n/s	n/s	1

- none -	Ham I	Willem Sophia	201775	318900	±10	131	1878	n/s	16	115	125	6	7,0	3

64	TOEB 2505/5635/001 (Schacht op Rauschenwerk)	Domaniale	203575	319022	±15	168	n/s	n/s	39	129	n/s	n/s	n/s	-
95	TOEB 2505/5634/016 (Oude Schacht)	Domaniale, Bostrop	203424	317819	±20	167	n/s	n/s	20	147	n/s	n/s	n/s	-
257	TOEB 2505/5636/012 (Instorting juni 1968 Sch.)	Domaniale	203612	319106	±5	166	n/s	n/s	40	126	n/s	n/s	n/s	-
- none -	TOEB 2504/5634/001 (Maschinenschacht Herrenkunst; Alter Schacht von Herrenkuhl)	Herrenkuhl	202877	317903	±15	155	n/s	n/s	23	132	n/s	n/s	n/s	-
- none -	TOEB 2504/5634/002 (Alter Schacht)	Herrenkuhl	202826	317886	±15	156	n/s	n/s	24	132	n/s	n/s	n/s	-
- none -	TOEB 2504/5634/003 (Alter Schacht)	Herrenkuhl	202840	317881	±15	156	n/s	n/s	24	132	n/s	n/s	n/s	-

black: original information
red: derived information

n/s: not specified

Appendix 2

Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

Bow-Tie-diagrams:
Shafts of historical mining
Shafts of industrial mining

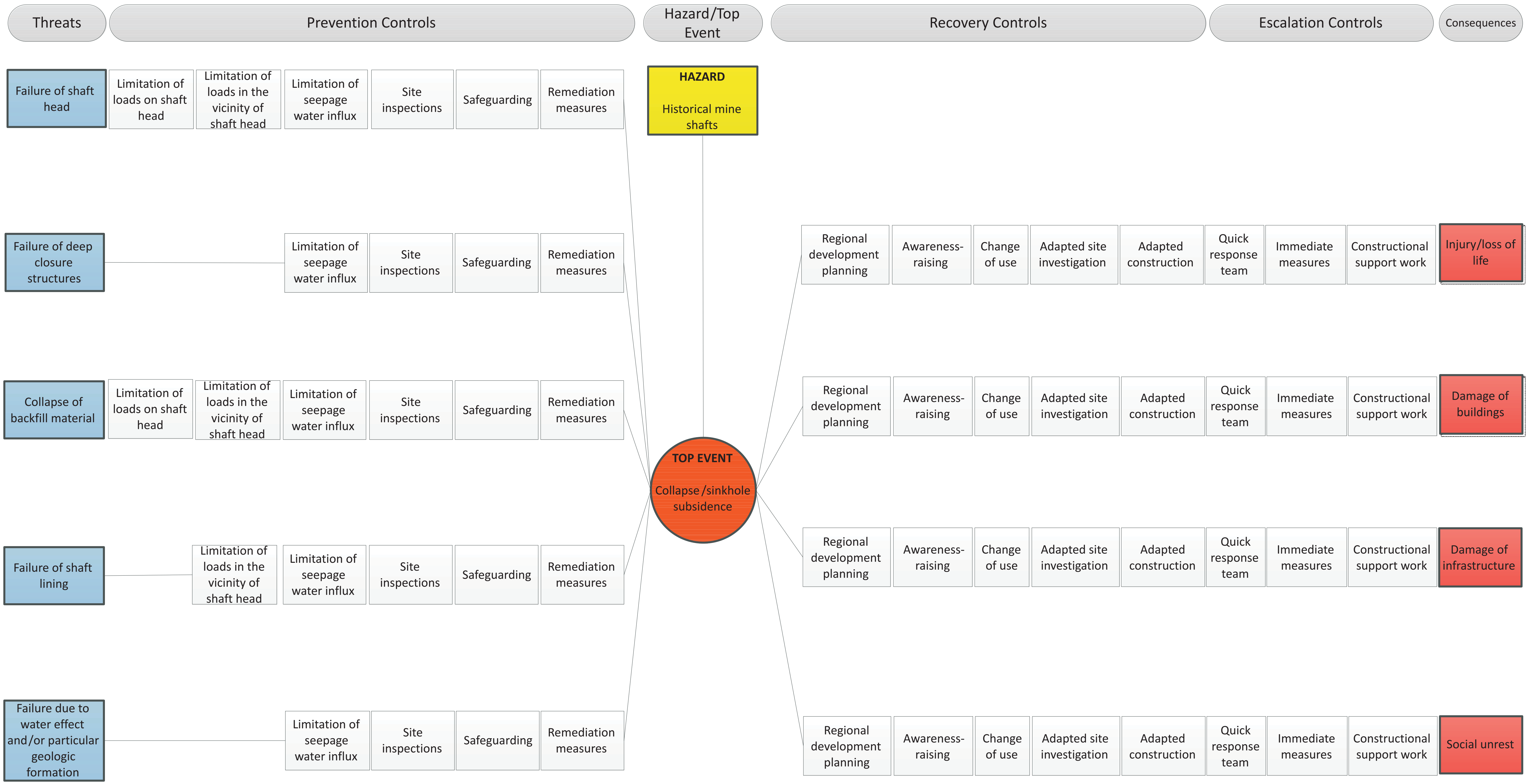
by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of
Ministerie van Economische Zaken - The Netherlands

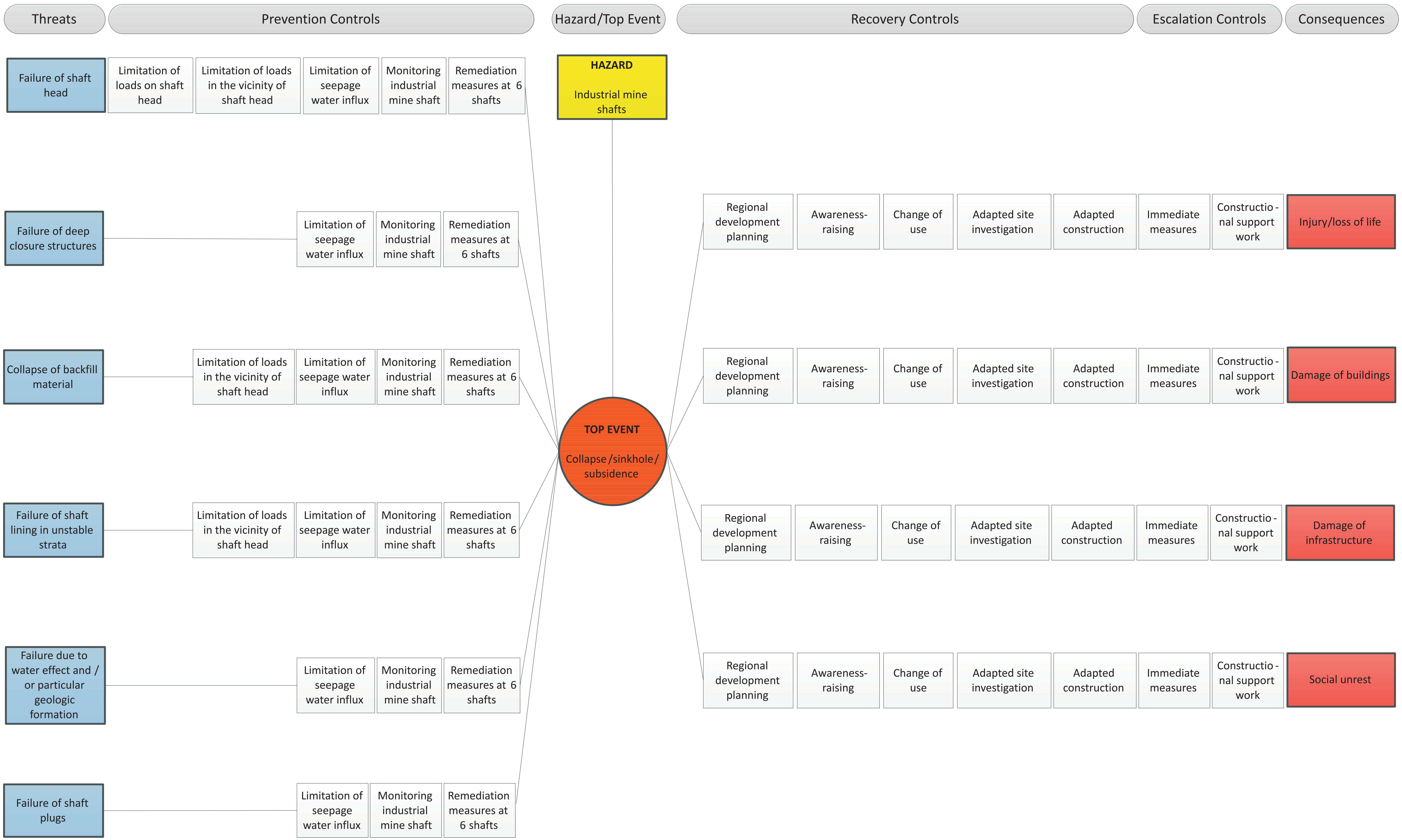
Aachen/Essen, 31. August 2016
(Rev. a: 02. December 2016)

5.2.2 Mine shafts



App. 2.1

5.2.2 Mine shafts



Appendix 3

Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

Bow-Tie-diagram: Near-surface mining

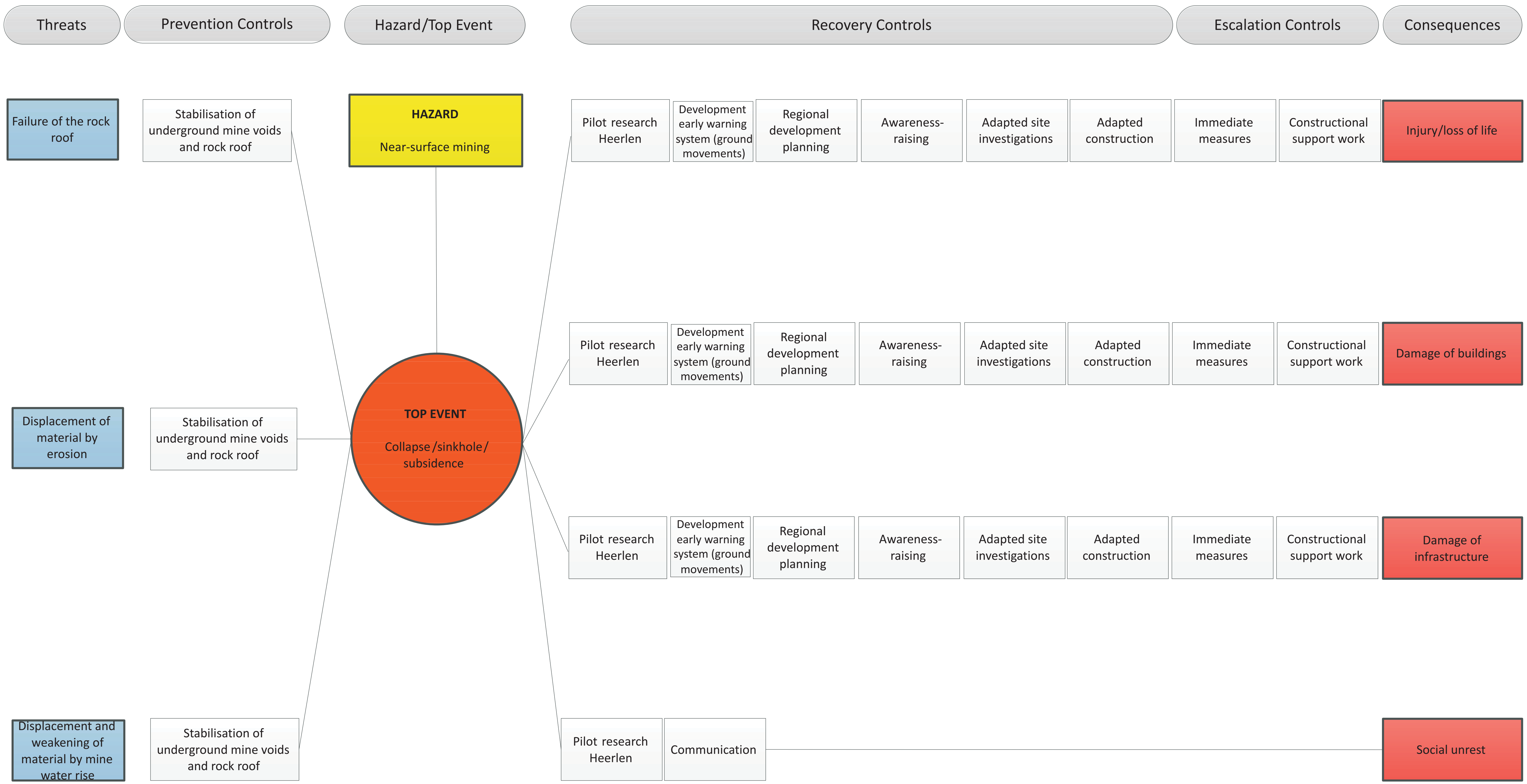
by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of
Ministerie van Economische Zaken - The Netherlands

Aachen/Essen, 31. August 2016
(Rev. a: 02. December 2016)

5.2.3 Near-surface mining



Appendix 4

Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

Reported results of the examination of the shaft documents

by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of
Ministerie van Economische Zaken - The Netherlands

Aachen/Essen, 31. August 2016
(Rev. a: 02. December 2016)

Contents

Figures

Tables

1 Introduction

In the context of the study “Na-ijlende gevolgen steenkolenwinning Zuid-Limburg” an extensive document collection about the abandonment of the industrial shaft could be compiled. This annex comprises a detailed examination of all available shaft documents. The shafts are discussed concession-wise; the location of the industrial shafts can be seen from Fig. 1.

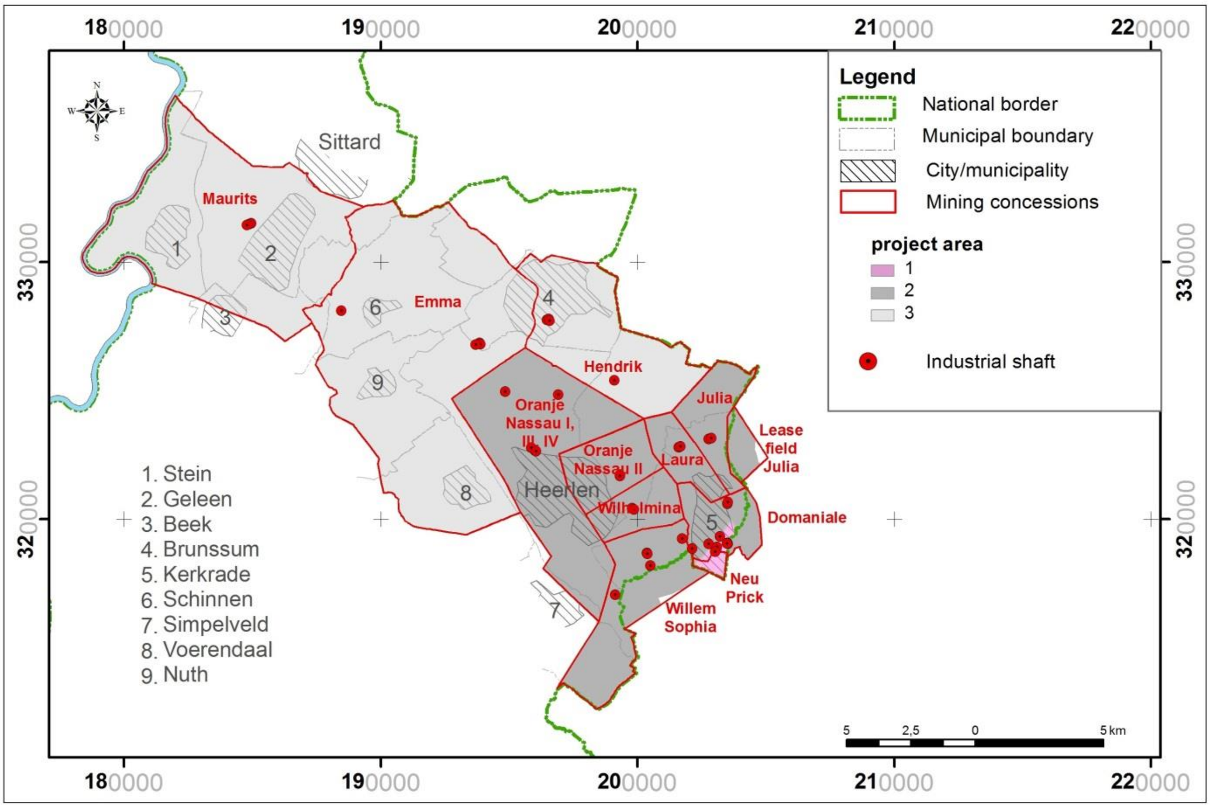


Fig. 1: Location of all industrial shafts in the South Limburg mining district

2 Domaniale Mijn

2.1 Buizenschacht

The vertical Buizenschacht of the pit Domaniale was drilled in 1904. In 1969 this shaft was backfilled and closed. According to documents available the shaft has an oval cross-section of 1,75 m x 1,25 m. The Buizenschacht was drilled to a total depth of 499 m and was used as ventilation shaft. The shaft wall was made of masonry (thickness of 0,50 m) /26/. There are no details available about any shaft fittings. In this area the overburden has a thickness of 42,55 m /26/.

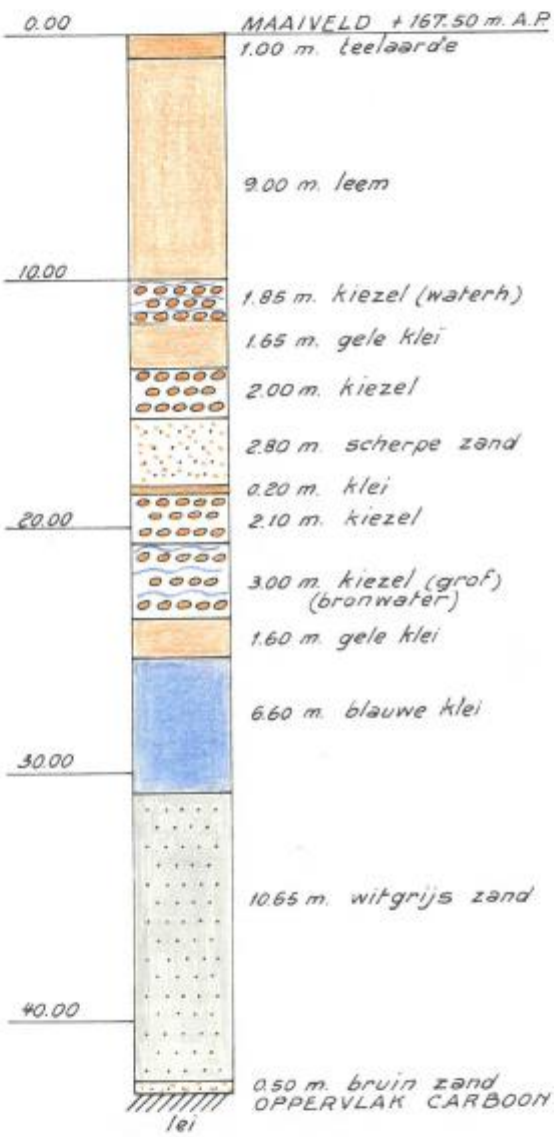


Fig. 2: Structure of the overburden in the range of the shafts Buizenschacht, Willem I and Willem II /26/

The overburden consists of topsoil, clay, gravel, silt and sand (Fig. 2).

The Buizenschacht has 17 documented insets /26/. The 40 m floor, as the topmost is located in a level of +121,41 m NAP and in a depth of 45,94 m /6/. In the year 1969 the shaft was closed on the 40 m floor (+121,41 m NAP) with a load bearing filling of a thickness of approximately 6 m. This filling consisted of a mixture of concrete with a quality of compactness of 325 H.A. (325 kg blast furnace cement, class A per m³ mixture) /9/ /25/.

For this purpose, approximately 2,5 m below the 40 m level, an inclined abutment was manufactured within the carbon at the shaft-landing /29/. After the ageing of the load bearing filling the open shaft column above was backfilled with a mixture of concrete with a quality of compactness of 300 H.A. (300 kg blast furnace cement, class A per m³ mixture) up to 2 m below the land surface.

In the range of the load bearing filling (40 m and 50 m floor) the connected gallery was sealed by means of pneumatic packing. Afterwards the shaft cover could be installed /9/.

Fig. 3 and 4 show the shaft barrier as sectional drawing.

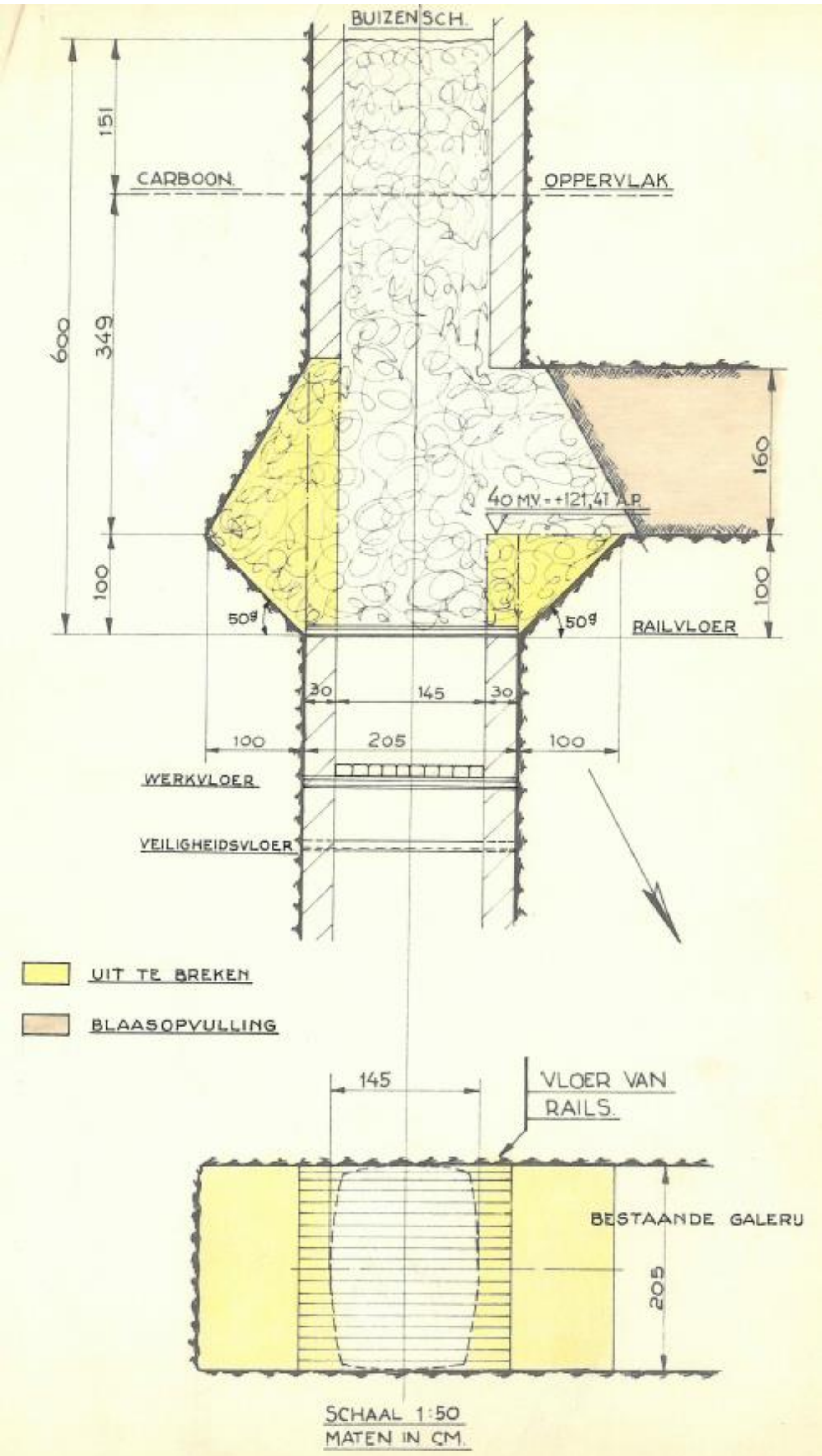
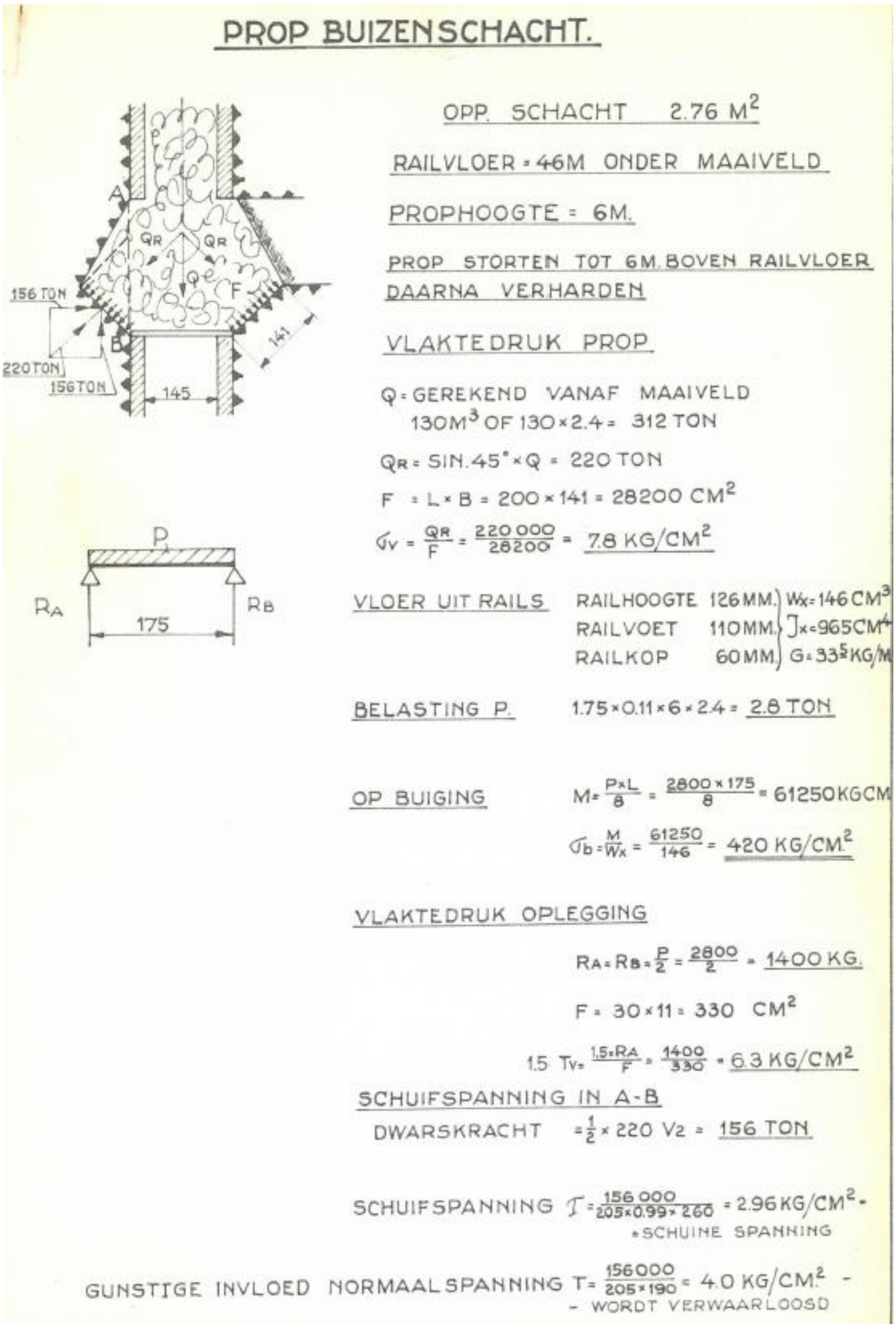


Fig. 3: Sectional drawing of the shaft barrier in the Buizenschacht /26/



OPP. SCHACHT 2.76 M^2

RAILVLOER = 46M ONDER MAAIVELD

PROPHOOGTE = 6M.

PROP STORTEN TOT 6M. BOVEN RAILVLOER
DAARNA VERHARDEN

VLAKTEDRUK PROP

Q = GEREKEND VANAF MAAIVELD
130M^3 OF 130 × 2.4 = 312 TON

Q_R = SIN.45° × Q = 220 TON

F = L × B = 200 × 141 = 28200 CM^2

$\sigma_V = \frac{Q_R}{F} = \frac{220\,000}{28200} = 7.8$ KG/CM^2

VLOER UIT RAILS — RAILHOOGTE 126 MM., RAILVOET 110 MM., RAILKOP 60 MM. — W_x = 146 CM^3, J_x = 965 CM^4, G = 33.5 KG/M

BELASTING P. 1.75 × 0.11 × 6 × 2.4 = 2.8 TON

OP BUIGING $M = \frac{P \times L}{8} = \frac{2800 \times 175}{8} = 61250$ KGCM

$\sigma_b = \frac{M}{W_x} = \frac{61250}{146} = 420$ KG/CM^2

VLAKTEDRUK OPLEGGING

$R_A = R_B = \frac{P}{2} = \frac{2800}{2} = 1400$ KG.

F = 30 × 11 = 330 CM^2

1.5 $T_V = \frac{1.5 \cdot R_A}{F} = \frac{1400}{330} = 6.3$ KG/CM^2

SCHUIFSPANNING IN A-B
DWARSKRACHT $= \frac{1}{2} \times 220\sqrt{2} = 156$ TON

SCHUIFSPANNING $\tau = \frac{156\,000}{205 \times 0.99 \times 260} = 2.96$ KG/CM^2 - = SCHUINE SPANNING

GUNSTIGE INVLOED NORMAALSPANNING $T = \frac{156000}{205 \times 190} = 4.0$ KG/CM^2 - - WORDT VERWAARLOOSD

Fig. 4: Details shaft barrier Buizenschacht /26/

The coordinates of the Buizenschacht are:

RD-x:	203493
RD-y:	319045
Elevation:	+167 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space, used as a playground, northeast of the road Finefrau (community Kerkrade) close to a residential allotment.

2.2 Willem I

The vertical Shaft Willem I of the pit Domaniale was drilled in 1828. In 1969 this shaft was backfilled and closed. According to documents available the shaft has a rectangular cross-section of 4,30 m x 2,60 m. Between the 200 m floor and the 380 m floor the shaft has a cross-section of 6,42 m x 2,60 m. The shaft Willem I was drilled to a total depth of 393,37 m and was used as drawing shaft. The shaft wall was made of masonry (thickness of 0,50 m) and steel support /26/. Within the level of +118,94 m NAP a dewatering (ø 300 mm) was installed /26/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 42,55 m /26/. Fig. 2 shows the structure of the overburden in the range of the shaft Willem I /26/.

The shaft Willem I has 21 documented insets /26/ /50/. The 40 m floor, as the topmost is located in a level of +121,41 m NAP and in a depth of 45,94 m /6/.

In the year 1969 the shaft was closed on the 40 m floor (+121,41 m NAP) with a load bearing filling of a thickness of approximately 6 m. This filling consisted of

113 m³ mixture of concrete with a quality of compactness of 325 H.A. (325 kg blast furnace cement, class A per m³ mixture) /9/ /25/ /50/.

For this purpose, approximately 2,5 m below the 40 m level, an inclined abutment was manufactured within the carbon at the shaft-landing //9/ /25/ /50/. After the ageing of the load bearing filling the open shaft column above was backfilled with 474 m³ mixture of concrete with a quality of compactness of 300 H.A. (300 kg blast furnace cement, class A per m³ mixture) up to 2 m below the land surface. In the range of the load bearing filling (40 m and 50 m floor) the connected gallery was sealed by means of pneumatic packing. Afterwards the shaft cover could be installed /9/.

In the range of the load bearing filling (40 m and 50 m floor) the connected gallery was sealed by means of pneumatic packing. Afterwards the shaft cover could be installed /9/ /50/.

During this backfilling the dewatering was closed likewise /26/. There is no further information about the robbing level up to which a withdrawing of this dewatering system had taken place.

Fig. 5 shows the shaft barrier on the 40 m floor.

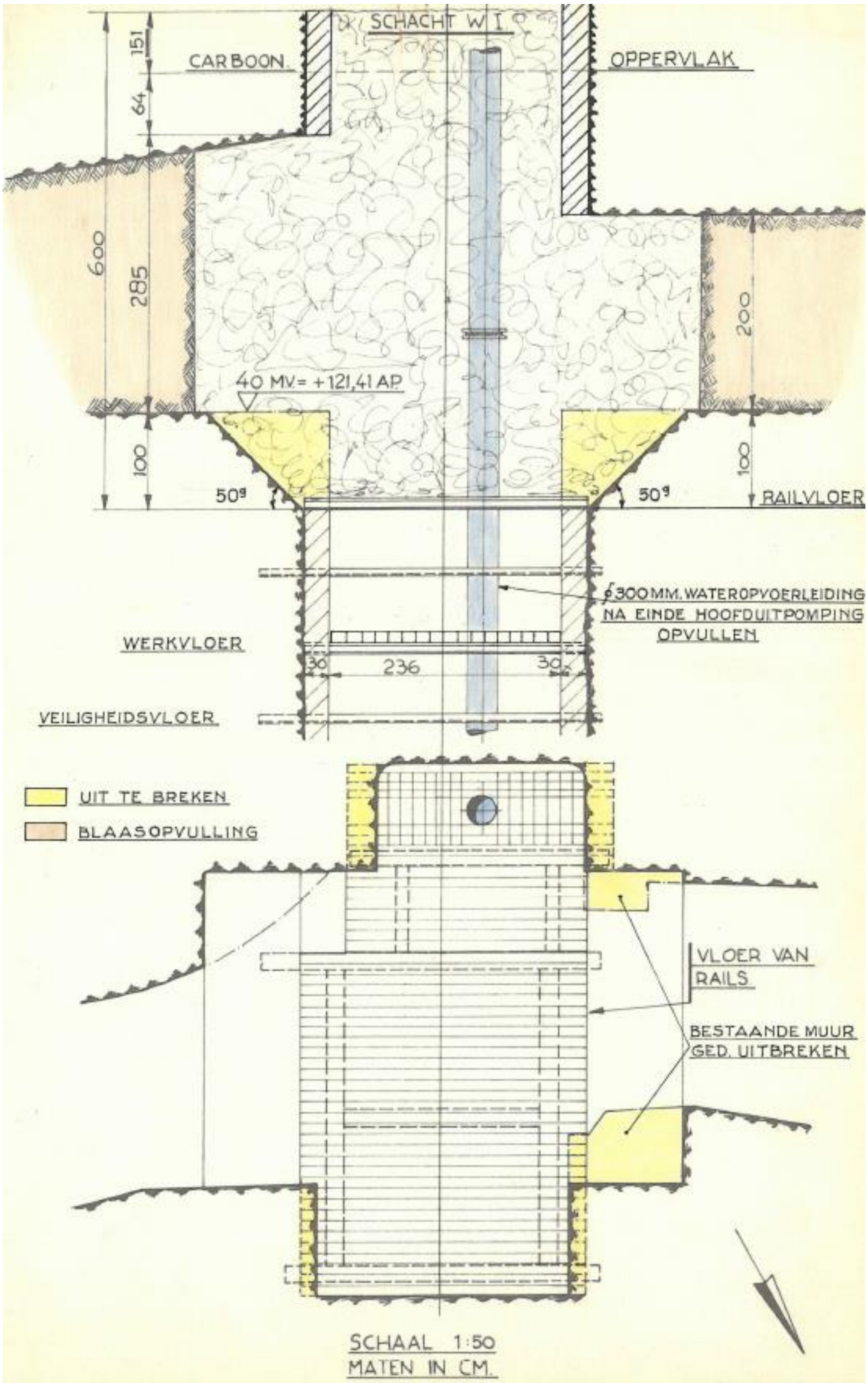
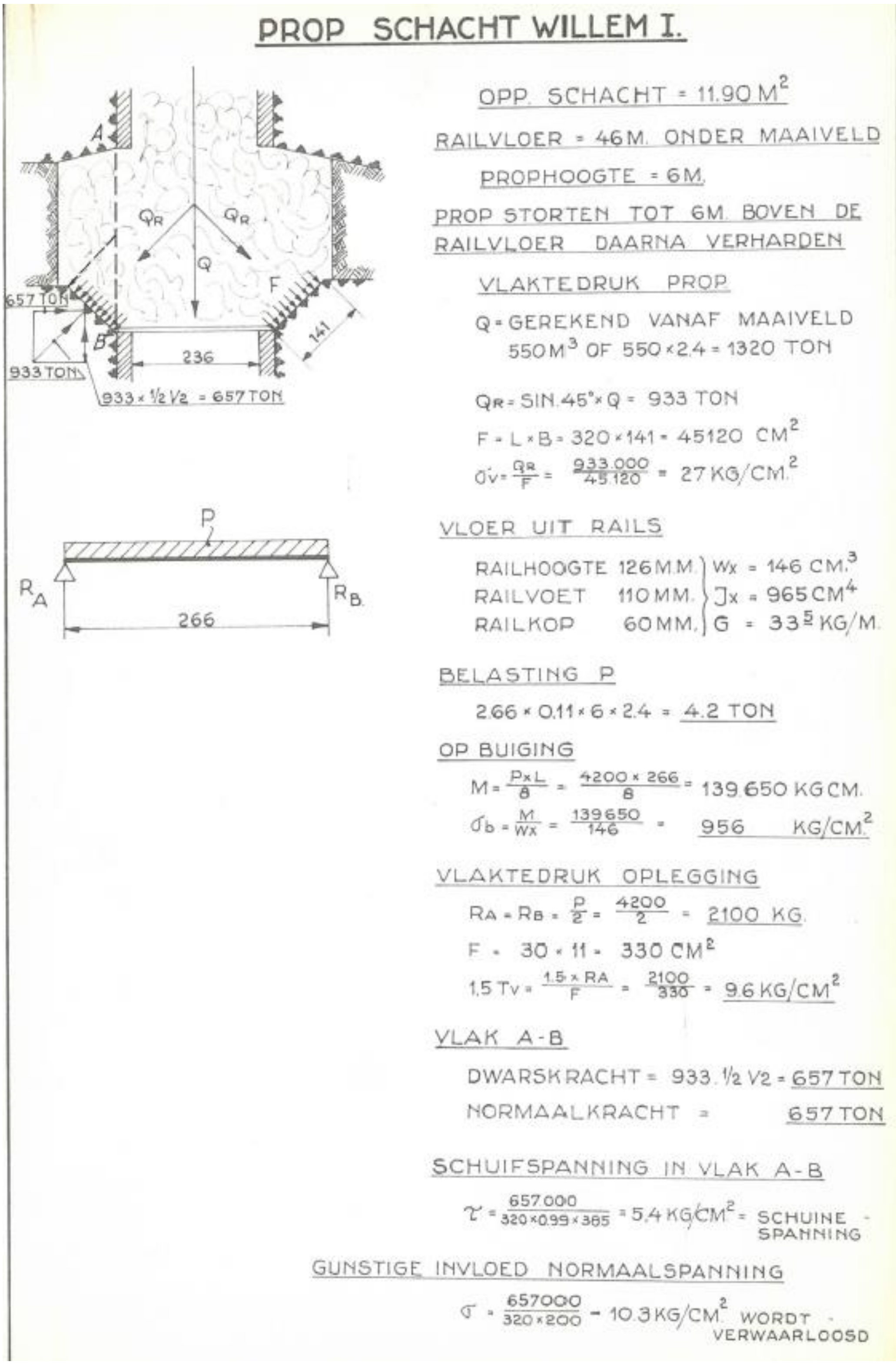


Fig. 5: Schema shaft barrier of shaft Willem I /26/



OPP. SCHACHT = 11.90 M²

RAILVLOER = 46 M. ONDER MAAIVELD

PROPHOOGTE = 6 M.

PROP STORTEN TOT 6 M. BOVEN DE RAILVLOER DAARNA VERHARDEN

VLAKTEDRUK PROP

Q = GEREKEND VANAF MAAIVELD
550 M³ OF 550 × 2.4 = 1320 TON

$Q_R = SIN. 45° × Q = 933$ TON

F = L × B = 320 × 141 = 45120 CM²

$\sigma_v = \frac{Q_R}{F} = \frac{933.000}{45.120} = 27$ KG/CM²

VLOER UIT RAILS

RAILHOOGTE 126 MM. | W_x = 146 CM.³
RAILVOET 110 MM. | J_x = 965 CM⁴
RAILKOP 60 MM. | G = 33⁵ KG/M.

BELASTING P

2.66 × 0.11 × 6 × 2.4 = 4.2 TON

OP BUIGING

$M = \frac{P \times L}{8} = \frac{4200 \times 266}{8} = 139.650$ KG CM.

$\sigma_b = \frac{M}{W_x} = \frac{139650}{146} = 956$ KG/CM²

VLAKTEDRUK OPLEGGING

$R_A = R_B = \frac{P}{2} = \frac{4200}{2} = 2100$ KG.

F = 30 × 11 = 330 CM²

$1.5\,T_v = \frac{1.5 \times R_A}{F} = \frac{2100}{330} = 9.6$ KG/CM²

VLAK A-B

DWARSKRACHT = 933 . ½ V2 = 657 TON

NORMAALKRACHT = 657 TON

SCHUIFSPANNING IN VLAK A-B

$\tau = \frac{657.000}{320 \times 0.99 \times 385} = 5.4$ KG/CM² = SCHUINE SPANNING

GUNSTIGE INVLOED NORMAALSPANNING

$\sigma = \frac{657000}{320 \times 200} = 10.3$ KG/CM² WORDT VERWAARLOOSD

Fig. 6: Details shaft barrier of shaft Willem I /26/

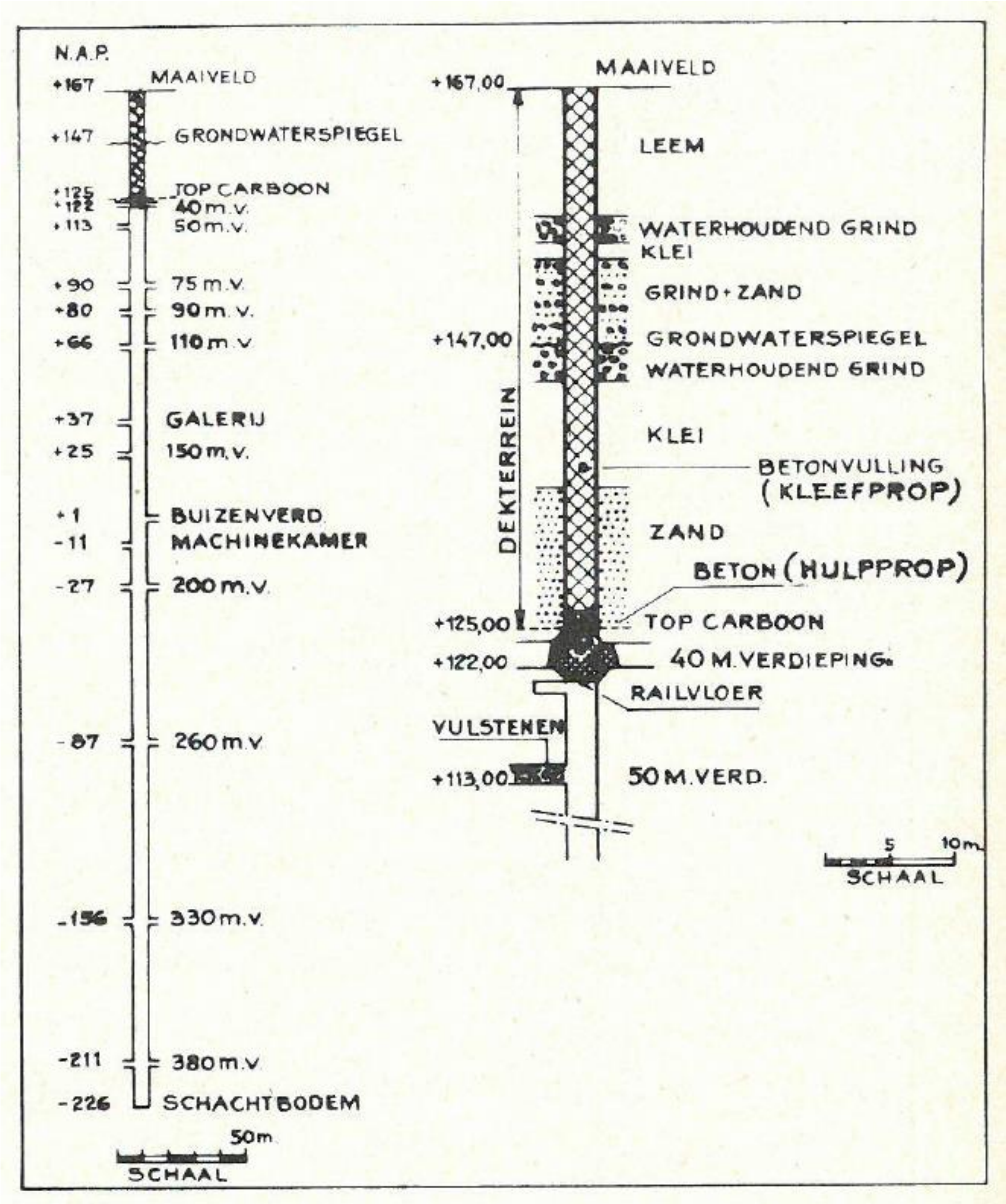


Fig. 7: Securing shaft Willem I /50/

The coordinates of the Shaft Willem I are:

RD-x:	203502
RD-y:	319058
Elevation:	+167 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space, used as a playground, northeast of the road Finefrau (community Kerkrade) close to a residential allotment.

2.3 Willem II

The vertical Shaft Willem II of the pit Domaniale was drilled in 1927. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a rectangular cross-section of 8,30 m x 3,70 m. Beneath the 620 m floor the cross-section tapers of to 5,30 m x 4,30 m /2/. The shaft Willem II was drilled to a total depth of 804 m and was used as drawing shaft. The shaft wall was made of masonry (thickness of 0,50 m) /26/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 42,90 m /26/. Fig. 2 shows the structure of the overburden in the range of the shaft Willem II /26/. The shaft Willem II has 19 documented insets /26/. The 40 m floor, as the topmost is located in a level of +121,41 m NAP and in a depth of 46 m /6/.

In the year 1970 there were installed seven heavy iron beams in a depth of 50 m. On these a landing consisting of several bars under a layer of concrete with a thickness of 1 m was mounted. These bars were embedded in the shaft wall, due to the fact that there was no connection to the next floor /29/. Following the shaft was closed with a load bearing filling of the thickness of approximately 8 m. This filling consisted of a mixture of concrete with a quality of compactness of 325 H.A. (325 kg blast furnace cement, class A per m³ mixture) and was drawn up to 1 m above the carbon /10/ /25/. After the ageing of the load bearing filling the openly shaft column above was backfilled with a mixture of concrete with a quality of compactness of 150 H.A. (150 kg blast furnace cement, class A per m³ mixture) up to 2 m below the land surface. Overall the load bearing filling has a length of approximately 50 m. Overall 1.084 m³ concrete were filled in /10/ /25/.

In order to measure the mine-water level, in 1980 the shaft barrier was perforated and equipped with a steel tube. The monitoring well is enclosable /18/. The drilling showed that the submitted plans for the shaft closure were not complied. The upper and lower part of the shaft cage rope were embedded in the load bearing filling and therefore the loose ends are hanging freely in the shaft over the total length of 600 m. Drill cores were obtained of the filling through it's whole length of 50 m. Analysis brought up results of only ½ the required compressive strength of the load bearing filling /18/. Wherein the unconfined compressive strength of the load bearing filling (depth 40,5 m to 47,3 m) was determined with results between 9,2 NM/m² and 19,9 MN/m². In the depth between 2,0 m and 33,0 m the compressive strength of the concrete was determined with results between 6,9 NM/m² and 15,1 MN/m² /26/.

The following figure shows the implementation planning of the shaft barrier.

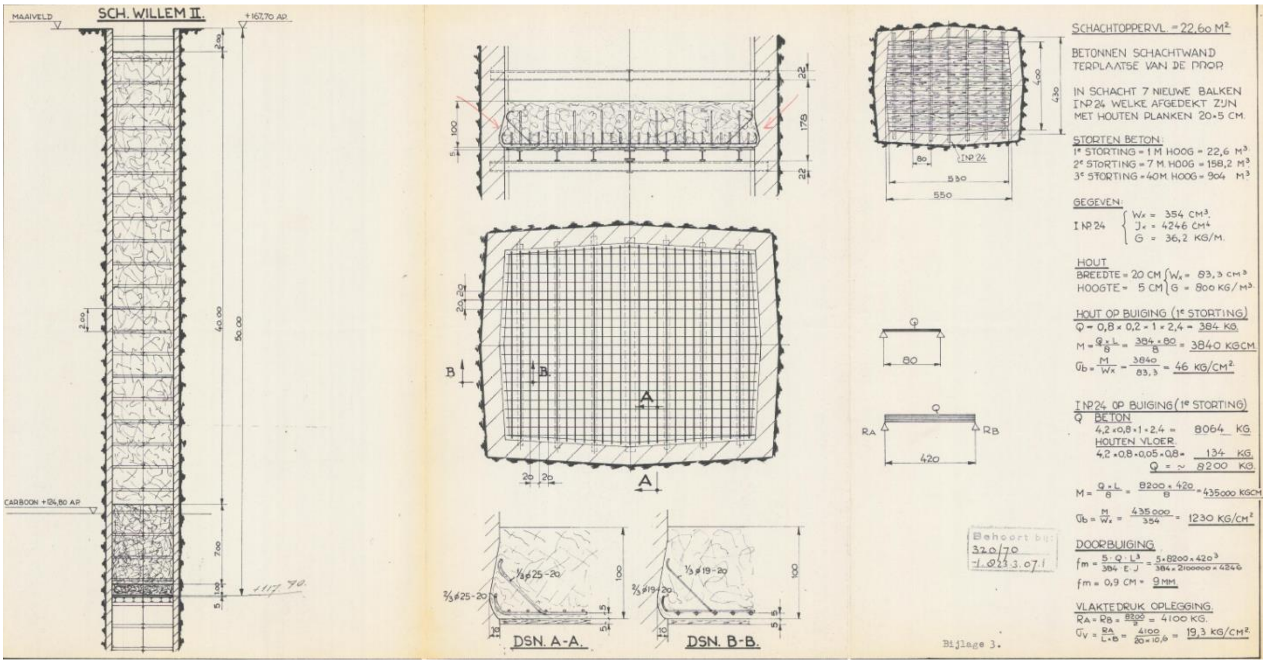


Fig. 8: Implementation planning load bearing filling shaft Willem II /32/

The coordinates of the Shaft Willem II are:

RD-x:	203529
RD-y:	319037
Elevation:	+168 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space, used as a playground, northeast of the road Finefrau (community Kerkrade) close to a residential allotment.

2.4 Beerenbosch I

The vertical Shaft Beerenbosch I was drilled in 1905 and sunk in 1928. In 1969 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 2,65 m diameter. The shaft Beerenbosch I was drilled to a total depth of 482 m and was used as ventilation shaft. In the range of the overburden the shaft has tubbing support. In the range of the carbon the shaft wall was made of masonry (thickness of 0,80 m) /26/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 45,35 m /26/. The shaft Beerenbosch I has 13 documented insets /26/. The 60 m floor, as the topmost is located in a level of +93,65 m NAP and in a depth of 53 m /6/.

In the year 1969 the shaft was closed on the 60 m floor (+93,65 m NAP) with 260 m³ load bearing filling of the thickness of approximately 6 m. This filling consisted of a mixture of concrete with a quality of compactness of 325 H.A. (325 kg blast furnace cement, class A per m³ mixture). The concrete seal is positioned upon two abutment surfaces in the carbon at a depth of 52,85 m /29/.

After the ageing of the load bearing filling the openly shaft column above was backfilled with a mixture of concrete with a quality of compactness of 200 H.A. (200 kg blast furnace cement, class A per m³ mixture) up to 2 m below the land surface /9/.

The coordinates of the shaft Beerenbosch I are:

RD-x:	203503
RD-y:	320588
Elevation:	+147 m NAP
Position accuracy:	+/- 1 m

According to the coordinates the shaft is located in a wooded area north the Berenbosweg (community Kerkrade).

2.5 Beerenbosch II

The vertical Shaft Beerenbosch II was drilled in 1917. In 1994 this shaft was backfilled and closed. According to documents available the shaft has a rectangular cross-section of 5,30 m x 3,80 m. The shaft Beerenbosch II was drilled to a total depth of 501,78 m and was used as ventilation shaft. Up to 1994 the shaft was used as pumping shaft.

The shaft lining is listed in Tab. 1.

Tab. 1: Overview shaft lining Beerenbosch II /27/

depth [m]	lining
0 – 4	approx. in-situ concrete, thickness 0,6 m
4 – 71	approx. natural stone, thickness 0,6 m
71 – 106	approx. in-situ concrete, thickness 0,6 m
106	local areas of repair within the masonry and concrete lining
106 – 107	approx. masonry, thickness 0,6 m
107 – 120	approx. in-situ concrete, thickness 0,6 m
120 – 129	approx. natural stone, thickness 0,6 m
129 – 501,78	approx. in-situ concrete, thickness 0,6 m

The rectangular shaft section consisted of four compartments (drawing compartment, ventilation compartment, travelling compartment and pumping compartment). The existing shaft fittings were a ventilation duct (NW 700), a cable, one pneumatic line (NW 50) and three ascending pipelines (NW 300) /27/.

In this area the overburden has a thickness of 46,15 m /26/. Therefore the carbon is located in a level of approximately +100 m NAP /27/. The overburden consists of a quaternary cover with a thickness of 1,4 m. Beneath this cover follow clay to a depth of 27,5 m and sand to approximately 47,0 m depth /27/.

A regional fault with a max. perpendicular displacement of 1,5 m passes north-south wards through the shaft on the level of Laag Merl (+67,0 m NAP). The whole strata sequence in this coal mining beneath approximately 47,0 m depth can be considered as “stable” by means of shaft sinking /27/. The shaft Beerenbosch II has 13 documented insets /26/. In Tab. 2 depth-dependent facts of the Shaft Berenbosch II are listed.

Tab. 2: Overview levels shaft Beerenbosch II /27/

designation	elevation	depth
pit bank	+145,48 m NAP	
air drift	+139,08 m NAP	6,40 m
75 m floor	+75,40 m NAP	70,08 m
200 m floor	-20,03 m NAP	165,51 m
260 m floor	-85,12 m NAP	230,60 m
280 m floor	-107,20 m NAP	252,68 m
380 m floor	-208,82 m NAP	354,30 m

500 m floor	-329,03m NAP	474,51 m
shaft floor	-356,30 m NAP	501,78 m

In 1994 the shaft was backfilled with waste rock from the shaft sump up to a depth of 196 m.

The following 10 m (from 196 m to 186 m) were backfilled with a drainage filter consisting of coarse debris (gravel and sand, whereas the sand prevents the destruction of the drainage filter by infiltrations of concrete) imbedding the strainer for a pump tube.

Between 186,0 m and 146,0 m depth the load bearing filling was made of concrete of a quality of B 15. Temporarily the section between 186,0 m and 166,0 m was used as deformation resistant abutment. Between 146,0 m and 86,0 m depth a concrete of a quality of B 5 was used and between 86,0 m and 4,0 m below the land surface a concrete of a quality of B 2 was used to backfill the shaft. For both concretes B 15 and B 5 there was used a cement NW-HS. Up to 200 m depth a pump tube is installed for potential dewatering of mine water /23/ /24/ /27/.

Within the section composed of cohesive and bearing concrete (186,0 m to 146,0 m) the present part of piping was removed. Hereby the tension stress upon the bearing parts is omitted /27/.

Up to 10 m underneath the cohesive filling a gauge (NW 300) was installed to measure the rising mine water level /27/.

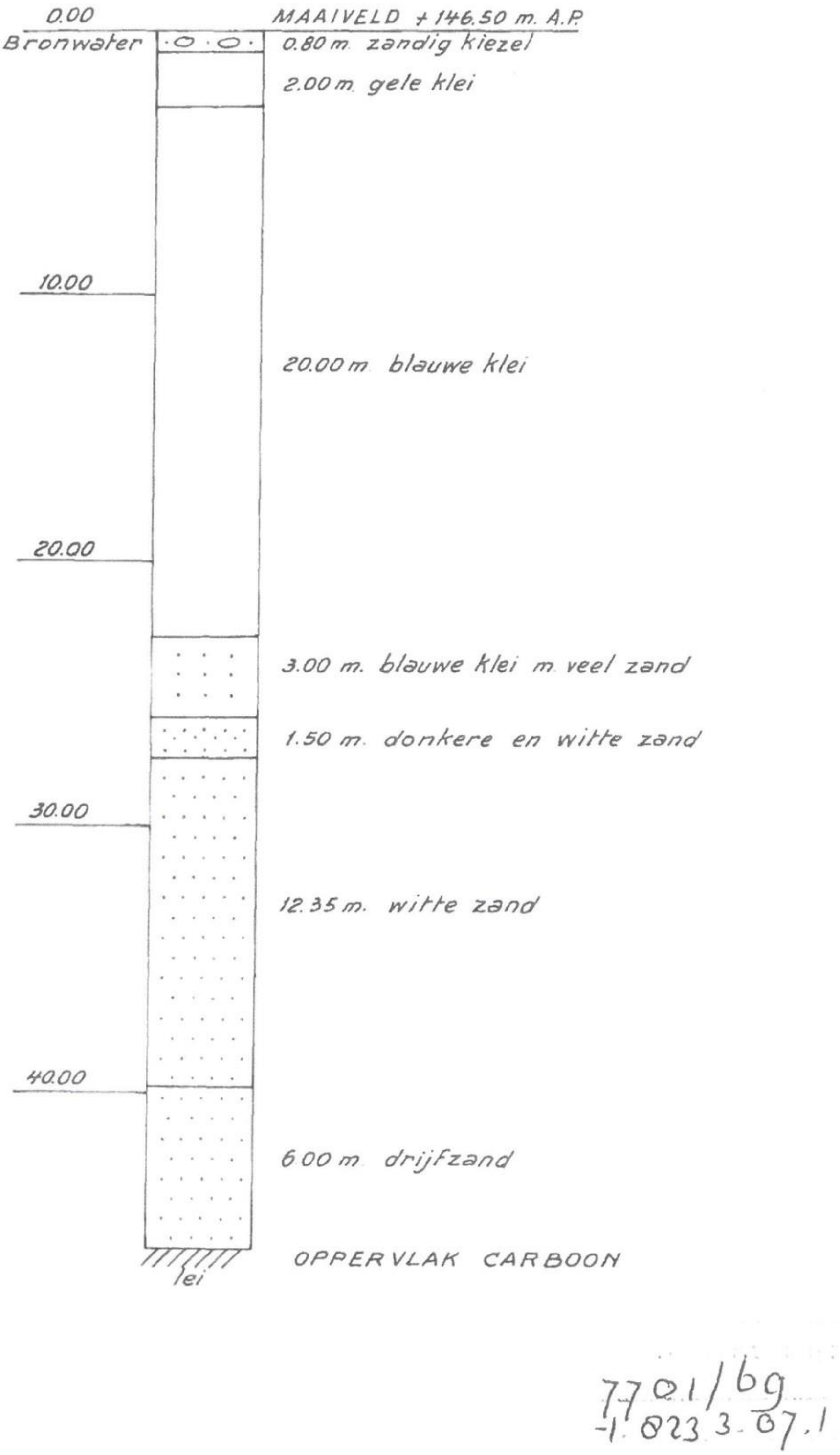


Fig. 9: Composition of the overburden at shafts Beerenbosch I and Beerenbosch II /79/

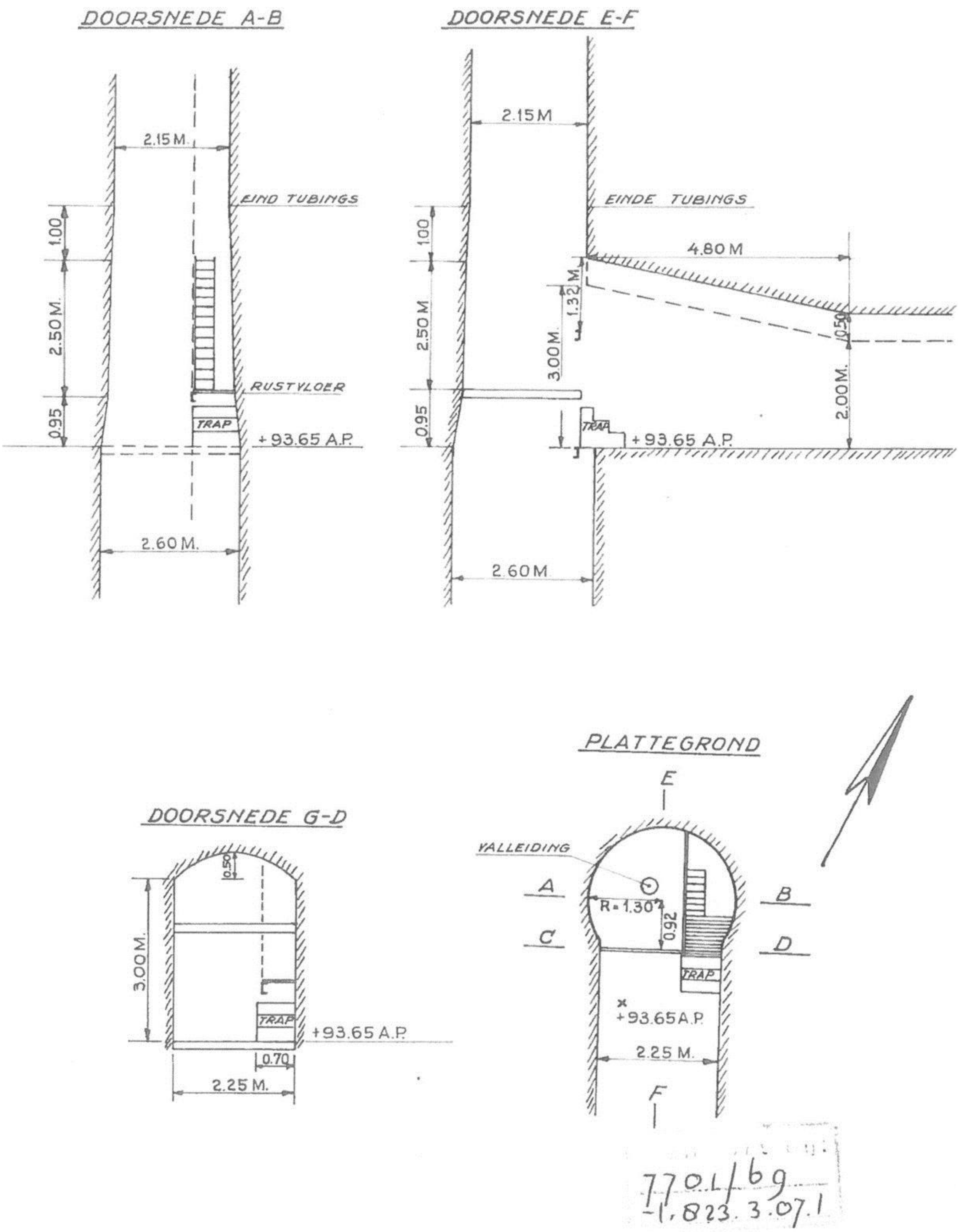


Fig. 10: Profiles in the range of the 60 m floor, shaft Beerenbosch I /79/

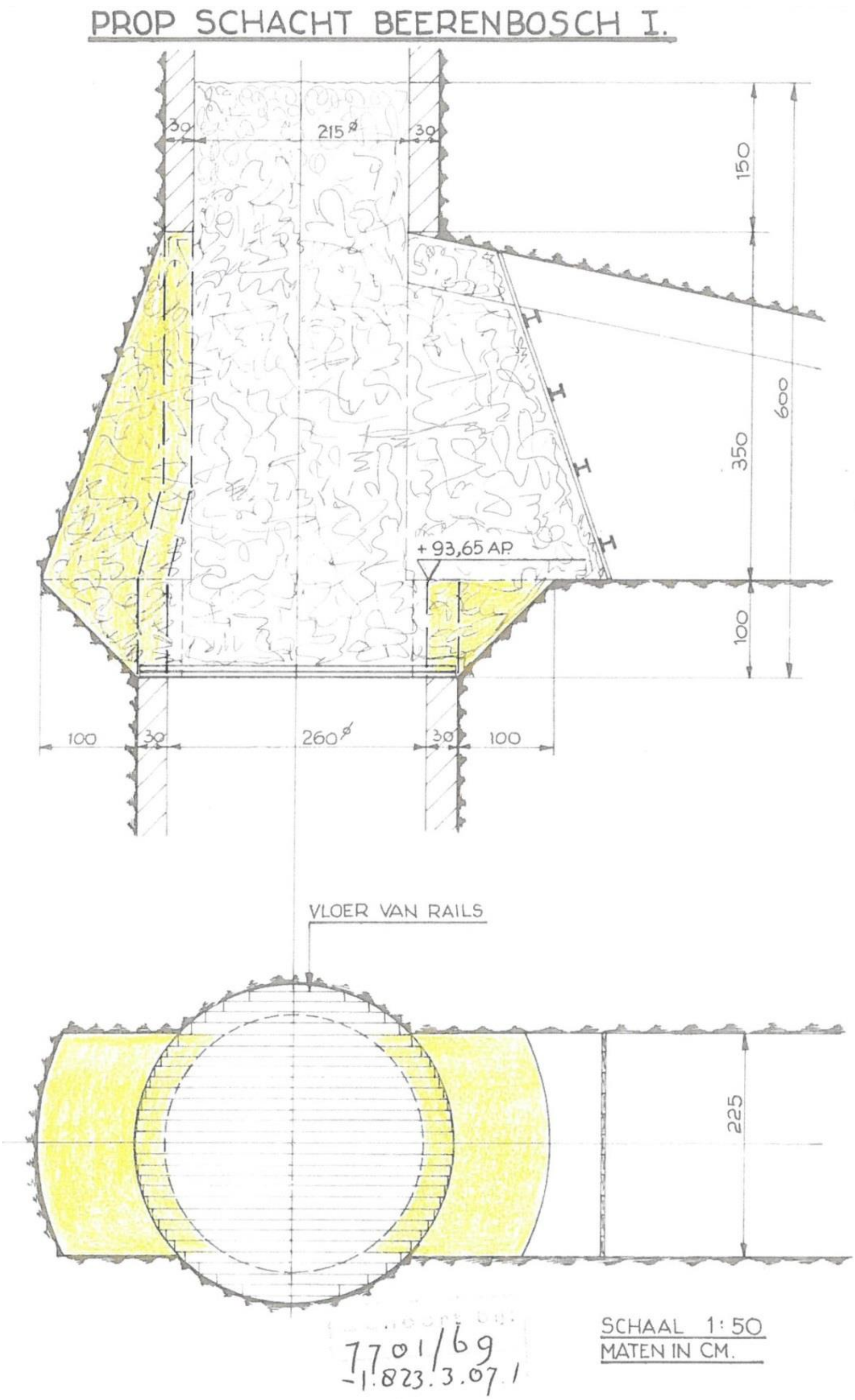
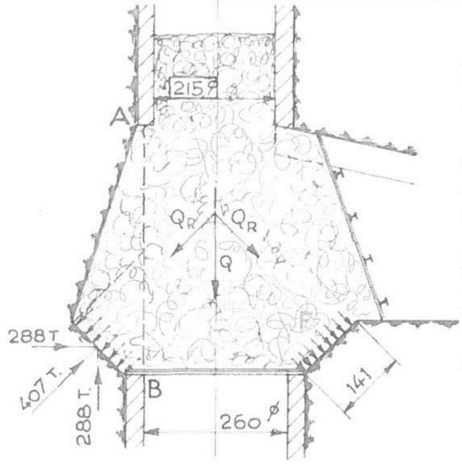


Fig. 11: Implementation planning load bearing filling shaft Beerenbosch I /79/

PROP SCHACHT BEERENBOSCH I.



OPPERVLAK SCHACHT = 5,3 M^2.
RAILVLOER = CA. 54 M. ONDER MAAIVELD.
VULLING SCHACHT TOT 2 M. ONDER MAAIVELD.

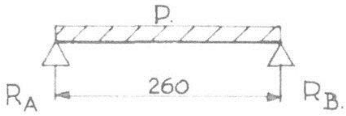
PROPHOOGTE = 6 M
PROP STORTEN TOT 6 M. BOVEN DE RAILVLOER
DAARNA VERHARDEN.

VLAKTEDRUK PROP
Q = GEREKEND VANAF 2 M ONDER MAAIVELD =
CA 240 M^3 OF 240 × 2,4 = 576 TON.

Q_R = SIN 45° × Q = 407 TON.
F = L × B = 2,25 × 1,41 = 3,17 M^2 = 31700 CM^2
$\sigma_V = \frac{Q_R}{F} = \frac{407000}{31700} = 12,8$ KG/CM^2.

VLOER UIT RAILS.
RAILHOOGTE 126 MM. | W_x = 146 CM^3
RAILVOET 110 MM. | J_x = 965 CM^4
RAILKOP 60 MM. | G = 33,5 KG/M.

BELASTING P. (PER RAIL).
P = 2,6 × 0,11 × 6 × 2,4 = 4,12 TON.



OP BUIGING.
$M = \frac{P \times L}{8} = \frac{4120 \times 260}{8} = 133\,900$ KGCM.
$\sigma_b = \frac{M}{W_x} = \frac{133.900}{146} = 917$ KG/CM^2

VLAKTEDRUK OPLEGGING.
$R_A = R_B = \frac{P}{2} = 2060$ KG.
F = 25 × 11 = 275 CM^2
$\sigma_V = \frac{R_A}{F} = \frac{2060}{275} = 7,5$ KG/CM^2.

VLAK A-B.
DWARSKRACHT = 407 × ½√2 = 288 TON
NORMAALKRACHT = 288 TON.

SCHUIFSPANNING IN VLAK A-B.
$\tau = \frac{288\,000}{440 \times 2.25} = 3$ KG/CM^2 = SCHUINE SPANNING.

GUNSTIGE INVLOED NORMAALSPANNING.
$\sigma = \frac{288\,000}{225 \times 200} = 6,4$ KG/CM^2 (WORDT VERWAARL.)

7701/69
-1.823.3.07.1

Fig. 12: Static calculation shaft Beerenbosch I /79/

The coordinates of the Shaft Beerenbosch II are:

RD-x:	203517
RD-y:	320662
elevation:	+145 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located to the north of Berenbosweg (community Kerkrade) within a fenced open space.

2.6 Nulland

The vertical Shaft Nulland was drilled in 1907. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 3,50 m diameter. The ventilation shaft Nulland was drilled to a total depth of 347 m and was used as travelling ventilation shaft. The shaft wall was made of masonry (thickness of 0,50 m) up to the 260 m floor and beneath that floor the shaft is made in steel support /26/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 41,45 m /9//26/. The ventilation shaft Nulland has 13 documented insets /26/. The 60 m floor, as the topmost is located in a level of +92,80 m NAP and in a depth of 63 m /6/.

In 1969 on the 60 m floor on each sides of the bedstop abutments were set for the installation of the shaft barrier /9/. In the year 1970 the shaft was closed on the 60 m floor (+92,80 m NAP) with 630 m³ load bearing filling of the thickness of approximately 6 m. This filling consisted of a mixture of cement and gravel with a quality of compactness of 325 H.A. (325 kg blast furnace cement, class A per m³ mixture) and was installed on an abutment of beams and bars /9/ /10/ /25/.

After the ageing of the load bearing filling the open shaft column above was backfilled with a mixture of concrete and gravel with a quality of compactness of 150 H.A. (150 kg blast furnace cement, class A per m³ mixture) up to 2 m below the land surface /9//10//25/.

The coordinates of the Shaft Nulland are:

RD-x:	202776
RD-y:	319031
elevation:	+156 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located at the Domaniale Mijnstraat (community Kerkrade). The former shaft building today is used as an art gallery and apartment.

2.7 Baamstraat

The vertical shaft Baamstraat was drilled in 1962. In 1967 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 2,40 m diameter in the overburden and a rectangular cross-section of 2,75 m x 2,30 m in the carbon. The shaft Baamstraat was drilled to a total depth of 20,94 m and was used as ventilation shaft and as access to the exploitation on Laag Merl /7/. Within the overburden the shaft wall was made of concrete (thickness of 0,45 m). There are no details available about any shaft fittings /28/.

In this area the overburden has a thickness of 14,16 m. The ventilation shaft Baamstraat has 1 documented reject, which is the floor on a level of 104,56 m NAP /6//28/.

The shaft was backfilled from above ground with approximately 108 m³ tailings by pneumatic packing up to the surface. Taking the shaft as stating point the east- and southwards tailing drifts of Laag Merl were backfilled with waste rock by pneumatic packing /7//28/. In 1978 the shaft was closed with a shaft covering (ø 4 m) /10//29/. In 1979 the terrain around the shaft was heaped up with waste rock for approximately 7,5 m. At the same time two 3 m rings (ø 2,8 m) were placed up on the shaft covering and backfilled to the top edge with sand. The rings were then closed with a covering of a thickness of 0,4 m. Today this covering is positioned 0,5 m below the surface /29//60//61/.

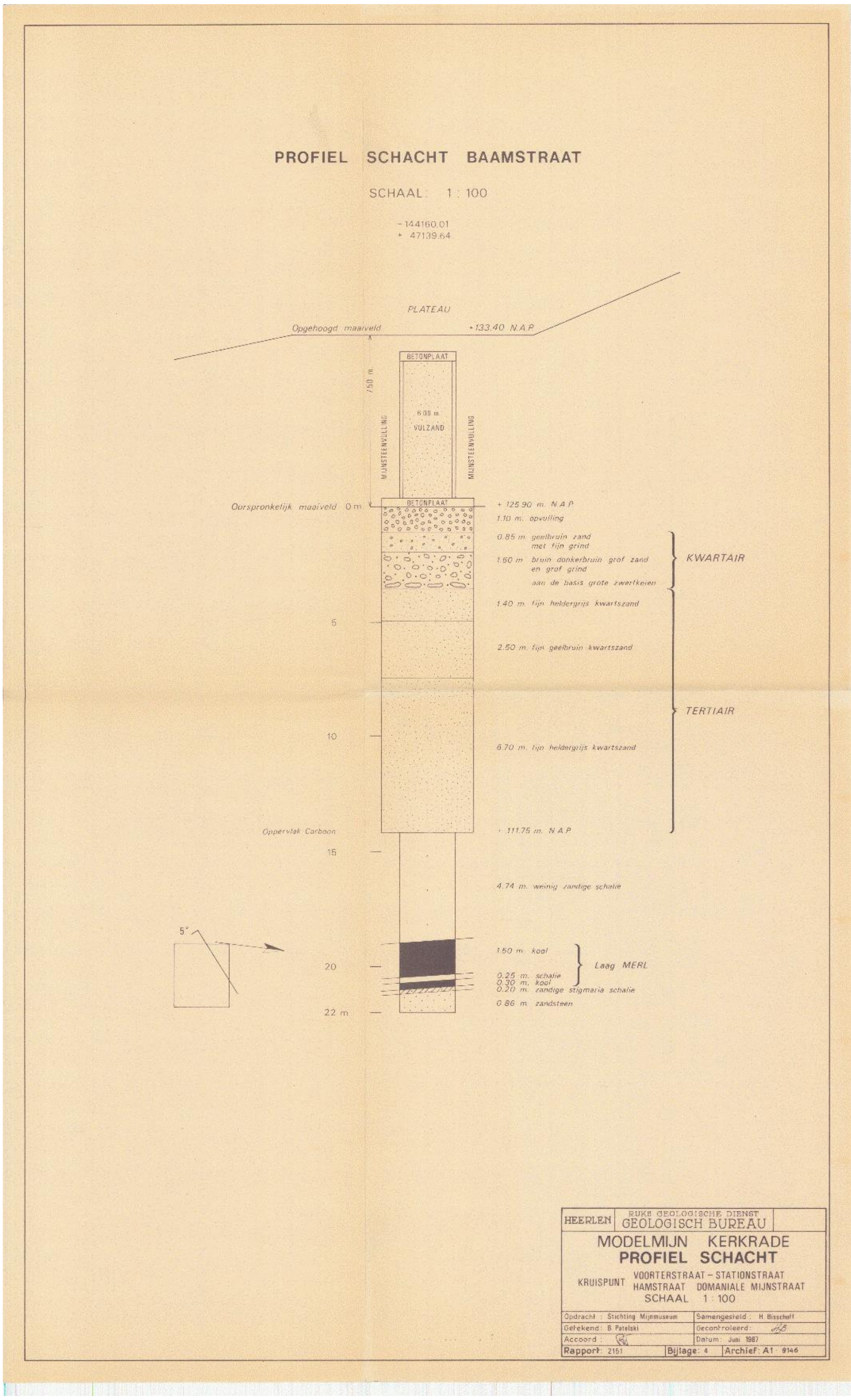


Fig. 13: Shaft profile shaft Baamstraat, status June 1987 /60/

The coordinates of the Shaft Baamstraat are:

RD-x:	202140
RD-y:	318840
elevation:	+132 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on an open space southwest of the roundabout of the roads Hamstraat and Voorterstraat (community Kerkrade). The shaft is marked by a sign.

2.8 Neuland

The vertical Shaft Neuland was drilled in 1828. In 1920 this shaft was backfilled and closed. According to documents available the shaft consists of two round double cylinders with a diameter of 1,60 m each. Both cylinders are separated from each other by masonry (thickness of 0,25 m). The shaft Neuland was drilled to a total depth of 189,64 m. One cylinder was used as travelling compartment, the other was used as drawing compartment. The shaft wall was made of masonry (thickness 0,5 m). There are no details available about any shaft fittings /1/ /2/ /3/ /64/.

In this area the overburden has a thickness of 39,50 m. The overburden has a stratification of 1,50 m topsoil, 6,50 m silt, 7,50 m sand mixed with gravel, 2,50 m of a rock water layer, 1,5 m clay and 11,50 m white sand /64/.

The shaft Neuland has 8 documented insets. The 60 m floor, as the topmost is located in a level of +92,80 m NAP and in a depth of 63 m /62/.

In the year 1919 in a depth of 85,0 m there were installed archs made of concrete (thickness of 0,75 m) in both cylinders. Used as abutment steel beams NP 30 were embedded. Afterwards the shaft was backfilled with debris. In 1980 the shaft was provided with a covering (thickness 0,5 m) on surface level /1//64/.

The coordinates of the Shaft Neuland are:

RD-x:	203101
RD-y:	318915
Elevation :	+164 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the property Grauweck 34 (community Kerkrade).

2.9 Louise

The vertical Shaft Louise was drilled in 1856. In 1907 this shaft was backfilled and closed /12/. According to documents available the shaft has an oval cross-section of 4,0 m x 3,30 m. The geological cross section of the Geological Bureau gives evidence of a total depth of 241,50 m for the shaft /65/.

In April 1907 the shaft was sounded with a total depth of 54 m and a groundwater level at a depth of 40 m. Replicate measurements in August 1972 only showed a changing groundwater level (38,50 m). At a depth of 15 m there could be detected water inflow at the shaft wall. Right below the shaft covering a pipeline (thickness 0,2 m) ends /65/. The shaft wall was made of masonry (thickness 0,5 m) /66/. The document 65 gives hints that the shaft Louise never had access to the mine workings /66/.

In this area the overburden has a thickness of 40,0 m /63/. The following figures give an overview of two shaft profiles.

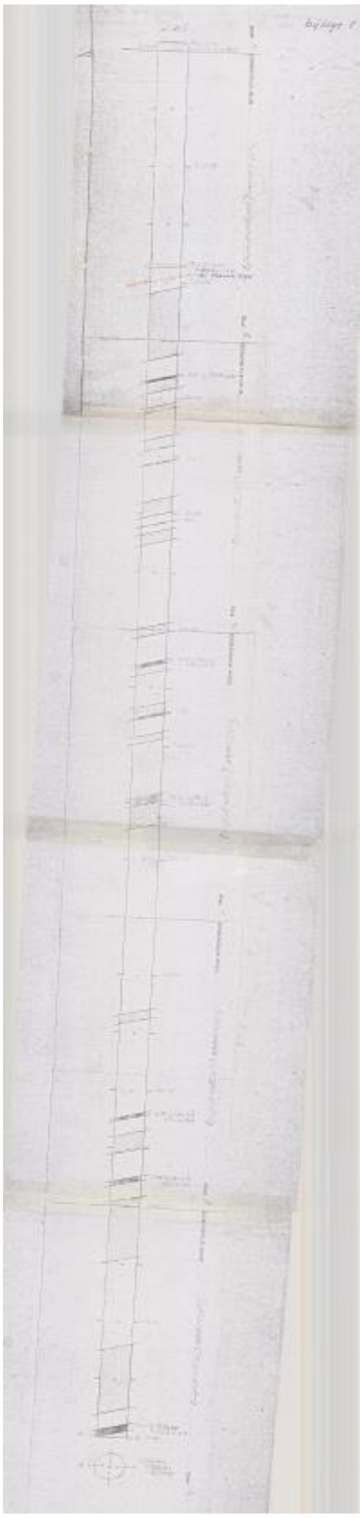


Fig. 14: Profile shaft Louise total depth of 241,50 m /65/

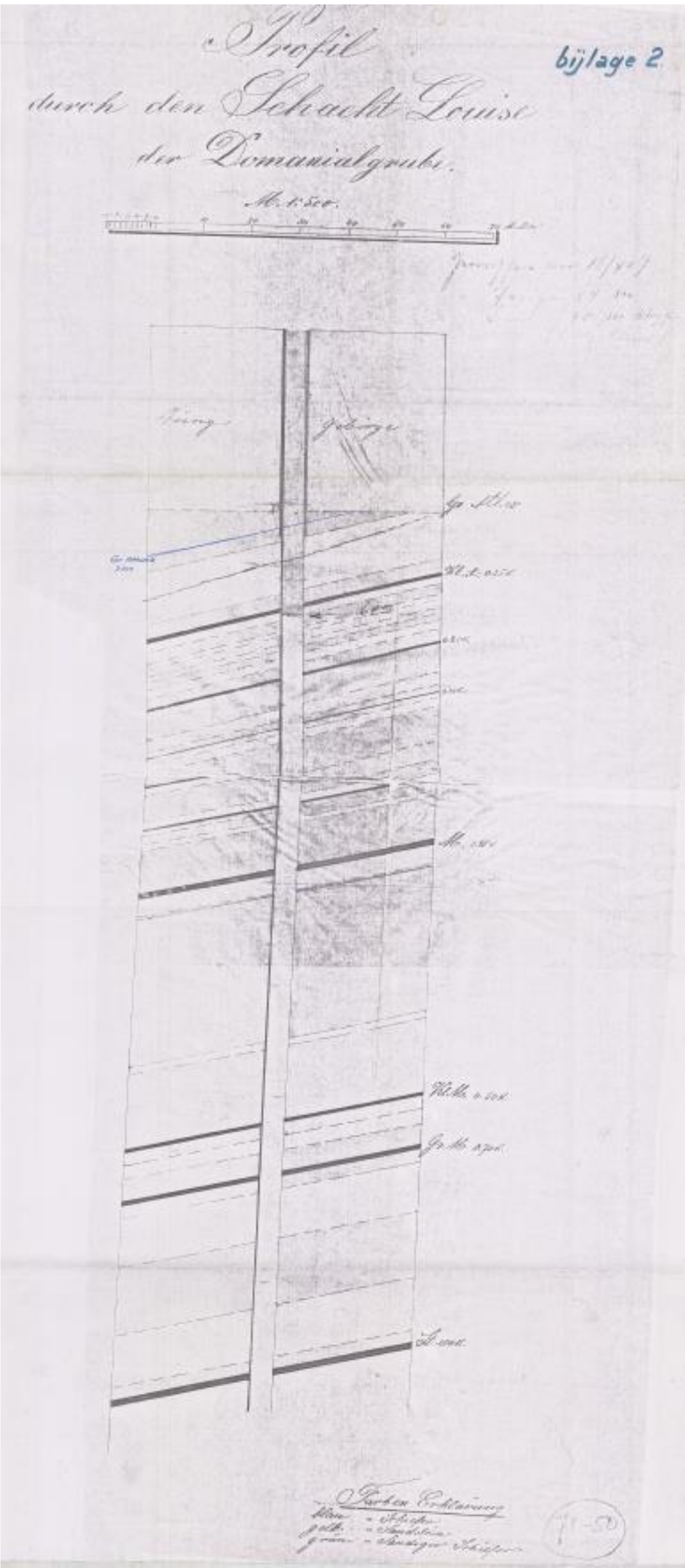


Fig. 15: Profile shaft Louise /65/

Afterwards the shaft was backfilled with debris. In 1980 the shaft was provided with a covering (thickness 0,5 m) on surface level /1//64/.

1972 the shaft was backfilled through the covering with debris (approx. 75,0 m) up to 44 m below the land surface. Between 44 m and 36 m depth (4 m below and 4 m above the carbon line) a load bearing filling consisting of 70 m^3 concrete of a thickness of 8 m and a quality of K 300 was backfilled through a drop pipe /63/. Following up to 5,5 m below the covering the shaft was backfilled with 309 m^3 waste rock. Between 5,5 m and 2 m up to the lower edge of the covering the shaft was backfilled with concrete (K 300) and finally was topped up with topsoil up to the land surface /12//63/.

The figure below shows the implementation planning of the back stowing of shaft Louise.

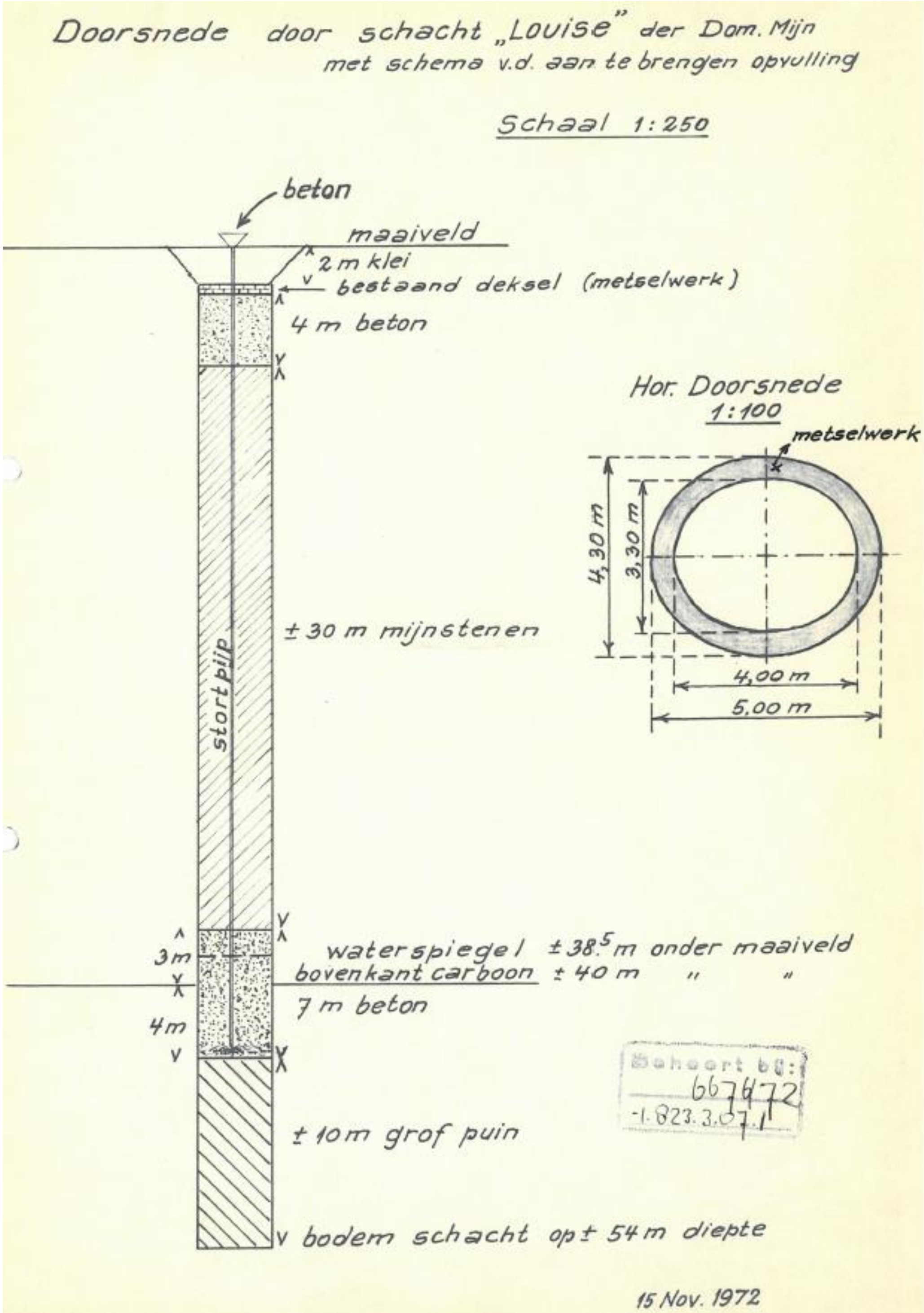


Fig. 16: Section drawing shaft Louise with implementation planning /63/

2002 within a construction project 0,5 m of the shaft covering (thickness 2,0 m) had to be scraped off. Upon the covering a layer of sand was applied and foundations were embedded. The load bearing of the beams takes place not via the covering but via bored piles /66/.

The coordinates of the Shaft Louise are:

RD-x:	203226
(RD-y:	319328
elevation:	+162 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located directly underneath the duplex house Johan Scholtesstraat 14-16 (community Kerkrade).

3 Neu Prick

3.1 Catharina

The vertical Shaft Catharina of the pit Neu Prick was drilled in 1838. In 1904 this shaft was backfilled and closed. According to documents available the shaft has a rectangular cross-section of 2,0 m x 3,0 m. The shaft Catharina was drilled to a total depth of 266 m and was used as drawing shaft. Extending over the overburden the shaft lining was made of masonry. Over the range of the carbon the shaft lining was made of wood /67/. There are no details available about any shaft fittings

In this area the overburden has a thickness of 41,0 m. In 1995 drilling results close to the shaft Catharina showed a strata sequence as followed: loess, gray clay, sand, gravel, grey sand, green clay and gravel with grey sand /67/. The following figure shows the rock mass composition close to the shaft Catharina /67/.

The shaft Catharina has 4 documented insets in the depth of 210,0 m and upon the 270 m floor (connection to the german pit Voccart upon the 218 m floor).

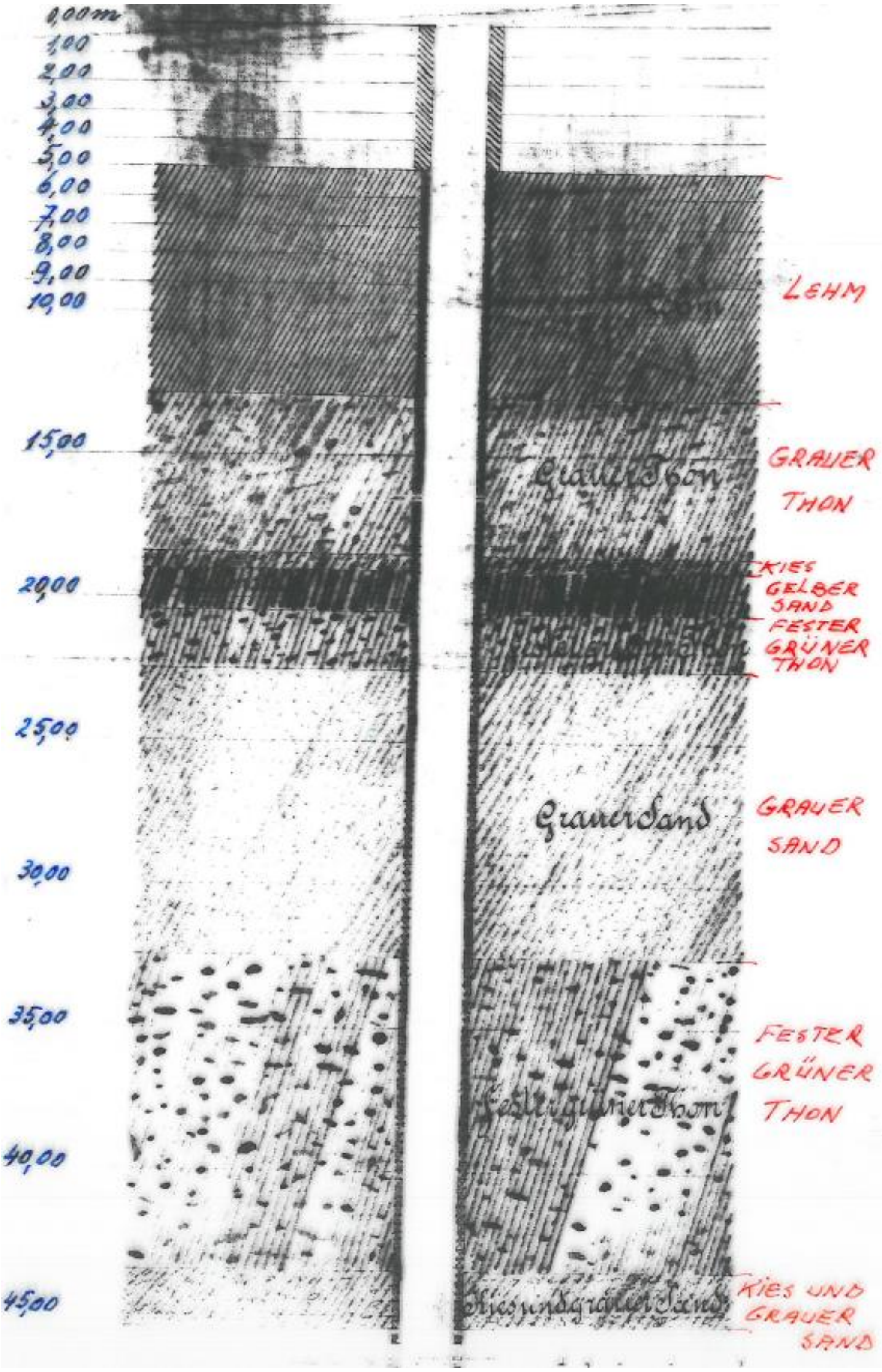


Fig. 17: Composition of overburden in the range of shaft Catharina /67/

In 1904 the shaft was backfilled from the 270 m floor up to the ground surface with soil and waste rock material /67/.

The rising mine water within the South-Limburg mining area was presumed to cause a potential danger of subsidence to the old shaft; therefore in 1996 the shaft Catharina was explored, analyzed and secured.

The concept to secure the shaft was a partial stabilization of the shaft column. Therefore the injection drill-holes were brought down to the depth of 90 m within the cross-section of the shaft.

Overall 195 t of blast furnace cement (HOZ35/PZ45F) were injected up to 5 m below the ground surface. Furthermore while drilling the injection drill-holes the loss of circulation required 45 t of insulating material. Overall 165 m³ material were injected /67/. This shows a stable backfilling of the shaft column by a successfully implemented injection. By means of the partial stabilization the shaft column can be seen as self-supporting.

Because no analysis of the load bearing capacity could be provided, at a depth of 45 m, 65 m and 85 m three extensometers (System GLÖTZL Typ GKSE 16) were embedded in the shaft filling. Furthermore a core drilling of a length of 90 m was brought down to verify the results /67/.

The coordinates of the shaft Catharina are:

RD-x:	203033
RD-y:	318726
elevation:	+168 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space at the Pricksteenweg (community Kerkrade).

4 Willem Sophia

4.1 Willem I

The vertical shaft Willem I of the pit Willem Sophia was drilled in 1900. In 1970 this shaft was backfilled and closed /30/. Within the overburden the shaft has a tubbing support. Below this the shaft wall was made of masonry (thickness 0,3 m) /4/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 60,0 m /50/. The shaft Willem I has 13 documented insets /50/. The 180 m floor, as the topmost is located in a level of approximately-23 m NAP and in a depth of 180 m /6/.

In the year 1969 the shaft was closed on the 180 m floor (-23 m NAP) using 100 m³ of a mixture of concrete as load bearing filling. Within the concept to secure the shaft diameter of 3,5 m as well as the shearing strain of 2 kg/cm² and a total length of 13 m for the load bearing filling made the use of armor within the filling unnecessary. Thereby the roughness of the shaft wall (masonry), the load bearing capacity of embedded beams as well as the fact that the filling could rest on one side of the floor, was not taken into account. The concrete was backfilled in two steps by the use of drop pipes. Afterwards the shaft was backfilled with approximately 1.360 m³ fine caving material by hydraulic stowing up to 4 m below the ground surface. Finally the shaft was provided with a concrete covering (thickness 3,5 m) with cast steel beams and an opening for refilling (ø 500 mm). By the end of 1970 the shaft column suffered a subsidence of 2,26 m /10//50/.

A static calculation is available /31/. The following figures show the calculations within the implementation planning.

Kleefprop op 180 m.v. Schacht Willem I

Diameter schacht 3.50 m.
Belasting op de kleefprop. (lengte prop 10 m)

1. t.g.v. waterkolom.
   $167 \times \frac{1}{4} \pi D^2 \times 1.$ = 1620 ton
2. t.g.v. stenen
   $20 \times 3.50 \times \frac{1}{4} \pi D^2 \times 0.8$ = 545 ton
3. t.g.v. prop (eig. gew.)
   $10 \times \frac{1}{4} \pi D^2 \times 2.4$ = 305 ton

2470 ton

Omtrek van de prop.
$10 \times \pi . D = 110 \ m^2$

Dit betekent een schuifspanning tussen prop en schachtwand ter grootte van

$$\tau = \frac{2470}{110} = 22.5 \ t/m^2 = 2{,}25 \ kg/cm^2$$

Uitvoering

Het betonneren vindt plaats in 2 etappes.
Eerst tot een hoogte van ca. 2.50 (even boven dak verdieping) en vervolgens de overige 10,5 m.

Gewicht van de eerste 2.50 m. is 6000 kg/m².
Overspanning 3.50 m.
Liggers h.o.h. 40 cm.

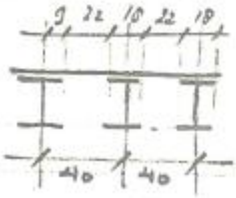
Moment per ligger

$M = \frac{1}{8} \times 0,40 \times 6000 \times 3.50^2 = 3700$ kgm.

Benodigd weerstandsmoment

$W = \frac{370000}{1400} = 265$ cm³

Profiel HE-A (DIE) 180
HE-B (DIN) 160



Profielen af te dekken met een stalen plaat 6/8 mm

$M = \frac{1}{10} \times 6000 \times 0,25^2 = 37,5$ kgm

$W = \frac{3750}{1400} = 2,6$ cm³

Plaat 6/8: $W = \frac{1}{6} \times 100 \times 0,6^2 = 6$ cm³

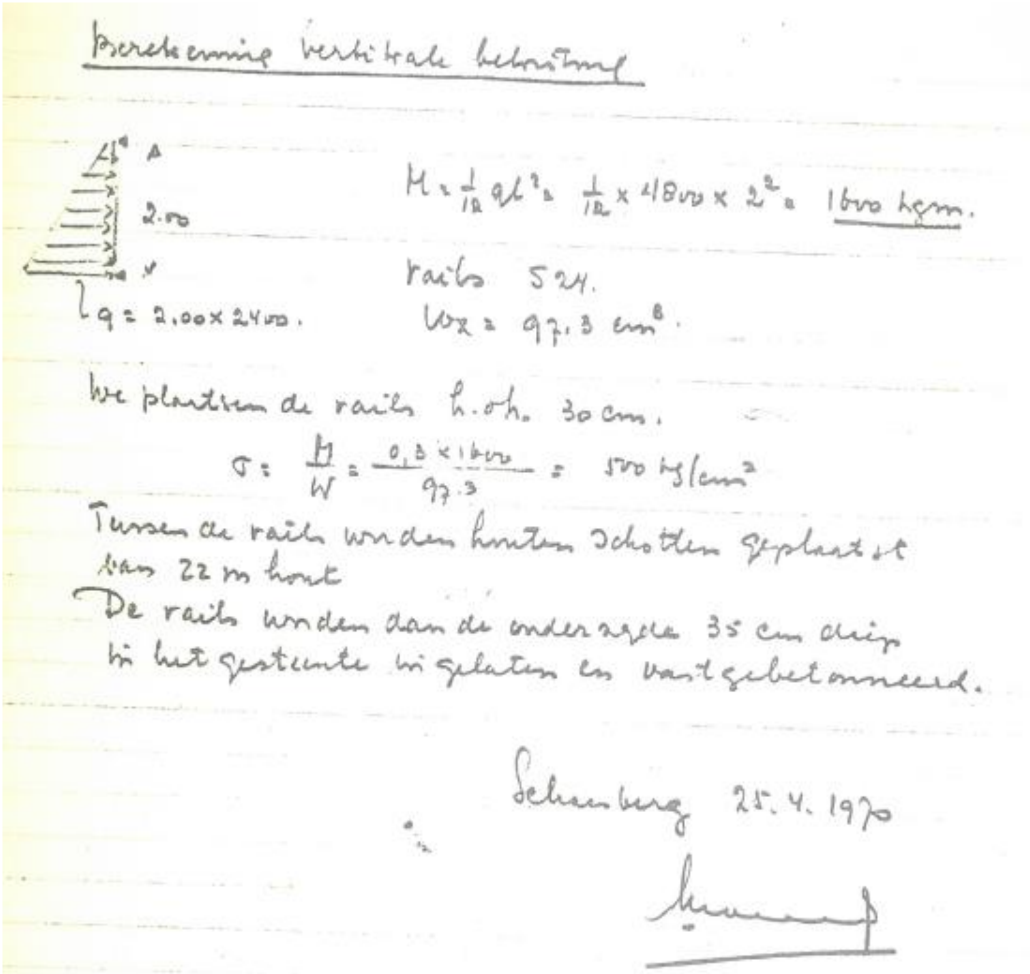


Fig. 18: Static calculation shaft Willem I /31/

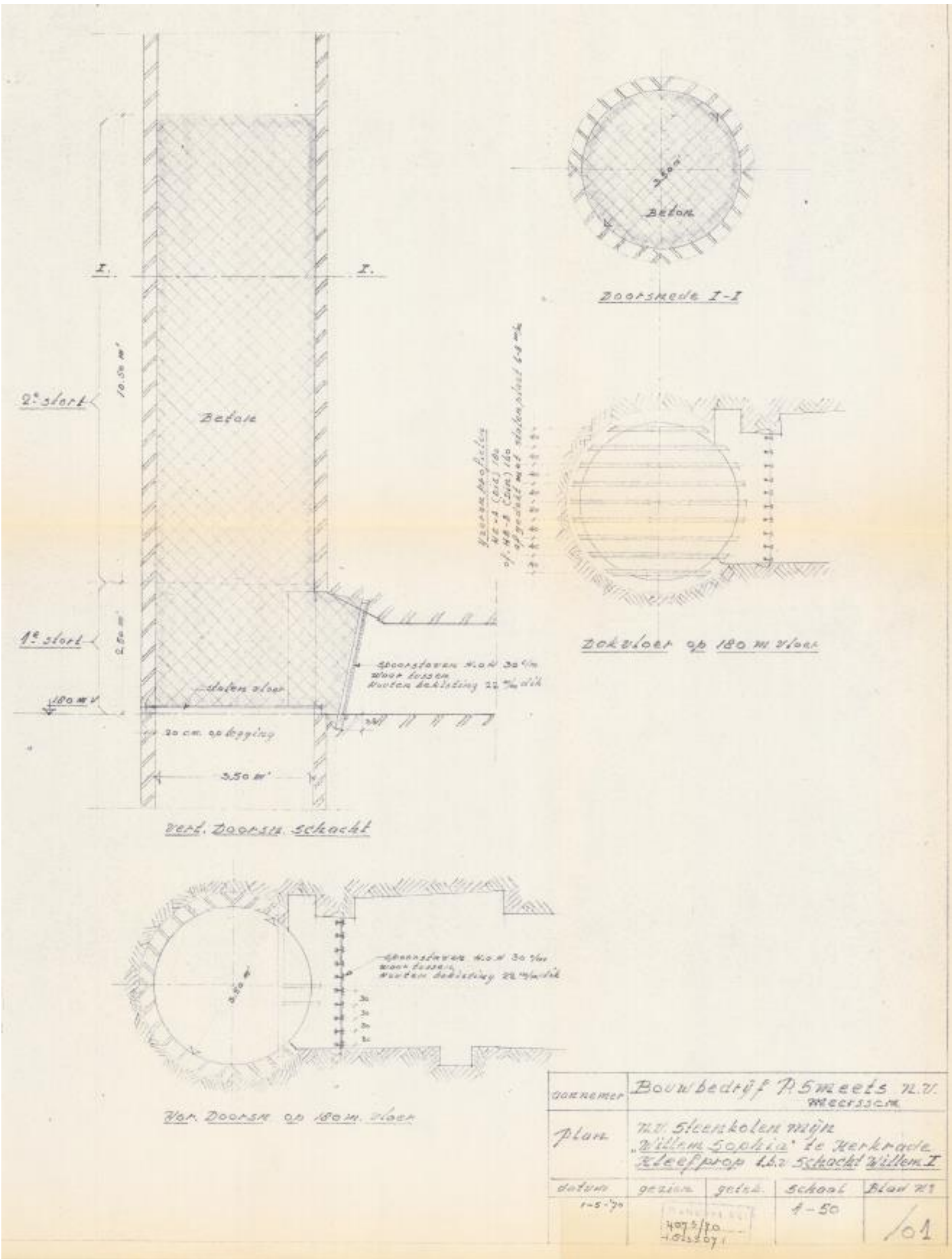


Fig. 19: Implementation planning load bearing filling shaft Willem I /31/

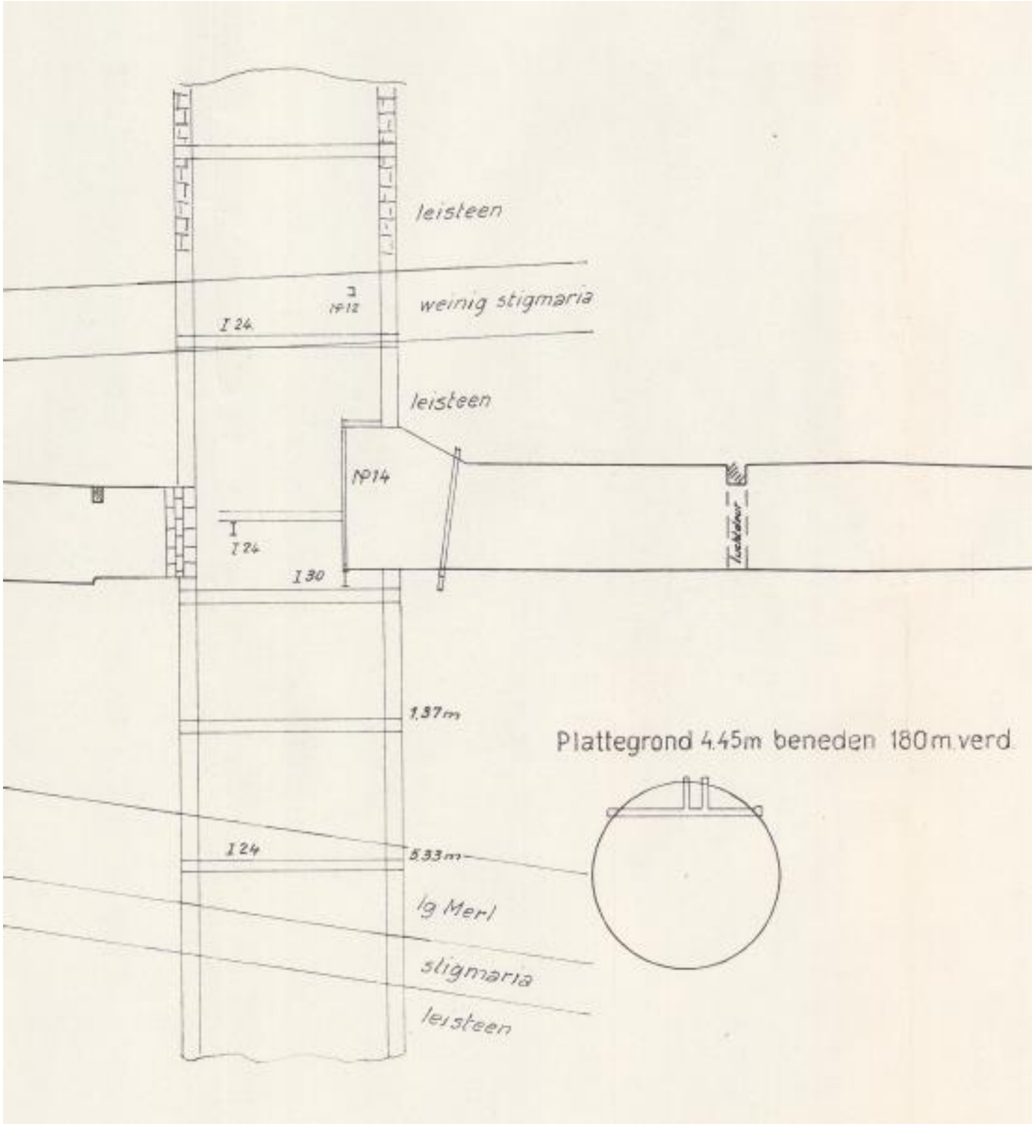


Fig. 20: Profile in the range of the 180 m floor with strata sequence /31/

Upon the 180 m floor mainly slate („leisteen“) as well as Laag Merl were found /31/.

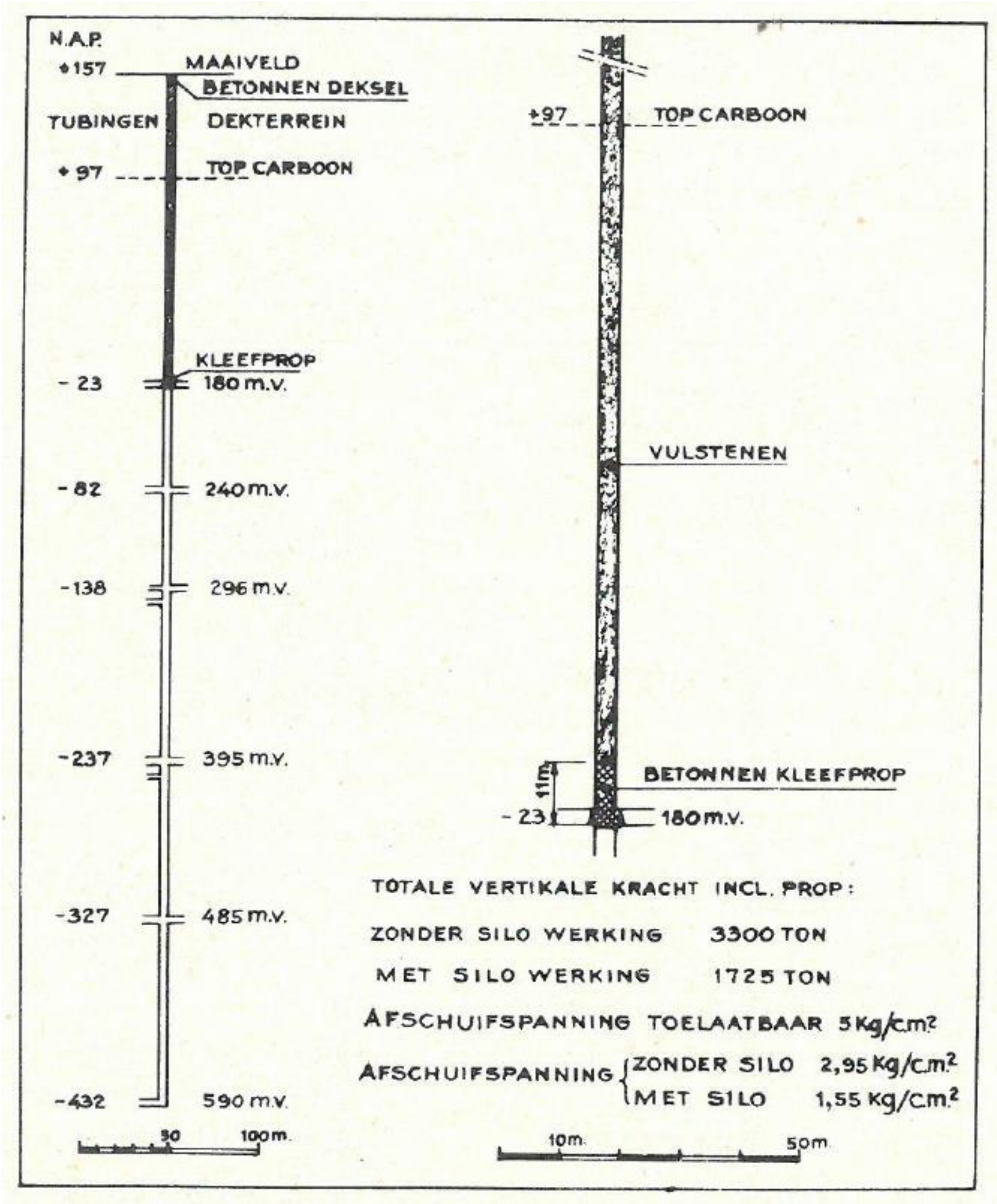


Fig. 21: Securing shaft Willem I /50/

The coordinates of the Shaft Willem I are:

RD-x:	200384
RD-y:	318635
elevation:	+158 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located southwards Industriestraat (community Kerkrade) and is used as soccer field of the sport club FC Kerkrade-West.

4.2 Willem II

The vertical shaft Willem II of the pit Willem Sophia was drilled in 1900. In 1970 this shaft was backfilled and closed /30/. According to documents available the shaft has a round cross-section of 3,60 m diameter. The shaft Willem II was drilled to a total depth of 651 m and was used as travelling, drawing and ventilation shaft /17/. Within the overburden the shaft has a tubbing support /50/. Below this the shaft wall was made of concrete (thickness 0,5 m) /4/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 60,0 m. The shaft Willem II has 15 documented insets. The 105 m floor, as the topmost is located in a level of +52,40 m NAP and in a depth of 105 m /6/.

In 1970 the shaft was closed on the 105 m floor using 80 m³ of a mixture of concrete as load bearing filling. The total length of the filling is 19 m. The seal rests on two sides of the shaft landing on the 105 m floor. The concrete was backfilled in three steps. Afterwards the shaft was backfilled with approximately 800 m³ fine caving material (<60 mm). Finally the shaft was provided with a concrete covering (thickness 3,5 m) with cast steel beams and an opening for refilling (ø 500 mm). By Sept. 7th 1970 the shaft column suffered a subsidence of 5,63 m /10//50/ and by the end of the year additionally 0,4 m. The shaft was provided with a line for compressed-air which was left behind for the purpose of controlling the mine water; it runs through the load bearing filling /10//50/.

By the end of 1973 the line for compressed-air was backfilled with 2 l gravel (5-12 mm), 2 l river sand, 10 l cement suspension, 60 l cement mortar und sand /33/.

The following figures show the implementation plannings of the shaft barrier for the shaft Willem II.

Afsluiting van de schacht Willem II
NV. Steenkolenmijnen Willem Sophia

De afsluiting van de schacht Willem II
zal plaats vinden door middel van een
kleefprop op de 105 m verdieping

De schachtdiameter bedraagt 3.60 m.
De lengte van de prop wordt 10 m

De belasting op de prop wordt
a) tgv. waterkolom
$(105-15) \times 1/4 \pi D^2 \times 1,- = 920$ ton
b) tgv. stenen
$20 \times 1/4 \pi D^2 \times (2,0 - 1,0) = 205$ ton
↑ moet zijn 1,2.
c) tgv. prop
$10 \times 1/4 \pi D^2 \times 2.4 = 245$ ton
1370 ton

Omtrek van de prop =
$10, \times \pi D = 113\ m^2$

Tussen schachtwand en prop treedt dus een
max schuifspanning op ter grootte van
$\tau = \frac{1370}{113} = 12\ t/m^2 = 1,2\ kg/cm^2$

Uitvoering

Het betonneren vindt plaats in drie etappes
Eerst over de kelderhoogte, vervolgens tot
onderkant dak verdieping en vervolgens
de prop van 10 m in de schacht.

De hoogte van het eerste stort is circa 4.00 m
Dit betekent een belasting van 9600 kg/m².

In de kelder wordt de schacht dichtgelegd
met stalen balken, waarop een stalen plaat
dikte 8 mm.
De balken wordt 30 cm h.o.h gelegd.

Moment per ligger

$M = \frac{1}{8} q l^2 = \frac{1}{8} \times 0{,}3 \times 9600 \times 3.60^2 = 4650$ kgm

$\bar{\sigma} = 1600$ kg/cm

Benodigd weerstands moment

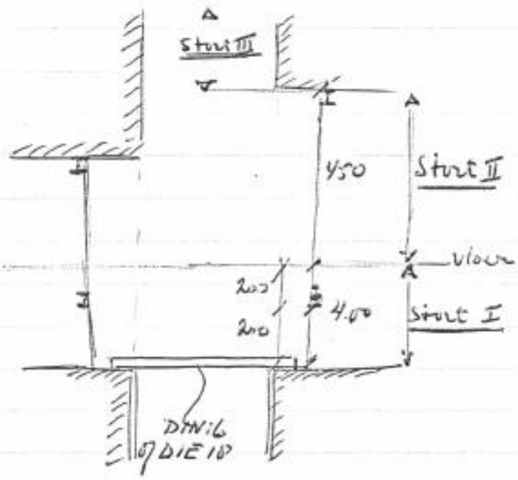
$W = \frac{465000}{1600} = 290 \text{ cm}^3$

Gekozen profiel HE 180 A ($W_x = 303 \text{ cm}^3$)
of HE 160 B ($W_x = 323 \text{ cm}^3$)

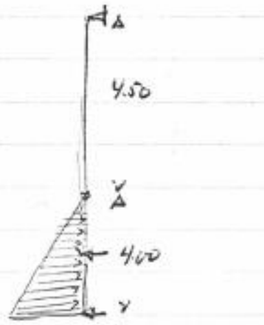
(HE-180 A = DIE 18
HE-160 B = DAN 16)

De afdekplaat behoeft niet berekend te worden, daar hier slechts zeer geringe overspanningen optreden nl. 12-15 cm

Berekening verticale belasting



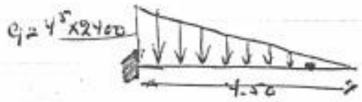
Situatie t.p.v. de 105 m verdieping



Belastingstoestand Stort I

Belastingstoestand Stort II

Voor berekening is belastingstoestand tijdens stort II maatgevend



Zie Stahl im Hochbau
Belastungstabellen

$M_{max} = \frac{Q\ell}{7.5} = \frac{4.5 \times 2400 \times 4.50}{7.5} = 6500$ kgm

Nemen rails S 24
$W_x = 97.3\ cm^3$.
We plaatsen de rails h.o.h. 20 cm

dan $\sigma = \frac{M}{W} = \frac{6500}{5 \times 97.3} = 1350\ kg/cm^2$.

Tussen de rails worden baddingen 6/16 geplaatst
De rails worden in de vloer en de kelder
Circa 20 cm diep ingelaten en gebetonneerd.

Schaesberg. 1 juni 1970.

[signature]

Fig. 22: Static calculation shaft Willem II /33/

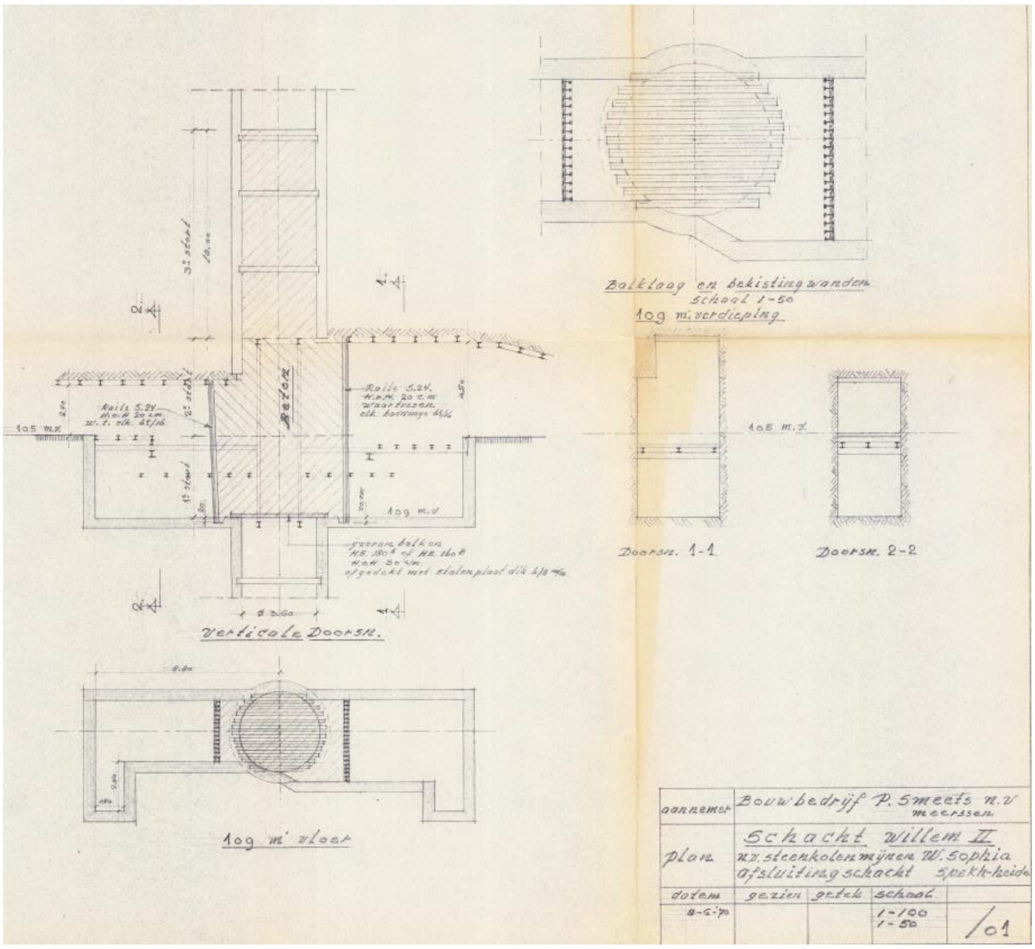


Fig. 23: Implementation planning load bearing filling shaft Willem II /33/

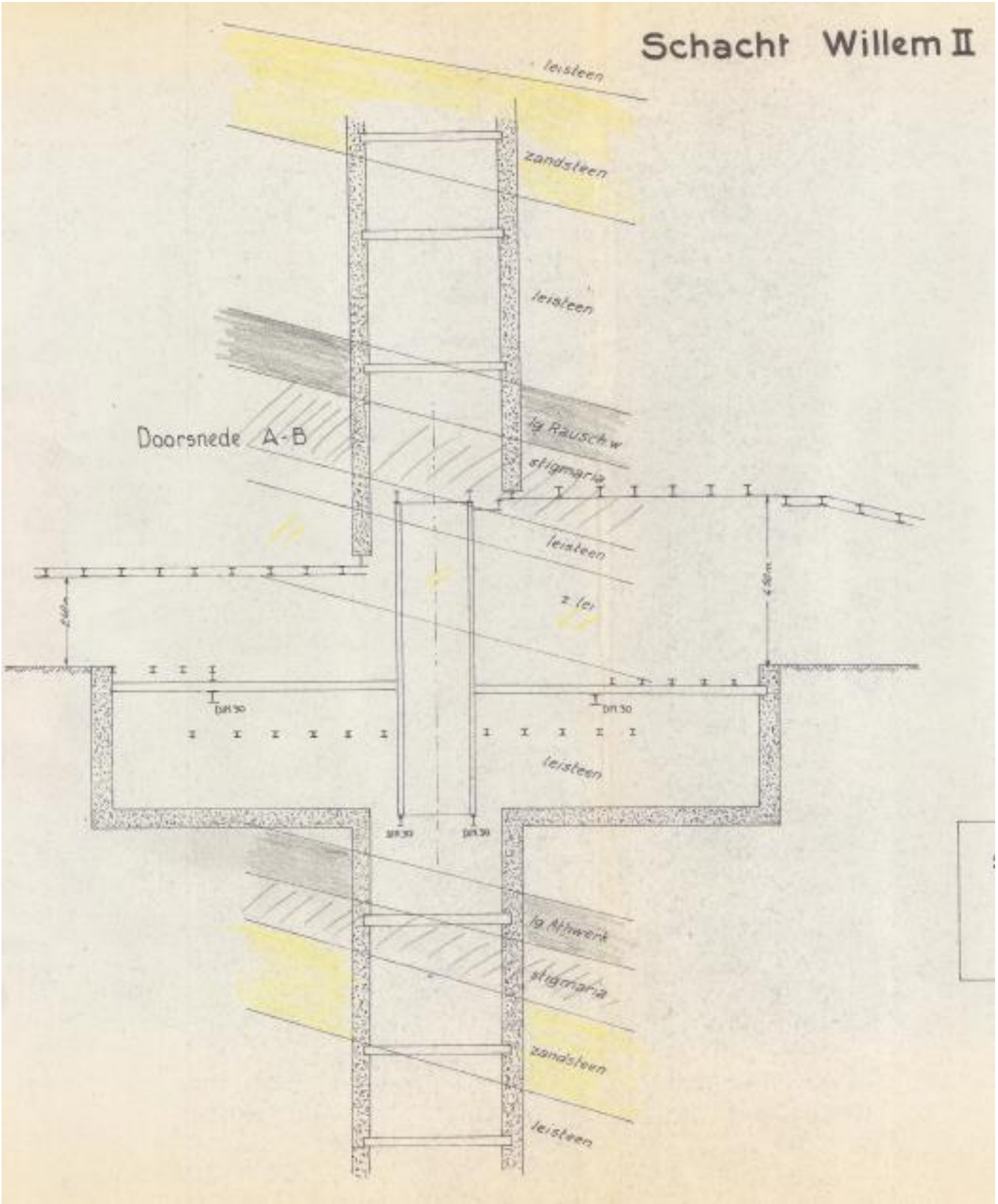


Fig. 24: Profile in the range of the 105 m floor /33/

In the range of the 105 m floor mainly slate and sandstone as well as Laag Rauschenwerk and Laag Athwerk were found /33/.

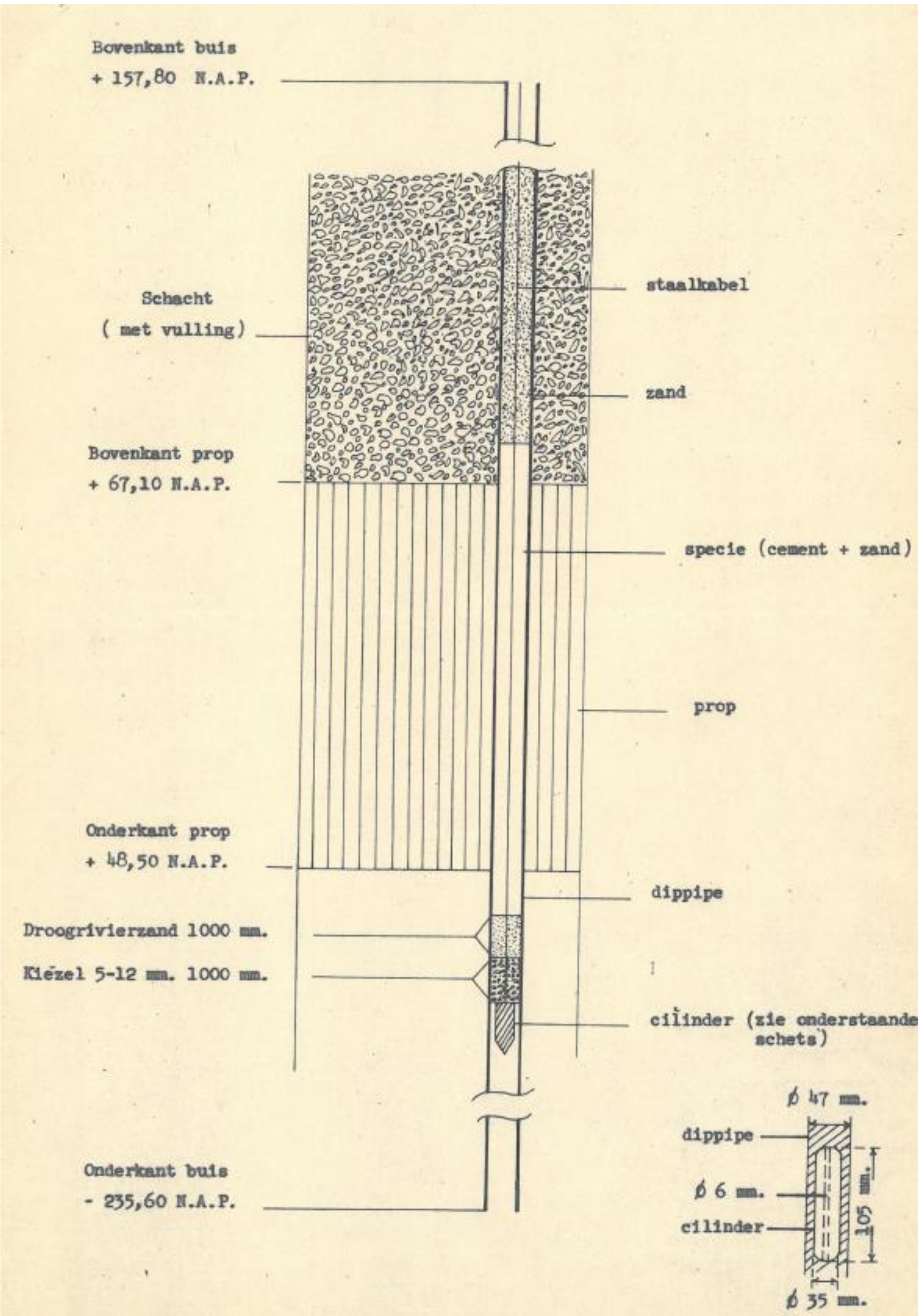


Fig. 25: Backfilling the line for compressed-air shaft Willem II /33/

The coordinates of the Shaft Willem II are:

RD-x:	200373
RD-y:	318668
elevation:	+158 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located southwards Industriestraat (community Kerkrade) and is used as soccer field of the sport club FC Kerkrade-West.

4.3 Sophia

The vertical shaft Sophia was drilled in 1949. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,50 m diameter. The shaft Sophia was drilled to a total depth of 328 m. From the overburden up to the carbon the shaft is structured as follows (from inside to the outside): masonry (0,665 m), bitumen joint (0,03 m), masonry (0,215 m) und concrete (0,19 m) /4//30/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 128,0 m.

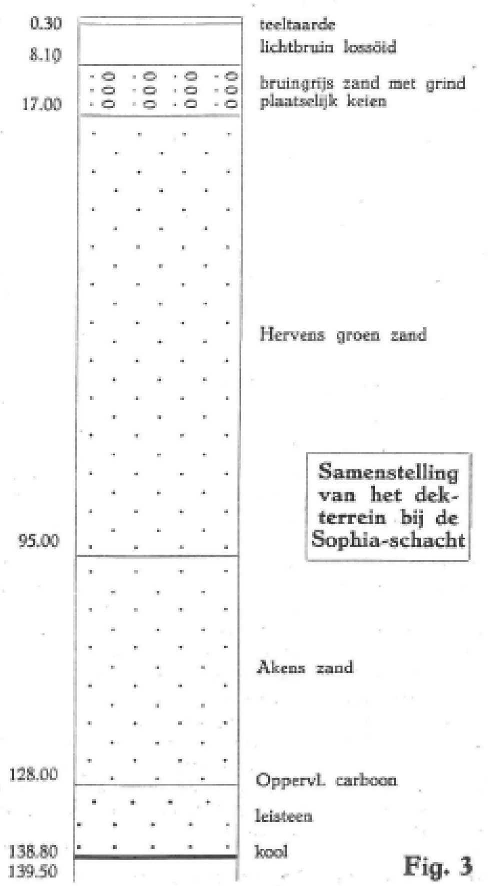


Fig. 26: Stratification overburden shaft Sophia /30/

The shaft Sophia has 8 documented insets. The 150 m floor, as the topmost is located in a level of +27,47 m NAP and in a depth of 149 m /6/.

In 1970 the shaft was closed on the 180 m floor with a load bearing filling of a length of 12 m consisting of 350 m³ of a mixture of concrete. The backfilling was conducted in three concreting sections. In the first on the floor level a platform consisting of iron beams was constructed. In the second step this platform was covered by a heavy reinforced concrete board which rests with its bend lower edge upon the surrounding rock to spread the pressure occurring from the load bearing filling and the backfilled loose material. As strengthening of the horizontal abutment underneath the 150 m floor there were additionally backfilled two meters of concrete. The back stowing was carried out in three steps by use of a drop pipe. After the ageing of the load bearing filling the open shaft column above was backfilled by hydraulic stowing with approximately 2100 m³ waste material and sand. Finally the shaft was provided with a concrete cover (thickness 3,5 m) with cast steel beams and an opening for refilling (ø 500 mm). By the end of 1970 the shaft column suffered subsidence of 5,10 m, /10//30/.

The figure below shows the shaft barrier of the shaft Sophia.

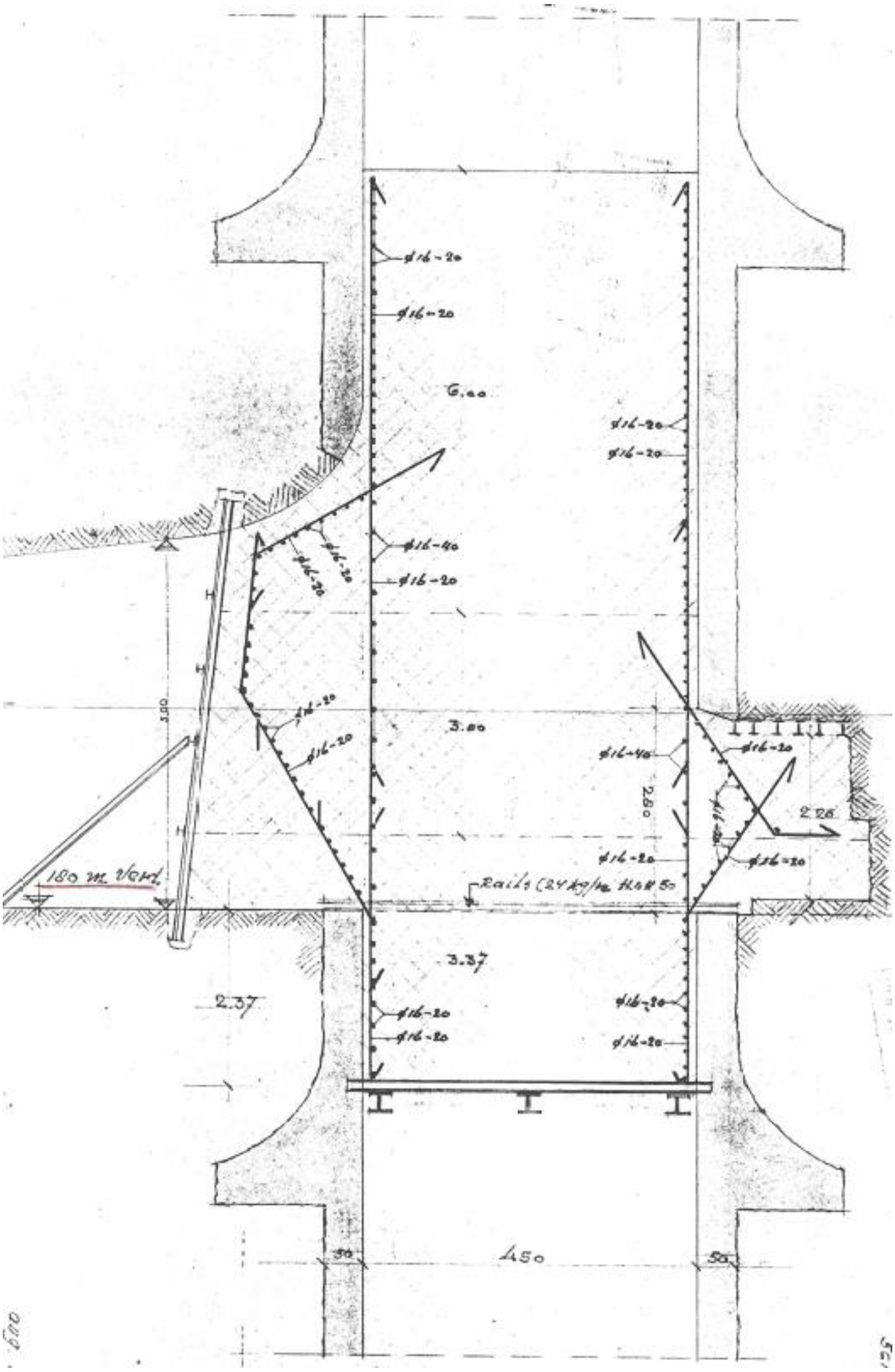


Fig. 27: Sectional drawing of the shaft barrier shaft Sophia /30/

The coordinates of the Shaft Sophia are:

RD-x:	199145
RD-y:	317044
elevation:	+176 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located southwestwards of a roundabout at the Avantisallee (community Heerlen). Directly westwards of the shaft an industrial estate was built.

4.4 HAM II

The vertical shaft HAM II was drilled in 1939. In 1970 this shaft was backfilled and closed /30/. According to documents available the shaft has a round cross-section of 4,8 m diameter. The shaft HAM II was drilled to a total depth of 32,0 m and was used as ventilation shaft /10/. A rise drift between the depth of 74 m and 32 m connected the shaft with the 70 m floor /33/. The shaft wall was made of concrete (thickness of 0,45 m) /4/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 21,0 m. The shaft HAM II has 2 documented insets /6/.

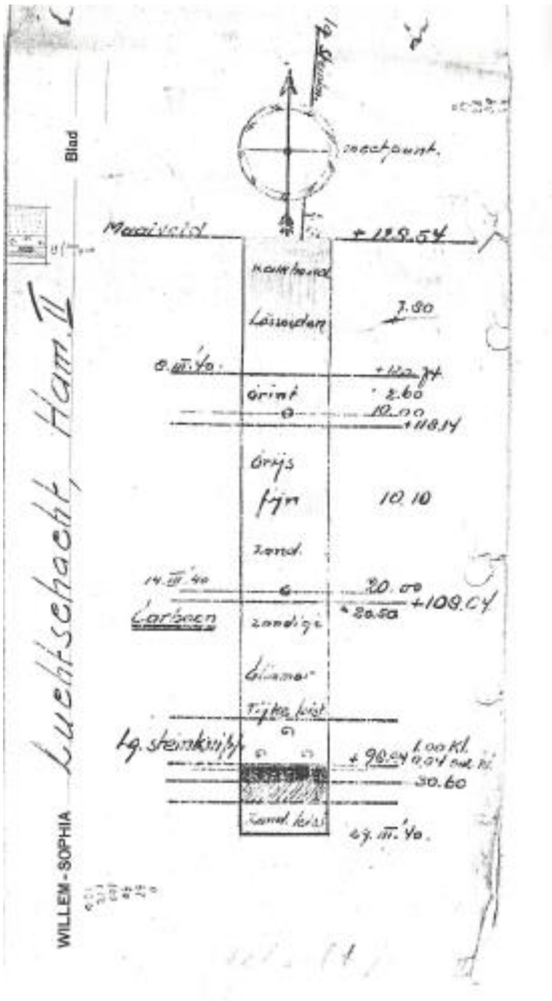


Fig. 28: Profile shaft HAM II /30/

In 1970 the shaft was backfilled with 575 m³ concrete up to one meter below the ground surface. The back stowing came to rest on an abutment of steel beams in a depth of 33,5 m (level of carbon). In a depth of 32 m the shaft has a smaller rectangular cross-section (rise drift). Above the shaft barrier clay was backfilled /10/. In the following figures the static calculation of the implementation planning of the load bearing filling is shown.

Afsluiting Hamschacht
NV. Steenkolenmijnen Willem Sophia

Deze schacht die dient doet als uittrekkend schachtje, kan op zeer eenvoudige wijze worden afgesloten

De aangewezen plaats hiervan is op circa 32 m minus maaiveld, waar het ronde schachtmetselwerk overgaat in opbraakramen.

Op de opbraakramen wordt een ondersteunings vloer gemaakt ~~[illegible]~~ teneinde het eigen gewicht van de gestorte beton te dragen.
De overspanning van de opbraak is 2.00 m.

Op afstanden van 40 cm hoh worden profielen DIN16 of DIE18 gelegd, waarop stalen plaat 6/8 mm.

Op deze vloer wordt een 8 m dikke kleiprop gestort.

Berekening overbalkje

$M = 0{,}4 \times \frac{1}{8} \times 8 \times 2400 \times 2.00^2 = 3840$ kgm

$W_{ben} = \frac{384000}{1400} = 270$ cm³.

Gekozen profiel is voldoende zwaar.

De berekening van de optredende schuifspanning tussen profiel en metselwerk laten we achterwege, daar deze zeer gering is (aanzienlijk kleiner dan 1 kg/cm²)

Schaesberg, 6 juni 1970

Fig. 29: Static calculation shaft HAM II /33/

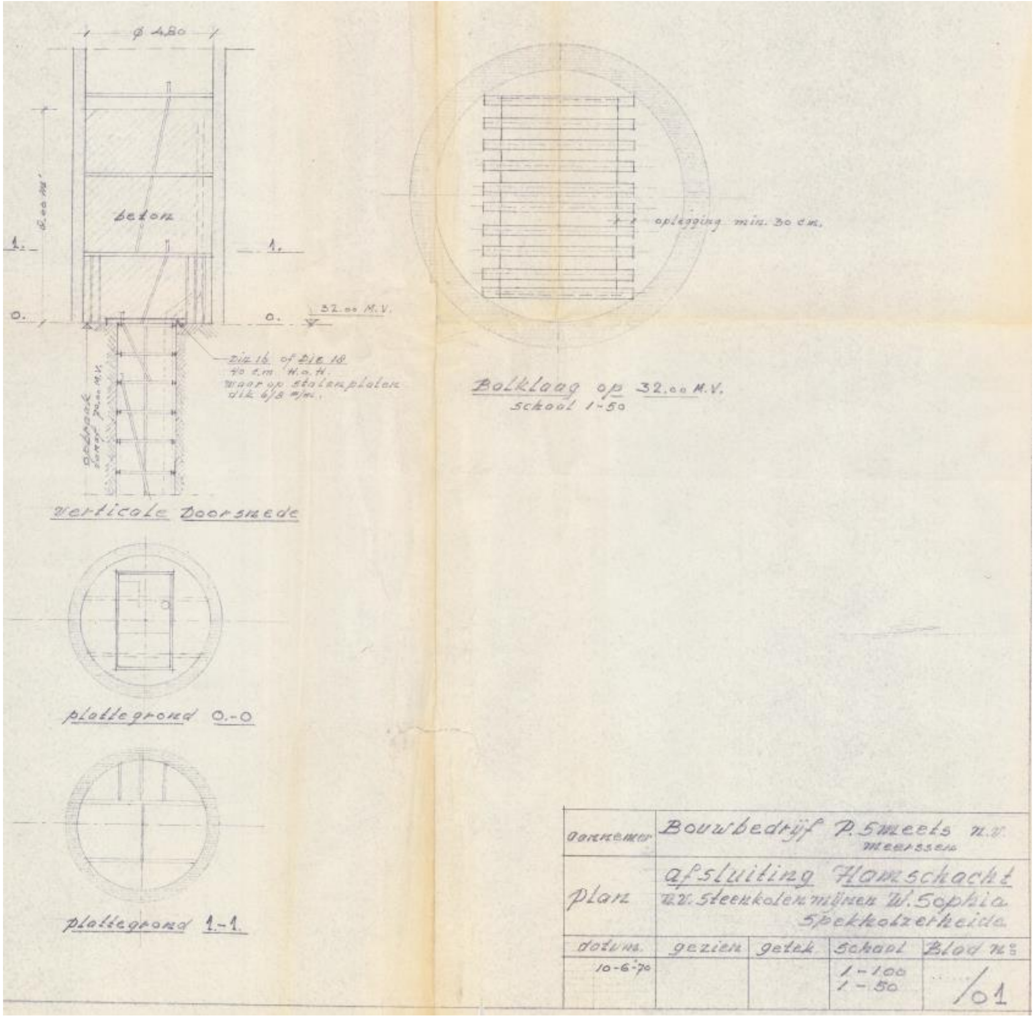


Fig. 30: Implementation planning load bearing filling shaft HAM II /33/

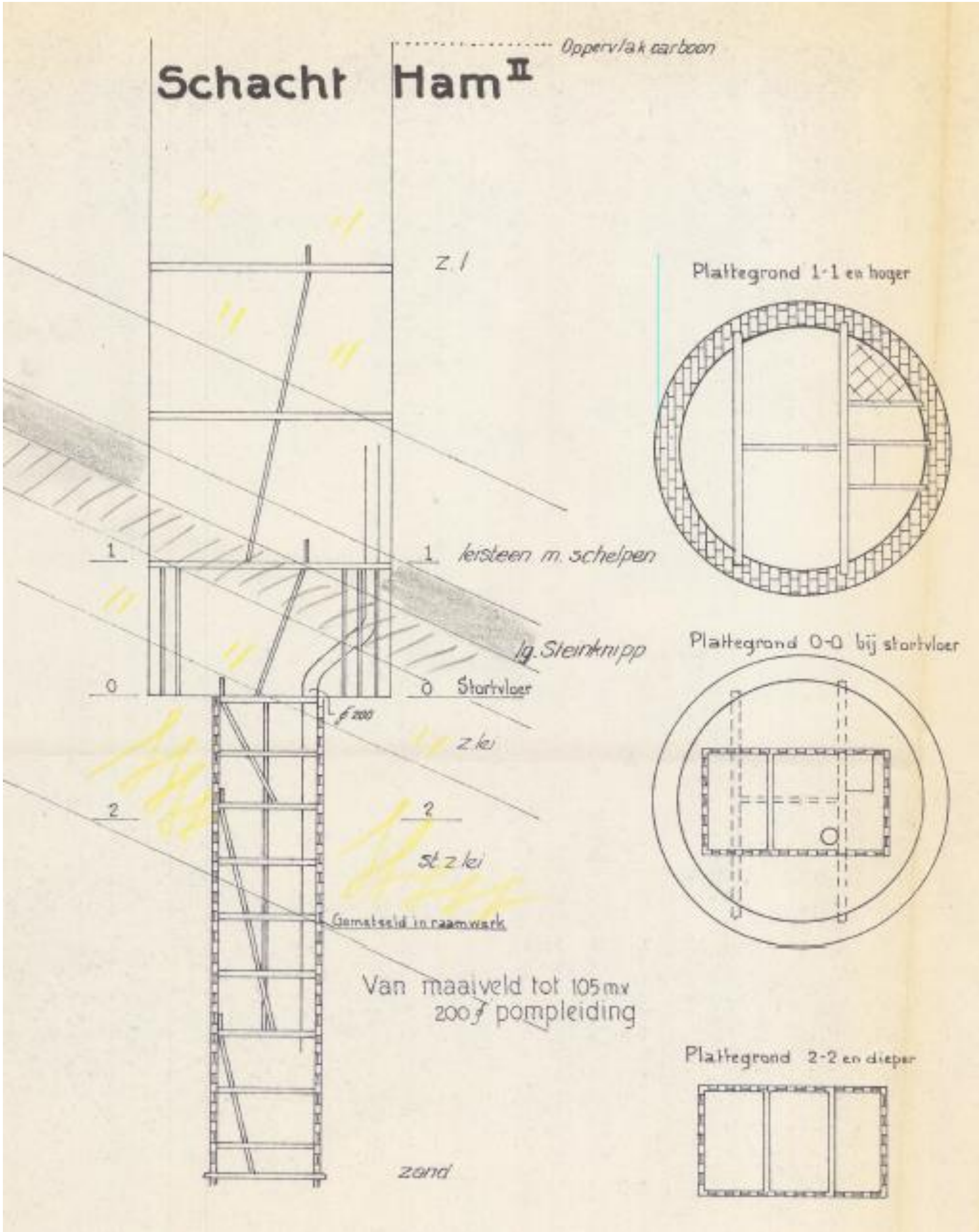


Fig. 31: Profile shaft HAM II /33/

Within the range of the barrier the following strata is occuring: slate, Laag Steinknipp and slate with shells /33/.

Fig. 32: Uncovered shaft head of HAM II on 14.04.2016

The coordinates of shaft HAM II could be determined by survey in April 2016; they are given by:

RD-x:	201746,00
RD-y:	319248,73
elevation:	+129 m NAP
positional accuracy:	+/- 0 m

The shaft is located in grazing land southwest of the cross-section Vauputsweg and Hammijnstraat (community Kerkrade) (c.f. Fig. 32).

4.5 Melanie

The vertical Shaft Melanie was drilled in 1955. In 1970 this shaft was backfilled and closed /30/. According to documents available the shaft has a round cross-section of 3,0 m diameter. The shaft Melanie was drilled to a total depth of 230,0 m and was used as equipment and ventilation shaft /30/. The shaft wall was made of concrete (thickness of 0,55 m) /4/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 65,30 m /30/. The shaft Melanie has 4 documented insets. The 100 m floor, as the topmost is located in a level of +53,27 m NAP and in a depth of 100 m /6/. The strata of the overburden consists is shown in the following figure.

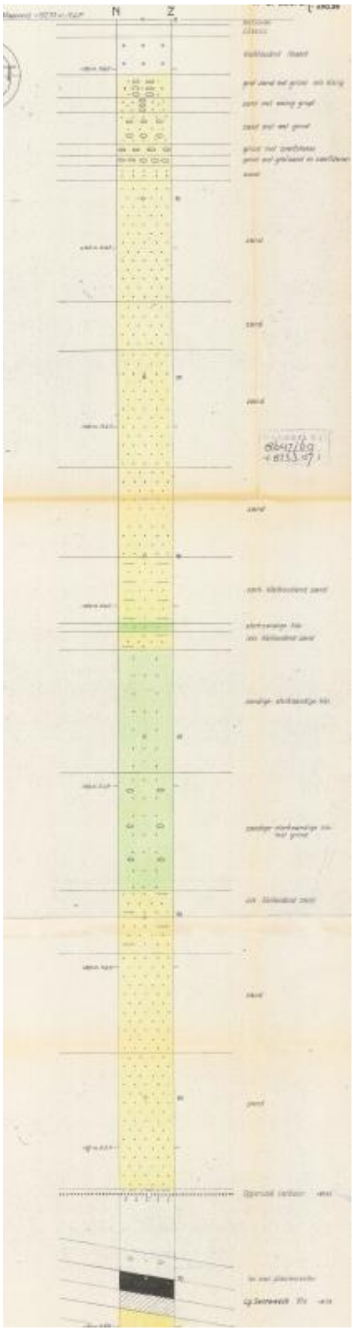


Fig. 33: Strata overburden shaft Melanie /51/

The following figure gives an overview of the rock layers in the range of the 100 m floor.

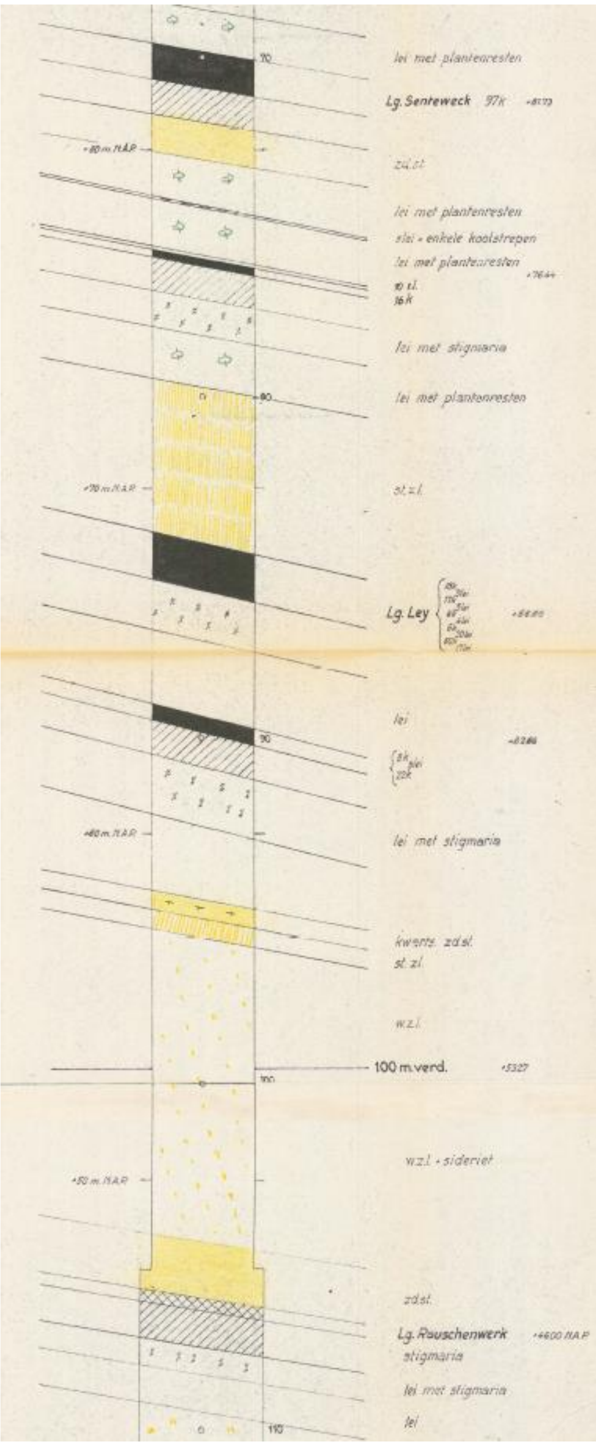


Fig. 34: Strata of the rock layer in the range of the 100 m floor /51/

In the range of the 100 m floor mainly slate (“leisteen”) and sandstone as well as Laag Rauschwerk are found /51/.

In 1970 the shaft was closed on the 100 m floor with a load bearing filling of a length of 25 m consisting of 330 m³ of a mixture of concrete. Hereby a connected waste material dugout was backfilled with concrete using drop pipes from above ground. Additionally the shaft column being still open was used as water reservoir. The shaft was secured with a grid consisting of steel beams upon the ground surface /10/.

In the following the shaft barrier of the shaft Melanie is shown.

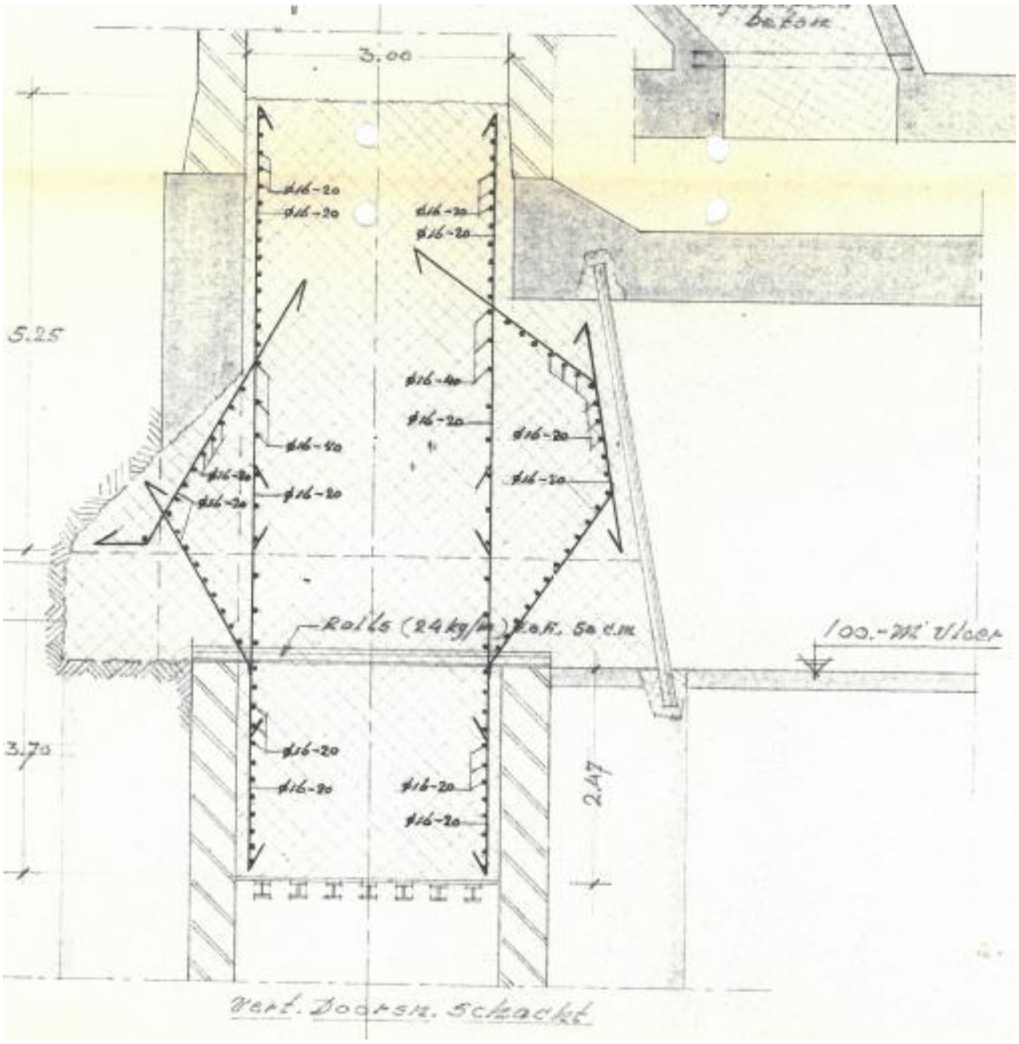


Fig. 35: Sectional drawing shaft barrier shaft Melanie /51/

The coordinates of the shaft Melanie are:

RD-x:	200515
RD-y:	318178
Elevation :	+153 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on an open space with trees, north of the Hamstraat(community Kerkrade)

5 Laura en Vereeniging

5.1 Laura I

The vertical Shaft Laura I was drilled in 1901. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,5 m diameter /4/. The shaft Laura I was drilled to a total depth of 730 m and was used as travelling, drawing and ventilation shaft. The shaft wall was made of masonry /4/. Within the overburden the shaft consists of tubbing support /50/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 99 m /50/. The shaft Laura I has 19 documented insets /50/. The 120 m floor, as the topmost is located in a level of -3,24 m NAP and in a depth of 119 m /6/.

In the following figure the strata in the range of shaft Laura I is pictured.

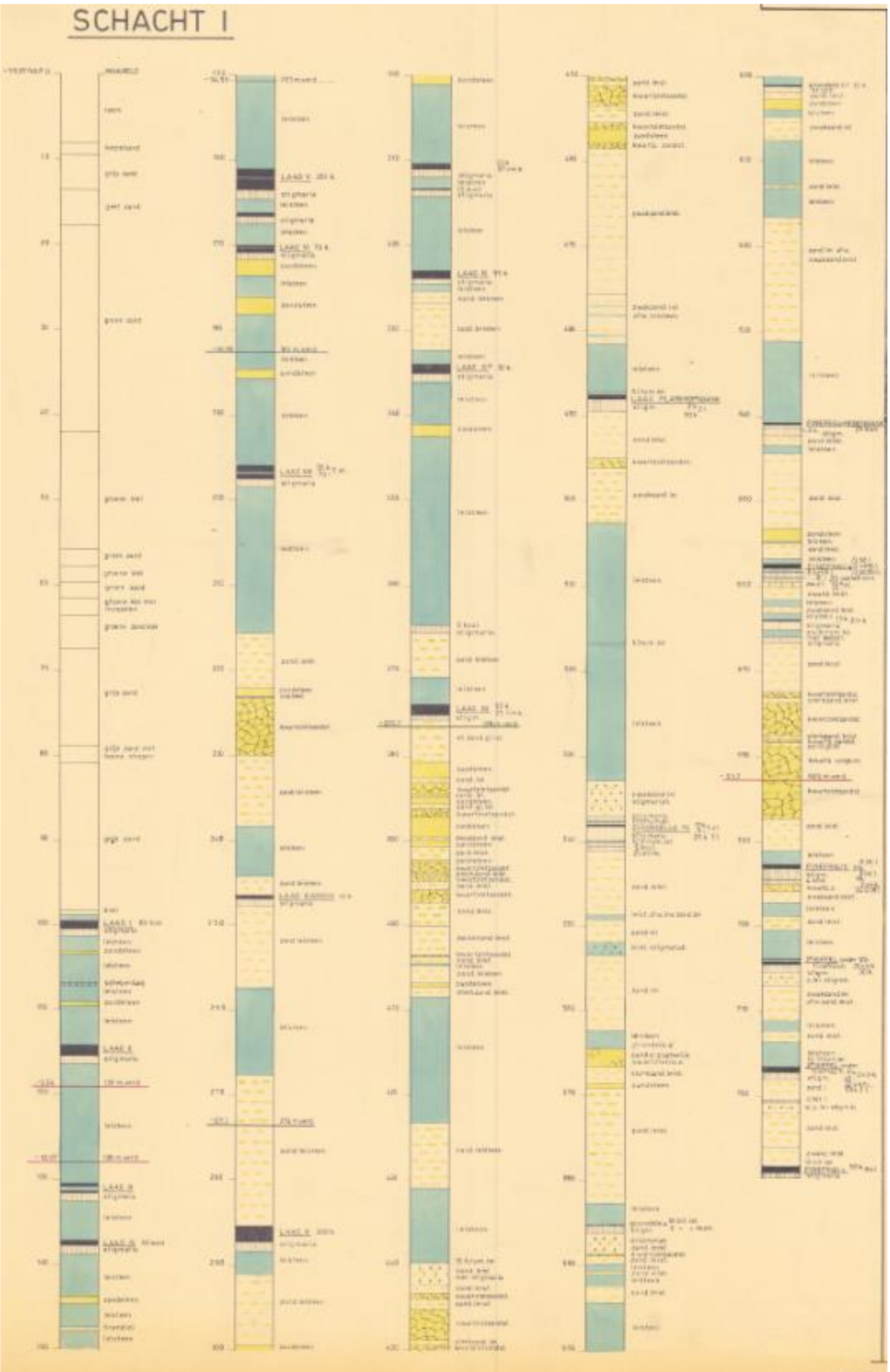


Fig. 36: Geological cross-section shaft Laura I /55/

In 1969 a shaft barrier was embedded on the 378 m (-260 m NAP) floor. 1969/1970 additionally to this barrier there were installed five load bearing fillings above it. The total length of all five fillings measures 73,90 m and required 1400 m³ of concrete. The shaft columns between the concrete sections were backfilled with 4.800 m³ of waste material. The load bearing filling was constructed for a shearing strain of 3 kg/cm². The static calculation for each filling included: the total water pressure measured from the level of the filling up to the ground surface, a column of loose waste material five times as high as the shaft cross-section (silo- effect) and the tare weight of the filling.

For the shaft barrier a mixture of cement and gravel with a quality of compactness of K 225 (resamples C 13/16) was used. Above the topmost load bearing filling the waste material was backfilled in free fall technique /9/. 1974 the shaft was provided with a reinforced concrete cover /14/.

The following figure shows the sectional drawing of the securing of the shaft Laura I.

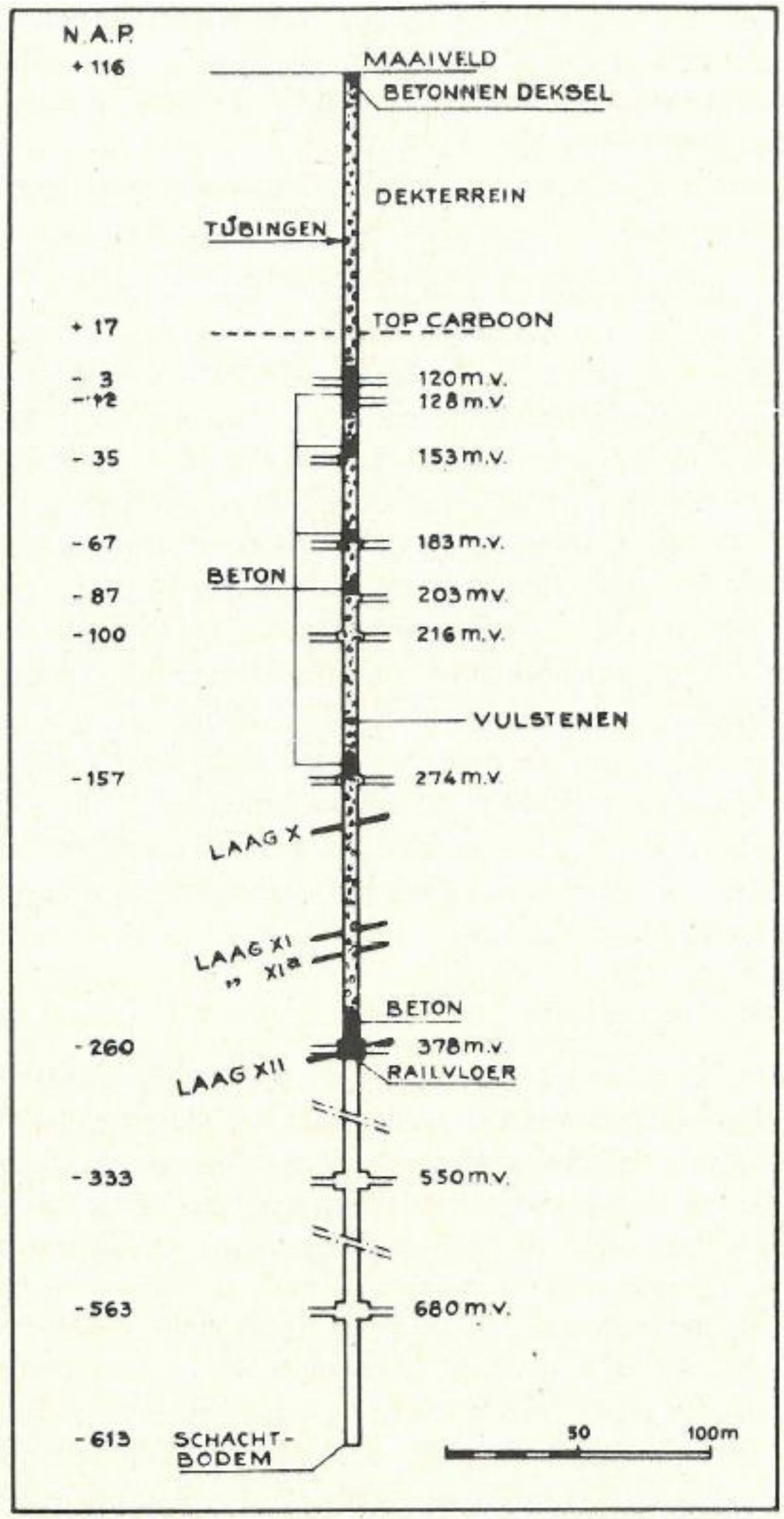


Fig. 37: Securing shaft Laura I /50/

The coordinates of shaft Laura I are:

RD-x:	201611
RD-y:	322793
elevation:	+113 m NAP
positional accuracy :	+/- 1 m

According to the coordinates the shaft is located in an open space northeast of Wackerstraat (community Kerkrade).

5.2 Laura II

The vertical Shaft Laura II was drilled in 1902. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,5 m diameter /4/. The shaft Laura II was drilled to a total depth of 401 m and was used as travelling, drawing and ventilation shaft. The shaft wall was made of masonry /4/. Within the overburden the shaft consists of tubbing support /50/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 99 m /50/. The shaft Laura II has 12 documented insets /50/. The 120 m floor, as the topmost is located in a level of -5,81 m NAP and in a depth of 121 m /6/.

In the following figure the strata in the range of shaft Laura II is pictured.

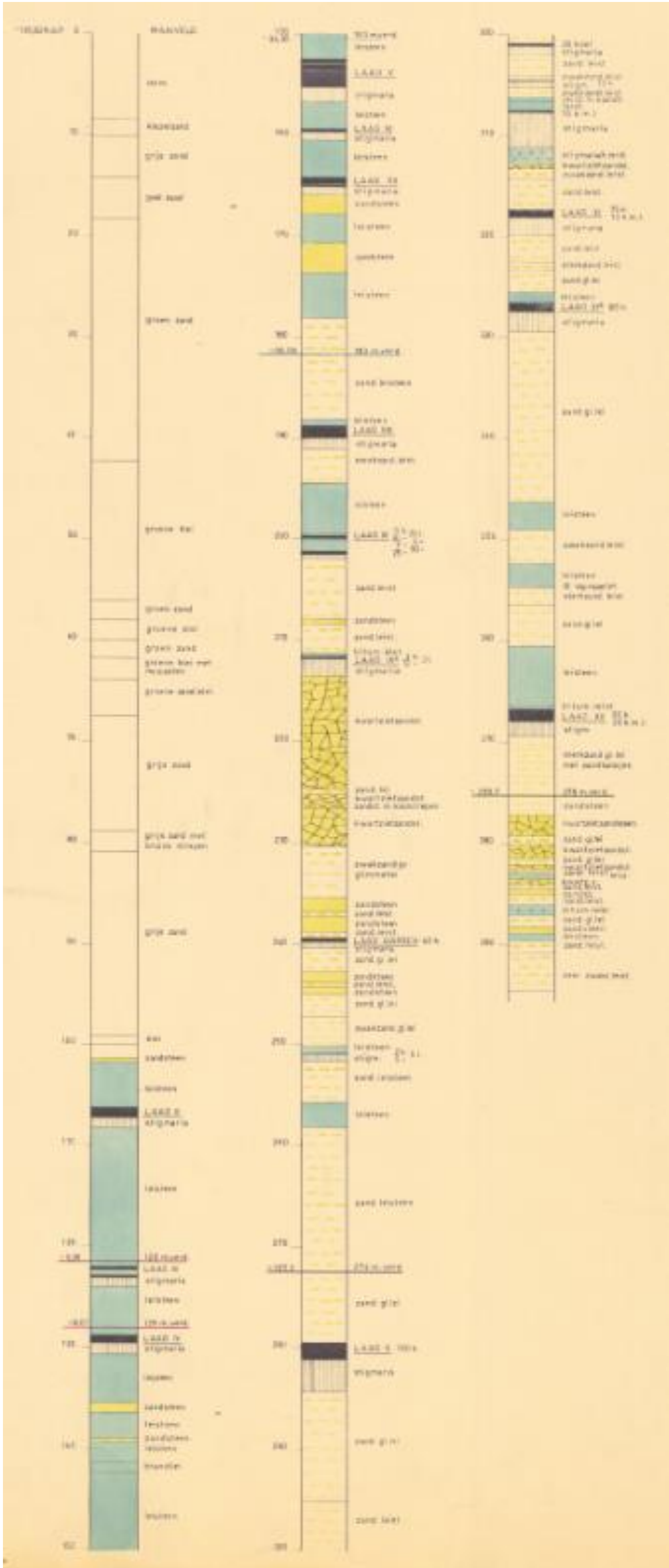


Fig. 38: Geological cross-section shaft Laura II /55/

In 1970 a shaft barrier out of a mixture of concrete (length 16,50 m) was embedded on the 378 m floor.

Beforehand the shaft was backfilled from the shaft sump up to 12 m above the 378 m floor with loose material. Using a drop pipe waste material was backfilled above the first load bearing filling 10 m above the 274 m floor. Then the second load bearing filling (12 m length) was made. Afterwards the shaft column was backfilled with waste material up to 10 m above the 183 m floor. The third embedded load bearing filling had a length of 8,5 m. On top of this, below 7 m of the 120 m floor the shaft was backfilled with further waste material. The topmost filling was embedded upon the floor and had a total length of 25 m. The load bearing filling was constructed for a shearing strain of 3 kg/cm². The static calculation for each filling included the total water pressure measured from the level of the filling up to the ground surface, a column of loose waste material five times as high as the shaft cross-section (silo- effect) and the tare weight of the filling.

For the shaft barrier a mixture of cement and gravel with a quality of compactness of K 225 (resamples C 13/16) was used. Above the topmost load bearing filling the waste material was backfilled in free fall technique /9//10/. Overall 1.200 m³ concrete and 5.280 m³ waste material were used for the back stowing /6/. 1974 the shaft was provided with a reinforced concrete cover /14/.

The coordinates of shaft Laura II are:

RD-x:	201680
RD-y:	322822
elevation:	+113 m NAP
positional accurracy:	+/- 1 m

According to the coordinates the shaft is located in an open space southwest of Edixhovenstraat (community Kerkrade).

5.3 Julia I

The vertical Shaft Julia I was drilled in 1926. In 1975 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 5,5 m diameter. The shaft Julia I was drilled to a total depth of 547,0 m and was used as travelling and drawing shaft. The shaft wall was made of masonry /4/. Within the overburden the shaft consists of tubbing support /50/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 216 m. The shaft Julia I has 6 documented insets. The 303 m floor, as the topmost is located in a level of -200,3 m NAP and in a depth of 303 m /6/.

In the following figure the strata in the range of shaft Julia I is pictured.

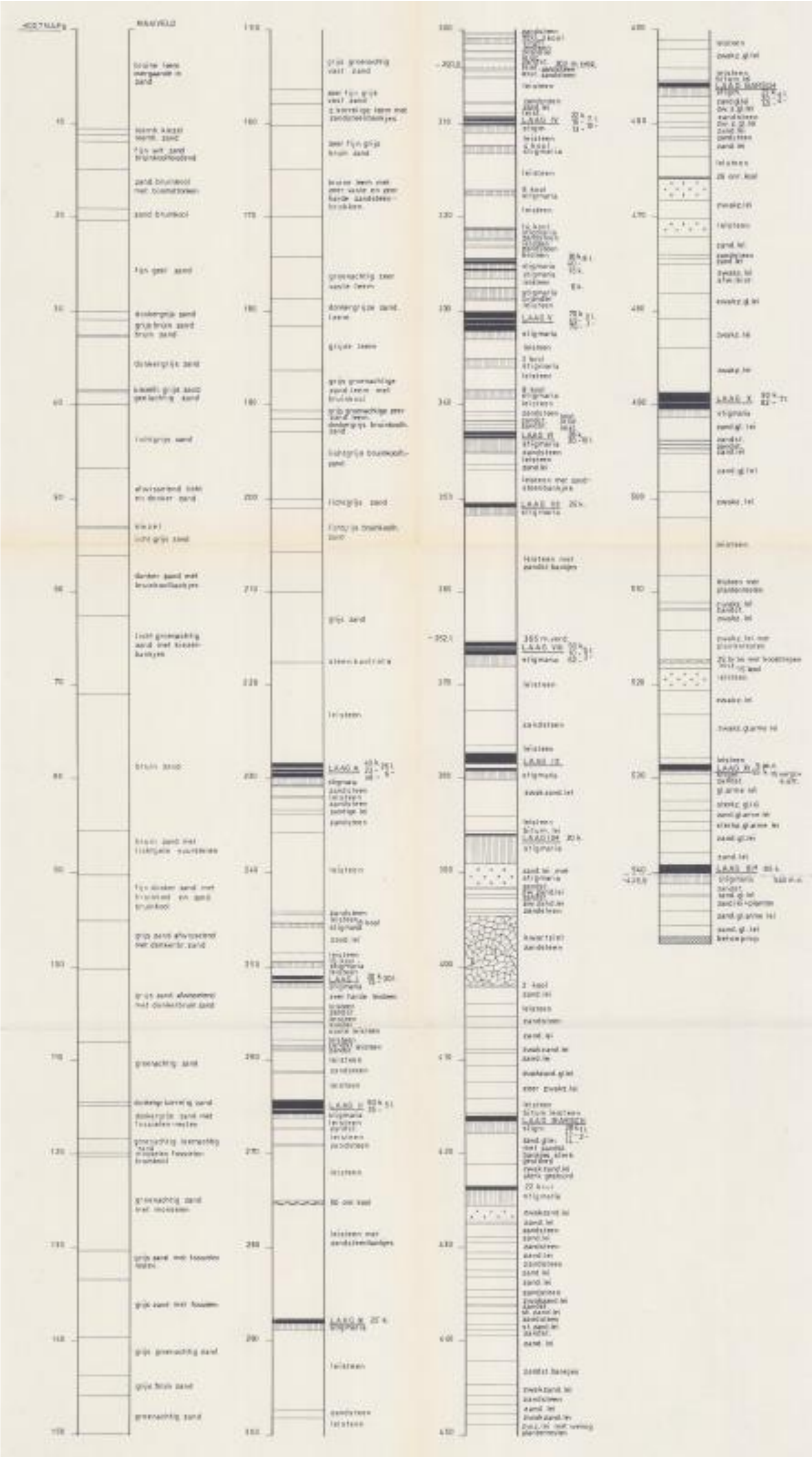


Fig. 39: Geological cross-section shaft Julia I /34/

In 1975 a load bearing filling of 420 m³ of a mixture of concrete (length of 17,0 m) was embedded approximately 12,0 m above the 303 m floor. Here the shaft cross-section measured 5,6 m in diameter. On the level of the 303 m floor an abutment (thickness 1,5 m) was embedded. The existing basement areas underneath the shaft landing were used to bear the filling. Upon this abutment the shaft column was backfilled over a length of roundabout 12 m with approximately 260 m³ waste material. On top of this waste material the load bearing filling was put. The concrete was backfilled in free fall technique. Upon the filling the shaft was backfilled with approximately 6.750 m³ fine grained waste material /15/. 1982 the shaft was provided with a concrete cover /20/.

The following figures show the implementation planning as well as the static calculation of the load bearing filling and the shaft cover for both shafts Laura I and Laura II.

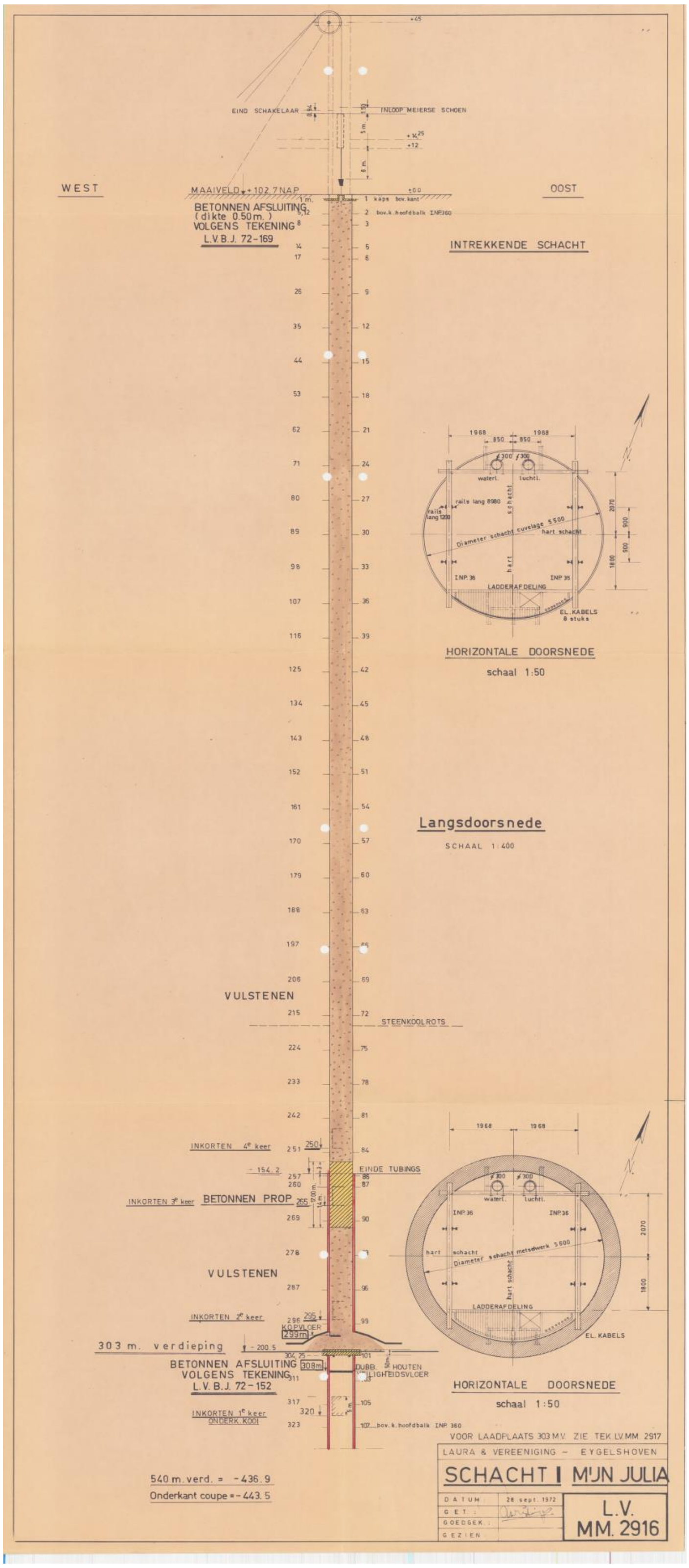


Fig. 40: Implementation planning shaft Julia I /34/

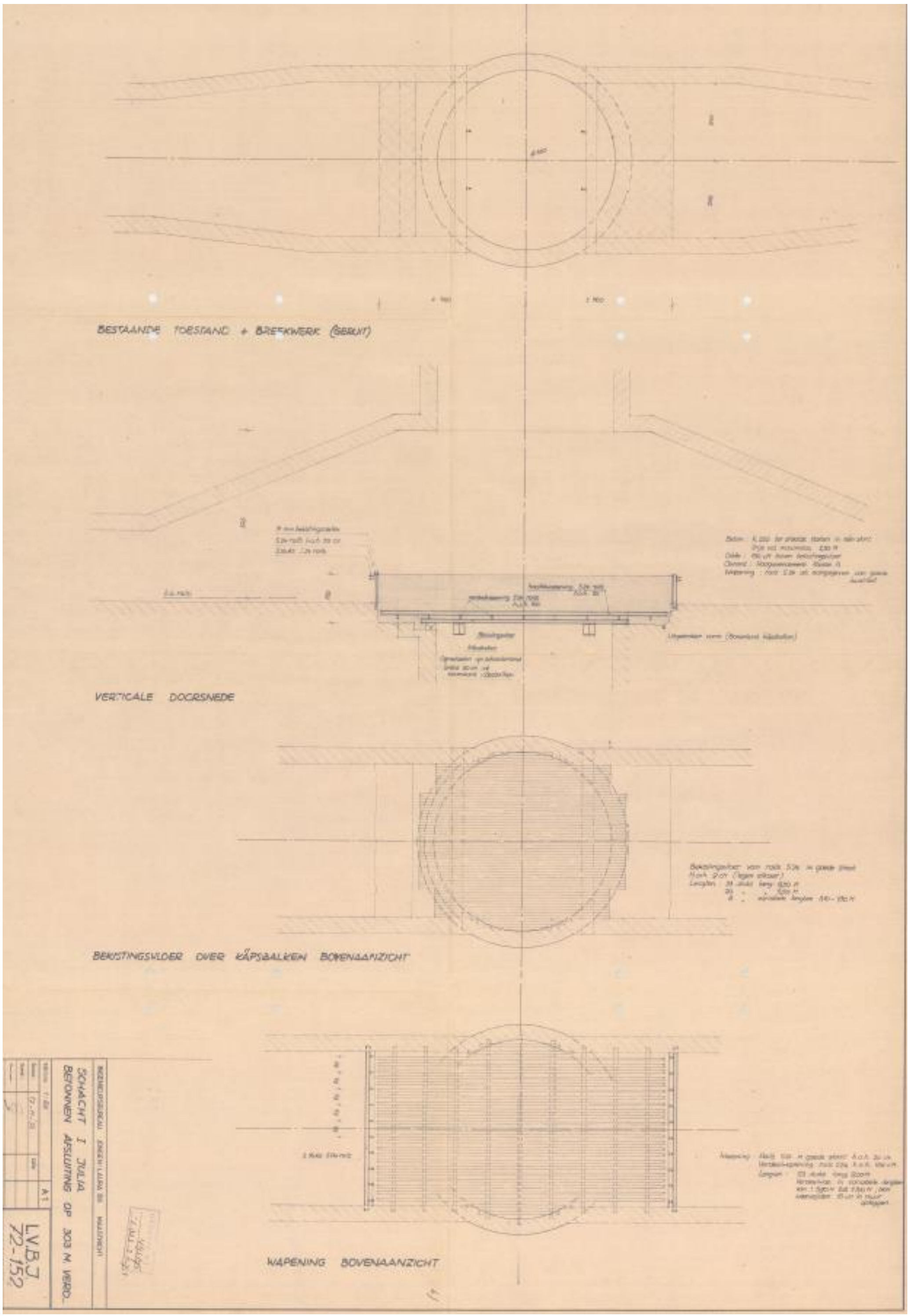


Fig. 41: Shaft cover shaft Julia I 303 m floor /34/

Betreft: Berekeningen lengteproppen in schacht I en II Julia
Tekeningen: L.V.M.M. 2916/2918

Diameter schachten: 5,60 m.
Omtrek : 17,60 m.
Oppervlakte : 24,6 m^2.

Belastingen op kleefproppen:

Van waterkolom tot 3,00 m - maaiveld: $250 \times 10^3 \times 24,6 = 6150 \times 10^3$ kg

Van schachtvulling tot een hoogte van 5 x diameter
schacht (silowerking): $800 \times 5 \times 5,60 \times 24,6 = 551 \times 10^3$ kg

Eigen gewicht betonprop met hoogte H: $2200 \times H \times 24,6 = 54,1 \times 10^3 \times H$ k

Afschuifweerstand: 30.000 kg/m^2.

Totaal: $30 \times 10^3 \times 17,6 \times H = 527 \times 10^3 \times H$ kg

$6150 \times 10^3 + 551 \times 10^3 + 54,1 \times 10^3 \times H = 527 \times 10^3 \times H \longrightarrow H = \frac{6701}{472,9} = \pm 14,00$ m.

Met 3,00 m. extra proplengte in het tubinggedeelte wordt de totale hoogte 17.00 m.

In schacht II is een gedeelte van de schachtwand onderbroken door de bunker-inrichting op 303 m. verd.

Open schachtwand: $\frac{17,60}{4} \times 6,50 = 28,6$ m^2. (gerekend tot tubings).

De ontbrekende afschuifweerstand van : $28,6 \times 30 \times 10^3 = 857 \times 10^3$ kg wordt vervangen door het horizontale steunvlak in de uitbouw.

Oplegdruk: $\frac{857 \times 10^3}{3,60 \times 3,20} = 75 \times 10^3$ kg/m^2 ($\bar{\sigma} = 600 \times 10^3$ kg/m^2)

Betreft: Bodem in schacht I Julia, 303 m.
Tekening: L.V.B.J. 72-152

Oppervlakte schacht: 24,6 m^2; omtrek 17,60 m.

Reken betonnen plaat dik 1,50 m.

Belastingen:

Eigen gewicht plaat 1,50 x 2400 = 3600 kg/m^2

Gewicht vulstenen tot schachtuitmonding 1800 x 4,50 = 8100 kg/m^2

De volgende lasten worden verspreid over een oppervlakte van 14,50 x 4,80 (45° lastspreiding)

Vulmateriaal van 195 - NAP tot 168 - NAP = 27 m ⟶

27.00 x 1,8 x 24,6 = 1200 ton

stortgewicht kleefprop 17,00 x 2,2 x 24,6 = 920 ton

2120 ton

$\frac{2120000}{14,50 \times 4,80}$ = 30500 kg/m^2

42200 kg/m^2

l_t gemiddeld = 6,00 m $M_{max} = \frac{42200}{8} \times 6,00^2 = 190.000$ kgm

Beton K 225. Wapening met spoorrails.

h_t = 1,50 m h = 1,20 m k_h = 0,275 f_y = 1,114 x 120 = 134 cm^2

Toepassen 5 stuks spoorrails per m^1 = 5 x 31,5 = 157,5 cm^2.

Controle schuine trekspanning

2/3 gedeelte van schachtomtrek kan dwarskracht opnemen = $\frac{2}{3} \times 17,60 = 11,70$m.

Op te nemen dwarskracht per m^1: $\frac{24,6 \times 42200}{11,70}$ = 89000 kg.

Schuine trekspanning: $\frac{1,5 \times 89000}{100 \times 150}$ = 8,9 kg/cm^2.

Deze spanning is toelaatbaar daar silowerking van het vulmateriaal niet in rekening is gebracht en de belasting van korte duur is.

Bekistingsvloer

Belasting door stortgewicht van 1,50 m. beton = 1,50 x 2400 = 3600 kg/m^2

l_t maximaal = 5,75 m $M_{max} = \frac{3600}{8} \times 5,75^2 = 14900$ kgm.

Ben.W. = $\frac{14900}{14}$ = 1065 cm^3

Rails S 24 tegen elkaar ⟶ $\frac{100}{9} \times 97,3 = 1100$ cm^3

Bekistingswand

Hoogte 1,00 m; maximale belasting door beton: 2400 kg/m^2

Horizontale steunbalk = A,

1,00 $R_A = 2400 \times 0,50 \times \frac{1,00}{3}$ ⟶ R_A = 400 kg/m^1 l_t = 5,00 m.

$M_{max} = \frac{400}{8} \times 5,00^2 = 1250$ kgm Ben.$W_x = \frac{1250}{14}$ = 90 cm^3

Toepassen 2 stuks rails S 24 ⟶ W_x = 194,6 cm^3

Betreft: Bodem in schacht II Julia; 303 m. verd.
Tekening: L.V.B.J. 72-153

Oppervlakte schacht : 24,6 m^2

Omtrek : 17,60 m

Reken betonnen plaat 1,50 m

Belastingen

Eigen gewicht plaat 1,50 x 2400 = 3600 kg/m^2

Gewicht vulstenen tot schachtuitmonding 1800 x 5,50 = 9900 kg/m^2

2120 ton van kolom materiaal in de schacht

is verspreid over een oppervlakte van 16,50 x 4,70 →

$\frac{2120000}{16,50 \times 4,70}$ = =27400 kg/m^2

40900 kg/m^2

l_t gem. = 6,00 m.

Belasting gunstiger dan plaat in schacht I

Afmeting, wapening en bekistingsvloer idem.

Fig. 42: Static calculation shafts Julia I and Julia II /34/

Betreft: Afdekking schachten Julia op maaiveld
Tekeningen L.V.B.J. 72-169 en 72-170

Algemeen: Belastingen vlgs. klasse 60 V.O.S.B. 1963:

Gelijkmatig verd. bel.	400 kg/m^2
Asdrukken $\frac{40.000}{4,00 \times 3,00}$ =	3330 kg/m^2
Gronddekking 1,00 x 1800	1800 kg/m^2
E.G. betonplaat: 0,50 x 2400	1200 kg/m2
	6730 kg/m^2

$l_y/l_x = 580/_{580} = 1$ $\quad 0,001 \times 6730 \times 5,80^2 = 227$

$M_{vx} = 227 \times 44 = 10000$ kgm $\quad k_h = 0,450 \quad f_y = 0,231 \times 45 = 10,38\ cm^2$

toepassen Ø 16-19 1^e laag

$M_{vy} = 227 \times 44 = 10000$ kgm $\quad k_h = 0,434 \quad f_y = 0,249 \times 43,4 = 10,8\ cm^2$

Ø 16-18^5 2^e laag.

fy oplegg. x = fy oplegg. y = Ø 16-37.

Hr. Schmitz
Jongen - Laura
Maastricht

Fig. 43: Static calculation shaft cover shafts Julia I and Julia II /34/

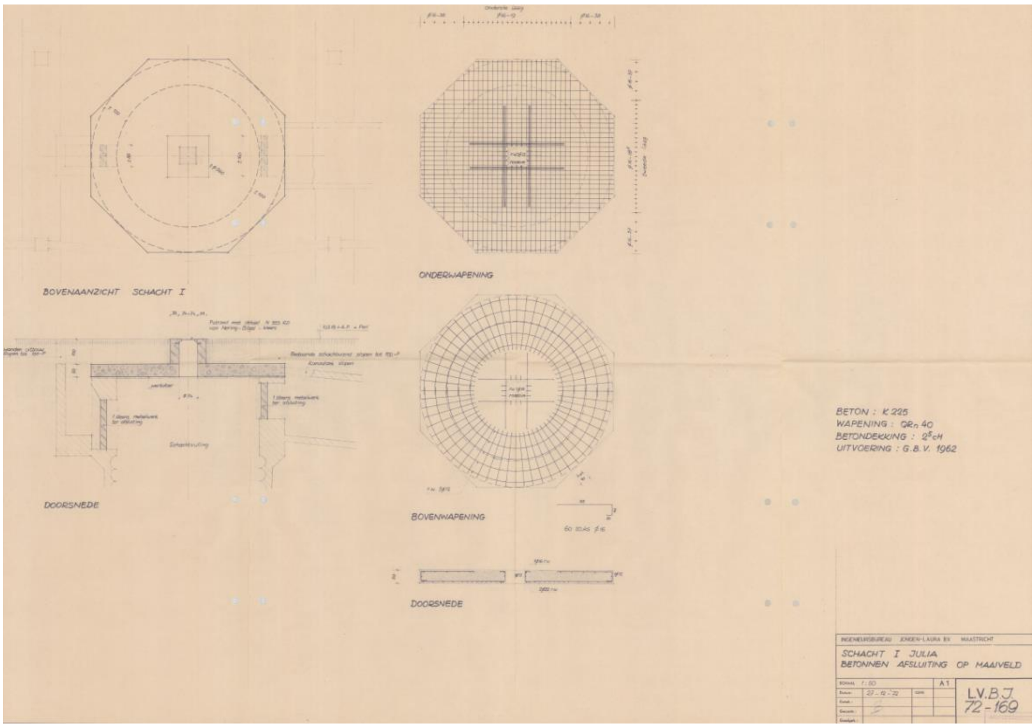


Fig. 44: Implementation planning shaft Julia I /34/

The coordinates of shaft Julia I are:

RD-x:	202781
RD-y:	323110
Elevation:	+102 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located westwards the Nievelsteenstraat (community Kerkrade) on the traffic area of an industrial estate westwards a building.

5.4 Julia II

The vertical Shaft Julia II was drilled in 1926. In 1975 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 5,5 m diameter. The shaft Julia II was drilled to a total depth of 568,0 m. The shaft head was made of masonry and tubbing support and beneath of concrete. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 212 m. The shaft Julia II has 6 documented insets. The 303 m floor, as the topmost is located in a level of -200,3 m NAP and in a depth of 303 m /6/.

In the following figure the strata in the range of shaft Julia II is pictured.

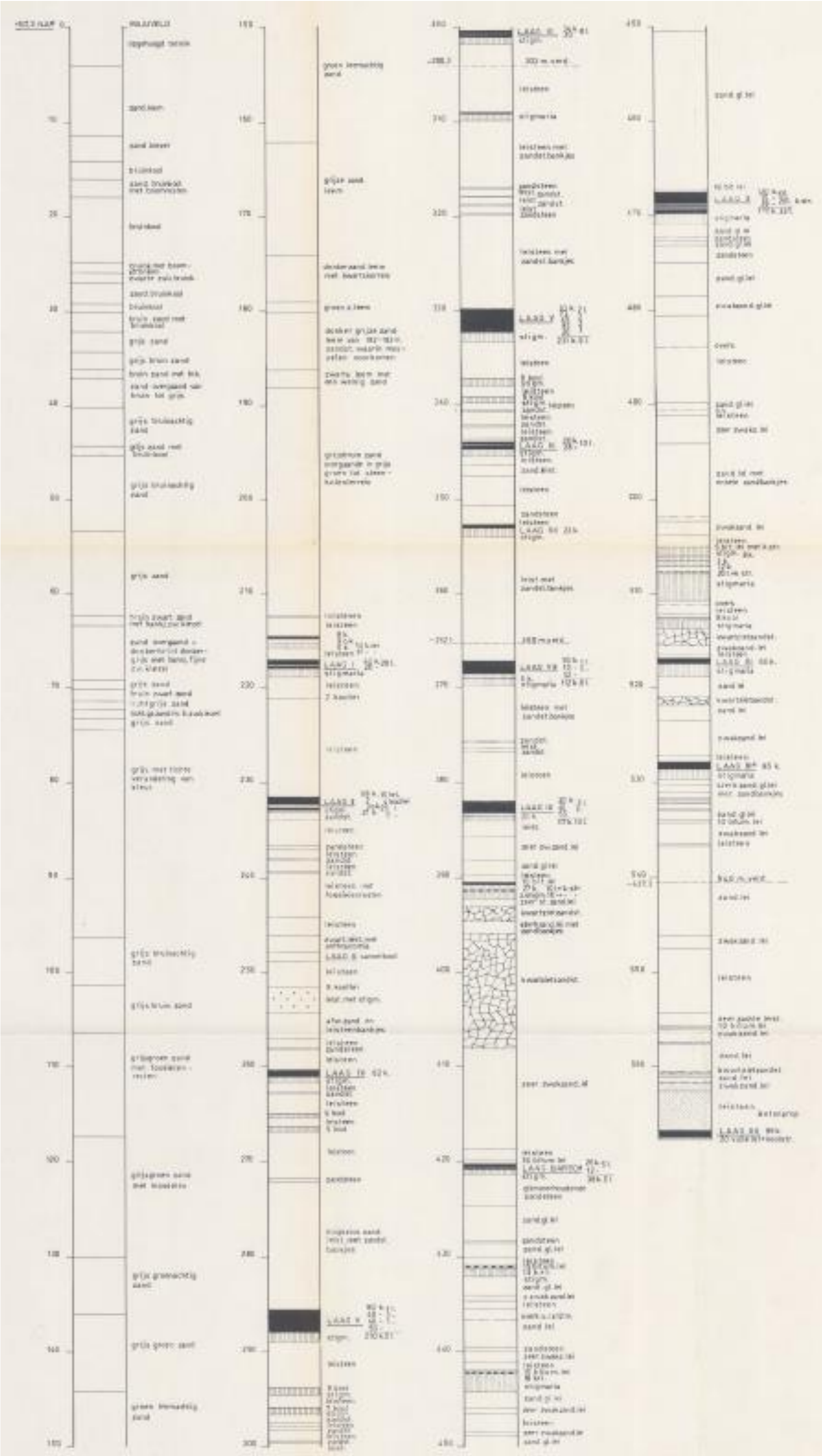


Fig. 45: Geological cross-section shaft Julia II /34/

In 1975 a load bearing filling of 420 m³ of a mixture of concrete (length of 17,0 m) was embedded approximately 12,0 m above the 303 m floor. Here the shaft cross-section measured 5,6 m. On the level of the 303 m floor an abutment (thickness 1,5 m) was embedded. The existing basement areas underneath the shaft landing were used to bear the filling. Upon this abutment the shaft column was backfilled over a length of roundabout 12 m with approximately 258 m³ waste material. On top of this waste material the load bearing filling was put. The concrete was backfilled in free fall technique. By this a waste material dugout connected to the shaft was backfilled as well. Up on the filling the shaft was backfilled with approximately 6.750 m³ fine grained waste material /15/. For the use of ground water monitoring an observation pipeline was installed between the 540 m floor and the level 2 m below the ground surface. 1982 the shaft was provided with a concrete cover /20/.

The following figures show the implementation planning as well as the static calculation of the load bearing filling and the shaft cover.

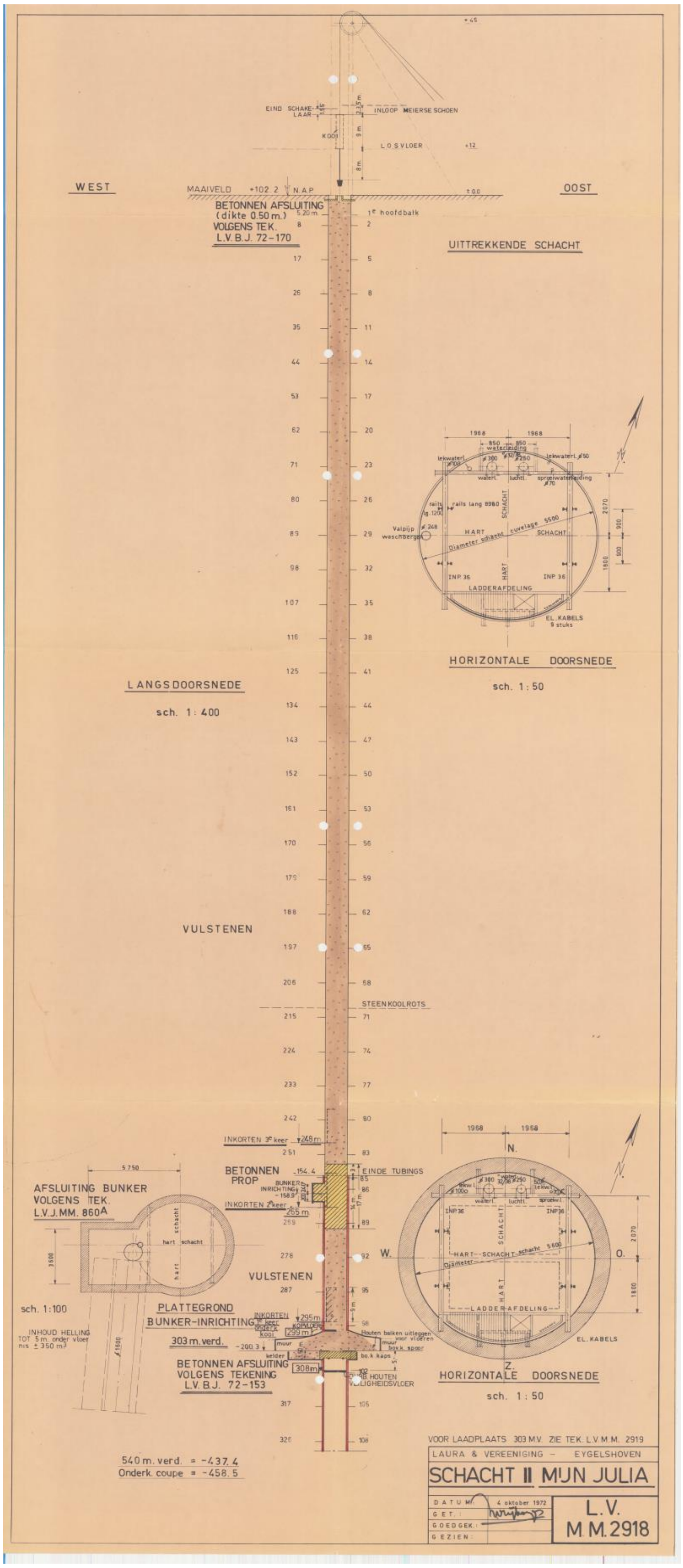


Fig. 46: Implementation planning shaft Julia II /34/

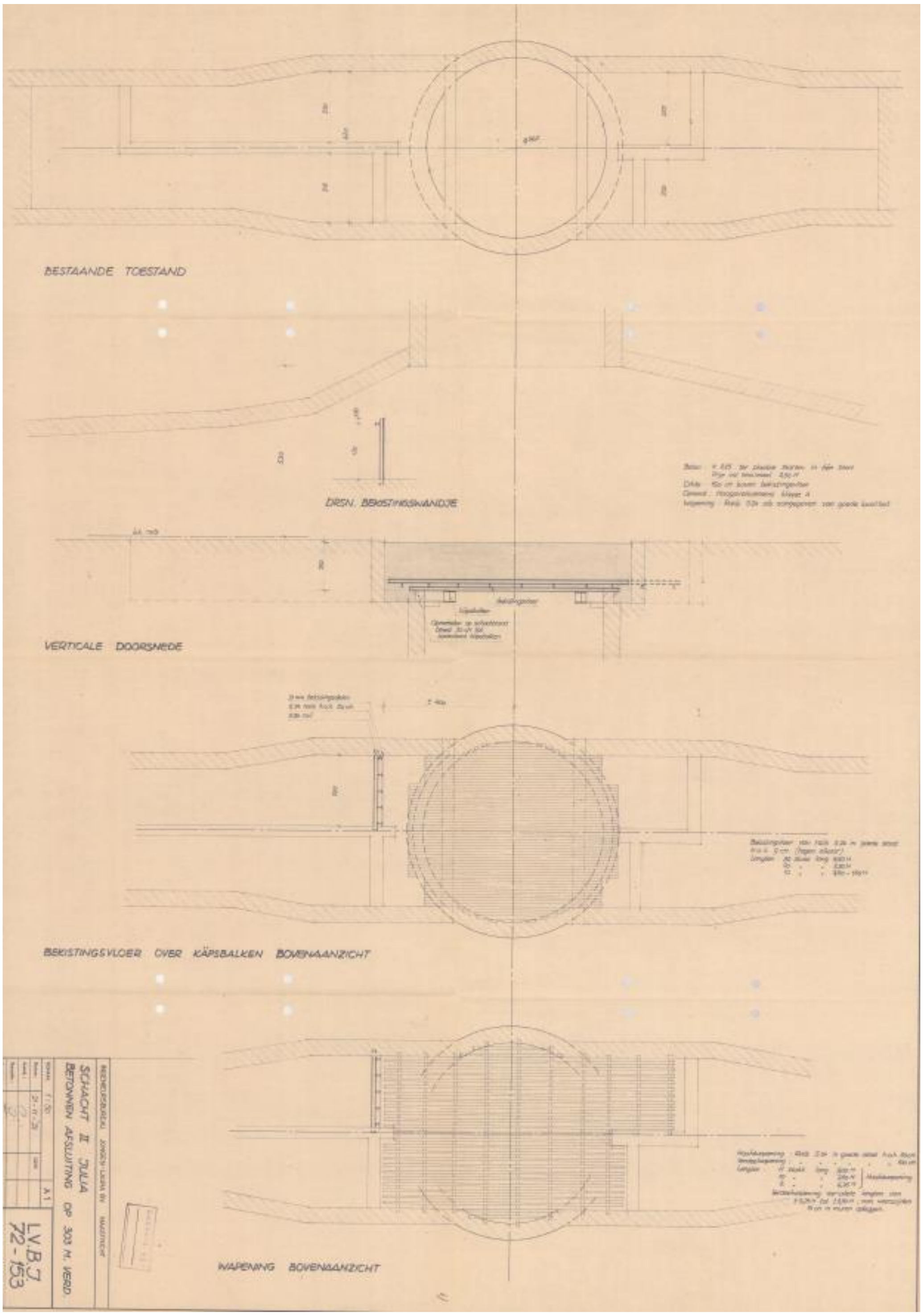


Fig. 47: Shaft cover shaft Julia II on the 303 m floor /34/

The static calculation of the load bearing filling is to be found in the chapter above (Shaft Julia I).

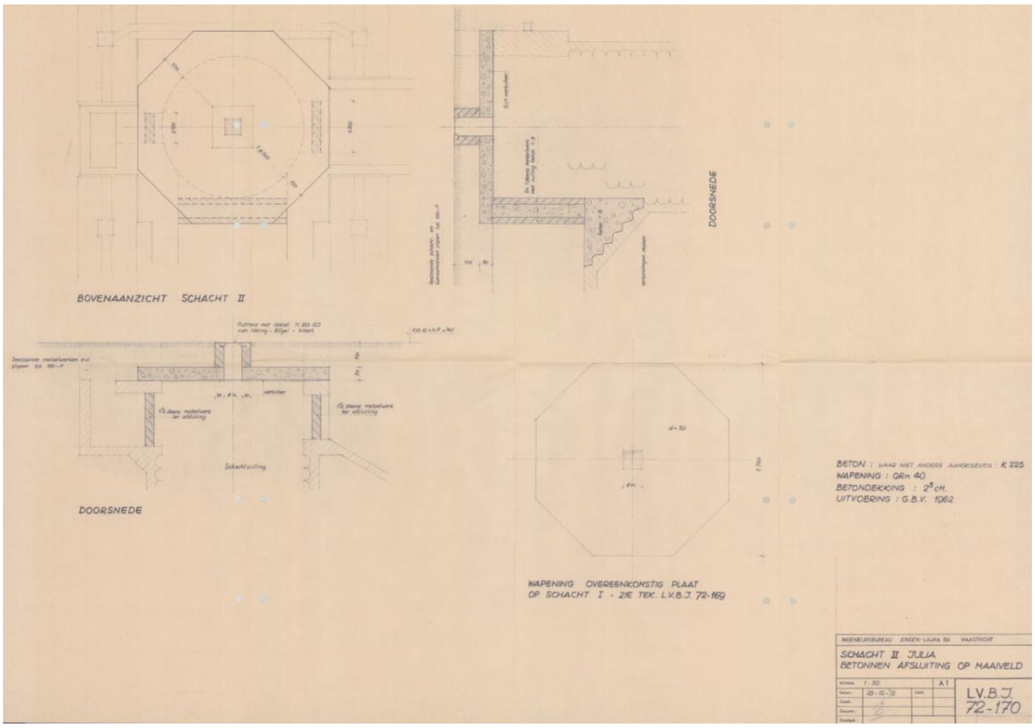


Fig. 48: Implementation planning shaft cover shaft Julia II /34/

The coordinates of shaft Julia II are:

RD-x:	202875
RD-y:	323143
elevation:	+102 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on an industrial estate north of Bart van Slobbestraat.

6 Oranje Nassau Mijnen

6.1 Shaft I, ON I

The vertical Shaft I of the pit Oranje Nassau I was drilled in 1894. In 1975 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 3,0 m diameter. The shaft was drilled to a total depth of 255,0 m and was used as upcast air shaft and drawing shaft. From the overburden to the carbon (level of -1,41 m NAP) the shaft consists of a tubbing support followed by masonry (thickness 0,5 m) /4//36/. The section between 0 m and 9 m was made of masonry (thickness 0,55 m) as well /36/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 95,61 m and has a layering sequence of sand and clay /36/. The shaft has 10 documented insets. The 136 m floor, as the topmost is located in a level of -26,50 m NAP and in a depth of 135 m /6//50/.

In the following figure the strata in the range of the 136 m floor is pictured (here mainly slate and sandstone).

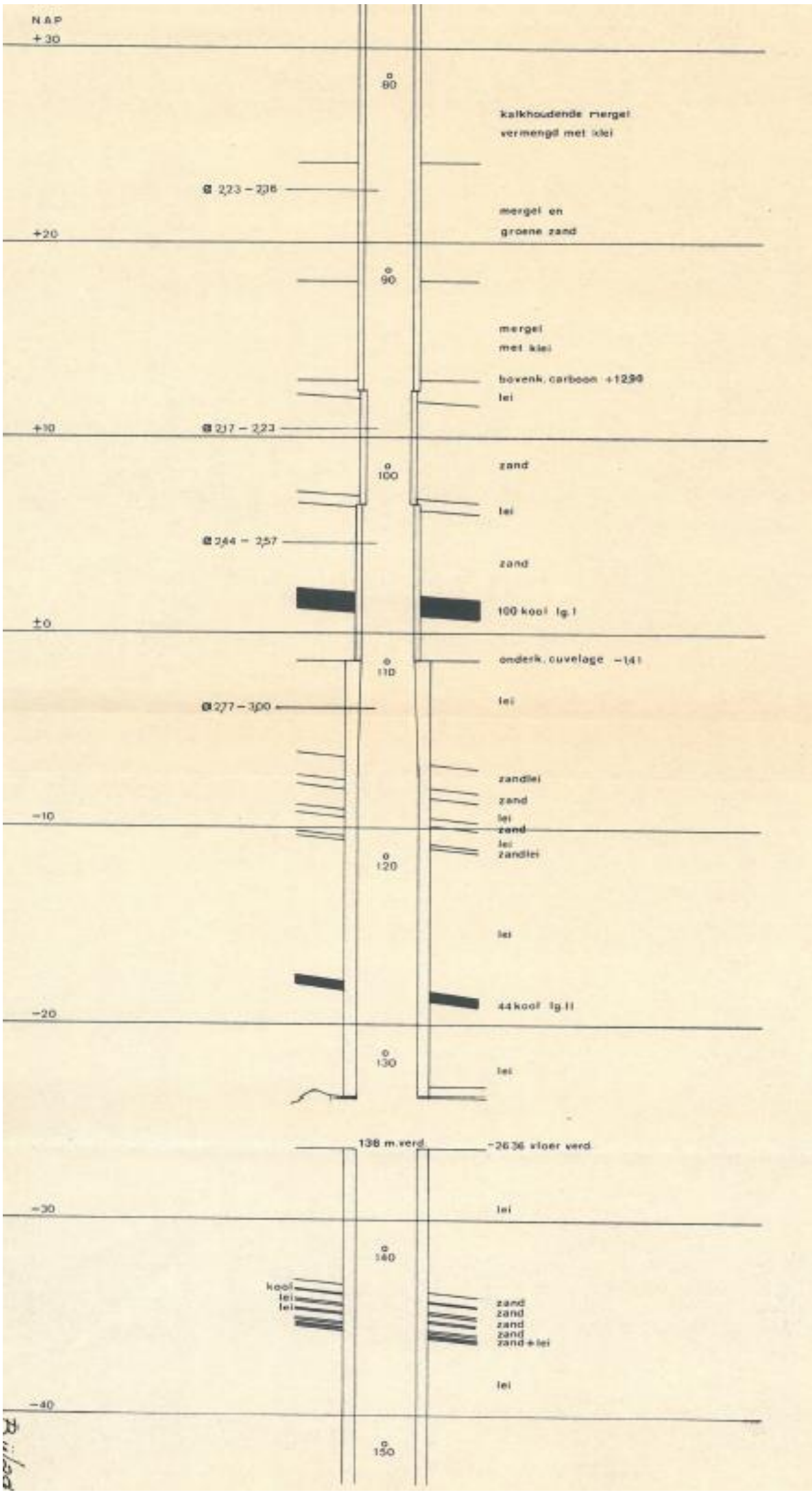


Fig. 49: Strata shaft I, ON I, 136 m floor /36/

In 1975 a load bearing filling out of 100 m³ of a mixture of concrete (length 8 m) was embedded in the 136 m floor (-26,50 m NAP). Additionally on the 136 m level a platform of iron beams covered by a heavy reinforced concrete board (thickness 1 m), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. The back stowing was carried out by the use of a drop pipe. Above the barrier the shaft column was backfilled with approximately 48 m³ waste material, 30 m³ concrete and additional approximately 300 m³ waste material up to the ground surface /14//35/. The shaft had to be topped up with 8 m³ waste material /35/. 1980 the shaft was provided with a concrete cover (thickness 3,27 m) out of 35 m³ concrete /18//37/. On top the cover was overlaid with 2 m of waste material and 0,8 m of soil /37/.

The following figures show the shaft barrier of shaft I in a schematic representation.

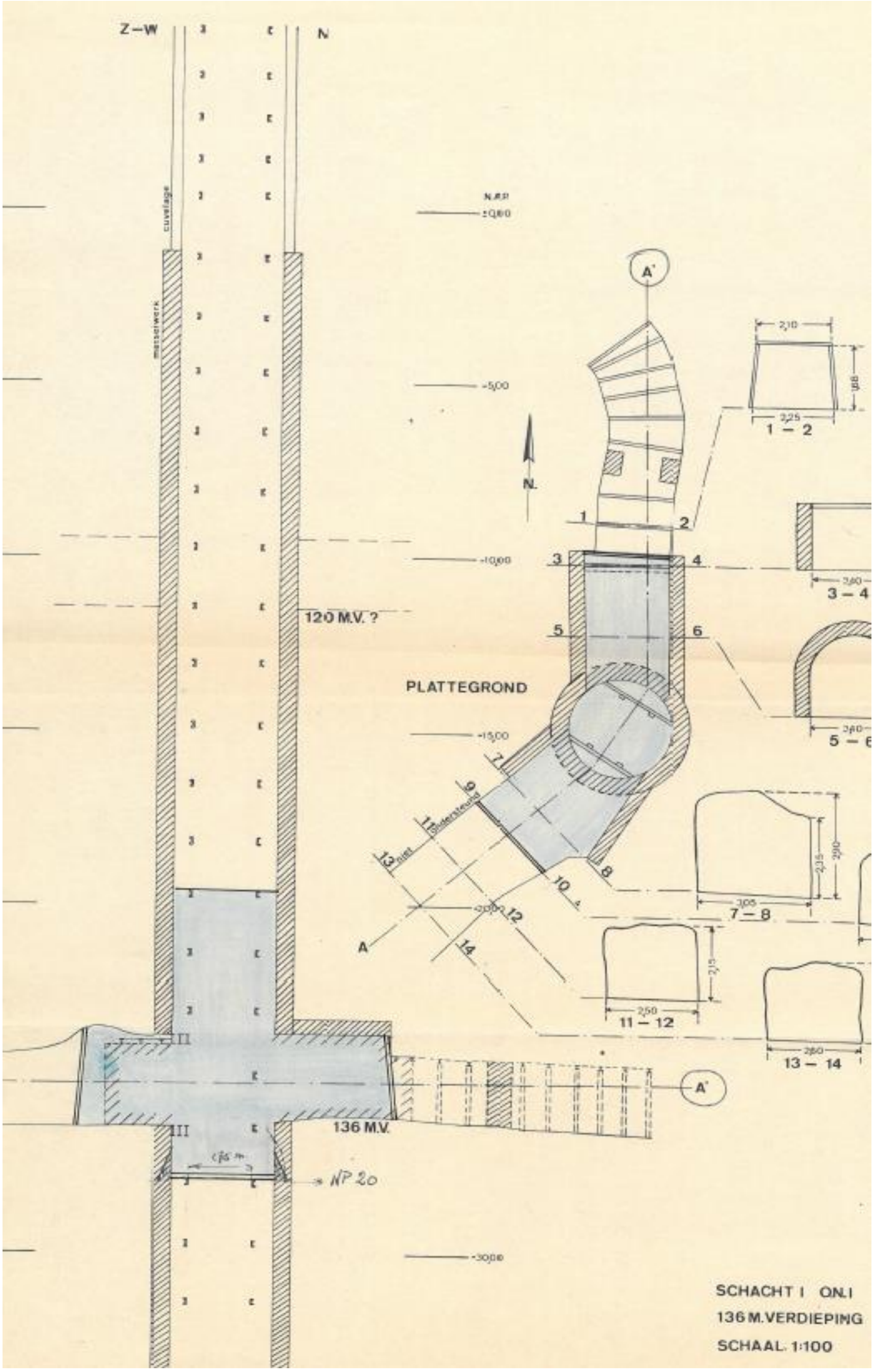


Fig. 50: Shaft barrier shaft I, ON I /36/

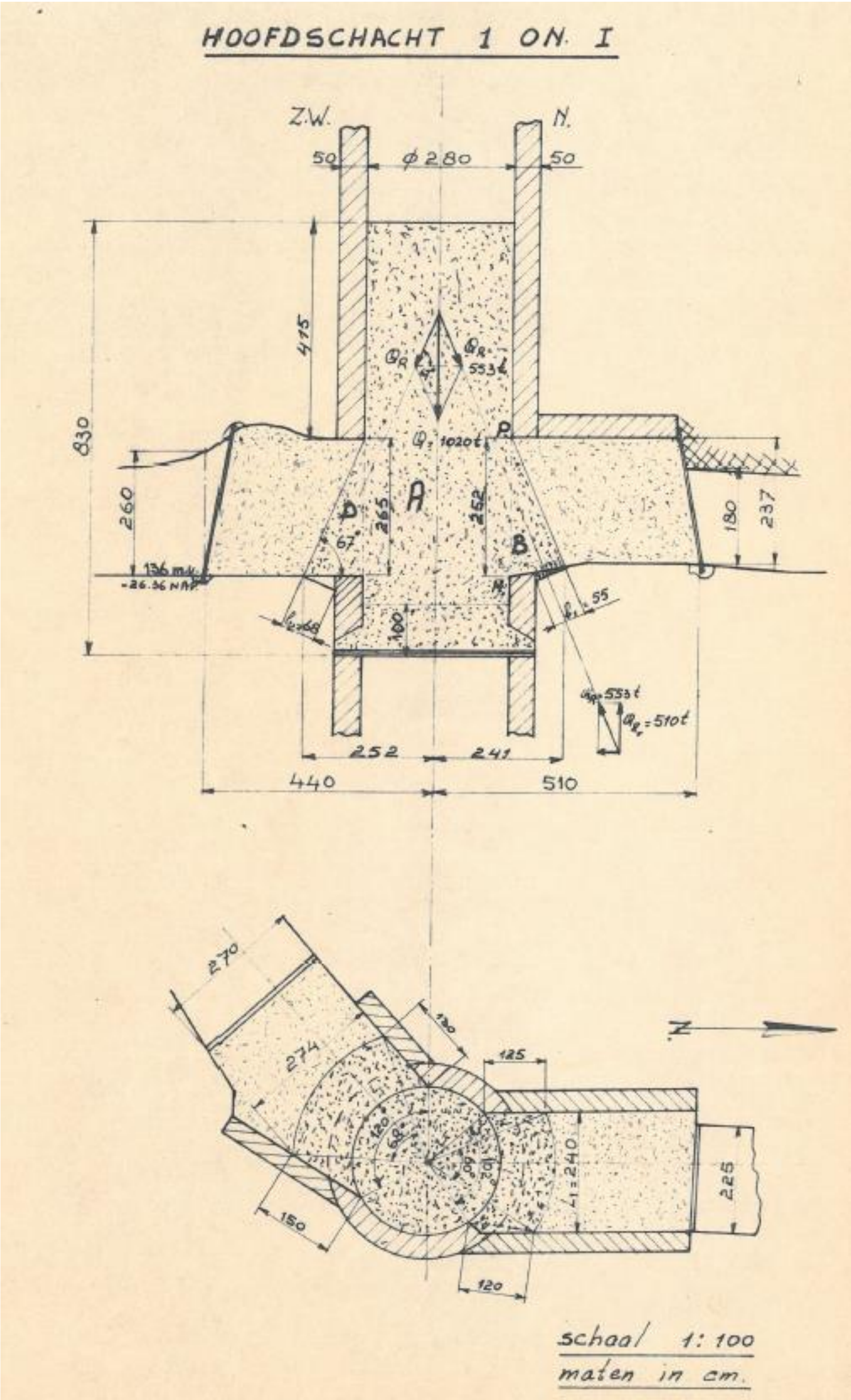


Fig. 51: Shaft barrier shaft I, ON I /36/

Static calculations of the shaft barrier of the shaft I, ON I are existent /36/. Compare the following figures.

Prop hoofdsch. 1 O.N. I

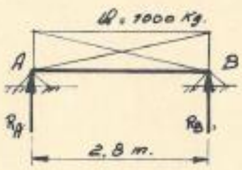
Diameter schacht = 2,80 m.
Opp. schachtdoorsn. = $\pi \times 1{,}40^2 = 6{,}15\ m^2$
Railvloer 136,5 m onder maaiveld.
Prophoogte : 1e gedeelte prop, hoog 1 m., op railvloer. Laten verharden.
Daarna 2e gedeelte storten tot 8,30 m. boven railvloer. Totale hoogte prop : 8,30 m.
S.g. beton = 2,4.

Vloer uit rails NP 46.

Railhoogte = 142 mm.
railvoet br. = 120 mm.
railkop br. = 72 mm.

$W_x = 231\ cm^3$
$I_x = 1640\ cm^4$
$G = 46{,}23\ Kg/m$.

Middelste rail:



Belasting Q.

Q = gew. beton hoog 1 m. + e.g. rail.

$Q = (2{,}8 \times 0{,}12 \times 1 \times 2{,}4) + \frac{46{,}23 \times 2{,}8}{1000}$

Q = ca. 1 ton.

Berekening: Op sterkte:

$M_b = \frac{Q \cdot l}{8} = \frac{1000 \times 280}{8} = 35000\ Kgcm.$

$\sigma_b = \frac{M_b}{W} = \frac{35000}{231} = 152\ Kg/cm^2$

Vlaktedruk oplegging rails in metselwerk van schachtwand.

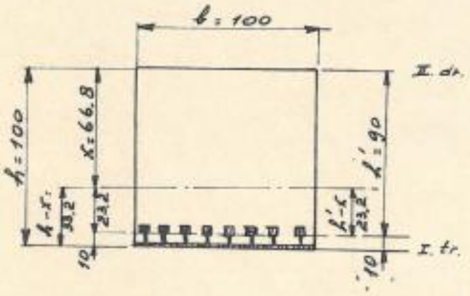
Toelaatbare vlaktedruk σ_v metselwerk = 20 Kg/cm^2
Lengte oplegging rail = 50 cm.

$R_A = R_B = \frac{1000}{2} = 500\ Kg.$ $F = 50 \times 12 = 600\ cm^2$

$\sigma_v = \frac{R_A}{F} = \frac{500}{600} = 0{,}835\ Kg/cm^2$

Vloer na 1e stort tot 1.00 m. dikte, berekend op buiging.

Wapening aaneengesloten vloer uit rails N.P. 46
Voetbreedte 12 cm. F_y rail = 58 cm² F per m' = $\frac{100}{12}$ x 58 = 485 cm²
Totale hoogte prop H = 8,30 m. $n = \frac{E_y}{E_b} = 15$.



Belasting q.

$q_{be} = H \times b \times s.g._{be} = 8{,}3 \times 1 \times 2400 = 19900$ Kg/m
e.g. yzer = $46{,}23 \times \frac{100}{12}$ = 385 ..

q = ± 20300 Kg/m.

$M_b = \frac{1}{8}\, Q\, l$. l = 2,8 + 0,3 = 3,1 m.

$M_b = \frac{1}{8}\,(20300 \times 3{,}1)\,3{,}1 = 24400$ Kgm.

M_b = 2.440.000 Kgcm.

Bepaling zwaartepunt x. $x = \frac{\Sigma F.z}{\Sigma F}$

$$x = \frac{\frac{1}{2}\, b.h^2 + n.F_y \times h'}{b.h + n.F_y}$$

$$x = \frac{(\frac{1}{2} \times 100 \times 100^2) + (15 \times 485 \times 90)}{100 \times 100 + 15 \times 485} = \frac{500000 + 655000}{10000 + 7280} \longrightarrow x = 66{,}8 \text{ cm}$$

$$I_{id.} = I_{be.} + n.I_y$$

$$= \frac{1}{3}\, b.x^3 + \frac{1}{3}\, b(h-x)^3 + n.F_y(h'-x)^2$$

$$= (\frac{1}{3} \times 100 \times 66{,}8^3) + (\frac{1}{3} \times 100 \times 33{,}2^3) + (15 \times 485 \times 23{,}2^2) \longrightarrow I_{id.} = 15.090.000 \text{ cm}^4$$

Drukzijde:

$$\sigma_{bd.} = \frac{M}{W_x} = \frac{M \times x}{I_{id}} = \frac{2.440.000 \times 66.8}{15.090.000} \longrightarrow \sigma_{bd.} = 10{,}8 \text{ Kg/cm}^2$$

Trekzijde:

$$\sigma_{bt} = \frac{M}{W_x} = \frac{M.(h-x)}{I_{id.}} = \frac{2.440.000 \times 33{,}2}{15.090.000} \longrightarrow \sigma_{bt} = 5.35 \text{ Kg/cm}^2$$

$$\sigma_{yt} = n \times \sigma_{bt} = 15 \times 5.35 \longrightarrow \sigma_{yt} = 80 \text{ Kg/cm}^2$$

Vlaktedruk prop

σ_v toelaatb. = 60 Kg/cm^2

Q gerekend vanaf maaiveld

Inhoud betonprop.

deel A: $\frac{\pi}{4} \cdot 2{,}8^2 \times 8{,}3 =$ 51 m^3

deel B : $\left\{\pi(2{,}41^2 - 1{,}4^2)\frac{60}{360} + \left(\frac{1{,}25 \times 0{,}44}{2}\right) + 1{,}2 \times \frac{0{,}4 + 0{,}16}{2}\right\} \frac{2{,}52}{2} =$ 3.32 m^3

deel D : $\left\{\pi(2{,}52^2 - 1{,}4^2)\frac{68}{360} + \left(\frac{1{,}5 \times 0{,}54}{2}\right) + \frac{1{,}3 \times 0{,}44}{2}\right\} \frac{2{,}65}{2}$ = 4.35 m^3

Totaal ± 60 m^3.

Gewicht betonprop = 60 x 2,4 = 144 ton

Gewicht waterkolom : $(136{,}5 - 8{,}3) \cdot \pi \times 1{,}4^2 \times 1$ = 790 ..

* Gewicht vulstenen onder water (silo-werking).
$= 5 \times 2{,}8 \times \pi \times 1{,}4^2 \times 1$ = 86 ..

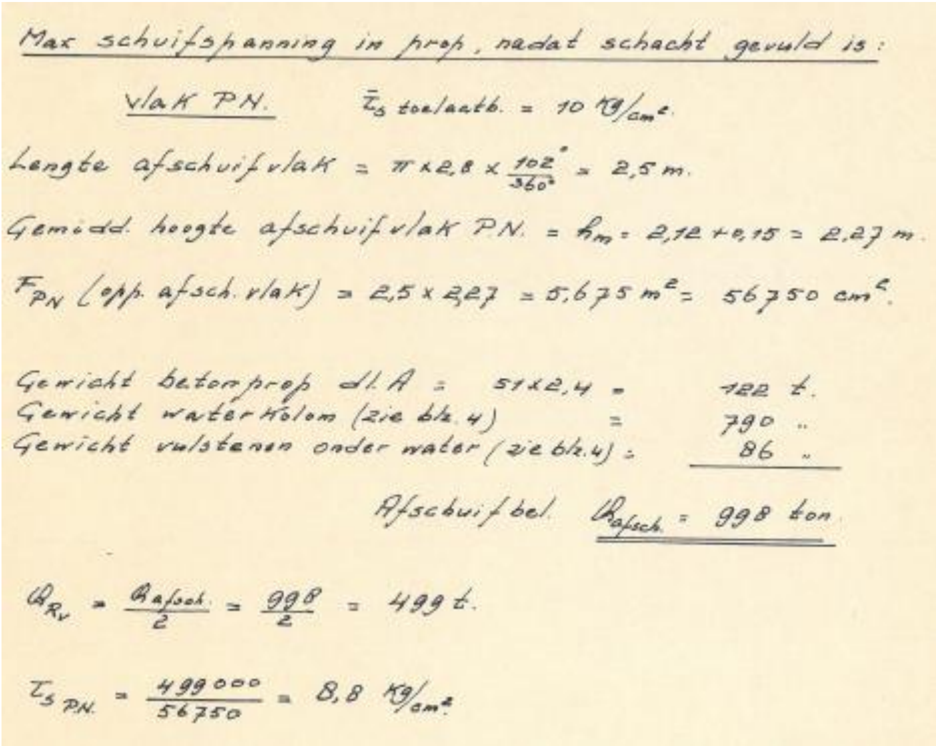
Q_{totaal} = 1020 ton

$$Q_R = \frac{\frac{1}{2} Q}{\cos 23} = \frac{1020}{2 \cos 23} = \frac{1020}{2 \times 0{,}9205} = 553 \text{ ton}$$

$$F = L_1 \times b_1 = 2{,}4 \times 0{,}55 = 1{,}32 \ m^2$$

$$\sigma_v = \frac{Q_R}{F} = \frac{553\,000}{13200} = 41{,}8 \ Kg/cm^2$$

* De werkzame massa van de vulstenen voor de druk op de prop wordt door de "Silo-werking" op 5 x diam. v.d. schacht – d.i. 5 x 2,8 m = 14 m. hoogte – gesteld.
De soortelyke massa van de vulstenen onder water is gesteld op 1.



Max schuifspanning in prop, nadat schacht gevuld is:

vlak P.N. τ_s toelaatb. = 10 kg/cm²

Lengte afschuifvlak = $\pi \times 2{,}8 \times \frac{102^\circ}{360^\circ} = 2{,}5$ m.

Gemidd. hoogte afschuifvlak P.N. = $h_m = 2{,}12 + 0{,}15 = 2{,}27$ m.

F_{PN} (opp. afsch. vlak) = $2{,}5 \times 2{,}27 = 5{,}675$ m² = 56750 cm².

Gewicht betonprop d.l. A = 51×2,4 = 122 t.
Gewicht waterkolom (zie blz. 4) = 790 ..
Gewicht vulstenen onder water (zie blz. 4) = 86 ..

Afschuifbel. Q_{afsch} = 998 ton

$Q_{R_v} = \frac{Q_{afsch}}{2} = \frac{998}{2} = 499$ t.

$\tau_{s\,PN} = \frac{499000}{56750} = 8{,}8$ kg/cm²

Fig. 52: Static calculation shaft barrier shaft I, ON I /36/

Furthermore a static calculation of inserted bulkheads in the insets on the 136 m floor is available /35/.

The coordinates of shaft ON I are:

RD-x:	196055
RD-y:	322643
elevation:	+109 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the property formally used by "CBS" (federal statistical office of the Netherlands, community Heerlen).

6.2 Shaft II, ON I

The vertical Shaft II of the pit Oranje Nassau I was drilled in 1894. In 1975 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 3,50 m diameter. The shaft was drilled to a total depth of 470,0 m and was used as upcast air shaft /38/. From the overburden to the carbon (level of -6,59 m NAP) the shaft consists of a tubbing support followed by masonry (thickness 0,5 m) /38/. The section between 0 m and 7 m was made of masonry (thickness 0,55 m) as well /38/. The shaft fittings are buntons, guide rails, eight electric cables, two pipes for compressed-air and two water pipelines /38/.

In this area the overburden has a thickness of 96,87 m and has a layering sequence of sand and clay /38/. The shaft has 14 documented insets. The 136 m floor, as the topmost is located in a level of -26,50 m NAP and in a depth of 135 m /6//50/.

In the following figure the strata in the range shaft II of the pit Oranje Nassau I is shown.

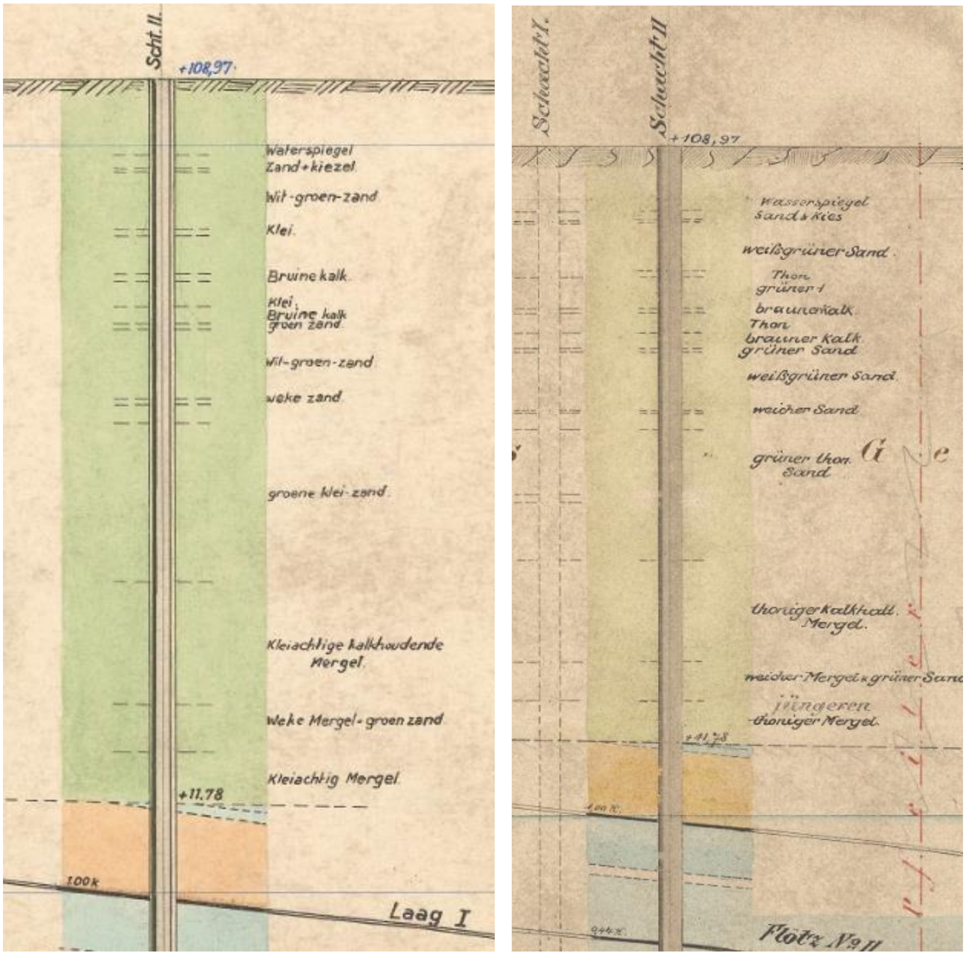


Fig. 53: Strata of the overburden shaft II, ON I /57/ /58/

In the following figure the strata in the range of the 136 m floor is pictured (here mainly slate and sandstone).

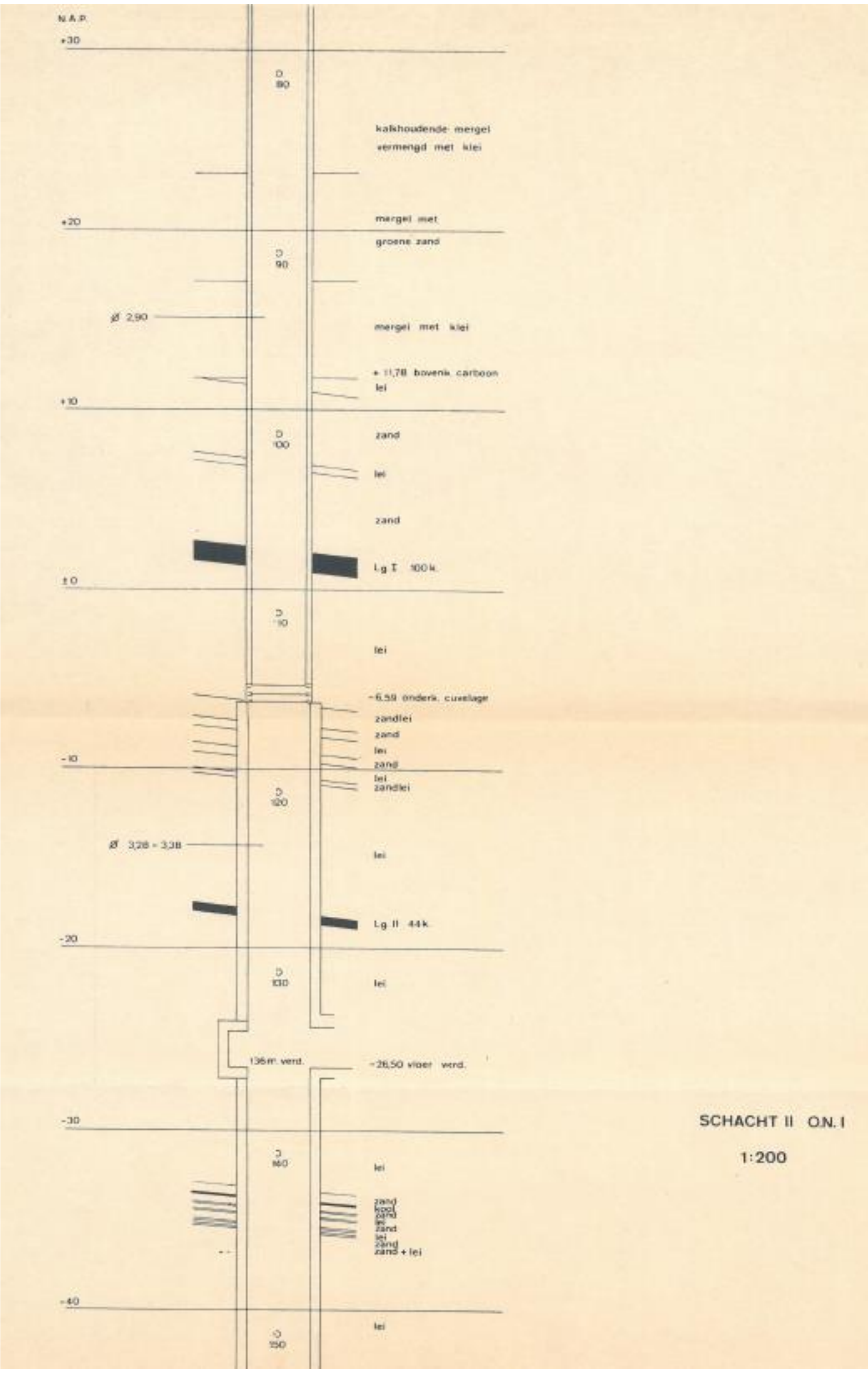


Fig. 54: Strata shaft II, ON I, 136 m floor /38/

In 1975 a load bearing filling out of 100 m³ of a mixture of concrete (length 8,2 m) was embedded in the 136 m floor on which the shaft has a cross-section of 3,3 m. Additionally about 2 m below the 136 m floor an abutment of iron beams covered by a concrete board (12 m³), which rests with its bend lower edge upon the surrounding rock was installed. The back stowing was carried out by the use of a drop pipe. Above the barrier the shaft column was backfilled with approximately 984 m³ waste material /15/ /35/ /38/. On the 120 m floor to provide a bearing for the waste material 40 m³ of concrete were backfilled /35/. For the use of ground water monitoring an observation pipeline was installed /15/. 1980 the shaft was provided with a concrete cover (thickness 0,4 m). In 1981 a monument of mining was set on the cover. The attached air drift was separated by a retaining wall /18//19//37//.

The following figures show the shaft barrier of shaft II of the pit Oranje Nassau I.

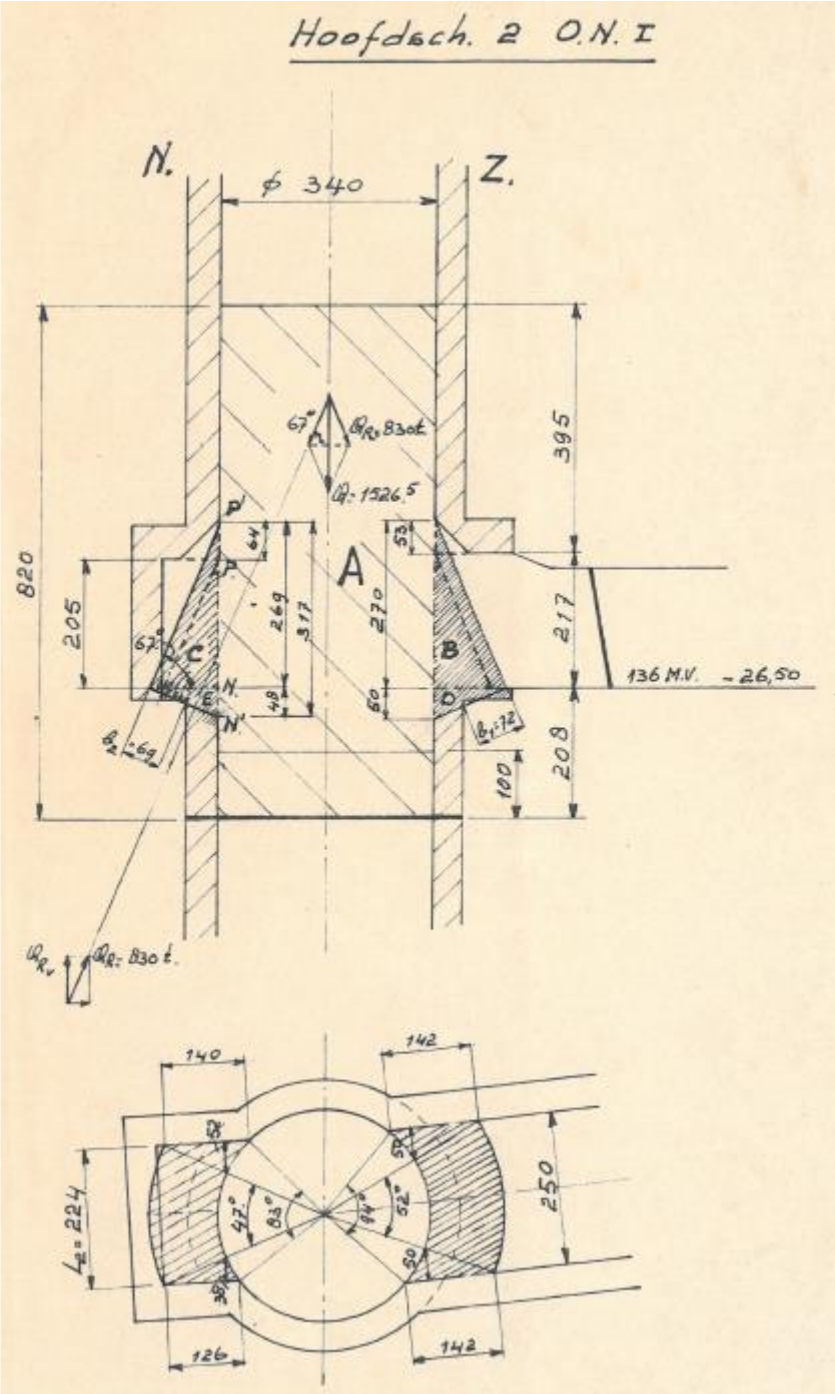


Fig. 55: Shaft barrier shaft II, ON I /38/

Static calculations of the shaft barrier of the shaft II, ON I are existent /36/. Compare the following figures.

Prop hoofdsch. 2 D.N. I

Diameter schacht = 3,40 m.
Opp. schachtdoorsn. = $\pi \times 1{,}7^2$ = 9,08 m²
Railvloer 137 m. onder maaiveld.

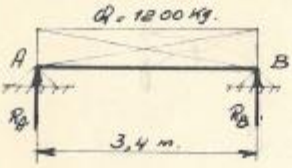
Prophoogte: 1e gedeelte prop, hoog 1 m., op railvloer.
Laten verharden.
Daarna 2e gedeelte storten tot 8,20 m. boven railvloer.
Totale hoogte prop H = 8,20 m.
s.g. beton = 2,4

Vloer uit rails N.P. 46

Railhoogte = 142 mm
railvoet br. = 120 mm.
railkop br. = 72 mm

W_x = 231 cm³
I_x = 1640 cm⁴
G. = 46,23 Kg/m'.

Middelste rail.



Belasting Q.

Q = gew. beton hoog 1 m. + e.g. rail.

$$Q = (3{,}4 \times 0{,}12 \times 1 \times 2{,}4) + \frac{46{,}23 \times 3{,}4}{1000}$$

Q = 1,14 t ∽ 1200 Kg.

Berekening: Op sterkte.

$$M_b = \frac{Q \cdot l}{8} = \frac{1200 \times 340}{8} = 50500 \text{ Kgcm.}$$

$$\sigma_b = \frac{M_b}{W_x} = \frac{50500}{231} = 222 \text{ Kg/cm}^2$$

Vlaktedruk oplegging rails in metselwerk van schachtwand.

Toelaatbare vlaktedruk $\bar{\sigma}_v$ metselwerk = 20 Kg/cm²
Lengte oplegging rail = 50 cm.

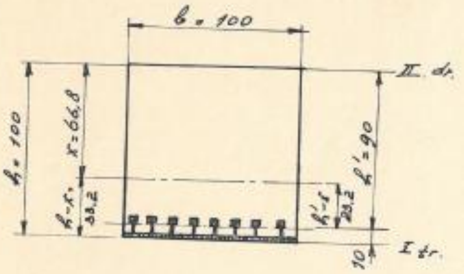
$$R_A = R_B = \frac{1200}{2} = 600 \text{ Kg.} \qquad F = 50 \times 12 = 600 \text{ cm}^2$$

$$\sigma_v = \frac{R_A}{F} = \frac{600}{600} = 1 \text{ Kg/cm}^2$$

Vloer na 1e stort, tot 1,00 m. dikte, berekend op buiging.

Wapening aaneengesloten vloer uit rails N.P. 46.

Voetbreedte 12 cm. F_y rail = 58 cm² F per m' = $\frac{100}{12} \times 58 = 485$ cm².
Totale hoogte prop $H = 8{,}20$ m. $n = \frac{E_y}{E_b} = 15$.



Belasting q.

$q_{bt} = H \times b \times s.g._b = 8{,}2 \times 1 \times 2400 = 19680$ Kg/m'

$eg_{ijzer} = 46{,}23 \times \frac{100}{12} =$ 385 "

$q = \pm\ 20100$ Kg/m'

$M_\ell = \frac{1}{8} \cdot Q \cdot \ell$ $\ell = 3{,}4 + 0{,}3 = 3{,}7$ m.

$M_\ell = \frac{1}{8}(20100 \times 3{,}7)\,3{,}7 = 34350$ Kgm $M_\ell = 3.435.000$ Kgcm.

Bepaling zwaartepunt x.

$$x = \frac{\Sigma F \times z}{\Sigma F}$$

$$x = \frac{\frac{1}{2} b.h^2 + n.F_y \times h'}{b.h + n.F_y}$$

$$x = \frac{(\frac{1}{2} \times 100 \times 100^2) + (15 \times 485 \times 90)}{100 \times 100 + 15 \times 485} = \frac{500.000 + 655.000}{10.000 + 7280} \rightarrow x = 66{,}8 \text{ cm}$$

$$I_{id} = I_{bt} + n.I_y$$

$$= \frac{1}{3} b.x^3 + \frac{1}{3} b(h-x)^3 + n.F_y(h'-x)^2$$

$$= (\frac{1}{3} \times 100 \times 66{,}8^3) + (\frac{1}{3} \times 100 \times 33{,}2^3) + (15 \times 485 \times 23{,}2^2) \rightarrow I_{id} = 15.090.000 \text{ cm}^4$$

Drukzijde:

$$\sigma_{bd} = \frac{M}{W_{II}} = \frac{M \times x}{I_{id}} = \frac{3.435.000 \times 66{,}8}{15.090.000} \rightarrow \sigma_{bd} = 15{,}3 \text{ Kg/cm}^2$$

Trekzijde:

$$\sigma_{bt} = \frac{M}{W_I} = \frac{M.(h-x)}{I_{id}} = \frac{3.435.000 \times 33{,}2}{15.090.000} \rightarrow \sigma_{bt} = 7{,}6 \text{ Kg/cm}^2$$

$$\sigma_{yt} = n \times \sigma_{bt} = 15 \times 7{,}6 \rightarrow \sigma_{yt} = 114 \text{ Kg/cm}^2$$

Max. schuifspanning in prop nadat de schacht gevuld is.

τ_s toelaatb = 10 kg/cm²

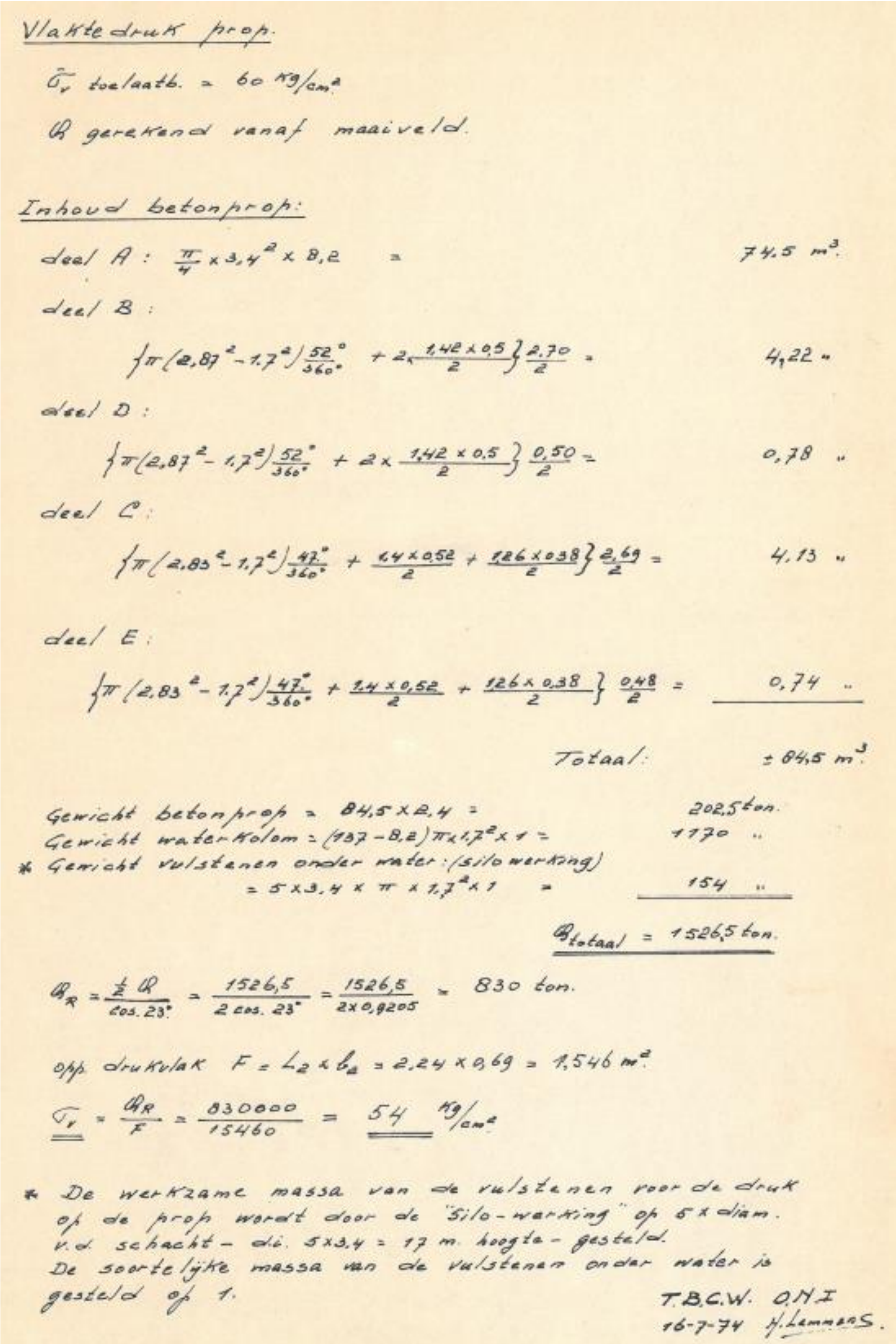
Afschuifvlak P.N. dient vergroot te worden

tot P'N' = 3.17 m.

$F_{P'N'} = P'N' \times L = 3.17 \times 2.47 = 7.83\ m^2 = 78\,300\ cm^2$

$\varphi_{Rv} = 751\,500\ kg$

$\tau_{s\,P'N'} = \frac{\varphi_{Rv}}{F_{P'N'}} = \frac{751\,500}{78\,300} = 9.6\ kg/cm^2$



Vlaktedruk prop.

$\bar{\sigma}_v$ toelaatb. = 60 kg/cm²

Q gerekend vanaf maaiveld.

Inhoud betonprop:

deel A: $\frac{\pi}{4} \times 3{,}4^2 \times 8{,}2$ = 74.5 m³.

deel B:

$\{\pi(2{,}87^2 - 1{,}7^2)\frac{52°}{360°} + 2 \times \frac{1{,}42 \times 0{,}5}{2}\}\frac{2{,}70}{2}$ = 4,22 "

deel D:

$\{\pi(2{,}87^2 - 1{,}7^2)\frac{52°}{360°} + 2 \times \frac{1{,}42 \times 0{,}5}{2}\}\frac{0{,}50}{2}$ = 0,78 "

deel C:

$\{\pi(2{,}83^2 - 1{,}7^2)\frac{47°}{360°} + \frac{1{,}4 \times 0{,}52}{2} + \frac{1{,}26 \times 0{,}38}{2}\}\frac{2{,}69}{2}$ = 4,13 "

deel E:

$\{\pi(2{,}83^2 - 1{,}7^2)\frac{47°}{360°} + \frac{1{,}4 \times 0{,}52}{2} + \frac{1{,}26 \times 0{,}38}{2}\}\frac{0{,}48}{2}$ = 0,74 "

Totaal: ± 84,5 m³.

Gewicht betonprop = 84,5 × 2,4 = 202,5 ton.
Gewicht waterkolom = $(137 - 8{,}2)\pi \times 1{,}7^2 \times 1$ = 1170 "
* Gewicht vulstenen onder water: (silowerking)
= $5 \times 3{,}4 \times \pi \times 1{,}7^2 \times 1$ = 154 "

Q_{totaal} = 1526,5 ton.

$Q_R = \frac{\frac{1}{2} Q}{\cos 23°} = \frac{1526{,}5}{2 \cos 23°} = \frac{1526{,}5}{2 \times 0{,}9205}$ = 830 ton.

opp. drukvlak $F = L_2 \times b_2 = 2{,}24 \times 0{,}69 = 1{,}546$ m².

$\sigma_v = \frac{Q_R}{F} = \frac{830000}{15460} = 54$ kg/cm²

* De werkzame massa van de vulstenen voor de druk op de prop wordt door de "Silo-werking" op 5 × diam. v.d. schacht – d.i. 5×3,4 = 17 m. hoogte – gesteld.
De soortelijke massa van de vulstenen onder water is gesteld op 1.

T.B.C.W. O.N.I
16-7-74 H. Lemmens.

Fig. 56: Static calculation shaft barrier shaft II, ON I /38/

Furthermore a static calculation of inserted bulkheads in the insets on the 136 m floor is available /38/.

The coordinates of shaft II are:

RD-x:	196019
RD-y:	322661
elevation:	+109 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located beneath a former shaft building, which now is a monument of mining /35/.

6.3 Shaft III, ON I

The vertical Shaft III of the pit Oranje Nassau I was drilled in 1905. In 1975 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 3,80 m diameter. The shaft was drilled to a total depth of 441,0 m and was used as drafting shaft /35/. The shaft wall was made of masonry (thickness 0,5 m). Within the overburden the shaft consists of tubbing support /35/. The shaft fittings are buntons, guide rails, three electric cables, one pipe for compressed-air and one water pipeline /35/.

In this area the overburden has a thickness of 96,64 m /35/. The shaft has 10 documented insets. The 136 m floor, as the topmost is located in a level of -26,50 m NAP and in a depth of 135 m /6//50/. In the following figure the strata in the range of the 136 m floor is pictured (here mainly slate and sandstone).

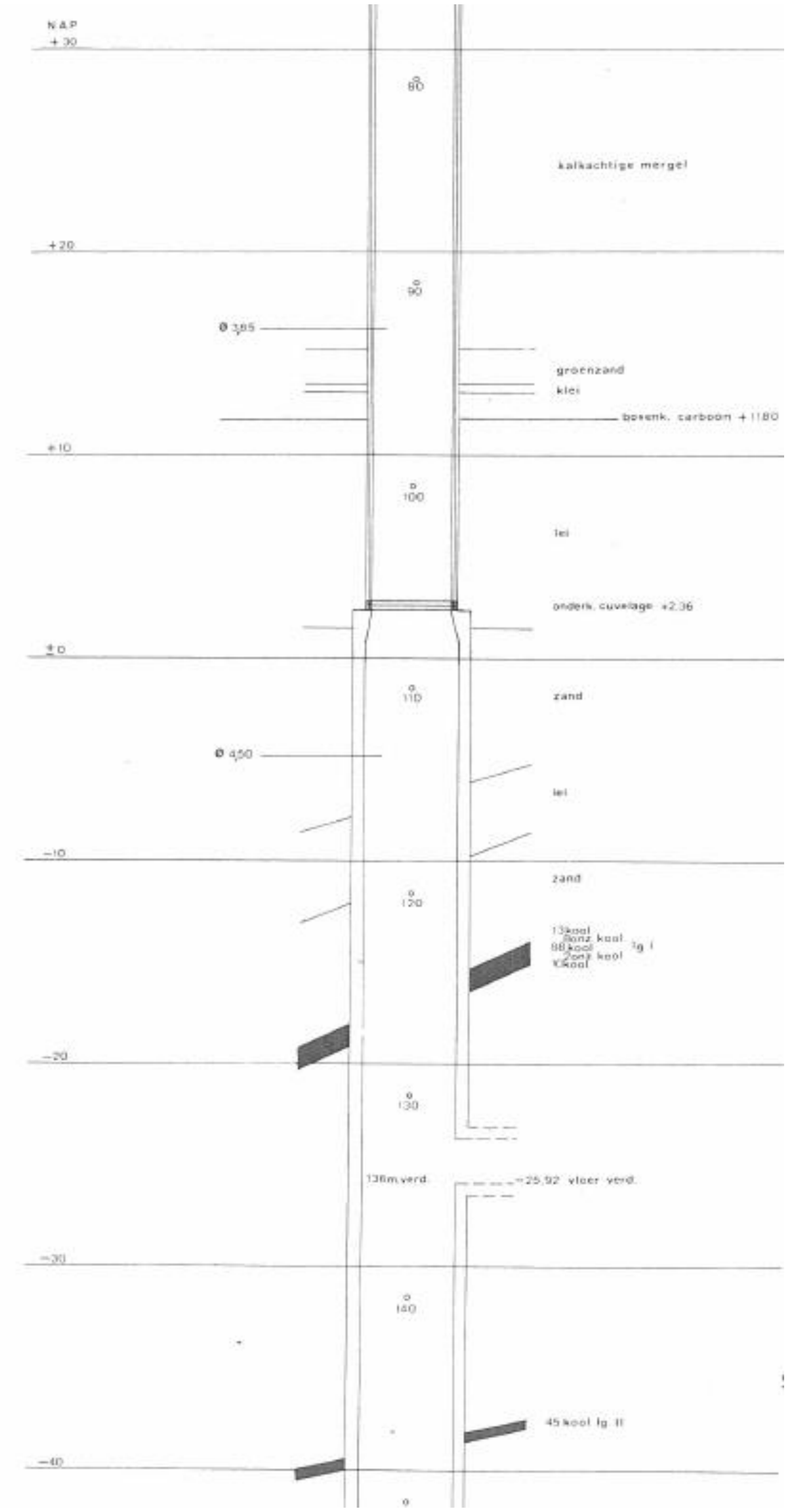


Fig. 57: Strata shaft III, ON I, 136 m floor /35/

In 1975 a load bearing filling out of 200 m³ of a mixture of concrete (length 10 m; 325 kg cement/m³ concrete) was embedded in the 136 m floor (-26,50 m NAP) on which the shaft has a cross-section of 4,5 m. Additionally about 2 m below the 136 m floor an abutment of iron beams covered by a concrete board (thickness 1,5 m; 24 m³), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. The back stowing was carried out in three segments of 55 m³, 50 m³ and 70 m³ by the use of a drop pipe. Hereby a connected waste material dugout was backfilled with concrete as well. Above this barrier the shaft column was backfilled with a total of 1.904 m³ waste material /15/. 1980 the shaft was provided with a concrete cover (thickness 0,5 m) 1,5 m below ground surface /35/. On top the cover was overlaid with 2 m of waste material and 0,8 m of topsoil /37/. The opening for refilling was closed with concrete /18/.

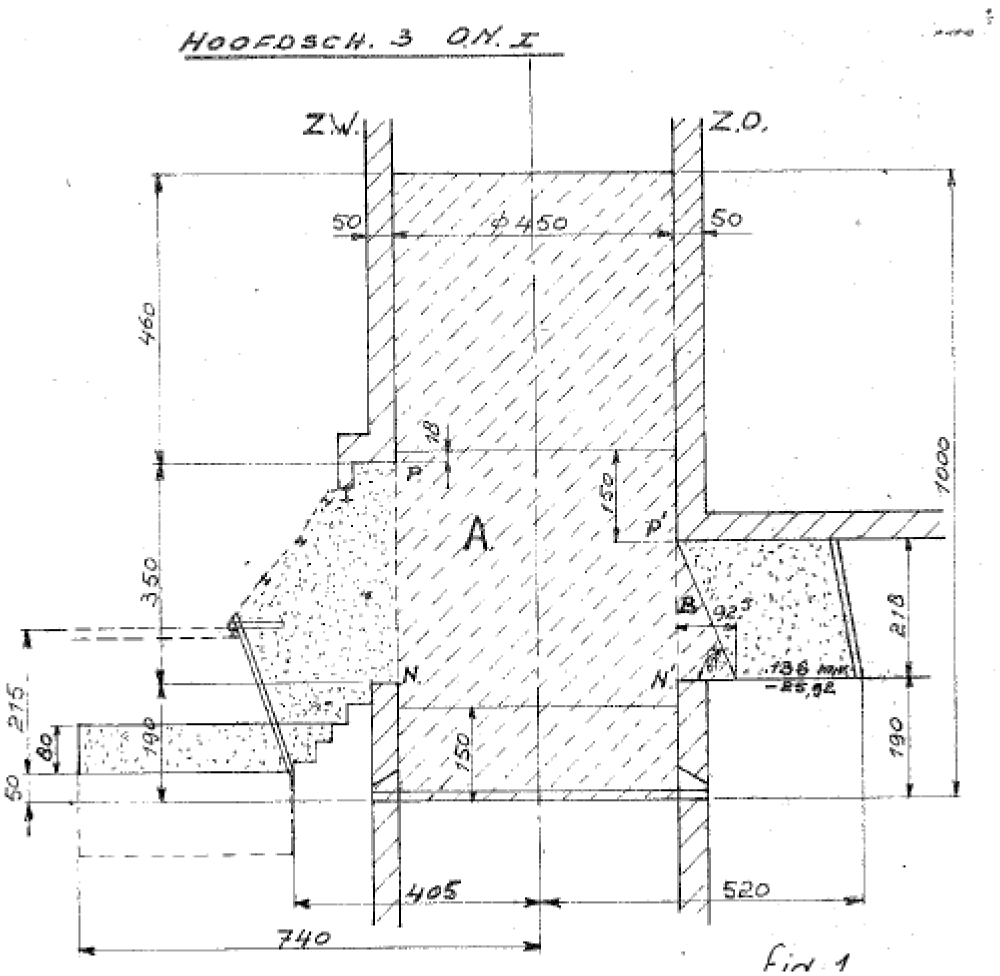


Fig. 58: Shaft barrier shaft III, ON I /35/

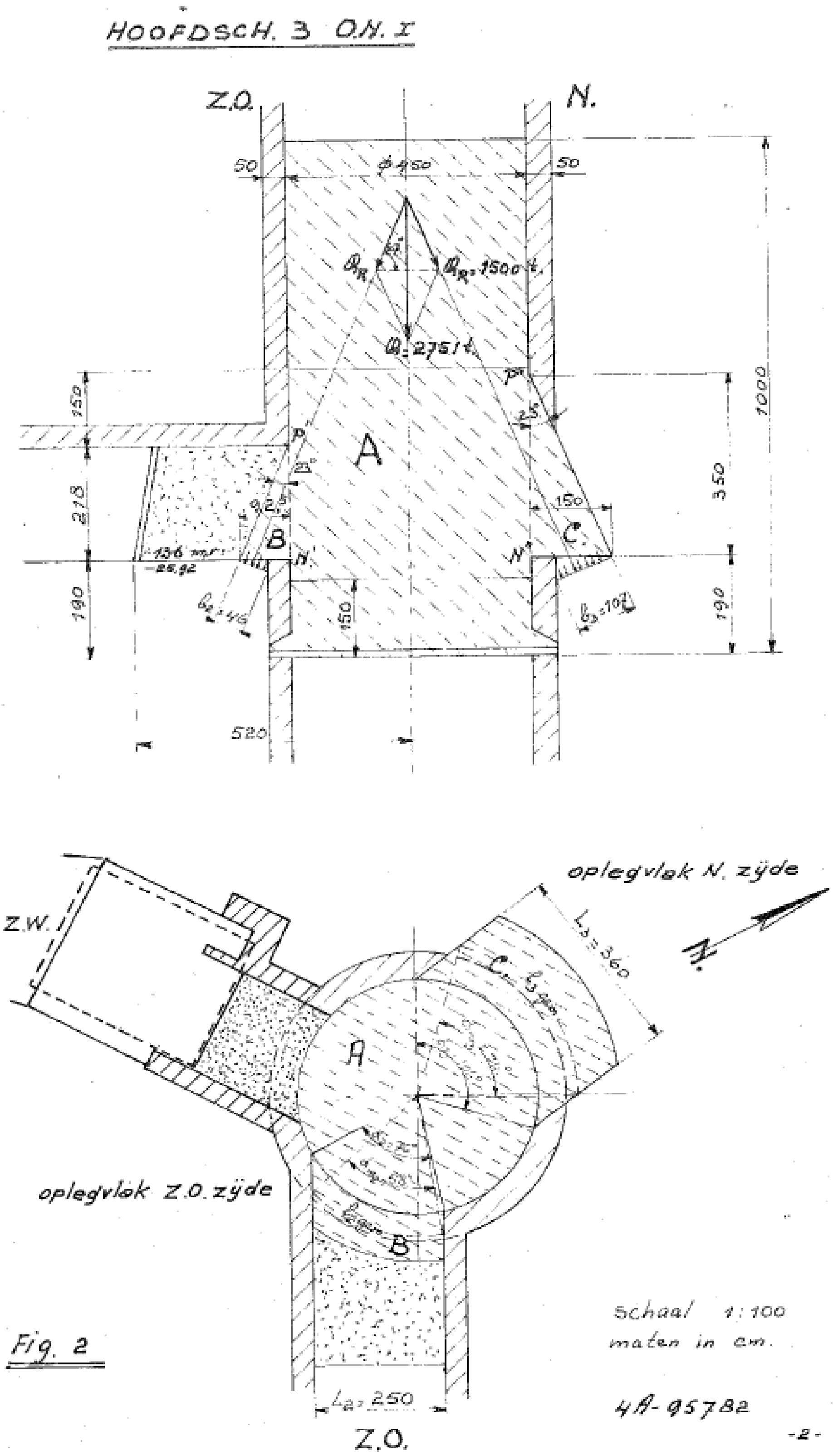


Fig. 59: Shaft barrier shaft III, ON I /35/

Static calculations of the shaft barrier of the shaft III, ON I are existent /35/. Compare the following figures.

Prop hoofdsch. 3 O.N. I

Diameter schacht = 4,50 m.

Opp. schachtdoorsn. = $\pi \times 2{,}25^2$ = 15,9 m^2.

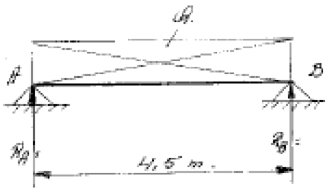
Railvloer 134,36 m. onder maaiveld.

Prophoogte: 1e gedeelte prop, hoog 1,50 m., op railvloer. Laten verharden.
Daarna 2e gedeelte storten tot 1,50 m. boven dak laadplaats Z.O.
Dan rest gedeelte storten tot 10 m. boven railvloer.
Totale hoogte prop H = 10 m.
s.g. beton = 2,4.

Vloer uit rails N.P. 46

Railhoogte = 142 mm.
railvoet br. = 120 mm.
railkop br. = 72 mm.
} $W_x = 231\ cm^3$, $I_x = 1640\ cm^4$, $G = 46{,}23\ Kg/m$.

Middelste rail.



Belasting Q.

Q = gew. beton, hoog 1.50 m + e.g. rail

$$Q = (4{,}5 \times 0{,}12 \times 1{,}5)\,2{,}4 + \frac{46{,}23 \times 4{,}5}{1000} =$$

Q = 2,2 t = 2200 Kg.

Berekening op sterkte

$$M_l = \frac{Q \cdot l}{8} = \frac{2200 \times 450}{8} = 124000\ Kgcm.$$

$$\sigma_b = \frac{M_l}{W_x} = \frac{124000}{231} = 540\ Kg/cm^2$$

Vlaktedruk oplegging rails in metselwerk van schachtwand.

Toelaatbare vlaktedruk σ_v metselwerk = 20 Kg/cm^2

Lengte oplegging rail = 50 cm.

$$R_A = R_B = \frac{2200}{2} = 1100\ Kg. \qquad F = 50 \times 12 = 600\ cm^2$$

$$\sigma_v = \frac{R_A}{F} = \frac{1100}{600} = 1{,}83\ Kg/cm^2.$$

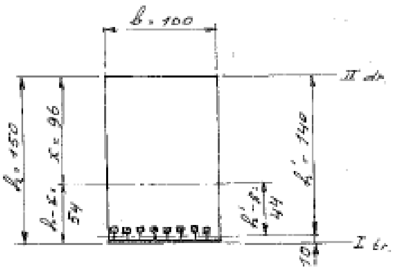
- 2 -

Vloer na 1e stort, tot 1,50 m. dikte, berekend op buiging.

Wapening aaneengesloten vloer uit rails N.P. 46.

Voetbreedte 12 cm. $F_{ij\ rail} = 58\ cm^2$ F per m' $= \frac{100}{12} \times 58 = 485\ cm^2$
Totale hoogte prop $H = 10$ m. $n = \frac{E_{ij}}{E_b} = 15$.

Belasting: q.



$q_{be} = H \times b \times s.g._{be} = 10 \times 1 \times 2400 = 24000\ Kg/m'$

$e.g._{ijzer} = 46,23 \times \frac{100}{12} = \qquad 385\ "$

$q = \pm\ 24400\ Kg/m'$

$M_b = \frac{1}{8} \cdot q \cdot l$ $\qquad l = 4,5 + 0,3 = 4,8$ m.

$M_b = \frac{1}{8}(24400 \times 4,8)\ 4,8 = 70300\ Kgm \rightarrow M_b = 7.030.000\ Kgcm$

Bepaling zwaartepunt x.

$x = \frac{\Sigma F \times Z}{\Sigma F}$

$x = \frac{\frac{1}{2} b . h^2 + n \times F_{ij} . h'}{b . h + n . F_{ij}}$

$x = \frac{(\frac{1}{2} \times 100 \times 150^2) + (15 \times 485 \times 140)}{100 \times 150 + 15 \times 485} = \frac{1125.000 + 1020.000}{15000 + 7280} \rightarrow x = 96\ cm.$

$I_{id} = I_{be} + n . I_{ij}$

$= \frac{1}{3} b . x^3 + \frac{1}{3} b . (h - x)^3 + n . F_{ij} (h' - x)^2$

$= (\frac{1}{3} \times 100 \times 96^3) + (\frac{1}{3} \times 100 \times 54^3) + (15 \times 485 \times 44^2) \rightarrow I_{id} = 48.824.400\ c.$

Drukzijde:

$\sigma_{bd} = \frac{M}{W_{II}} = \frac{M \times x}{I_{id}} = \frac{7.030.000 \times 96}{48.824.000} \rightarrow \sigma_{bd} = 13,8\ Kg/cm^2$

Trekzijde:

$\sigma_{bt} = \frac{M}{W_I} = \frac{M \times (h - x)}{I_{id}} = \frac{7030000 \times 54}{48.824.000} \rightarrow \sigma_{bt} = 7,8\ Kg/cm^2$

$\sigma_{ijt} = n \times \sigma_{bt} = 15 \times 7,8 \rightarrow \sigma_{ijt} = 117\ Kg/cm^2$

Vlaktedruk prop.

$\bar{\sigma}_v$ toelaatb. = 60 kg/cm²

Q gerekend vanaf maaiveld.

In verband met de zich aan Z.W. zijde bevindende bunker, (zie bijlage 5 doorsn. 1-2), wordt het oplegvlak aan Z.W.-zijde voor de berekening op vlaktedruk, buiten beschouwing gelaten.

Uit voorenstaande volgt dus dat aan Noordzijde een nieuw oplegvlak voor de prop, breed 3,60 m., gemaakt moet worden. zie fig. 8.

Inhoud beton prop:

deel A : $\frac{\pi}{4} \times 4{,}5^2 \times 10$ . . . . 159 m³

deel B : $l_{2gem.} \times b_2 \times \frac{h_{P'N'gem.}}{2}$ =

$(\pi \times R_{m_2} \times \frac{\alpha_{m2}}{180}) b_2 \times \frac{h_{P'N'gem.}}{2}$

$\pi(2{,}25 + \frac{0{,}925}{2})\frac{58}{180} \times 0{,}925 \times \frac{1{,}96}{2}$ = ... 2,48 "

deel C : $l_{3gem.} \times b_2 \times \frac{h_{P''N''}}{2}$ =

$(\pi \times R_{m_3} \times \frac{\alpha_{m3}}{180}) b_2 \times \frac{h_{P''N''}}{2}$ =

$\pi(2{,}25 + \frac{1{,}5}{2})\frac{74}{180} \times 1{,}5 \times \frac{3{,}5}{2}$ = ... 10,20 "

Inhoud totaal = ∽ 172 m³

Gewicht betonprop = 172 × 2,4 = 413 t.
Gewicht waterkolom = $(134{,}36 - 10)\frac{\pi}{4} \times 4{,}5^2 \times 1$ = 1980 "
* Gewicht vulstenen onder water: (silo werking):
$(5 \times 4{,}5) \times \frac{\pi}{4} \times 4{,}5^2 \times 1$ = 358 "

Q_{totaal} = 2751 t.

* De werkzame massa van de vulstenen, voor de druk op de prop, wordt door de „silo"-werking op 5 × diam. v.d. schacht – d.i. 5 × 4,5 m. = 22,5 m. hoogte – gesteld.
De soortelijke massa van de vulstenen onder water is gesteld op 1.

$$Q_R = \frac{\frac{1}{2} Q_{tot}}{\cos. 23^\circ} = \frac{2757}{2 \cos. 23} = \frac{2751}{2 \times 0{,}9205} = 1500\ t.$$

$$Q_{R_{tot}} = 2 \times Q_R = 2 \times 1500 = 3000\ t.$$

Opp. drukvlakken = $F = F_B + F_C$ (resp. Z.O. en N.)

$$F_B = b_2 \times L_2 = 0{,}46 \times 2{,}5 = 1{,}15\ m^2$$

$$F_C = b_3 \times L_3 = 1{,}07 \times 3{,}6 = 3{,}85$$

$$F = 5{,}00\ m^2 = 50000\ cm^2$$

$$\sigma_v = \frac{Q_{R\,tot}}{F} = \frac{3000000}{50000} = 60\ kg/cm^2$$

Max. schuifspanning in prop, nadat schacht gevuld is.

· vlak: PN + P'N' + P"N" (resp. Z.W.; Z.O.; en N.)

τ_s toelaatb. = 10 kg/cm²

Z.W.: Gemidd. hoogte v. afschuifvlak PN = h_{m_1} (doorsn. 3-4 bijlage 5)

$h_{m_1} = \frac{F}{L_1} = \frac{68770}{210} = 327{,}5$ cm.

Lengte afschuifvlak: L_{PN}

$L_{PN} = \pi \cdot D \cdot \frac{\alpha_1}{360^\circ} = \pi \times 450 \times \frac{57}{360} = 224$ cm.

Z.O.: Gemidd. hoogte v. afschuifvlak P'N': h_{m_2} (doorsn. 5-6 bijlage 5

$h_{m_2} = \frac{F}{L_2} = \frac{49150}{250} = 196$ cm.

Lengte afschuifvlak = $L_{P'N'}$

$L_{P'N'} = \pi \cdot D \cdot \frac{\alpha_2}{360^\circ} = \pi \times 450 \times \frac{76}{360} = 298$ cm.

N.: Hoogte v. afschuifvlak P"N" = h_{m_3}

$h_{m_3} = 350$ cm.

Lengte afschuifvlak = $L_{P''N''}$

$L_{P''N''} = \pi \cdot D \cdot \frac{\alpha_3}{360} = \pi \times 450 \times \frac{106}{360} = 416$ cm.

Opp. afschuifvlakken: $F = F_{PN} + F_{P'N'} + F_{P''N''}$

$F_{PN} = h_{m_1} \times L_{PN} = 327{,}5 \times 224 = 73500$ cm²

$F_{P'N'} = h_{m_2} \times L_{P'N'} = 196 \times 298 = 58500$ "

$F_{P''N''} = h_{m_3} \times L_{P''N''} = 350 \times 416 = 145500$ "

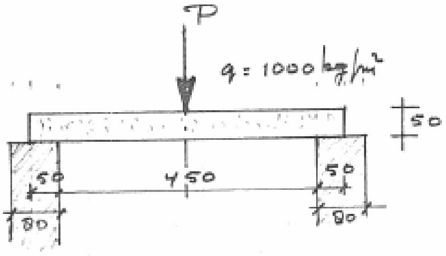
$F_{tot.} = 277500$ cm²

Fig. 60: Static calculations of the shaft barrier of the shaft III, ON I /35/

Furthermore a static calculation of inserted bulkheads in the insets on the 136 m floor is available /35/. Compare the following figure.

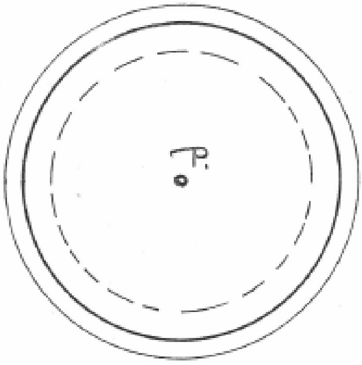
Berekening

Afdekplaat schacht 3 O.N. Mijn I te Heerlen



Beton K. 225
Wapening. GR. 24

$\ell_x = \ell_y = 450 + 50 = 500$ cm
Dikte plaat = 50 cm
$h_x = 50 - 2,5 - 0,8 = 46,7$ cm
$h_y = 50 - 2,5 - 1,6 - 0,8 = 45,1$ cm
$b_x = b_y = 0$ voor P.



e.g. plaat 0,5 x 2400 = 1200 kg/m²
nl 1000 kg/m²
q = 2200 kg/m²

$M_{xq} = M_{yq} = 0{,}044 \times 2200 \times 5^2 = 2420$ kgm
$M_{xP} = M_y P = \frac{1}{3} \times 10.000 = 3333$ "
M_x tot $= M_y$ tot $= 5753$ kgm

$\kappa = h \sqrt{\frac{b}{m}} = 45{,}1 \sqrt{\frac{1}{5753}} = 0{,}595$

$A_x = A_y = 0{,}076 \times \frac{m}{h}$

$= 0{,}0767 \times \frac{5753}{45{,}1} = 9{,}78$ cm²

onder wap ϕ16-20 = 10,05 cm²
boven wap ϕ12-20 = 5,65 cm²

Schuifspanning

Totale belasting $= \frac{1}{4} \pi \times 5^2 \times 1000 + 10.000 = 29.635$ kg
omtrek: $\pi \times 450 = 1414$ cm

$\tau_s = \frac{3 \times 29635}{2 \times 1414 \times 50} = 0{,}63$ kg/cm²

Oplegdruk

omtrek $= \pi \times 500 = 1571$ cm Opp $= 1571 \times 50 = 78550$ cm²

τ middelwerk 29.635 : 78550 = 0,38 kg/cm²

Maatschappij tot Exploitatie
van Onroerend Goed Oranje-Nassau
(O.G.O.N.) B.V.

23-10-78

[signature]

Fig. 61: Static calculation of the shaft cover shaft III,ON I /35/

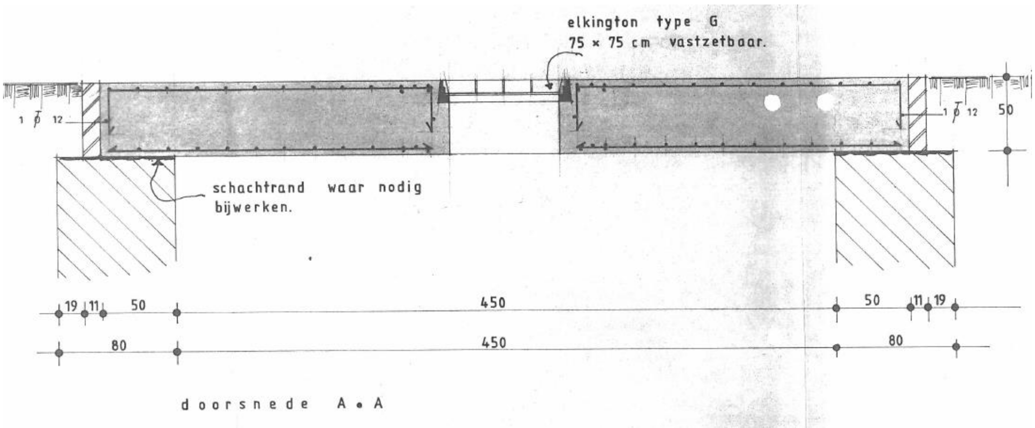


Fig. 62: Sectional drawing shaft cover shaft III, ON I /35/

The coordinates of shaft III are:

RD-x:	195874
RD-y:	322783
Elevation:	+109 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the property formally used by "CBS" (federal statistical office of the Netherlands, community Heerlen). On top of the shaft in 2009 a 15 m pillar made of glass as artworks was set up /35/.

6.4 Shaft I, ON II

The vertical Shaft I of the pit Oranje Nassau II was drilled in 1898. In 1971 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,0 m diameter. The shaft was drilled to a total depth of 477,0 m and was used as drawing shaft. From the overburden to the carbon (level of +13,40 m NAP) the shaft consists of a tubbing support followed by masonry (thickness 0,5 m) /36/. The shaft fittings are buntons, guide rails, some electric cables, one pipe for compressed-air and two water pipelines /39/.

In this area the overburden has a thickness of 129,74 m and has a layering sequence of sand, clay and lignite /36/. The carbon is located on a level of +22,60 m NAP /53/.The shaft has 16 documented insets /39/. The 163 m floor, as the topmost is located in a level of -10,40 m NAP and in a depth of 163 m /6//50/.

In the following figure the strata in the range of the 136 m floor is pictured (here mainly slate and sandstone with intercalated hard coal beds).

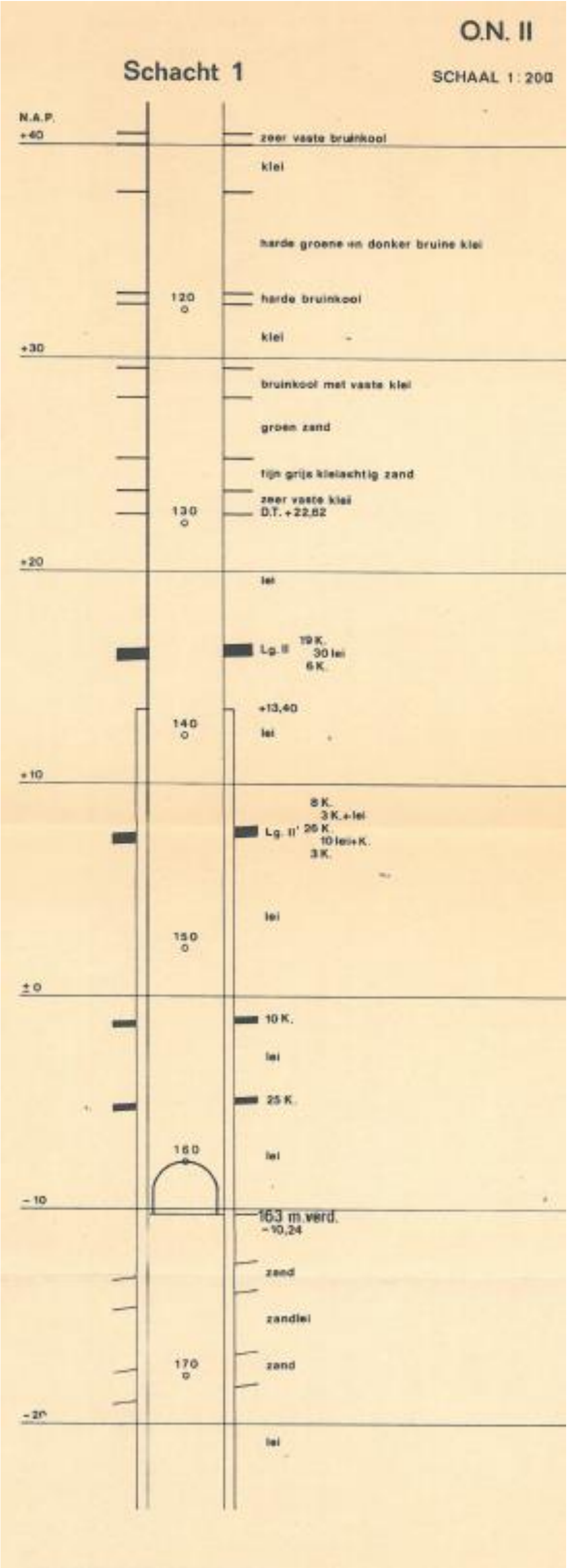


Fig. 63: Strata shaft I, ON II, 163 m floor /39/

In 1971 a load bearing filling out of 125 m³ of a mixture of concrete (length 10 m) was embedded in the 163 m floor (-10,40 m NAP).

Additionally an abutment of iron beams covered by a concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. For the constructed two-way slab an abutment had to be embedded in the carbon on one side of the shaft-landing. The concrete was backfilled by free fall technique using an existing water pipeline. Above the barrier the shaft was backfilled completely with approximately 1.925 m³ waste material of the grain size 0-120 mm. Until the end of 1971 respectively 1972 the shaft column subsided 0,01 m each time /11//12/. In 1972 the shaft was provided with a reinforced concrete cover (0,25 m below floor) with an opening for refilling /4/ /11/ /12/ /53/.

The following figure shows the shaft cover.

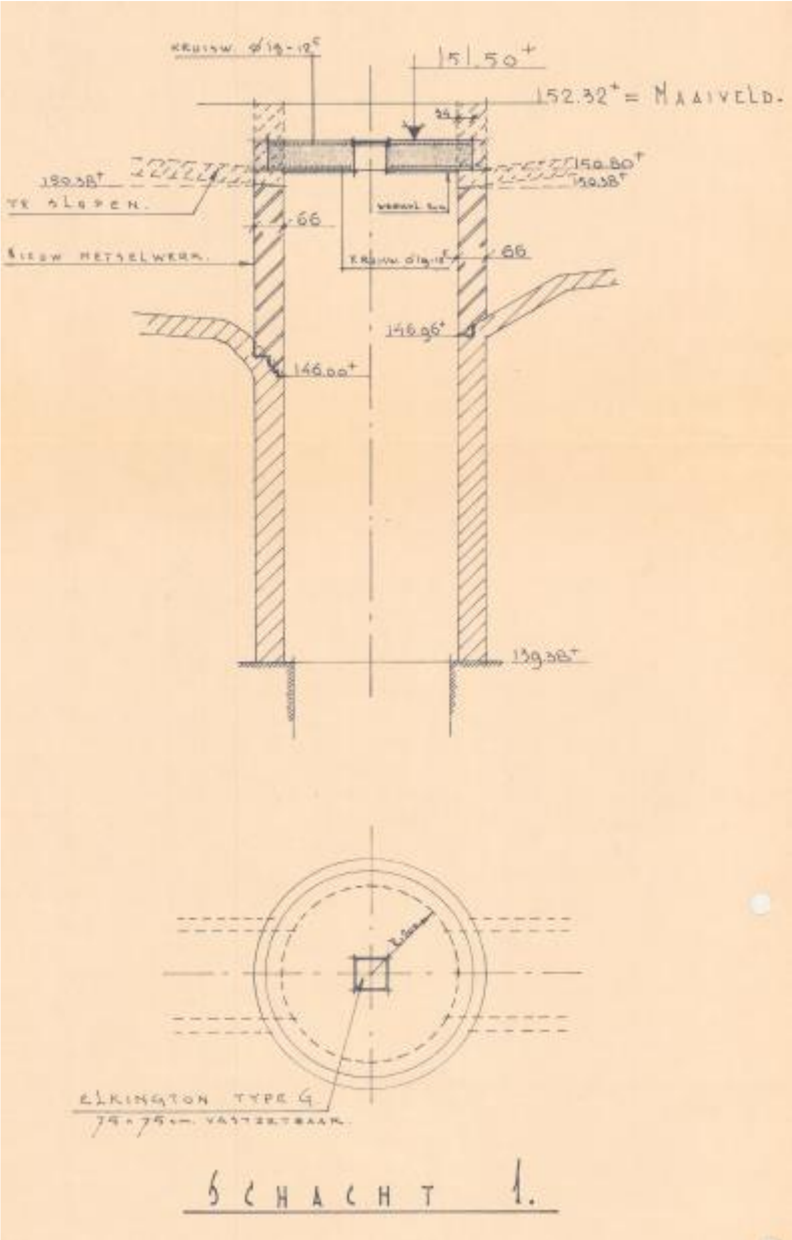


Fig. 64: Shaft cover shaft I, ON II /53/

The coordinates of shaft I are:

RD-x:	199322
RD-y:	321717
elevation:	+153 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space at Aan de Schacht and Koelmoer (community Landgraaf).

6.5 Shaft II, ON II

The vertical Shaft II of the pit Oranje Nassau II was drilled in 1898. In 1971 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 5,40 m diameter. The shaft was drilled to a total depth of 433,0 m. Within the overburden the shaft consists of tubbing support /50/. The shaft fittings are buntons, guide rails, some electric cables, one pipe for compressed-air and one water pipeline /39/.

In this area the overburden has a thickness of 131 m and has a layering sequence of sand, clay and lignite /53/. The carbon is located on a level of +23,0 m NAP /53/. The shaft II has 16 documented insets. The 163 m floor, as the topmost is located in a level of -10,40 m NAP and in a depth of 163 m /6//50/.

In the following figure the strata in the range of the 136 m floor is pictured (here mainly slate and sandstone with intercalated hard coal beds).

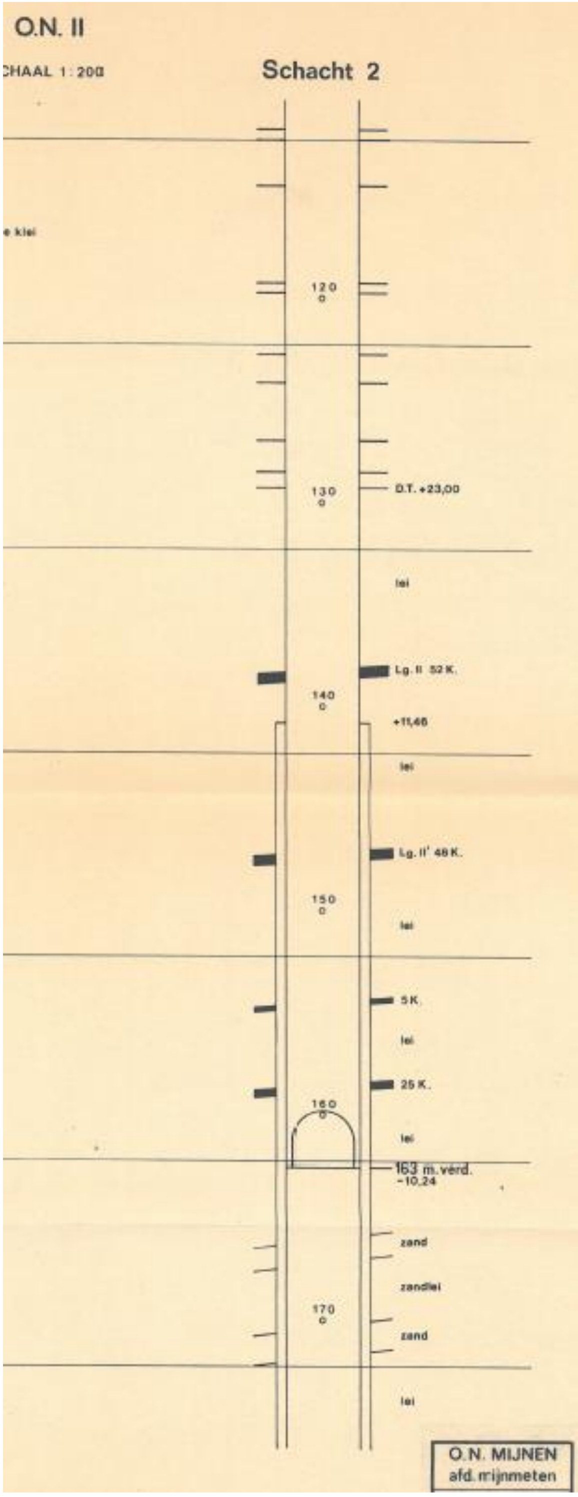


Fig. 65: Strata shaft II, ON II, 163 m floor /39/

In 1971 a load bearing filling out of 230 m³ of a mixture of concrete (length 10 m) was embedded in the 163 m floor.

Additionally an abutment of iron beams covered by a concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. For the constructed two-way slab an abutment had to be embedded in the carbon on one side of the shaft-landing. The concrete was backfilled by free fall technique using an existing pipe for compressed-air. Above the barrier the shaft was backfilled completely with approximately 3.500 m³ waste material of the grain size 0-120 mm. Until the end of 1971 respectively 1972 the shaft column subsided 0,01 m each time /11//12/. In 1972 the shaft was provided with a reinforced concrete cover (0,71 m below floor) with an opening for refilling /4/ /11/ /12/ /53/.

The following figure shows the shaft barrier of the shaft II of the pit Oranje Nassau II.

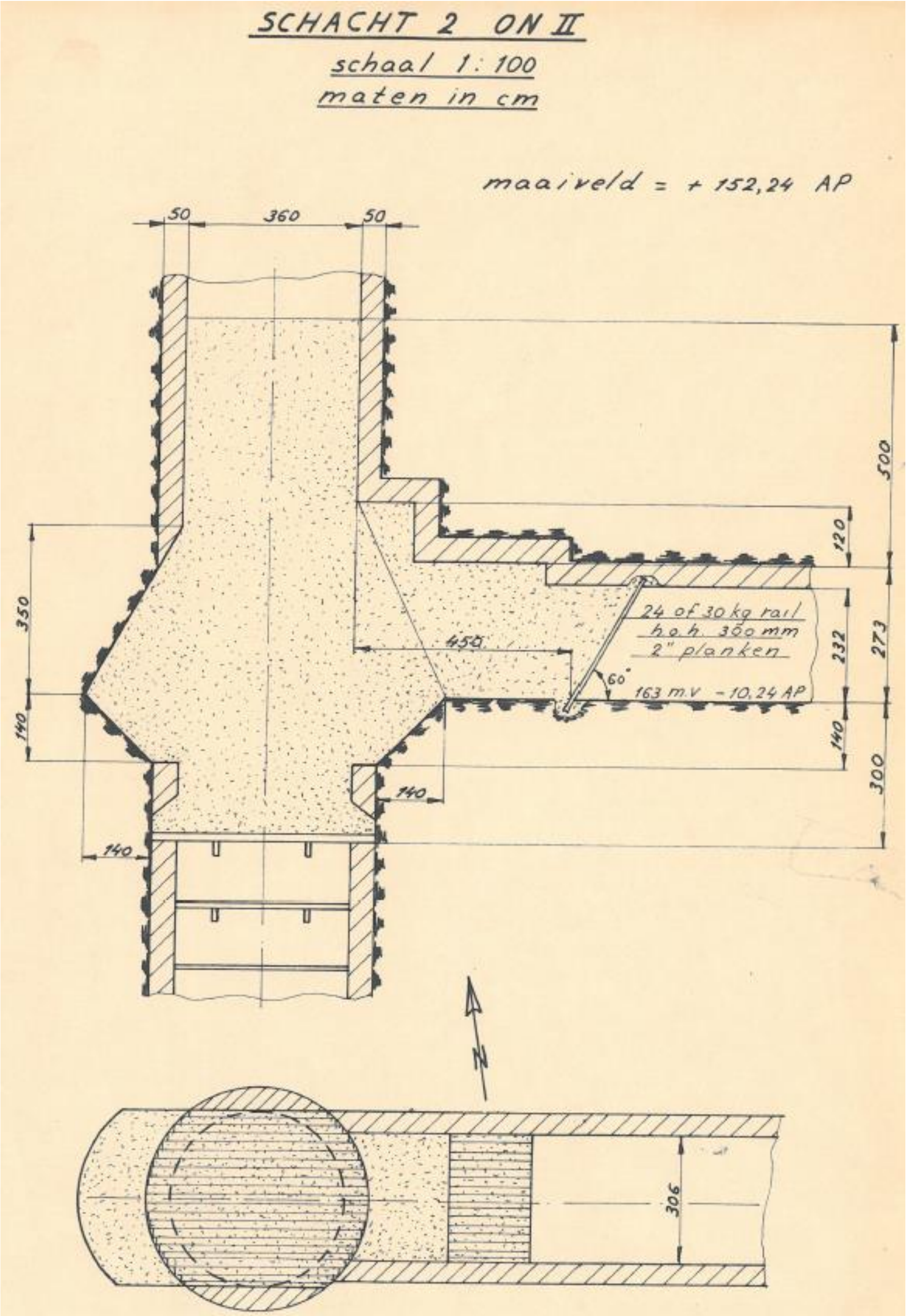


Fig. 66: Shaft barrier shaft II, ON II /39/

Static calculations of the shaft barrier of the shaft II, ON II are existent /39/. Compare the following figures.

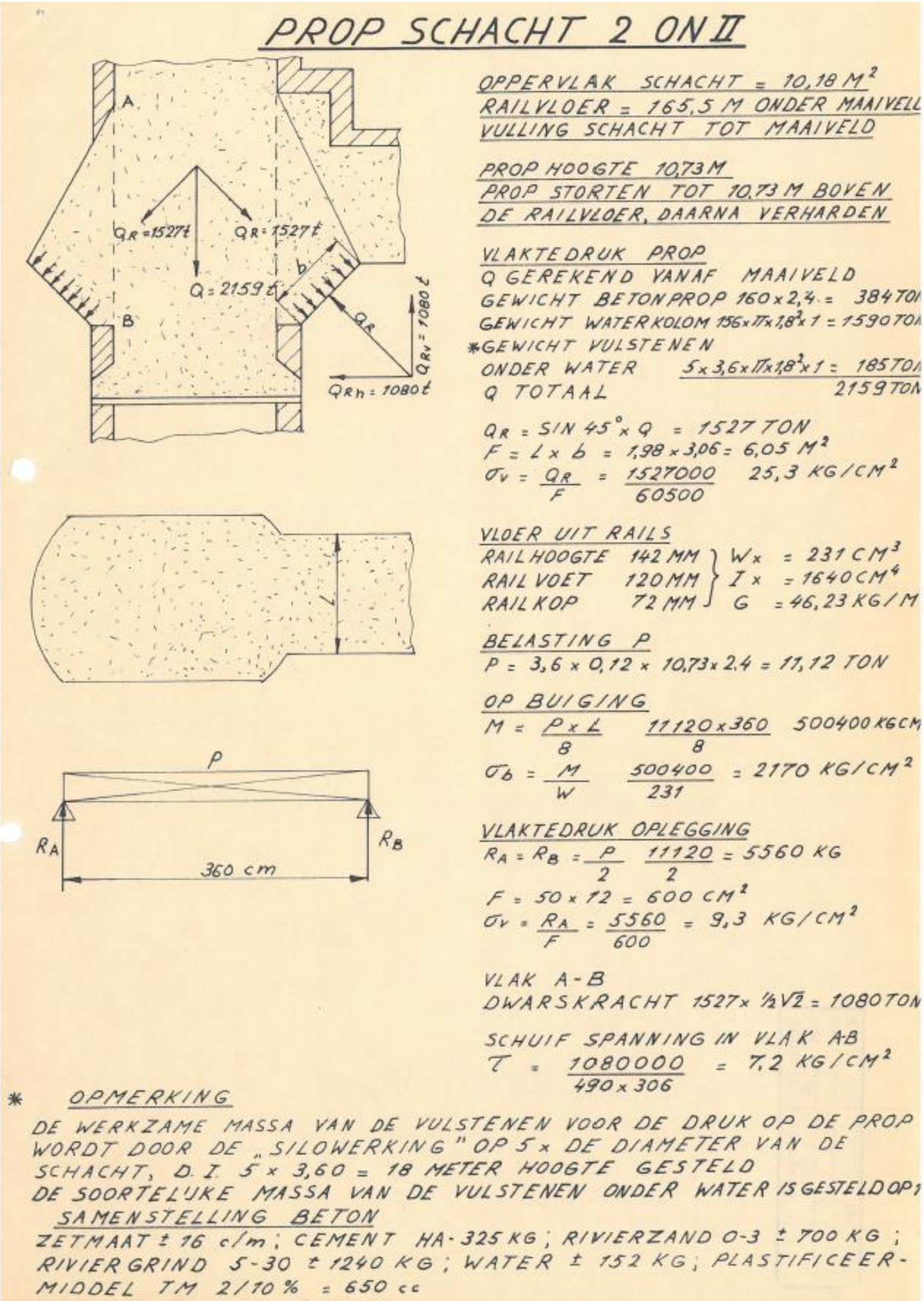


Fig. 67: Static calculation shaft barrier shaft II, ON II /39/

Furthermore a static calculation of the cover is existent /39/. Compare the following figure.

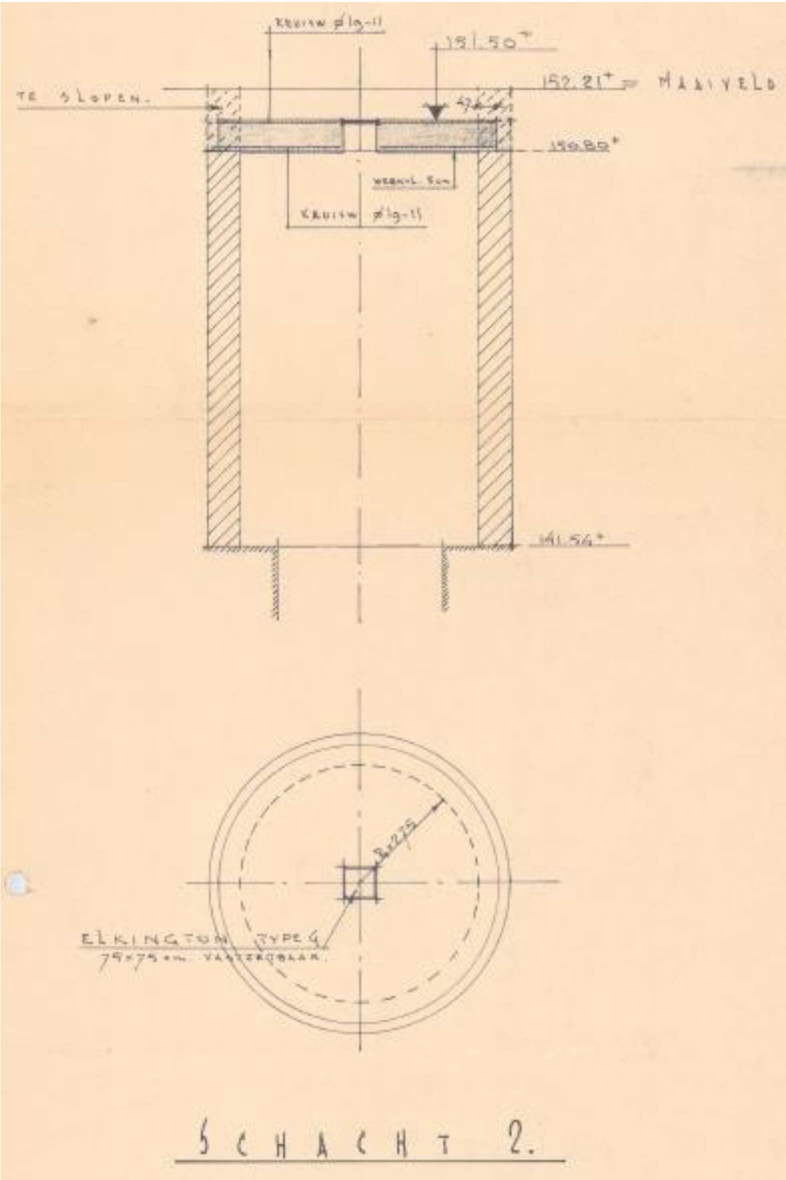


Fig. 68: Shaft cover shaft II, ON II /53/

The coordinates of shaft II are:

RD-x:	199315
RD-y:	321677
elevation:	+152 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space at Aan de Schacht and Koelmoer (community Landgraaf).

6.6 Shaft, ON III

The vertical Shaft of the pit Oranje Nassau III was drilled in 1912 /40/. In 1973 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 7,20 m diameter. The shaft was drilled to a total depth of 844,0 m and was used as drafting shaft and drawing shaft /13/. Within the overburden the shaft consists of tubbing support /50/. The shaft fittings are buntons, guide rails, electric cables, pipe for compressed-air and one pump line /40/.

In this area the overburden has a thickness of 147,42 m and has a layering sequence of sand and clay /40/. The shaft has 12 documented insets. The 225 m floor, as the topmost is located in a level of -133,26 m NAP and in a depth of 227 m /6//50/.

In the following figures a sectional drawing through the shaft is pictured /40//59/.

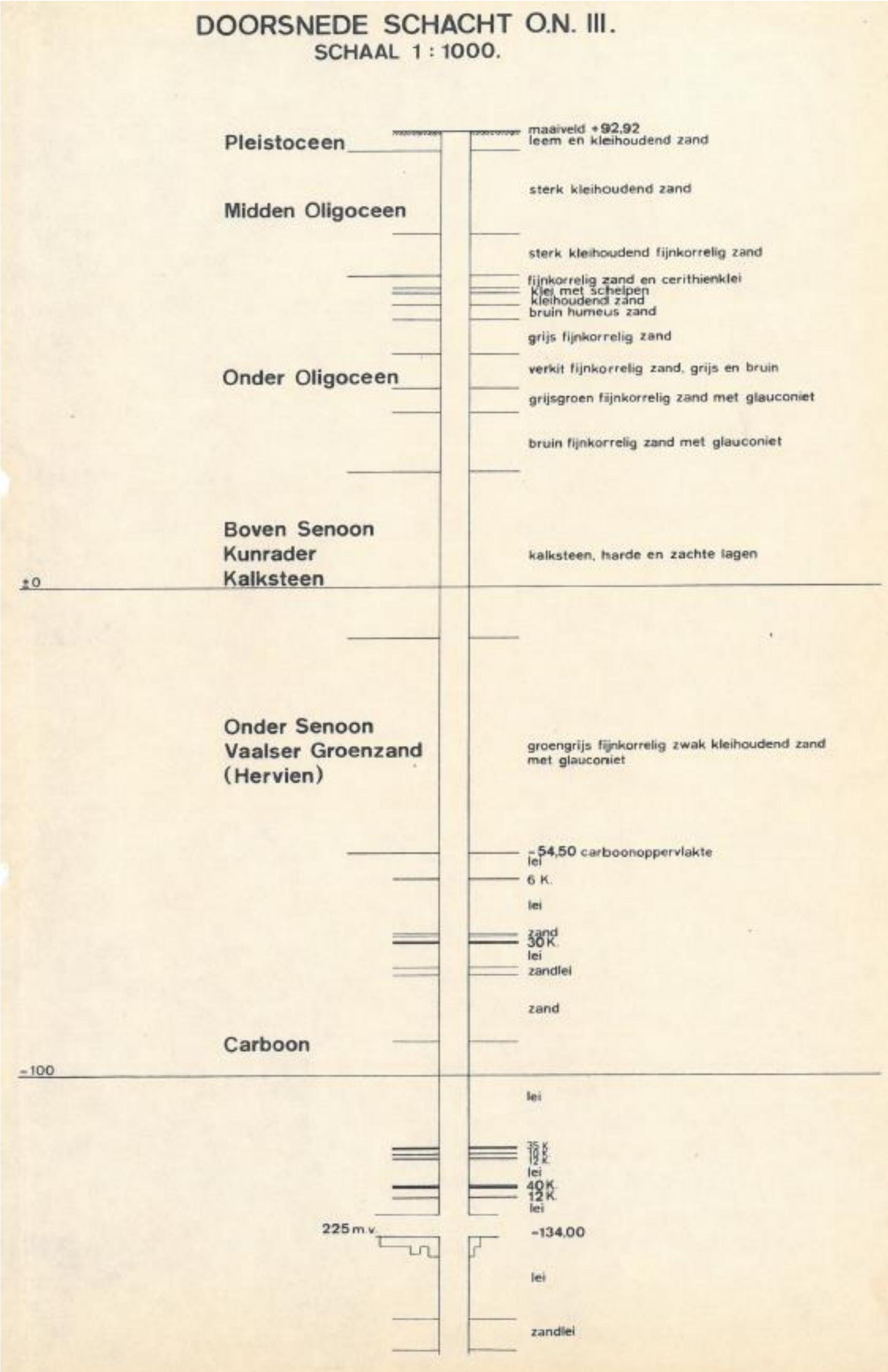


Fig. 69: Sectional drawing through the shaft, ON III including the strata /40/

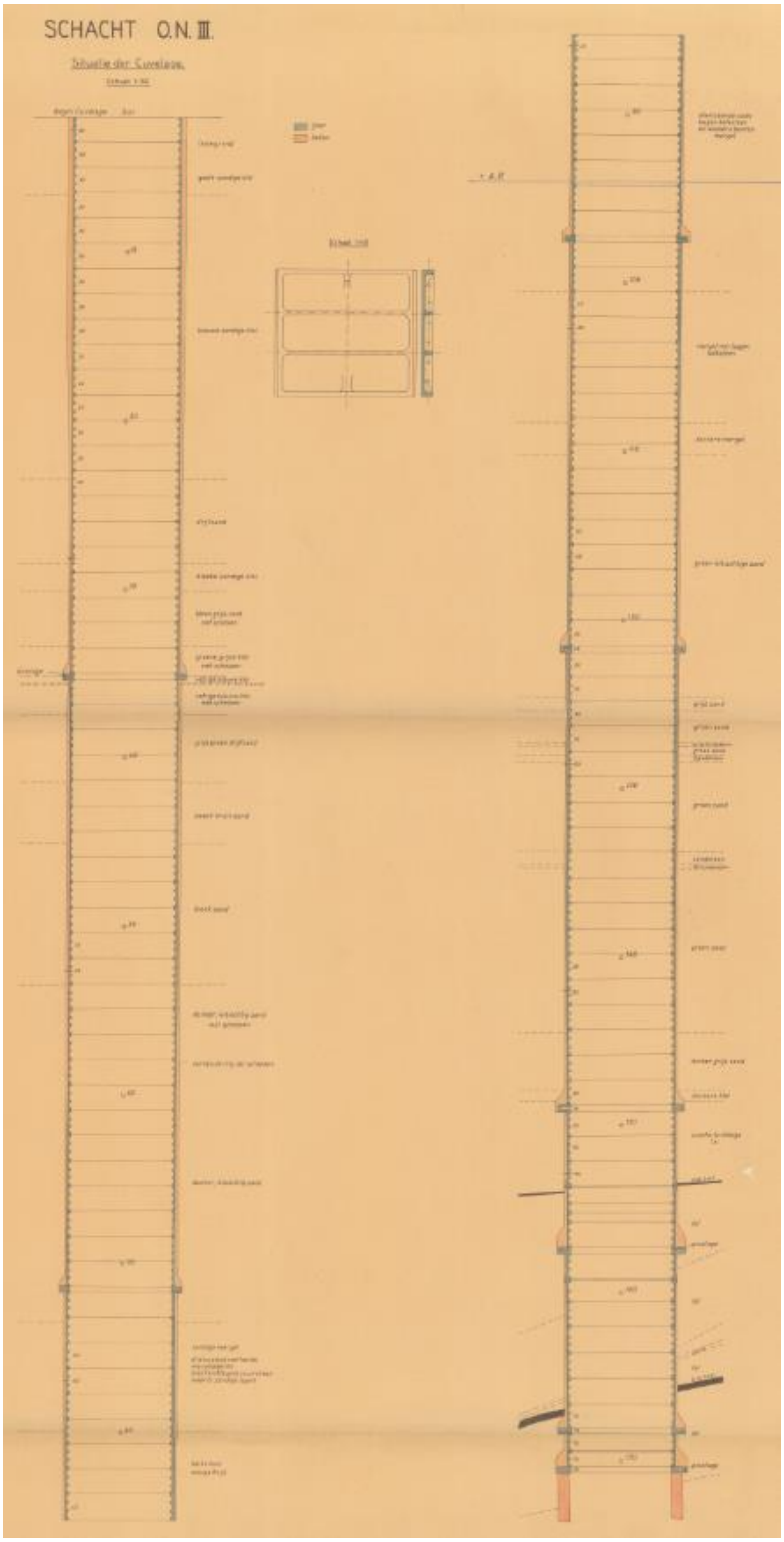


Fig. 70: Sectional drawing through the shaft, ON III including the strata /59/

In the following figure the strata in the range of the 225 m floor is pictured (here mainly slate and sandstone with intercalated hard coal beds) /40/.

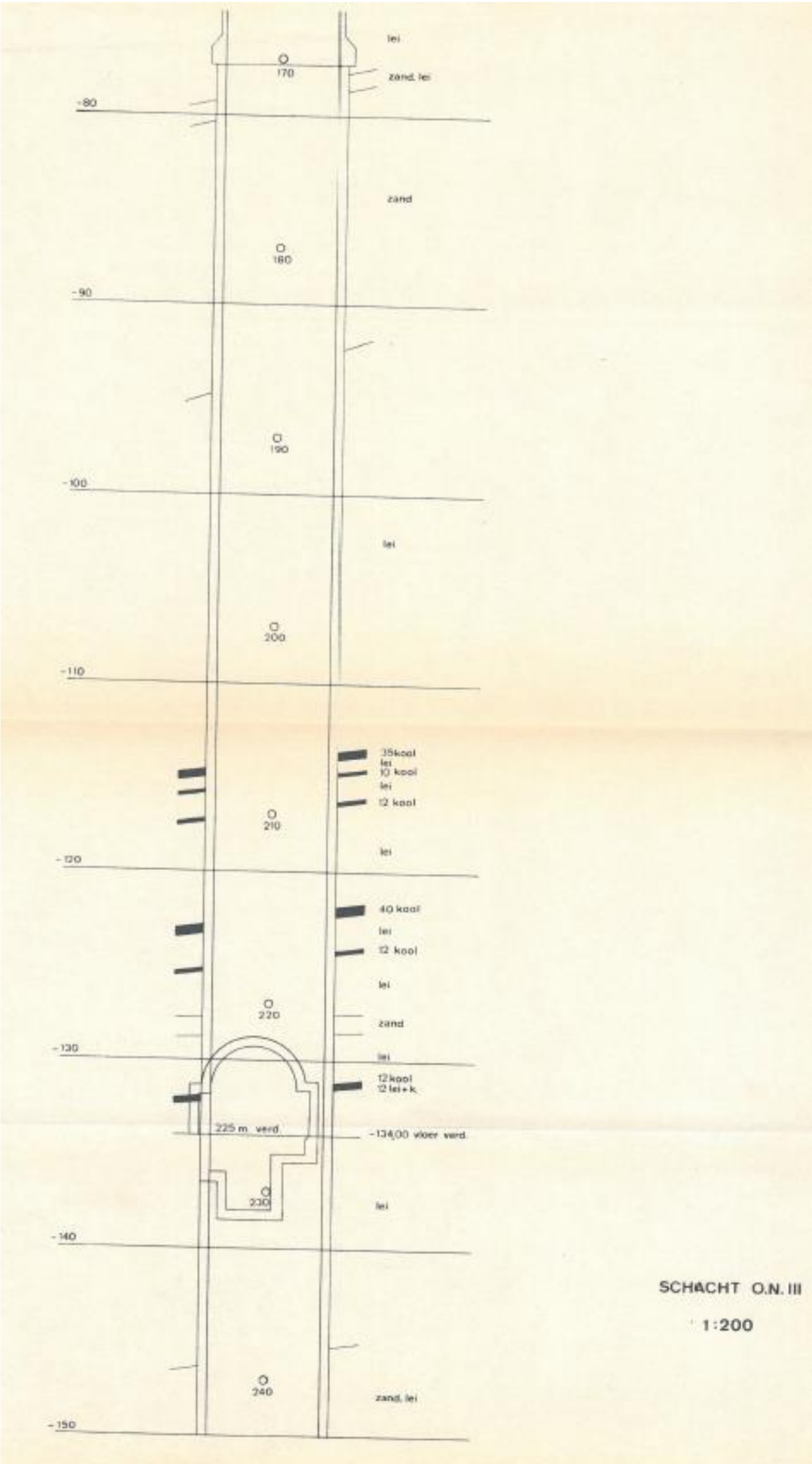


Fig. 71: Strata shaft, ON III, 225 m floor /40/

In 1973 a load bearing filling out of 3.450 m³ of a mixture of concrete was embedded in the shaft-landing of the 225 m floor (-134,00 m NAP). The backfilling was carried out using a back stowing plant. In this depth the shaft has a diameter of 6,0 m. The existing basement areas underneath the shaft landing were used for bearing the filling. First of all on the 225 m floor an abutment of iron beams covered by a concrete board, which rests with its bend lower edge upon the surrounding rock, was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. The remaining part of the shaft was backfilled with 7.035 m³ waste material by hydraulic stowing. By the end of 1973 the shaft column subsided 7,5 m /13//14/. Finally in 1975 the shaft was provided with a concrete cover and an opening for refilling /15/. 1976 this opening was closed with concrete /41/.

In the following figure the shaft barrier of the main shaft of the pit Oranje Nassau III is pictured.

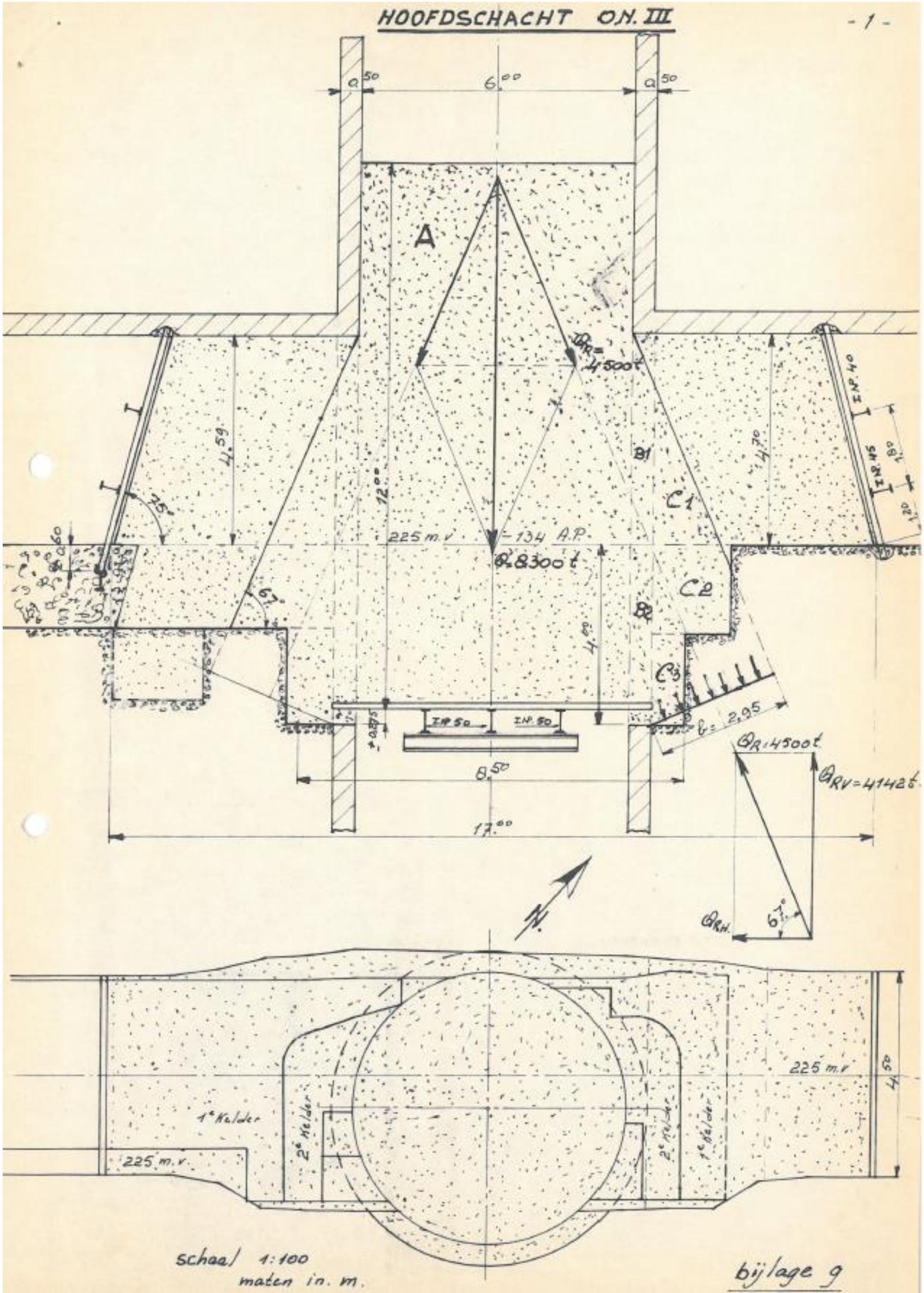


Fig. 72: Shaft barrier, ON III /40/

Static calculations of the shaft barrier of the main shaft, ON III are existent /39/.

Compare the following figures.

Prop hoofdschacht O.N. III

Diameter schacht = 6 m.

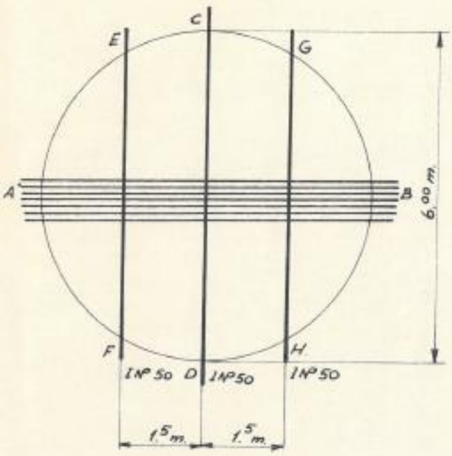
Opp. schachtdoorsn. = $\pi \times 3^2 = 28{,}274\ m^2$

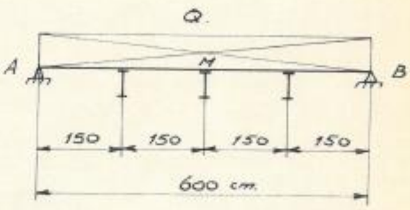
Railvloer 227 m. onder maaiveld op de 225 m. verd.

Prophoogte: 1^e gedeelte prop, hoog 2 m, laten verharden.
Daarna 2^e gedeelte storten tot dak verdieping d.i. 4,7 m. boven de 225 m. verd. Laten verharden.
Daarna 3^e gedeelte storten tot 12 m. boven railvloer.
s.g. beton = 2,4.

Vloer uit rails NP 46

Railhoogte = 142 mm.	$W_x = 231\ cm^3$	mat. st. 60/70
railvoet br. = 120 mm.	$I_x = 1640\ cm^4$	
railkop br = 72 mm.	$G = 46{,}23\ kg/m$	





Middelste rail   Q per rail = gewicht prop (2 m) + e.g. rails

Q per rail $= 6 \times 0{,}12 \times 2 \times 2{,}4 + \frac{6 \times 46{,}23}{1000}$

$= 3{,}456 + 0{,}277$

$= 3{,}74\ t. = 3740\ kg.$

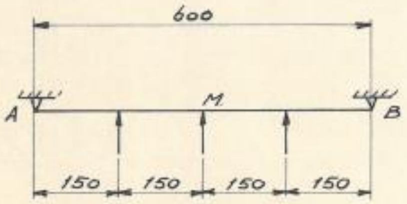
Max. doorbuiging rail = $f_{mr.}$ (in M.)   Toelaatbare doorbuiging: $f_{toel.}$

$f_{m.r.} = \frac{5 \cdot Q \cdot l^3}{384 E \cdot I} = \frac{5 \times 3740 \times 600^3}{384 \times 2{,}2 \times 10^6 \times 1640} = 2{,}9 \text{ cm.}$

$f_{toel.} = \frac{l}{600} = \frac{600}{600} = 1 \text{ cm.}$

De te grote doorbuiging van de rail, $(f_{mr} - f_{toel.}) = 2{,}9 - 1 = 1{,}9 \text{ cm}$, wordt opgenomen door de liggers CD, EF en GH.

Berekening van de door de liggers op te nemen krachten P.



$f_M = f_{mr} - f_{toel.} = 1{,}9$ cm. op te heffen door de krachten P.

$f_M = \frac{19 P \cdot l^3}{384 E \cdot I}$

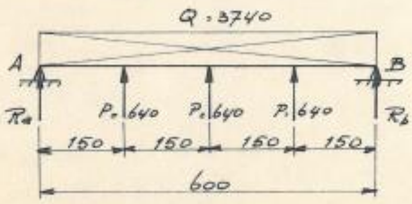
$1{,}9 = \frac{19 P \times 600^3}{384 \times 2{,}2 \times 10^6 \times 1640}$

$P = \frac{1{,}9 \times 384 \times 2{,}2 \times 10^6 \times 1640}{19 \times 600^3} = 640 \text{ kg/rail}$

Berekening vlaktedruk oplegging rail in metselwerk van schachtwand.

Toelaatbare vlaktedruk $\bar{\sigma}_v$ metselwerk $= 20 \text{ kg/cm}^2$

Lengte oplegging rail = 50 cm.



$(\Sigma M)_A = 0 \rightarrow$

$(3740 \times 300) - (640 \times 150) - 640 \times 300 - (640 \times 450) - R_B \times 600 = 0$

$R_B = \frac{(3740 \times 300) - (640 \times 900)}{600} = 910 \text{ kg}$

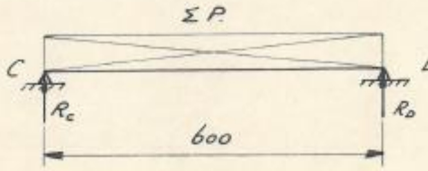
$R_A = R_B = 910 \text{ kg}$

$\sigma_v = \frac{R_A}{F} = \frac{910}{12 \times 50}$

$\sigma_v = 1{,}5 \text{ kg/cm}^2$

Berekening ligger CD.

INP 50 voorzien van oplegplaten (500x300x25)
$W_x = 2750\ cm^3$ $I_x : 68740\ cm^4$ $b = 185\ mm.$



$P = 640\ kg/rail$

$\Sigma P = 640 \times \frac{600}{12} = 32000\ kg$

$R_c = R_D = \frac{\Sigma P}{2} = 16000\ kg.$

a) op sterkte:

$\sigma_b = \frac{M}{W}$

$\sigma_b = \frac{\frac{1}{8} \times 32000 \times 600}{2750} = \frac{2400.000}{2750} = 875\ kg/cm^2$

b) op doorbuiging:

$f_{toel.} = \frac{l}{600} = \frac{600}{600} = 1\ cm.$

$f_{max.} = k . \frac{\Sigma P \times l^3}{I_x}$

$f_{max.} = \frac{6,2 \times 32 \times 6^3}{68740} = \frac{42800}{68740}$

$f_{max.} = 0,625\ cm. < f_{toel.}$

c) Vlaktedruk oplegging in metselwerk van schachtwand.

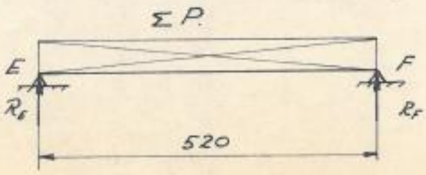
Opleglengte $l : 50\ cm.$ $R_c = R_D = 16000\ kg.$

Oplegbreedte $b : 30\ cm$

$\sigma_v = \frac{R_c}{F} = \frac{16000}{50 \times 30} = 10,7\ kg/cm^2 < \bar{\sigma}_v$

Liggers EF en GH

INP 50 voorzien van oplegplaten (a/m. 500x300x25).



$\Sigma P = 640 \times \frac{520}{12} = 27700\ kg$

$R_E = R_F = \frac{27700}{2} = 13850\ kg$

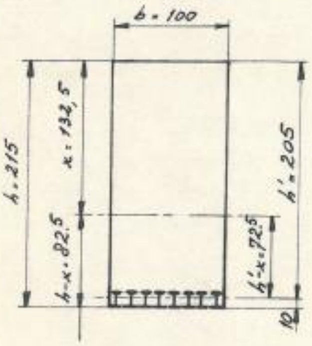
$\sigma_v = \frac{13850}{50 \times 30} = 9,2\ kg/cm^2$

Vloer na 1e stort tot 2,15 m dikte, berekend op buiging

Wapening aaneengesloten vloer uit rails NP46.

Voetbreedte = 12 cm. F_y rail = 58 cm²

F per m' = $\frac{100}{12} \times 58 = 485$ cm² $n = \frac{E_y}{E_b} = 15$.



Belasting q.

$q_{be} = H \times b \times s.g._{be} = 12 \times 1 \times 2400 = 28.800$ kg/m.

$s.g._{yzer} = 46 \times \frac{100}{12} =$ 385 ..

q = 29.200 kg/m

$M_b = \frac{1}{8} Ql$

$M_b = \frac{1}{8}(29.200 \times 6,3) \times 6,3$

$M_b = 144868$ kgm = ± 14.500.000 kgcm.

Bepaling zwaartepunt x

$x = \frac{\frac{1}{2} b h^2 + n \times F_y \times h'}{b \cdot h + n \cdot F_y}$

$= \frac{\frac{1}{2} \times 100 \times 215^2 + 15 \times 485 \times 205}{100 \times 215 + 15 \times 485} = \frac{3802250}{28780} = 132,5$ cm.

$I_{id} = I_{be} + n . I_y$

$= \frac{1}{3} b . x^3 + \frac{1}{3} b (h-x)^3 + n . F_y (h'-x)^2$

$= \frac{1}{3} \times 100 \times 132,5^3 + \frac{1}{3} \times 100 \times 82,5^3 + 15 \times 485 \times 72,5^2$

$I_{id} = 134400000$ cm⁴

$\sigma_{b.d.} = \frac{M}{I} \times x$.

$\sigma_{b.d.} = \frac{14.500.000}{134.400.000} \times 132,5 = 14,2$ kg/cm²

$\sigma_{b.t.} = \frac{M(h-x)}{I_{id}} = \frac{14.500.000 \times 82,5}{134.400.000} = 8,9$ kg/cm²

$\sigma_{y_t} = n \times \sigma_{bt} = 15 \times 8,9 = 133,5$ kg/cm²;

Vlaktedruk prop

$\bar{\sigma}_v$ toelaatbaar = 60 kg/cm²

Q gerekend VANAF maaiveld.

Inhoud betonprop

deel A: $12 \times \pi \times 3^2 =$ 339,3 m³

deel B_1: $2\left\{\left(3,5 \times \frac{3,47+3,40}{2}\right) + \left(\frac{3,5+1,57}{2} \times 2,25\right) - \left(\frac{140}{360} \times \pi \times 3^2\right)\right\} 4,7 -$

$2\left(\frac{0,5 \times 1,178}{2} \times 5,6\right) =$ 60,- "

deel B_2: $2\left\{\left(\frac{3,5+2,875}{2} \times 2,95\right) + \left(\frac{3,5+1,31}{2} \times 2,05\right) - \left(\frac{120}{360} \cdot \pi \cdot 3^2\right)\right\} 4 =$ 39,3 "

deel C1: $2\left(\frac{1,495 \times 3,522}{2} \times \frac{5,6+5,48}{2}\right) =$ 29,2 "

deel C2: $2 \times \frac{1}{2} \times 2\left(1,495 + 2,348\right) \times 4,1$ 31,5 "

deel C_3: $2 \times 0,75 \times 4,1 = 6,15$ m³

$2 \times 1,05 \times 4,1 =$ 8,61 "

14,8 "

Totaal 514,- m³

Gewicht betonprop = 514 × 2,4 = 1234 ton

Gewicht waterkolom = $219 \times \pi \times 3^2 \times 1$ 6194 "

* Vulstenen onder water (silo-werking) = $5 \times 6 \times \pi \times 3^2 \times 1$ 850 "

Q_{totaal} ± 8300 ton

$$Q_R = \frac{\frac{1}{2} Q}{\cos 23} = \frac{8300}{2 \cos 23} = \frac{8300}{2 \times 0,9205} = \frac{8300}{1,841} = 4500 \text{ ton.}$$

$$F = l \times b = \frac{5+4,5}{2} \times 2,95 = 4,75 \times 2,95 = 14 \text{ m}^2$$

$$\sigma_v = \frac{Q_R}{F} = \frac{4500000}{140000} = 32,1 \text{ kg/cm}^2$$

* De werkzame massa van de vulstenen voor de druk op de prop wordt door de „silo-werking" op 5x diam. v.d. schacht, d.i. 5×6 = 30 m hoogte, gesteld. De soortelijke massa van de vulstenen onder water is gesteld op 1.

<u>Max. schuifspanning in draagvloer bij storten prop.</u>

τ_s toelaatbaar = 15 kg/cm² Hoogte draagvloer = 2 m

Afschuifvlak van 1e gedeelte prop = F cm²

$F = 2\left(\pi \times 6 \times \frac{70}{360}\right) 2 = 14{,}7\ m^2 = 147\,000\ cm^2$

$P_s = (12-2)\,\pi \times 3^2 \times 2{,}4 = 680\ t = 680.000\ kg.$

$\tau_s = \frac{P_s}{F} = \frac{680.000}{147.000} = 4{,}62\ kg/cm^2$

<u>Max. schuifspanning in prop, nadat schacht gevuld is (vlak ED)</u>

Lengte afschuifvlak

a) <u>boven 225 m.v.</u> $\alpha = 140°$

$l_a = \pi \times 6 \times \frac{140}{360} = 7{,}3\ m.$ $h_a = 4{,}7\ m.$

b) <u>boven 1e kelder</u>

$l_b = \pi \times 6 \times \frac{87-70}{360} = 0{,}89\ m.$ $h_b = 2\ m.$

c) <u>boven 2e kelder.</u>

$l_c = \pi \times 6 \times \frac{70}{360} = 3{,}67\ m.$ $h_c = 4\ m.$

<u>Opp. afschuifvlak F_{ED}</u>

$(7{,}3 \times 4{,}7) + (0{,}89 \times 2) + (3{,}67 \times 4) = 50{,}76\ m^2 = 507600\ cm^2$

$Q_{RV} = Q_R \sin 67° = 4500 \times 0{,}9205 = 4142\ t. = 4142000\ kg.$

$\tau_s(ED) = \frac{Q_{RV}}{F_{ED}} = \frac{4142000}{507600} = 8{,}16\ kg/cm^2$

Fig. 73: Static calculation shaft barrier, ON III /40/

BEREKENING BETONPROP IN SCHACHT O.N. III 225 M/V.

Volume beton (zie tekening)

cilinder deel A $12.00 \times \pi \times 3.00^2 =$ 424 m^3.

deel B1 $(8.50 \times 5.50 - 280/360 \times \pi \times 3.00^2 - 2.30 \times 2.00)\ 4.60 =$ 93 m^3.

deel B2 $\left\{\frac{(4.20 + 2.70)}{2}\ 2.50 + \frac{(4.20 + 2.20)}{2}\ 2.50 - 110/360 \times \pi \times 3.00^2 + \frac{(4.20 + 3.40)}{2}\ 3.00 + \frac{(4.20 + 2.20)}{2}\ 2.20 - 125/360 \times \pi \times 3.00^2\right\} 2.00 =$ 33 m^3.

deel B3 (zie B2) 33 m^3.

deel C $\left(\frac{4.50 + 5.50}{2} \times 4.00\right) 2.00 \times 2 =$ 80 m^3.

deel D $\left\{\left(\frac{5.50 + 4.50}{2} \times 4.00\right)2 + \left(\frac{5.50 + 4.60}{2} \times 2.00\right)2\right\} \times \frac{4.60}{2} =$ 138 m^3.

Totaal 801 m^3.

Gewicht

betonprop $800 \times 2,4 =$ 1920 ton.

waterkolom $216 \times \pi \times 3.00^2 \times 1 =$ 6104 ton.

vulstenen onder water $5 \times 6 \times \pi \times 3.00^2 \times 1 =$ 848 ton.

Totaal 8872 ton.

afgerond 8900 ton.

Ontbonden onder 45° per oplegvlak :

$$\frac{Q}{\sqrt{2}} = \frac{8900}{\sqrt{2}} = \frac{8900\sqrt{2}}{2} =$$ 6230 ton.

Oppervlakte oplegvlak :

$$\frac{4.50 + 4.00}{2} \times 4.00\sqrt{2} =$$ 23,8 m^2.

Oplegdruk

$$\frac{6230000}{238000} =$$ 26,2 kg/cm^2.

Vertikale druk onverhard beton op draagvloer $15/10 \times 2,4 =$ 3,6 kg/cm^2.

Fig. 74: Calculation load bearing filling, ON III /40/

Furthermore a static calculation of inserted bulkheads in the insets on the 225 m floor is available /40/.

The coordinates of the main shaft are:

RD-x:	194845
RD-y:	324962
elevation:	+93 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the schoolyard of the elementary school “De Schacht” westwards Belemnieterf (community Heerlen).

6.7 Shaft, ON IV

The vertical Shaft of the pit Oranje Nassau IV was drilled in 1910. In 1973 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 5,2 m diameter. The shaft ON IV was drilled to a total depth of 740,0 m and was used as upcast air shaft /13/.

Up to the level of -103,4 m NAP the shaft was made of masonry /35/. Within the overburden the shaft consists of tubbing support. The shaft fittings are buntons, guide rails, electric cables, one pipe for compressed-air, one water pipeline and two pump lines /42/.

In this area the overburden has a thickness of 188,88 m and has a layering sequence of sand and clay /35//42/. The shaft ON IV has 10 documented insets. The 240,0 m floor, as the topmost is located in a level of -130,41 m NAP and in a depth of 239 m /6//50/.

The following figure gives an overview of the strata in the range of the 240 m floor (here mainly slate, sandstone with intercalated beds of hard coal) /42/.

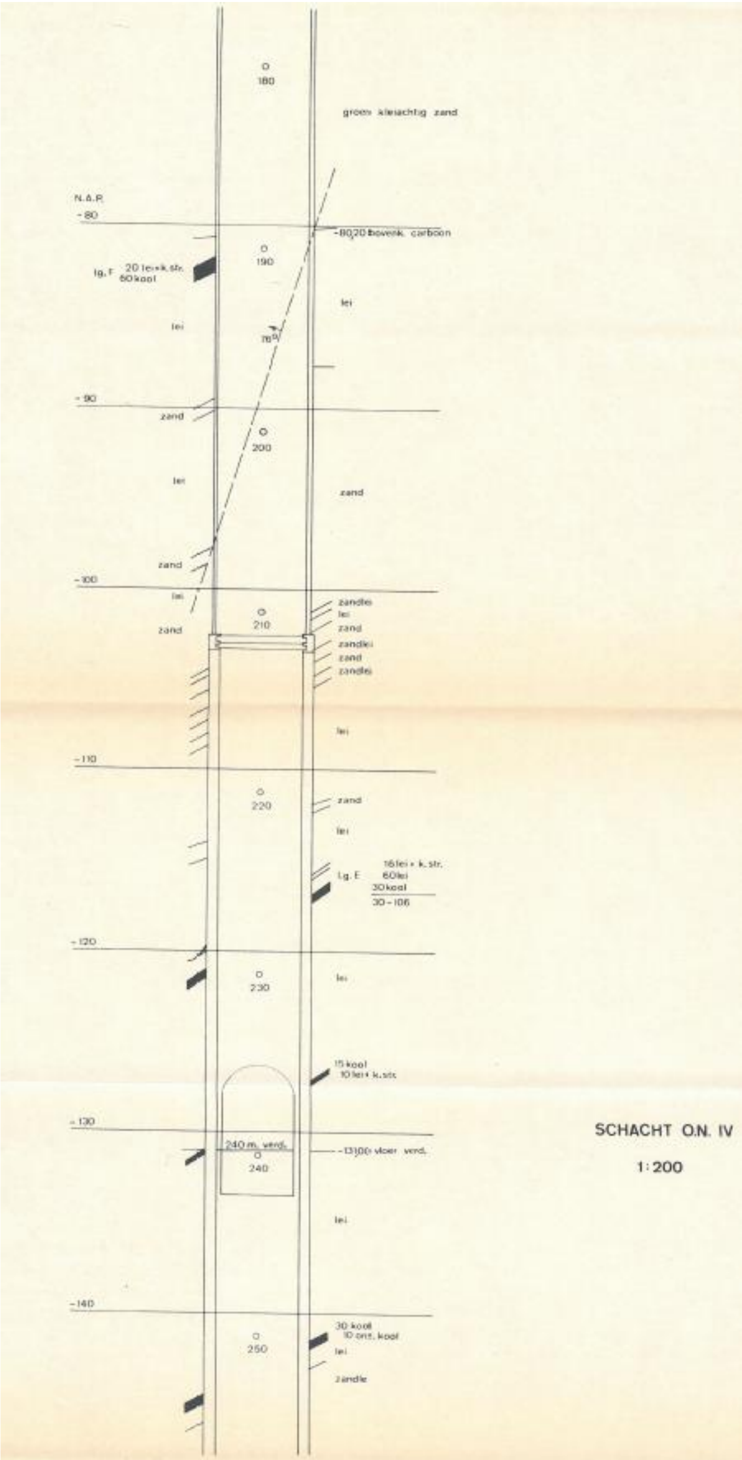


Fig. 75: Strata shaft ON IV, 240 m floor /42/

In 1973 a shaft barrier out of 2.000 m³ of a mixture of concrete was embedded in the range of the shaft-landing on the 240 m floor (-131 m NAP), on which the shaft has a diameter of 4,5 m. The back stowing was carried out through an existing pipe (ø 250 mm) using a back stowing plant. The insets on both sides of the shaft were used as abutments. Before the shaft fittings had to be drawn off. Additionally on the 240 m floor an abutment of iron beams covered by a reinforced concrete board (thickness 2,7 m), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. Above the barrier the shaft column was backfilled with waste material of the grain size 0-120 mm (thickness 25 m). The remaining shaft was backfilled up to the air drift close to the ground surface with 3.235 m³ sand overall (Ts) using hydraulic stowing /13//35/. By the end of 1973 the shaft column subsided 1,27 m /13/. 1974 the shaft was provided with a reinforced concrete cover with cast steel beams and an opening for refilling. In 1976 this opening was closed with concrete /43/.

In the following figure the shaft barrier of the main shaft of the pit Oranje Nassau IV is pictured.

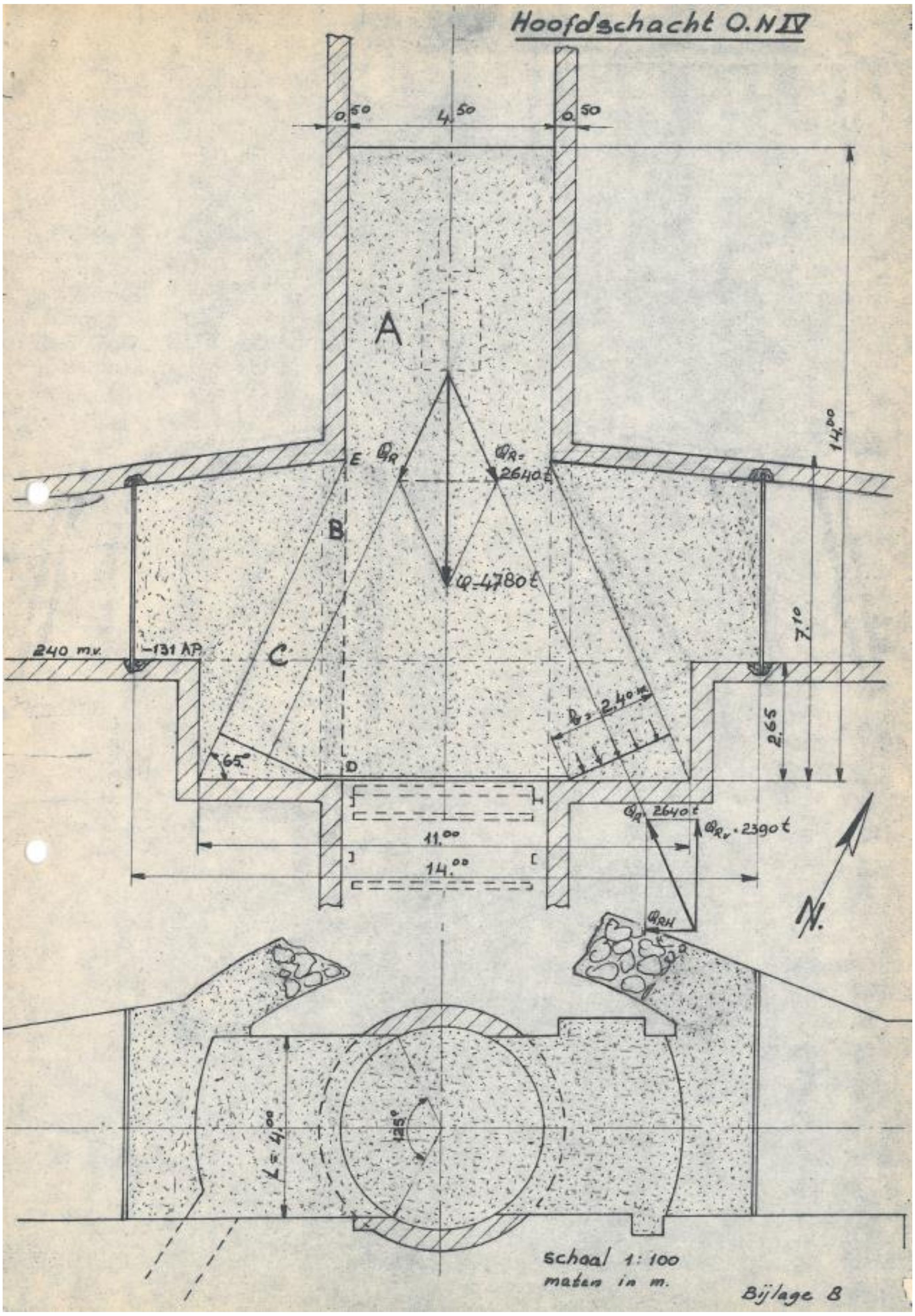


Fig. 76: Shaft barrier, ON IV /42/

Static calculations of the shaft barrier of the shaft ON IV are existent /42/.

Compare the following figures.

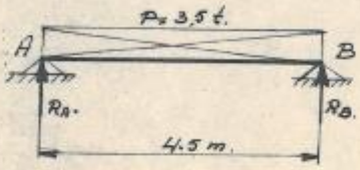
Prop hoofdschacht O.N. IV

Diameter schacht = 4,5 m.
Opp. schachtdoorsn. = $\pi \times 2{,}25^2 = 15{,}9\ m^2$.
Railvloer 242 m. onder maaiveld.
Prophoogte: 1e gedeelte prop hoog 2,5 m. Laten verharden.
Daarna 2e gedeelte storten tot 14 m. boven railvloer.
S.g. beton = 2,4.

Vloer uit rails NP 46.

Railhoogte = 142 mm.
railvoet br. = 120 mm.
rail Kop br. = 72 mm.

$W_x = 231\ cm^3$
$I_x = 1640\ cm^4$
$G = 46{,}23\ Kg/m'$



Belasting: P.

P = gew. beton hoog 2,5 m + e.g. rail.

$$P = (4{,}5 \times 0{,}12 \times 2{,}5 \times 2{,}4) + \frac{46{,}23 \times 4{,}5}{1000}$$

P = ca. 3,5 t.

Op buiging:

$$M = \frac{P \times l}{8} = \frac{3500 \times 450}{8} = 197000\ Kgcm.$$

$$\sigma_b = \frac{M}{W} = \frac{197000}{231} = 855\ Kg/cm^2$$

Vlaktedruk oplegging rails

$$R_A = R_B = \frac{3500}{2} = 1750\ Kg.$$

$$F = 50 \times 12 = 600\ cm^2.$$

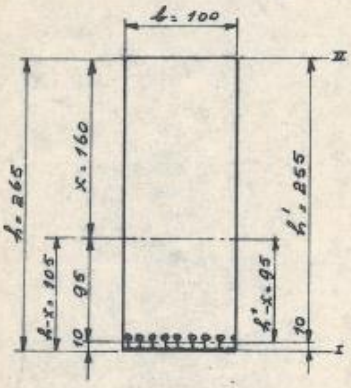
$$\sigma_r = \frac{1750}{600} = 2{,}9\ Kg/cm^2$$

<u>Vloer na 1e stort tot 2,65 dikte berekend op buiging</u>

Wapening aaneengesloten vloer uit rails N.P. 46

Voetbreedte = 12 cm . F_y rail = 58 cm².

F per m' = $\frac{100}{12} \times 58$ = 485 cm² $n = \frac{E_y}{E_b} = 15$.



<u>Belasting q.</u>

$q_{b_c} = F_{schacht} \times 6 . g_{b_c} = 16 \times 2400 = 38400$ Kg/m¹

e.g. yzer : $46.23 \times \frac{100}{12}$ = 385 "

q = 38 785 Kg/m¹

$M_b = \frac{1}{8} . Q . l$

$M_b = \frac{1}{8} \times (38785 \times 4,8) \times 4,8 = 112000$ Kgm.

M_b = 11.200.000 Kgcm.

<u>Bepaling zwaartepunt x.</u>

$x = \frac{\Sigma F . Y}{\Sigma F}$

$X = \frac{1/2 . b . h^2 + n \times F_y \times h'}{b . h + n . F_y}$

$X = \frac{1/2 \times 100 \times 265^2 + 15 \times 485 \times 255}{100 \times 265 + 15 \times 485} = \frac{5.375.000}{33.680} = 160$ cm.

$I_{id.} = I_{b_c} + n . I_y$

$I_{id.} = 1/3 \, b . x^3 + 1/3 \, b (h-x)^3 + n . F_y (h'-x)^2$

$= 1/3 \times 100 \times 160^3 + 1/3 \times 100 \times 105^3 + 15 \times 485 \times 95^2$

$I_{id.}$ = 241.200.000 cm⁴.

$\underline{\underline{\sigma_{bt}}} = \frac{M}{W_I} = \frac{M (h-x)}{I_{id.}} = \frac{11.200.000 \times 105}{241.200.000} = \underline{\underline{4.9}}$ Kg/cm².

$\underline{\underline{\sigma_y}} = n \times \sigma_{bt} = 15 \times 4,9 = \underline{\underline{73,5}}$ Kg/cm².

Vlaktedruk prop. 4

σ_v toelaatbaar = 60 Kg/cm²
Q gerekend vanaf maaiveld.

Inhoud beton-prop

deel A: $14 \times \pi \times 2{,}25^2$ = 224 m³
deel B: $\frac{250}{360} \times \pi(2{,}75^2 - 2{,}25^2)\, 7.1$ = 38,2 "
deel C: $2(6{,}90 \times \frac{3}{2} \times 4)$ = 82,8 "
totaal 345 m³

Gewicht betonprop = $345 \times 2{,}4$ = 830 ton.
Gewicht waterkolom = $226 \times \pi \times 2{,}25^2 \times 1$ = 3592 "
* Gewicht vulstenen onder water: (silo werking)
$= (5 \times 4{,}5) \times \pi \times 2{,}25^2 \times 1 =$ 358 "

Q totaal = 4780 ton.

$$Q_R = \frac{\frac{1}{2} Q}{\sin 65} = \frac{4780}{2 \times 0{,}9063} = 2640 \text{ ton.}$$
$$F = l \times b = 4 \times 2{,}40 \text{ m} = 9{,}6 \text{ m}^2$$

$$\sigma_v = \frac{Q_R}{F} = \frac{2640000}{96000} = 27{,}5 \text{ Kg/cm}^2$$

max. schuifspanning in draagvloer bij storten prop.

τ_s toelaatbaar = 15 Kg/cm²

Afschuifvlak van 1e gedeelte prop = F

$$F = 2(\pi \times 4{,}5 \times \tfrac{125}{360}) \times 2{,}5 = 24{,}5 \text{ m}^2$$
$$P_s = (14 - 2{,}5)\, \pi \times 2{,}25^2 \times 2{,}4 = 438 \text{ ton.}$$

$$\tau_s = \frac{P_s}{F} = \frac{438000}{245000} = 1{,}79 \text{ Kg/cm}^2$$

max schuifspanning in prop nadat schacht gevuld is (vlak ED)

Lengte afschuifvlak = $\pi \times 4{,}50 \times \frac{125}{360} = 4{,}9$ m. hoogte ED = 7,1 m.

$F_{ED} = 4{,}9 \times 7{,}1 = 34{,}79$ m²

$$\tau_{s(ED)} = \frac{Q_{Rv}}{F_{ED}} = \frac{2390000}{347900} = 6{,}9 \text{ Kg/cm}^2$$

* Opmerking:

De werkzame massa van de vulstenen voor de druk op de prop wordt door de "Silo werking" op 5x diam. v.d. schacht – d.i. 5x4,5 = 22.5 m hoogte – gesteld.
De soortelijke massa van de vulstenen onder water is gesteld op 1.

T.B. C.W. O.N. I
21-3-73
[signature]

Fig. 77: Static calculation shaft barrier, ON IV /42/

BEREKENING BETONPROP IN SCHACHT O.N. IV 240 M/V.

Volume beton (zie tekening)

cilinder deel A	$16{,}00 \times \pi \times 2{,}25^2 =$	254 m^3
deel B	$250/360 \times \pi\ (2{,}75^2 - 2{,}25^2)7{,}10 =$	39 m^3
deel C	$185/360 \times \pi\ (5{,}50^2 - 2{,}25^2)\ 2{,}50/2 =$	51 m^3
deel D	$2(\frac{5{,}50 + 6{,}20}{2}\ 3{,}75 + 5{,}50 \times 0{,}40)4 =$	192 m^3
	Totaal	536 m^3

Gewicht

betonprop	$536 \times 2{,}4 =$	1286 ton
waterkolom	$226 \times \pi \times 2{,}25^2 \times 1 =$	3592 ton
vulstenen onder water	$5 \times 4{,}50 \times \pi \times 2{,}25^2 \times 1 =$	358 ton
	Totaal	5236 ton

Druk per oplegvlak $5236\ \frac{1}{2}\ \sqrt{2}$ = 3700 ton.

$F = 2{,}50.\sqrt{2} \times 4{,}00 =$ 14,14 m^2

Oplegdruk $= \frac{3700000}{141400} =$ 26,2 kg./cm^2.

Fig. 78: Calculation load bearing filling, ON IV /42/

Furthermore a static calculation of inserted bulkheads in the insets on the 136 m floor is available /42/.

The coordinates of shaft ON IV are:

RD-x:	196912
RD-y:	324846
elevation:	+109 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the premises „Sigrano-groeve“ of Sibelco company at Koolkoelenweg (community Heerlen).

7 De Staatsmijnen

7.1 Shaft I, Wilhelmina

The vertical Shaft I of the state mine Wilhelmina was drilled in 1905. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,50 m diameter. The shaft was drilled to a total depth of 825,0 m and was used as downcast shaft and drawing shaft /44/. Within the overburden the shaft consists of tubbing support /50/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 99 m respectively the carbon surface is located on +60 m NAP /6/. The shaft has 14 documented insets. The 162 m floor, as the topmost is located in a level of -6,0 m NAP and in a depth of 165 m /6//50/.

In the following figure the strata of the overburden in the range of shaft I Wilhelmina is pictured. Here mainly occur layers of sand and clay.

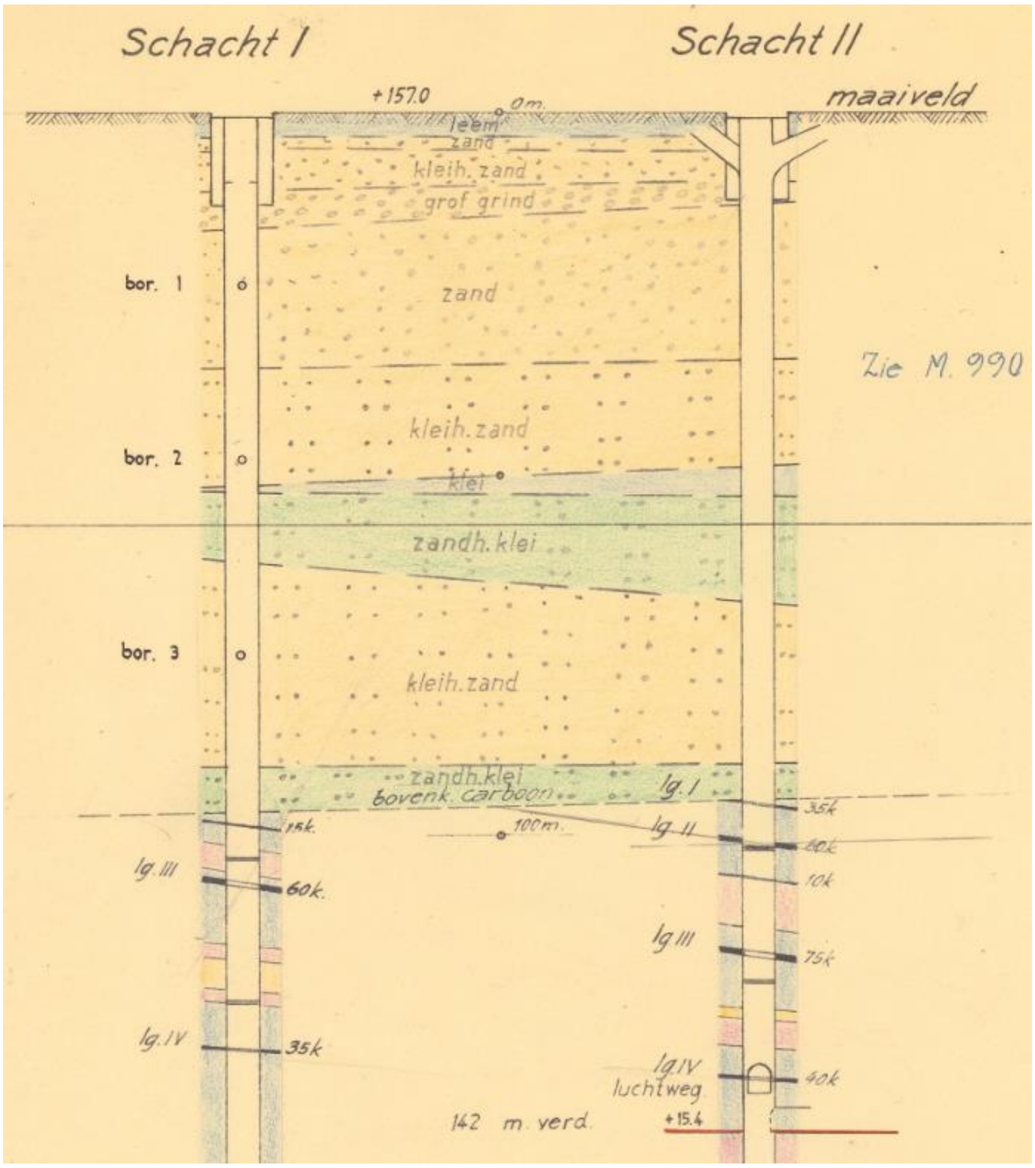


Fig. 79: Strata shaft I Wilhelmina /56/

In 1969 a shaft barrier out of 327 m³ of a mixture of concrete (thickness 8 m) was embedded in the 162 m floor /44/. First of all on the 240 m floor an abutment of iron beams covered by a reinforced concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. For the strengthening of the shaft wall beneath the barrier the shaft was backfilled with a plug of concrete (thickness 1 m) /9/. In 1970 the entire shaft column was backfilled with approximately 3.002 m³ waste material above the barrier and was closed with a reinforced cover including an opening for refilling. For ground water monitoring an existing pipe for compressed-air was installed in the shaft and implemented in the plug /10//44/. In 1970 the shaft column subsided 0,75 m /10/ and for another 0,01 m in 1971 /11/. Finally in 1975 the opening for refilling was closed with a mixture of concrete /15//44/.

In the following figure the shaft barrier of the shaft Wilhelmina I is pictured.

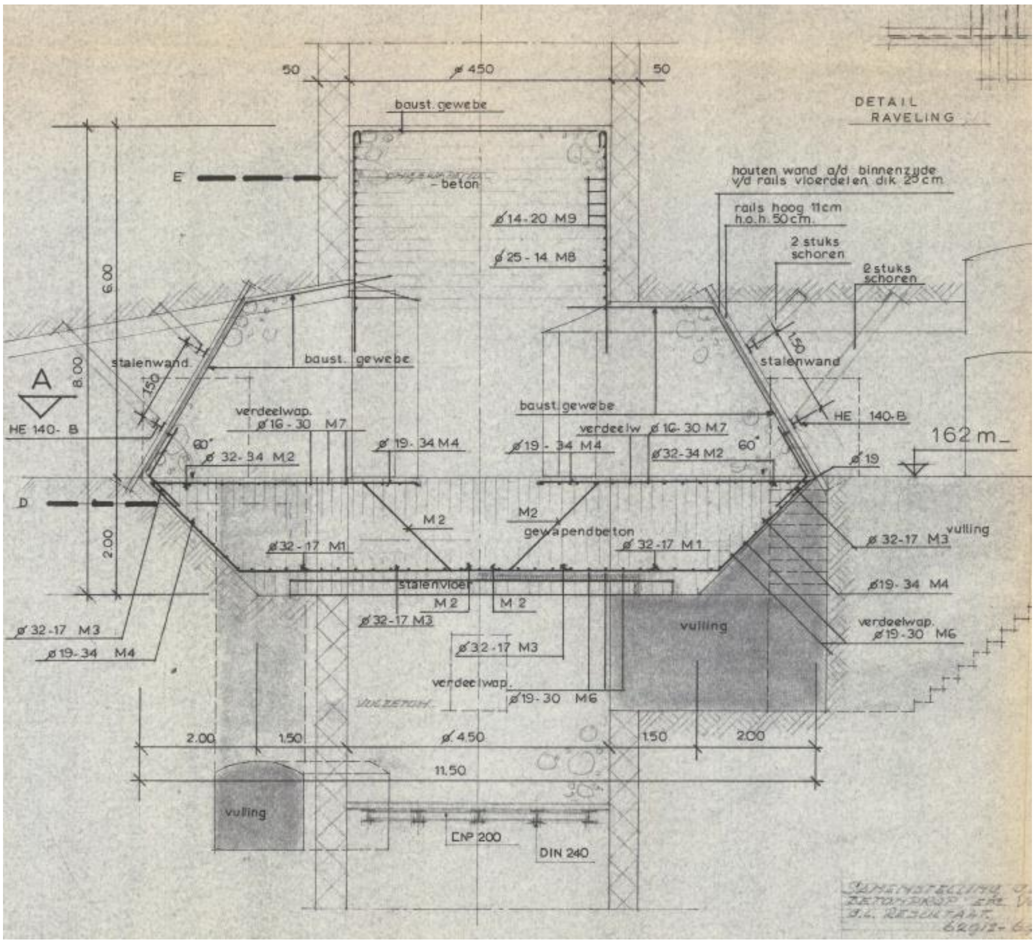


Fig. 80: Shaft barrier shaft I Wilhelmina /44/

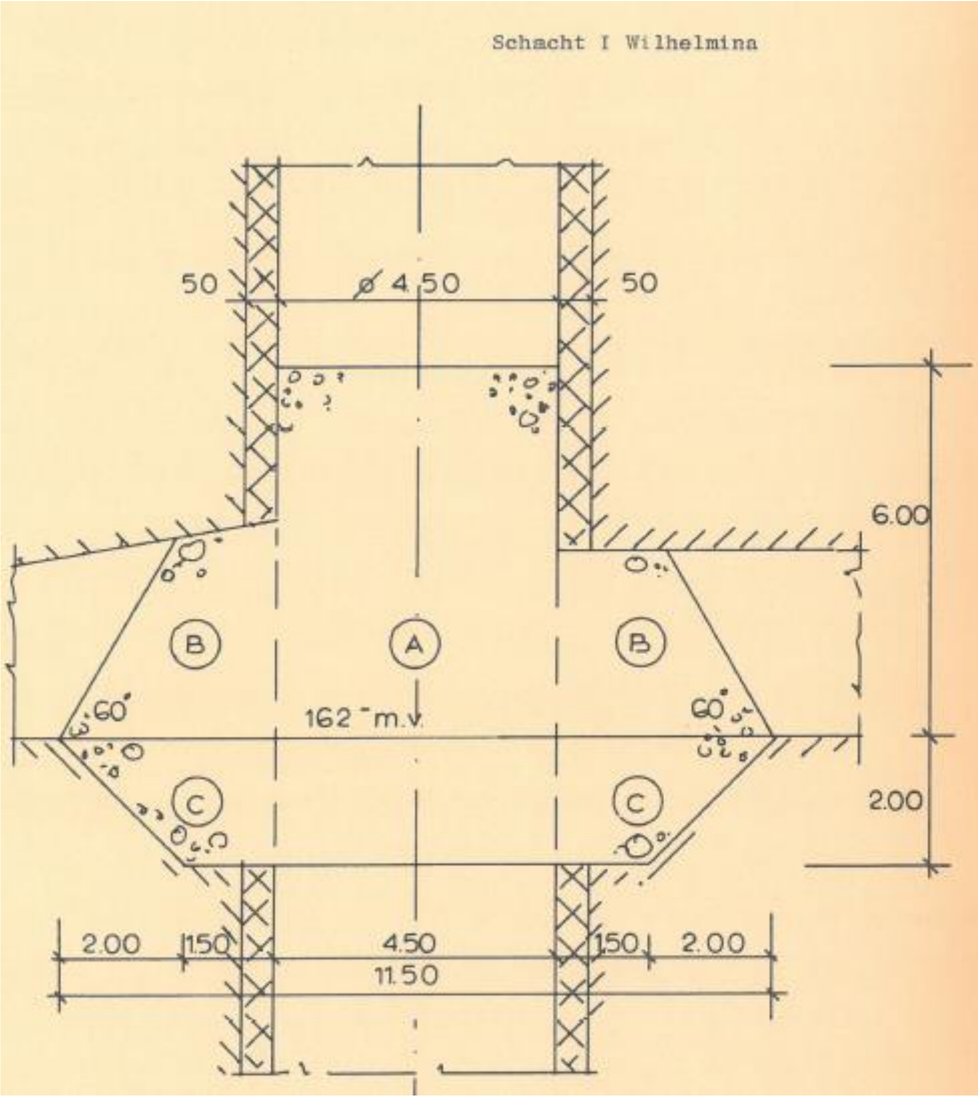


Fig. 81: Shaft barrier shaft I Wilhelmina /44/

Static calculations of the shaft barrier are existent /44/. Compare the following figures.

NV NEDERLANDSE STAATSMIJNEN

Nieuwbouw DSM

Geleen, juni 1969

Nr. 2842

STATISCHE BEREKENING BETONPROP IN SCHACHT I

SM WILHELMINA OP 162 M VERDIEPING

Tekening 4B 44256

Betonprop

Bepaling volume beton (zie tekening).

Cilinder deel (A) $\frac{\pi}{4}$. $4,50^2$. 8,00 = 128 m^3

Deel B: (vanwege de kromming is de breedte van de teen
20 cm groter genomen)

2 x $\frac{1}{2}$ (1,95 + 3,70) . 3,00 . 4,00 = 68 m^3

Deel C

2 x $\frac{1}{2}$ (3,70 + 1,70) . 2,00 . 4,00 = 44 m^3

V Totaal 240 m^3

Vulbeton

Trap naar 2e uitstapvloer:

1,50 . 3,75 . 1,90 + 3,00 . 1,00 . 3,90 + $\frac{1}{2}$. 4,20 . 1,00 . 4,00 =
= 10,7 + 11,7 + 8,4 = 30,8 m^3

Trap naar 1e uitstapvloer:

$\frac{1}{2}$. 2,40 . 1,90 . 1,00 + $\frac{1}{2}$. 2,00 . 2,00 . 1,50 = 2,3 + 3 = 5,3 m^3

Schacht naar gang op 169-P

$\frac{\pi}{4}$. $1,65^2$. 5,00 = 10,7 m^3

Gang op 169-P 1,65 x 1,50 x 10,50 = 26 m^3

Watergalerij (gedeeltelijk)

1,00 x 1,20 x 2,75 + 1,75 x 3,00 x 2,00 = 3,3 + 10,5 = 13,8 m^3

Totaal 86,6 m^3

Opmerking

Gerekend wordt op het ongunstigste geval: n.l. geen water onder de prop, en boven de prop is een vulling van met water verzadigde wasstenen.
Vanwege de "silowerking" kan gerekend worden met een equivalente vulhoogte van 3 x de diameter van de schacht.

te weinig

Daar deze kolom wasstenen zich onder water bevindt, wordt voor het gewicht gerekend met 1,2 ton/m^3.
Daarbij komt voor de verticale kracht het eigengewicht van de betonprop en het gewicht van een waterzuil ter hoogte van 156 m'.

(Schacht I)

Gewicht betonprop totaal

$240 \cdot 2,4 =$ 575 ton

Gewicht waterkolom

$\frac{\pi}{4} \cdot 4,50^2 \cdot 156 \cdot 1,00 =$ 2500 ton

Met "silowerking" extra van vulstenen onder water

te weinig

$3 \cdot 4,50 \cdot \frac{\pi}{4} \cdot 4,50^2 \cdot 1,2 =$ *5/3 ×* 255 ton *425*

P Totaal = 3330 ton

Reken 3350 ton

P Totaal ontbonden onder een hoek van 45° geeft: $R = \frac{1}{2}\sqrt{2} \cdot 3350 = 2400$ ton

Oplegvlak: $0 = 2,00\sqrt{2} \cdot 4,00 = 11,3\ m^2$

Oplegdruk: $\sigma_d = \frac{2400}{11,3} = 21,2\ kg/cm^2$

A) Stalen vloer op 164 m' -mv. geheel als schacht II.

B) Betonplaat d ≈ 160 cm geheel als schacht II.

c) Wapening cilinder ponsspanning + schuifspanning als schacht II.

d) Wandbekisting zie ook schacht II.

Vloer voor manchet (schacht I)

Volume beton $\frac{\pi}{4} \cdot 4{,}50^2 \cdot 4{,}00 = \sim 64\ m^3$

Betonplaat dik 4 m

a) Eigen gew. 4 x 2400 = 9600 kg/m^2

b) Eigen gew. stalen vloer (reken) 300 kg/m^2

Q Totaal = 9900 kg/m^2

1) Moerbalken h.o.h. 1,00 m' l = 2,50 m'

$q = 1{,}00 \times 9900 = 9900\ kg/m^2$

$M_{max} = 1/8 \times 9900 \times 2{,}50^2 = 7750\ kgm$

$W\ vereist = \frac{775000}{1400} = 560\ cm^3$

W aanwezig DIN 24 $\rightarrow W_x = 938\ cm^3$

2) Rails vormen dek

H = 70 mm B = 58 mm

Moerbalken hart op hart 1,00 m'

$\sigma_t = 700\ kg/cm^2 \qquad W_x = 24{,}4\ cm^3$

Per m' $\rightarrow$ 17 stuks = $17 \times 24{,}4 = 415\ cm^3/m'$

$M_{max} = 1/8 \times 9900 \times 1{,}00^2 = 1235\ kgm$

$W\ vereist = \frac{123500}{700} = 176\ cm^3/m' < 415\ cm^3/m'$

3) Kettingen 13 stuks

Draagvermogen minimaal 13 x 15 ton = 195 ton.

Gewicht manchet = 64 x 2400 = 154 ton.

Eigen gew. stalen vloer reken

$300\ kg/m^2 =$

$300 \times \frac{\pi}{4} \times 4{,}5^2 =$ 4,8 ton

Totaal 158,8 ton < 195 ton

Fig. 82: Static calculation shaft barrier shaft I Wilhelmina /44/

The coordinates of shaft I Wilhelmina are:

RD-x:	199802
RD-y:	320412
elevation:	+157 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located directly at the access road of a riding area northwards “Tunnelweg” (community Kerkrade).

7.2 Shaft II, Wilhelmina

The vertical Shaft II of the state mine Wilhelmina was drilled in 1904. In 1970 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,50 m diameter. The shaft was drilled to a total depth of 537,0 m and was used as downcast shaft /44/. Within the overburden the shaft consists of tubbing support /50/. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 99 m respectively the carbon surface is located on +60 m NAP /6/. The shaft has 12 documented insets. The 142 m floor, as the topmost is located in a level of -15,0 m NAP and in a depth of 144 m /6//50/.

In the following figure the strata of the overburden in the range of shaft II Wilhelmina is pictured. Here mainly occur layers of sand and clay.

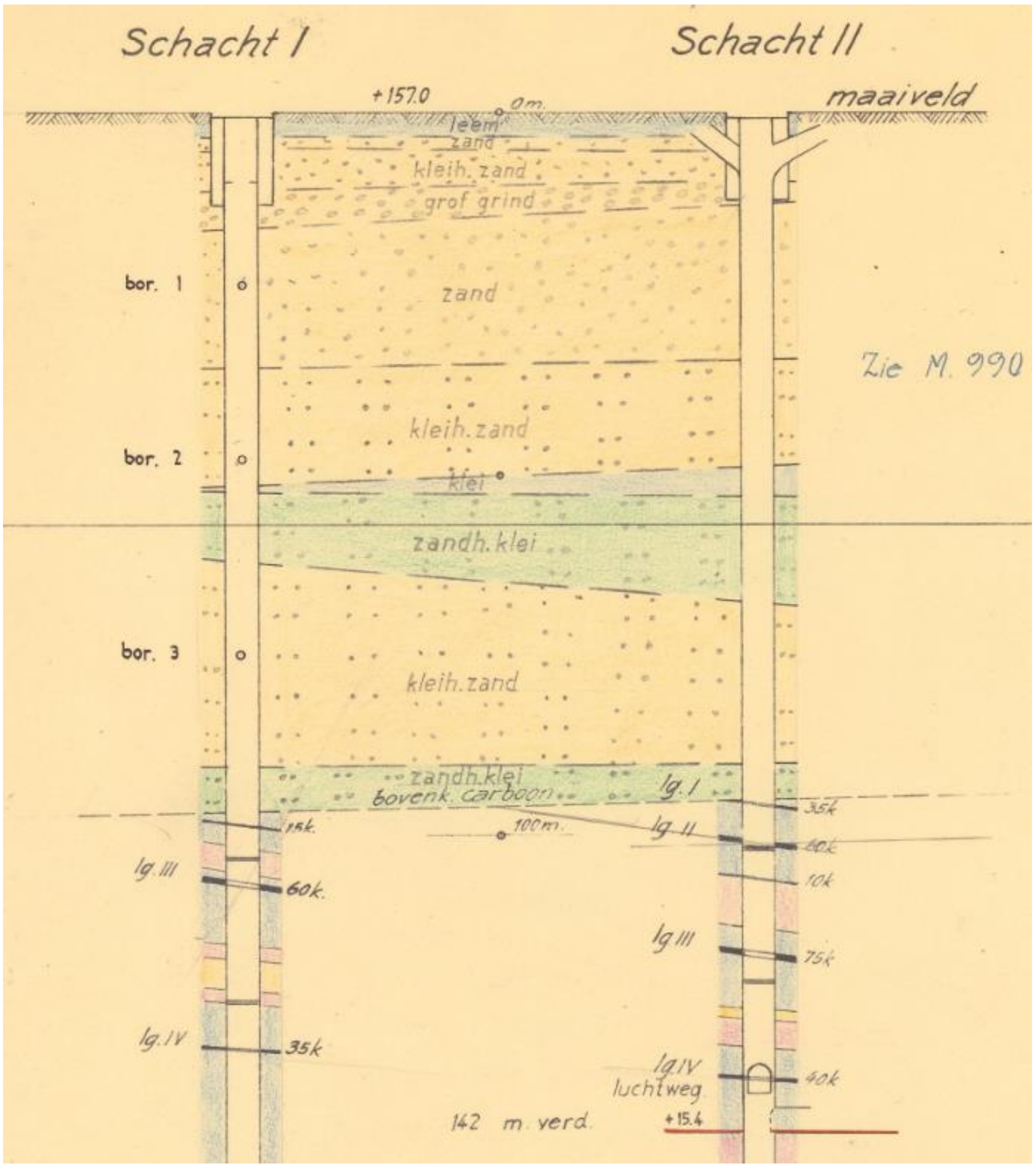


Fig. 83: Strata shaft II Wilhelmina /56/

In 1969 a shaft barrier out of 327 m³ of a mixture of concrete (thickness 10,75 m) was embedded in the 162 m floor /44/. First of all on the 240 m floor an abutment of iron beams covered by a reinforced concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. Additionally the shaft barrier was provided with a column of 400 m³ concrete to seal off the insets of two galleries (142 m floor and 134 m floor) /9//44/. In 1970 the entire shaft column was backfilled with approximately 2.340 m³ waste material above the barrier and was closed with a reinforced cover including an opening for refilling /10//44/. In 1970 the shaft column subsided 0,19 m /10/ and for another 0,01 m in 1971 /11/. Finally in 1975 the opening for refilling was closed with a mixture of concrete /15//44/.

In the following figure the shaft barrier of the shaft II Wilhelmina is pictured.

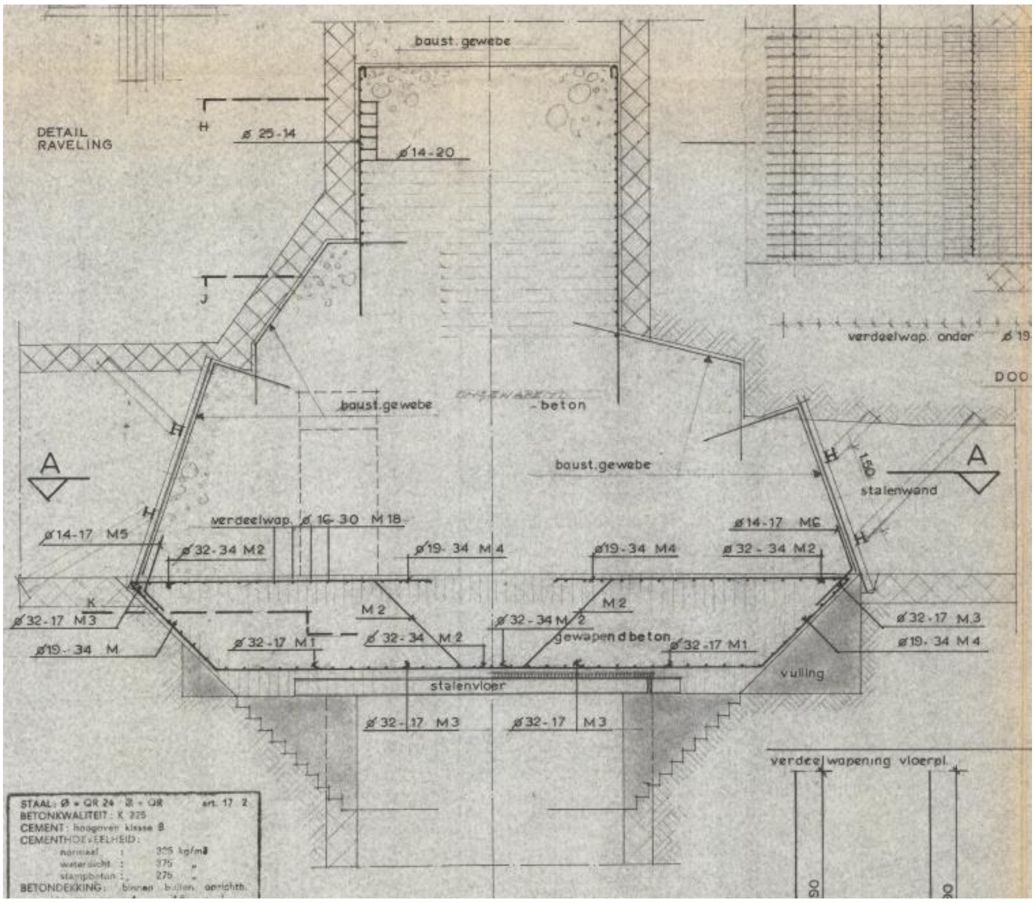


Fig. 84: Shaft barrier shaft II Wilhelmina /44/

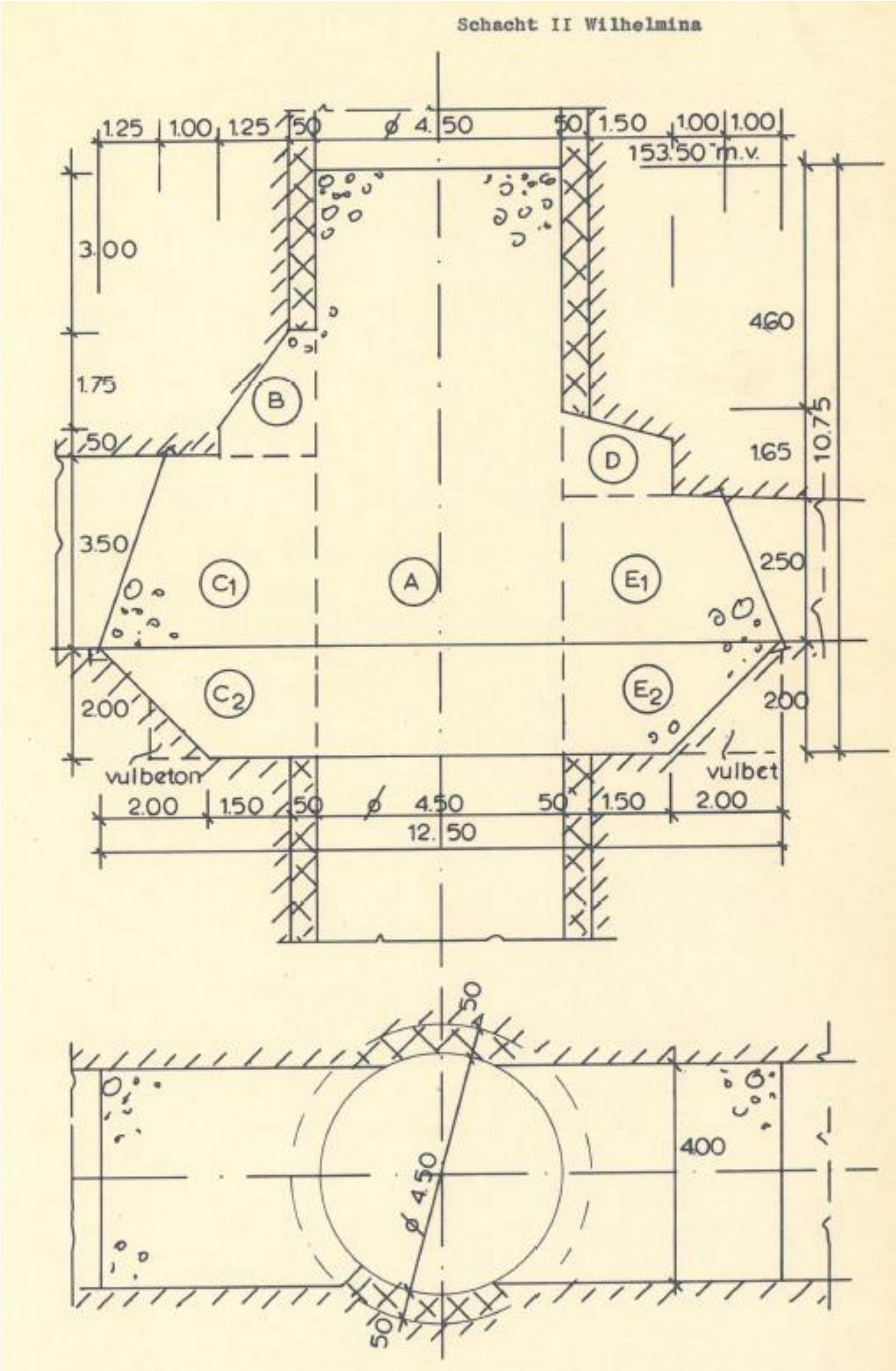


Fig. 85: Shaft barrier shaft II Wilhelmina /44/

Static calculations of the shaft barrier are existent /44/. Compare the following figures.

NV NEDERLANDSE STAATSMIJNEN
Nieuwbouw DSM

Geleen, juni 1969

Nr. 2843

STATISCHE BEREKENING BETONPROP IN SCHACHT II
SM WILHELMINA OP 162 M" VERDIEPING
Tekeningen 4B 44257, 4B 44258 en 4B 44259

I. Betonprop

Bepaling volume beton (zie tekening)

Cilinder deel A $\frac{\pi}{4}$. $4{,}50^2$. $4{,}50^2$. 10,75	= 172 m^3
Deel B : $\frac{1}{2}$ (0,70 + 1,95) 2,25 . 4,00	= 12 m^3
Deel C_1: $\frac{1}{2}$ (1,95 + 4,20) 3,50 . 4,00	= 43 m^3
Deel C_2: $\frac{1}{2}$ (4,20 + 2,20) 2,00 . 4,00	= 25,6 m^3
Deel D : 2,20 . 1,30 . 4,00	= 11,4 m^3
Deel E_1: $\frac{1}{2}$ (3,20 + 4,20) 2,50 . 4,00	= 37 m^3
Deel E_2: $\frac{1}{2}$ (4,20 + 2,20) 2,00 . 4,00	= 25,6 m^3
V Totaal	326,6 m^3

Vanwege de kromming is de breedte van de teen 20 cm groter genomen (deel B t/m E_2).

Reken vulbeton $\frac{1}{2}$. 2,00 . 2,00 . 4,00 = 8
$\frac{1}{2}$. 1,00 . 1,00 . 4,00 = 2

vulbeton 10 m^3

Opmerking:

Er wordt gerekend met het ongunstigste geval:

a) onder de prop geen water;

b) boven de prop een vulling van wasstenen die met water is verzadigd.

Vanwege de "silowerking" kan volgens bijgaande bijlage II voor de wasstenen gerekend worden met een equivalente vulhoogte van 3 x de diameter van de schacht.

Zie opm. bl. I

Het soortelijk gewicht γ van de kolom wasstenen welke verzadigd is

met water volgt uit: $\gamma = \gamma_s \left(1 - \frac{\gamma_v}{\gamma_k}\right)$ (zie hiervoor eveneens bijlage II)

Hierin is γ_k = soortelijk gewicht van de korrels (ca. 2,5)

γ_v = soortelijk gewicht van de omringende vloeistof (ca. 1)

γ_s = soortelijk gewicht van het droge stortmateriaal (ca. 2)

γ = schijnbare soortelijk gewicht van de stortmassa verzadigd met water

indien $\gamma_s = 2$, dan is $\gamma = 1,2$

Gerekend wordt met dit gewicht.

De totale verticale kracht volgt door sommering van de volgende componenten:

a) gewicht van de totale betonprop;

b) de hydrostatische druk;

c) de door het vulmateriaal op de prop uitgeoefende druk.

Wordt bij het berekenen van de oplegkracht rekening gehouden met de kleef tussen prop en wanden, dan moet de verticale kracht verminderd worden met de kleefkracht (kleefopp. x kleefkracht per opp. eenheid).

A) Berekening oplegdruk waarbij de kleeft van de prop t.o.v. de schachtwand en de laadplaatsen verwaarloosd is.

a) betonprop: volume x s.g. = 327 x 2,4 = 785 ton

b) waterkolom: $\frac{\pi}{4} \cdot 4,50^2 \cdot 153,50 \cdot 1$ = 2450 ton

c) vulstenen: 3 . 4,50 . $\frac{\pi}{4} \cdot 4,50^2 \cdot 1,2$ = 260 ton ca. 425

P Totaal = 3495 ton

Reken P = 3500 ton

ontbinding van P tot. onder een hoek van 45° geeft

$R = \frac{1}{2}\sqrt{2}\ 3500 = 2500$ ton

Oplegvlak $O = 200\sqrt{2} \cdot 4,00 = 11,3\ m^2$

Oplegdruk $\sigma_d = \frac{R}{O} = \frac{2500}{11,3} = 222$ ton/m², dus 22,2 kg/cm².

B) Berekening van de oplegdruk waarbij wel rekening wordt gehouden met de kleef van de prop t.o.v. de schacht- en laadplaatswanden. Bij de berekening wordt de kleef aangenomen op 2,5 kg/cm² er van uitgaande dat de laadplaatswanden ruw zijn en de schachtwand bewust geruwd wordt.

Nuttig kleefoppervlak:

Laadplaats	$\{2 \times \frac{1}{2}(4 + 7,5) + 2 \times \frac{1}{2}(4 + 6)\}$	= 86 m²
Schachtwand	2 x 10,50 x 3	= 63 m²
Totaal		149 m²

Bij de bepaling v/h nuttig kleefoppervlak van de schachtwand zijn alleen de segmentgedeelten loodrecht op de as v/d laadplaats in rekening gebracht.

Totale kleefkracht v/d betonprop 149 x 25 = 3700 ton.

Resterende kracht 3500 - 3700 = - 200 ton.

II. Stalen vloer op 164 m' - mv

Deze stalen vloer dient als bekistingsvloer voor de 1,60 m' dikke betonplaat.

a) Eigen gew. betonvloer h x s.g. = 1,60 x 2400 = 3850 kg/m^2

b) Eigen gew. stalenvloer (reken) = 300 kg/m^2

Q totaal 4150 kg/m^2

1) Moerbalken h.o.h. 1,00 m' l = 6,00 m'

Opmerking: De gemetselde schachtwanden zijn van dermate slechte kwaliteit, dat deze balken buiten de schachtwanden op het carboongesteente moeten worden opgelegd.

$q = 1,00 \times 4150 = 4150\ kg/m^2$

$M_{max} = 1/8 \times 4150\ .\ 6,00^2 = 18700\ kgm$

$W\ vereist = \frac{1870000}{1400} = 1330\ cm^3$

Kies DIN 28

$W_x = 1380\ cm^3$

$I_x = 19270\ cm^4$

$G = 103\ kg/m'$

Randbalken

$q = 0,65 \times 4150 = 2700\ kg/m^2$ l = 4,50 m'

$M_{max} = 1/8 \times 2700\ .\ 4,50^2 = 6850\ kgm$

$W\ vereist = \frac{685000}{1400} = 490\ cm^3$

Kies INP 28

$W_x = 542\ cm^3$

$I_x = 7590\ cm^4$

$G = 47,9\ kg/m'$

Rails vormen dek (smalspoorrails)

H = 70 mm B = 58 mm

Moerbalken hart op hart 1,00 m'

σ_t = 700 kg/cm^2 W_x = 24,4 cm^3

per m' → 17 stuks = 17 x 24,4 = 415 cm^3/m'

M_{max} = 1/8 x 4150 x 1,00^2 = 520 kgm/m'

W vereist $\frac{52000}{700}$ = 74 cm^3/m' < 415 cm^3/m'

Over deze smalspoorrails een houten dekvloer d = 2,5 cm

III. Betonplaat

De betonplaat moet het gewicht van de betonprop dragen. Hoogte betonprop reken 7,50 m'.

Dus q = 7,50 x 2400 = 18000 kg/m^2

q over strook breed 4,00 m' = $\frac{4,50}{4,00}$ + 18000 = 20300 kg/m'

Veldmoment = 1/8 x 20300 x 6^2 = 91500 kgm/m'

b = 1,00 m' h_t = 160 cm h = 155 cm

$$\sigma_b / \sigma_a = -/1400$$

$k_o = \frac{M}{bh^2} = \frac{91500}{1 \times 155^2} = 3,8$

ω_o = 0,294 % = 45,5 cm^2/m'

Wapening = Ø 25 - 10 = 49 cm^2/m' of
Ø 32 - 17 = 47,5 cm^2

Verdeelwap. = 1/5 x 45,5 = 9,1 cm^2/m'
Wapening = Ø 16 - 20 = 10 cm^2/m' of
Ø 19 - 30 = 9,45 cm^2/m'

Steunpuntsmoment = reken

$1/12 \times 20300 \times 6^2 = 61000$ kgm/m'

$b = 1,00$ m' $h_t = 160$ cm $h = 155$ cm

$\sigma_b/\sigma_a = -/1400$

$k_o = \frac{M}{bh^2} = \frac{61000}{1 \times 155^2} = 2,5.$

$\omega_o = 0,191$ % = 29,6 cm^2/m'

Wapening = Ø 25 + Ø 14 - 20 = 32,2 cm^2/m' of
Ø 32 + Ø 19 - 34 = 32 cm^2/m'

Verdeelwapening = 1/5 x 29,6 = 6 cm^2/m'
Wapening = Ø 14 - 25 = 6,2 cm^2/m' of
Ø 16 - 30 = 6,5 cm^2/m'

Dwarskracht in plaat

$h_t = 1,60$ m'

Reken b = 2 x 5,00 = 10 m' (ontwikkeld)

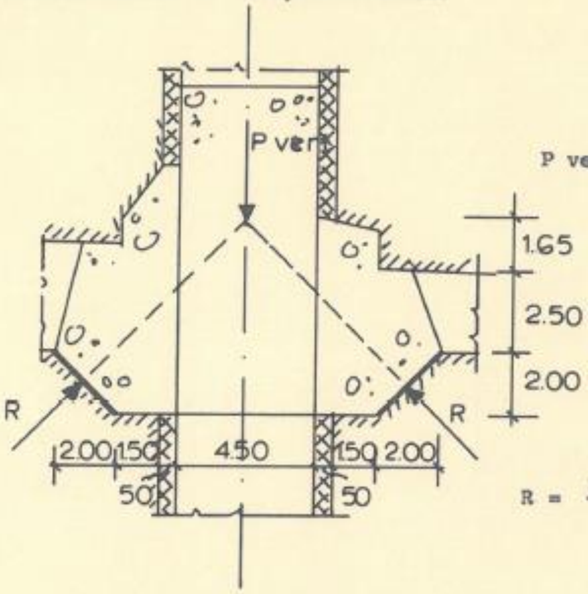
D = gewicht betoncilinder =

$10,75 \times \frac{\pi}{4} \cdot 4,5^2 \cdot 2,4 = 172 \cdot 2,4 = 412,5$ ton

$\rho = 3/2 \cdot \frac{412500}{1000 \times 1,60} = 2,6$ $kg/cm^2 < 7$

Geen opgebogen wapening vereist.

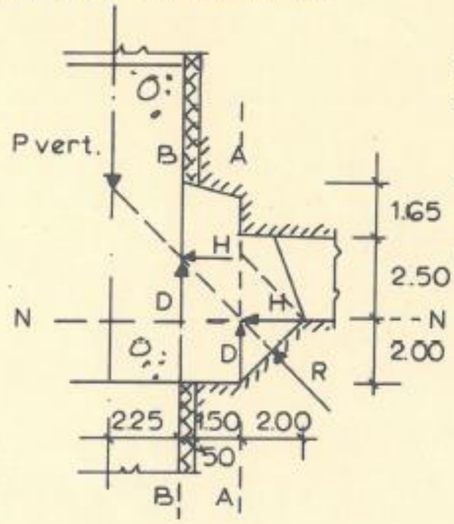
Gesteente-druk exclusief kleef v/d betonprop t.o.v.
de schachtwand en de laadplaatswanden.



P vert. = waterkolom = 2450 t
vulstenen = 260 t
betoncilinder = 413 t
Totaal = 3123 t

$R = \frac{3123}{2} \times \sqrt{2} = 1561^5 \sqrt{2}$
$= 2220$ ton

Hoofd- trek- en drukspanning



in vlak A-A werken
t.g.v. R = 2220 ton

a) een dwarskracht D van 1561^5 ton
b) een horizontale component H van 1561^5 ton excentrisch op doorsn. A-A

In het snijpunt van N-N en A-A komen de volgende spanningen

Doorsn. A-A

Horizontale = verticale schuifspanning

$$\tau = \frac{1561500}{400 \times k_z \times 445} = \frac{1561500}{400 \times 0,985 \times 445} = 8,9 \text{ kg/cm}^2$$

Mom. tengevolge van D in A = 1561500 x 1 = 1561500 kg/m

$$k_o = \frac{M}{bh^2} = \frac{1561500}{4 \times 450^2} = \frac{1561500}{810000} \sim 2$$

$k_z = 0,985$

Normaalspanning

$$\sigma = \frac{1561500}{400 \times 400} = \frac{1561500}{160000} = 9{,}75 \text{ kg/cm}^2$$

Hoofdspanning

$$\sigma' = \frac{9{,}75}{2} \pm \sqrt{\left(\frac{9{,}75}{2}\right)^2 + (8{,}9)^2}$$

$$= 4{,}88 \pm 10{,}1$$

Hoofddrukspanning = 4,88 + 10,1 = 14,98 kg/cm^2

Hoofdtrekspanning = 4,88 - 10,1 = - 5,22 kg/cm^2

Doorsnede B-B

Nom. tengevolge van D in B =

1561500 x 3 = 4684500 kgm

$$k_o = \frac{4684500}{4 \times 615^2} = \frac{4684500}{1500000} = 3{,}1$$

$$k_z = 0{,}976$$

Horizontale = verticale schuifspanning

$$\tau = \frac{1561500}{400 \times 0{,}976 \times 610} = \frac{1561500}{238000} = 6{,}6 \text{ kg/cm}^2$$

Normaalspanning

$$\sigma = \frac{1561500}{400 \times 430} = \frac{1561500}{172000} = 9{,}1 \text{ kg/cm}^2$$

Hoofdspanning

$$\sigma' = \frac{9{,}1}{2} \pm \sqrt{\left(\frac{9{,}1}{2}\right)^2 + (6{,}6)^2}$$

$$= 4{,}55 \pm 8{,}03$$

Hoofddrukspanning = 4,55 + 8,03 = 12,59 kg/cm^2

Hoofdtrekspanning = 4,55 - 8,03 = -3,48 kg/cm^2

Ponsspanning in doorsnede B-B

De ponsspanning bedraagt

hoogte afschuifvlak 6,15 m

breedte afschuifvlak 2/3 x π x 4,50 = 9,45 m'

Ponsspanning

$$\rho = \frac{31230000}{945 \times 615} = 5,4 \text{ kg/m}^2$$

Stellen wij: kubusdrukvastheid k_o = 225 kg/cm^2

kubustrekvastheid k_t = 25 kg/cm^2

Normaalspanning t.g.v. H = 1561,5 ton

σ = 9,1 kg/cm^2

De ponsvastheid

$$\rho_k = \sqrt{(k_o - \sigma)(k_t + \sigma)}$$

$$= \sqrt{(225 - 9,1)(25 + 9,1)}$$

$$= 86 \text{ kg/cm}^2$$

De veiligheidsfactor is dan $\frac{86}{5,4}$ = 15,9

Wapening cilinder

Minimum wapening volgens V.V.A.A. = 0,3 %

$$A = 0,3 \cdot \frac{\pi}{4} \cdot \frac{4,50^2}{100} = 477 \text{ cm}^2$$

Wapening Ø 25 - 14 (99 stuks Ø 25 = 485 cm^2)

Berekening wandbekisting

Gerekend wordt met één trek per kwartier; per trek wordt gestort 3 m^3.
Na 4 uur begint de beton op te stijven, zodat de zijwaartse druk 4 uur lang toeneemt en daarna constant blijft.
In 4 uur wordt gestort 4 x 3 x 4 = 48 m^3.

Gemiddelde lengte v/d betonprop = $\frac{12,50 + 10,00}{2}$ = 11,25 m'

Breedte v/d prop = 4,5 m'
De storthoogte behorende bij 48 m^3 is dan

$h = \frac{48}{11,25 \; . \; 4,50} = 0,95$ m' Reken 1,00 m'

De rails h = 11 cm worden onder 60^o geplaatst en op 2 plaatsen ondersteund door horizontale moerbalken.
Elk veld 1,50 m' lang. De rails staan onderling 50 cm hart op hart.

$q = 0,50 \times 2400 = 1200$ kg/m' $\sigma_t = 700$ kg/m^2

$M_{max} = 1/8 \times 1200 \; . \; 1,50^2 = 340$ kgm

W vereist = $\frac{34000}{700} = 48,6$ cm^3

W. aanw. = $0,06 \; . \; h^3 = 0,06 \; . \; 11^3 = 80$ cm^3

Houten wand aan binnenzijde tegen de rails

$q = 2400$ kg/m^2 $l = 0,50$ m'

$M_{max} = 1/12 \; . \; 2400 \; . \; 0,5^2 = 50$ kgm/m' $\sigma_t = 70$ kg/cm^2

W vereist = $\frac{5000}{70} = 71,5$ cm^3

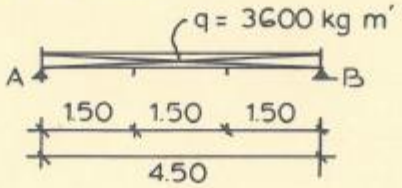
W aanwezig = $1/6 \; . \; 100 \; . \; d^2 = 71,5$ $d^2 = \frac{6 \; . \; 71,5}{100} = 4,3$ cm

d = 2,1 cm kies wand 2,5 cm dik

Moerbalken h.o.h. 1,50 m' l = 4,50 m'

De moerbalken worden op afstanden van 1,50 m' gesteund door schoren (op 2 plaatsen).

q = 1,5 x 2400 = 3600 kg/m'



M_{max} = 1/8 x 3600 x $1,5^2$ =

~ 1000 kgm

W vereist = $\frac{100000}{1400}$ = 75 cm^3

Aanwezig voorspanbalken

DIR. 12
- W_x = 288 cm^3
- G = 52,1 kg/m'

DIN 14
- W_x = 216 cm^3
- G = 33,7 kg/m'

Drukkracht in schoren 1,5 x 3600 = 5400 kg = 5,4 ton

Berekening wandbekisting v. gang op 142 m verdieping

Gerekend wordt met één trek per kwartier; per trek wordt gestort 3 m^3.
Na 4 uur begint de beton op te stijven, zodat de zijwaartse druk 4 uur lang toeneemt en daarna constant blijft.
In 4 uur wordt gestort 4 x 3 x 4 = 48 m^3.
De storthoogte behorende bij 48 m^3 is dan H.
Stel hoogte is 2 m dan

$\frac{\pi}{4} \times 4{,}50^2 \times 2 + 3{,}5 \times 2{,}5 \times 2 = 32 + 17{,}5 = 49{,}5\ m^3$.

Dus 2 m hoogte aanhouden.
De zijwaartse druk is dan 2 x 2400 = 4800 kg/m'.

$\sigma_t = 700\ kg/cm^2$

1) Rails

Neem afstand rails 40 cm.
De rails H = 11 cm en op h.o.h. 0,75 m' ondersteund

q = 0,40 x 4800 = 1920 kg/m'

$M_{max} = 1/8 \times 1920 \times 0{,}75^2 = 135$ kgm

W vereist $= \frac{13500}{700} = 19{,}3\ cm^3$

W aanwezig $0{,}06\ h^3 = 0{,}06 \times 11^3 = 80\ cm^3$

2) Liggers

q = 0,75 x 4800 = 3600 kg/m'

$M_{max} = 1/8 \times 3600 \times 2{,}6^2 = 3050$ kgm

W vereist $= \frac{305000}{1400} = 217\ cm^3$

Neem HE 160 - B $\longrightarrow W_x = 311\ cm^3$

<u>Houten wand a/d binnenzijde v/d rails</u>

$q = 4800\ kg/m^2 \qquad l = 0,4\ m'$

$M_{max} = 1/12 \times 4800 \times 0,4^2 = 64\ kgm$

$\sigma_t = 70\ kg/cm^2$

$W\ vereist = \frac{6400}{70} = 91,5\ cm^3$

$W\ aanwezig = 1/6 \times 100 \times d^2 = 91,5$

$d^2 = \frac{91,5 \times 6}{100} = 5,5\ cm$

Neem d = 2,5 cm

Berekening wandbekisting v. gang op 134 m verdieping geheel als wandbekisting op 142 m verdieping.

Fig. 86: Static calculation shaft barrier shaft II Wilhelmina /44/

AANVULLENDE BEREKENING BEHORENDE BIJ NR. 2043 NIEUWBOUW D.S.M.

Bij de berekening van de oplegdruk van de prop is schacht II op de pagina's 4 en 5 is geen rekening gehouden met de invloed van de "kurk" welke boven op de prop (volgens tekening 4B 44257) gestort wordt tot vlak boven de 134 m. verd.
Wordt de "kurk" wel in de berekening betrokken dan kan men de volgende mogelijkheden onderscheiden:

A. Er wordt <u>geen</u> rekening gehouden met <u>Kleef</u>
In dit geval wordt het te dragen gewicht

de betonprop 327 x 2,4 =	785 ton
de kurk 25 x 16 x 2,4 =	960 ton
waterkolom $\frac{\pi}{4} \times 4,5^2 \times 128,50 \times 1$ =	2050 ton
vulstenen $3 \times 4,5 \times \frac{\pi}{4} \times 4,50^2 \times 1,2$ =	<u>200 ton</u>
Totaal	4055 ton

Oplegdruk wordt dan $\frac{\frac{1}{2}\sqrt{2} \times 4055}{11,3}$ = 254 ton/m2 = 25,4 kg/cm2

B. Er wordt <u>wel</u> rekening gehouden met <u>Kleef</u>
Bij een totaal Kleef-oppervlak van 149 (van prop) + 25 x π x 4,5 (van kurk)= 504 m2 wordt de oplegdruk reeds gelijk aan 0 kg/cm2 als de Kleef gelijk is aa
$\frac{4055000}{5040000}$ = o,81 kg/cm2

w.g. ir. Zurhaar,

Fig. 87: Additional calculation shaft barrier shaft II Wilhelmina /44/

The coordinates of shaft II Wilhelmina are:

RD-x:	199863
RD-y:	320378
elevation:	+157 m NAP
positional accuray:	+/- 1 m

According to the coordinates the shaft is located in a wooded area southwards "Tunnelweg" (community Kerkrade).

7.3 Shaft I, Emma

The vertical Shaft I of the state mine Emma was drilled in 1909. In 1974 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 6,0 m diameter. The shaft was drilled to a total depth of 900,0 m and was used as travelling shaft, drawing shaft and downcast drafting shaft. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 198 m respectively the carbon surface is located on -92 m NAP /6/. The shaft I Emma has 12 documented insets. The 259 m floor, as the topmost is located in a level of -153,0 m NAP and in a depth of 259 m /6//50/.

In the following figure the strata in the range of the 259 m floor is pictured. Here mainly occur layers of slate as well as Laag III /52/.

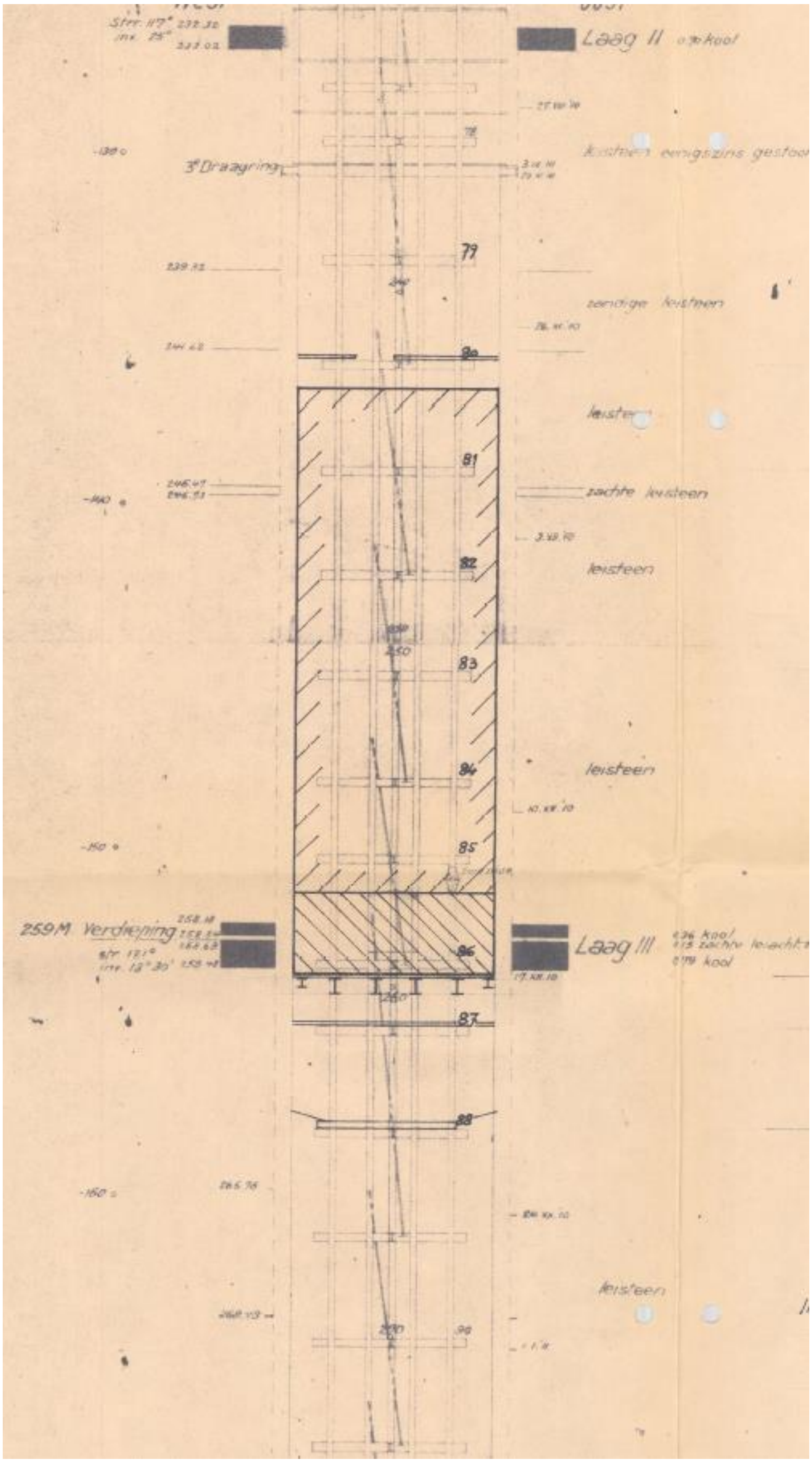


Fig. 88: Strata shaft I Emma, 259 m floor /52/

In 1974 a load bearing filling out of 511 m³ of a mixture of concrete (length 17,60 m) was embedded in the shaft underneath the 259 m floor. Within the filling a steel tube (ø 1.000 mm) was inserted /52/. The reason for inserting the steel tube was to provide an opening to install submersible pumps to potentially lower the mine water level. The upper end of the steel tube was sealed with a layer of 1,42 m of concrete. The lower end was left open. The shaft barrier was used as load bearing filling. For a maximum friction of the filling the shaft walls were cleaned and drawn off. Finally the shaft was covered up with a welded steel panel /14/. In 1977 the shaft was backfilled with 7.299,5 m³ sand /17/.

The following figures show the shaft barrier of shaft I Emma.

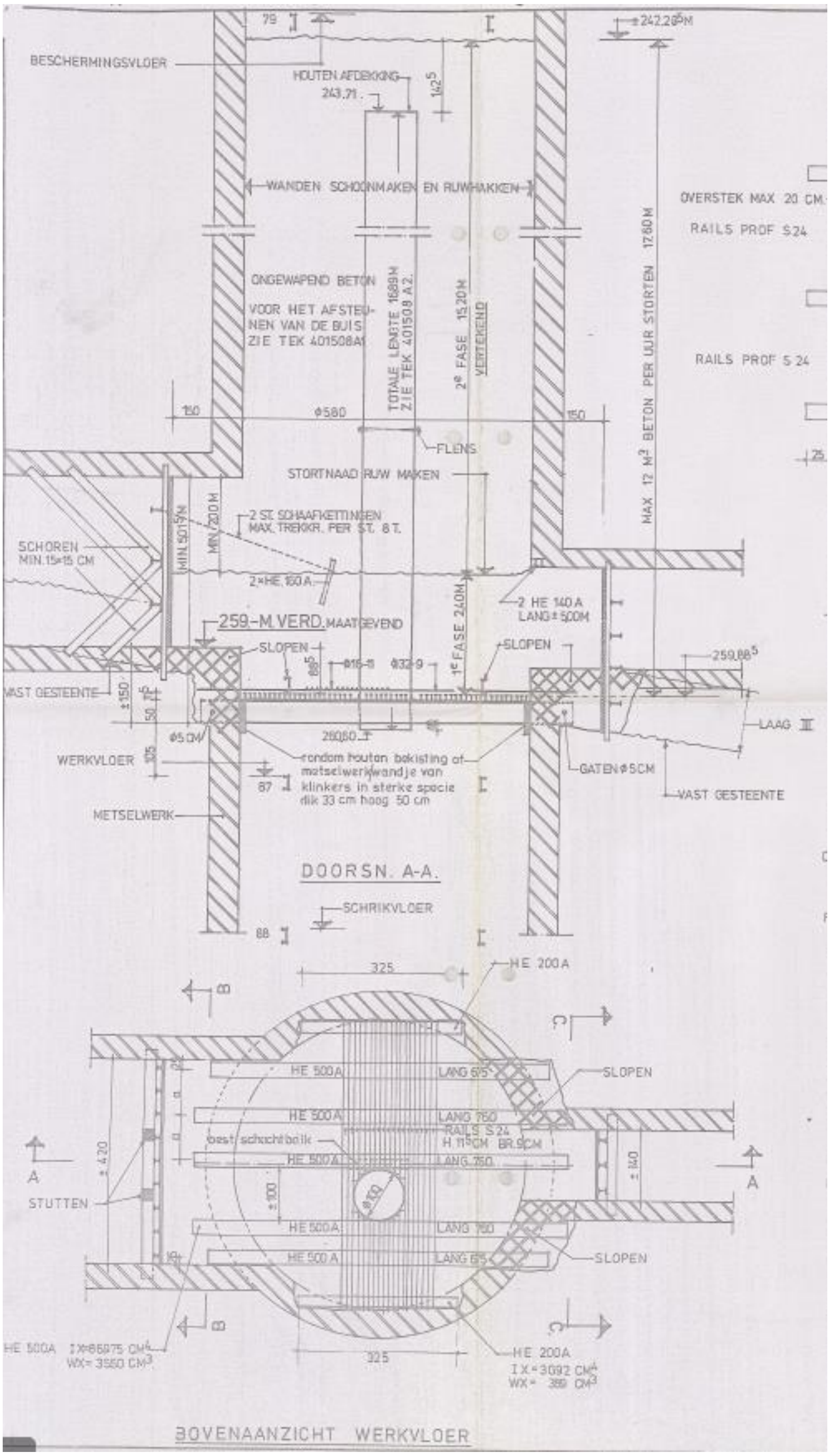
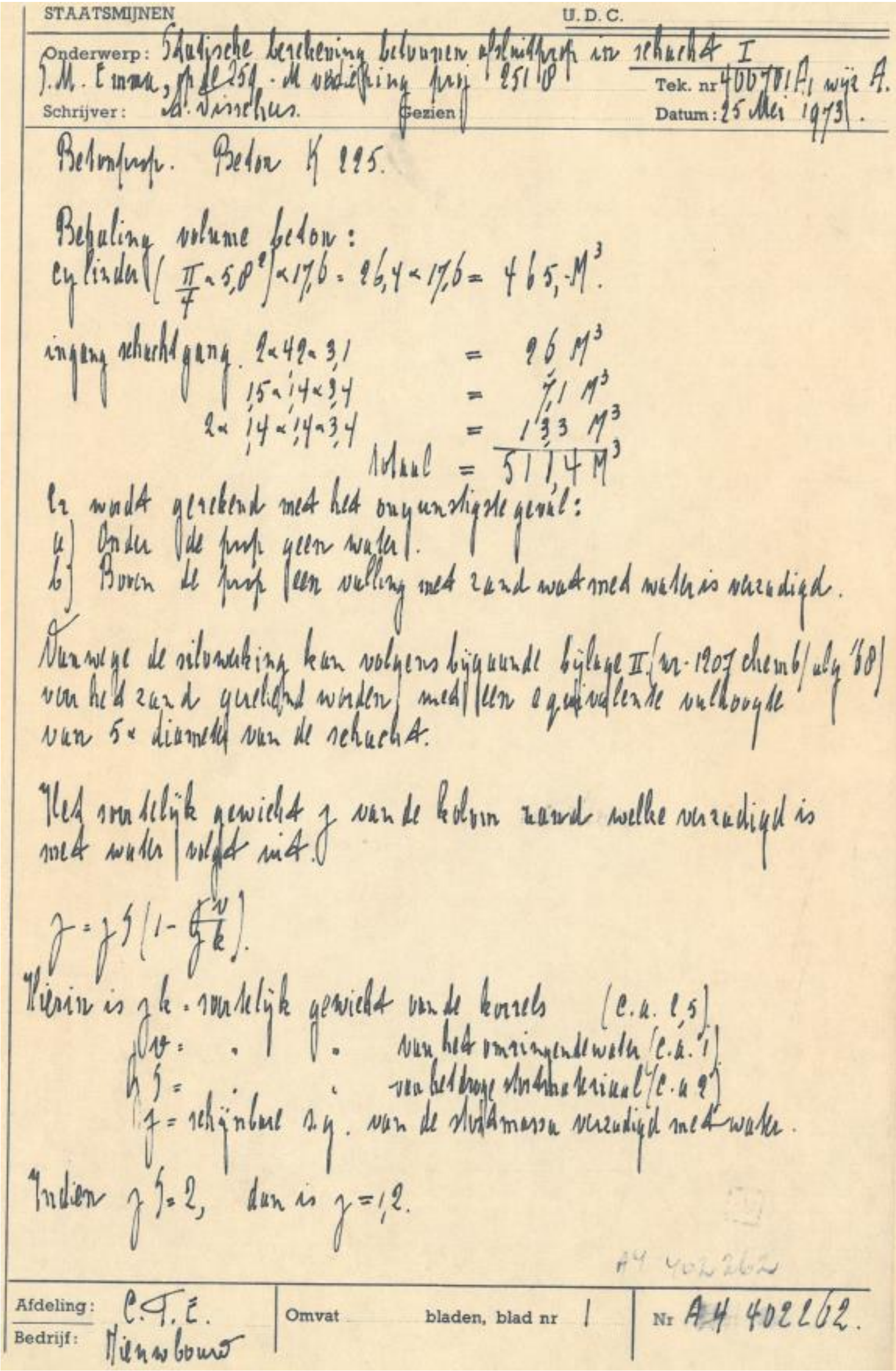


Fig. 89: Shaft barrier shaft I Emma /52/

Static calculations of the shaft barrier of the shaft I Emma are existent /52/. Compare the following figures.



STAATSMIJNEN U.D.C.

Onderwerp: Statische berekening betonnen afsluitprop in schacht I S.M. Emma, op de 259 - M verdieping prop 251 M

Tek. nr 400701 A1 wijz. A.

Schrijver: A. Vinchus. Gezien:

Datum: 25 Mei 1973.

Betonprop. Beton K 225.

Bepaling volume beton:

cylinder ($\frac{\pi}{4} \times 5{,}8^2$) $\times 17{,}6 = 26{,}4 \times 17{,}6 = 465{,}-$ M³.

ingang schachtgang. $2 \times 4{,}2 \times 3{,}1 = 26$ M³

$1{,}5 \times 1{,}4 \times 3{,}4 = 7{,}1$ M³

$2 \times 1{,}4 \times 1{,}4 \times 3{,}4 = 13{,}3$ M³

totaal $= 511{,}4$ M³

Er wordt gerekend met het ongunstigste geval:

a) Onder de prop geen water.

b) Boven de prop een vulling met zand wat met water is verzadigd.

Vanwege de silowerking kan volgens bijgaande bijlage II (nr. 1207 chemb/ cly '68) van het zand gerekend worden met een equivalente vulhoogte van 5× diameter van de schacht.

Het soortelijk gewicht γ van de kolom zand welke verzadigd is met water volgt uit:

$$\gamma = \gamma_s \left(1 - \frac{\gamma_v}{\gamma_k}\right).$$

Hierin is γ_k = soortelijk gewicht van de korrels (c.a. 2,5)

γ_v = " " van het omringende water (c.a. 1)

γ_s = " " van het droge stortmateriaal (c.a. 2)

γ = schijnbare s.g. van de stortmassa verzadigd met water.

Indien $\gamma_s = 2$, dan is $\gamma = 1{,}2$.

A4 402262

Afdeling: C.T.E.

Bedrijf: Nieuwbouw

Omvat bladen, blad nr 1

Nr A4 402262.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitprop schacht I, S.M. Emma. projekt 2518.

Tek. nr 400701 A1 wijz. A.

Schrijver: Gezien: Datum:

De totale vertikale kracht volgt door sommering van de volgende komponenten.

A.) Gewicht van de cylinder van de betonprop.
B.) De hydrostatische druk.
C.) De door het vulmateriaal op de prop uitgeoefende druk (silowerking)

Totale vertikale kracht.

A) Betonprop.	465 × 2,3 =	1070	ton.
B) Waterkolom	1 × 26,4 × 243 =	6420	ton.
C) Vulsteen/zand	26,4 × 5 × 5,8 × 1,2 =	920	ton.
	totaal =	8410	ton

We berekenen de prop als wrijvingsprop, waarbij de kleef op 3 kg/cm² aangenomen wordt.

Een en ander gezien:

1) De onrondheid van de schacht, die in orde van grootte cm's bedraagt.

2) De krimp van de prop, die in orde van grootte mm's zal bedragen.

3) De ruwheid van de schacht- en landgangwanden, er van uitgaande dat gladde stukken bewust geruwd zullen worden.
(De schachtwanden zijn van metselwerk.)

Afdeling: C.T.E. Omvat bladen, blad nr 2. Nr A4 402262.
Bedrijf: Hoensbroek.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitprop schacht I, S.M. Emma
Projekt 951 P.

Tek. nr 400 701 A, wijz A.

Schrijver: Gezien: Datum:

Gedeelte van de schachtwand onderbroken.

door laadgangen $2 \times 4{,}2 \times 3{,}1 = 26{,}- \; M^2$

Dit komt overeen met rondom een schachtwandhoogte van.

$$\frac{26}{\pi \times 5{,}8} = \frac{26}{18{,}2} = 1{,}43 \text{ M.}$$

Berekening prophoogte.

Benodigde prophoogte. $\frac{8410}{30 \times 18{,}2} + 1{,}43 = \frac{8410}{546} + 1{,}43 = 15{,}40 + 1{,}43 = 16{,}83$ M.

(aanw. 17,60 M.)

e.g. natte betonplaat $2{,}4 \times 2600 = 6250$ kg/M²
e.g. stalen sleen rekenen. 600

Totaal = 6850 kg/M²

Afdeling: C.T.E. | Omvat bladen, blad nr 3. | Nr A4 402 262.

Bedrijf: Hoensbroek

STAATSMIJNEN U.D.C.

Onderwerp: Schmidtgroep schacht I, S.M. Emma.
Projekt 2511 P. Tek. nr 400701 A,

Schrijver: Gezien: Datum:

Moerbalken. h.o.h. $\frac{4,20 - 2 \times 0,25}{3} = \pm 1,34$ M. $l = 5,8 + 0,6 = 6,4$ M.

$M_{max} = 1/8 \times 6850 \times 6,4^2 \times 1,34 = 46000$ kgm.

W_x ben $= \frac{4600000}{1400} = 3350$ cm³. H.E. 500 A. $W_x = 3550$ cm³; $I_x = 86975$ cm³; $G = 155$ kg/M'

$\sigma_b = \frac{4600000}{3550} = 1320$ kg/cm²

$f = \frac{5}{384} \times \frac{1,34 \times 68,5 \times 640^4}{2,1 \times 10^6 \times 86975} = 1,14$ cm. $\frac{640}{1,14} = 560 = \frac{1}{560}\, l$.

σ oplegdruk $\frac{(2,9 \times 1,34 \times 6850) \times 2}{60 \times 30} = \frac{53300}{1800} = 29,5$ kg/cm²

Randbalken $l = 3,25$ M. breedte $\frac{\frac{5,00 - 4,20}{2} - 0,25}{2} + 0,25 = 0,53$ cm

$q = 0,53 \times 6850 = 3630$ kg/M'. $M = 1/8 \times 3630 \times 3,25^2 = 4730$ kgm.

W ben $= \frac{473000}{1400} = 338$ cm³. H.E. 200 A $W_x = 389$ cm³; $I_x = 3692$ cm⁴; $G = 42,3$ kg/M

$f = \frac{5}{384} \times \frac{36,3 \times 325^4}{2,1 \times 10^6 \times 3692} = 0,7$ cm. $\frac{320}{0,7} = \frac{1}{460}\, l$.

σ oplegdruk. $\frac{1,62 \times 3630 \times 2}{50 \times 20} = \frac{11000}{1000} = 11,0$ kg/cm²

Afdeling: C.T.E. Omvat bladen, blad nr 4. Nr A4 402262.
Bedrijf: Nieuwbouw

STAATSMIJNEN U.D.C.

Onderwerp: afdichtprop schacht I, S.M. Emma.
project 1251P.

Tek. nr 400701 A1.

Schrijver: Gezien: Datum:

Dek gevormd door railprofielen S 24 h.o.h. gem.

$l = \frac{4{,}20 - 2 \times 0{,}25}{3} = \pm 1{,}25$ M.

profiel S. 24. ⟶ Reken i.v.m. slijtage:

$W_x = 0{,}8 \times 97{,}3 = 77\ cm^3$
$I_x = 0{,}8 \times 569 = 455\ cm^4$
$\bar{\sigma}_t = 700\ kg/cm^2$

We berekenen het dek nu verder per M'. breedte.

$q = 1 \times 6850 = 6850\ kg/M'$

$M_{max} = 1/8 \times 6850 \times 1{,}25^2 = 1340$ kgm.

W_x nodig $= \frac{134000}{700} = 192\ cm^3$ W_x aanw $= 11 \times 77 = 850\ cm^3$

Ber. I_x voor $f < \frac{1}{600} l = 37{,}2 \times 1{,}25 \times 6{,}850 \times 1{,}25^2 = 510$.

I_x aanwezig $= 11 \times 455 = 5050\ cm^4$

M uitkraging $= 1/2 \times 6850 \times 0{,}4^2 = 550$ kgm.

I_x ben voor $f < \frac{1}{400} l = 1{,}5 (357 \times 6{,}8 \times 0{,}4^3) = 234\ cm^4$

I_x aanwezig $= 5050\ cm^4$

Afdeling: C.T.E.
Bedrijf: Nieuwbouw.

Omvat bladen, blad nr 5.

Nr A4 402262.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitpropp schacht I, S.M. Emma.
Project 1251.
Tek. nr 400701 A1 wijz A.

Schrijver: Gezien: Datum:

Gewapend betonplaat. hoogte 2,40 M. lengte 6,30 M.

Beton: K. 225 Staal QR. 40.

De betonplaat moet het gewicht van de nog te storten betonprop dragen.

te storten beton 15,2 × 2,6 = 39,50 ton/M².
e.g. betonplaat 2,4 × 2,4 = 5,75 ton/M².
Totaal = 45,25 × 26,4 = 1200 ton.

Oppervlak van de dragende middenstrook 4,20 M (breed) ± 5,- M (lang) = 21,- M².

Belasting op de middenstrook per M² = $\frac{1200}{21}$ = 57,2 ton/M²

M veld: 1/8 × 57200 × 6,3² = 286000 kgm. × 1,5 = 430.000

234 = h₀ √ 286 000   h₀ = 0,438   f = 0,24 × 234 = 56,2 c².
hoofdwap. Ø 32-13 = 61 c²   verdeelwap. 1/5 × 56,2 = 11,2 c²   Ø 16-17 = 11,8 c².

τ max = $\frac{1200\ 000}{2 \times 430 \times 0{,}85 \times 234} = \frac{1200\ 000}{171\ 000}$ = 7 kg/c²

Max opleg kracht van betonprop. = 1200 000 kg
e.g. belasting 26,4 × 600 = 15 700 kg
1215 700 kg

oplegdruk $\frac{1215700}{(60+60) \times 420} = \frac{1215700}{50400}$ = 24,1 kg/c².

De beton draagt ook nog op het vast gesteente.

Afdeling: C.T.E. Omvat bladen, blad nr 6. Nr A4 402262.
Bedrijf: Nieuwbouw.

STAATSMIJNEN IN LIMBURG U. D. C.

Onderwerp: Statische berekening betonnen afsluitdop in schacht I S.M. Emma, op de 259-m verdieping prof. 25 118 / 2641 Tek. nr 400701 A1 wijz B.

Schrijver: A. Visschers. Gezien: Datum: 10 Mei 1974.

Wijziging op blz. 6.

t.g.v. het plaatsen van de buis ø 100cm is de belasting.

Aangenomen tot 1,5 × 57,2 ton/m² = 86 ton/m²

M veld = 1/8 × 86 × 6,3² = 430 000 kgm.

234 = ko √430000 ko = 0,357. fy = 0,385 × 234 = 90 cm² wap ø32-9.

Afdeling: C.T.E. Omvat bladen, blad nr 6A. N: A4-402262.

Bedrijf: Nieuwbouw

STAATSMIJNEN U. D. C.

Onderwerp: afdichting schacht I, S.M. Emma. Project 2510.

Tek. nr 400701 A1.

Schrijver: Gezien: Datum:

Berekening gangafdichting.

Gerekend wordt met een hoeveelheid per kwartier van 3.- M³ beton
Na 4 min begint de beton op te stijven, dan gestort 4.- × 4.- × 3.- = 48 M³
Snelste stijghoogte bij het vullen van de ronde schachtdoorsnede.
Stijghoogte in 4 min. $\frac{48}{26,4} = 1,82$ M. Max druk $1,82 \times 2600 = 4720$ Kg/M².

Hout l max = 0,5 M.

$M = \frac{1}{10} \times 4720 \times 0,5^2 = 118$ Kgm. $W_{ben} = \frac{11800}{90} = 168$ cm³

W aanwezig $1/6 \times 100 \times 3,2^2 = 170$ cm³

Rails l max = 1,- M.

max belasting $= 0,5 \times 4720 = 2360$ Kg/M′ $M = 1/8 \times 2360 \times 1^2 = 295$ Kgm.

W ben. $= \frac{29500}{700} = 42,2$ cm³ W rails per stuk = 77 cm³.

Horizontale liggers l max = 1,75 M (linker gang en rechter gang)

$M = 1/8 \times 4720 \times 1,75^2 = 1800$ Kgm. $W_{ben} = \frac{180000}{1400} = 129$ c³ H.E. 140 A. $W_x = 155$ cm³

Trekkracht per steunpunt. $1,75 \times 4720 = 8250$ kg
Max trekkracht op liggers (kleine gang rechts) $1 \times 8250 = 8250$ Kg (in het midden)
$M = 1/4 \times 8250 \times 1,6 = 3300$ Kgm. (uit trekkracht naar links)
$M' = 1/8 \times 0,7 \times 4720 \times 1,6^2 = 1060$ Kgm. (betondruk naar rechts)
totaal = 4360 Kgm.

$W_{ben} = \frac{4360}{1400} = 160$ cm³ H.E. 160 A. $W_x = 220$ cm³.

Afdeling: C.T.E. | Omvat bladen, blad nr 7. | Nr A4 402262.
Bedrijf: Nieuwbouw

STAATSMIJNEN IN LIMBURG U.D.C.

Onderwerp: STATISCHE BEREKENING BETONNEN AFSLUITPROP IN SCHACHT I S.M. EMMA, OP DE 259 M VERDIEPING. Proj: 2518; 2641 Tek. nr 400 701 - A1 wijz. B, 401 508 - A2

Schrijver: Schlösser. J.J. Gezien: Datum: 8 MEI 1974

AANVULLING T.G.V. STALEN DOORVOERPIJP.

Berekening betonhoogte boven st. pijp.

Opp. hor. schachtdrsn. $= \frac{\pi}{4} \times 5{,}8^2 =$ 26,4 m^2

Tot. vert. kracht = 8410 ton, dit is $\frac{8410}{26{,}4} =$ 320 ton/m^2

Diam. houten deksel = 1,05 m. → opp. = 0,87 m^2

Schuifspanning $= \frac{320 \times 0{,}87}{1{,}35 \times \pi \times 1} = 67\ t/m^2 = 6{,}7$ kg/cm^2.

De benodigde betonhoogte is dus 1,35 m.

Houten deksel

dikte geschaafd = 71 mm. dikte ongeschaafd = 75 mm.

Belasting = 1,4 × 3000 = 4200 kg/m^2.

$M = 1/8 \times 4200 \times 1{,}01^2$ = 540 kgm.

$W = 1/6 \times 100 \times 7{,}1^2$ = 840 cm^3

$I = 1/12 \times 100 \times 7{,}1^3$ = 2980 cm^4

$\sigma_b = \frac{54000}{840} = 65$ kg/cm^2 < 70 kg/cm^2

Ben. $I = 0{,}651 \times 4200 \times 1{,}01^3 = 2820\ cm^4$ < 2980 cm^4

$\sigma_{opl} = \frac{0{,}87 \times 4200}{316} = 12$ kg/cm^2. < 20 kg/cm^2.

Stalen pijp ⌀ 1016/996 mm.

Storthoogte bij 48 $m^3 = \frac{4 \times 12}{\frac{\pi}{4} \times 5{,}8^2} = \frac{48}{26{,}4} = 1{,}82$ m

Max. druk (hor.) = 1,82 × 3000 = 5500 kg/m^2.

Reken voor eenzijdige uitwendige druk →

$M = 1/36 \times 5500 \times 0{,}51^2 \times 3{,}37 = 144$ kgm

$\sigma = \frac{14400}{1/6 \times 100 \times 1^2} = \frac{14400}{16{,}66} = 870$ kg/cm^2

Afdeling: OTE Omvat bladen, blad nr 8 Nr A4 402 262.
Bedrijf: NWB

Fig. 90: Static calculation shaft barrier shaft I Emma /52/

The coordinates of shaft I Emma are:

RD-x:	193855
RD-y:	326853
elevation:	+106 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space close to a company car park westwards the roundabout of Emmaweg and Plato-Straat (community Brunssum).

7.4 Shaft II, Emma

The vertical Shaft II of the state mine Emma was drilled in 1909. In 1974 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,5 m diameter. The shaft was drilled to a total depth of 570,0 m and was used as travelling shaft, drawing shaft and downcast drafting shaft. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 200 m respectively the carbon surface is located on -95 m NAP /6/. The shaft II Emma has 8 documented insets. The 259 m floor, as the topmost is located in a level of -153,0 m NAP and in a depth of 258 m /6//50/.

In the following figure the strata in the range of the 259 m floor is pictured. Here mainly occur layers of slate and sandstone as well as Laag III /52/.

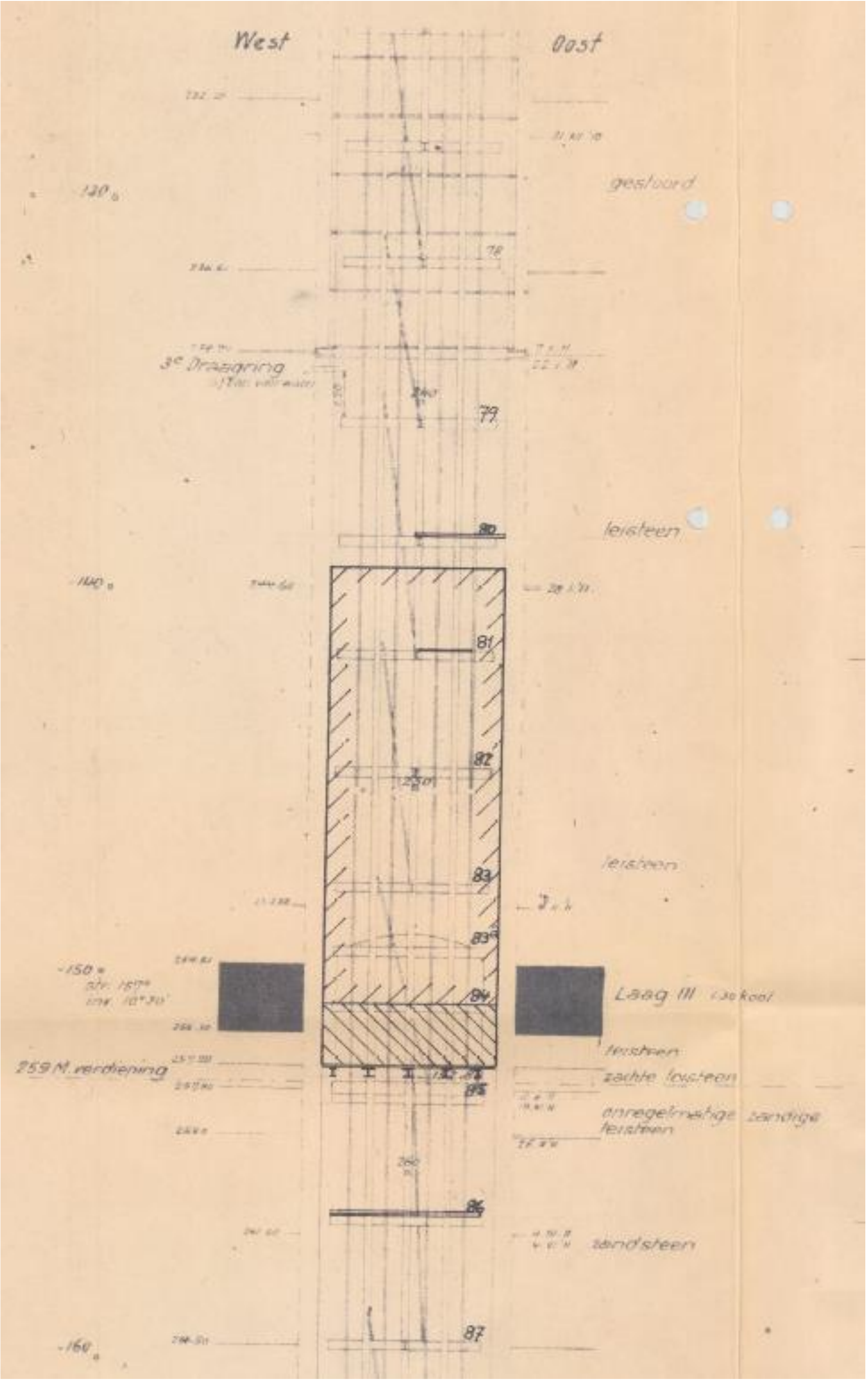


Fig. 91: Strata shaft II Emma, 259 m floor /52/

In 1974 the shaft was closed on the 259 m floor with a shaft barrier of a length of 13,4 m consisting of approximately 284 m³ of a mixture of concrete /52/. The barrier was constructed as load bearing filling. The remaining shaft column above the barrier was backfilled with approximately 3.900 m³ waste material /14/. In 1975 the shaft was provided with a concrete cover and two openings for refilling. Finally 1983 the two openings were closed with concrete /21//49/.

The figure below shows the shaft barrier of the shaft II Emma.

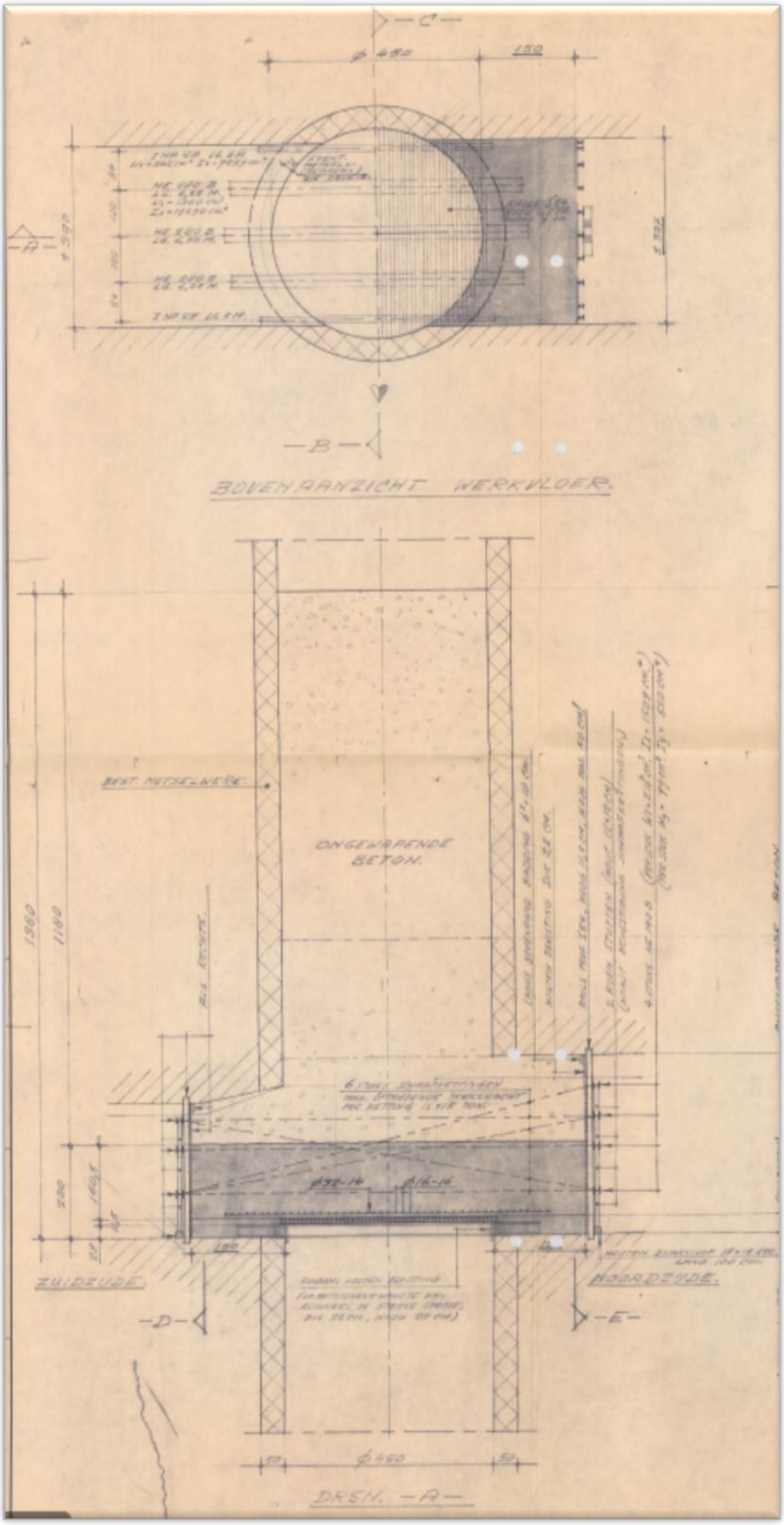


Fig. 92: Shaft barrier shaft II Emma /52/

Static calculations of the shaft barrier of the shaft II Emma are existent /52/. Compare the following figures.

Onderwerp: STATISCHE BEREKENING BETONNEN AFSLUITPROP IN SCHACHT II, SM. EMMA, OP DE 259 M. VERDIEPING. PROJ. 2518. **Tek. nr** 48 45.159.
Schrijver: SCHLÖSSER . J.J. **Gezien:** **Datum:** 12 MRT. '73
Wijz. A: 22 MEI '73
Wijz. B: 3 JULI '74

BETONPROP BETON: K 225.

BEPALING VOLUME BETON:

CILINDER	$: (\frac{\pi}{4} \times 4{,}50^2) \times 14{,}15$	=	225 M^3
BLOK NOORD	$: \{(3{,}15 \times 3{,}82) - (\frac{115}{360} \times 15{,}9)\} \times 3{,}60$	= ±	26 M^3
BLOK ZUID	$: \{(3{,}20 \times 3{,}90) - (\frac{122}{360} \times 15{,}9)\} \times 3{,}10$	= ±	23 M^3
UITDIEP. L.G.	$: 2 \times 1{,}40 \times 3{,}90 \times 0{,}65$	= ±	10 M^3
	TOTAAL	= ±	284 M^3

Er wordt gerekend met het ongunstigste geval:
a) Onder de prop geen water
b) Boven de prop een vulling van wasstenen die met water is verzadigd.

Vanwege de "SILOWERKING" kan volgens bijgaande BIJLAGE II (NR. 1207 CHEMB/ALG – '68) voor de wasstenen gerekend worden met een equivalente vulhoogte van 5× de diameter van de schacht.

Het soortelijk gewicht γ' van de kolom wasstenen welke verzadigd is met water volgt uit:

$$\gamma' = \gamma_s \left(1 - \frac{\gamma_v}{\gamma_k}\right)$$

Hierin is γ_k = soortelijk gewicht van de korrels (ca. 2,5)
γ_v = " " van de omringende vloeistof (ca. 1)
γ_s = " " van het droge stortmateriaal (ca. 2)
γ' = schijnbare s.g. van de stortmassa verzadigd met water

Indien $\gamma_s = 2$, dan is $\gamma' = 1{,}2$

Afdeling: CTE **Omvat** 12 **bladen, blad nr** 1B **Nr** 2910
Bedrijf: NIEUWBOUW

STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, ST. EMMA.
PROJECT 2518.
Tek. nr 4845159
Schrijver: Gezien: Datum: 12 MRT. '73
Wijz.A: 22 MEI '73
Wijz.B: 3 JULI '74

De totale verticale kracht volgt door sommering van de volgende componenten:
A/ Gewicht van de cilinder van de betonprop.
B/ De hydrostatische druk.
C/ De door het vulmateriaal op de prop uitgeoefende druk (silowerking)

TOTALE VERTICALE KRACHT:

A/ BETONPROP	: $225 \times 2{,}3$	=	518 TON
B/ WATERKOLOM	: $(\frac{\pi}{4} \times 4{,}5^2) \times 245 \times 1$	=	3896 "
C/ VULSTENEN	: $(\frac{\pi}{4} \times 4{,}5^2) \times 5 \times 4{,}5 \times 1{,}2$	=	430 "
	TOTAAL	=	4844 TON

We berekenen de prop als wrijvingsprop, waarbij de kleef op 3 kg/cm² aangenomen wordt.
Een en ander gezien:
1/ De onrondte van de schacht, die in orde van grootte cm's bedraagt.
2/ De krimp van de prop, die in orde van grootte mm.'s zal bedragen.
3/ De ruwheid van de schacht- en laadgangwanden, en van uitgaande dat gladde stukken bewust geruwd zullen worden. (De schachtwanden zijn van metselwerk.)

Afdeling: CTE
Bedrijf: NWB
Omvat 12 bladen, blad nr 2B Nr 2910

STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, SM. EMMA. PROJECT 2518.

Tek. nr 48 45 159

Schrijver: Gezien: Datum: 12 MRT. '73

Wijz. A: 22 MEI '73

Wijz. B: 3 JULI '74

GEDEELTE VAN SCHACHTWAND ONDERBROKEN DOOR LAADGANGEN:

NOORDZIJDE: $\left(\frac{115}{360} \times \pi \times 4{,}5\right) \times (3{,}5 - 0{,}4) = 14\ M^2$

ZUIDZIJDE : teken idem $= 14$ "

TOTAAL $= 28\ M^2$

DIT KOMT OVEREEN MET RONDOM EEN SCHACHTWAND-HOOGTE VAN $28 : 14{,}15 = 1{,}98$ m.

BEREKENING PROPHOOGTE

BENODIGDE PROPHOOGTE: $\frac{4844}{30 \times (\pi \times 4{,}5)} + 1{,}98 + 0{,}40$ (werkvl.) = **13,80 M**

(gerekend vanaf vloer laadgang)

STALEN VLOER OP 259 M. VERDIEPING.

Deze vloer dient als bekistingsvloer voor de 1,605 M. dikke betonplaat.

E.G. NATTE BETONPLAAT $1{,}905 \times 2600 = 4950$ kg/m²

E.G. STALEN VLOER REKEN $= 600$ "

$q = 5550$ kg/m²

Afdeling: CIE Bedrijf: NWB | Omvat 12 bladen, blad nr 3B | Nr 2910

STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, SM. EMMA.
PROJECT 2518.

Tek. nr 48 45159

Schrijver: Gezien: Datum: 12 MRT. '73

Wijz. B: 3-7-74

MOERBALKEN h.o.h. 1m , l = 4,92 m.

$q = 1 \times 5550 = 5550$ kg/m'

$M_{max} = 1/8 \times 5550 \times 4{,}92^2 = 16800$ kgm.

Max. oplegkracht $= (2{,}25 + 0{,}42) \times 5550 = 14800$ kg.

KIES HE 280 B. → $W_x = 1380$ cm^3
$I_x = 19270$ cm^4
$G = 103$ kg/m'

$$\sigma_b = \frac{1680000}{1380} = 1230 \text{ kg/cm}^2$$

$$f = \frac{5 \times 55{,}5 \times 492^4}{384 \times 2{,}1 \times 10^6 \times 19270} = 1{,}05 \text{ cm dit is } \frac{1}{463} l.$$

$$\sigma_{opl.druk} = \frac{2 \times 14800}{28 \times 42} = 25 \text{ kg/cm}^2$$

(verder nog spreiding door beton tussen balken)

RANDBALKEN l = 3,50 m

$$q = \left[(0{,}42 + 0{,}41) \times 4800 + \frac{404}{2/3 \times 0{,}84}\right] \times \frac{5550}{4800} = 5480 \text{ kg/m'}$$

$$M_{max} = \left[1/8 \times 4720 \times 3{,}5^2\right] \times \frac{5550}{4800} = 8400 \text{ kgm}$$

$$\text{Max. oplegkr.} = \left[\{(1{,}8 \times 0{,}48) + (1{,}2 \times 2/3 \times 0{,}35)\} \times 4800 + 1/2 \times 2/3 \times 720\right] \times \frac{5550}{4800} = 7000 \text{ kg}$$

KIES I NP 28 → $W_x = 542$ cm^3
$I_x = 7587$ cm^4
$G = 48$ kg/m'

$$\sigma_b = \frac{840000}{542} = 1550 \text{ kg/cm}^2$$

$$f = \frac{5 \times 54{,}8 \times 350^4}{384 \times 2{,}1 \times 10^6 \times 7587} = 0{,}682 \text{ cm dit is } \frac{1}{510} l.$$

$$\sigma_{oplegdruk} = \frac{2 \times 7000}{11{,}9 \times 42} = 28 \text{ kg/cm}^2$$ (verder nog spreiding ete)

Afdeling: OTE | Omvat 12 bladen, blad nr 48 | Nr 2910

Bedrijf: NWR

STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, SM. EMMA.
PROJECT 2518.
Tek. nr 4B 45 159
Schrijver: Gezien: Datum: 12 MRT. '73
Wijz. B: 3-7-'74

DEK GEVORMD DOOR RAILPROFIELEN S24 h.o.h. 9 cm. $l = 1$ m.

PROFIEL S24 → REKEN I.V.M. SLIJTAGE:

$W_x = 0{,}8 \times 97{,}3 = 77\ cm^3$

$I_x = 0{,}8 \times 569 = 455\ cm^4$

$\sigma_t = 700\ kg/cm^2$

WE BEREKENEN HET DEK NU VERDER PER M' BREEDTE.

$q = 1 \times 5550 = 5550\ kg/m'$

$M_{max.} = 1/8 \times 5550 \times 1^2 = 695\ kgm.$

$W_x\ vereist = \frac{69500}{700} = 100\ cm^3$

$W_x\ aanw. = \frac{100}{9} \times 77 = 850\ cm^3$

Ben. I_x voor $f < \frac{1}{600} l = 37{,}2 \times 5{,}55 \times 1^3 = 210\ cm^4$

$I_x\ aanw = \frac{100}{9} \times 455 = 5050\ cm^4$

$M_{uitkr.} = 1/2 \times 5550 \times 0{,}41^2 = 470\ kgm\ (< 695\ kgm)$

Ben. I_x voor $f < \frac{1}{900} l = 1{,}5(357 \times 5{,}55 \times 0{,}41^3) = 210\ cm^4$

$I_x\ aanw = 5050\ cm^4$

Afdeling: CTE
Bedrijf: NWB
Omvat 12 bladen, blad nr 5B
Nr 2910

STAATSMIJNEN **U. D. C.**

Onderwerp: AFSLUITPROP SCHACHT II, ST. EMMA

Tek. nr 48 45159

Schrijver: **Gezien:** **Datum:** 12 MRT '73

STORTHOOGTE BOVEN GEW. BETONPLAAT (bij 48 m³) Wijz. B: 3-7-'74

STORTHOOGTE TOT TOP ZUID. LAADGANG 1,35 M

BIJBEHORENDE BETON 29,8 × 1,35 = 40,20 M³

STORTH. TOT BEGIN. BOOG N. LAADGANG 0,26 M

BIJBEH. BETON (5,9 × 3,78) × 0,26 = 5,80 M³

STORTH. TOT TOP N. LAADGANG 0,50 M

BIJBEH. BETON {15,9 + ($\frac{2}{3}$ × 1,5 × 3,78)} × 0,5 = 19,68 × 0,5 = 9,85 M³

DE STORTHOOGTE VAN DE EERSTE 48 M³ BETON BOVEN DE PLAAT IS DUS: $1{,}35 + 0{,}26 + \frac{48 - 40{,}2 - 5{,}80}{19{,}68} = 1{,}71$ m.

REKEN q = 5000 kg/m² → toel. storthoogte $\frac{5000}{2600} = 1{,}92$ m

I.v.m. de ongelijkmatige stijging van de beton is bij de volgende 48 m³ beton de storthoogte met 0,18 m te verminderen.

STORTHOOGTE VANAF 0,40 M ONDER TOP N. LAADGANG

(1,35 + 0,26 + 0,50 − 1,71 = 0,40)

Om de druk op de gangafdichting constant te laten blijven, mogen we de eerste 4 uur de beton niet meer dan 1,92 − 0,18 = 1,74 m laten stijgen.

In deze 4 uur mag dus gestort worden:

(40,2 + 5,8 + 9,85 − 48) + (1,74 − 0,40) × 15,9 = 29,15 M³ beton.

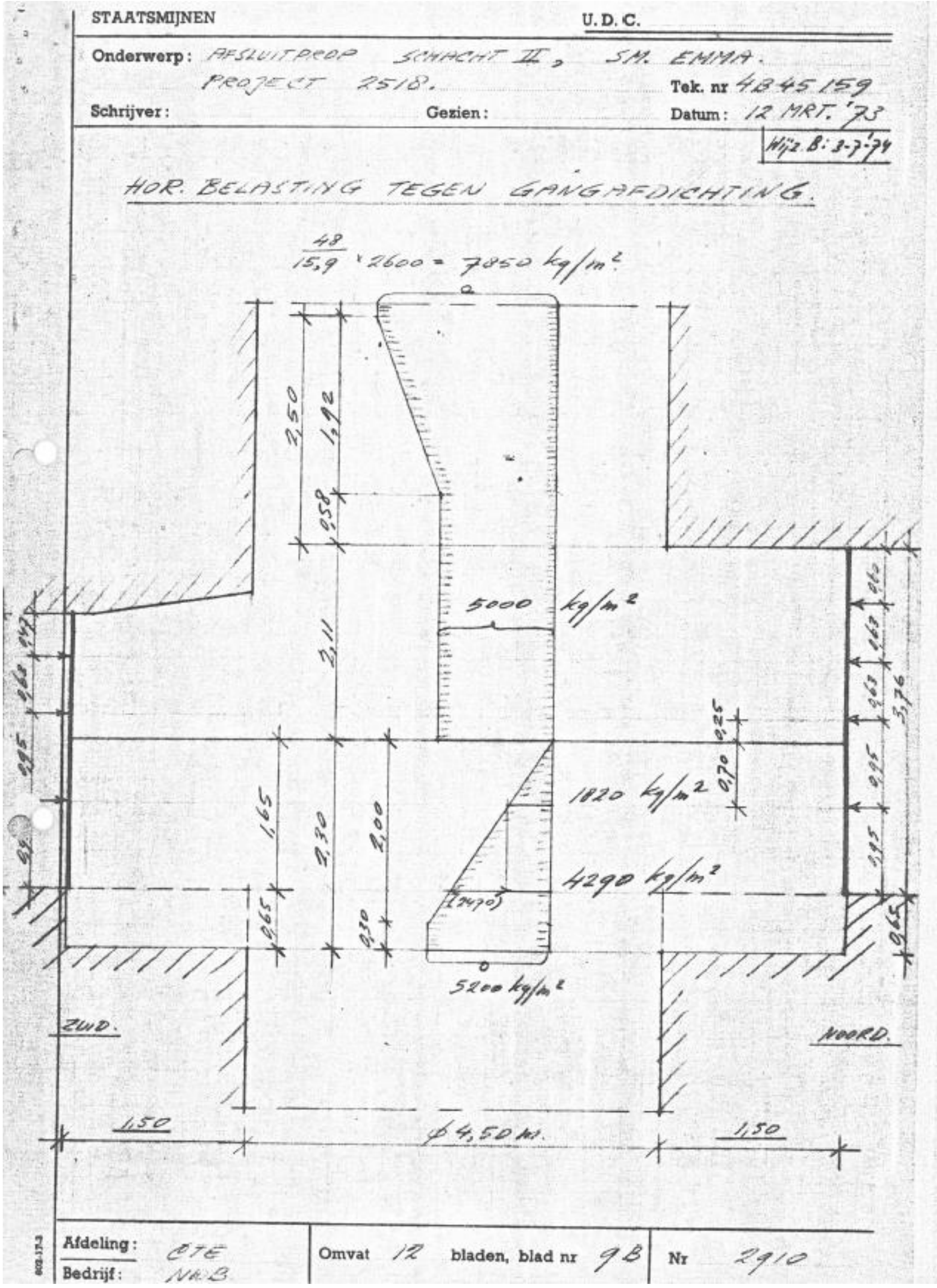
We nemen vanaf top N. laadgang een trek per half uur

Na 4 uur is dan $7{,}85 + (4 - \frac{7{,}85}{48} \times 4) \times 6 = 27{,}85$ M³ beton gestort.

STORTHOOGTE FASE 2: $\frac{4 \times 6}{15{,}9} = 1{,}51$ m < 1,74 m

Afdeling: CTE **Omvat** 12 **bladen, blad nr** 8 8 **Nr** 2910

Bedrijf: NWB



STAATSMIJNEN
U. D. C.
Onderwerp: AFSLUITPROP SCHACHT II, SM. EMMA.
PROJECT 2518.
Tek. nr 4B45 159
Schrijver:
Gezien:
Datum: 12 MRT. '73
Wijz. B: 3-7-'74
HOR. BELASTING TEGEN GANGAFDICHTING.
48/15,9 × 2600 = 7850 kg/m²
5000 kg/m²
1820 kg/m²
4290 kg/m²
5200 kg/m²
ZUID.
NOORD.
1,50
φ 4,50 m.
1,50
Afdeling: CTE
Bedrijf: NkB
Omvat 12 bladen, blad nr 9B
Nr 2910

STAATSMIJNEN **U. D. C.**

Onderwerp: AFSLUITPROP SCHACHT II, ST. EMMA.
PROJECT 2518.

Tek. nr 4345159

Schrijver: **Gezien:** **Datum:** 12 MRT. '73

HOUTEN BEKISTING

We nemen $\sigma_b = 80$ kg/cm²

$M_{veld} = M_{st.p} = \frac{1}{10} \times 5000 \times 0{,}5^2 = 125$ kgm.

$M_{uitkr} = 1/2 \times 5000 \times 0{,}22^2 = 125$ kgm.

HOUTDIKTE : 32 mm

$W_x = 1/6 \times 100 \times 3{,}2^2 = 170$ cm³

$I_x = 1/12 \times 100 \times 3{,}2^3 = 272$ cm⁴

$\sigma_b = \frac{12500}{170} = 74$ kg/cm²

$f = \frac{5}{384} \times \frac{50}{100.000} \times \frac{50^4}{272} = 0{,}15$ cm d.i. $\frac{1}{334}$ l

(door inkl. gunstiger)

$f_{uitkr.} = \frac{50 \times 22^4}{8 \times 100.000 \times 272} = 0{,}054$ cm. d.i. $\frac{1}{407}$ l

Afdeling: CTE **Omvat** 12 **bladen, blad nr** 10 B **Nr** 2910

Bedrijf: NWB

STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, SM. EMMA.
PROJECT 2518 Tek. nr 48 45 159

Schrijver: Gezien: Datum: 12 MRT.'73

Wijz. B: 3-7-'73

VERTICALE STIJLEN h.o.h. max. 50 cm

$l = 0{,}95$ m.

PROFIEL 524 $W_x = 77\ cm^3$ $I_x = 455\ cm^4$

$\bar{\sigma}_t = 700\ kg/cm^2$. (zie pag. 5)

$q = 0{,}5 \times 5000 = 2500\ kg/m'$

$M = 1/8 \times 2500 \times 0{,}95^2 = 283\ kgm.$

$\sigma_b = \frac{28300}{77} = 370\ kg/cm^2\ (< 700)$

$f \leq \frac{1}{600} l \rightarrow$ Ben. $I_x = 37{,}2 \times 2{,}5 \times 0{,}95^3 = 95\ cm^4\ (< 455)$

Oplegkracht max $= 0{,}5\{(0{,}475 \times 2730) + \frac{2}{3}(0{,}475 \times 2470)\} = 1040$ kg.

σ oplegdruk $= \frac{2 \times 1040}{9 \times 12} = 19{,}3\ kg/cm^2$

HOR. BALKEN (Door wijz B minder belasting: 2730 kg/m² wordt nu 1820 kg/m²)

Belasting onderste balk:

$1{,}25\{(0{,}475 \times 2730) + \frac{1}{3}(0{,}475 \times 2470)\}$ = 2120 kg/m'

$(0{,}475 \times 260) + \frac{2}{3}(0{,}475 \times 2470)$ = 910 "

TOTAAL = 3030 kg/m'

Belasting bovenste balk:

$0{,}315 \times 5000$ = 1600 kg/m'

$1{,}25(0{,}30 \times 5000)$ = 1900 "

TOTAAL = 3500 kg/m'

Afdeling: ETE
Bedrijf: NWB

Omvat 12 bladen, blad nr 11 B Nr 2910

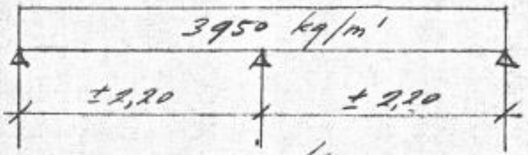
STAATSMIJNEN U.D.C.

Onderwerp: AFSLUITPROP SCHACHT II, ST. EMMA
PROJECT 2518 Tek. nr 4B45159

Schrijver: Gezien: Datum: 12 MRT. '73

Wijz. B 3-7-'74

Belasting middenbalken:
(0,315 + 0,475) × 5000 = 3950 kg/m². MAATGEVEND.



M t.g.v. hor. bel. = 1/8 × 3950 × 2,2² = 2400 kgm
M e.g. = reken = 100 "
2500 "

Max. oplegkracht = 1,1 × 3950 = 4350 kg.

Max. trekkr. (ketting) = 1,25 (2,2 × 3950) = 11000 kg.

KIES HE 140 B → Wx = 216 cm³
Ix = 1509 cm⁴
G = 43 kg/m'

$\sigma = \frac{250000}{216} = 1160$ kg/cm²

$f = \frac{5 \times 395 \times 220^4}{384 \times 2{,}1 \times 10^6 \times 1509}$ = 0,385 cm dit is $\frac{1}{570}$ l

σ oplegdruk $\frac{2 \times 4350}{14 \times 21}$ = 29,5 kg/cm²

(Oplegkracht is in werkelijkheid veel gunstiger; zie pag. 9)

Afdeling: CTE Omvat 12 bladen, blad nr 12 8 Nr 2910
Bedrijf: NWR

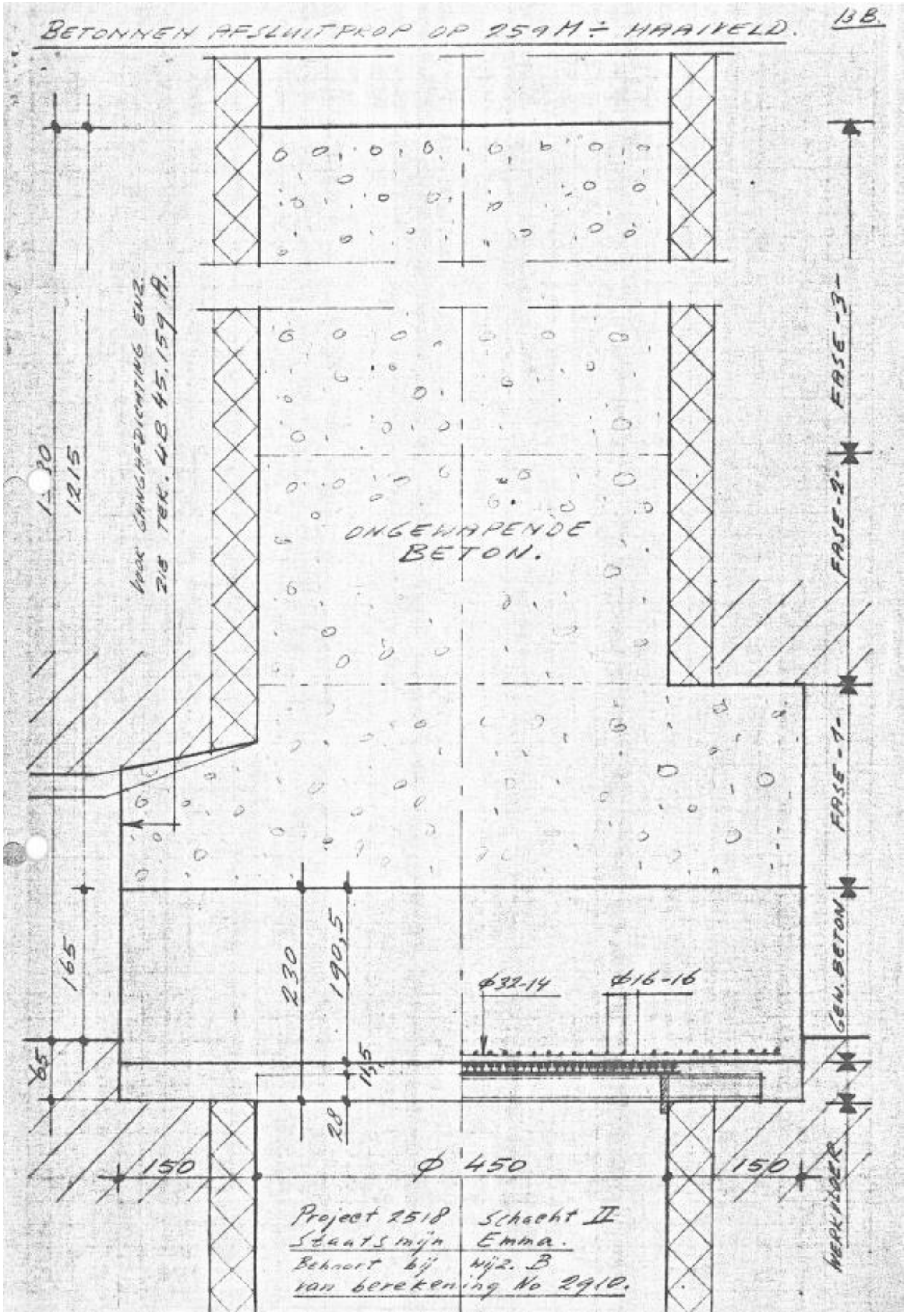


Fig. 93: Static calculation shaft barrier shaft II Emma /52/

The coordinates of shaft II Emma are:

RD-x:	193889
RD-y:	326800
elevation:	+105 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located southwards the roundabout of Emmaweg and Plato-Straat (community Brunssum).

7.5 Shaft III, Emma

The vertical Shaft III of the state mine Emma was drilled in 1937. In 1974 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 6,0 m diameter. The shaft was drilled to a total depth of 980,0 m and was used as drawing shaft and upcast drafting shaft. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 203 m respectively the carbon surface is located on -989 m NAP /6/. The stratigraphic horizons of the overburden are shown in the following figure /68/.

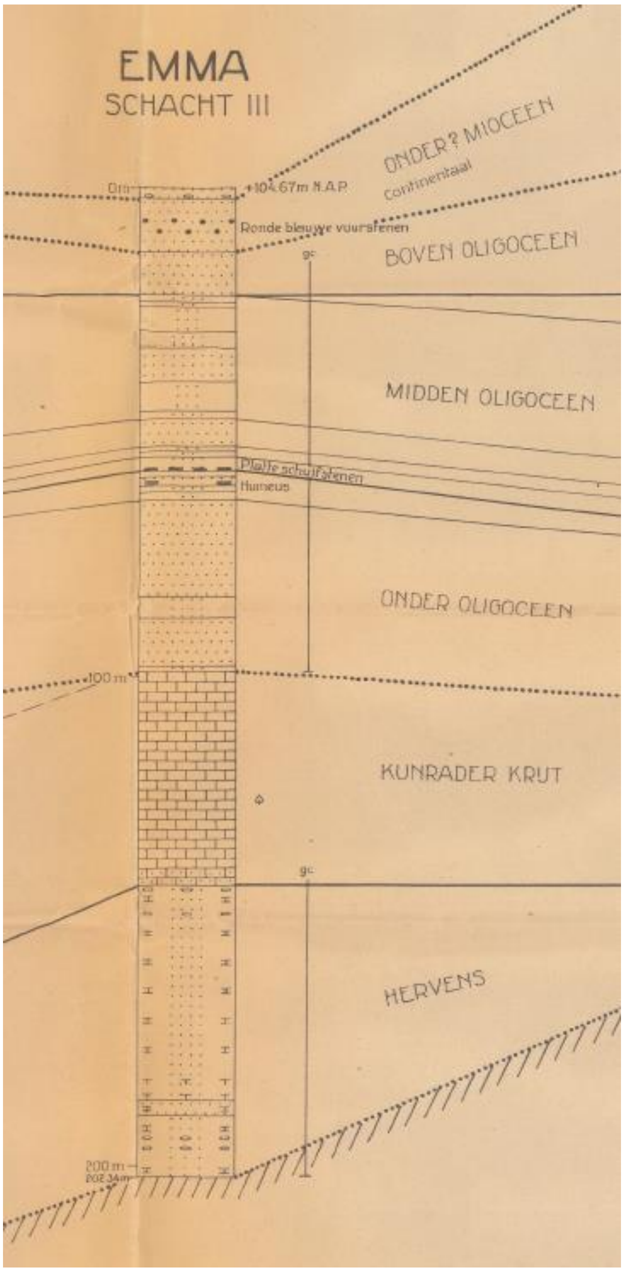


Fig. 94: Stratigraphic horizons of the overburden, shaft III Emma /68/

The shaft III Emma has 8 documented insets. The 259 m floor, as the topmost is located in a level of -153,0 m NAP and in a depth of 258 m /6/ /50/.

In the following figure the strata in the range of the 259 m floor is pictured (here mainly layers of slate and sandstone as well as Laag IV /52/.

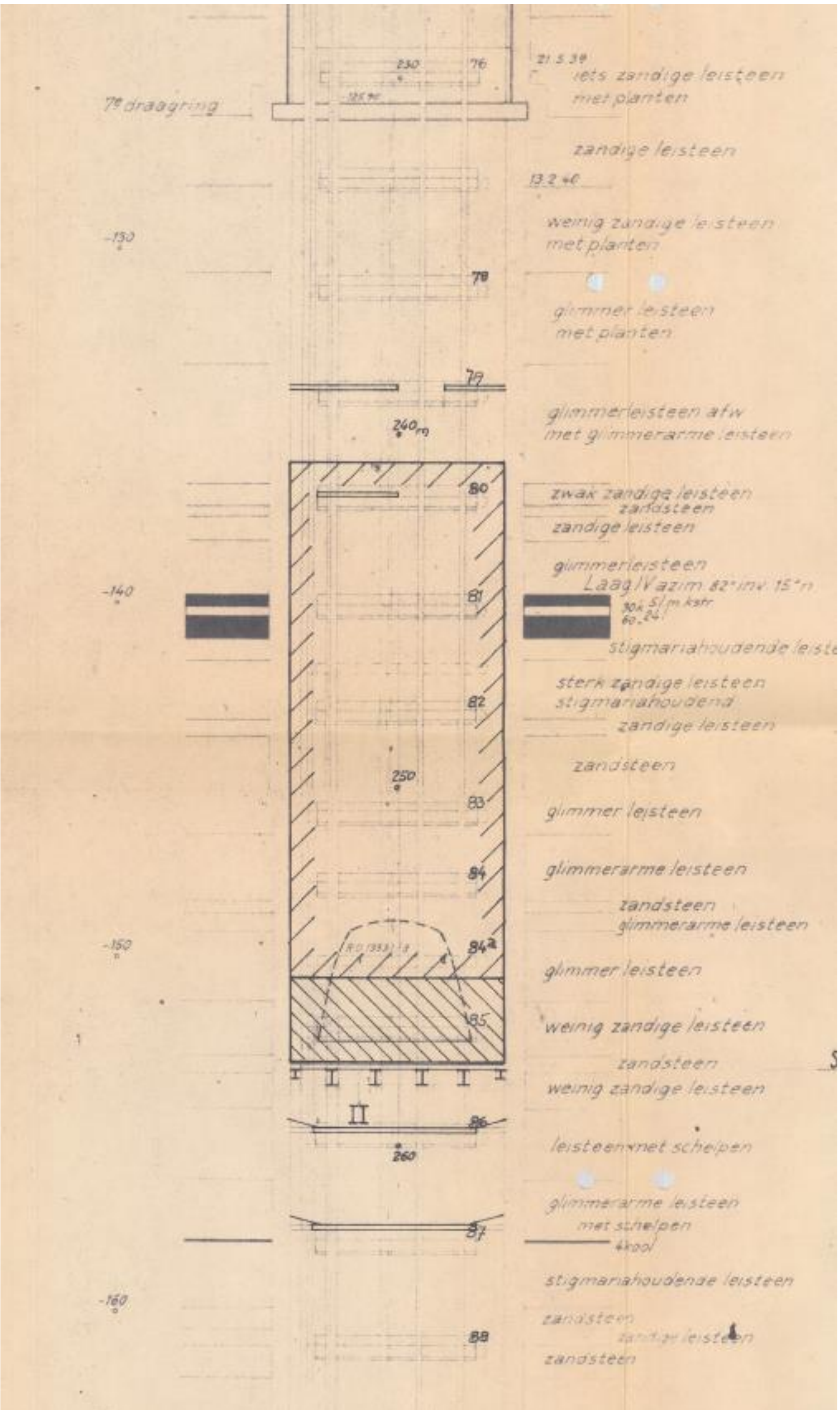


Fig. 95: Strata shaft III Emma, 259 m floor /52/

In 1974 a load bearing filling out of 510 m³ of a mixture of concrete (length 17,85 m) was embedded in the shaft underneath the 259 m floor. Within the filling a steel tube (ø 1.000 mm) was inserted /52/. The reason for inserting the steel tube was to provide an opening to install submersible pumps to potentially lower the mine water level. The upper end of the steel tube was sealed with a layer of 1,42 m of concrete, the lower end was left open. The shaft barrier was used as load bearing filling. For a maximum friction grip of the filling the shaft walls were cleaned and drawn off. Finally the shaft was covered up with a welded steel panel /14/. In 1977 the shaft was backfilled with 7.984 m³ sand. Because of difficulties at the fore shaft, he was backfilled separately with 1.285 m³ sand by hydraulic stowing. The adverse inclination of the attached suction channel, he as well was backfilled separately with 526 m³ sand by hydraulic stowing /17/. In 1981 the shaft was provided with a concrete cover /49/.

The following figures show the shaft barrier of shaft III Emma.

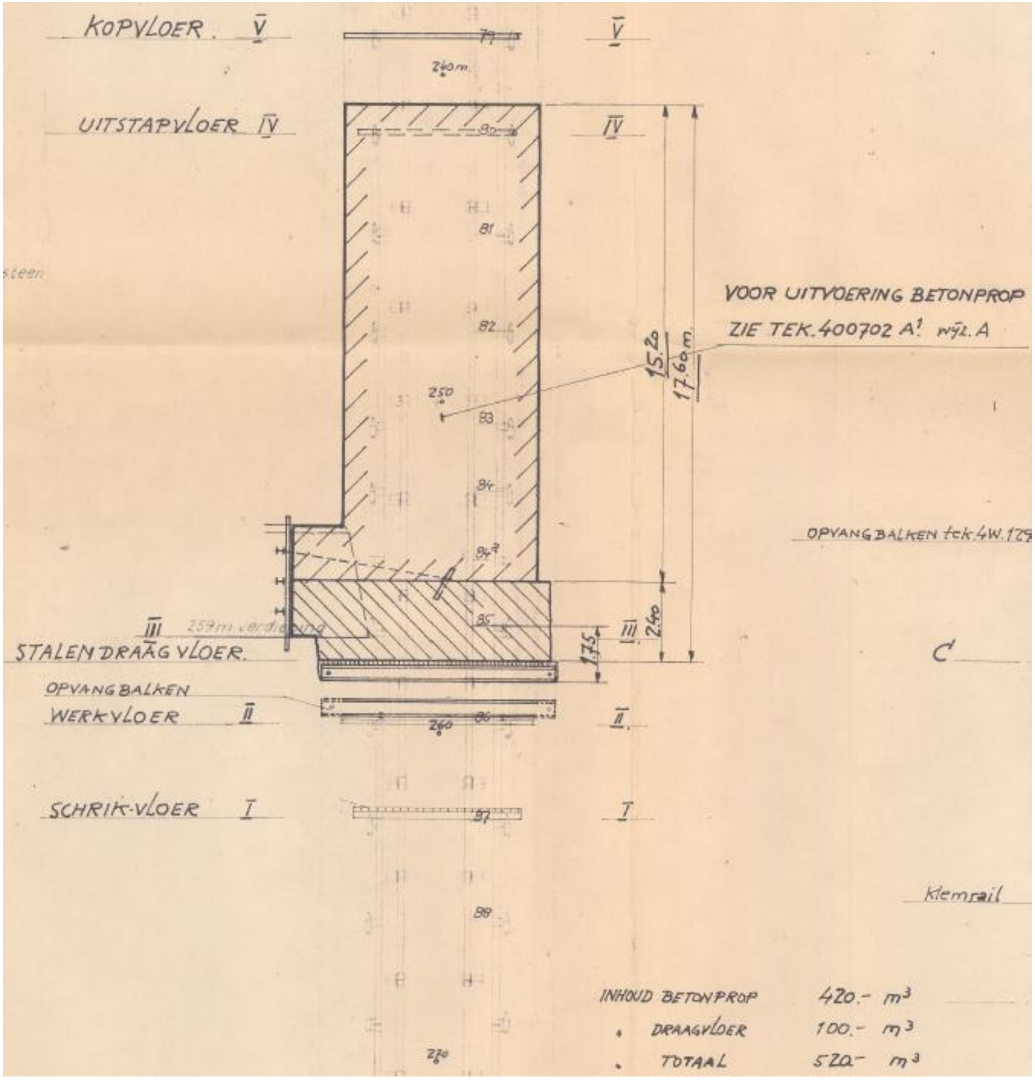


Fig. 96: Shaft barrier shaft III Emma /52/

Static calculations of the shaft barrier of the shaft III Emma are existent /52/. Compare the following figures.

STAATSMIJNEN U. D. C.

Onderwerp: Statische berekening betonnen afsluitprop in schacht II. S.M. Emma, op de 259,- M verdieping proj. 251 P.

Tek. nr 400702 A1, wijz. A.

Schrijver: A. Visschers. Gezien:

Datum: 25 Mei '73.

Betonprop. Beton K.225.

Bepaling volume van het beton:

cylinder $\left(\frac{\pi}{4} \times 6^2\right) \times 17,6 = 28,2 \times 17,6 = 496\ M^3$

ingang schachtgang $2,- \times 3,- \times 4,- = 24\ M^3$

totaal $= 520\ M^3$

Er wordt gerekend met het ongunstigste geval:

a) Onder de prop geen water.

b) Boven de prop een vulling van zand wat met water is verzadigd.

Vanwege de silowerking kan volgens bijgaande bijlage II. (nr-1207 chemb/alg-'68) van het zand gerekend worden met een equivalente vulhoogte van 5 x diameter van de schacht.

Het soortelijk gewicht γ van de kolom zand welke verzadigd is met water volgt uit:

$$\gamma = \gamma s \left(1 - \frac{\gamma v}{\gamma K}\right)$$

Hierin is γk = soortelijk gewicht van de korrels. (c.a. 2,5)

γv = " " van het omringende water (c.a. 1.)

γs = " " van het droge stortmateriaal (c.a. 2).

γ = schijnbare s.g. van de stortmassa verzadigd met water.

Indien $\gamma s = 2$, dan is $\gamma = 1,2$.

Afdeling: C.T.E. Omvat bladen, blad nr 1. Nr 3001.

Bedrijf: Nieuwbouw.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitprop schacht II, S.M. Emma.
Projekt 2510.

Tek. nr 400702 A, wijz. A.

Schrijver: Gezien: Datum:

De totale vertikale kracht volgt door sommering van de volgende componenten.

A.) Gewicht van de cylinder van de betonprop.
B.) De hydrostatische druk.
C.) De door het vulmateriaal op de prop uitgeoefende druk (silowerking)

<u>Totale vertikale kracht.</u>

A.) Betonprop. 49,6 × 23 = 1140 ton.
B.) Waterkolom, 1 × 28,2 × 243 = 6850 ton.
C.) Vulstenen/zand 28,2 × 5 × 6 × 1,2 = 1030 ton.
totaal = 9020 ton.

We berekenen de prop als wrijvingsprop, waarbij de kleef op 3 kg/cm² aangenomen wordt.

Een en ander gezien:
1) De rondte van de schacht, die in orde van grote cm's bedraagt.
2) De krimp van de prop, die in orde van grote mm's zal bedragen.
3) De ruwheid van de schacht- en landingswanden, en van uitgaande dat gladde stukken bewust geruwd zullen worden.
(De schachtwanden zijn van gewapend beton).

Afdeling: C.G.E. Omvat bladen, blad nr 1. Nr 3001
Bedrijf: Nieuwbouw

STAATSMIJNEN U.D.C.

Onderwerp: afsluitprop schacht II., S.M. Emma
project 1951

Tek. nr 400702 A1. wijz A.

Schrijver: Gezien: Datum:

Gedeelte van de schachtwand onderbroken
door de laadgang a/d zuidkant ± 4 × 3 = 12 M²
idem noordkant = 12 M²
totaal = 24 M²

Dit komt overeen met rondom een schachtwandhoogte van.

$\frac{24}{\pi \times 6} = \frac{24}{18,8} = 1,28$ M.

Berekening prophoogte.

Benodigde prophoogte $\frac{9020}{30 \times 18,8} + 1,28 = \frac{9020}{565} + 1,28 = 16 + 1,28 = 17,28$ M.

(uitvoering 17,60 M.)

Stalen vloer op 250 M. verdieping.

Deze vloer dient als bekistingsvloer voor de 2,40 M. dikke betonplaat.

e.g. natte betonplaat 2,4 × 2600 = 6250 Kg/M²
e.g. stalen vloer etc. = 600 Kg/M²
totaal = 6850 Kg/M²

Afdeling: C.T.E. Omvat bladen, blad nr 3. Nr 3001.
Bedrijf: Nieuwbouw.

STAATSMIJNEN U.D.C.

Onderwerp: Afsluitplaat schacht III S.M. Emma.
Project 25118.
Tek. nr 400702 A1

Schrijver: Gezien: Datum:

Moerbalken h.o.h. $\frac{4.00 - 2 \times 0,25}{3} = \pm 1,20$ M. $l = 6,60$ M.

$M_{max} = 1/8 \times 6850 \times 6,6^2 \times 1,2 = 857 \times 43,5 \times 1,2 = 44700$ Kgm.

W_x ben $= \frac{4470000}{1400} = 3200$ cm^3. H.E. 500 A $W_x = 3550$ cm^3
$I_x = 86975$ cm^3
$G = 155$ Kg/M'

$\sigma_b = \frac{4470000}{3550} = 1260$ Kg/cm^2.

$f = \frac{5}{384} \times \frac{1,2 \times 68,5 \times 660^4}{2,1 \times 10^6 \times 86975} = 1,1$ cm. $\frac{660}{1,1} = 600 = \frac{1}{600}\ l.$

σ opleg druk. $\frac{(3 \times 1,2 \times 6850) \times 2}{60 \times 30} = \frac{49300}{1800} = 27,4$ Kg/cm^2.

Randbalken. $l = 3,45$ M.

breedte $\frac{\frac{6,00 - 4,00}{2} - 0,25}{2} + 0,25 = 0,63.$

$q = 0,63 \times 6850 = 4320$ Kg/M'. $M = 1/8 \times 4320 \times 3,45^2 = 6480$ Kgm.

Wben $= \frac{648000}{1400} = 463$ cm^3 H.E. 220 A. $W_x = 515$ cm^3
$I_x = 5410$ cm^4
$G = 50,5$ Kg/M.

$f = \frac{5 \times 43,2 \times 350^4}{384 \times 2,1 \times 10^6 \times 5410} = 0,75$ cm. $\frac{345}{0,75} = \frac{1}{460}\ l.$

σ opleg druk. $= \frac{1,72 \times 4320 \times 2}{50 \times 22} = 13,5$ Kg/cm^2.

Afdeling: C.T.E. Omvat bladen, blad nr 4. Nr 3001.
Bedrijf: Nieuwbouw.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitplaat schacht III S.M. Emma.
Projekt 12510. Tek. nr 400702 A1.

Schrijver: Gezien: Datum:

Dek gevormd door railprofielen S24 h.o.h. 9 cm.

$l = \frac{4.00 - 2 \times 0{,}25}{3} = \pm 1{,}20$ M.

profiel S24. → Reken i.v.m. slijtage:

$W_x = 0{,}8 \times 97{,}3 = 77$ cm³

$I_x = 0{,}8 \times 569 = 455$ cm⁴.

$\sigma_t = 700$ kg/cm².

We berekenen het dek nu verder per M' breedte.

$q = 1 \times 6850 = 6850$ kg/M'

$M_{max} = 1/8 \times 6850 \times 1{,}2^2 = 1250$ kgm.

W_x vereist $= \frac{125000}{700} = 178$ cm³. W_x aanw $= 11 \times 77 = 850$ cm³

Ber. I_x voor $f < \frac{1}{600} l = 37.2 \times 1{,}2 \times 6850 \times 1{,}2^2 = 440$ cm⁴.

I_x aanwezig. $11 \times 455 = 5050$ cm⁴.

M uitkraging $= 1/2 \times 6850 \times 0{,}40^2 = 550$ kgm.

I_x ber voor $f < \frac{1}{900} l = 1{,}5(357 \times 6{,}8 \times 0{,}4^3) = 234$ cm⁴.

I_x aanwezig. $= 5050$ cm⁴

Afdeling: C.G.E. | Omvat bladen, blad nr 5. | Nr 3001.

Bedrijf: Nieuwbouw.

STAATSMIJNEN U.D.C.

Onderwerp: afsluitprop schacht III S.M. Emma. projekt 2518.

Tek. nr 400702 A1 wijz A.

Schrijver: Gezien: Datum:

<u>Gewapend betonplaat.</u> hoogte 2,40 M. lengte 6,50 M.

Beton K 225. Staal Q.R. 40.

De betonplaat moet het gewicht van de nog te storten betonprop dragen.

te storten beton 15,2 × 2,6 = 39,50 ton/M².
e.g. betonplaat 2,4 × 2,4 = 5,75 "
totaal = 45,25 × 28,2 = 1275 ton.

Oppervlak van de dragende middenstrook 4,- M (breed) × 5,5 (lang) = 22,- M²

Belasting op de middenstrook per M² = $\frac{1275}{22}$ = 58 ton/M².

M veld = 1/8 × 58000 × 6,5² = 5,25 × 58000 = 304000 kgm.

234 = $k_0 \sqrt{304000}$ k_0 = 0,425 A = 0,259 × 234 = 60,7 cm²

hoofdwap. ⌀32-13 = 61 cm². verdeelwap. $\frac{1}{5}$ × 60,7 = 12,14 cm²
⌀16-16 = 12,5 cm²

$\tau_{max} = \frac{1275000}{2 \times 450 \times 0,85 \times 234} = 7,1$ kg/cm²

op te nemen schuifkracht door de staalprofielen 8 × 49 × 1,2 × 0,58 × 1400 = 383000 kg.

Max oplegkracht van betonprop = ~~1275000 kg~~ = 1275000 kg
e.g. bekisting = 28,2 × 600 kg = 16900 kg
totaal = 1291900 kg

oplegdruk = $\frac{1291900}{(30+60) \times 400}$ = 36 kg/cm² of $\frac{0,5 \times 1291900}{35 \times 400}$ = 46,5 kg/cm² rechts.

Afdeling: C.T.E. Omvat bladen, blad nr 6. Nr 3001.
Bedrijf: Mijnbouw

STAATSMIJNEN IN LIMBURG — U.D.C.

Onderwerp: Statische berekening betonnen afsluitplaat in schacht III S.M. Emma, op de 259,- m Ondieping proj. 2510. 2641.

Tek. nr 400702 A1 wijz 3.

Schrijver:

Gezien:

Datum: 8 Mei 1974

Wijziging op blz. 6.

t.g.v. het plaatsen van de buis ø 100 cm de belasting.

Aangenomen last $1,5 \times 58 = 87$ ton/m^2.

$M_{uit} = 1/8 \times 87 \times 6,5^2 = 455000$ kgm.

$234 = k_o \sqrt{455000}$ $k_o = 0,347$. $f_y = 0,4 \times 234 = 93$ cm^2 wap ø32-8.5

Afdeling: C.T.E.

Bedrijf: Nieuwbouw

Omvat bladen, blad nr 6A.

Nr 3001.

STAATSMIJNEN U.D.C.

Onderwerp: afdichtingsproject schacht III, S.M. Emma. projekt 25/10.

Tek. nr 400702 A1.

Schrijver: Gezien: Datum:

Berekening gangafdichting.

Gerekend wordt met een stek per kwartier van 3. M^3 beton.
Na 4 uur begint de beton op te stijven. gestort $4 \times 4 \times 3 = 48\ M^3$

Snelste stijghoogte bij het vullen van de ronde schachtdoorsnede.

Stijghoogte in 4 uur $\frac{48.-}{28.2} = 1,70\ M.$

Max druk. $1,7 \times 2600 = 4430\ Kg/M^2$

Hout $l\ max = 0,5\ M.$

$M = 1/10 \times 4430 \times 0,5^2 = 111\ Kgm$ $W_{ben} = \frac{11100}{70} = 158\ cm^3$

W vuren $1/6 \times 100 \times 3,2^2 = 170\ cm^3$

Rails $l\ max\ 1.\ M.$

max belasting $0,5 \times 4430 = 2215\ Kg/M'$
$M = 1/8 \times 2215 \times 1^2 = 277\ Kgm.$
$W_{ben} = \frac{27700}{700} = 40\ cm^3$ W_{rails} per stuk $= 77\ cm^3$

Horizontale liggers. $l\ max = 1,75\ M.$

$M = 1/8 \times 4430 \times 1,75^2 = 1670\ kgm.$ $W_{ben} = \frac{167000}{1400} = 120\ cm^3$

H.E. 140 A. $W_x = 155\ cm^3$

Afdeling: C.T.E.
Bedrijf: Nieuwbouw

Omvat bladen, blad nr 7.

Nr 3001.

STAATSMIJNEN U.D.C.

Onderwerp: Projekt afsluitplaat 1951 schacht II.

Schrijver: Gezien: Tek. nr 400702 A1. Datum:

Max trek- of drukkracht. 1,5 × 4430 = 6650 Kg.

Knikkracht van H.E. 160 A. 13 × 0,6 × 1400 × 0,6 = 6700 Kg.

De stijghoogte is hier kleiner vanwege de grotere doorsnede d.p.

Afdeling: C.T.E. Omvat bladen, blad nr 8. Nr 3001.

Bedrijf: Hulpbouw.

STAATSMIJNEN IN LIMBURG U.D.C.

Onderwerp: Statische berekening betonnen afsluitplaat in schacht III S.M. Emma, op 259,- M verdieping proj 2518

Tek. nr 401534 A2

Schrijver: A. Vischers Gezien: Datum: 21 Mei 1974.

Uitkepen van de hoofdliggers H.E. 500 met 9 cm

Verzwakking van de flens. $2{,}3 \times 9 + 2{,}3 \times 2{,}5 = 20{,}7 + 5{,}8 = 26{,}5$ cm²

Netto breedte van de koppelplaat in het midden.

$30\text{ cm} - (9 + 2 \times 1 + 2{,}5) = 30 - 13{,}5 = 16{,}5$ cm. Opp: $16{,}5 \times 2 = 33$ cm²

Bouten M24 kwaliteit 8 G. Schuifkracht per bout 6430 kg

Over te brengen kracht $26{,}5 \times 1400 = 37000$ kg

aantal bouten $\frac{37000}{6430} = 6$ stuks.

Voor nauwkeurig werken 8 stuks genomen.

stuikdruk $= \frac{6430}{2{,}4 \times 2} = \frac{6430}{4{,}8} = 1340$ kg/cm²

Afdeling:
Bedrijf:

Omvat bladen, blad nr 8A Nr 3001.

STAATSMIJNEN IN LIMBURG U.D.C.

Onderwerp: Statische berekening betonnen afsluitkap in schacht III
S.M. Emma, diepte 259.- M verdieping proj. 2641. Tek. nr 400702 A wijz. I.
Schrijver: A. Witzelers. Gezien: Datum: 30 Mei '74.

Uitkeping van 85 mm in de onder- en bovenflens.

Contrôle van de spanning en doorbuiging.

H.E. 500 A. hoogte 490 mm. I_x = 86975 cm⁴
breedte 300 mm. W_x = 3550 cm³

Verzwakking van de onder- en bovenflens = $2 \times a^2 F = 2 \times 23{,}4^2 \times 8{,}5 \times 2{,}3 =$
$550 \times 39 = 21500$ cm⁴

I_x verzwakt 86975 cm⁴
af 21500 cm⁴
rest = 65475 cm⁴

W_x verzwakt $\frac{65475}{24{,}5} = 2670$ cm³

balk lengte = dagmaat + opleglengte = 5,85 + 0,60 = 6,45 M.

$M = \frac{1}{8} \times 6850 \times 1{,}2 \times 6{,}45^2 = 6{,}25 \times 6850 = 42700$ Kgm. $q = 1{,}2 \times 6850 = 8220$ Kg/M'

$\sigma = \frac{4270000}{2670} = 1600$ Kg/cm² veiligheidsfactor $= \frac{2400}{1600} = 1{,}5$.

De buitenste balken buigen minder door (kleinere lengte)
De rails brengen een gedeelte v/d belasting v/d middenbalken op de buitenbalken over.

Doorbuiging. $f = \frac{5}{384} \times \frac{q\,l^4}{E\,I} = \frac{5}{384} \times \frac{82{,}2 \times 645^2 \times 645^2}{2.100.000 \times 65475} = \frac{411 \times 415000 \times 415000}{384 \times 2.100.000 \times 65475}$

$= \frac{411 \times 172000}{806 \times 65475} = \frac{1}{2} \times 2{,}63 = 1{,}32$ cm

Doorbuiging. $\frac{645}{1{,}32} = \frac{1}{490}\,l$.

Afdeling:
Bedrijf:
Omvat bladen, blad nr 8B
Nr 3001.

STAATSMIJNEN IN LIMBURG U.D.C.

Onderwerp: STATISCHE BEREKENINGEN BETONNEN AFSLUITPROP IN SCHACHT III
S.M. EMMA, OP DE 259 M VERDIEPING. PROJ. 2518; 2641 Tek. nr 400702-A1 wijz. 8 401508-A2
Schrijver: Schlösser J.J. Gezien: Datum: 8 MEI 1974.

AANVULLING T.G.V. STALEN DOORVOERPIJP.

Berekening betonhoogte boven st. pijp.

Opp. hor. schachtdrsn. = $\frac{\pi}{4} \times 6^2$ = 28,25 m^2.

Tot. vert. kracht = 9020 ton, dit is $\frac{9020}{28,25}$ = 320 ton/m^2.

Schuifspanning = $\frac{320 \times 0,87}{1,35 \times \pi \times 1}$ = 67 t/m^2 = 6,7 kg/cm^2.

De benodigde betonhoogte is dus 1,35 m.

Houten deksel

dikte geschaafd = 71 mm. dikte ongeschaafd = 75 mm.

Belasting = 1,4 × 3000 = 4200 kg/m^2.

M = 1/8 × 4200 × $1,01^2$ = 540 kgm.

W = 1/6 × 100 × $7,1^2$ = 840 kgm.

I = 1/12 × 100 × $7,1^3$ = 2980 cm^4

$\sigma_b = \frac{540.00}{840}$ = 65 kg/cm^2. < 70 kg/cm^2.

Ben. I = 0,651 × 4200 × $1,01^3$ = 2820 cm^4. < 2980 cm^4

$\sigma_{opl.} = \frac{0,87 \times 4200}{316}$ = 12 kg/cm^2. < 20 kg/cm^2.

Stalen pijp ϕ 1016/996 mm.

Storthoogte bij 48 m^3 = $\frac{48}{\frac{\pi}{4} \times 6^2} = \frac{48}{28,25}$ = 1,70 m.

Max. druk (hor.) = 1,70 × 3000 = 5100 kg/m^2.

Reken voor eenzijdige uitwendige druk →

M = 1/36 × 5100 × $0,51^2$ × 3,37 = 134 kgm.

$\sigma = \frac{13400}{1/6 \times 100 \times 1^2} = \frac{13400}{16,66}$ = 810 kg/cm^2.

Afdeling: CTE Omvat bladen, blad nr 9 Nr 3001.
Bedrijf: NWB

Fig. 97: Static calculation shaft barrier, shaft III Emma /52/

The coordinates of the shaft III Emma are:

RD-x:	193704
RD-y:	326791
elevation:	+105 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on sidewalk northwards the roundabout of Emmaweg and Akerstraat Noord (community Brunssum).

7.6 Shaft IV, Emma

The vertical Shaft IV of the state mine Emma was drilled in 1947. In 1971 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,5 m diameter. The shaft was drilled to a total depth of 653,0 m and was used as travelling shaft and downcast drafting shaft. Within the overburden the shaft consists of reinforced concrete. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 215 m respectively the carbon surface is located on -148,9 m NAP /6/. The shaft IV Emma has 6 documented insets. The 325 m floor, as the topmost is located in a level of -200,2 m NAP and in a depth of 266 m /6//50/.

In 1971 a shaft barrier (length 18 m) out of 1.053 m³ of a mixture of concrete and a quality of compactness of 240 H.A. (240 kg blast furnace cement, class A per m³ mixture with 60 kg ADI-filler) was inserted at the insets on the 325 m floor. In the first on the floor level a platform consisting of iron beams was constructed. In the second step this platform was covered with a heavy reinforced concrete board which rests with its bend lower edge upon the surrounding rock. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. One existing bunker for waste material had to be eradicated. The remaining parts of the bunker were backfilled with concrete as well. Above the barrier the shaft column was backfilled with approximately 5136 m³ debris and covered up with a shaft cover made of concrete with an opening for refilling. By the end of 1971 the shaft column subsided 5,16 m /11/. Therefore the shaft was backfilled with additionally 40 m³ waste material. 1972 the fill material subsided an extra 0,49 m. Finally in 1975 the opening for refilling was closed with a mixture of concrete /15/.

The figure below shows the shaft barrier of the shaft IV Emma.

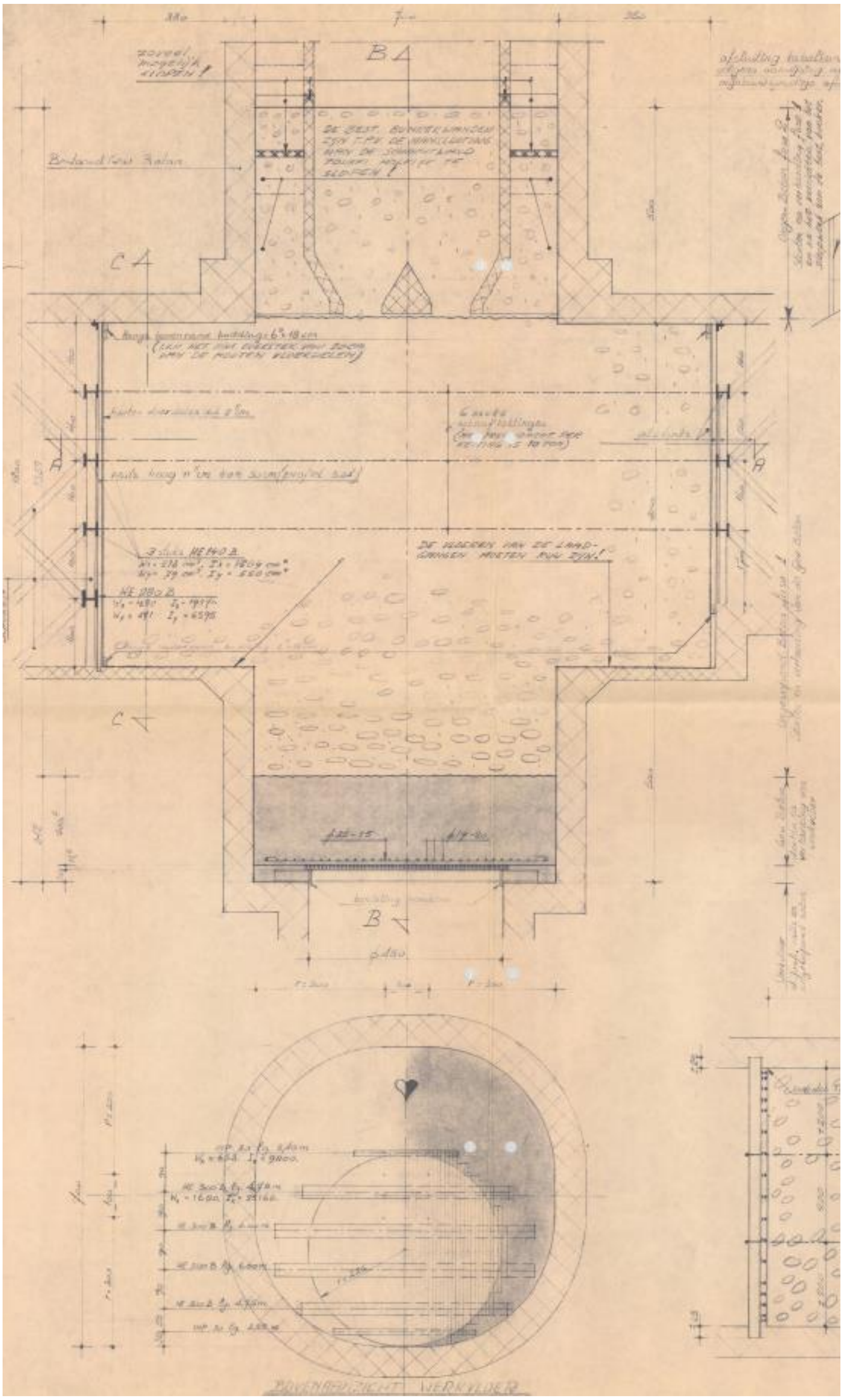


Fig. 98: Shaft barrier shaft IV Emma /52/

Static calculations of the shaft barrier of the shaft IV Emma are existent /52/. Compare the following figures.

NIEUWBOUW – DSM. GELEEN, JAN. 1974.

BEREKENING NR. 2932.
PROJECT NR. 2518.

BYBEH. TEK. NR. 4845471

STATISCHE BEREKENING BETONNEN AFSLUITPROP IN SCHACHT IV, STAATSMIJN E^a, OP DE 325 M. VERDIEPING.

I BETONPROP. BETON K 225.

ER WORDT GEREKEND MET HET ONGUNSTIGSTE GEVAL:
A) ONDER DE PROP GEEN WATER.
B) BOVEN DE PROP EEN VULLING VAN WASSTENEN DIE MET WATER VERZADIGD IS.

VANWEGE DE „SILOWERKING" KAN VOLGENS BYGAANDE BYLAGE II (NR. 1207 CHEMB/ALG – '68) VOOR DE WASSTENEN GEREKEND WORDEN MET EEN EQUIVALENTE VULHOOGTE VAN 3× DE DIAMETER VAN DE SCHACHT.

HET SOORTELYK GEWICHT γ VAN DE KOLOM WASSTENEN WELKE VERZADIGD IS MET WATER VOLGT UIT:

$$\gamma = \gamma_s \left(1 - \frac{\gamma_v}{\gamma_k}\right)$$

HIERIN IS γ_k = SOORTELYK GEWICHT VAN DE KORRELS (CA. 2,5)
γ_v = " " VAN DE OMRINGENDE VLOEIST. (CA. 1)
γ_s = " " VAN HET DROGE STORTMAT. (CA. 2)
γ = SCHYNBARE SOORTELYK GEWICHT VAN DE STORTMASSA VERZADIGD MET WATER.

DUS $\gamma = 2\left(1 - \frac{1}{2,5}\right) = \underline{1,2}$

BEPALING OPPERVLAKKEN:

DRSN. SCHACHT : $\frac{\pi}{4} \times 6^2 + 13 \times 1 = \underline{41,26\ M^2}$

DRSN. LAADGANG : $\frac{118}{360} \times \frac{\pi}{4} \times 4,80^2 = 5,94\ M^2$

$2 \times \left(\frac{58,5}{360} \times \frac{\pi}{4} \times 14^2 - 2 \times 3,95\right) = 30,-$ "

$\frac{60}{360} \times \frac{\pi}{4} \times 12^2 - 4,40 \times 2,5 = \underline{4,73}$ "

$\underline{40,67\ M^2}$

DRSN. SCHACHT Ø 4,5 M : $\frac{\pi}{4} \times 4,50^2 = \underline{15,90\ M^2}$

BEPALING VOLUME BETON:

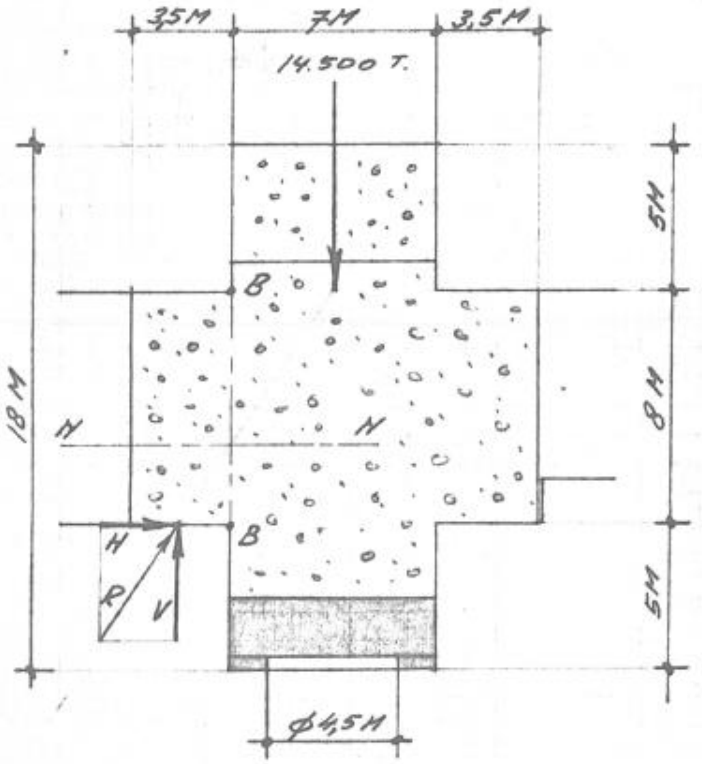
VERT. GEDEELTE IN SCHACHT: 41,26 x 18 =	743	M³
BLOK IN LAADGANG NOORD: =	155	"
BLOK IN LAADGANG ZUID: =	155	"
TOTAAL VOLUME	1053	M³

DE TOTALE VERTICALE KRACHT VOLGT DOOR SOMMERING VAN DE VOLGENDE COMPONENTEN:

A/ GEWICHT VAN DE BETONPROP
B/ DE HYDROSTATISCHE DRUK.
C/ DE DOOR HET VULMATERIAAL OP DE PROP UITGEOEFENDE DRUK. (SILOWERKING)

A/ BETONPROP: 1053 x 2,4 (ca)	= 2530	TON
B/ WATERKOLOM: 41,26 x 261 x 1	= 10780	"
C/ VULSTENEN: 41,26 x 3 x 7,42 x 1,2	= 1100	"
P TOTAAL	= 14410	TON.

REKEN 14.500 TON



$V = \frac{1}{2} \times 14500 = 7250$ TON.
$R = \frac{9{,}56}{8} \times 7250 = 8660$ TON.
$H = \frac{5{,}25}{8} \times 7250 = 4760$ TON
$\sigma_{opl.} = \frac{7250}{3{,}5 \times 5} = 415\ T/M^2 = 41{,}5\ kg/cm^2$

OPL. KAN MAX. 0,70 x 7250 = 5080 TON HOR. OPNEMEN.

BGS Q.K.

HOOFD- TREK- EN DRUKSPANNING.

IN VLAK B-B WERKEN T.G.V. $R = 8660$ TON
A) EEN DWARSKRACHT $V = 7250$ TON
B) EEN HOR. COMPONENT $H = 4760$ TON
EXCENTRISCH OP DRSN. B-B

$\sigma = \frac{4760}{4{,}75 \times 5{,}70} = 176\ \text{ton/m}^2 = 17{,}6\ \text{kg/cm}^2$

$\tau_V = \frac{7250}{5 \times 0{,}973 \times 8} = 186\ \text{ton/m}^2 = 18{,}6\ \text{kg/cm}^2$

$\tau_H = \frac{4760}{7 \times 5} = 136\ \text{ton/m}^2 = 13{,}6\ \text{kg/cm}^2$ (DRSN N-N)

HOOFDSP. $\sigma = \frac{17{,}6}{2} \pm \sqrt{\left(\frac{13{,}6}{2}\right)^2 + 18{,}6^2}$

$\sigma = 8{,}8 \pm 20{,}6 = {+29{,}4 \atop -11{,}8}\ \text{kg/cm}^2$

HOOFDDRUKSPANNING = $29{,}4\ \text{kg/cm}^2$

HOOFDTREKSPANNING = $11{,}8\ \text{kg/cm}^2$

PONSSPANNINGEN. IN DRSN. B-B.

$\sigma_{pons} = \frac{7250 \times 1{,}5}{40{,}67} = 267\ \text{T/m}^2 = 26{,}7\ \text{kg/cm}^2$

STELLEN WIJ: KUBUSDRUKVASTH. $K_d = 225\ \text{kg/cm}^2$
KUBUS TREKVASTH $K_t = 25\ \text{kg/cm}^2$

DE PONSVASTHEID $\rho_k = \sqrt{(225-17{,}6)(25+17{,}6)} = 94\ \text{kg/cm}^2$

DE VEILIGHEIDSFACTOR IS DAN $\frac{94}{26{,}7} = 3{,}53$

II STALEN VLOER OP 325 M. VERDIEPING.

DEZE VLOER DIENT ALS BEKISTINGSVLOER VOOR DE 2M. DIKKE BETONPLAAT.

EIG. GEWICHT BETONPL. (NAT) 2×2600 = 5200 kg/m²
" " ST. VLOER REKEN = 600 "
q = 5800 "

MOERBALKEN. l = 5,50 M. H.O.H. 0,9 M.

$q = 0{,}9 \times 5800 = 5220$ kg/m'

$M_{max} = 1/8 \times 5220 \times 5{,}50^2 = 19800$ kgm.

$W_{vereist} = \frac{19.800.00}{1400} = 1420$ cm³

KIES HE 300B → $W_x = 1680$ cm³, $I_x = 25166$ cm⁴

$f = \frac{5 \times 52{,}2 \times 550^4}{384 \times 2{,}1 \times 10^6 \times 25166} = 1{,}20$ cm dit is 1/460 l.

$G_{opl.} = \frac{2 \times 2{,}75 \times 5220}{42 \times 28} = 25$ kg/cm²

RANDBALKEN l = 3,40 M.

$q_{max.} = (0{,}245 + 0{,}41) \times 5800 + \frac{490}{0{,}49} = 4800$ kg/m'

$M_{max.} = 1/8 \times 4800 \times 3{,}40^2 = 7000$ kgm.

$W_{vereist} = \frac{7000.00}{1400} = 500$ cm³

KIES INP 30 → $W_x = 653$ cm³, $I_x = 9800$ cm⁴

$f = \frac{5 \times 48 \times 340^4}{384 \times 2{,}1 \times 10^6 \times 9800} = 0{,}41$ cm dit is 1/830 l.

$G_{opl.} = \frac{2 \times 1{,}7 \times 4800}{35 \times 12{,}5} = 37{,}5$ kg/cm²

RAILS (PROF. S24) VORMEN DEK. l = 0,9 M.

$\overline{G}_t = 700$ kg/cm² $W = 0{,}8 \times 97{,}3 = 77$ cm³ (I.V.M. SLYTAGE)

$M_{max.} = 1/8 \times 5800 \times 0{,}9^2 = 590$ kgm/m'

$W_{vereist} = \frac{590.00}{700} = 85$ cm³/m'

$W_{aanw.} = \frac{100}{9} \times 77 = 850$ cm³/m'

$M_{uitkr. max.} = 1/2 \times 5800 \times 0{,}41^2 = 490$ kgm/m' (< 590 kgm/m')

III WANDBEKISTING.

GEREKEND WORDT MET ÉÉN TREK PER KWARTIER;
PER TREK WORDT GESTORT 3 M³.
NA 4 UUR BEGINT DE BETON OP TE STYVEN,
ZODAT DE ZYWAARTSE DRUK 4 UUR LANG
TOENEEMT EN DAARNA CONSTANT BLYFT.

IN 4 UUR WORDT GESTORT $4 \times 3 \times 4 = 48$ M³ BETON.

DE DAARBY BEHORENDE STORTHOOGTE IS
A) T.P.V. DE SCHACHT $48/41{,}26 = 1{,}16$ M.
B) T.P.V. DE LAADGANGEN $48/(41{,}26+35) = 0{,}63$ M.
REKEN 0,80 M.

HOUTEN BEKISTING TEGEN RAILS.

$q = 1{,}16 \times 2600 = 3020$ kg/m²

$M_{max} = 1/12 \times 3020 \times 0{,}50^2 = 63$ kgm/m'

$W_{vereist} = \frac{6300}{70} = 90$ cm³ $W_{aanw.} = 1/6 \times 100 \times 2{,}4^2 = 96$ cm³

WE NEMEN HOUTEN WOERDELEN DIK 2,5 cm.

RAILS S24 H.O.H. 0,50 M $l = 1{,}60$ M.

$q = 0{,}80 \times 2600 = 2080$ kg/m²

PER M' $q = 0{,}50 \times 2080 = 1040$ kg/m'.

$M_{max.} = 1/8 \times 1040 \times 1{,}60^2 = 340$ kgm.

$W_{vereist} = \frac{34000}{700} = 49$ cm³. $W_{aanw.} = 0{,}8 \times 97{,}3 = 77$ cm³.

$f = \frac{5 \times 10{,}4 \times 160^4}{384 \times 2{,}1 \times 10^6 \times 0{,}8 \times 569} = 0{,}094$ cm dit is $1/1700\ l$.

$\sigma_{opl.} = \frac{2 \times 0{,}8 \times 1040}{9 \times 10} = 19$ kg/cm².

MOERBALK. (LINKSONDER 1x) $l = 6{,}50$ M.

$q_x = 1{,}6 \times 2080 = 3330$ kg/m'
$M_x = 1/8 \times 3330 \times 6{,}5^2 = 17.600$ kgm.

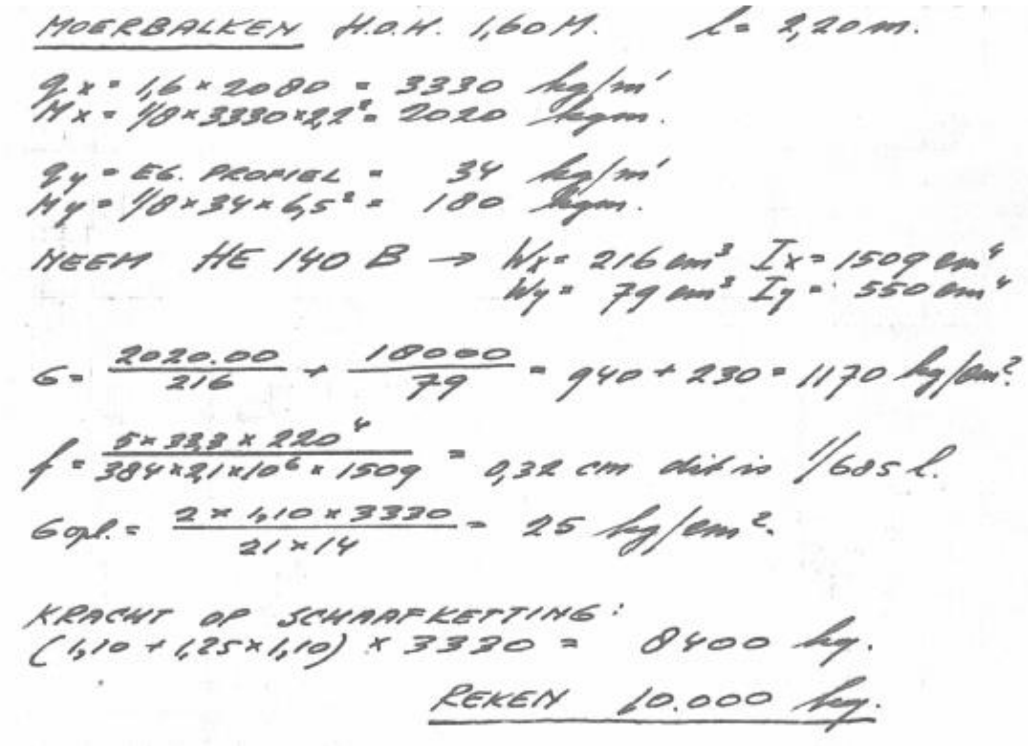
q_y = E.G. PROFIEL = 103 kg/m'
$M_y = 1/8 \times 103 \times 6{,}5^2 = 545$ kgm.

NEEM HE 280 B → $W_x = 1380$ cm³. $W_y = 471$ cm³
$I_x = 19270$ cm⁴. $I_y = 6595$ cm⁴

$\sigma = \frac{1760000}{1380} + \frac{54500}{471} = 1280 + 120 = 1400$ kg/cm².

$f = \frac{5 \times 33{,}3 \times 650^4}{384 \times 2{,}1 \times 10^6 \times 19.270} = 1{,}92$ cm dit is $1/340\ l$.

$\sigma_{opl.} = \frac{2 \times 3{,}25 \times 3330}{25 \times 28} = 31$ kg/cm².


MOERBALKEN H.O.H. 1,60 M. l = 2,20 m.

q_x = 1,6 × 2080 = 3330 kg/m'

M_x = 1/8 × 3330 × 2,2² = 2020 kgm.

q_y = EG. PROFIEL = 34 kg/m'

M_y = 1/8 × 34 × 6,5² = 180 kgm.

NEEM HE 140 B → W_x = 216 cm³ I_x = 1509 cm⁴

W_y = 79 cm³ I_y = 550 cm⁴

$\sigma = \frac{202000}{216} + \frac{18000}{79} = 940 + 230 = 1170$ kg/cm²

$f = \frac{5 \times 33,3 \times 220^4}{384 \times 2,1 \times 10^6 \times 1509} = 0,32$ cm dit is 1/685 l.

$\sigma_{opl} = \frac{2 \times 1,10 \times 3330}{21 \times 14} = 25$ kg/cm².

KRACHT OP SCHAAFKETTING:

(1,10 + 1,25 × 1,10) × 3330 = 8400 kg.

REKEN 10.000 kg.

Fig. 99: Static calculation shaft barrier shaft IV Emma /52/

1992 the baseline risk assessment of the mining authority Staatstoezicht op de Mijnen required for any construction activity a distance of a radius 7,5 m from the shaft center /22/.

The coordinates of the shaft IV are:

RD-x:	188473
RD-y:	328112
elevation:	+66 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located northern Straße Borgerfietspad (Gemeinde Schinnen) on the property of the US Army Garrison Schinnen (used as supply base)

7.7 Shaft I, Hendrik

The vertical Shaft I of the state mine Hendrik was drilled in 1913. In 1967 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 6,0 m diameter. The shaft was drilled to a total depth of 902,0 m and was used as travelling shaft, drawing shaft and drafting shaft /47/. The shaft was made out of masonry (thickness 0,55 m) and reinforced concrete (thickness 0,35 m). Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 222 m respectively the top carbon is located on -92 m NAP /6/. The shaft has 14 documented insets. The 272 m floor, as the topmost is located in a level of -175,0 m NAP and in a depth of 272 m /6//50/.

In the following figure the strata in the range of the 272 m floor is pictured. Here mainly occur layers of slate as well as Laag III /52/.

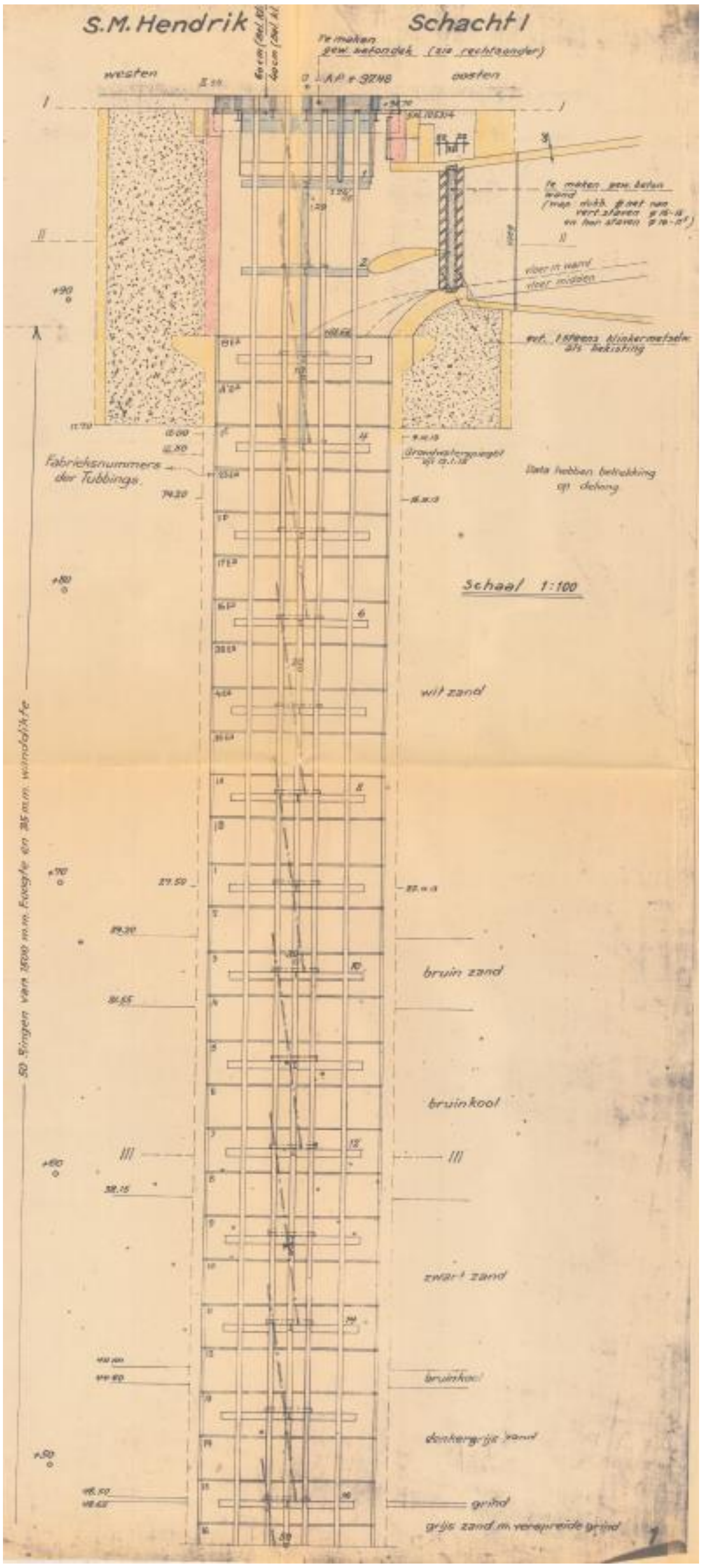


Fig. 100: Strata shaft I Hendrik up to a depth of 50 m /45/

In 1967 a load bearing filling out of 615 m³ of a mixture of concrete (length 11 m) was embedded in the 272 m floor (-175,0 m NAP).

Additionally on the level of the floor an abutment of iron beams covered with a concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. Afterwards the shaft column was backfilled from above the shaft barrier to the ground surface with 12000 t (6890 m³) waste material /7/ /45/ /47/ /50/. In 1969 the shaft was closed with a shaft cover (thickness 0,6 m) with an integrated opening for refilling /9/ /45/ /47/. In 1970 and in 1971 the shaft subsided 0,02 m respectively 0,01 m /10/ /11/. Finally the shaft was closed in 1975 by backfilling the opening for refilling with a mixture of concrete /15//45/.

In the following figures the stabilization respectively the shaft barrier for the shaft I Hendrik are shown.

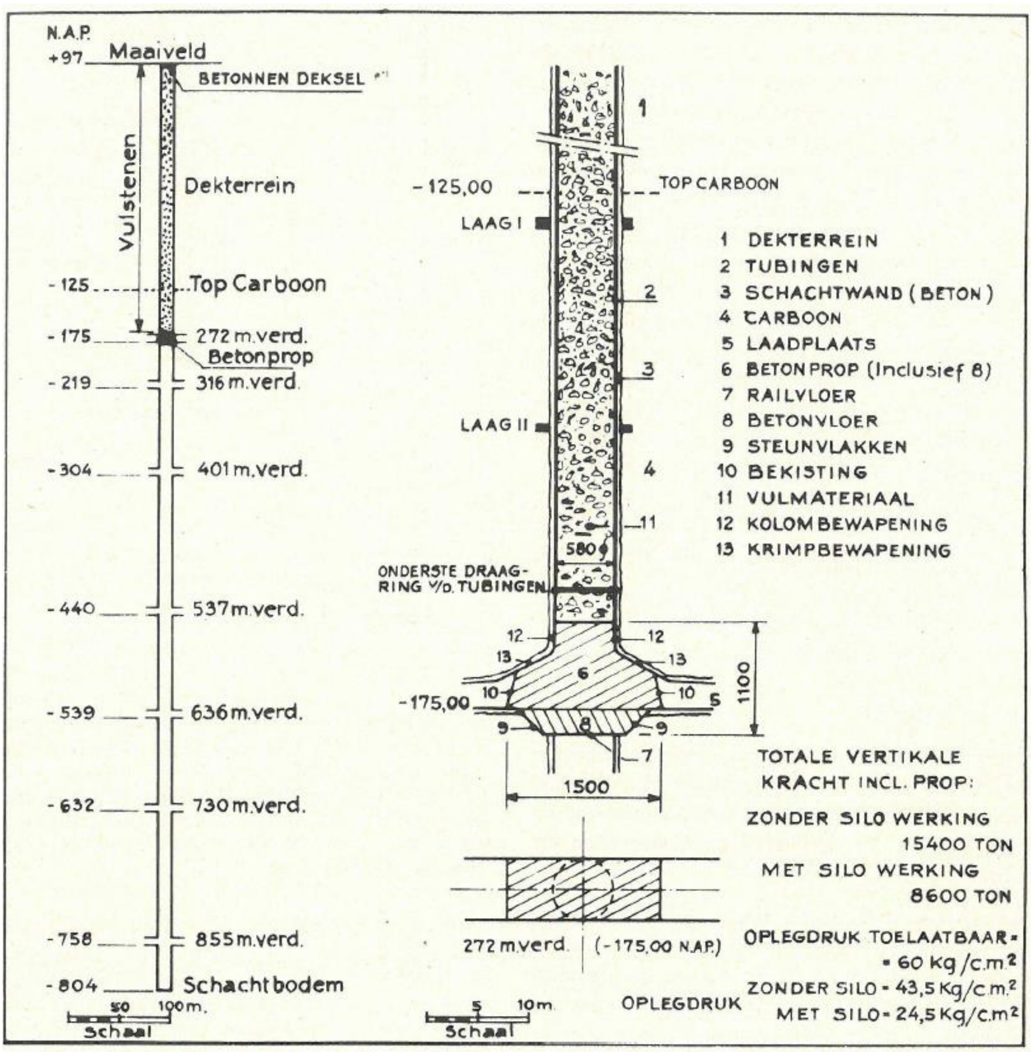


Fig. 101: Stabilization shaft I Hendrik /50/

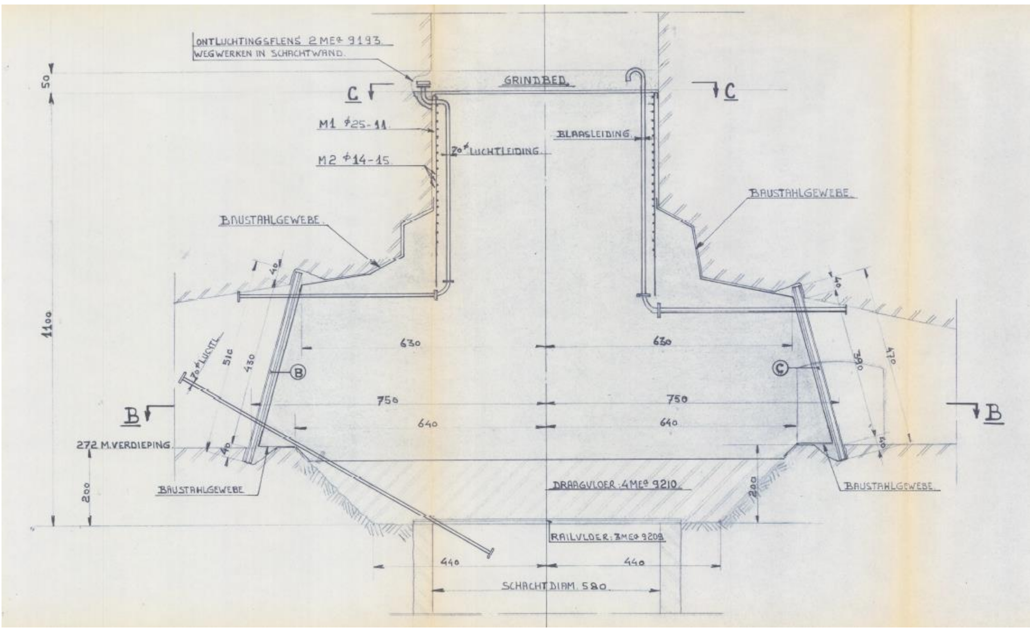


Fig. 102: Shaft barrier shaft I Hendrik /45/

Static calculations of the shaft barrier are existent /45/. Compare the following figures.

D.S.M. N.V. Nederlandse Staatsmijnen

1322 A.B. Heerlen, 2 februari 1967

Statische berekening betonprop in Schacht I Staatsmijn Hendrik op 272 m' verdieping

A. Stalen vloer op 274 - p.

Deze stalen vloer dient als bekistingsvloer voor de 1,50 m' dikke betonplaat.

Eigen gewicht betonvloer: 1,50.2400 =	3600 kg/m2
Eigen gewicht stalen vloer reken	200 kg/m2 +
q_{tot} =	3800 kg/m2

M_{max} = 1/8 x 3800 x $6,25^2$ = 18500 kgm/m'

$\sigma = \frac{M}{W} \Rightarrow W_{vereist} = \frac{M}{\sigma} = \frac{1850000}{1400}$ = 1320 cm3/m'

Aanwezig 20 smalspoorprofielen h = 125 mm'

$W_x = 20 \{0,06 \times 12,5^3\}$ = 2340 cm3/m' > 1320 cm3/m'

S.H.B. pag. 73

B. Betonplaat d = 150 cm

De betonplaat moet het gewicht van de betonprop dragen. Voor de hoogte van de betonprop inclusief de eigen dikte van de betonplaat is te rekenen 9 m'. Dus:

q = 9,00 x 2400 = 21.600 kg/m2.

Veldmoment

M_{max} = 1/8 x 21600 = 5,80 m2 = 91000 kgm/m'

b = 1,00 m' h_t = 150 cm' h = 145 cm' $\sigma_b/\sigma_a = 34^5/1400$

A = 48,8 ^{cm2}/m2 : WAP: Ø 25-10 = 49 ^{cm2}/m'

v.w. = 1/5 x 48,8 = 9,76 ^{cm2}/m' : Ø 14-15 = 10,2 cm2/m'

Steunpuntsmoment: M_{reken} 1/12 x 21600 x $5{,}80^2$ = 60500 kgm/m

b = 1,00 m' h_t = 150 cm' h = 145 cm'

σ_b / σ_a = ‾/1400 A = 32,2 cm2/m'

WAP: Ø 25 + Ø 16-20 = 34,5 cm/m'

W.W. = 1/5 x 32,2 = 6,44 cm2/m' Ø 14-15 = 10,2 cm2/m'

Dwarskracht D = 5,80 m'

$$\rho = \frac{3}{2} \times \frac{\frac{\pi}{4} \cdot D^2 \cdot 21600}{\pi \cdot D \cdot 150} = 3{,}14 \text{ kg/cm2} < 7$$

Geen opgebogen wapening vereist

Betonprop

Bepaling volume beton (zie figuur)

Deel A :	$\frac{\Pi}{4} \times 5{,}8^2 \times 3$	= 79,2 m3
Deel B :	5,8 x 7,8 x 2	= 90,5 m3
Deel C :	½ (12,60 + 15,00) x 4 x 5,80	= 320,0 m3
Deel D :	½ (12,80 + 8,80) x 2 x 5,80	= 125,3 m3 +
	Totaal	615,0 m3

Gewicht betonprop : 615 x 2,4 = 1475 ton

Gewicht wasstenen: (S.G. = 1,9 t/m3) $\frac{\Pi}{4} \times 5{,}80^2 \times 263 \times 1{,}9 =$ 13200 ton

Totaal vertikaal: 13200 + 1475 = 14675 ton

Ontbonden onder hoek van 45° geeft:

$R = \frac{1}{2}\sqrt{2} \times 14675 = 10350$ ton

Opmerking: Gerekend is op het gewicht van de totale zuil vulstenen. Dus silo-werking is verwaarloosd.

Oplegvlak: B = 5,80 m' L = 4,25 m'.

O = 5,80 x 4,25 = 24,7 m2 Reken 25 m2.

Oplegdruk = $\frac{10350}{25}$ = 415 t/m2 = 41,5 kg/cm2

Als schacht gevuld is met water:

Gewicht betonprop: 615 x 1,4 =	860 ton
Gewicht stenen $\frac{\Pi}{4} \times 5{,}8^2 \times 263 \times 0{,}9 =$	6240 ton +
totaal:	7100 ton

Oplegdruk = $\frac{7100}{25}$ = 284 t/m2 = 28,4 kg/cm2

Ponsspanning:

$P_{vertikaal}$ = 14675 ton

Omtrek cilinder = π x 5,8 = 18,25 m'

Hoogte cilinder = 2,00 + 4,00 + 2,00 = 8 m'

$\sigma_{pons} = \frac{14675000}{1825 \times 800} = 10$ kg/cm2 < 15 kg/cm2

Wapening cilinder:

Minimum wapening volgens V.V.A.A. = 0,3%

$A = 0{,}3 \times \frac{II}{4} \times \frac{580^2}{100} = 792$ cm2

Wap 162 Ø 25 (Ø 25-11) = 795 cm2

of 99 Ø 32 (Ø 32 - 18) = 797 cm2

Ir. H.A.M. Quekel
C.B.D. - C.B.

Fig. 103: Static calculation shaft barrier shaft I Hendrik /45/

A static calculation of the retaining wall within the suction channel as well as the concrete cover are existent /45/.

The coordinates of the shaft I Hendrik are:

RD-x:	196480
RD-y:	327759
elevation:	+97 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located at the kerb of Prins Hendriklaan (community Brunssum).

7.8 Shaft II, Hendrik

The vertical Shaft II of the state mine Hendrik was drilled in 1912. In 1967 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 4,0 m diameter. The shaft was drilled to a total depth of 855,0 m and was used as drawing shaft and drafting shaft /47/. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 223 m respectively the top carbon is located on -126 m NAP /6/. The shaft II Hendrik has 14 documented insets. The 272 m floor, as the topmost is located in a level of -175,0 m NAP and in a depth of 272 m /6//50/.

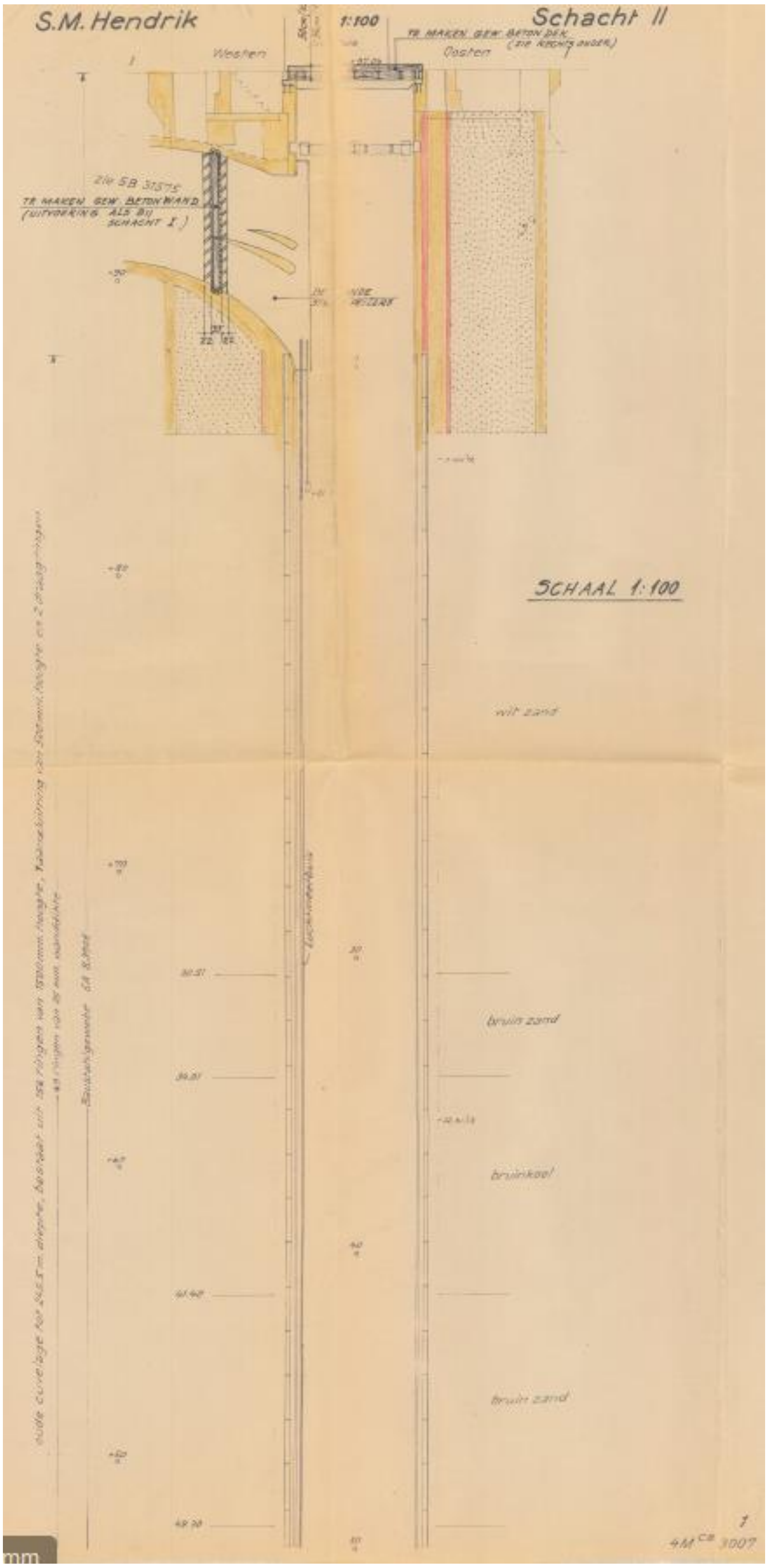


Fig. 104: Strata shaft II Hendrik up to a depth of 50 m /45/

In 1967 a load bearing filling out of 280 m³ of a mixture of concrete (length 9 m) was embedded in the 272 m floor. Additionally on the level of the floor an abutment of iron beams covered with a concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. Afterwards the shaft column was backfilled from above the shaft barrier to the ground surface with 6.800 t (3.445 m³) waste material /7/ /47/ /50/. In 1969 the shaft was closed with a shaft cover (thickness 0,5 m) with an integrated opening for refilling /9/ /45/ /47/. In 1970 and in 1971 there was no subsidence within the shaft filling /10/ /11/. Finally the shaft was closed in 1975 by backfilling the opening for refilling with a mixture of concrete /15//45/.

In the following figures the shaft barrier for the shaft II Hendrik is shown.

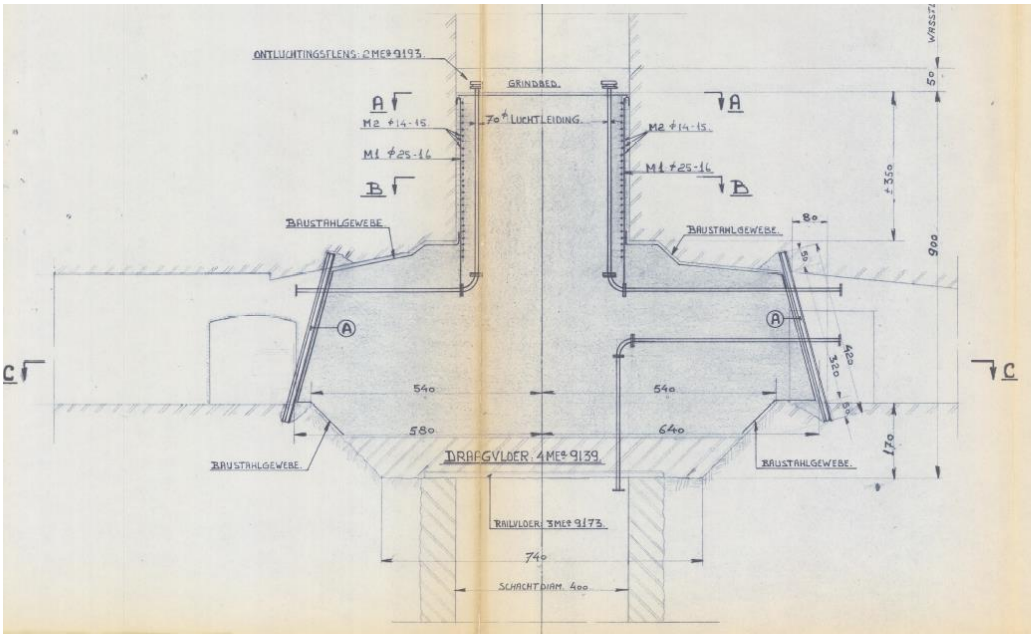


Fig. 105: Shaft barrier shaft II Hendrik /45/

Static calculations of the shaft barrier are existent /45/. Compare the following figures.

DSM NV NEDERLANDSE STAATSMIJNEN

1298 AB

Heerlen, 13 januari 1967.

Statische berekening betonprop in Schacht II Hendrik op 272 m verdieping.

A. Stalen vloer op 273-P.

Deze stalen vloer dient als bekistingsvloer voor de 90 cm dikke betonplaat.

eigen gewicht betonvloer: 0,90 x 2400 = 2160 kg/m2
eigen gewicht stalen vloer reken + 200 kg/m2
q tot 2360 kg/m2

reken q = 2400 kg/m2.

$M_{max} = 1/8 \times 2400 \times 4{,}50^2 = 6075$ kgm/m'

$\sigma = \frac{M}{W}$ $W_{vereist} = \frac{M}{T} = \frac{607500}{1400} = 435$ cm^3/m'

Aanwezig 20 smalspoorprofielen h = 125 mm'

$W_x = 20 \times (0{,}06 \times 12{,}5^3) = 2340$ cm3/m' >> 435

(zie S.I.H.B. pag. 73)

B. Betonplaat d = 90 cm

De betonplaat moet het gewicht van de betonprop dragen.
Voor de hoogte van de betonprop inclusief de eigen dikte van de betonplaat is te rekenen 8,00 m. Dus: q = 8,00 x 2400 = 19200 kg/m2.

Veldmoment: $M_{max.} = 1/8 \times 19200 \times 4{,}00^2 =$ 38400 kgm/m'

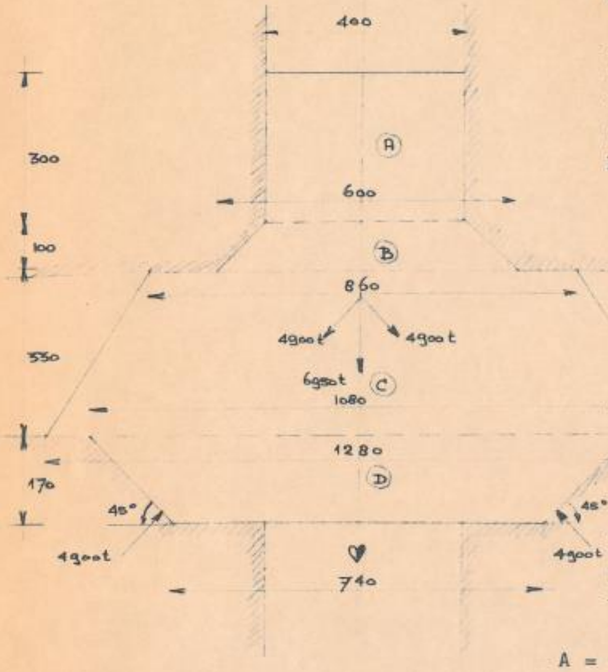
b = 1,00 m h_t = 90 cm' h = 85 cm' $\sigma b/\sigma a$ = 39,5/1400
A = 36 cm2/m' Wapening: Ø 25 - 12 = 41 cm2/m'
verdeelwapening: 1/5 x 36 = 7,2 cm2/m'
Ø 14 - 20 = 7,7 cm2/m'

Steunpuntsmoment. M reken $1/12 \times 19200 \times 4{,}00^2 = 25600$ kgm/m'
b = 1,00 m' h_t = 90 cm' h = 85 cm $\sigma b/\sigma a$ = 31/1400
A = 23,4 cm2/m' Wapening: Ø 19 - 12 = 23,5 cm2/m'
verdeelwapening: Ø 14 - 20 = 7,7 cm2/m'

Dwarskracht D = 4,00 m'

$$\rho = \frac{3}{2} \times \frac{\frac{\pi}{4} \cdot D^2 \times 19200}{\pi \cdot D \cdot 90} \quad 3{,}2 \text{ kg/cm2} < 7$$

Geen opgebogen wapening vereist.



Ponsspanning:

P vertikaal = 6950 ton

omstrek cilinder = $\pi \cdot 4$ = 12,56 m'

hoogte cilinder = 3,30 + 1,70 + 1,00 = 6,00 m'

$$\sigma \text{ pons} = \frac{6950000}{1256 \times 600} = 9{,}25 \text{ kg/cm2} < 15 \text{ kg/cm2}$$

Wapening: cilinder:

Minimum wapening volgens V.V.A.A. = 0,3%

$$A = 0{,}3 \times \frac{\pi}{4} \cdot \frac{400^2}{100} = 377 \text{ cm2}$$

Wap. 77 Ø 25 (Ø 25 - 16) = 378 cm2

Betonprop

Bepaling volume beton (zie figuur)

Deel /A/:	$\frac{\pi}{4} + 4^2 \cdot 3$	= 37,7 m3
Deel /B/:	1/2 (6,00 + 4,00) . 1,00 . 4,00	= 20,0 m3
Deel /C/:	1/2 (8,60 + 12,80) . 3,30 . 4,00	=141,3 m3
Deel /D/:	1/2 (10,80 + 7,40) . 1,70 . 4,00	= 62,0 m3
Totaal:		261,0 m3

Gewicht betonprop: 261 x 2,4 = 625 ton krimpwap.: ⌀ 14 - 20
Gewicht wasstenen: s.g. = 1,9 ton/m3

$\frac{\pi}{4} \cdot 4{,}00^2 \cdot 265 \cdot 1{,}9 = 6325$ ton

Totaal vertikaal 6325 + 625 = 6950 ton
Ontbonden onder hoek van 45° geeft:

$R = \frac{1}{2}\sqrt{2} \cdot 6950 = 4900$ ton

Opmerking: Gerekend is op het gewicht van de totale zuil vulstenen: Dus silowerking is verwaarloosd.

Oplegvlak: B = 4,00 m' l = 4,00 m' O = 4 x 4 = 16 m2
Oplegdruk: $= \frac{4900}{16} = 306$ t/m2 = 30,6 kg/cm2

Als schacht gevuld is met water:

Gewicht betonprop = 261 x 1,4 = 365 ton
Gewicht stenen $\frac{\pi}{4} \cdot 4^2 \cdot 265 \cdot 0{,}9 = 3000$ ton
Tot: 3365 ton

Oplegdruk $= \frac{3365}{16} = 210$ t/m2 = 21 kg/cm2.

Fig. 106: Static calculation shaft barrier shaft II Hendrik /45/

A static calculation of the retaining wall within the suction channel as well as the concrete cover is existent /45/.

The coordinates of the shaft II Hendrik are:

RD-x:	196543
RD-y:	327791
elevation:	+97 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the property of the NATO Joint Force Headquarters southern of Rimburger Weg (community Brunssum).

7.9 Shaft III, Hendrik

The vertical shaft III of the state mine Hendrik was drilled in 1929. 1967/1968 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section of 5,4 m diameter. The shaft was drilled to a total depth of 454,0 m and was used as drafting shaft. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 199 m respectively the carbon surface is located on -39 m NAP /6/. The shaft III Hendrik has 4 documented insets. The 183 m floor, as the topmost is located in a level of -85,0 m NAP and in a depth of 245 m /6//50/.

In 1967 the shaft was closed on the 316 m floor with a load bearing filling consisting of concrete (length 22 m) and additionally above on the 183 m floor with a second load bearing filling of a length of 14 m. This back stowing had to be performed from above ground therefore concrete and demolition waste were backfilled into the shaft alternately. Furthermore above the filling a protective layer of sand was inserted /7/. Overall 720 m³ concrete and 700 m³ demolition waste were backfilled. Finally in 1968 the shaft was backfilled with another 10.300 m³ waste material /47//50/. Finally the shaft was provided with a reinforced concrete cover and an opening for refilling on ground level /8//47/. 1970 the shaft column subsided 0,02 m, 1971 another 0,01 m and 1972 additionally 0,01 m /10//11//12/. In 1975 the opening for refilling was backfilled with a mixture of concrete /15//45/.

The following figures show the implementation planning for the shaft III Hendrik.

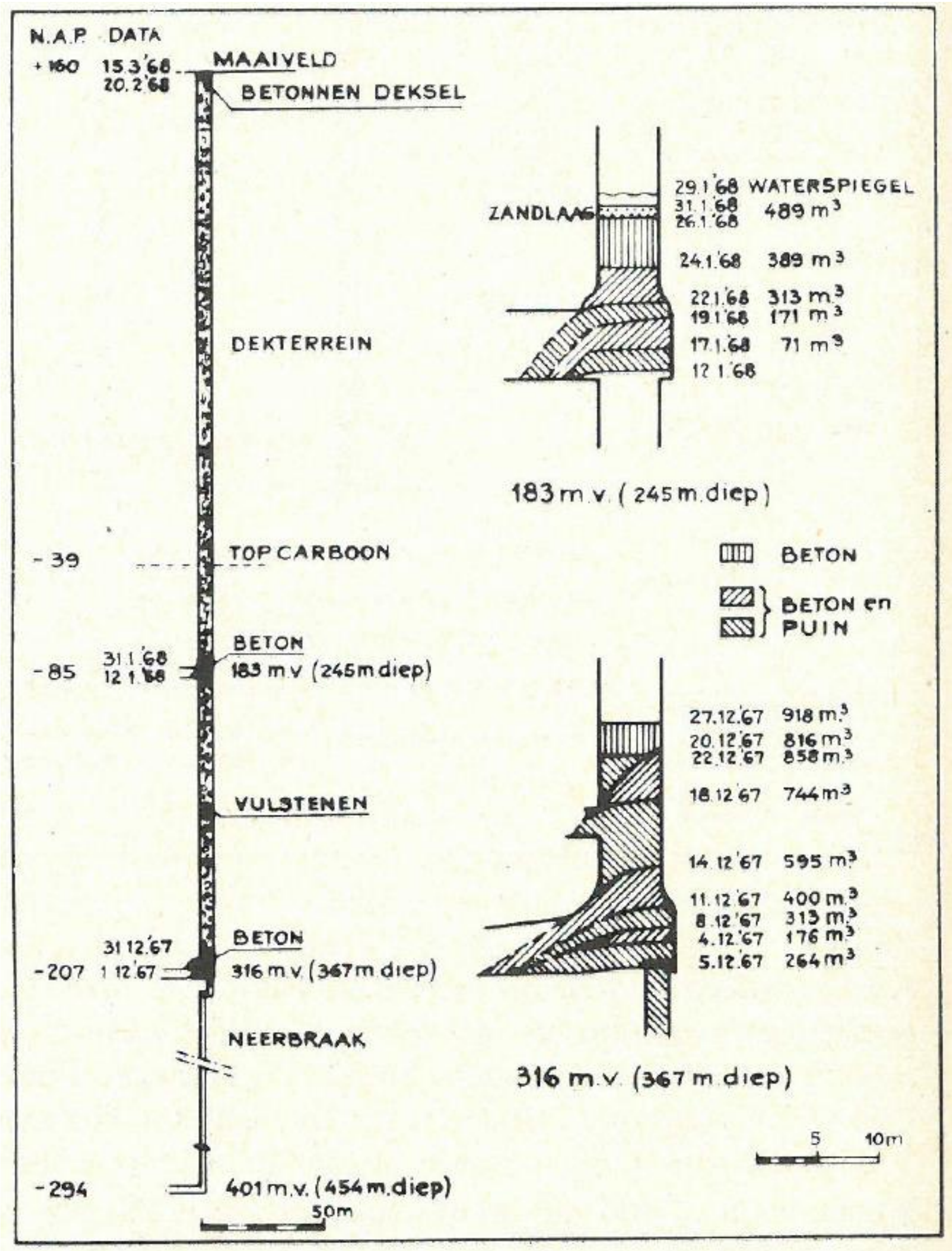


Fig. 107: Stabilization, shaft III Hendrik /50/

The coordinates of shaft III Hendrik are:

RD-x:	199096
RD-y:	325391
elevation:	+163 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on a sports field at Schachtstraat (Community Landgraaf). Within the shaft area a shelter was build /46/.

7.10 Shaft IV, Hendrik

The vertical Shaft IV of the state mine Hendrik was drilled in 1953. In 1969 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section with an inside diameter of 6,60 m. The shaft was drilled to a total depth of 1.058,0 m and was used as travelling shaft. Within the overburden the shaft consists of tubbing support and within the carbon he was made of masonry (thickness 0,6 m). There are no details available about any shaft fittings.

In this area the overburden has a thickness of 219 m respectively the top carbon is located on -124 m NAP /6/. In the following figure the strata of the overburden in the range of the shaft IV Hendrik is pictured /68/.

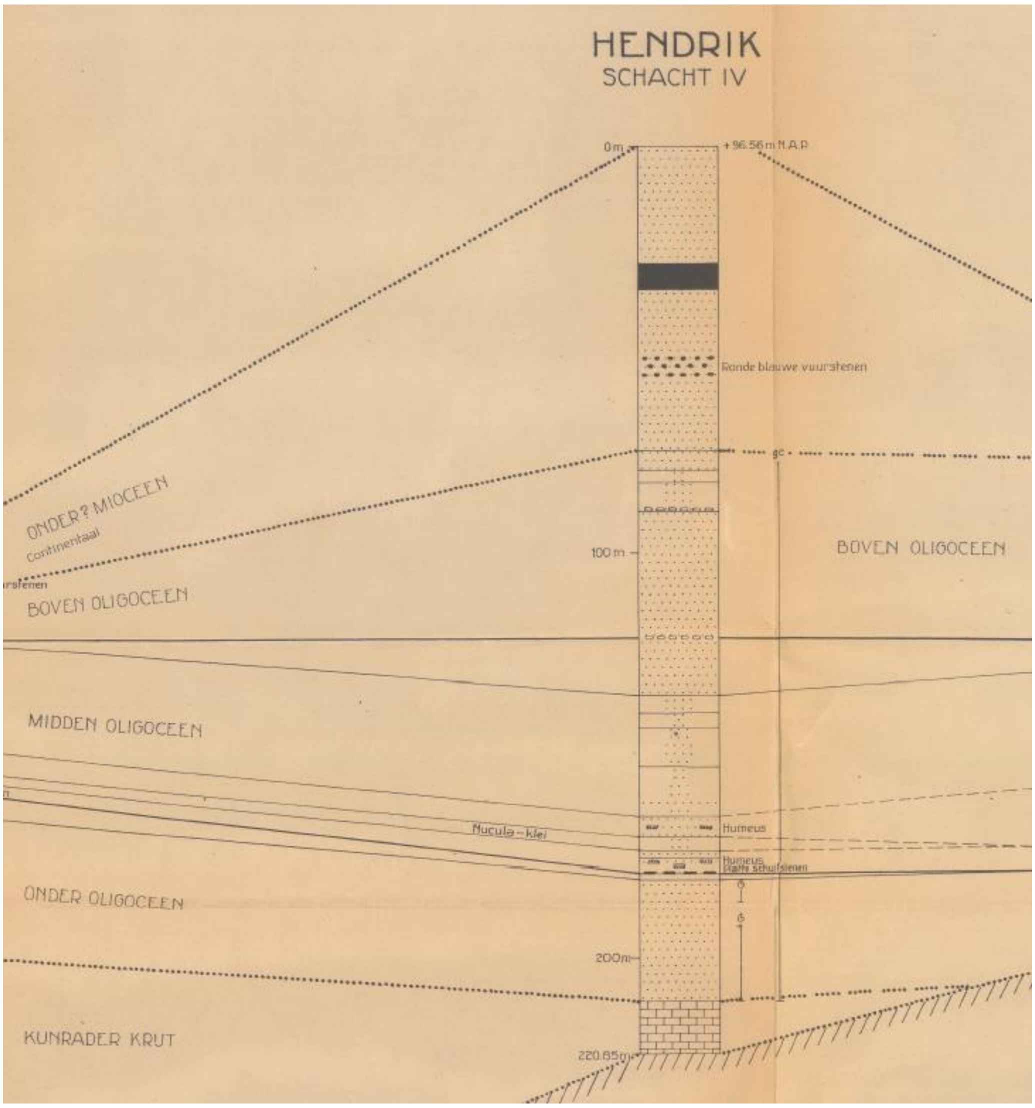


Fig. 108: Strata of the overburden, shaft IV Hendrik /68/

The shaft IV Hendrik has 16 documented insets. The 272 m floor, as the topmost is located in a level of -174,5 m NAP and in a depth of 270 m /6//50/.

In 1969 a load bearing filling out of 774 m³ of a mixture of concrete was embedded in the 272 m floor. In the first on the level of the 272 m floor an abutment of iron beams covered with a concrete board, which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling and the backfilled loose material is spread best. The base of the barrier (thickness 2,5 m) was produced in two sections. A temporary ventilation steel pipeline (ø 1.000 mm) was embedded in the base of the barrier. During the following back stowing with 600 m³ concrete (length 10,5 m) this steel pipeline was extended with an additional steel pipeline (diameter 0,3 m). In the end the pipeline as backfilled with concrete completely. Above the barrier the shaft column was backfilled with 9.275 m³ waste material /7//9//47//50/. 1970 the shaft was provided with a concrete cover with an opening for refilling /9/ /10/ /47/. In 1970 the shaft column subsided 0,01 m /10/. Later on there was no further subsidence /11/. In 1975 the opening for refilling was backfilled with a mixture of concrete /15//45/.

1992 a number of point-baring piles were founded surrounding the shaft. On top of the piles a beam foundation was installed for development. By this means the shaft barrier is not pressurized with the weight of the buildings. Between shaft mouth and development openings for ventilation were left. These measures were executed by recommendation of the mining authority Staatstoezicht op de Mijnen /22/.

The following figures show the shaft barrier of shaft IV Hendrik.

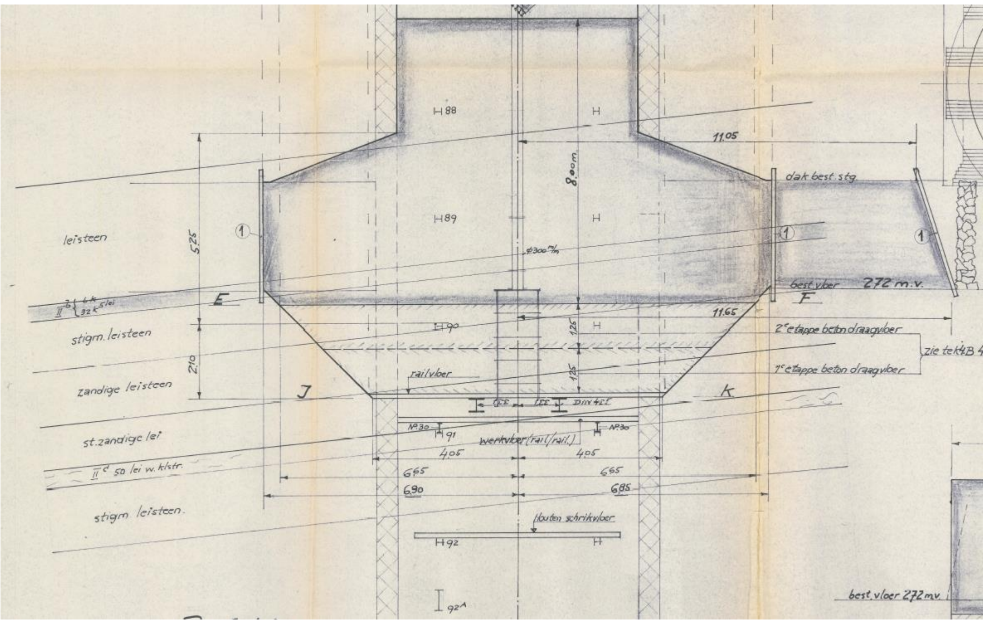


Fig. 109: Shaft barrier shaft IV Hendrik with genuine rock layers /47/

In the range of the shaft barrier mainly slate occurs.

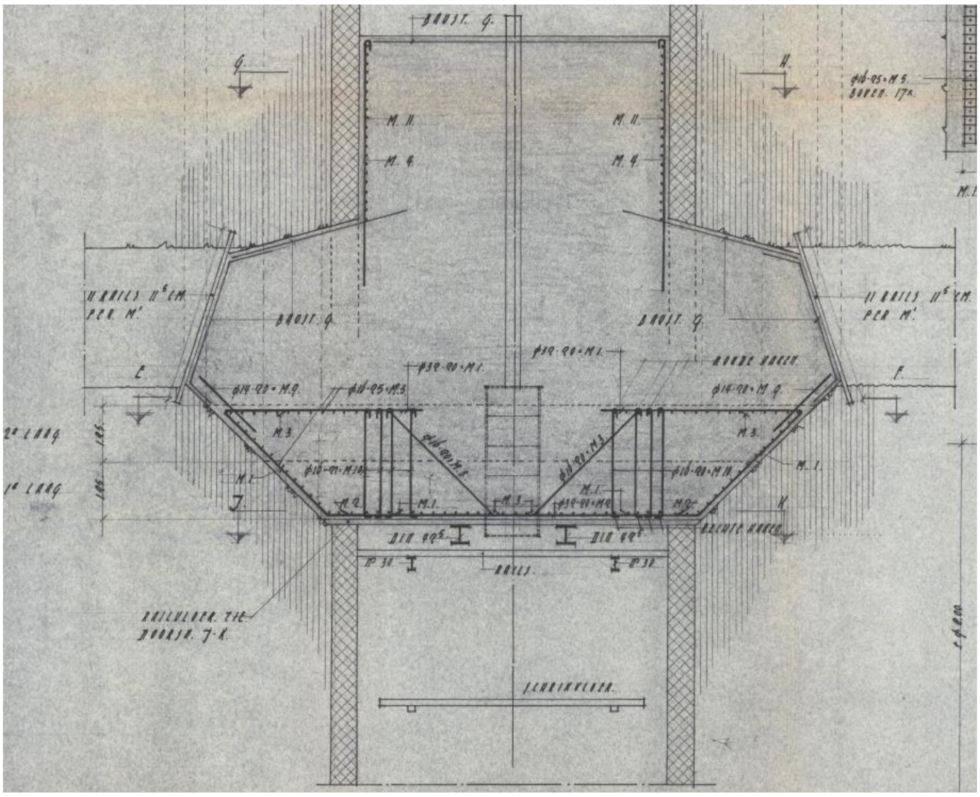


Fig. 110: Shaft barrier, shaft IV Hendrik /47/

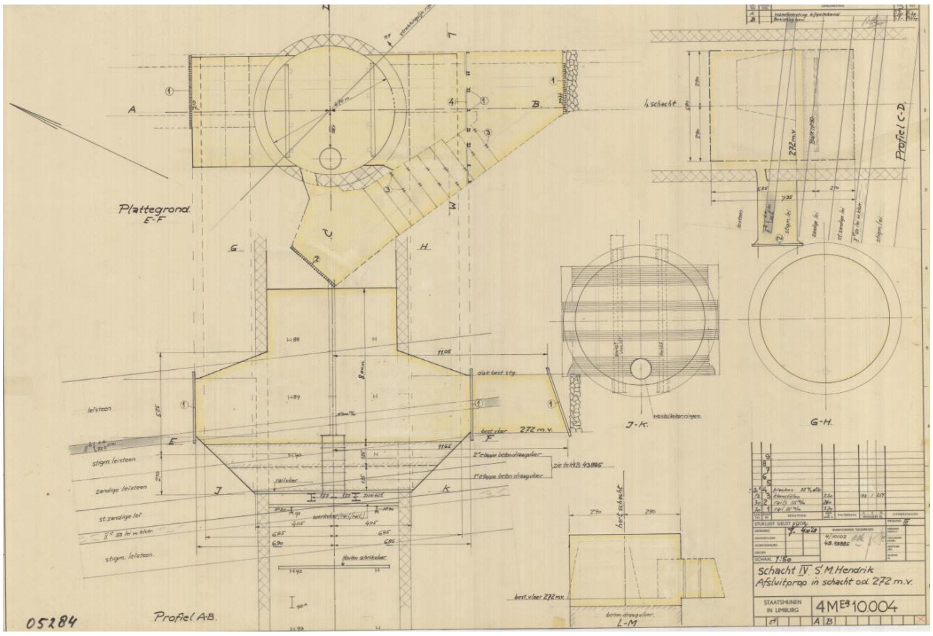


Fig. 111: Shaft barrier, shaft IV Hendrik /54/

Static calculations of the shaft barrier are existent /39/. Compare the following figures.

Staatsmijn Emma/Hendrik, 16 december 1968
Nr. 1186 Ea/1.1.5

STATISCHE BEREKENING BETONPROP IN SCHACHT IV HENDRIK OP DE 272 METER VERDIEPING

I. Betonprop (zie tekening 4M Ea 10.004)

De totale inhoud van de betonprop inclusief H-balken bedraagt	610,3 m^3
inhoud H-balken (met gedemonteerde stroomkappen)	0,2 m^3
inhoud wentelkoker over proplengte	2,6 m^3
inhoud 300 mm leiding over proplengte	0,5 m^3
inhoud prop exclusief H-balken, wentelkoker en Ø 300 leiding	607,0 m^3

inhoud betonplaat exclusief wentelkoker:

(incl.is + 2,4 m^3 — 1,2 m^3 / 1,2 m^3) onderste deel 70,6 m^3; bovenste deel 89,5 m^3

totaal 160,1 m^3

inhoud "natte prop" exclusief wentelkoker en Ø 300 leiding 446,9 m^3

V_{totaal} = 610 m^3; gerekend à 2,4 ton/m^3 is dit ca. 1465 ton.

Er wordt gerekend met het ongunstigste geval:

a. onder de prop geen water;

b. boven de prop een vulling van wasstenen die met water is verzadigd.

Vanwege de silowerking kan volgens bijgaande bijlage II voor de wasstenen gerekend worden met een equivalente vulhoogte van 3x de diameter van de schacht.

Het soortelijke gewicht γ van de kolom wasstenen welke verzadigd is met water volgt uit: $\gamma = \gamma_s \left(1 - \frac{\gamma_v}{\gamma_k}\right)$ (zie hiervoor eveneens bijlage II)

Hierin is
γ_k = soortelijk gewicht van de korrels (ca. 2,5)
γ_v = soortelijk gewicht van de omringende vloeistof (ca. 1)
γ_s = soortelijk gewicht van het droge stortmateriaal (ca. 2)
γ = schijnbare soortelijke gewicht van de stortmassa verzadigd met water.

Indien $\gamma_s = 2$, dan is $\gamma = 1,2$. Gerekend wordt met dit gewicht.

De totale verticale kracht volgt door sommering van de volgende componenten:

a. gewicht van de totale betonprop;
b. de hydrostatische druk;
c. de door het vulmateriaal op de prop uitgeoefende druk.

Wordt bij het berekenen van de oplegkracht rekening gehouden met de kleef tussen prop en wanden, dan moet de verticale kracht verminderd worden met de kleefkracht (kleefopp. x kleefkracht per opp. eenheid).

A. Berekening oplegdruk waarbij de kleef van de prop t.o.v. de schachtwand en de laadplaatsen verwaarloosd is.

ad. a) Betonprop: volume x s.g. = 610 . 2,4 ca. 1470 ton

ad. b) Waterkolom: $\frac{\pi}{4} \cdot (6,7)^2 \cdot 266 \cdot 1 =$ 9380 ton

ad. c) Vulstenen: $3 \cdot 6,7 \cdot \frac{\pi}{4} \cdot (6,7)^2 \cdot 1,2 =$ 850 ton

P totaal 11700 ton

Ontbinding van $P_{tot.}$ onder een hoek van 45° geeft

$R = \frac{1}{2}\sqrt{2} \cdot 11700 =$ 8270 ton

oplegvlak $0 = 3,3 \times \sqrt{2} \times 5,8 =$ 27,3 m^2

Oplegdruk $\sigma_d = \frac{R}{0} = \frac{8270}{27,3} = 303$ ton/m^2, dus ca. 30,3 kg/cm^2

B. Berekening van de oplegdruk waarbij wel rekening wordt gehouden met de kleef van de prop t.o.v. de schacht- en laadplaatswanden.
Bij de berekening wordt de kleef aangenomen op 2,5 kg/cm^2, er van uitgaande dat de laadplaatswanden ruw zijn en de schachtwand bewust geruwd wordt.

Nuttig kleefoppervlak:

laadplaats $4 \times \frac{1}{2}(3,5 + 7,5) \times 4 =$ 88 m^2

schachtwand $2 \times 10,5 \times 5,6 =$ 117,6 m^2

205,6 m^2

In de zuidelijke laadplaatsaansluiting is door ca. 70 m^3 beton extra te storten aansluiting gemaakt met de vaste laadplaatswand.

Bij de bepaling van het nuttig kleefoppervlak van de schachtwand zijn alléén de segmentgedeelten loodrecht op de as van de laadplaats in rekening gebracht.

Totale kleefkracht van de betonprop 206 x 25 =	5150 ton
Resterende verticale kracht 11700 - 5150 =	6550 ton
Resterende kracht loodrecht op oplegvlak 6550 x $\frac{1}{2}\sqrt{2}$ =	4640 ton
Resterende oplegdruk 4640 : 273 =	17 kg/cm^2

II. Stalen vloer op -275 AP

Deze stalen vloer dient als bekistingsvloer voor de onderste helft van de 2,50 meter dikke gewapend betonnen draagvloer.
Na verharding van de onderste helft dient deze als kistvloer voor de bovenste helft.

De belasting op deze stalen vloer bedraagt:

a. gewicht onderste helft betonvloer

h x s.g. = 1,25 x 2400 = 3000 kg/m^2

b. eigen gewicht stalen vloer: (reken) = 300 kg/m^2

q_{totaal} 3300 kg/m^2

Railsvormen dek: h = 110 mm
b = 90 mm

moerbalken hart op hart 2,25 m'

$\bar{\sigma}_t = 700$ kg/cm^2. W. aanwezig : $0,06 \times 11^3 = 80$ cm^3

per m' 11 stuks = 11 x 80 = 880 cm^3/m'

$M_{max.} = 1/10 \times 3300 \times 2,25^2 = 1680$ kgm/m'

$W_{vereist} = \frac{1680,00}{700} = 240$ cm^3/m' < 880

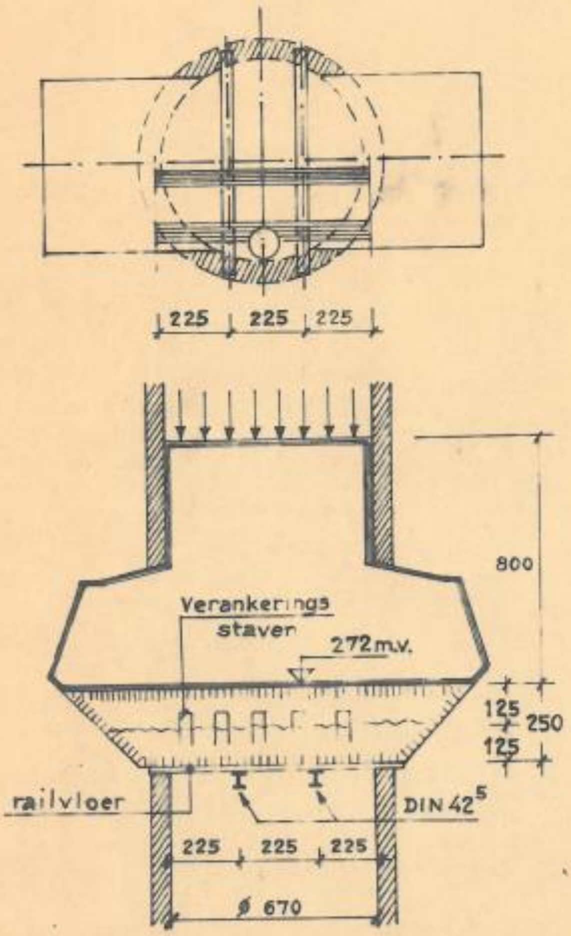
Moerbalken 2,25 m' hart op hart ℓ = 7,00 m'

q = 2,25 x 3300 = 7400 kg/m'

$M_{max.}$ = 1/8 x 7400 x 7^2 = 45500 kgm

$W_{vereist}$ = $\frac{45500.00}{1400}$ = 3240 cm^3

Kies DIN 42^5 (Wx = 3270 cm^3 G = 166 kg/m')

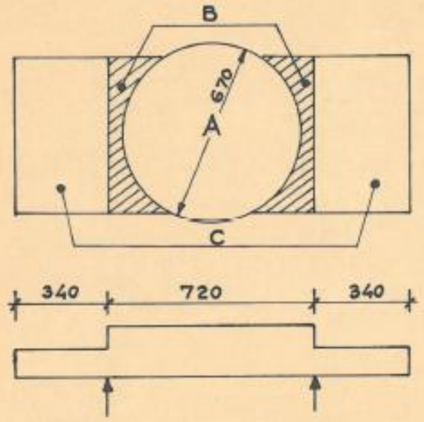


III. Betonplaat

De betonplaat moet het gewicht van de betonprop (inclusief eigen gewicht) dragen.

$d_{betonplaat}$ =2,50 m' $h_{betonprop}$=10,5 m'

gewicht deel A =	890 ton
gewicht deel B =	155 ton
deel A + B =	1045 ton
deel C =	420 ton
deel A + B + C	1465 ton



Veldmoment = $\frac{1}{8}$. 1045 . (7,20) - 210 x 1,25 = 680 tm

Beschikbare breedte = 5,6 meter

M veld per m' $\frac{680}{5,6}$ = 122 ton

h_t = 250 cm h = 245 cm' σ_b / σ_a = -/1400

$\frac{M}{bh^2}$ = $\frac{122000}{1x245^2}$ = 2,02 ω_o = 0,155 % = 38 cm^2/m'

Wap. ø 25 - 12,5 = 39,2 cm^2/m' of ø 32 - 21 = 38,3 cm^2/m'

Verdeelbewapening $\frac{1}{5}$ x 38 = 7,6 cm^2/m' Wap ø 16 - 26 = 7,7 cm^2/m' of
ø 19 - 37 = 7,7 cm^2/m'

Steunpuntsmoment

$M_{steunpunt}$ reken $\frac{1}{12}$. 1045 . 7,20 = 630 tm , over een breedte van 5,80 m.

Per m' = $\frac{630}{5,80}$ = 110 ton

h_t = 250 cm' h = 245 cm' σ_b / σ_a = -/1400

$\frac{M}{bh^2}$ =1,84 ω_o = 0,140 % = 34,2 cm^2/m'

Wap. ø 25 + ø 14 - 18 = 35,9 cm^2/m' of
ø 32 + ø 19 - 30 = 36 cm^2/m'

Verdeelbewapening $= \frac{1}{5} \cdot 34{,}2 = 6{,}8\ cm^2/m'$

Wap. Ø 14 - 20 = 7,6 cm^2/m
Ø 16 - 25 = 8 cm^2/m

Dwarskracht in plaat

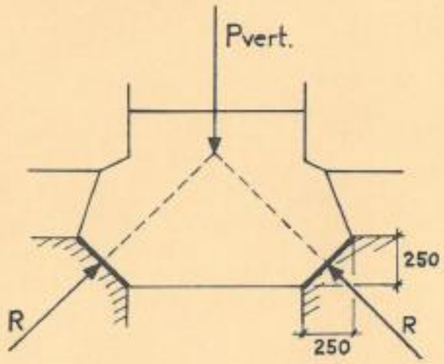
$h_t = 2{,}50\ m'$ $b = 5{,}80\ m'$

D = gewicht betoncilinder 930 ton

$\varrho = \frac{3}{2} \cdot \frac{930000}{2 \times 580 \times 250} = 4{,}8\ kg/cm^2 = \ < 7$

Geen opgebogen wapening vereist.

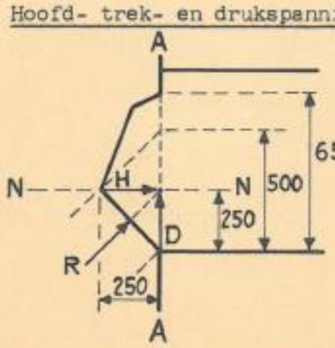
Gesteente-druk exclusief kleef van de betonprop t.o.v. de schachtwand en de laadplaatswanden



$P_{vert.}$:	waterkolom	9.380 ton
	vulstenen	850 ton
	betoncilinder	930 ton
	totaal	11.160 ton

$R = \frac{11.160}{2} \times \sqrt{2} = 5580\ \sqrt{2}$
$= 7900$ ton

Hoofd- trek- en drukspanning



In vlak AA werken t.g.v. R = 7900 ton

a. een dwarskracht D van 5580 ton
b. een horizontale component H van 5580 ton exentrisch op doorsnede AA.

In het snijpunt van NN en AA komen de volgende spanningen

horizontale = verticale schuifspanning $\tau = \frac{5580000}{580 \times 0{,}99 \times 645} = 15{,}1\ kg/cm^2$

Normaalspanning $\sigma = \frac{5580000}{580 \times 500} = 19{,}3$ kg/cm^2

Hoofdspanning $\sigma' = \frac{19{,}3}{2} \pm \sqrt{(\frac{19{,}3}{2})^2 + (15{,}1)^2}$

$= 9{,}65 \pm 17{,}9$

Hoofddrukspanning $= 9{,}65 + 17{,}9 = 27{,}55$ kg/cm^2) *)

Hoofdtrekspanning $= 9{,}65 - 17{,}9 = - 8{,}25$ kg/cm^2)

*) in vlakken die een hoek maken met NN die bepaald is door

$tg\ 2\varphi = \frac{2 \times 15{,}1}{19{,}3} = 1{,}565$

$\varphi = 29^\circ$

Ponsspanningen in doorsnede AA

De ponsspanning bedraagt $\rho = \frac{11160000}{2/3 \times \quad \times 670 \times 650} = 12{,}3$ kg/cm^2

Stellen wij: kubusdrukvastheid $K_d = 225$ kg/cm^2

kubustrekvastheid $K_t = 25$ kg/cm^2

normaalspanning t.g.v. H = 5580 ton: $\sigma = 19{,}3$ kg/cm^2

De ponsvastheid $\rho_k = \sqrt{(225 - 15)\ (25 + 15)}$

$= 91{,}5$ kg/cm^2

De veiligheidsfactor is dan $\frac{91{,}5}{12{,}3} = 7{,}45$

Wapening cilinder

Minimum wapening volgens V.V.A.A. = 0,3 %.

$A = 0{,}3 \cdot \frac{\pi}{4} \cdot \frac{6{,}702}{100} = 1060$ cm^2

Wap. Ø 25 - 10 (215 stuks Ø 25) = 1055 cm^2

Berekening wandbekisting

Gerekend wordt met één trek per kwartier; per trek wordt gestort 3 m^3.
Na 4 uur begint de beton op te stijven, zodat de zijwaartse druk 4 uur lang toeneemt en daarna constant blijft.
In 4 uur wordt gestort 4 x 3 x 4 = 48 m^3.

Gemiddelde lengte van de betonprop = $\frac{14,20 + 12,20}{2}$ = 13,20 m'

Breedte van de prop = 5,80 m
De storthoogte behoren bij 48 m^3 is dan:

$$h = \frac{48}{13,20 \cdot 5,80} \approx 0,70 \text{ m'}$$

De zijwaartse druk wordt dus 0,70 x 2400 = 1680 kg/ m^2.
De rails worden onder een hoek van ± 60° geplaatst en steunen in het dak en in de vloer σ_t = 700 kg/cm2.

$l \approx$ 4,00 m'. $M_{max} = 1/8 \cdot 1680 \cdot 4,00^2 = 3360$ kgm/m'

$$W_{vereist} = \frac{336000}{700} = 480 \text{ cm}^3$$

rails: h = 110 mm' b = 90 mm' per m' breedte dus 11 stuks

$$W_{aanwezig} = 11 \times 0,06 \cdot 11^3 = 880 \text{ cm}^3 > 480 \text{ cm}^3$$

Wijziging 18 april 1969

Fig. 112: Static calculation, shaft barrier, shaft IV Hendrik /47/

The coordinates of the shaft are:

RD-x:	196577
RD-y:	327721
elevation:	+97 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located on the property of the NATO Joint Force Headquarters northern of Venweg (community Brunssum).

7.11 Shaft I, Maurits

The vertical Shaft I of the state mine Maurits was drilled in 1916. In 1968 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section with an inside diameter of 5,8 m. The shaft was drilled to a total depth of 856,0 m and was used as drafting shaft, travelling shaft and drawing shaft /48/. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 303 m respectively the top carbon is located on -233 m NAP /6/. The strata of the overburden is pictured in figure 113. The shaft I Maurits has 10 documented insets. The 391 m floor, as the topmost is located in a level of -319,0 m NAP and in a depth of 389 m /6//50/. In 1967 a load bearing filling out of 704 m³ of a mixture of concrete was embedded in the 391 m floor. In the first on the level of the 391 m floor an abutment of iron beams covered with a concrete board (325 kg Portland A-cement pro m³), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling (240 kg blast furnace cement A pro m³) and the backfilled loose material is spread best. Furthermore above the filling a protective layer of gravel was inserted /8/. 1968 the shaft column was backfilled above the barrier with a total of 13500 m³ waste material /7/ /8/ /48/. 1969 the shaft was provided with a reinforced concrete cover (thickness 0,7 m) with an opening for refilling /9/ /48/. In 1970 the shaft column subsided 0,73 m, thereof only 0,07 m in 1971 /10//11/. In 1973 the shaft column subsided another 0,03 m. Up to that date the subsidence overall measured 17,43 m /12/. In 1973 there was no further subsidence /13/. In 1974 the shaft surrounding fore-shaft (depth of 20 m) was backfilled with sand /14/. In 1976 a new subsidence required a back stowing with additionally 60 m³ of sand and 110 m³ of water /16/. In 1981 the opening for refilling was backfilled with a mixture of concrete /49/.

The figure below shows the shaft barrier of the shaft I Maurits.

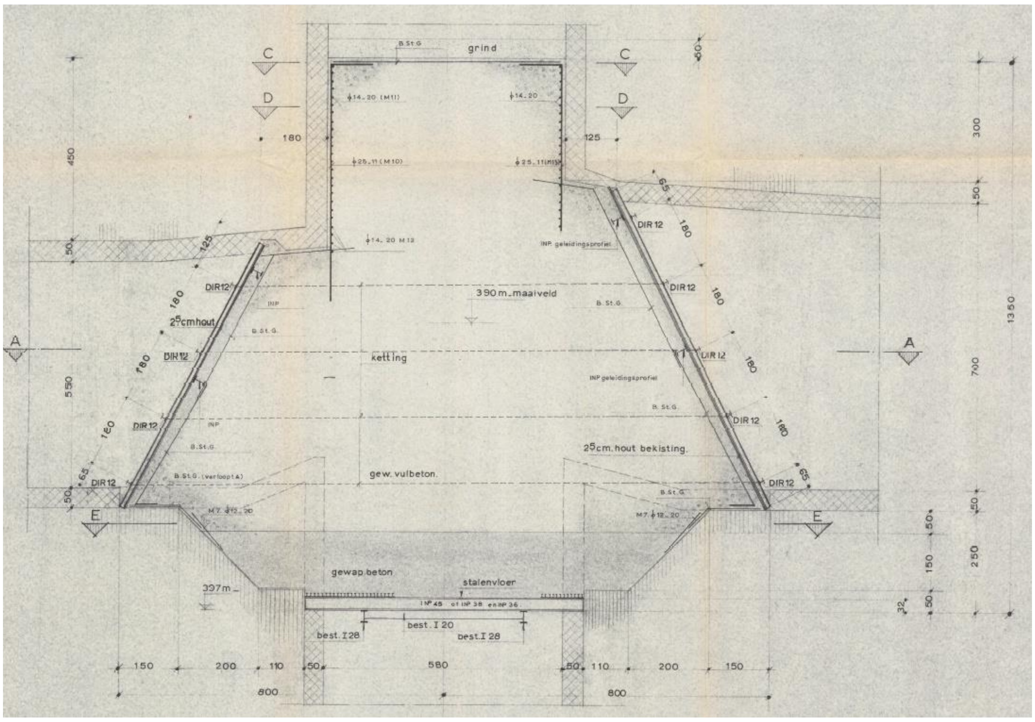


Fig. 113: Shaft barrier, shaft I Maurits /48/

Static calculations of the shaft barrier are existent /48/. Compare the following figures.

NV NEDERLANDSE STAATSMIJNEN Heerlen, 27 april 1967

nr. 1463 A.B.

Statische berekening v.d. Betonprop in Schacht I
Staatsmijn Maurits op de 391 m' verdieping.

BETONPROP

Bepaling volume beton (zie figuur)

Cilinder deel /A/: Ø 5,80 m' h = 3,00 m'

Volume: $\frac{\pi}{4}$. 5,80^2.3,00 = 79 m3

Deel /B/: ½ (16,00 + 8,60).7,50.5,00 = 461 m3

Deel /C/: ½ (14,00 + 7,80).3,00.5,00 = 164 m3

V_{Totaal} = 704 m3

Gewicht betonprop totaal: 704 x 2,4 = 1690 ton.

Reken 1700 ton

Vulstenen: ℓ = 397-13,50 = 383,50 m' S.G. = 1,9 t/m3

Gewicht vulstenen: $\frac{\pi}{4}$. 5,80^2.383,50.1,9 = 19300 ton

(8260 m3) Totaal: 21000 ton

Ontbonden onder hoek van 45° geeft:

R = $\frac{1}{2}\sqrt{2}$.2100 = 14850 ton.

Opmerking: gerekend is op het gewicht van de totale zuil vulstenen. Dus "silowerking" is verwaarloosd.

Opleg vlak: Breedte = 5,00 m'
Lengte reken 6,00 m'
Oppervlakte = 6 x 5 = 30 m2

Opleg druk = $\frac{14850}{30}$ = 495 t/m2 = 49,5 kg/cm2

ALS SCHACHT IS GEVULD MET WATER

Gewicht betonprop = 704 x 1,4 = 985 ton
Gewicht vulstenen = $\frac{\pi}{4}$. 5,80^2.383,50.0,9 = 9135 ton
Totaal 10120 ton

R = ½V2-10120 = 7150 ton
Opleg druk = $\frac{7150}{30}$ = 238 t/m2 = 23,8 kg/cm2

A. Stalen vloer op 397 - P

Deze stalen vloer dient als bekistingsvloer voor de 1,50 m' dikke betonplaat.

Eigen gewicht betonvloer: 150 x 2400 =	3600	kg/m2
Eigen gewicht stalen vloer reken	200	"
q Totaal	3800	kg/m2

Rails vormen dek: h = 110 mm' b = 90 mm'
per m' aanwezig 11 stuks.
W aanwezig = 11 x h 0,06.11^3 = 880 cm3/m' $\bar{\sigma}$ = 700 kg/cm2
Moerbalken 1,65 m' h.o.h.
M max. = 1/10.3800.1,65^2 = 1310 kgm/m'
W vereist = $\frac{131000}{700}$ = 187 cm3/m' << 880.

Moerbalken: h.o.h. 1,65 m' l = 6,00 m'
q = 1,65.3800 = 6275 kg/m'
M max. = 1/8.6275.6,00^2 = 28200 kgm
W = $\frac{2820000}{1400}$ = 2020 cm3

DIN 32 { Wx = 2016 cm3
G = 134,5 kg/m'
G Totaal = 4 x 6 x 134,5 = 3230 kg.

Opmerking: Schachtbalken (I28 h.o.h. 4,10 m') zijn niet sterk genoeg om moerbalken te dragen.
Moerbalken moeten dus doorlopen tot schachtrand.

B. Betonplaat d = 150 cm'

De betonplaat moet het gewicht van de betonprop dragen. Voor de hoogte van de betonprop inclusief de eigen dikte van de betonplaat is te rekenen 9 m'.

Dus q = 9,00 x 2400 = 21600 kg/m2.

VELDMOMENT

$M_{max.} = 1/8.21600.5,80^2 = 91000$ kgm/m'

b = 1,00 m' h_t = 150 cm' h = 145 cm' $\sigma_b/\sigma = 34^5/1400$.

A = 48,8 cm2/m': WAP: Ø 25-10 = 49 cm2/m'

Verdeelwapening = 1/5.48,8 = 9,76 cm2/m'

WAP: Ø 14-15 = 10,2 cm2/m'

STEUNPUNTSMOMENT

$M_{reken} = 1/12.21600.5,80^2 = 60500$ kgm/m'

h = 1,00 m' h_t = 150 cm h = 145 cm σ_b/σ_A = –/1400

A = 32,2 cm2/m': WAP: Ø 25 + Ø 16-20 = 34,5 cm2/m'

Verdeelwapening = 1/5 x 32,2 = 6,44 cm2/m'

WAP: Ø 14-20 = 7,7 cm2/m'

DWARSKRACHT

Diameter = 5,80 m'
Omtrek = π 5,80 = 18,25 m'
Lengte afschuifvlak = 18,25 - 2 x 3 = 12,25 m'

$$\tau = \frac{3}{2} \times \frac{\frac{\pi}{4}.D^2.q}{(\pi D-6)\cdot 150} = \frac{3}{2} \times \frac{\frac{\pi}{4} \times 5,80^2 \times 21600}{1225 \times 150} = 4,67 \text{ kg/cm2} < 7$$

Geen opgebogen wapening vereist.

PONSSPANNING IN TOTALE PROP

$P_{verticaal}$: uit betoncilinder: $\frac{\pi}{4} \times 5,80^2.13,50.2,4 =$ 850 t

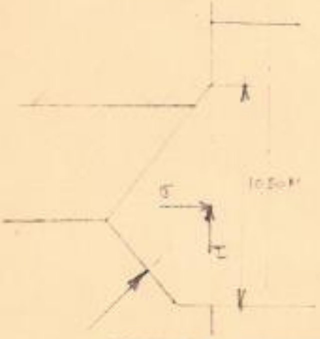
uit vulstenen 19300 t

$P_v =$ 20150 t

Lengte afschuifvlak = Π.D-2 x 3 = 18,25-6 = 12,25 m'
Hoogte afschuifvlak = 10,50 m'

τ pons = σD = $\frac{20+50000}{1225x1050}$ = 15,7 kg/cm2

HOOFDSPANNINGEN IN PROP



Hoofdtrekspanning: $\sigma_1 = \frac{\sigma}{2} + \sqrt{\frac{\sigma^2}{4} + \tau^2}$

$\sigma_1 = -7,85 + \sqrt{\frac{15,7^2}{4} + 15,7^2} = -7,85 + 7,85.\sqrt{5}$

$\sigma_1 = +7,85.1,24 =$ 9,75 kg/cm2

Hoofddrukspanning: $\sigma_1 = + \frac{\sigma}{2} - \sqrt{\frac{\sigma^2}{4} + \tau^2}$

$\sigma_1 = -7,85 - 7,85\sqrt{5} = -7,85.3,24 =$ -25,4 kg/cm

WAPENING CILINDER

Minimum wapening volgens V.V.A.A. = 0,3%

A = 0,3 . $\frac{\pi}{4}$. $\frac{5,80^2}{100}$ = 792 cm2

WAP: 162 Ø 25 (Ø 25-11) = 795 cm2

òf 99 Ø 32 (Ø 32-18) = 797 cm2

BEREKENING WANDBEKISTING

Gerekend wordt met één trek per kwartier per trek wordt
5 m3 gestort.
Na 4 uur begint de beton op te stijven, zodat de zijwaartse
druk vier uur lang toeneemt en daarna constant blijft.

In 4 uur wordt gestort 4 x 5 x 4 = 80 m3.

Gemiddelde breedte van de betonprop
= $\frac{16.00 + 8,60}{2}$ = $\frac{24,60}{2}$ = 12,30 m'

De storthoogte behorende bij 80 m3 is dan:

h = $\frac{80}{12,30 \times 5,00}$ = 1,30 m'

De zijwaartse druk wordt dus 1,30 x 2400 = 3100 kg/m2

De rails worden verticaal geplaatst en op 4 plaatsen ondersteund door horizontale moerbalken, elk veld 2,50 m' lang.

$M_{max.} = 1/10.q\ell^2 = 1/10.3100.2{,}50^2 = 1940$ kgm/m'

$W_{vereist} = \frac{194000}{700} = 276$ cm3/m' << 880.

$W_{aanwezig} = 880$ cm3/m'

Moerbalken

$q = 2{,}50 \times 3100 = 7750$ kg/m' $\ell = 5{,}50$ m'

$M_{max.} = 1/8.7750.5{,}50^2 = 29300$ kgm.

$W_{vereist} = \frac{2930000}{1400} = 2100$ cm3

Kies minimaal DIN 36 ($W_u = 2400$ cm3
$I_x = 43190$ cm4)

Doorbuiging:

$f_{optredend} = \frac{5}{384} \cdot \frac{q\ell^4}{E.I} = \frac{5}{384} \times \frac{77{,}5.5^4.10^8}{2{,}1.10^6.43190} =$ 97 cm'

Oplegreactie: $R = \frac{5 \times 7750}{2} = 19300$ kg

Oplegdruk op betonnen wand $= \frac{19300}{30x30} = 21{,}5$ kg/cm2

Fig. 114: Static calculation, shaft barrier, shaft I Maurits /48/

Furthermore static calculations of the shaft cover are existent /48/.

The coordinates of the shaft are:

RD-x:	184956
RD-y:	331506
elevation:	+70 m NAP
positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space on the side of the industrial complex Chemelot northwards of the company railway.

7.12 Shaft II, Maurits

The vertical Shaft II of the state mine Maurits was drilled in 1918. In 1968 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section with an inside diameter of 5,8 m. The shaft was drilled to a total depth of 810,0 m and was used as drafting shaft, travelling shaft and drawing shaft /48/. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 302 m respectively the carbon surface is located on -230 m NAP /6/. The strata of the overburden is pictured in figure 113. The shaft II Maurits has 10 documented insets. The 391 m floor, as the topmost is located in a level of -319,0 m NAP and in a depth of 391 m /6//50/.

In 1968 a load bearing filling out of 691 m³ of a mixture of concrete was embedded in the 391 m floor /48/. In the first on the level of the 391 m floor an abutment of iron beams covered with a concrete board (325 kg Portland A-cement pro m³), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling (240 kg respectively 275 kg blast furnace cement A pro m³) and the backfilled loose material is spread best. Furthermore above the filling a protective layer of gravel was inserted /8/. 1968 the shaft column was backfilled above the barrier with a total of 13.320 m³ waste material /7/ /8/ /48/. 1969 the shaft was provided with a reinforced concrete cover (thickness 0,7 m) with an opening for refilling /9/ /48/. In 1970/1971 the shaft column subsided 1,46 m, thereof only 0,45 m in 1971 /10//11/. In 1972 the shaft column subsided another 0,03 m. Up to that date the subsidence overall measured 14,05 m /12/. In 1973 there was no further subsidence /13/. In 1974 the shaft surrounding fore-shaft (depth of 20 m) was backfilled with sand /14/. In 1976 anew subsidence required a back stowing with additionally 98 m³ of sand and 145 m³ of water /16/. In 1981 the opening for refilling was backfilled with a mixture of concrete /49/.

Static calculations of the shaft barrier are existent /48/.

The coordinates of the shaft are:

RD-x:	184881
RD-y:	331478
elevation:	+72 m NAP
Positional accuracy:	+/- 1 m

According to the coordinates the shaft is located in an open space on the side of the industrial complex Chemelot northwards of the company railway.

7.13 Shaft III, Maurits

The vertical Shaft III of the state mine Maurits was drilled in 1955. In 1968 this shaft was backfilled and closed. According to documents available the shaft has a round cross-section with an inside diameter of 6,7 m. The shaft was drilled to a total depth of 894,0 m and was used as drafting shaft /48/. Within the overburden the shaft consists of tubbing support. There are no details available about any shaft fittings.

In this area the overburden has a thickness of 301 m respectively the carbon surface is located on -230 m NAP /6/. The strata of the overburden within the range of the shaft III Maurits is shown below /68/.

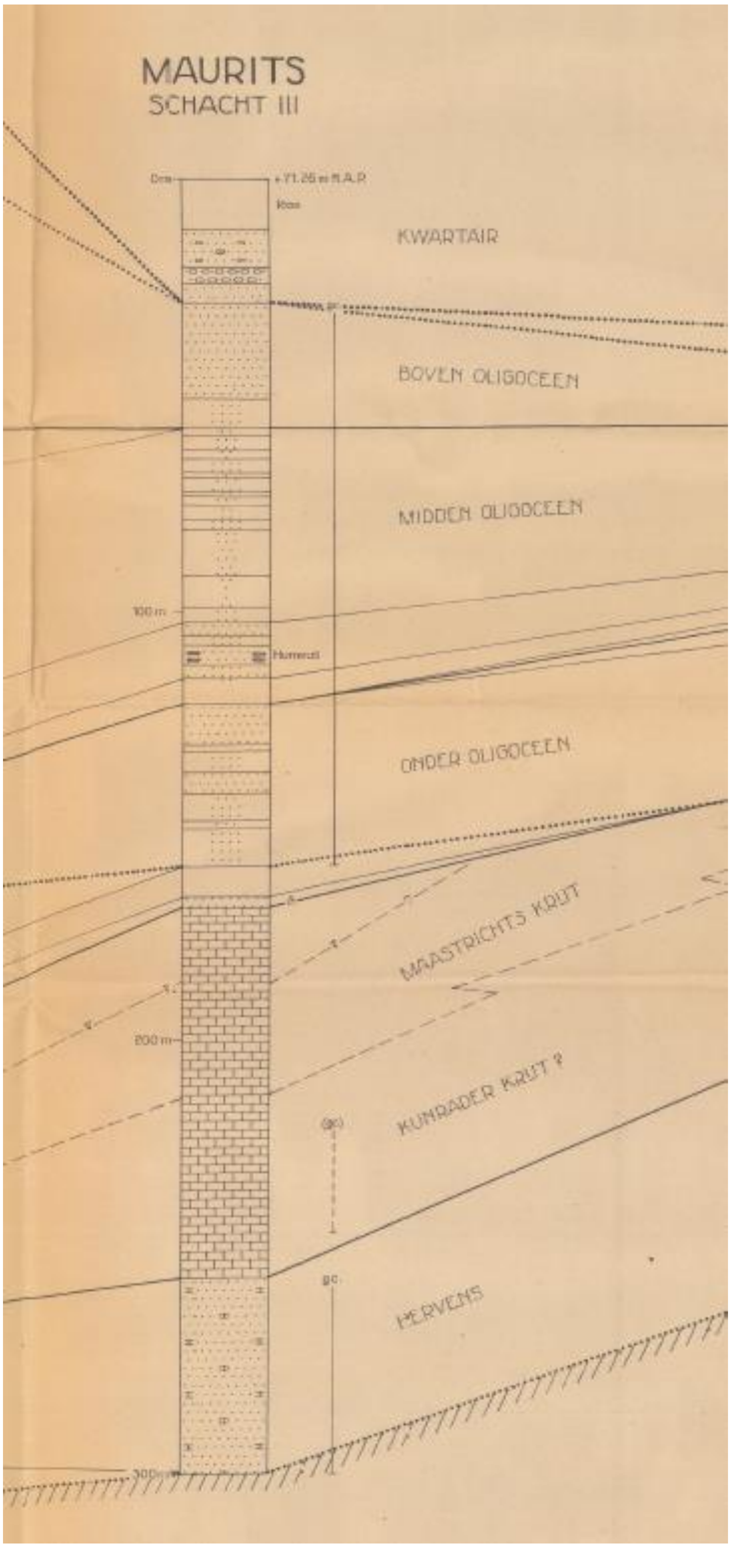


Fig. 115: Strata of the overburden, shaft III Maurits /68/

The shaft III Maurits has 10 documented insets. The 391 m floor, as the topmost is located in a level of -319,0 m NAP and in a depth of 390 m /6//50/.

In 1968 a load bearing filling (length 15,5 m) out of 939 m³ of a mixture of concrete was embedded in the 391 m floor /48/. In the first on the level of the 391 m floor an abutment of iron beams covered with a concrete board (325 kg Portland A-cement pro m³), which rests with its bend lower edge upon the surrounding rock was installed. By this mean the pressure occurring from the load bearing filling (240 kg respectively 275 kg blast furnace cement A pro m³) and the backfilled loose material is spread best. Furthermore above the filling a protective layer of gravel was inserted /8/. 1968 the shaft column was backfilled above the barrier with a total of 18.040 m³ waste material /7/ /8/ /48/. 1969 the shaft was provided with a reinforced concrete cover (thickness 0,85 m) with an opening for refilling /9/ /48/. In 1970/1971 the shaft column subsided 0,71 m, thereof only 0,32 m in 1971 /10//11/. In 1972 the shaft column subsided another 0,18 m. Up to that date the subsidence overall measured 3,72 m /12/. In 1973 the shaft column subsided another 0,05 m /13/. In 1976 anew subsidence required a back stowing with additionally 105 m³ of sand and 226 m³ of water /16/. In 1981 the opening for refilling was backfilled with a mixture of concrete /49/.

In the following figure the shaft barrier of shaft III Maurits is shown.

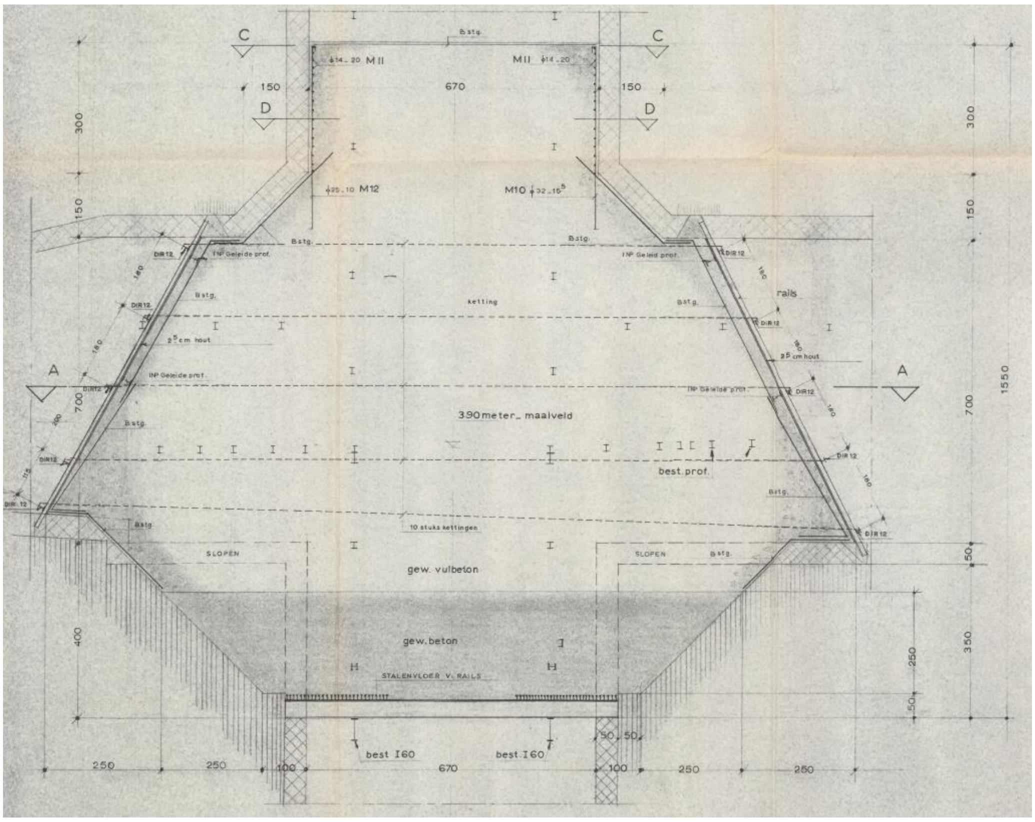


Fig. 116: Shaft barrier, shaft III Maurits /48/

Static calculations of the shaft barrier are existent /48/. Compare the following figures.

NV NEDERLANDSE STAATSMIJNEN

Heerlen, 13-6-1967
Nr. 1517 AB

Statische berekening van de betonprop in schacht III
Stm. Maurits op de 391 m' verdieping

Betonprop

Bepaling volume beton

Cilinder deel A Ø 6,70 m', h = 3,00 m'.

Volume: $\frac{\pi}{4}$. $6{,}70^2$. 3,00 = 106 m3
Deel B: ½ (6,70 + 9,70) . 1,50 . 6,00 = 74 m3
Deel C: ½ (11,70 + 18,70) . 7,00 . 6,00 = 638 m3
Deel D: ½ (16,70 + 8,70) . 4,00 . 6,00 = 305 m3
V totaal = 1.123 m3

Gewicht betonprop totaal = 1123 x 2,4 = 2700 ton.
Vulstenen: h = 397 - 15,50 = 381,50 m', s.g. = 1,9 t/m3.-
Gewicht vulstenen: $\frac{\pi}{4}$. $6{,}70^2$. 381,50 . 1,9 = 25.500 ton.
(13450 m3) totaal = 28.200 ton
Ontbonden onder hoek van 45° geeft:
R = ½ $\sqrt{2}$. 28.200 = 20.000 ton

Opmerking: gerekend is op het gewicht van de totale zuil vulstenen. Dus "silowerking" is verwaarloosd.
Oplegvlak: breedte = 6,00 m'
lengte reken = 7,50 m'
oppervlakte = 6 x 7,5 = 45 m2.
Oplegdruk: $\frac{20.000}{45}$ = 445 t/m2 = 44,5 kg/cm2.

Als schacht is gevuld met water
Gewicht betonprop = 1123 x 1,4 = 1575 ton
Gewicht vulstenen = $\frac{\pi}{4}$. $6{,}70^2$. 381,50 . 0,9 = 12100 ton
totaal = 13675 ton

R = ½ $\sqrt{2}$. 13675 = 9650 ton.

Oplegdruk: $\frac{9650}{45}$ = 215 t/m2 = 21,5 kg/cm2

A. Stalen vloer op 396-P

Deze stalen vloer dient als bekistingsvloer voor de 2,50 m' dikke betonplaat.

Eigen gewicht betonvloer : 2,50 x 2400 = 6000 kg/cm2
Eigen gewicht stalen vloer reken 300 kg/cm2

g totaal = 6300 kg/cm2

Rails vormen dek h = 110 mm', b = 90 mm'

per m' aanwezig 11 stuks.

W aanwezig= 11 x 0,06 . 11^3 = 880 cm3/m' $\bar{\sigma}$ = 700 kg/m'
Moerbalken h.o.h. 1,90 m'

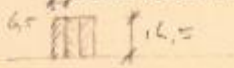
M max.= 1/10 . 6300 . $1{,}90^2$ = 2270 kgm/m'.

W vereist= $\frac{227000}{700}$ = 325 cm3/m' < 880 cm3/m'.

Houten balken vormen dek $\bar{\sigma}$ = 70 kg/cm2.

W vereist= $\frac{227000}{70}$ = 3250 cm3/m'.

Kies $6^5/16^5$



W aanwezig = $\frac{100}{6{,}5}$. 1/6 . 6,5 . $16{,}5^2$ = 4530 cm3/m'
> 3250 cm3/m'.

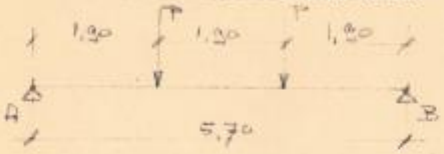
Moerbalken opgelegd op aanwezige schachtbalken DIN 60.
Moerbalken h.o.h. 1,90 m', l = 4,50 m'.
q = 1,90 . 6300 = 12000 kg/m'.
M max.= 1/8 . 12000 . $4{,}50^2$ = 30400 kgm.

W vereist = $\frac{3040000}{1400}$ = 2170 cm3.

Kies DIN 34 W aanwezig = 2160 cm3

Oplegkracht op DIN 60 = $\frac{1{,}20 + 4{,}50}{2}$. 12000 = 34200 kg.

Controle aanwezige DIN 60



l = 5,70 m'
$R_A = R_B$ = 34200 kg
M_{mAX} = 34200 . 1,90 = 65000 kgm
W vereist = $\frac{6500000}{1400}$ = 4650 cm3
W aanwezig = 5700 cm3 > 4650 cm3.

Concl. DIN 60 voldoet.

B. <u>Betonplaat</u> d = 250 cm'

De betonplaat moet het gewicht van de betonprop dragen. Voor de hoogte van de betonprop inclusief de eigen dikte van de betonplaat is te rekenen 10 m'. Dus q = 10,00x2400 + 24000 kg/m2.

q over strook 6,00 m breed = $\frac{6,70}{6,00}$. 24000 = 26500 kg/m2.

<u>Veldmoment</u>

M max. = 1/8 . 26500 . $7,20^2$ = 172000 kgm/m'.

b = 1,00 m' h_t = 250 cm' h = 245 cm' σ_b / σ_a = -/1400

A = 54,5 cm2/m' <u>Wap:[redacted] Ø 32 - 15 = 54 cm3/m'</u>.

Verdeelwapening = 1/5 x 54,5 = 10,9 cm2/m'
<u>[redacted] Ø 19 - 25 = 11,3 cm2/m'</u>

<u>Steunpuntsmoment</u>

M reken = 1/12 . 26500 . $7,20^2$ = 115000 kgm/m'.

b = 1,00 m' h_t = 250 cm' h = 245 cm' σ_b / σ_a = -/1400

A = 36 cm2/m' <u>Wap:[redacted] Ø 32 + Ø 19 - 30 = 36,5 cm2/m'</u>

Verdeelwapening: 1/5 . 36 = 7,2 cm2/m'
<u>Ø 16 - 25 = 8 cm2/m'</u>

<u>Dwarskracht in plaat</u>

Reken b = 2 x 7 = 14 m'.
h_t = 2,50 m'

D = gewicht betoncilinder = $\frac{\pi}{4}$. $6,70^2$. 2,4 . 15,50 = 1300 ton

$\rho = \frac{3}{2} \times \frac{1300000}{1400.250}$ = 3,7 kg/cm2 < 7

Geen opgebogen wapening vereist.

<u>Ponsspanning</u>

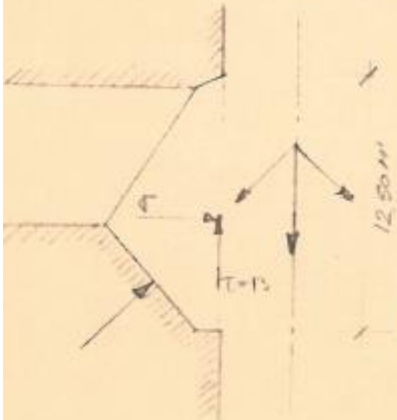
P verticaal uit beton cilinder $\frac{\pi}{4}$. $6,70^2$. 15,50 . 2,4 = 1300 ton

uit vulstenen <u>25500 ton</u>

P_v = 26800 ton

Lengte afschuifvlak $\pi . D - 4{,}50 = \pi . 6{,}70 - 4{,}50 =$
$21{,}00 - 4{,}50 = 16{,}50$ m'.
Hoogte afschuifvlak = 12,50 m'.

τ pons $= \sigma = \frac{26800}{16{,}50 \times 12{,}50} = 130$ t/m2 = <u>13 kg/cm2</u>



<u>Hoofdspanningen in prop</u>

Hoofdtrekspanning =

$$\sigma_1 = \frac{\sigma}{2} + \sqrt{\frac{\sigma^2}{4} + \tau^2} =$$

$$\sigma_1 = -6{,}5 + \sqrt{\frac{13^2}{4} + 13^2} =$$

$$-6{,}5 + 6{,}5\sqrt{5} = 6{,}5 \;.\; 1{,}24 = + \underline{8{,}10 \text{ kg/cm2}}$$

Hoofddrukspanning =

$$-6{,}5 - 6{,}5\sqrt{5} = -6{,}5 \;.\; 3{,}24 = - \underline{21 \text{ kg/cm2}}$$

<u>Wapening cilinder</u>

Minimum wapening volgens V.V.A.A. = 0,3%.

$$A = 0{,}3 \;.\; \frac{\pi}{4} \;.\; \frac{6{,}70^2}{100} = 1060 \text{ cm2.}$$

<u>Wap.</u>

<u>Ø 32 - 15^5 (132 stuks Ø 32) = 1065 cm2)</u>

<u>Berekening wandbekisting</u>

Gerekend wordt met één trek per kwartier, per trek wordt 5 m3 gestort.
Na 4 uur begint de beton op te stijven, zodat de zijwaartse druk vier uur lang toeneemt en daarna constant blijft.
In 4 uur wordt gestort 4 x 5 x 4 = 80 m3.
Gemiddelde breedte van de betonprop =

$$\frac{18{,}70 + 11{,}70}{2} = \frac{30{,}40}{2} = 15{,}20 \text{ m'.}$$

De storthoogte behorende bij 80 m3 is dan:

$$h = \frac{80}{15{,}20 \times 6{,}00} \quad 0{,}90 \text{ m'.}$$

De zijwaartse druk wordt dus 0,90 x 2400 = 2160 kg/m2.
De rails worden verticaal geplaatst en op 4 plaatsen ondersteund door horizontale moerbalken, elk veld 2,30 m' lang.

$$M \text{ max.} = 1/10 + ql^2 = 1/10 \;.\; 2160 \;.\; 2{,}30^2 = 1140 \text{ kgm/m'.}$$

$$W \text{ vereist} = \frac{114000}{700} = 163 \text{ cm3/m'} \ll 880$$

W aanwezig = 880 cm3/m'.

Moerbalken

$q = 2{,}30 \times 2160 \approx 5000$ kg/m' $l = 6{,}50$ m'

M max. $= 1/8 \;.\; 5000 \;.\; 6{,}50^2 = 26400$ kgm.

W vereist $= \frac{2640000}{1400} = 1885$ cm3.

Kies minimaal DIN 34 $\left\{ \begin{array}{l} W_x = 2160 \text{ cm3} \\ I_x = 36660 \text{ cm4} \end{array} \right.$

Doorbuiging:

f optredend $= \frac{5}{384} \cdot \frac{ql^4}{EI} = \frac{5}{384} \times \frac{50 \;.\; 6{,}5^4 \;.\; 10^8}{2{,}1 \;.\; 10^6 \;.\; 36660} = 1{,}5$ cm'

Oplegreactie : $R = \frac{6 \times 5000}{2} = 15000$ kg.

Oplegdruk op betonnen wand $= \frac{15000}{30 \times 30} = 16{,}7$ kg/cm2.

Fig. 117: Static calculation, shaft barrier, shaft III Maurits /48/

Furthermore static calculation of the shaft cover are existent /48/.

The coordinates of the shaft are:

RD-x:	184788
RD-y:	331443
elevation:	+71 m NAP
positional accuracy :	+/- 1 m

According to the coordinates the shaft is located in an open space on the side of the industrial complex Chemelot northwards of the company railway.

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Appendix 5

Na-ijlende gevolgen steenkolenwinning Zuid-Limburg

Final report on the results of the working groups
5.2.2 - risks from mine shafts
5.2.3 - risks from near-surface mining

Sampled data of industrial shafts

by

Projectgroup
"Na-ijlende gevolgen van de steenkolenwinning in Zuid-Limburg"
(projectgroup GS-ZL)

on behalf of
Ministerie van Economische Zaken - The Netherlands

Aachen/Essen, 31. August 2016
(Rev. a: 02. December 2016)

No.	Mine shaft	Concession	Easting (RD new) [m]	Northing (RD new) [m]	Ground surface level [mNAP]	Closing date	Overburden thickness [m]	Bedrock surface level [mNAP]	Shaft depth [m]	Sump [mNAP]	Number of floors	Depth of topmost floor [m bgl]	Level of topmost floor [mNAP]	Height difference between topmost floor and bedrock surface level [m]	Shaft lining in the overburden	Shaft dimension / diameter [m]	Cross-sectional area in the overburden [m^2]
1	Buizenschacht	Domaniale	203493	319045	167	1969	42	125	499	-332	8	46	121	4	brickwork	1,75 x 1,25	3
2	Willem I	Domaniale	203502	319058	167	1969	42	125	393	-226	10	45	122	3	brickwork	4,30 x 2,60	12
3	Willem II	Domaniale	203529	319037	168	1970	43	125	804	-636	10	46	122	3	brickwork	8,30 x 3,70	24
4	Beerenbosch I	Domaniale	203503	320588	147	1969	48	99	482	-335	7	53	94	5	tubbing	2,65	4
5	Beerenbosch II	Domaniale	203517	320662	147	1994	47	100	502	-355	7	53	94	6	concrete	5,30 x 3,80	20
6	Nulland	Domaniale	202776	319031	156	1970	41	115	347	-191	6	63	93	22	brickwork	3,50	10
7	Baamstraat	Domaniale	202140	318840	133	1967/1978	14	119	21	112	1	21	105	7	concrete	2,40	4
8	Neuland	Domaniale	203101	318915	164	1920	40	124	190	-26	4	63	101	23	brickwork	1,60 x 1,60 dual cylinders	4
9	Louise	Domaniale	203226	319328	162	1907	40	122	242	-80	-	-	-	-	brickwork	4,00 x 3,30	13
10	Catharina	Neu Prick	203033	318726	168	1904	41	127	266	-98	2	210	-42	169	brickwork	2,00 x 3,00	6
11	Willem I	Willem Sophia	200384	318635	158	1970	61	97	590	-432	6	181	-23	120	tubbing	3,50	8
12	Willem II	Willem Sophia	200373	318668	158	1970	61	97	651	-493	8	106	52	45	tubbing	3,60	8
13	Sophia	Willem Sophia	199145	317044	176	1970	126	50	328	-152	5	148	28	2	brickwork	4,50	15
14	HAM II	Willem Sophia	201746	319249	129	1970	21	108	74	55	1	74	55	53	brickwork	4,80	18
15	Melanie	Willem Sophia	200515	318178	153	1970	66	87	230	-77	2	100	53	34	concrete	3,00	7
16	Laura I	Laura-Julia	201611	322793	116	1969	99	17	730	-614	9	119	-3	20	tubbing	4,50	16
17	Laura II	Laura-Julia	201680	322822	116	1970	100	16	401	-285	5	122	-6	22	tubbing	4,50	16
18	Julia I	Laura-Julia	202781	323110	103	1975	216	-113	547	-444	4	304	-201	88	tubbing	5,50	23
19	Julia II	Laura-Julia	202875	323143	103	1975	213	-110	568	-465	4	304	-201	91	tubbing	5,50	23
20	Shaft I	Oranje Nassau I	196055	322643	109	1975	96	13	255	-146	5	135	-12	25	tubbing	3,00	7
21	Shaft II	Oranje Nassau I	196019	322661	109	1975	96	13	470	-361	7	135	-12	25	tubbing	3,50	7
22	Shaft III	Oranje Nassau I	195874	322783	108	1975	96	12	441	-333	5	135	-27	39	tubbing	3,80	12
23	Shaft I	Oranje Nassau II	199322	321717	152	1971	132	20	477	-325	9	162	-10	30	tubbing	4,00	9
24	Shaft II	Oranje Nassau II	199315	321677	152	1971	131	21	433	-281	7	162	-10	31	tubbing	5,40	9
25	Shaft	Oranje Nassau III	194845	324962	94	1973	149	-55	844	-750	6	228	-134	79	tubbing	7,20	27
26	Shaft	Oranje Nassau IV	196912	324846	109	1973	189	-80	740	-631	4	240	-131	51	tubbing	5,20	16
27	Shaft I	Wilhelmina	199802	320412	157	1970	97	60	822	-665	7	163	-6	66	tubbing	4,50	16
28	Shaft II	Wilhelmina	199863	320378	157	1970	97	60	537	-380	7	163	-6	66	tubbing	4,50	16
29	Shaft I	Emma	193855	326853	106	1974	198	-92	900	-794	6	259	-153	61	tubbing	6,00	25
30	Shaft II	Emma	193889	326800	105	1974	200	-95	570	-465	4	258	-153	58	tubbing	4,50	16
31	Shaft III	Emma	193704	326791	105	1974	203	-98	980	-875	6	258	-153	55	tubbing	6,00	27
32	Shaft IV	Emma	188473	328112	66	1971	215	-149	653	-587	3	266	-200	51	steel/concrete	4,50	16
33	Shaft I	Hendrik	196480	327759	97	1967	222	-125	902	-805	7	272	-175	50	tubbing	6,00	26
34	Shaft II	Hendrik	196543	327791	97	1968	223	-126	855	-758	7	272	-175	49	tubbing	4,00	13
35	Shaft III	Hendrik	199096	325391	160	1968	199	-39	454	-294	3	245	-85	46	tubbing	5,40	22
36	Shaft IV	Hendrik	196577	327721	96	1969	221	-125	1.058	-962	8	272	-175	51	tubbing	6,60	35
37	Shaft I	Maurits	184956	331506	70	1968	303	-233	856	-786	5	389	-319	86	tubbing	5,80	26
38	Shaft II	Maurits	184881	331478	72	1969	302	-230	810	-738	5	391	-319	89	tubbing	5,80	26
39	Shaft III	Maurits	184788	331443	71	1969	300	-229	894	-823	5	390	-319	90	tubbing	6,70	35

No.	Mine shaft	Concession	Sealing element[1)]	Plug material[1)]	Installation technique of the plug	Lower edge of the plug [mNAP]	Plug length [m]	Total amount of concrete [m³]	Height difference between top of the plug and bedrock surface level [m]	Ratio plug length:shaft diameter	Verifiable structural analysis available	Length of loose material backfilling column [m]	Total amount of loose material [m³]	Void capacity below the plug [m³]	Mine water level in 2014 [mNAP]	Diameter of shaft-protection-zone [m]
1	Buizenschacht	Domaniale	shear plug IIa	concrete 325 H.A	unknown	121	6	135	-3	2,57	yes	-	-	991	35,66*	89,75 x 89,25
2	Willem I	Domaniale	shear plug IIa	concrete 325 H.A	unknown	121	6	580	-3	0,81	yes	-	-	3.891	35,66*	92,3 x 90,6
3	Willem II	Domaniale	shear plug IIa	concrete 325 H.A	unknown	121	8	1.150	-1	0,84	no	-	-	23.278	35,66*	98,3 x 93,7
4	Beerenbosch I	Domaniale	shear plug IIa	concrete 325 H.A	unknown	94	6	260	-1	2,26	no	-	-	2.366	35,66*	102,65
5	Beerenbosch II	Domaniale	cohesive backfilling	B15, B5, B2	loose dumping	-20	20	3.660	99	-	yes	-	-	9.043	35,66*	103,3 x 101,8
6	Nulland	Domaniale	shear plug IIa	concrete 325 H.A	unknown	93	6	630	16	6,29	no	-	-	2.732	35,66*	89,5
7	Baamstraat	Domaniale	loose material and cover plate	-	loose dumping	-	-	-	-	-	no	21	108	0	35,66*	34,4
8	Neuland	Domaniale	loose materials on arched roofing	concrete / steel beams	unknown	79	0,75	-	45	-	no	83	334	422	35,66*	87,7 x 85,6
9	Louise	Domaniale	shear plug (IIb)	concrete K 300	fall pipe	118	8	70	4	2,00	no	31	309	-	35,66*	88 x 87,3
10	Catharina	Neu Prick	injection grouting of the loose material	-	-	-	-	165	-	-	no	-	-	-	35,66*	88 x 89
11	Willem I	Willem Sophia	shear plug IIb	concrete	fall pipe	-23	13	100	107	3,71	yes	170	1.360	3.935	35,66*	129,5
12	Willem II	Willem Sophia	shear plug IIb	concrete	unknown	52	19	80	27	5,28	yes	100	800	5.547	35,66*	129,6
13	Sophia	Willem Sophia	floor-supported plug	concrete	fall pipe	0	12	350	37	2,67	no	140	2.100	2.417	35,66*	200
14	HAM II	Willem Sophia	shear plug IIb	concrete	unknown	95	33,5	575	-19	6,98	yes	-	-	0	35,66*	50,8
15	Melanie	Willem Sophia	shear plug IIb	concrete	fall pipe	53	25	330	9	8,33	no	-	-	919	35,66*	139
16	Laura I	Laura-Julia	shear plug IIc	K 225	unknown	-12	73,9	1.400	12	16,42	no	295	4.270	5.646	28,36*	200
17	Laura II	Laura-Julia	shear plug IIc	K 225	fall pipe	-12	62	1.200	8	13,78	no	330	5.280	0	28,36*	200
18	Julia I	Laura-Julia	shear plug IIb	concrete	fall pipe	-170	17	420	58	3,09	yes	285	6.750	5.773	11,92**	200
19	Julia II	Laura-Julia	shear plug IIb	concrete	fall pipe	-170	17	420	58	3,09	yes	285	6.750	6.272	11,92**	200
20	Shaft I	Oranje Nassau I	floor-supported plug	concrete	fall pipe	-27	8	130	32	2,67	yes	125	656	848	20,86***	199
21	Shaft II	Oranje Nassau I	floor-supported plug	concrete	fall pipe	-27	8,2	152	30	2,34	no	128	984	3.223	20,86***	199,5
22	Shaft III	Oranje Nassau I	floor-supported plug	concrete 325	fall pipe	-27	10	200	28	2,63	yes	126	1.904	3.470	20,86***	199,8
23	Shaft I	Oranje Nassau II	floor-supported plug	concrete	fall pipe	-10	10	125	22	2,50	no	152	1.925	3.958	28,36*	200
24	Shaft II	Oranje Nassau II	floor-supported plug	concrete	fall pipe	-10	10	230	22	1,85	yes	152	3.500	6.206	28,36*	200
25	Shaft	Oranje Nassau III	floor-supported plug	concrete	fall pipe	-133	12	3.450	67	1,67	yes	215	7.035	25.080	20,86***	200
26	Shaft	Oranje Nassau IV	floor-supported plug	concrete	fall pipe	-130	14	2.000	36	2,69	yes	225	3.235	10.619	20,86***	200
27	Shaft I	Wilhelmina	floor-supported plug	concrete	unknown	-6	8	380	58	1,78	yes	155	3.002	10.481	28,36*	200
28	Shaft II	Wilhelmina	floor-supported plug	concrete	unknown	-6	10,75	760	55	2,39	yes	140	2.340	5.948	28,36*	200
29	Shaft I	Emma	shear plug IIb	concrete	unknown	-153	17,6	511	43	2,93	no	241	7.300	18.124	20,86***	200
30	Shaft II	Emma	shear plug IIb	concrete	unknown	-153	13,4	284	45	2,98	no	245	3.900	4.962	20,86***	200
31	Shaft III	Emma	floor-supported plug	concrete	unknown	-153	17,85	510	37	2,98	no	240	7.984	20.414	20,86***	200
32	Shaft IV	Emma	floor-supported plug	concrete	unknown	-200	18	1.053	33	4,00	no	248	5.136	6.155	20,86***	200
33	Shaft I	Hendrik	floor-supported plug	concrete	unknown	-175	11	615	39	1,83	yes	265	6.890	17.813	20,86***	200
34	Shaft II	Hendrik	floor-supported plug	concrete	unknown	-175	9	280	40	2,25	yes	265	3.445	7.326	20,86***	200
35	Shaft III	Hendrik	shear plug IIc	concrete/demolition material	loose dumping	-85	**36**	1.420	32	6,67	no	333	10.300	4.787	20,86***	200
36	Shaft IV	Hendrik	floor-supported plug	concrete	unknown	-175	13	774	37	1,97	yes	265	9.275	26.891	20,86***	200
37	Shaft I	Maurits	floor-supported plug	concrete 240 H.A	unknown	-319	13,6	704	72	2,34	yes	380	13.500	12.339	?	200
38	Shaft II	Maurits	floor-supported plug	concrete 240 H.A	unknown	-319	13	691	76	2,24	no	380	13.320	11.070	?	200
39	Shaft III	Maurits	floor-supported plug	concrete 240 H.A	unknown	-319	15,5	939	74	2,31	yes	380	18.040	17.769	?	200

36 measured value
248 estimated value

35,66* 02.12.2014
11,92** 05.11.2014
20,86*** 16.11.2014

1) for further details see report and App. 4

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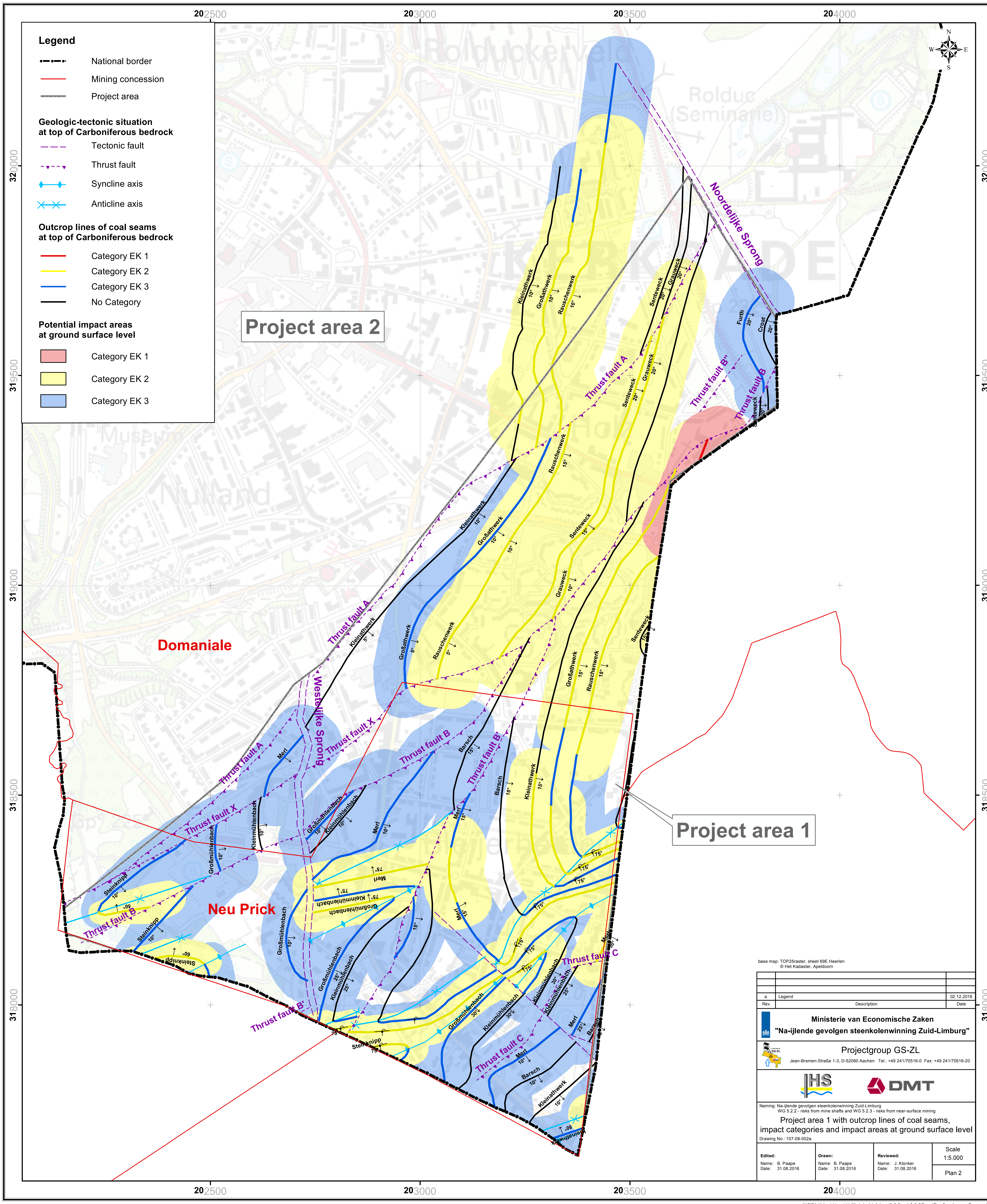

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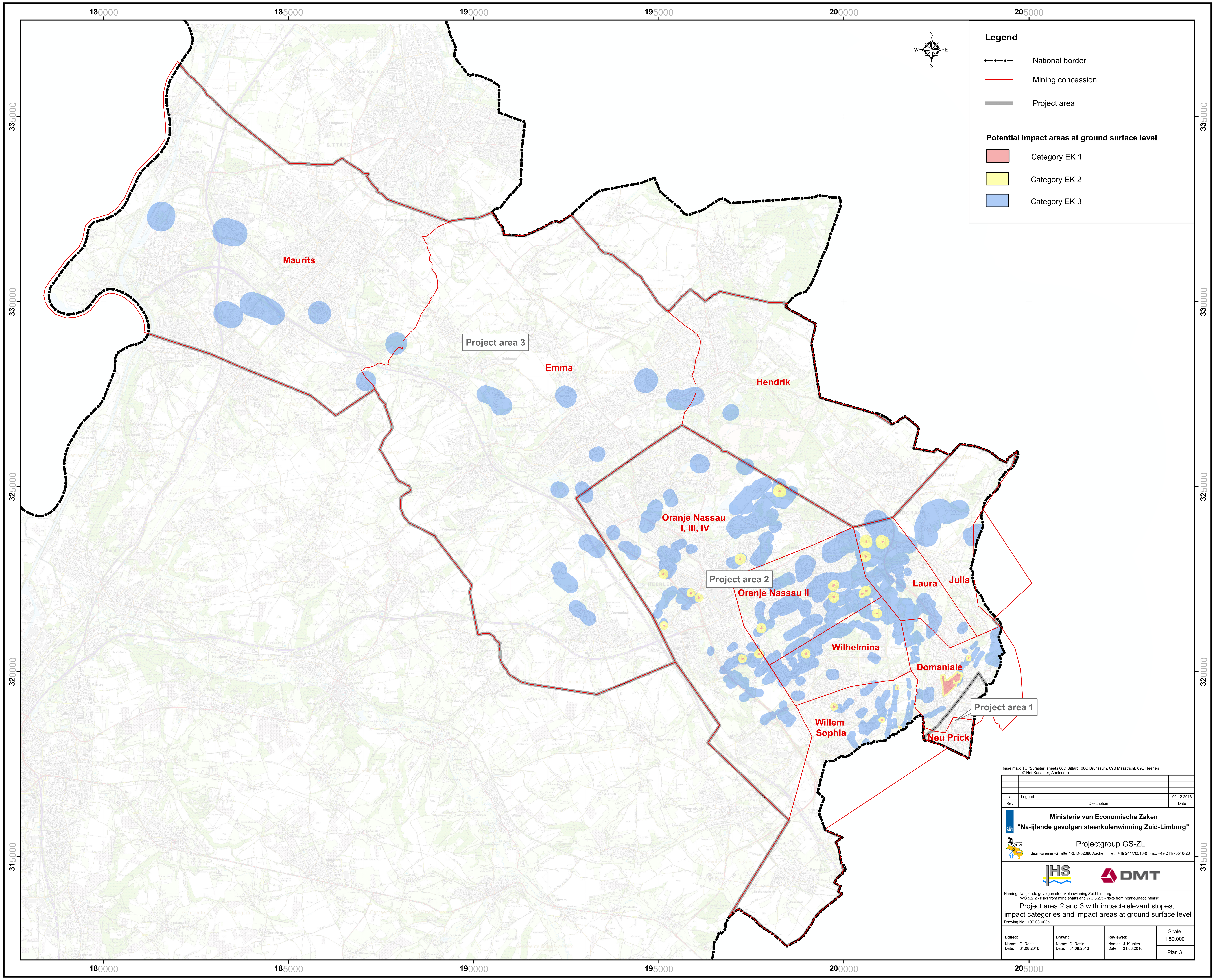

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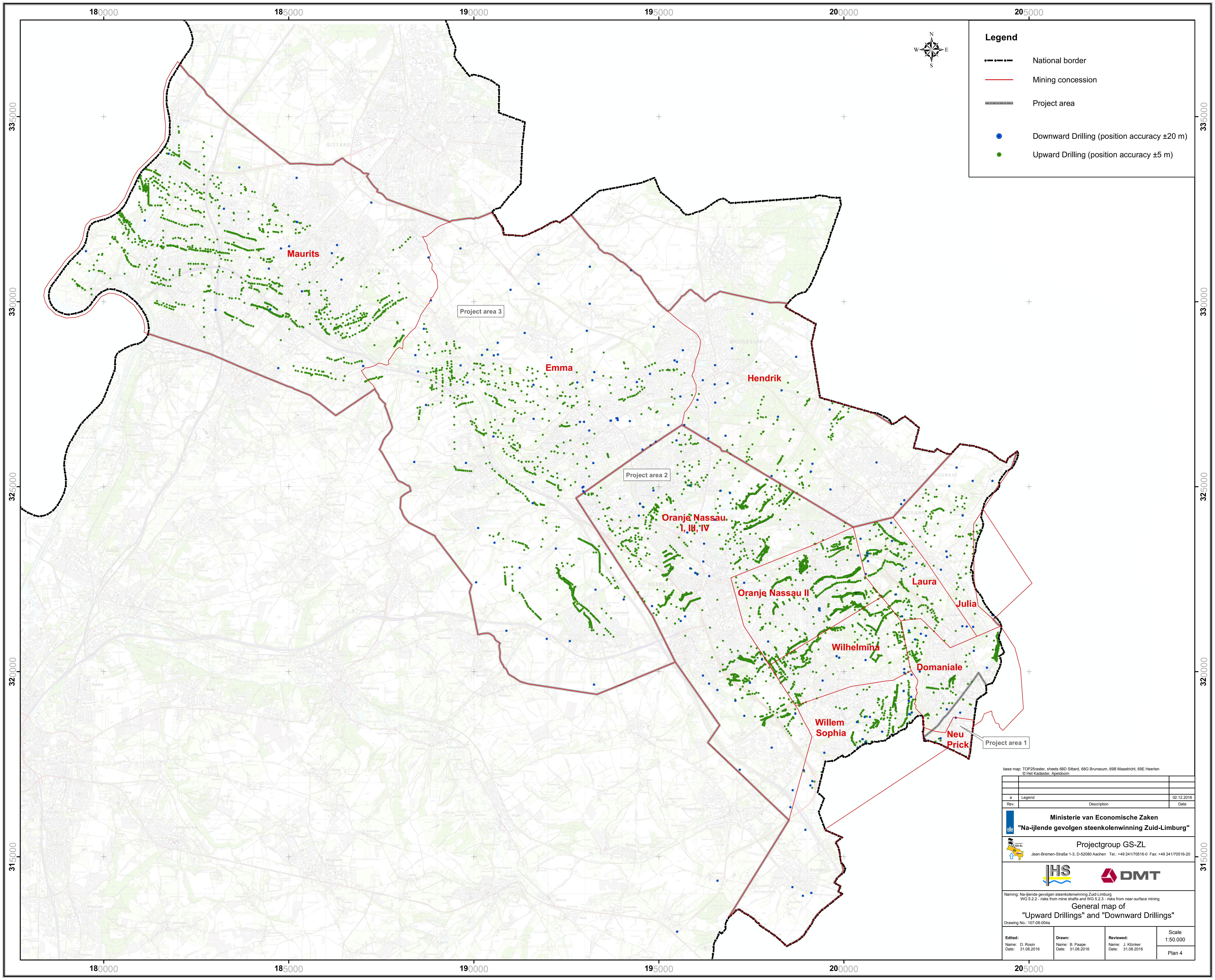

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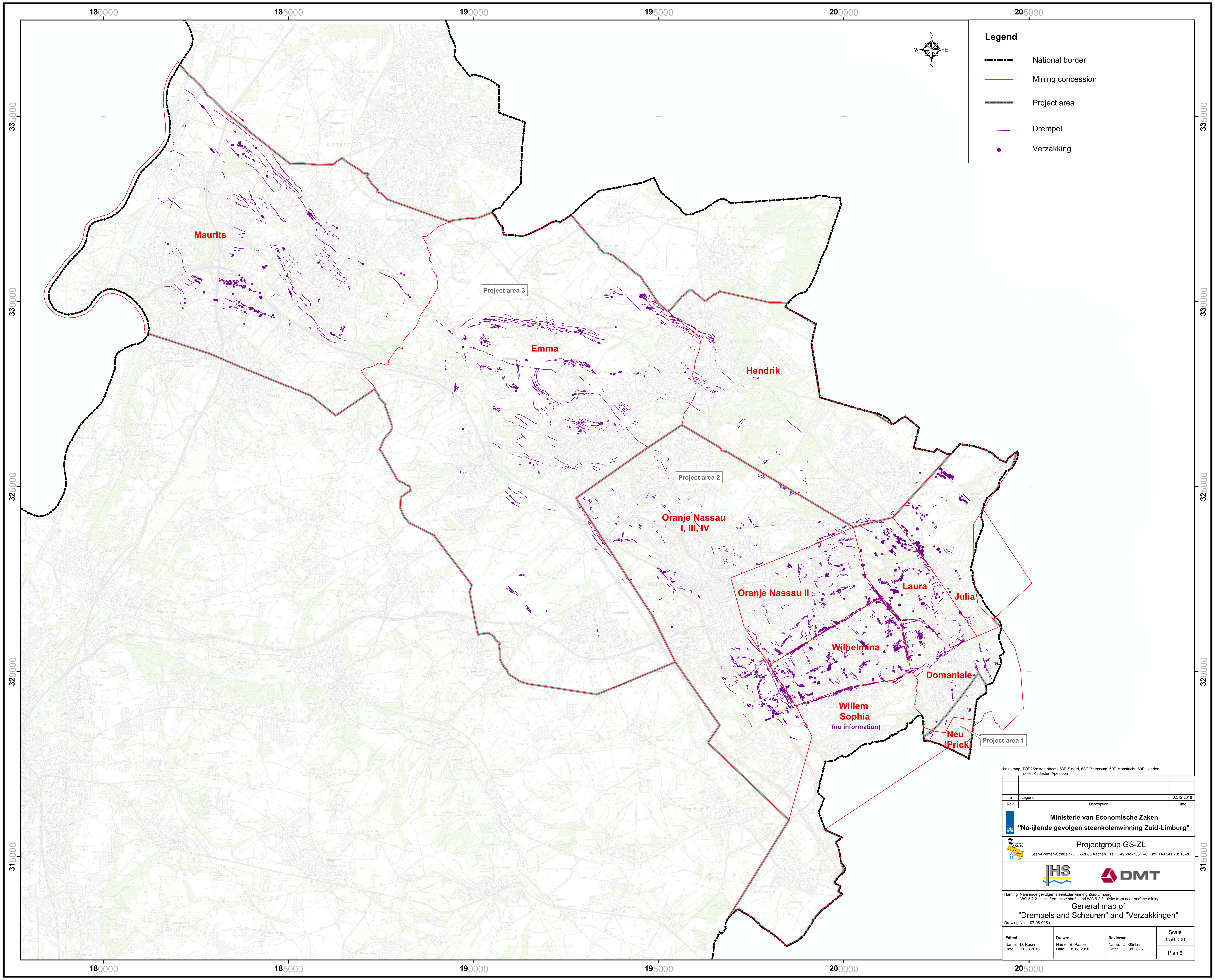

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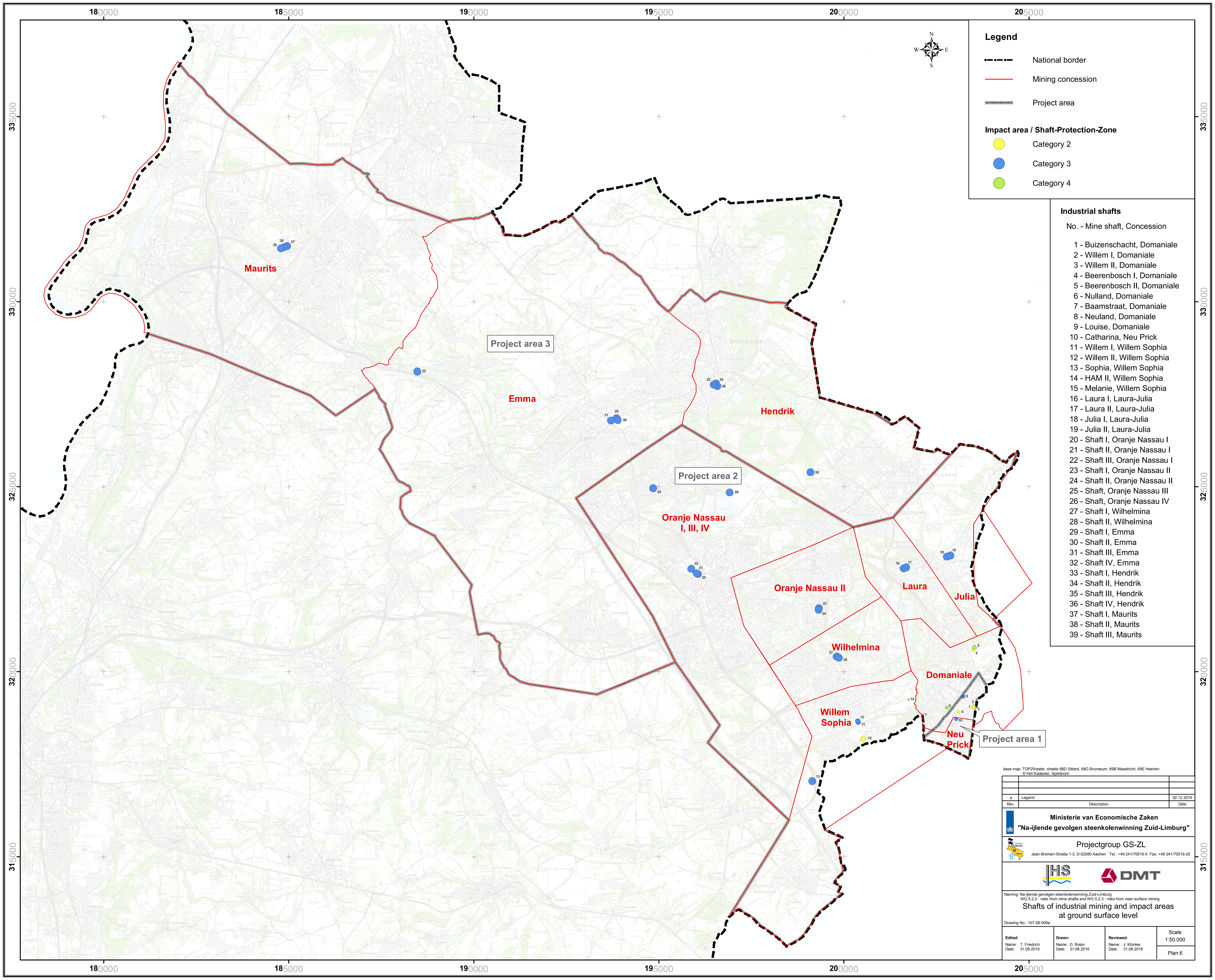

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