Rehabilitation of large profiles lacking long-term stability

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Many large interceptor sewers, predominantly installed under public roads, have been in use for over 100 years. They are made of brickwork or compressed concrete and often show serious damage, so that it would seem that long-term stability can no longer be guaranteed. As an alternative to replacing large profiles, various renovation procedures offer significant advantages with respect to investment costs, environmental protection and urban pollution. Depending on the procedure, it may well be possible to extend the service life, in order to match that of a replacement. However, beforehand, an in-depth study of the current stability and residual load capacity of the existing sewer is required, along with needs-based selection and dimensioning of the rehabilitation procedure. The new standard DWA-A 143-2 serves as a good basis for the necessary static calculations.

1. Specific features of large interceptor sewers

1.1 Materials and profile shapes

In line with the requirements and technical construction possibilities available at the start of the last century, the sewers were often made of brickwork, occasionally using natural stone, but predominantly using man-made bricks (fired solid bricks/clinkers). Depending on the dimensions and loads at the time, the masonry shells are one, two or even three layers thick, while the wall thickness quite often varies within the cross-section. Furthermore, in the early days, sewers were already made from compressed concrete, but they now vary greatly in terms of strength. The concrete was only rarely reinforced and, in most cases, does not comply with our current requirements for reinforced concrete ***(Figure 1)***.

The standard profile shapes with circular or egg-shaped cross-sections prevalent in the smaller sewers are giving way, with the increasing size of the interceptors, to other operationally and structurally more favourable profile shapes. Most large profiles can be classified as either jaw-shaped or hood profiles. However, different versions of the individual cross-section shapes can be found, which can be more precisely described in terms of their attributes – for example, whether they are wide, squashed, stretched or raised. At the time, the designers and structural engineers were aiming to achieve load transfer via one pressure arch because the materials used were very limited in terms of their ability to withstand tensile stresses. This load-bearing behaviour means that the arch must have an appropriate base or footing with sufficient load capacity and subsoil stiffness.

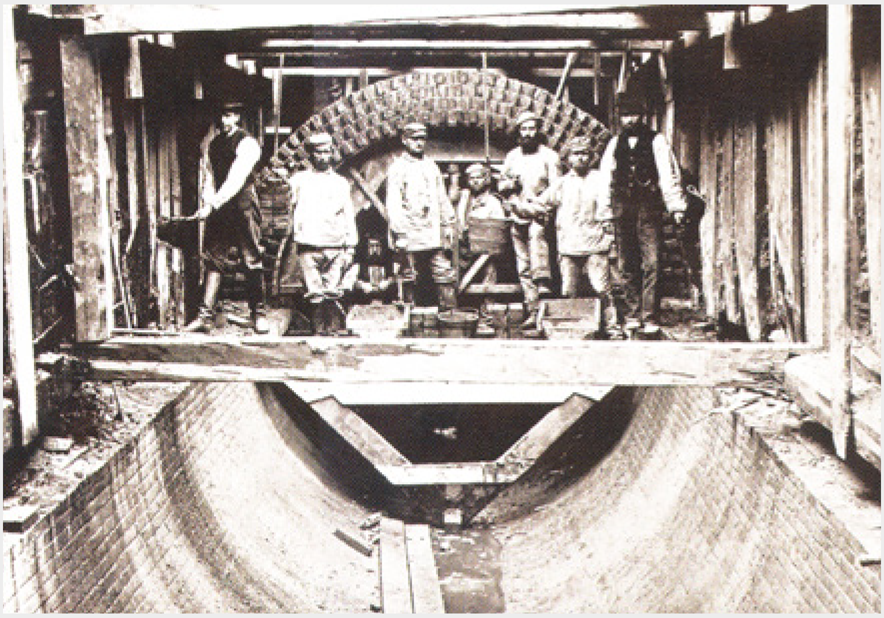

Figure 1: *The Geststammsiel sewer under construction (Hamburg 1872)*
source: Hamburger Stadtentwässerung (Hamburg Urban Drainage)

The invert of the sewer is, in many cross-sections, constructed as a separate arch, which must

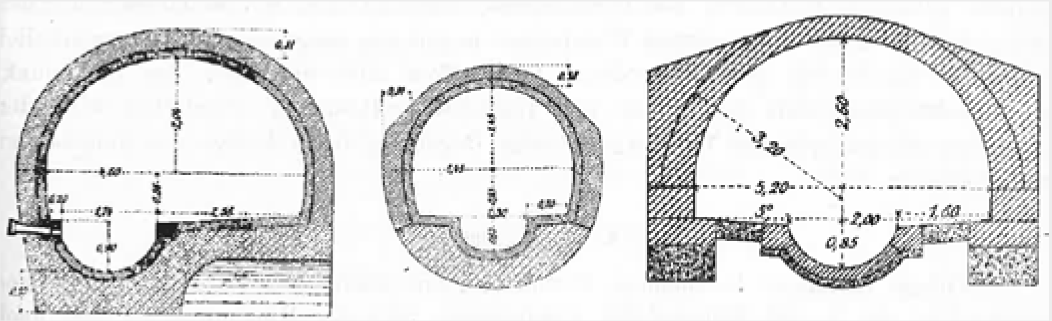

Figure 2: *Historical cross-section drawings of large profiles in Cologne (left, centre) and Stuttgart (right), source: Handbuch der Ingenieurwissenschaften in fünf Teilen [Engineering Handbook in five parts]*

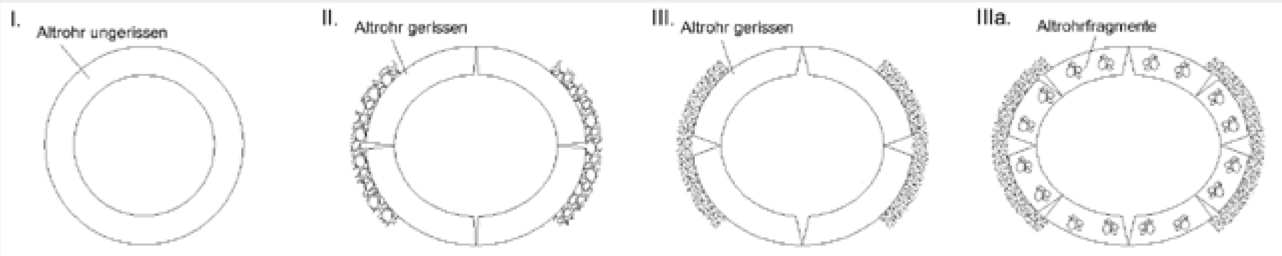

Figure 3: *Host pipe conditions (from I to III): I, II, III and IIIa, source: S&P Consult GmbH*

be viewed as structurally detached from the main arch and (at least above the groundwater level) it does not usually affect the overall stability of the sewer.
Figure 2 shows examples of various cross-section shapes.

1.2 Assessment of condition

1.2.1 Basis for assessing the condition

For large profiles, condition assessment is based closely on the recently published worksheet DWA-A 143-2 (July 2015) [2], which governs the static calculations for the rehabilitation of drains and sewers using lining and assembly methods. Although this set of regulations relates mainly to pipes and ducts with (smaller) circular and egg-shaped cross-sections, the basic method is also adopted for the calculation and dimensioning of large profiles.
For a basic assessment of the stability of the sewers to be rehabilitated (referred to as host pipe in the DWA-A 143-2), three basic "host pipe conditions" are differentiated (***Figure 3***):

» **Host pipe condition I**: The host pipe is able to bear the load alone. In this case, the liner only has to ensure sewer tightness. Since the host pipe alone is able to bear all externals loads, the liner is only stressed by the groundwater that seeps through the untight sewer wall and increases external pressure corresponding to the groundwater level.
» **Host pipe condition II**: The host pipe alone is not able to bear the load and has longitudinal cracks at four points on its circumference (crown, springline and invert). The resulting quarter shells have turned against each other, so that the cross-section has ovalised. The clearance height of the cross-section has been reduced, but the cross-section has also become broader and pushed itself into the soil in the springline area. The soil supports the sewer and a load-bearing system results, consisting of the cracked host pipe and the supporting soil - the "host pipe-soil system". If this host pipe soil system is structurally stable with the necessary assurances, the host pipe is in host pipe condition II. In this case, similar to host pipe condition I, the liner only has to ensure sewer tightness. Since the host pipe alone is able to bear all externals loads, the liner is only stressed by the groundwater that flows through the untight sewer and increases external pressure. In contrast to host pipe condition I, an oval, pre-formed liner may be required. Host pipe-soil system is structurally stable with the necessary assurances and the host pipe is in host pipe condition II.
» **Host pipe condition III**: This host pipe condition corresponds to host pipe condition II with the crucial difference that the stability of the host pipe-soil system is no longer verifiable. In this case, the liner has to bear not only groundwater loads, but must at least partially help to cope with all influences, such as soil, traffic and superimposed loads. In contrast to the host pipe conditions I and II, the influences on the liner in host pipe condition III are generally much greater.

In the DWA-A 143-2, a further host pipe condition was created and is referred to as host pipe condition IIIa, but it was only included in the informative annex of the regula-

***Table 1**: Criteria for classifying a sewer as host pipe condition III based on DWA-A 143-2*

Criteria	Limit value
Joint ring deformation	greater than 6 %
Cover over crown	smaller than the cross-section width
Change in load (e.g. increase in superimposed load)	yes
Cracks are larger	yes
Cavitation in the soil due to infiltration	yes

tions. In contrast to the existing host pipe condition III, it is assumed in this case that the quarter shells between the joints (cracks) of the host pipe do not remain intact and break, thus eliminating any supportive effect of the host pipe. Large profiles which must be classified as host pipe condition IIIa can only be rehabilitated in exceptional cases involving special measures and are therefore not considered further in this report.

Table 1 shows the criteria defined in the DWA-A 143-2 for distinguishing between host pipe conditions II and III. For large profiles, the data in this table can be used at best as an initial guideline. Instead, precise and in particular individual studies of the cross-section are required and are justified because, on the one hand, the risk potential is significantly greater than with small pipes and, on the other hand, there is a huge potential for savings if rehabilitation takes place.

1.2.2 Historic research, surveying and monitoring

The first step in individual condition assessment is to evaluate any existing documentation regarding the cross-section in question. However, experience shows that, in most cases, plans can no longer be found and quite often even the year of construction, material and cross-sectional geometry are completely unknown.

Hence, initial information is not gathered until the entire length of the interceptor is inspected, in order to

» determine the material and
» document any statically relevant damage (cracks, corrosion, exposed reinforcement, etc.).

This is followed by an initial appraisal of the stability, paying particular attention to any imminent danger of collapse, which may call for emergency measures, such as blocking (heavy) traffic.

Figure 4 shows an example of a statically relevant longitudinal crack in a concrete sewer that does not represent a serious risk to the stability of the sewer. Further investigations and, in particular, a structural analysis are needed, in order to determine whether long-term stability can be ensured without structural reinforcement.

In ***Figure 5***, the longitudinal crack in the crown of the masonry is so pronounced that serious risk cannot be excluded. Here immediate measures are required, especially since the invert has already opened up and the entire cross-section is considerably deformed.

As part of the visual inspection, initial examinations can be conducted (e.g. determining the reinforcement content by profometer measurements) or monitoring with plaster or crack marks can be introduced.

1.2.3 Structural inspections

Due to the large number of different cross-section shapes, which also often vary from region to region, condition assessment includes measuring the inner contour, in order to include it with sufficient accuracy in the calculation model.

***Figure 4**: Longitudinal crack in cast-in-place concrete sewer, source: S&P Consult GmbH*

Source: S&P Consult GmbH

***Figure 5**: Significant longitudinal crack in the crown and in the invert, heavily deformed cross-section*

Figure 6: *GRP short pipe lining, source: HOBAS Rohre GmbH*

Figure 7: *Pipe liner, source: Aarsleff GmbH*

The outer contour of the sewer can be ascertained by determining the wall thicknesses, if the historical documents do not help draw adequate conclusions in this respect.
In addition to the geometry, the strength of the materials used in their current condition is crucial in terms of helping to verify the stability of the sewer. In order to determine the material properties, a sufficient number core samples must be retrieved using a core drill with a diameter of 100 mm (for concrete) or 150 mm (for masonry) and subjected to a strength test in the laboratory.

1.2.4 Soil parameters

The stability of large profiles is determined, to a large extent, by the characteristics of the subsoil. Not only must the soil provide a secure (vertical) foundation, especially for sewers that are already cracked, it also plays an important role in the side (horizontal) bedding of the system. The load-bearing structure does not consist exclusively of the sewer wall alone, it also includes the interaction of the masonry or concrete with the surrounding subsoil. For this reason, knowledge of the soil parameters is essential for the structural analysis. In order to obtain reliable values, a soil laboratory or at least a specialist in geotechnical engineering must be consulted.
After field tests and collection of soil samples, the subsoil parameters relevant for the structural analysis are determined in geotechnical laboratory tests. In addition to the specific weight of all the soil layers on top of and next to the sewer, the following two parameters of the soil at springline level can significantly affect the stability of the sewer:

» Friction angle φ' for defining the load capacity of the side bedding
» Stiffness modulus E_s for defining the stiffness of the side bedding.

Any groundwater must be known not only at its highest, but also the lowest level, because it is generally not until the calculation is performed that it becomes clear which groundwater situation is decisive when it comes to dimensioning a liner to be sized for host pipe condition III. The groundwater levels must be ascertained by the subsoil specialist.

1.2.5 Results of the condition assessment for calculation and dimensioning of the liner

The condition assessment described in the previous chapters provides the following information and input values for the structural analysis of the sewer:

» Geometry of the sewer cross-section
» Host pipe condition of the sewer
» Position, width and length of statically relevant cracks
» Other damage (e.g. reduction in wall thickness due to corrosion)
» Material properties of the sewer wall
» Subsoil characteristics

Thus, a static model can be created based on the finite element method. The result is a fairly reliable statement about the stability of the sewer in unrehabilitated condition. After a sensitivity analysis of the individual input parameters, a classification as host pipe condition II or III is undertaken, thus creating the most important basis for the calculation and dimensioning of the liners.

2. Rehabilitation procedure (suitability for large profiles in condition III)

At present, only renovation procedures are used for the structural reinforcement of large, man-accessible interceptors, namely:

» the pipe lining method (up to approx. DN 2000),
» single pipe lining,
» the PVC spiral wound method with filling the annular gap with load-bearing grout and
» the shotcrete method with statically effective reinforcement

2.1 Pipe lining method

Overall (including man-accessible and non-man-accessible types), the pipe lining method currently represents the most

Source: Sekisui GmbH

***Figure 8**: SPR spiral wound process*

Source: S&P Consult GmbH

***Figure 9**: Reinforcement of the shotcrete shell*

Source: S&P Consult GmbH

***Figure 10**: Spraying process*

widely used renovation method. In large-scale profile rehabilitation, however, the nominal diameter upper limit is currently approx. DN 2000. Furthermore, the applicability depending on the profile shape needs to be checked (especially for dry weather channel in the invert).

2.2 Single pipe lining (short pipe lining)

In single pipe lining, factory-made pipes are inserted, through excavations, into the section to be rehabilitated. In large profile rehabilitation, the individual pipes are usually connected to each other in the section. The resulting annular gap is then grouted.

This method can be used for all host pipe conditions and for most cross-section shapes (also e.g. jaw-shaped profile with dry weather channel). Restrictions regarding applicability may be encountered if there is very soft soil in the bedding zone and in the case of very large interceptors (problems relating to transporting the prefabricated pipes). The sewer and the bedding must be at least sufficiently stable for the time being (during work in the sewer). The section to be rehabilitated must be free of flow obstacles and must be taken out of operation for the duration of the work. In case of heavy groundwater infiltration, pre-sealing is required.

The method is not linked to any special pipe materials. In most cases, GRP pipes are used because they are most adaptable to the different cross-section shapes. The pipe joints are made with sleeves or slip-over couplings, as adhesive joints or over-laminate joints (possibly combinations).

The pipes are inserted into the section to be rehabilitated via special transport equipment, known as trolleys, onto which the pipes are placed. After that, the pipes are fixed using spacers and secured against the forces (especially buoyancy) that occur during the annular gap grouting. This work is carried out manually, for instance, by backfilling with quick-setting grout or softwood wedges, using special support saddles or spacers. Amongst other things, the annular gap grouting is intended to achieve:

» Positional stability of the lining pipe
» Creation of defined bedding of the lining pipe
» Preventing water transport through the annular gap
» Preventing soil penetration through the cracks in the host pipe
» Preventing gas accumulations in the annular gap
» Uniform transmission of external loads.

2.3 PVC spiral wound method with filling the annular gap with load-bearing grout

The only example of this method is the 'Sewage Pipe Renewal' method, or SPR method, from Japan, see ***Figure 8***. It is characterised by winding a lining pipe made from PVC rib profiles with steel reinforcement within the rehabilitation section. The PVC rib profile is wound off a reel and fed through a manhole to a self-propelling winding machine, which moves along a defined space adapted to the contour of the sewer to be lined. The SPR method can be used for rehabilitating gravity lines in the nominal widths DN 800 to DN 5500. The liner can adapt to many host pipe cross-sections, such as egg-shaped profiles, rectangular shapes and mouth profiles. Even curved sections with a curve radius that equivalent to five times the nominal width can be rehabilitated. Rehabilitation is also possible with controlled dry weather flow. House connection pipes can remain fully operational. This method requires little space on-site.

In order to ensure the SPR lining pipe can accommodate the external pressure of the backfill material during annular gap grouting, with larger cross-sections, adjustable and collapsible support frames are installed as bracing.

The defined annular gap between SPR liner and host pipe is filled with an easy-flow, high-strength backfill material. The backfilling creates a mineral 'pipe in pipe' that has a high stiffness and is used to bear the load alone. The PVC-U lining pipe does not help bear the load, but does protect the supporting pipe against chemical and mechanical attack from within.

2.4 Shotcrete with statically effective reinforcement

The reinforcement is connected with the old sewer walls by anchors.

In addition to the compaction, the high throwing energy released during the spraying process also helps create a good bond with the substrate.

Depending on the type of starting mixture, a distinction is generally made between the dry and wet spraying method. In the dry spraying method, the premix, consisting of cement, aggregate and, if required, powdered additives, is fed in a dry state to the conveying pipe and pneumatically conveyed as a thin stream to the spray nozzle where additional water and, if need be, liquid concrete admixtures are mixed with it. In the wet spraying method, the premix, consisting of cement, aggregate, additional water and, if required, additives, is fed in a wet state to the conveying pipe and conveyed as either a thin or thick stream. In practice, the dry spraying process is predominant because there are fewer problems with providing the starting mixture and in relation to the interruption of work. When leaving the nozzle, the mixture possesses a high kinetic energy that causes the shotcrete or sprayed mortar to be compacted when it hits the substrate.

Reinforced shell and spraying process are shown in ***Figure 9*** and ***Figure 10***.

3. Static dimensioning of rehabilitation systems

The nature and extent of the static dimensioning of rehabilitation systems to be used for strengthening the man-accessible interceptors under consideration here depend significantly on the condition of the host pipe, i.e. the residual load capacity of the existing system. Since July 2015, as the successor to the ATV-M 127-2, the DWA-A 143-2 has provided an essential basis for the static verification of lining and assembly methods. We describe below the changes or additions applied in the DWA-A 143-2 compared to the ATV-M 127-2 as far as they relate to the safety concept and any imperfections that need to be addressed. Finally, using examples, information regarding static modelling is provided and the effects of the new normative regulations are highlighted.

Table 2*: Partial safety coefficients for*
a) Effects,
b) Resistances or material properties and
c) Combination coefficients, Table from [2]

a) Effect	γ_F
Permanent loads (G) (Soil loads, dead load, surface load if applicable, concentrated surface loads)	1.35
Variable loads (Q) (traffic loads excluding road traffic, groundwater, etc.)	1.5
Traffic loads	1.35
Temporary flood (based on long-term modulus of elasticity)	1.1
Internal pressure (incl. pressure surge)	1.5
Test pressure	1.2
Temperature variations	1.1
Imposed deformations	1.1

b) Resistances (pipe material)	γ_M
Plastic liner, cured in the sewer	1.35
Plastic liner, factory manufactured (extrusion and other methods)	1.25
Mortar liner (taking account of any notch effects in material testing)	1.5
Stainless steel	1.15
Resistance acting favourably (e.g. imposed deformation in the case of host pipe condition III)	1.0
Host pipe made from concrete and stoneware for verification of host pipe pressure zones	1.5

c) Combination coefficients (combination with)	ψ
Temperature variations with external water pressure	0.7
External water pressure with soil and traffic loads	0.9
Equivalent load for external water pressure with soil and traffic loads	0.7

3.1 Application of the partial safety concept

One of the main new features of the DWA-A 143-2 compared with the ATV-M 127-2 lies in the transition from the global safety concept to the partial safety concept. Previously, the structural analysis was conducted using the characteristic material properties (strength $\sigma_{Br,k}$ and moduli of elasticity E_k, e.g. defined in the DIBt [Deutsches Institut für Bautechnik/German Institute for Construction Technology] certification). The "k" index describes characteristic material values. The stresses calculated under the effect of the working load (σ_k) were required to demonstrate a sufficient safety margin in relation to the material strength (e.g. host pipe condition II: req. $\gamma = 2.0$, for host pipe condition III: req. $\gamma = 1.5$). Furthermore, evidence of deformation and stability had to be provided. ***Table 2a*** and ***Table 2b*** contain the partial safety coefficients γ_F and γ_M, specified separately in the DWA-A 143-2 for effects (loads) and resistances (material properties) and ***Table 2c*** contains the combination coefficients ψ that can be used when considering

***Table 3**: Comparison of global safety according to the ATV-M 127-2 with the partial safety concept of the DWA-A 143-2*

Product, host pipe condition	γ according to ATV-M 127-2 [1]		$\gamma_F \cdot \gamma_M$ according to DWA-A 143-2 [2]
Pipe liner, HPC II, p_a	2.0	≈	1.5 · 1.35 = 2.025
Grouted GRP pipe, HPC II, p_a	2.0	>	1.5 · 1.25 = 1.88
Mortar liner, HPC II, p_a	2.0	<	1.5 · 1.5 = 2.25
Pipe liner, HPC III, p_E, p_V	1.5	<	1.35 · 1.35 = 1.82
Grouted GRP pipe, HPC III, p_E, p_V	1.5	<	1.35 · 1.25 = 1.69
Mortar liner, HPC III, p_E, p_V	1.5	<	1.35 · 1.5 = 2.025

load combinations (interaction verification). In the context of the static calculation according to the DWA-A 143-2, there is a geometrically non-linear analysis of the problem with iterative load increase up to γ_F times the working load, whereby the material properties ($E_d = E_k / \gamma_M$, $\sigma_{Br,d} = \sigma_{Br,k} / \gamma_M$) reduced by the partial safety γ_M must be included. The "d" index describes the design values of the material.
The stresses σ_d determined under γ_F times load are thus compared with the strength $\sigma_{Br,d}$ reduced by γ_M. Thus capacity utilisation rates (in the stress analysis $\sigma_d/\sigma_{Br,d}$) are determined, which may take on a maximum value of 1.0 (100 % utilisation). Since the stress analysis is performed non-linear under γ_F times load (effect), the stability verification is the analysis included. The deformation analysis is conducted as before based on the characteristic material properties (E_k) under working load effect. A number of consequences that result from the partial safety coefficients listed in Table 2 as well as from application of the partial safety concept are set forth below:
If one considers as an approximation for global safety γ the product of the partial safeties $\gamma_F \cdot \gamma_M$, the result for rehabilitation methods applied in the field of large profile rehabilitation is the comparison of safeties shown in ***Table 3***. It becomes clear that both sets of regulations DWA-A 143-2 and ATV-M 127-2 for pipe lining rehabilitation in host pipe condition II require a quasi-identical level of safety (2.0 ≈ 2.025). This is of particular importance because the design tables of the DWA-M 144-3 retain their validity despite the change in the basis of design. It should be noted in this respect that, in addition, within the framework of static verification of the design tables, comparative calculations have also been performed based on the DWA-A 143-2 design. A further prerequisite for the applicability of the tables is of course still observance of the required material properties and imperfection approaches. Furthermore, the comparison of the safety levels of both sets of regulations presented in Table 2 also shows that for grouted GRP pipes, because of the partial safety coefficient $\gamma_M = 1.25$ for host pipe condition II, the result is a decrease in safety to req. $\gamma \approx 1.88$ (previously req. $\gamma = 2.0$). Where host pipe condition III, i.e. soil and traffic loads, must be taken into account, in all cases considered the DWA-A 143-2 results in an increase in the required safety compared with the ATV-M 127-2.
In connection with fluctuating groundwater levels, the question of how to deal with the load case of the flood effect came up occasionally in the past. Designing a plastic liner based on the long-term material properties as well as a required global safety of $\gamma = 2.0$ (HPC II) seemed too conservative and uneconomical. By specifying a partial safety coefficient of $\gamma_F = 1.1$, the DWA-A 143-2 offers a way of verification. A pre-requisite for application of this coefficient is the consideration of long-term material properties (modulus of elasticity and strength) in the static calculation. With the previously mentioned approximation ($\gamma \approx \gamma_F \cdot \gamma_M$), the result is therefore a safety of approx. $\gamma_F \cdot \gamma_M = 1.1 \cdot 1.25 = 1.375$ (< 1.5 · 1.25 = 1.88). The short-term nature of the flood event is therefore taken into account by reducing the level of safety compared with the case of a long-term external water pressure load. This procedure is therefore only permissible for materials that demonstrate a load-deformation behaviour depending on the load time, i.e. for plastics. Otherwise (e.g. mortar liner, shotcrete inner shell, etc.), $\gamma_F = 1.5$ is to be expected, analogous to the case of long-term external water pressure load. It should be noted that for plastic systems there is the possibility, as before, to conduct the static calculation for the load time of the flood event using the material characteristics valid for the duration of the load case, e.g. from the creep rupture test in combination with a partial safety coefficient $\gamma_F = 1.5$. For both means of proof, the client or the client's representatives should be asked what exposure time is to be expected. The second means of proof shown above may well be decisive, e.g. with longer exposure times or even with plastics that show a relatively rapid reduction of the material properties.
Finally, the consideration of constraining forces ($\gamma_F = 1.1$) of rehabilitation systems is new in the DWA-A 143-2. They can arise due to the effect of temperature, for example, but may also be caused by soil and traffic loads in the case of host pipe condition III. In the latter case, the cross-section of the soil-embedded system of a cracked (in four shells) host pipe is (due to the formation of longitudinal cracks or due to lack of transfer of tensile stress e.g. in masonry) deformed by soil and traffic loads. This deformation acts as imposed deformation on the liner. Whether the load effect or the constraining effect is dominant depends on the stiffening effect of the liner [5]. Since this question cannot be answered at the outset, the two cases must be considered, based on the different partial safety coefficients in the context of static verification.
In addition to the structural analysis of individual effects, the permissibility of effect combinations must be established, whereby the combination coefficients ψ listed in Table 1c may be used. The most commonly required interaction verification relates to consideration of the

simultaneous effect of soil and traffic load (p_v) and external water pressure (p_a). This can be performed by applying the equations (1) to (3).

Stress analysis (tension zone):

$$\left(\frac{\max.\sigma_d(q_v)}{\sigma_{bZ,d}}\right)^{2,0}+\left(\frac{\max.\sigma_d(p_a)}{\sigma_{bZ,d}}\right)^{1,0} \leq 1.0 \qquad (1)$$

$\max.\sigma_d$: maximum tensile stress
$\sigma_{bZ,d}$: bending tensile strength
q_v: vertical soil and traffic load
p_a: external water pressure

Stress analysis (pressure zone):

$$\left(\frac{\min.\sigma_d(q_v)}{\sigma_{D,d}}\right)^{2,0}+\left(\frac{\min.\sigma_d(p_a)}{\sigma_{D,d}}\right)^{1,0} \leq 1.0 \qquad (2)$$

$\min.\ \sigma_d$: maximum value of compressive stress
$\sigma_{D,d}$: Compressive strength

Stability verification:

$$\left(\frac{q_{v,A,d}}{\text{krit}.q_v}\right)^{2,0}+\left(\frac{p_{a,d}}{\text{krit}.p_{a,d}}\right)^{1,0} \leq 1.0 \qquad (3)$$

$q_{v,A,d}$: vertical soil and traffic load upon buoyancy
$\text{crit}.q_v$: Snap-through buckling load due to soil and traffic load
$\text{crit}.p_a$: Snap-through buckling load due to external water pressure

The equations are already in ATV-M 127-2 in a similar form and were merely converted in DWA-A 143-2 into design values (index "d" in equations 1 to 3). In the context of the calculation, the soil load components of the vertical load are determined taking into account the buoyancy effect, also an annular gap approach may be disregarded when calculating $\text{crit}.p_a$. A new feature has resulted from the DWA-A 143-2, the possibility to perform a calculation on the entire system taking into account all influences.
From the above explanations regarding the partial safety concept of the DWA-A 143-2 it is clear that, particularly in the case of HPC III, numerous combinations of load cases ensue together with various applicable coefficient combinations $\gamma_F/\gamma_M/\psi$, which must be examined statically [6]. From the calculation according to DWA-A 143-2 result more calculation steps compared with the ATV-M 127-2, from which the stability and serviceability of the liner must be interpreted.

3.2 Static modelling, imperfection approaches, verification

The following section only considers trenchless methods that are used for structural reinforcement of large interceptors. For static verification of such rehabilitation systems, use of the finite element method (FEM) is advisable

» because existing interceptors often have special profiles (see above) and using the FEM any cross-section can be included in the static modelling,
» because a model created for inventory assessment (host pipe classification) can continue to be used by adding the components of the rehabilitation system to it,
» because this way, different, relatively straightforward rehabilitation options can be statically examined and their anticipated success can be assessed,
» because relatively light soil layers, which with large profiles may well change around the interceptor height area, can be considered and
» finally because all loads can be included in one calculation, which offers advantages in terms of interaction verification (see above).

Ultimately, the basics of static modelling have not changed with the appearance of the DWA-A 143-2. The applicable static model is essentially dependent on the host pipe condition I, II, III or IIIa of the existing interceptor and on the load effect.
In the case of host pipe condition II, the liner is primarily stressed by the effect of external water pressure. In accordance with DWA-A 143-2 and ATV-M 127-2, the structural calculation is performed using the system of a quasi-rigid liner cross-section embedded in the host pipe. While the standard software available primarily enables the calculation of circular and regular egg profiles, a wide variety of cross-section shapes (e.g. mouth profile, profiles with dry-weather channels, etc.) can be included in the static modelling through use of the FEM. The liner can be

Table 4: *Overview of the line installation methods in urban and rural regions*

Profile	Position φ_V	Symmetry to the vertical axis	Size ω_V	Opening angle $2\varphi_1$	Notes
Non-normal egg profile W:H ≠ 2:3	Middle of flat area, lopsided	no	0.5 % of H	Size of flat area	-
Mouth profile	180° (invert)	symmetrical and the case is to be examined	> 0.5 % of the invert radius, but ≤ 10 mm	Size of invert area	Also examine lop-sided (asymmetric) position
Other large profiles W or H > 1.5 m	1)	If applicable	10 mm to 20 mm	1)	Examine several positions and sizes, if need be

1) to be determined by an engineer at an unfavourable point

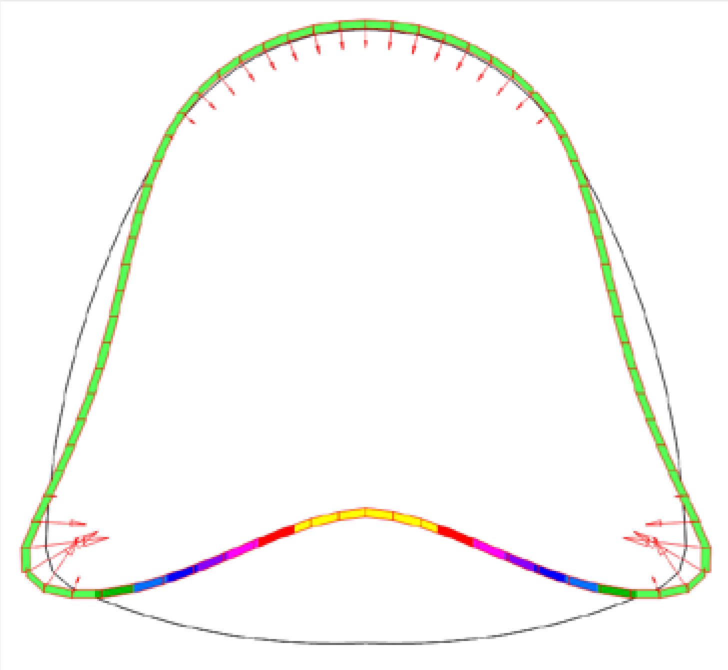

***Figure 11**: Host pipe condition II, liner deformed under external water pressure (exaggerated view) and activated contact pressure forces between the liner and the host pipe*

Source: IngSoft GmbH

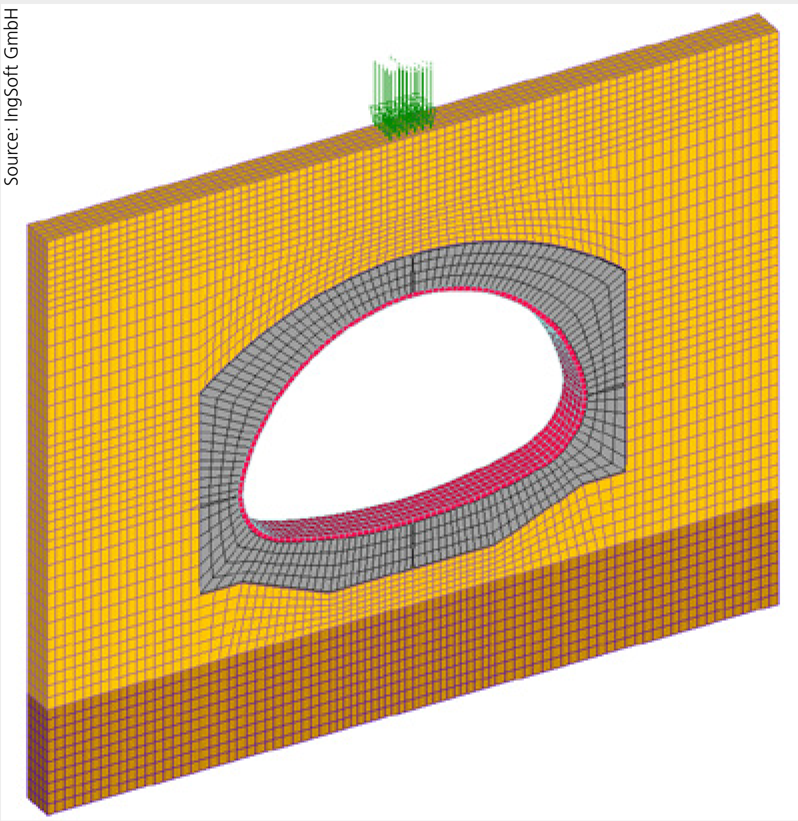

***Figure 12**: Continuum model of a mouth profile rehabilitation, soil cover and LM 1 as wheel load in the pipe center*

mapped using a wide variety of element types (beam, disc, shell or volume elements). The bedding of the liner in the host pipe-soil system is simulated, for example by means of support provided by compression springs distributed over the liner's outer wall or by defining a corresponding contact condition. In addition to the liner thickness, the modulus of elasticity represents the essential stiffness property of the liner. In accordance with DWA-A 143-2, the design value E_d must be used. The external water pressure is applied as a line or surface load on the liner outer wall and iteratively increased in a geometric non-linear analysis up to γ_F times the working load. Stress and stability analysis must be performed taking into account the design values, while observance of the permissible liner deformations must also be demonstrated taking into account the characteristic values. Since the issue is a stability problem in this case, imperfections (gap formation, local pre-deformation and joint ring pre-deformation) have to be taken into account in the geometric modelling of the liner. It is to be regarded as very positive that in the DWA-A 143-2 the information regarding the imperfection approaches was considerably expanded. Specifically, further details were provided regarding other processes (single pipe lining, spiral wound pipe lining, grouted-in-place lining process) and, in addition to circular and regular egg profiles, regarding other cross-section shapes. Large interceptors, in particular, often have special profile shapes. According to DWA-A 143-2, in these cases, for spiral wound pipe and grouted-in-place lining processes, an inward joint ring pre-deformation $w_{Gr,V}$ of the crown must be taken into account. For grouted methods, the $w_{Gr,v}$ approach is omitted. The local imperfection must be placed at the site of the anticipated buckling. Known damage patterns or test results (e.g. with circular profiles in the invert area, with egg profiles in the springline area) can provide information on the crucial buckling area. Table 3 from the DWA-A 143-2 provides information regarding local pre-deformations to be placed for special cross-section shapes (concerns, in particular, large profile rehabilitation).

Figure 11 shows an example of a liner deformed invertly under external water pressure in a jaw-shaped host pipe (exaggerated view). The rather flat arched invert has a tendency to snap-through buckling and activates high supporting forces in the corners. A correspondingly major impact on the stability is produced by the gap between the host pipe and liner, and unfavourable local pre-deformation of the invert.

Due to the high sensitivity with respect to the corresponding imperfections "local pre-deformation" and "gap", the approaches must be carefully determined and implemented on the safe side in statics.

If host pipe condition III is present, the liner, host pipe cross-section and surrounding soil are included in the

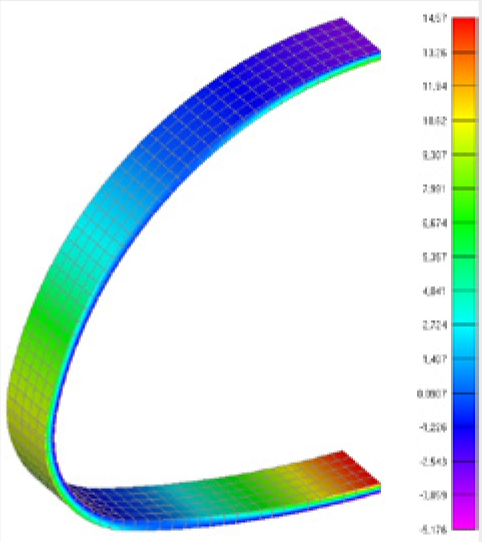

Figure 13: *Maximum main stress [MPa] in operational state under the influence of soil and traffic loads*

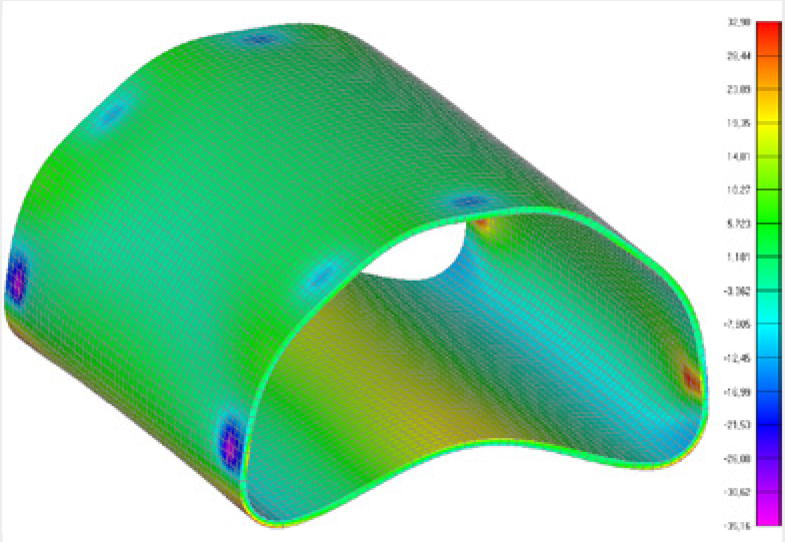

Figure 14: *Maximum main stress [MPa] in operational state (grouting process), Deformed model (exaggerated view)*

static modelling. Using the FEM, these components can be included with their actual geometry (pipe cross-sections) and layering (soil). Discretisation is usually performed as a planar FE model or as a three-dimensional shell using shell or volume elements. In the transitions between the liner and host pipe and between host pipe and soil, contact conditions (pure compressive stress transfer) have to be considered. The host pipe cross-section is displayed as a link chain. While for circular profiles in HPC III, four overload-induced longitudinal cracks (joints) in crown, invert and springlinees have to be included, with special profiles, the locations of longitudinal crack formation are not necessarily known at the outset. Here, if applicable, a calculation must first be performed as a rigid host pipe cross-section. The joint approach finally takes place at points where the maximum tensile stresses arise. ***Figure 12*** shows the modelling of a mouth-shaped GRP pipe in host pipe condition III. Due to the low depth of cover, a wheel load was applied as the traffic load. Dead weight loads are taken into account, in that the material-specific weight is assigned to the elements, traffic loads over area loads on the model surface. The geometrically non-linear analysis is performed using the design values and the $g\gamma_F$ times loads. Stress, stability and deformation analysis are to be performed. ***Figure 13*** shows an example of the development of the maximum main stresses under the influence of the soil and traffic loads. One half of the model has been calculated and displayed, since the problem is symmetric. ***Figure 14*** shows the development of the maximum main stresses of a GRP mouth profile in construction status. The local stress peaks result from wedging at the support points, which have an area of approximately 10 x 10 cm. To get a better idea, the deformed model is shown with an exaggerated deformation.

Conclusions

The worksheet DWA-A 143-2, published in July 2015, provides a sound basis for the preparation of static calculations in the context of rehabilitation of large profiles. However, the large dimensions and associated relatively high costs of carrying out the rehabilitation justify a closer and therefore more elaborate static assessment and calculation because, on the one hand, there is also a high risk potential for the public street space and, on the other hand, there is great potential for savings.
Therefore, the "host pipe condition" should not be determined merely by visual inspection, but by means of an extended assessment of the condition of the sewer, which ideally includes the following investigations:

- » Sight of as-built plans and, if applicable, historical documents
- » Inspection of the sewer and documentation of statically relevant damage (cracks, corrosion)
- » Assessment of the current safety status and imminent danger of collapse
- » System of monitoring fields with plaster and crack marks
- » Determination of the geometry of the inner and outer contour of the sewer wall (surveying, determination of wall thickness)
- » Taking samples from the sewer wall (profometer measurements, retrieval of core samples, if necessary removal of reinforcement bars)
- » Laboratory tests to determine the statically relevant material properties
- » Soil investigations in and next to the former trench
- » Laboratory investigations to determine the statically relevant soil parameters

The results of the extended condition assessment then make it possible to determine the host pipe condition in accordance with DWA-A 143-2 and to select the rehabilitation method. The subsequent static calculations both for the actual condition and for calculation of the liner can generally no longer be carried out with the formulas of the DWA-A 143-2 because the application limits are exceeded in many ways. Instead, a static analysis must be performed in full compliance with the regulations using the finite elements method, which requires structural engineers to have, in particular, in-depth knowledge of the numerical methods of mechanics in addition to accurate knowledge of the rehabilitation process.
The corresponding statics established provide optimal dimensioning of rehabilitation method (the liner) incl. all the necessary verifications with respect to the sewer's long-term stability and suitability for use.
This ensures that even large interceptors can be rehabilitated as economically as possible and yet remain functional over the entire period envisaged without risk to the surface traffic.

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