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LINING STABILITY: AN ANALYSIS OF DAMAGED SEWERS

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ABSTRACT: With the assessment of sewers and drains generally recognised criteria such as tightness, stability, hydraulic capability and efficiency are valid. For the assessment of the *stability*, however, there are often uncertainties. In this connection the ATV-DVWK Advisory Leaflet ATV M 127-2 represents a practical development of the system of rules and standards for the stability of earth-bedded pipelines that is called upon in the case of dimensioning for rehabilitation procedures. Nevertheless, further assumptions about the pipe-soil system are necessary which often are based on individual appraisals and experiences of the designer. To that end, in the years 1999 to the beginning of 2002, extensive experiments and theoretical investigations were carried out at the Universities of Bremen and Muenster as well as at the Bremen hanseWasser GmbH in order to assess more reliably the load-carrying capacity in particular of damaged host pipes in the ground. The research was supported by the Federal Ministry for Education and Research (BMB+F) under the short title ASSUR. The paper gives a view of the essential contents of the research; further information can be taken from the final report to ASSUR (Steffens et. al. 2002).

1. INTRODUCTION

For the assessment of the structural substance of sewers and drains criteria are necessary by which, on the one hand, the priority of a rehabilitation can be seen and, on the other hand, the necessary wall thickness of the epoxy liner can be determined conclusively.

To classify the degree of damage from the static viewpoint the host pipe stages defined in the German ATV-DVWK Advisory Leaflet ATV-M 127-2 are helpful. The *Host Pipe Stages I and II* describe old pipes without and with longitudinal cracks; there is, however, a load bearing old pipe-soil system also on a long-term basis. For the dimensioning the external water pressure is almost exclusively decisive. For the buckling-behaviour of liners under external water pressure numerous experimental and theoretical papers have been published, cf. (Falter et. al. 1996) for example for a summary.

Host Pipe Stage III is, according to Advisory Leaflet ATV-M 127-2, defined as follows: the old pipe is cracked four times in the longitudinal direction, the resultant pipe-soil system is not, on a long-term basis, bearing safe - with that earth and traffic loads are also to be examined. Although a requirement exists for such investigations in order to determine the wall thickness of a liner economically and with certainty, there are up until now relatively few works available for Host Pipe Stage III, s. Table 1 and (Law & Moore 2003).

Table 1. Experiments for Host Pipe Stage III, broken host pipes with (III) and without liners (IIIo)

Year	Source	Liner	Host pipe s = wall thickness	Soil group	Host Pipe Stage Test	Maximum test-load (Load plate)	Imperfections			Measure- ment
							w _D	w _{FH,D}	w _G	
1	2	3	4	5	6	7	8	9	10	11
1988	Watkins Shupe Osborn	-	Concrete ND 762	G3	IIIo TB	p ₀ = 420 kN/m ²	-	0	-	Δd _D
		CIPP			III TB				*)	
1999	Falter Grunwald Steffens	-	Masonry Oval 800/1200 s = 120/250 mm	G3	IIIo Insitu	F = 430 kN (0.5×0.5 m)	-	0	-	8×3 LVDT
2002	Doll Achmus Rizkallah	CIPP	Concrete ND 800 s = 130 mm	G1, G3	III TB	p ₀ = 482 kN/m ²	-	0	*)	Δd _D σ _z
2002	Steffens Falter Harder (ASSUR Report)	-	Concrete ND 400 s = 65 mm a) unbroken b) faulty pipe zone	G1	IIIo TB	F = 300 kN a) 1×1m b) 0.5×0.5m	-	6%	-	LVDT σ _z ; σ _x
		-	Vitrified clay ND 400 s = 41 mm	G1	IIIo TB	F = 300 kN 1×1 m	-	6%	-	LVDT σ _z ; σ _x
		-	Concrete frag- ments ND 400 a) new, s = 80 mm b) 100 years old, s = 50 mm	-	IIIo -	F a) = 180 kN b) = 60 kN	-	3%, 6%, 9%	-	Strain
		-	Masonry, oval B/H = 1310/1700 s ≈ 260 mm	G3	IIIo In situ	F = 430 kN (0.5×0.5m)	-	0	-	6×1 LVDT

Abbreviations: TB: test box, p₀: surface load, F: concentrated surface load (wheel load), a×b: size of the load plate, w_D: local deformation, w_{FH,D}: four-hinge deformation, w_G: annular gap, LVDT: linear variable differential transformer, σ_z, σ_x: vertical and horizontal soil pressure (soil pressure transducer)

*) No information (typical value for CIPP laminate: 0.2 to 0.5 % of the liner's radius)

While the theoretical approaches to dimensioning essentially agree for Host Pipe Stages I and II at national and international research level (s. comparison calculations in Thépot 2001), the opinions on Host Pipe Stage III which are represented below for a system without groundwater are definitely very different:

1. On the one hand the opinion exists, that it is a question of a consolidated system with the broken host pipe in the ground and therefore liner in the case of an absence of groundwater or of other pressure conditions hazardous to stability, danger can be dimensioned very thin.

2. On the other hand the view is taken that the host pipe-soil system is still subjected to loads even after the installation of a liner, for example through ground relocation, subsequent work near the pipeline route and live loads (with small covering).

Those experimental papers published up to the appearance of the ASSUR report vary considerably, see Table 1. In this way some investigations were carried out in the test box (TB), other load experiments took place in existing sewers (in situ). Uniform loading of the surface of the test area (p_0 as defined in ATV-A 127) and subarea loads over the area of a load plate (single load F , plate size $a \times b$) were chosen as operations. Finally, the amount and quality of measurement were also different: In some cases pipe deformation only was measured, in other cases both the vertical and horizontal soil pressures. In addition "Zero experiments" in the test box *without* a pipe are also reported upon with the aim of calibrating the computation models for further investigations *with* pipes.

2. OBJECTIVES OF THE ASSUR RESEARCH PROJECT

In the ASSUR study *experimental and theoretical stability verifications* were combined. On the one hand, with the aid of the findings, the basis of the theoretical verification is improved, on the other hand already well-proven experimental procedures are transferred to the assessment of the remaining service life time of sewers and drains.

On the question of *material conservation* the following investigation topics were, inter alia, covered:

- Procedures for the static assessment of partially damaged sewers and special structures,
- Definition of crucial degrees of damage with masonry main sewers (spalling, cross-sectional deformation, corrosion of the mortar, cracks, missing stones),
- Simplification of the measurement technology with load experiments through clarification, whether and in which cases a measuring base independent of the load might be dispensed with.

For *sewer renovation* the following tasks are important

- Further development of static dimensioning procedures,
- Inclusion of time-controlled processes (permanency, success forecasting for renovation measures),
- Definition of liner location misalignment in host pipes and their relevance for the stability of liners,
- Static assessment of disturbances limited to being parallel to the axis (e.g. in cases of fragment formation, pipe misalignment, defects, side connections) using spatial models,
- Loading of broken host pipes through earth and traffic loads,
- Buckling calculation for oval profiles, in particular systematic investigations on the influence of the springlines on the stability in connection with installation tolerances attainable in practice,
- Mathematical and experimental determination of the influence of neighbouring construction works on renovated sewers with the aim of a minimisation of damage.

3. TEST BOX

3.1. Pressure distribution of subarea loads (Tests 1 - 3)

In order to test the usability of the earth pressure transducers employed and to determine the necessary number and order, "zero experiments" using a body of soil without a pipe were carried out. The soil in the test box is incorporated in layers and compacted according to specification, and inductive measurement facilities (several linear variable differential transformers (LVDT) and 10 earth pressure transducers) are emplaced. Using a load plate measuring 1×1 m², in total three zero experiments (sand without pipe) were applied.

In Experiments 1 and 2 the compaction density of the soil is varied. In the first experiment a compaction density $D = 0.70$, in the second experiment $D = 0.89$ is achieved.

To simulate a realistic trench situation (side support through soil layers) and to avoid corruption of the results through deformation of the test box walls - beginning with the Experiment 3 - a layer of expanded plastics is installed between the walls and the body of soil.

The measurements of the vertical soil pressures at the level of the later pipe zone and the deformation measurements at the sidewalls show that, even without elastic side bedding, no appreciable silo effect occurs. The integration of the soil pressures at discrete points fulfils approximately the vertical equilibrium.

With increasing load a stress concentration results under the load plate.

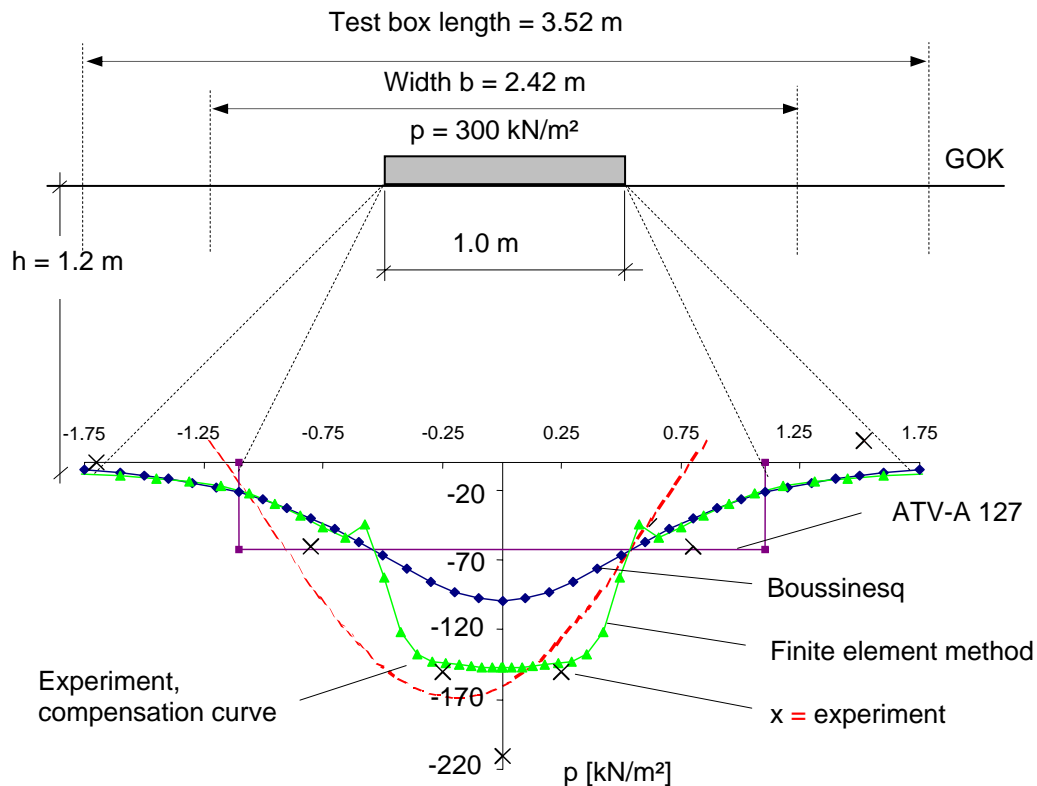


Figure 1. Comparison of the soil pressure with Zero Experiment 1 (sand not compacted)

The evaluation of the analytical results and comparison with the measured data recorded in the experiment show that the maximum vertical soil pressure (with 2.5 times the 60 t HGV load) from the experiments and Finite Elements (FE) calculations lie above the values of ATV-DVWK Standard ATV-DVWK-A 127, i.e. in the case of shallow covering the pipe stresses at the level of the crown are, according to ATV-DVWK-A 127, significantly underestimated, see Figure 1. This has consequences, for example, for pipes with smaller nominal diameters as well as for short pipes, as these can lie in the area of stress peaks and the load distribution determined according to ATV-DVWK-A 127 is too favourable. For pipelines with limited damage higher load approaches are therefore necessary, for example with partial rehabilitation.

3.2. Concrete pipe cracked four times along the axis (Test 5)

Four continuous cracks in the crown, the invert and the springlines are a typical damage scenario for rigid pipes, see Figure 2. The rigid pipe is converted into a flexible four-hinge system which must be supported by the side soil. For load-carrying capacity experiments on this pipe-soil system a concrete pipe systematically cracked four times is installed into the test box in accordance with Figure 2 with ca. 60 cm

earth cover and the surface is loaded via a 1×1 m² plate. As an initial deflection of the four hinge system 6 % is chosen before application of the load.

The pipe deformation is determined using two and three dimensional models (Figure 3) as well as according to ATV-DVWK Advisory Leaflet ATV M-127-2 and is compared with the experimental results, cf. Table 2.

Table 2. Comparison of the measured and computed pipe deformation for Test 5

Model (FE = Finite Elements)		-	Truss model	2D-FE-Model for Host Pipe Stage III	3D model
Source		Test 5	ATV-M 127-2 Appendix 6	ATV-M 127-2	ABAQUS
Model dimension		3D	2D	2D	3D
Crown deformation, inside	mm	-2.0	-	-1.56	-6.34
Invert deformation, inside	mm	-0.5	-	+1.56	-2.80
Pipe deformation Δd_D	mm	1.5	3.6	3.12	3.54
Pipe deformation δ_D related to the radius	%	0.4	0.9	0.78	0.88

Note on the deformation direction: + above, - below

The computed pipe deformation (Columns 4 to 6) agree well, they are however above the measured values. With that the calculation models of the German design standard for the dimensioning of sewer rehabilitation are conservative.

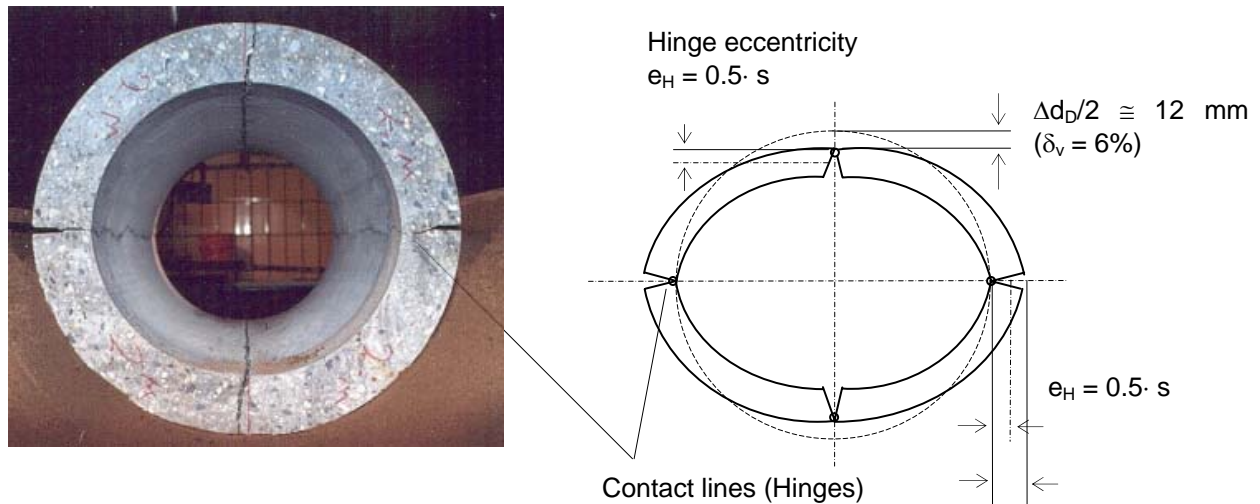


Figure 2. Concrete pipe ND 400 cracked four times (Test 5)
 Left: Installation in the test box, Right: Kinematics with 12 mm crown deformation

3.3 Non-bearing pipe zone (Test 6)

As in Test 5 a concrete pipe systematically cracked four times is installed. In order to consider possible loosening of the soil beside the pipeline zone, for example through groundwater flow or infiltration, the side bedding of the pipes is disturbed at the level of the springline. For this purpose fire hoses are installed at the sides and are filled with water. During the experiment the water was bled from the fire hoses causing cavities in the ground.

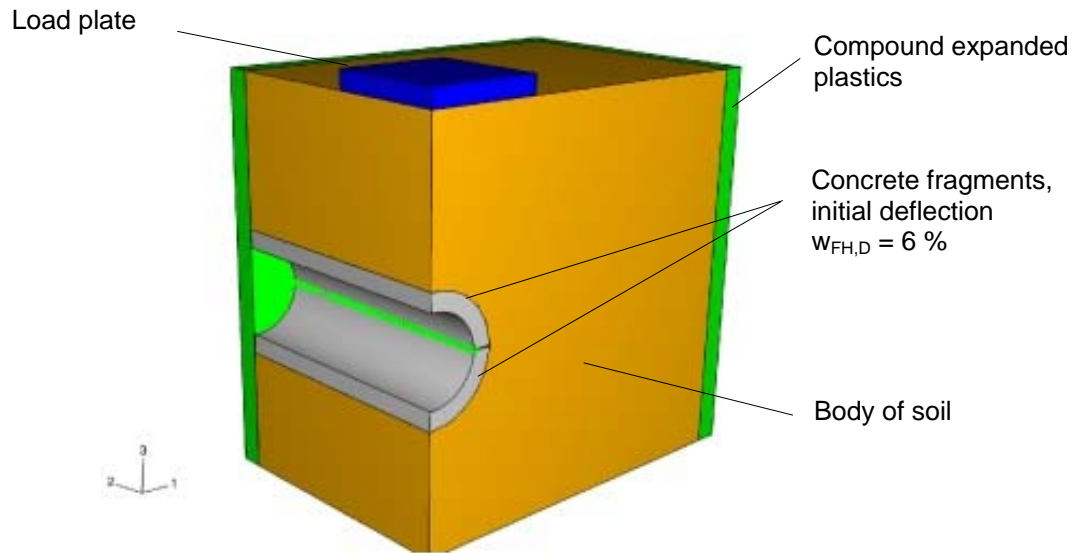


Figure 3. Test 5: Test box with cracked concrete pipe, three dimensional Finite Element model

Under the maximum load of $F = 300 \text{ kN}$ doubled pipe deformation $\Delta d_D \cong 3.18 \text{ mm} > 1.5 \text{ mm}$ (measured value) arise when compared with Test 5. Through the greater pipe deformation the vertical soil pressures above the crown are more heavily reduced than in Test 5 (where pipe zone was well compacted).

Then the load is applied via a smaller load plate ($a \times b = 0.5 \times 0.5 \text{ m}^2$) in order to concentrate the pressure on the pipe crown. This led, already at the load stage $F = 170 \text{ kN}$, to great deformation with the beginning of collapse of the pipe-soil system.

3.4 Excavation and backfilling with an irregularly cracked vitrified clay pipe (Test 7)

For the processing of an irregular crack scenario with a greater number of fragments an experiment with a vitrified clay pipe ND 400 was carried out. After reassembling the pipe cracked by crown pressure load only point contact is to be recognised between the fragments, s. Figure 4. After the constant load tests and continuous load tests the pipeline is dug up and the trench backfilled again. This corresponds to the case, where the pipe-soil system is influenced and/or disturbed by later construction activities near the pipeline route.

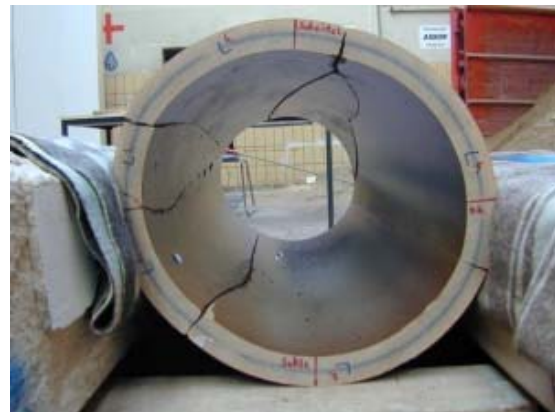


Figure 4.
Test 7: Vitrified clay pipe irregularly cracked in 6 fragments

(Photo in Steinzeug Abwassersysteme GmbH in Cologne)

During *the first load test* (load plate $a \times b = 0.5 \times 0.5 \text{ m}^2$, max $F = 135 \text{ kN}$) the pipe's crown is deformed by ca. 1.1 mm. During the *excavation* an elevation of the pipe crown of ca. 0.4 mm is measured, the vertical soil pressures in this region are reduced by ca. 12 kN/m^2 .

The vertical crown deformation remains almost unchanged during the *backfilling*, cf. Figure 5. The soil was compacted using a medium weight electric compactor after each ca. 30 cm filling height. During the compaction in one layer of 32 cm above the crown, the deformation changes by ca. 0.25 mm (s. Figure 5 at 225 min). A compaction at a height of 66 cm above the pipe crown also has even smaller modifications of the crown deformation (s. Figure 5 at 250 min) and of the vertical soil pressure about the pipe crown as a result.

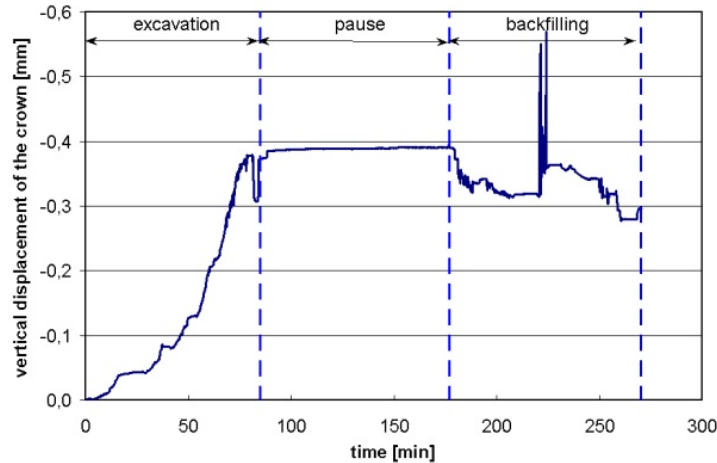


Figure 5. Vertical deformation of the pipe during excavation and backfilling with Test 7

The tests show that an excavation and backfilling over a damaged host pipe lead to loads and deformation that are to be explained by changes of overload. Such impact on liners coming from outside construction activities are taken into account in ATV-DVWK Advisory Leaflet ATV-M 127-2 through Host Pipe Stage III.

A second load test with a load plate $a \times b = 0.5 \times 1.0 \text{ m}^2$ and 310 kN maximum load leads to 1.04 % ovalisation. The calculation using Appendix 6 of ATV-M 127-2 shows an ovalisation of 3.0 %, which is greater than 1.04 %. The system examined from six irregularly broken fragments apparently does not behave more flexibly than the theoretical model with four hinge lines assumed in the Leaflet M 127-2.

During the reloading a further short diagonal crack occurred which was caused possibly by an undetected internal hairline crack before applying the load.

4. LOAD-CARRYING CAPACITY OF THE PRESSURE AREAS OF BROKEN HOST PIPES

An assessment of the structural safety of an old pipe is only possible if it also keeps on being able to transmit vertical loads in the pressure areas of the springline. For this reason experiments with concrete pipe fragments are carried out in a testing machine in the same way as for a broken and deformed host pipe, cf. Figures 6 and 7.

In Table 3 the ultimate pressure strengths of new pipe fragments and those of a ca. 100 years old sewer in Muenster are presented. The new pipe has the concrete quality B 45. The pressure resistance of the old pipe is in this case only ca. 30 % of the new pipe samples.

Table 3. Ultimate pressure strength σ_D of the pipe tests in N/mm^2

Test	1	2	3	4	Mean value
N (new sample)	46,2	45,0	63,4	-	51,5
A (old sample)	17,8	15,2	18,1	17,1	17,0

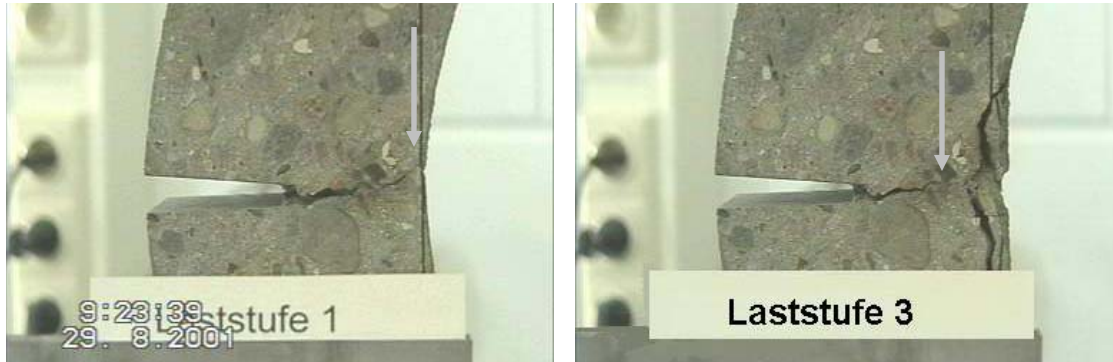


Figure 6. New pipe sample, deformed 9 % at $F = 10 / 30 \text{ kN}$ (= 8 / 25 % of the ultimate load)

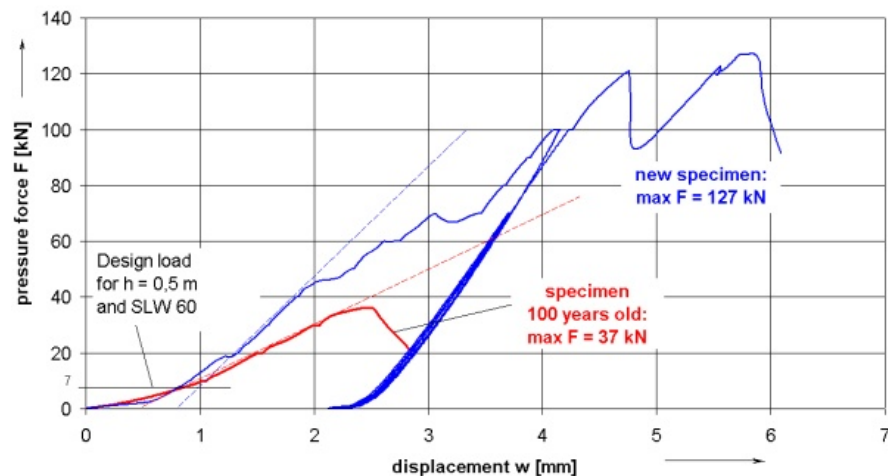


Figure 7. Old pipe sample, deformed 9 % at $F = 10 / 30 \text{ kN}$ (= 15 / 45 % of the ultimate load)

The comparison of the load displacement curves for new (N) and old (A) pipe tests in Figure 8 can be summarised as follows.

For the cases examined with 3, 6 and 9 % deformation the ultimate loads of the new tests are 2.7 to 3.5 times higher than for the old ones. The load displacement curves of the old sample are flatter, e. g. by loading old pipes higher compression strains in the structure are to be expected - this leads in turn to additional loading of the liner after rehabilitation.

Figure 8. Compression tests on new and old samples, deformed 9 %, load-deflection curves up to the ultimate load



5. INSITU-EXPERIMENTS (LOAD VEHICLE BELFA)

Within the investigation project ASSUR in situ experiments were planned on sewers employing the new load vehicle BELFA, see Figure 9, which has been designed to test the load capacity of bridges in Germany.



Figure 9.
Above: load vehicle BELFA,
right: load-plate of BELFA vehicle

In Bremen the crown axis of a masonry main sewer with oval shape B/H = 1310/1700 mm was loaded up to max $F = 450$ kN via a 1×1 m² load plate analogous to a previous load test using a crane (Falter et. al. 1999). The soil cover height was approx. 1 m.

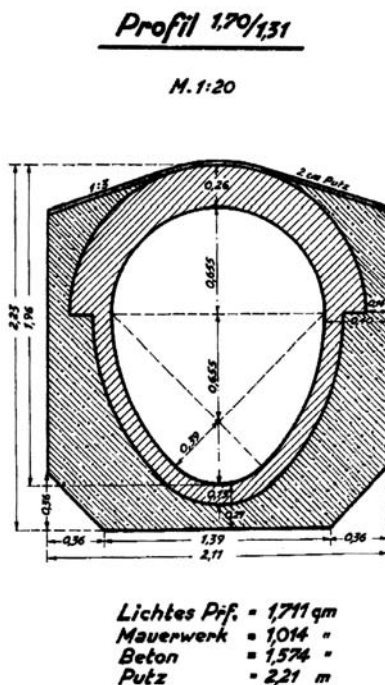


Figure 10.
Left: Sectional view of the masonry main sewer with oval shape 1310/1700 mm in Bremen,
Above: Measurement base unaffected by the load using 6 linear variable differential transformers (LVDT)

Instead of the $8 \times 3 = 24$ linear variable differential transformers (LVDT) (Falter et. al. 1999), in order to reduce expense only a total of 6 transformers were placed on the independent measurement base, cf. Figure 10. Nevertheless, an unambiguous reconstruction of the displacement state of the main sewer wall is possible from the results of measurement in Figure 11. The crown deformation under the maximum test load were 2.06 mm (test) and 1.87 mm (calculation).

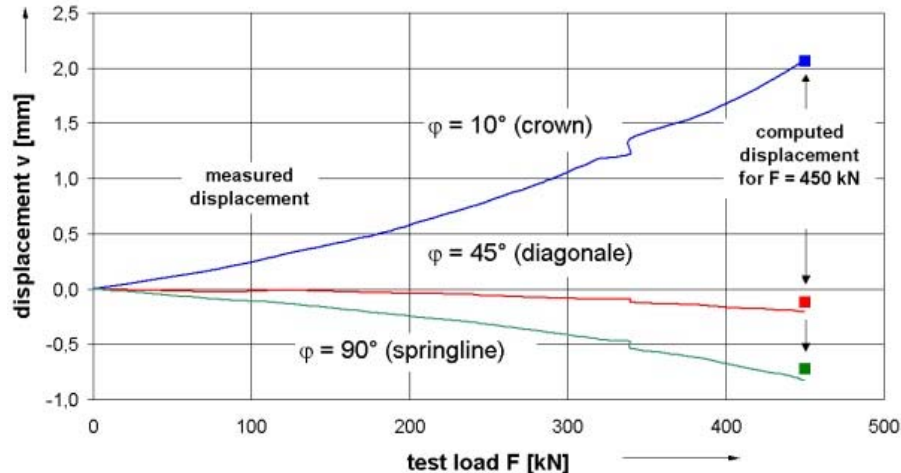


Figure 11. Load-deflection curve on loading the masonry 1310/1700 mm oval main sewer in Bremen

From the deformation behaviour of the main sewer under load it is possible to find suitable calculation models for the structural safety of the construction. This was possible on the one hand experimentally by applying the γ -time load from a 60 t truck (HGV 60), on the other hand with the aid of the soil parameters gained from ground explorations (here: clay classified as soil group G3, stratified ground) by numeric verification of the entire stability and the sectional resistance. Further information is presented in (Steffens et. al. 2002)0.

6. SUMMARY

With the results of the ASSUR research project (Steffens et. al. 2002) the calculation models in ATV-DVWK Advisory Leaflet ATV-M 127-2 are checked experimentally for the first time. With the updating of the dimensioning rules the following findings are to be considered:

- The influence of concentrated area forces is found to be higher than previously assumed.
- The initial deformation must be differentiated more precisely in particular for larger main sewers (for example masonry oval sections).
- The influence of the pressure areas in the assumed hinge lines on host pipe and liner is to be specified, corresponding verifications are to be supplemented.
- Notes on use for experimental stability verifications are to be included in design standards.

The pilot experiments and in situ measurements, inter alia, showed the following results:

- Safe structural modelling for a masonry main sewer is possible also in ground with soil parameters varying over the height,
- The necessary amount of measurements can be reduced on the basis of static models already proved,
- Material conservation is possible even in cases with old pipe-soil systems which cannot be tested or tested only at great expense.

The experiments and mathematical relationships give an insight into the parameters whose knowledge is necessary for the assessment of the stability of damaged sewers or main sewers. At the same time it is

shown, through which parameters a collapse can be caused when these deteriorate through further usage (strength, Modulus of Elasticity, local cracks etc.).

The parameters of the old pipe stage are also important for an installed liner, thus compression in high pressure areas of the host pipe can lead to increases of the liner loading.

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