Field Verification of Ring Compression Conduit Design

J. DEMMIN, Armco-Thyssen, Dinslaken, Germany

•IN July 1963, Armco-Thyssen, a joint venture of Armco Steel Corporation, Middletown, Ohio, and August Thyssen-Huette, Duisburg-Hamborn, Germany, carried through live-load and loading-to-failure tests. The test structure was a 7-gage multi-plate pipe arch of 20-ft 7-in. span and 13-ft 2-in. rise, Armco's largest structure of this shape on record.

The live-load test was conducted to prove to the German Federal Railway that large corrugated steel structures are safe for use as conduits and underpasses in railway embankments. Therefore, the live-load test was to be conducted under the severest possible loading conditions required by the German Federal Railway design criteria, considering a safety factor of 3. The loading-to-failure test was conducted to provide scientific data on the behavior of corrugated steel structures under loading conditions, especially to determine under what load the structure would finally collapse and how this collapse developed. Both tests were conducted on the same test structure. Only the cover height and the positioning of the load were varied according to the different test purposes.

Size and gage of the structure were primarily designed for practical considerations suggested by the test purposes. For general acceptance of corrugated steel structures by the Federal Railway, it had to be proved that even the largest structure designed, of the most unfavorable shape, would satisfy performance requirements. Pipe arches, in particular, were considered statically unfavorable. Therefore, Armco's largest pipe arch was chosen as a test structure. Since the cover was low and the live load was fairly small, the wall thickness was not determined by ring-compression methods, but by empirical data applying to the structure during backfilling. Therefore, the wall thickness was designed by the "flexibility factor."

The suggested maximum flexibility factor is 5.0×10^5; $FF = D^2/J$. Since the periphery of this pipe arch is 20 ft 7 in. × 13 ft 2 in. = 207π (see Armco Catalog MP-1663), for the pipe-arch structure $D = 207$. In addition, the moment of inertia of multi-plate wall for 7-gage thickness is given by $J = 0.1080$ in.^{4}/in. and for 8-gage thickness by $J = 0.0961$ in.^{4}/in. Therefore, the 7-gage flexibility factor $FF = 207^2/0.1080 = 3.97 \times 10^5$ (o.k.), and the 8-gage flexibility factor $FF = 207^2/0.0961 = 4.46 \times 10^5$ (o.k.). With a special view to the loading-to-failure test and since the same structure was to be used for both tests, a wall thickness of 7 gage was chosen.

TEST SETUP

Test Structure and Backfilling

The pipe-arch structure—20-ft 7-in. span and 13-ft 2-in. rise—to which loads were to be applied, consisted of two rings, each of 8-ft length, which could freely deflect (Fig. 1). This 16-ft long test structure was completely within the pressure area of the applied load under the selected cover heights. Additional pipe sections were attached to this central body. The section at the open end was also made up of two rings bolted together, whereas only one ring section was added to the rear which was closed by a wooden cover and backed up with earth. The pipe sections adjacent to the central body were only to serve for widening the upper grade surface, thus reducing the danger of subgrade failure. They were separated from the center body by 4-in. wide gaps to

NOTE: In this paper kp (kilopond) is equivalent to kilogram (kg).

Paper sponsored by Committee on Culverts and Culvert Pipe.

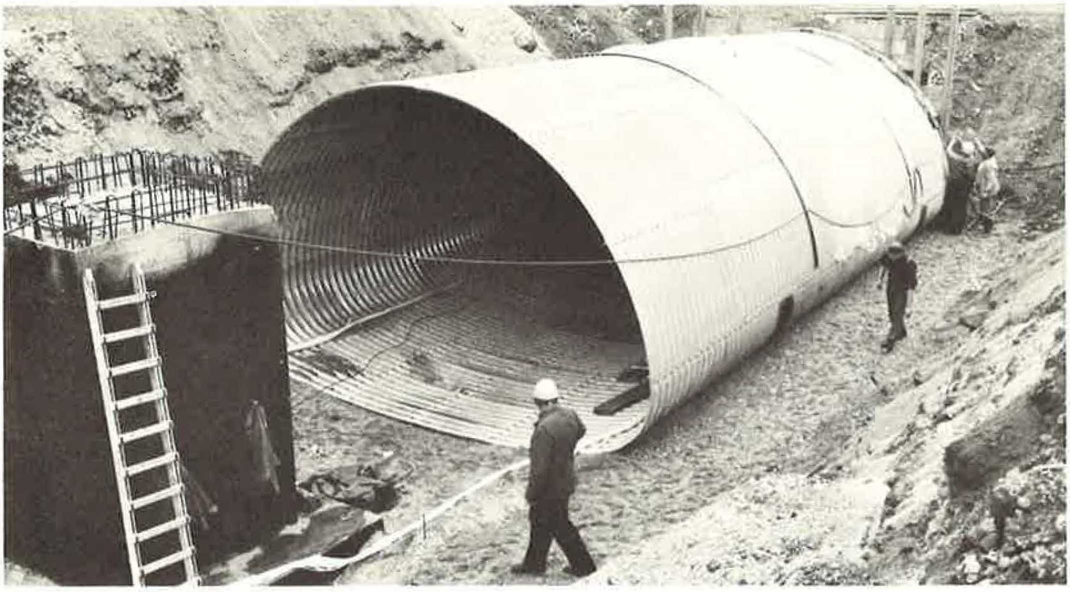


Figure 1. Test structure before backfilling.

Figure 2. Placement and compaction of backfill.

permit independent deflection of the actual test structure. Only the lower corner plates of all sections were firmly connected. To prevent soil seepage, the gaps between the pipe sections were covered with 10-gage corrugated metal strips of 1.5-ft length.

Backfill material was placed in lifts of 8 in., with each layer tamped separately (Fig. 2). Gravel was used as backfilling material and surface vibrators were employed for compaction. Tamping operations were continuously checked by drop-penetration testing. A laboratory Proctor test showed a soil density of 107 percent of single Proctor density (see Appendix).

The test was carried out in the works area of the August Thyssen-Huette plant. An excavated site that was to accommodate heavy column foundations served as a trench. The test structure was installed between two strong concrete pillars.

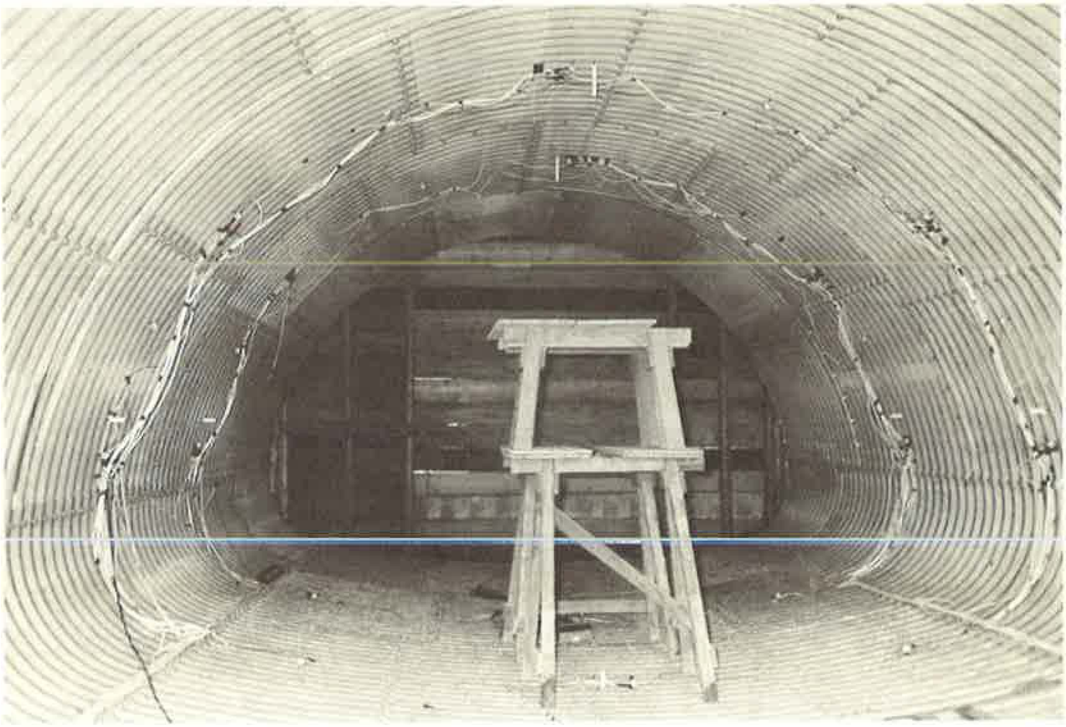


Figure 3. Inside view of structure.

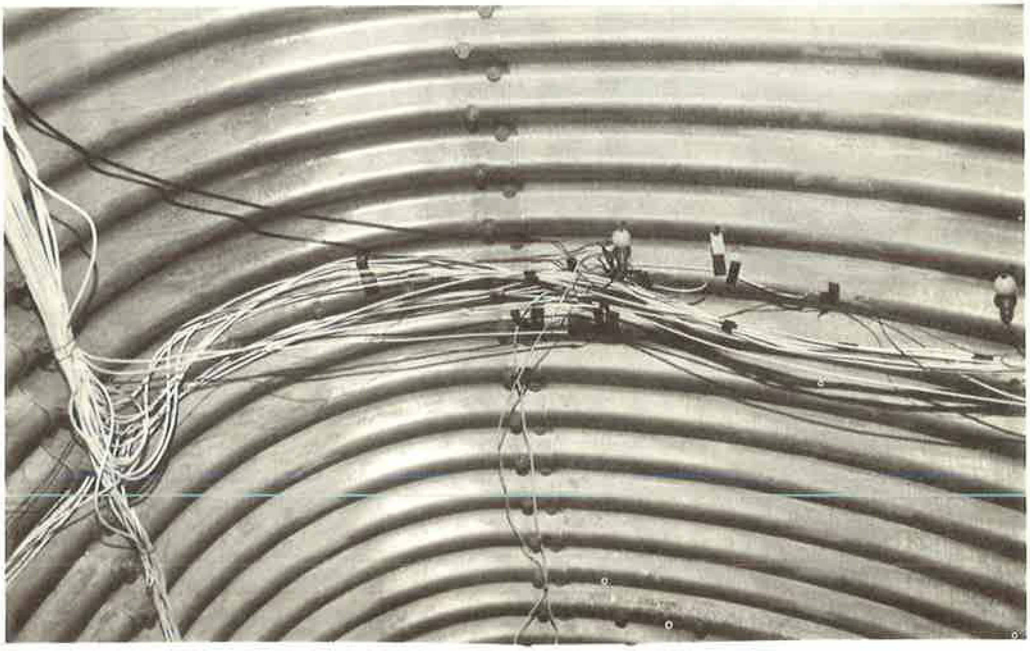


Figure 4. Partial view with measuring elements.

Measuring Instruments

Strain measurements were taken with strain gage strips. Three gages were installed at each of six measuring points (in the trough, and the crest of the corrugation and near the axis through the center of gravity of the corrugated profile) in two sectional planes. They were glued on with the special X-60 adhesive. To compensate for the influence of thermal expansion, compensation strips were placed near the gage points. To accomplish this, gages were stuck into small test coupons of the pipe-arch material. These were attached to the pipe arch so that they would undergo the same thermal expansion as the test structure without suffering any strain through the imposed load. The actual

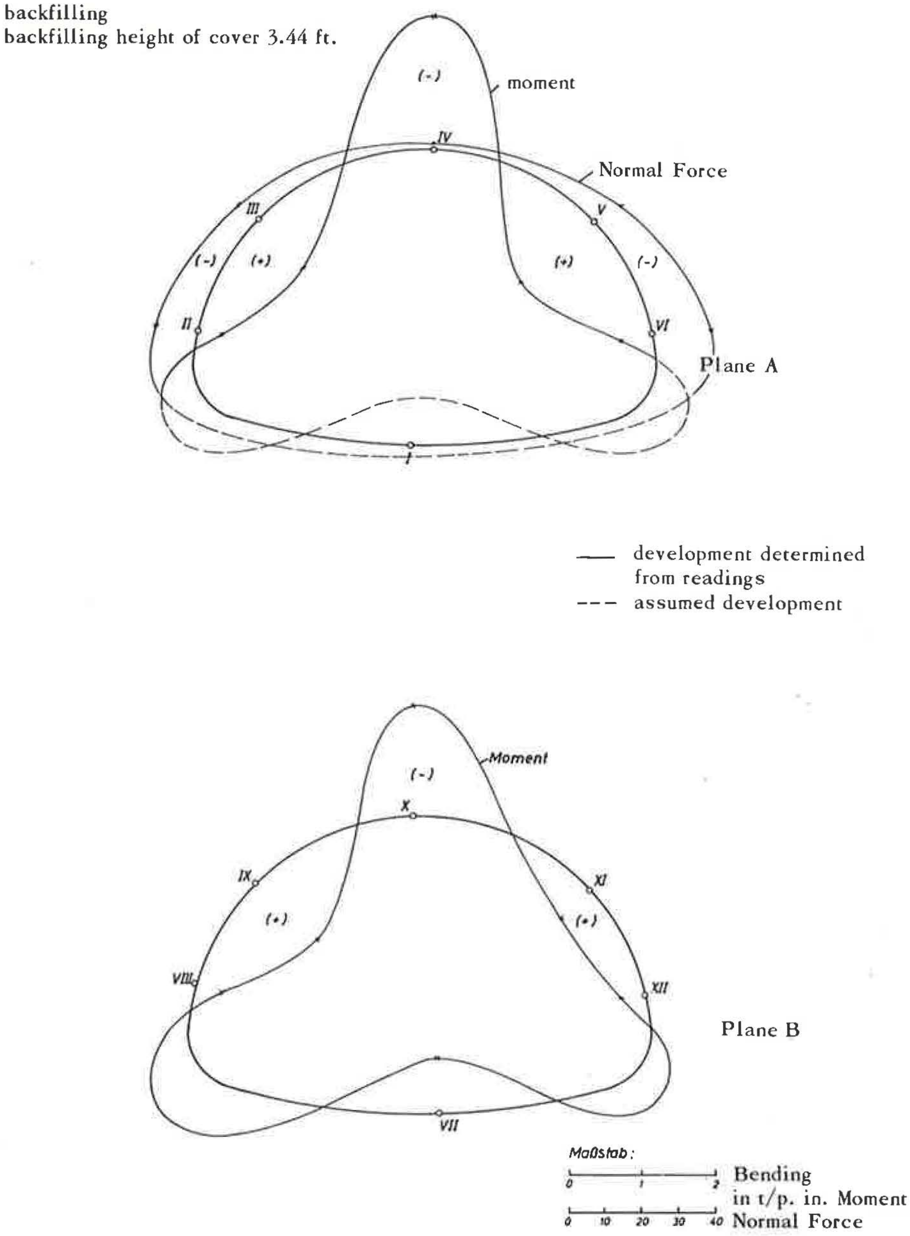


Figure 5. Development of normal forces and bending moments from reading taken.

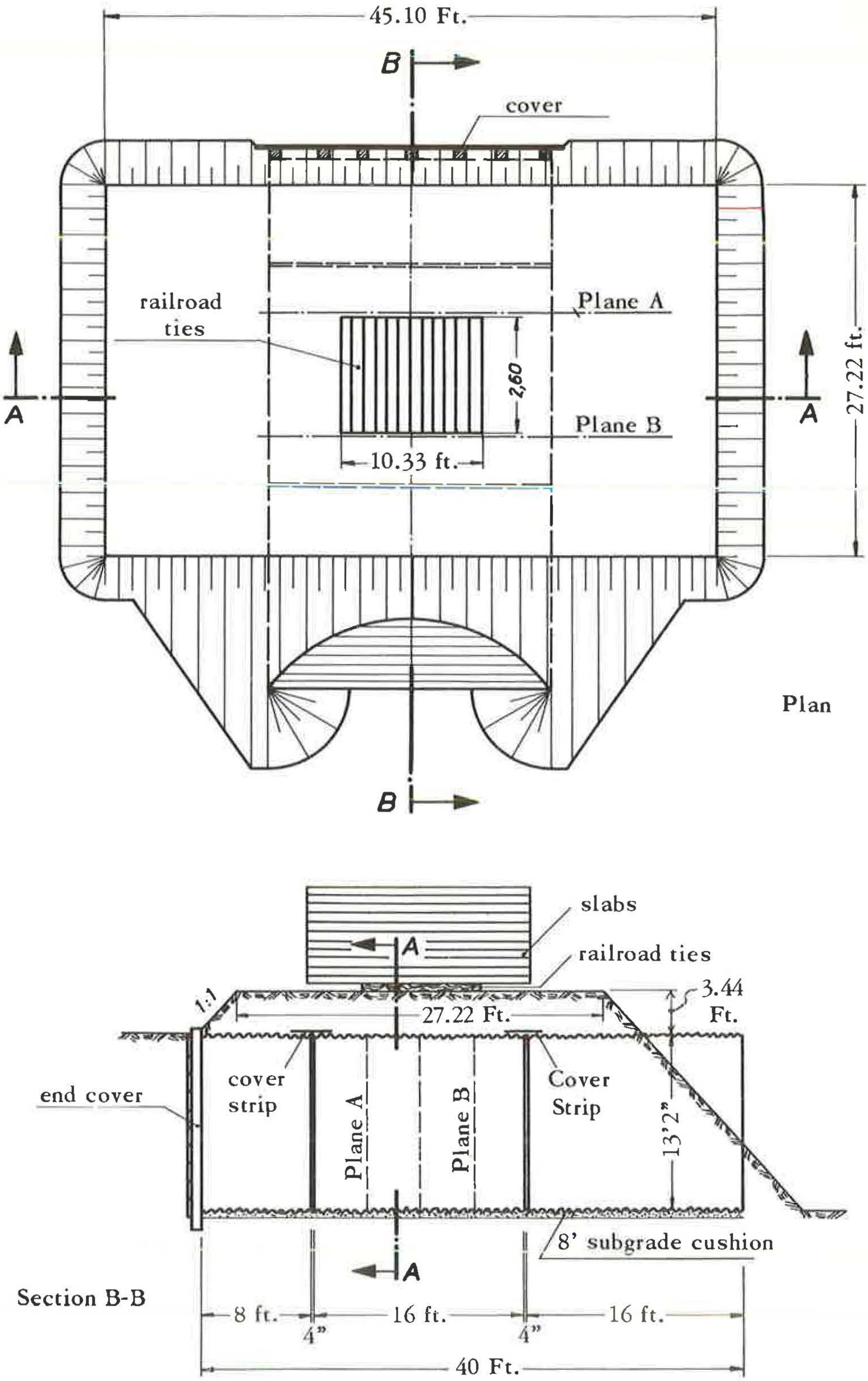


Figure 6. Setup for live-load-test.

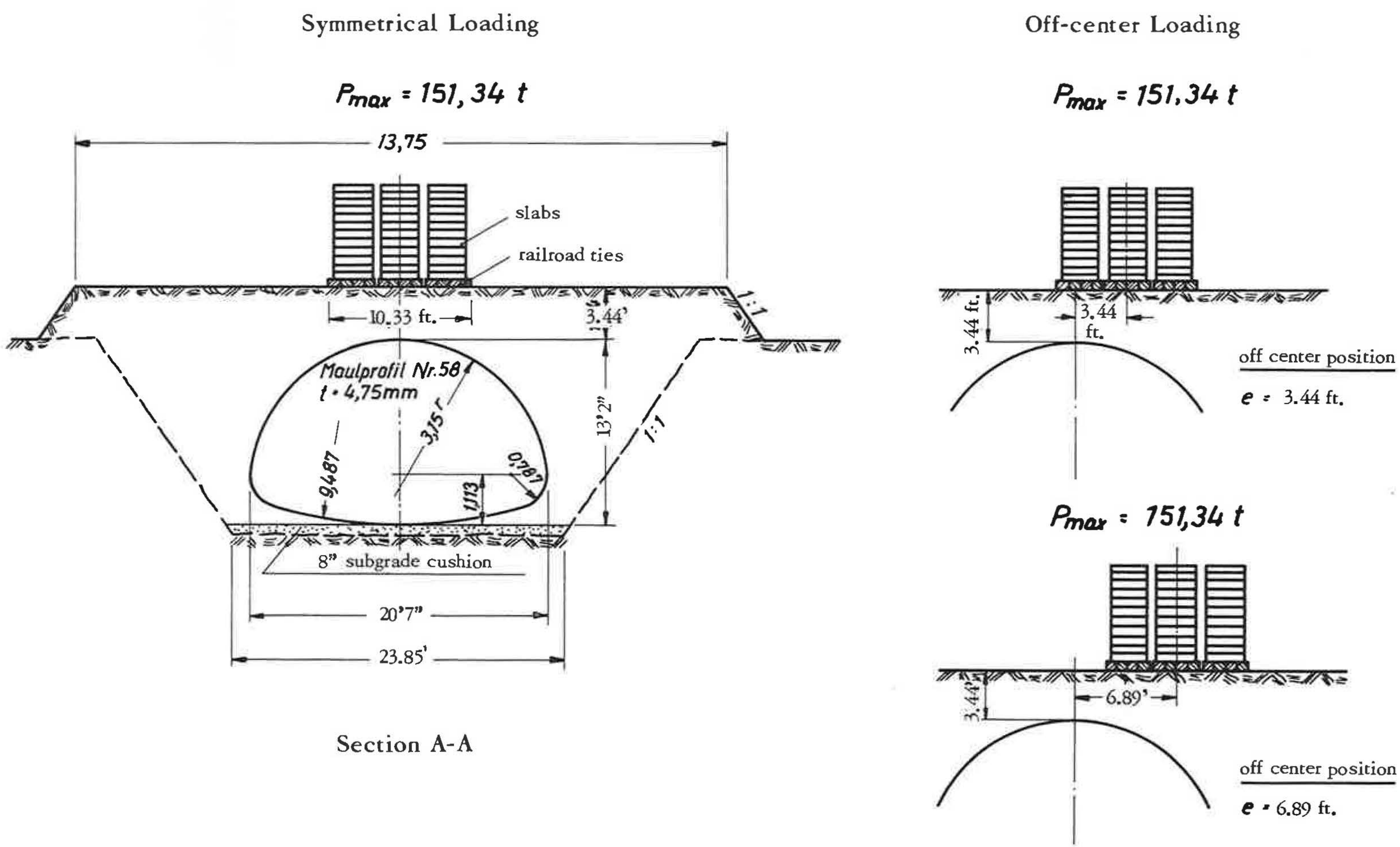


Figure 6. Continued.

strain was determined by establishing the difference between the measured value and a base reading taken before loading (Figs. 3 and 4).

Deformation was measured photographically. Measuring lights, with a black dot in the middle of each bulb, were installed next to and between the strain gage points. Zeiss-Jena phototheodolites registered the displacements of the dots so that the magnitude and direction of the displacements could subsequently be determined from the photographs by a stereocomparator. This procedure permitted indication of the movement of measuring points with an accuracy of 0.02 in. To be able to determine pipe-arch deformation on the spot at any time, additional gage pins were placed in the crest, the invert and on the side walls. By means of a theodolite, the displacement of the leveled points could then be read off immediately.

LIVE-LOAD TEST

After installation of the instruments required for strain and deformation measurements, backfilling and covering operations were begun on June 18, 1963. During backfilling and earth tamping, considerable vertical deflection of the pipe arch was noted. With 3.44 ft of cover, the horizontal diameter had decreased by 2.64 in., and there was an elongation of 3.86 in. in the vertical diameter; i.e., the crest was pushed up by 3.62 in. while the invert settled 0.24 in. (zero reading: pipe free in trench). At the same time, there was a considerable increase in the extreme fiber strains and, consequently, the extreme fiber stresses.

Extreme fiber stresses due to backfilling and cover of 3.44 ft over pipe center (Fig. 5) were in crest point IV σ_1 = +31,931 psi, σ_3 = -39,299 psi, and in crest point X σ_1 = +28,759 psi, σ_3 = -30,466 psi. To support the load, 8.53-ft long railway ties were placed side by side on the surface grade parallel to the pipe-arch axis, covering a

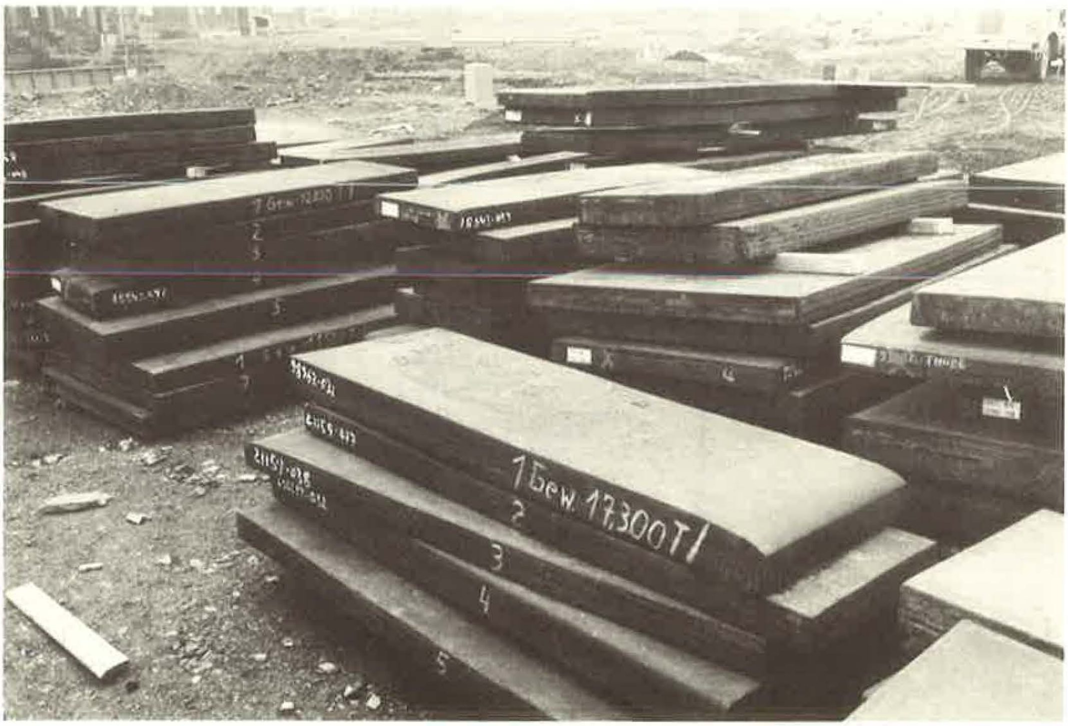


Figure 7. Slabs ready to be placed on structure.

width of 3 × 3.44 = 10.33 ft, or about half the clear span of the pipe arch. Thus, the supporting area was 8.53 × 10.33 ft = 88.11 sq ft (Fig. 6).

Steel slabs from the August-Thyssen steel mill were used as a load (Fig. 7). For the live-load test, the Munich Central Office of Federal Railways had determined that 50 tons was the most severe load a structure of similar span might have to carry. This represents the load transmitted by a two-axle railway car, each axle weighing 25 tons. Considering a safety factor of 3, the total load for the live-load test was to be 150 tons. This load was to be applied by three independent slab piles placed axially on the pipe arch and also on one side only, since for arched supporting structures, off-center loading will often constitute the severest condition. For the loading-to-failure test the steel slabs were also used as a load. As the actual carrying capacity was unknown, a maximum load of 1,000 tons based on a computation with the ring compression formula was planned to be applied for this test. With a supporting area of 88.11 sq ft, this load could be imposed only by piling the slabs crosswise.

On June 21, 1963, a cover height of 3.44 ft, or one-sixth the span, was reached, so that loading could begin. The steel slabs weighing between 5 and 10 tons, weighed in advance, were positioned on the ties by a crane. Strain and deformation were measured at 25-ton load increments. These measurements showed that deflections and strains resulting from the overhead load were small in comparison to those that had resulted from backfilling and acted in the opposite direction. To start with, a load of 151.32 tons was applied axially over the pipe arch (Fig. 8). Until then, no marked changes in deflections and strains appeared. Results from application of this load (Fig. 9) were as follows:

Plane 1—downward deflection in crest 0.374 in.;
Plane 2—downward deflection in crest 0.339 in.;

Figure 8. Live-load test, test structure with 151.32-ton axial load.

Date of reading: 27. Juni 1963

Time : 6 30 Uhr

load in tons : 151

Remarks : reading taken after 151 ton load had remained unchanged for 6 days

Scale of profil 1:30

Scale of deflection 1:1

measurements indicated in millimeters

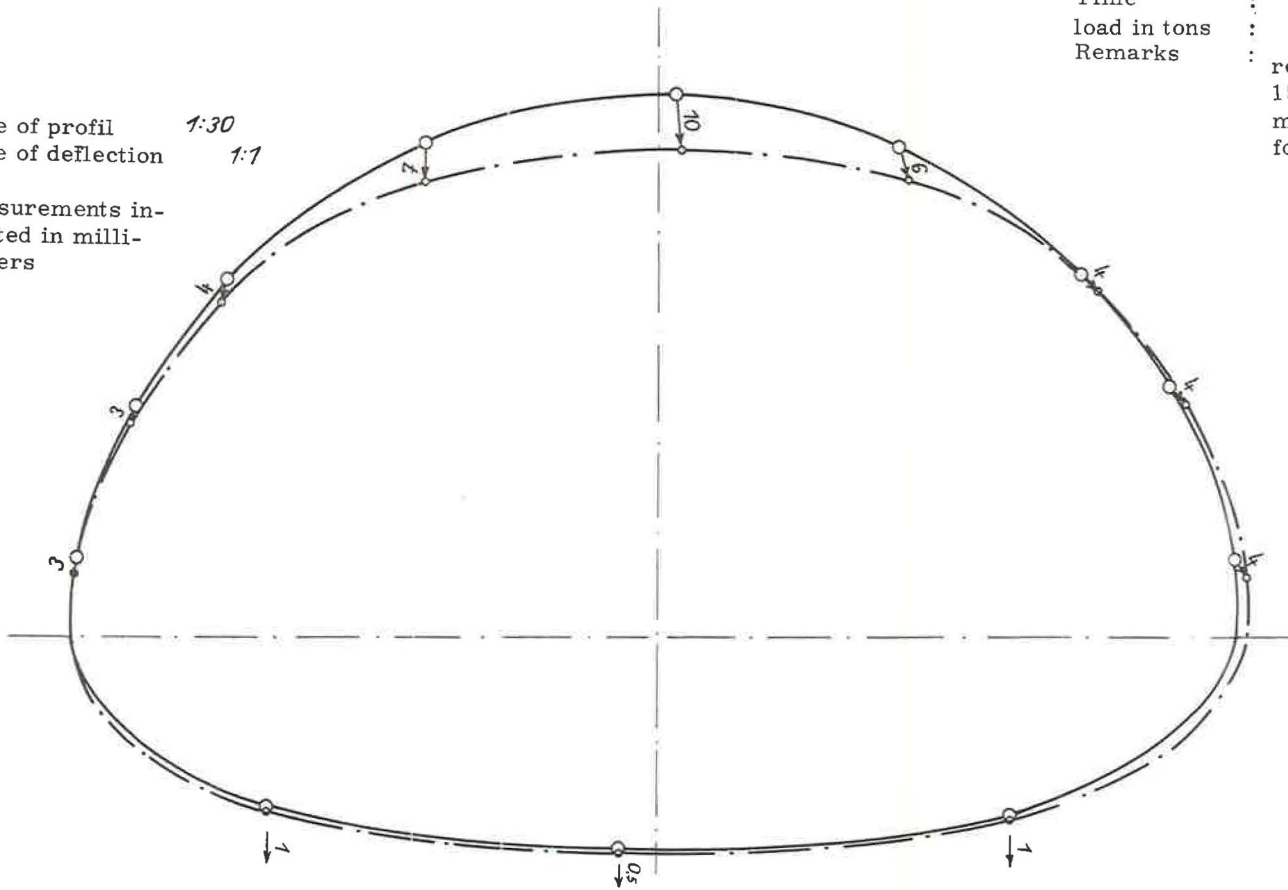


Figure 9. Live-load test, deformation measurement.

live-load-test
applied load P = 151, 32 t
readings 6 to 12

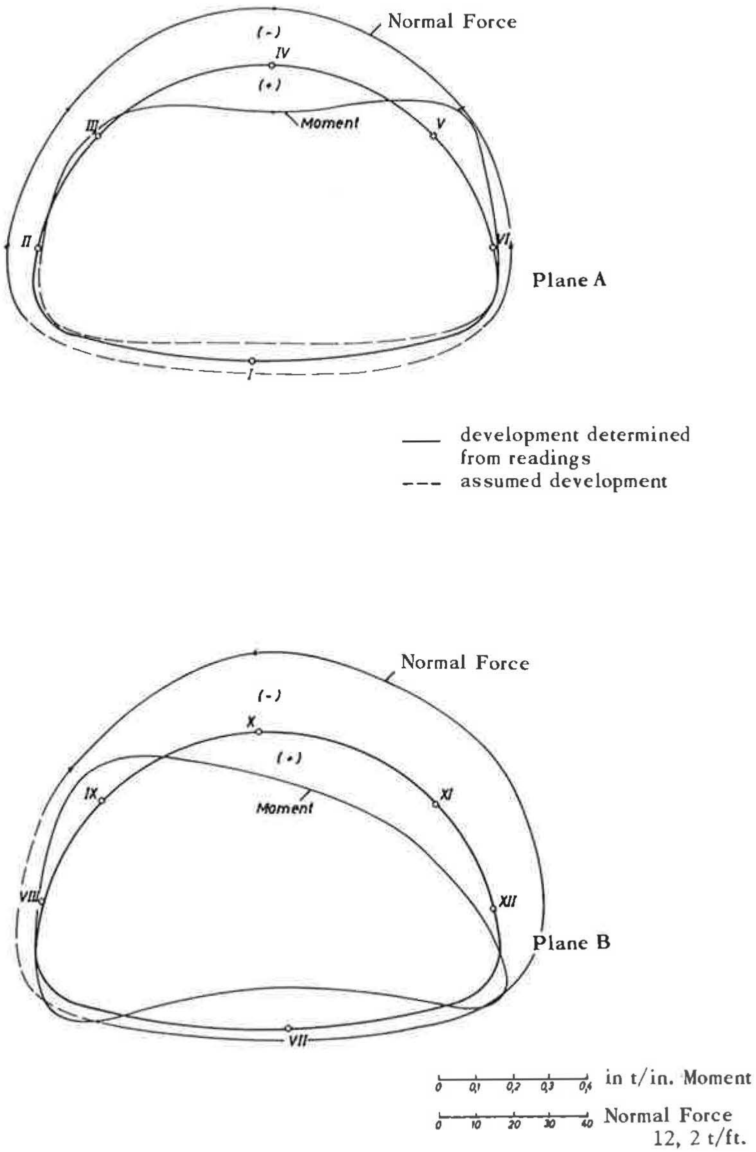


Figure 10. Development of normal forces and bending moments from reading taken.

Changes in extreme fiber stresses (Fig. 10) in crest point IV σ_1 = -6,088 psi, σ_3 = -626 psi; and in crest point X σ_1 = -7,353 psi, σ_3 = -2,717 psi.

The load of 151.32 tons was left in place for 6 days, and readings were taken each day. Both the strain and deflection measurements varied at different times. During the 6 days and, indeed, during loading operations, there was a shift in soil pressures which, however, died away after a few days. Thus, practically no further change in deflection could be noted on the third day. It was also observed that deflections and strains were not symmetrical, although the gage points were located symmetrically and

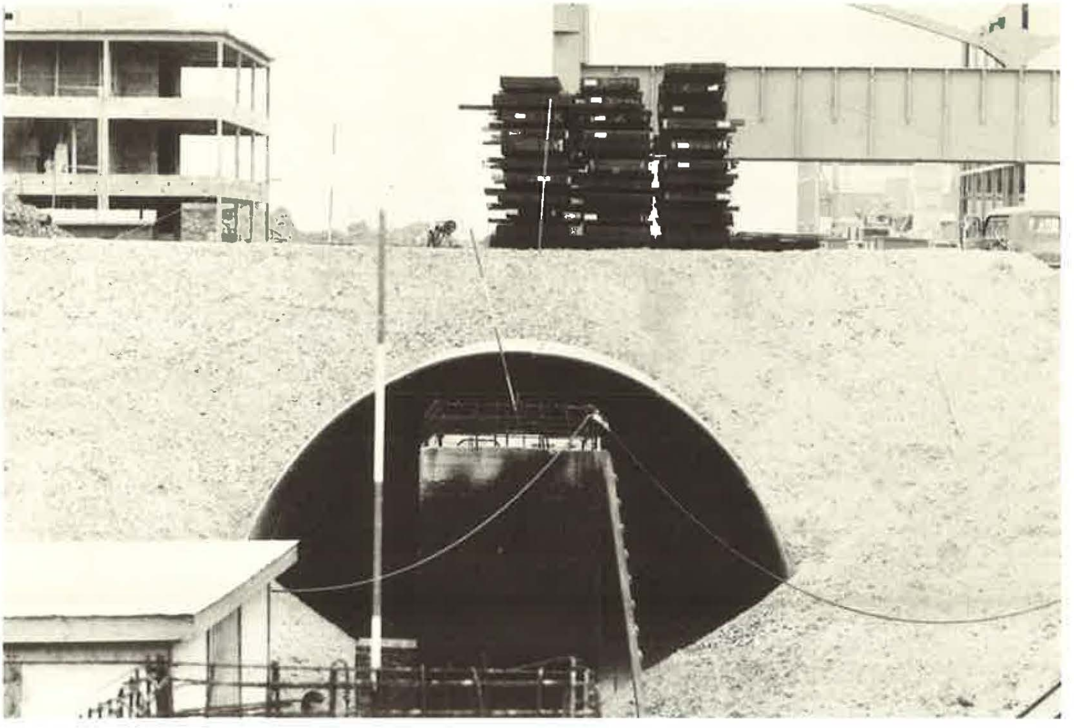


Figure 11. Test structure under load of 151.32 tons applied in 3.44-ft off-center position.

Figure 12. Test structure under load of 151.32 tons applied in 6.89-ft off-center position.

care had been taken to place the load as near as possible over the center. Apart from inevitable off-center loadings, this development may be traced primarily to non-uniform backfill material. The deflections caused by backfilling were only slightly diminished under this load.

After 6 days the load was removed to one side by shifting the slabs (Fig. 11). First one of the outer piles was moved to the other side, and after that the center pile. Loading was then 6.89 ft off-center (Fig. 12). Only very slight strains and deformations

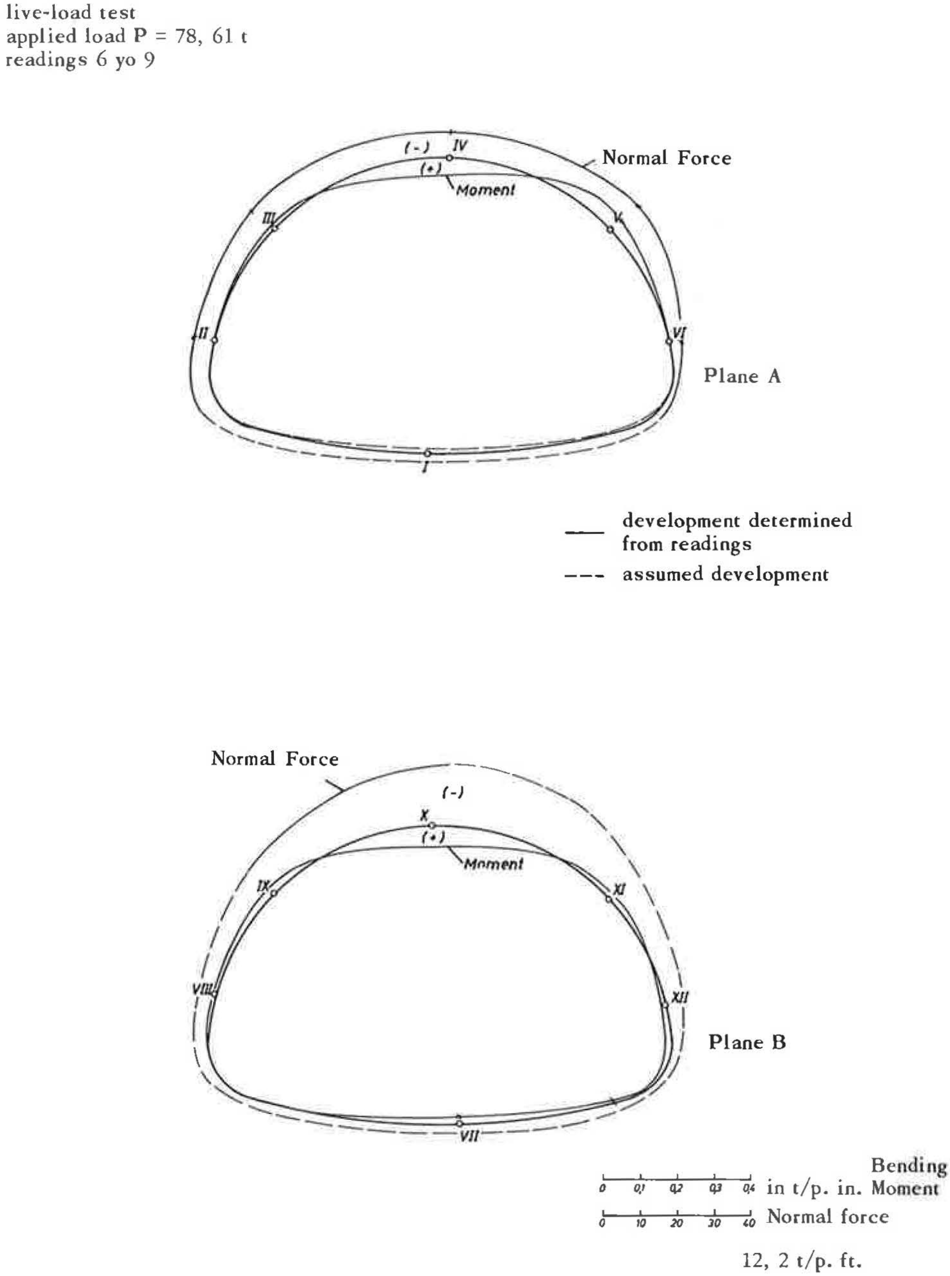


Figure 13. Development of normal forces and bending moments from reading taken.

were caused by this load shifting. The pipe-arch crest which had moved to the right by 0.04 in. under the axial load, moved back 0.12 in. to the left with the load in the off-center position.

Results of Strain Measurements

Figures 5, 10, and 13 show the results of the strain measurements made in the course of the live-load test. In some gage points several readings reveal that strain development along the section height is not linear. This is not in agreement with Euler-Benouilli's hypothesis that sections will remain even, which generally is considered true enough also in the plastic sphere. This strain pattern deviating from linearity may be explained in that the pipe-arch wall of corrugated metal sheet represents a plane load-bearing structure consisting of curved half-sections of a cylinder. Since the rigidity of the pipe arch along the centerline is very small as compared with that across the axis, the load will be primarily distributed along the ring, and the supporting structure may be regarded as a curved beam with a corrugated cross-section. Under concentrated pressures induced by rock in the backfilling material, however, the metal wall may in places react as a plane load-carrying structure, thus developing localized strains opposed to the hypothesis of linearity of strains along the section height.

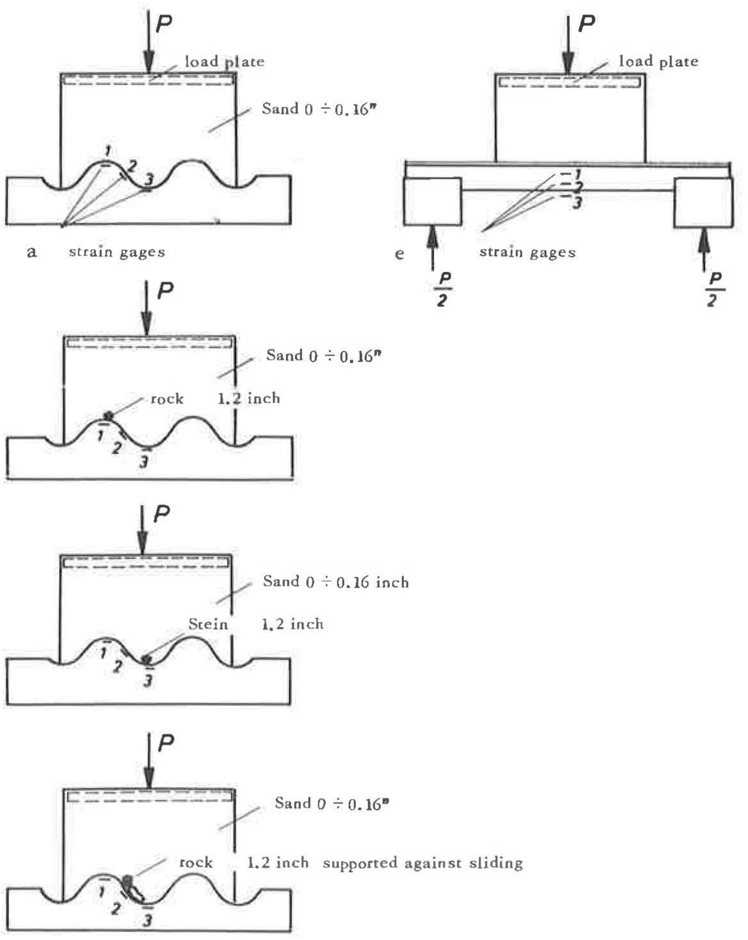


Figure 14.

The same effect also became apparent in tests conducted at the Institute for Statics and Steel Construction, for the purpose of clarifying this question. An Armco-Thyssen corrugated metal sheet was submitted to bending stress in a manner shown in Figure 14. These tests proved that the constantly acting lateral pressure, resulting from the corrugated profile, cannot possibly be the reason for the nonlinear strain development when using uniform granular material.

Although even localized pressures in the wave crest or the wave trough hardly affected the linearity, lateral pressures resulting from the presence of rock in the backfilling material would cause strain developments opposed to linearity.

For the determination of stresses and sectional forces from the strains, it was assumed that all strains were within the range of elasticity. Due to the low bending strength of the pipe arch, the acting bending moments will produce high extreme fiber strains which may exceed the yield point. Particularly during the placement of fill, considerable bending moments will be encountered in the absence of support by surrounding soil. Since the instruments for strain measuring were not installed until erection was completed, the yield development in the respective places could not be registered. During backfilling and loading operations, stresses induced on the pipe arch changed several times. Changes of this kind occurring in the plastic sphere will generate residual stresses that are superposed on the load stresses. It is not possible to study the stress pattern accurately, since stresses during erection are unknown and, furthermore, the stress curve will fluctuate as various loads are being applied or removed. The best results are obtained when the stresses are derived independently from the strains introduced at each individual load increment without considering initial stresses. This procedure was followed when evaluating the measurements. Even if it were possible to register all the influences affecting the strain measurements, a summation of strains or stresses would not provide much clarity inasmuch as the effects of the individual load increments would be concealed.

The computed stresses and sectional forces shown in the tables as "stresses from readings" and "sectional forces from readings" will, therefore, only approximately represent the forces to which the pipe arch was subjected but will permit qualitative conclusions as to the behavior of the structure under loading conditions.

In addition to the sectional forces resulting from backfilling and loading, which are shown in the tables, a rough estimation may indicate the range of sectional forces developing by erection. The pipe arch was erected by attaching and bolting together pipe elements of differing curvature, starting from the invert and continuing toward the sides. Due to inevitable production tolerances when curving the plates, and as a result of the weight of the structure, the rings consisting of individual sections can be closed only by pulling the open ends together or by parting overlapping ends.

Since it is impossible to determine the necessary amount of adjustment after assembly has been completed, the rough estimate of stresses during erection of the pipe arch will be based on an empirical adjustment value of $C_2 = \pm 1.64$ ft. In the most unfavorable instance, this adjustment and the effect of the pipe-arch weight will cause the following bending moments to arrive at the points marked (Fig. 15):

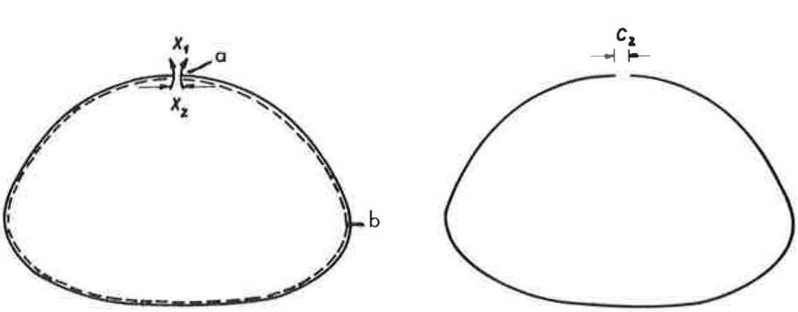


Figure 15. Sectional forces resulting from adjustment of ring ends.

Date of reading : 19. Juni 1963

Time : 18⁰⁰

load in tons : ./.

Remarks : Backfilling and cover up to about 3/4 rise

Scale of Profile 1:30

Scale of deflection 1:5

measurements indicated in millimeters

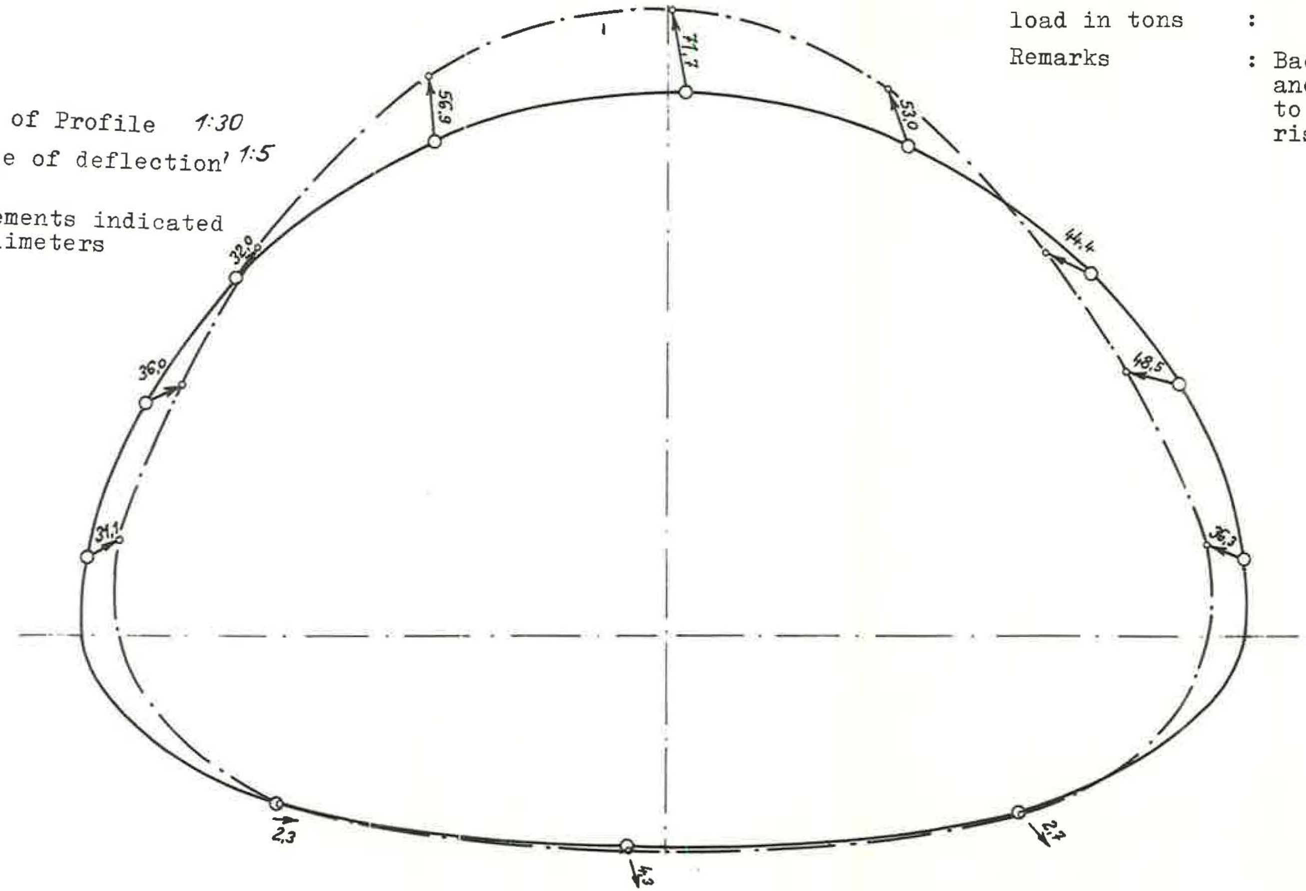


Figure 16. Live-load test, deformation measurement.

Date of reading: 21. Juni 1963

Time : 8 00

load in tons: 0

Remarks : Base reading prior for loading test

Scale of Profil 1:30

Scale of deflection 1:5

measurements indicated in millimeters

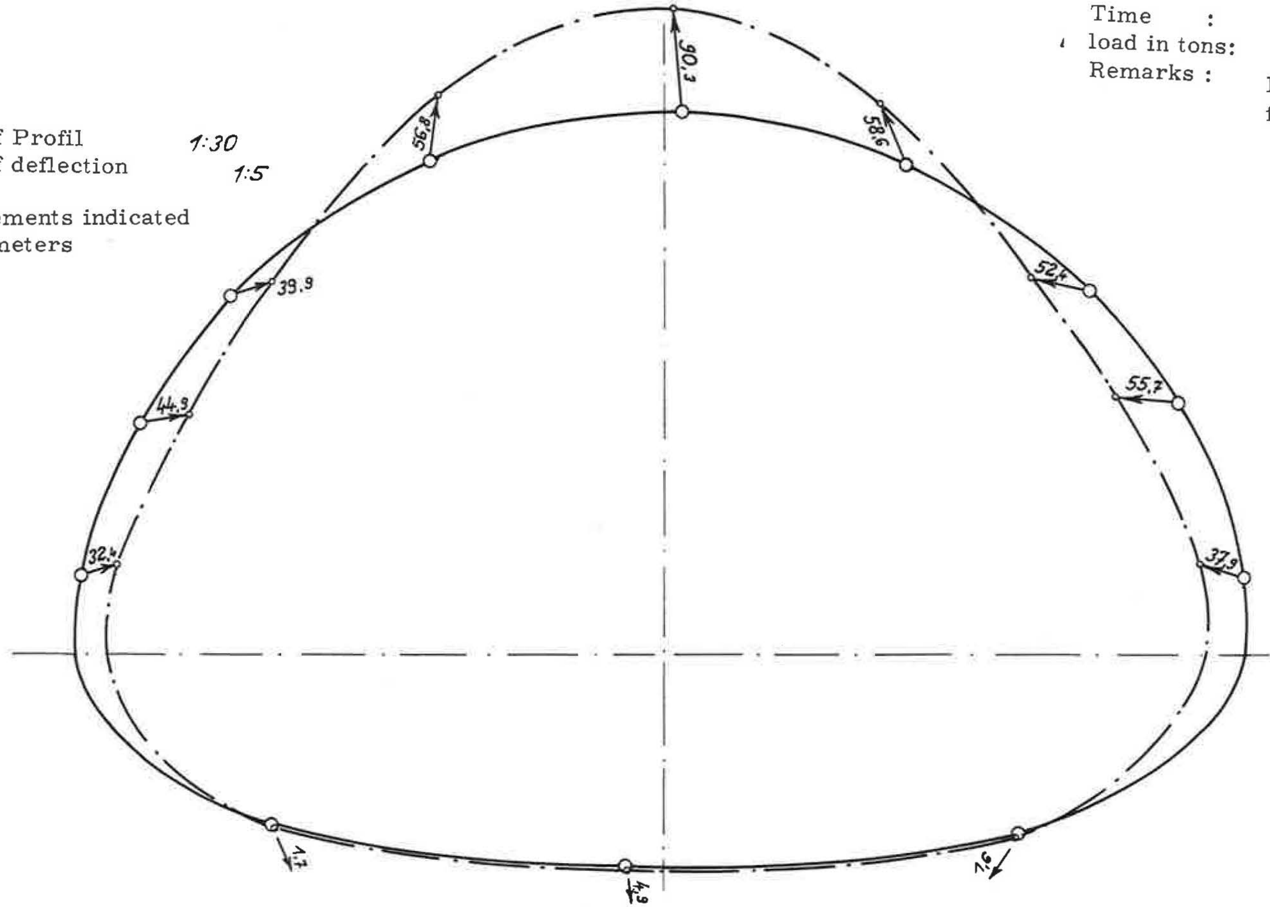


Figure 17. Live-load test, deformation measurement.

Date of reading : 21. Juni 1963
Time : 17⁰⁰ Uhr
load in tons : 151
Remarks : readings immediately after application of load

Scale of Profil: 1:30
Scale of deflection: 1:1

measurements indicated in millimeters

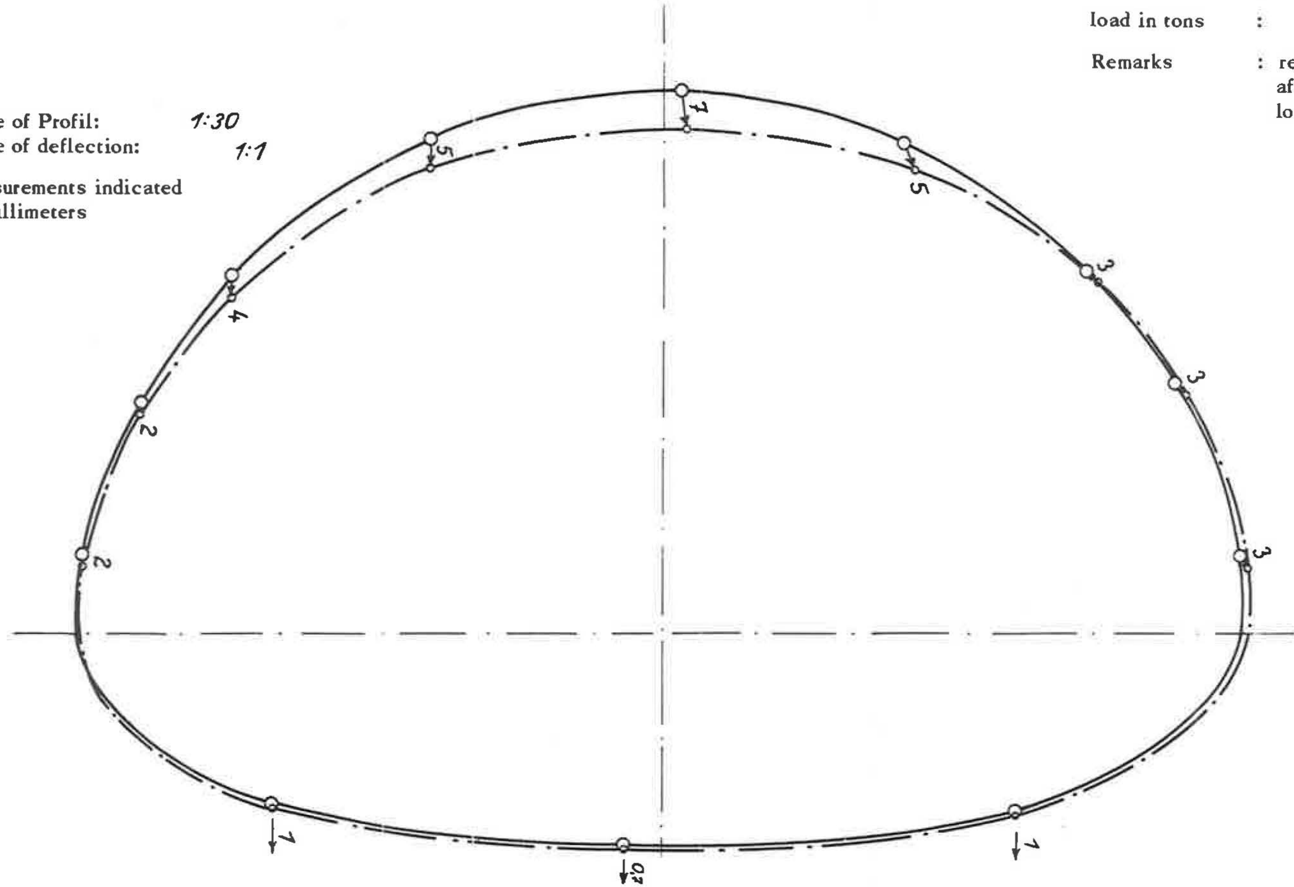


Figure 18. Live-load test, deformation measurement.

Date of reading: 28. Juni 1963
Time : 7⁰⁰ Uhr
load in tons : 151
Remarks : Reading after 17 hours off-center laoding

Scale of Profil 1:30
Scale of deflection 1:1

measurements indicated in millimeters

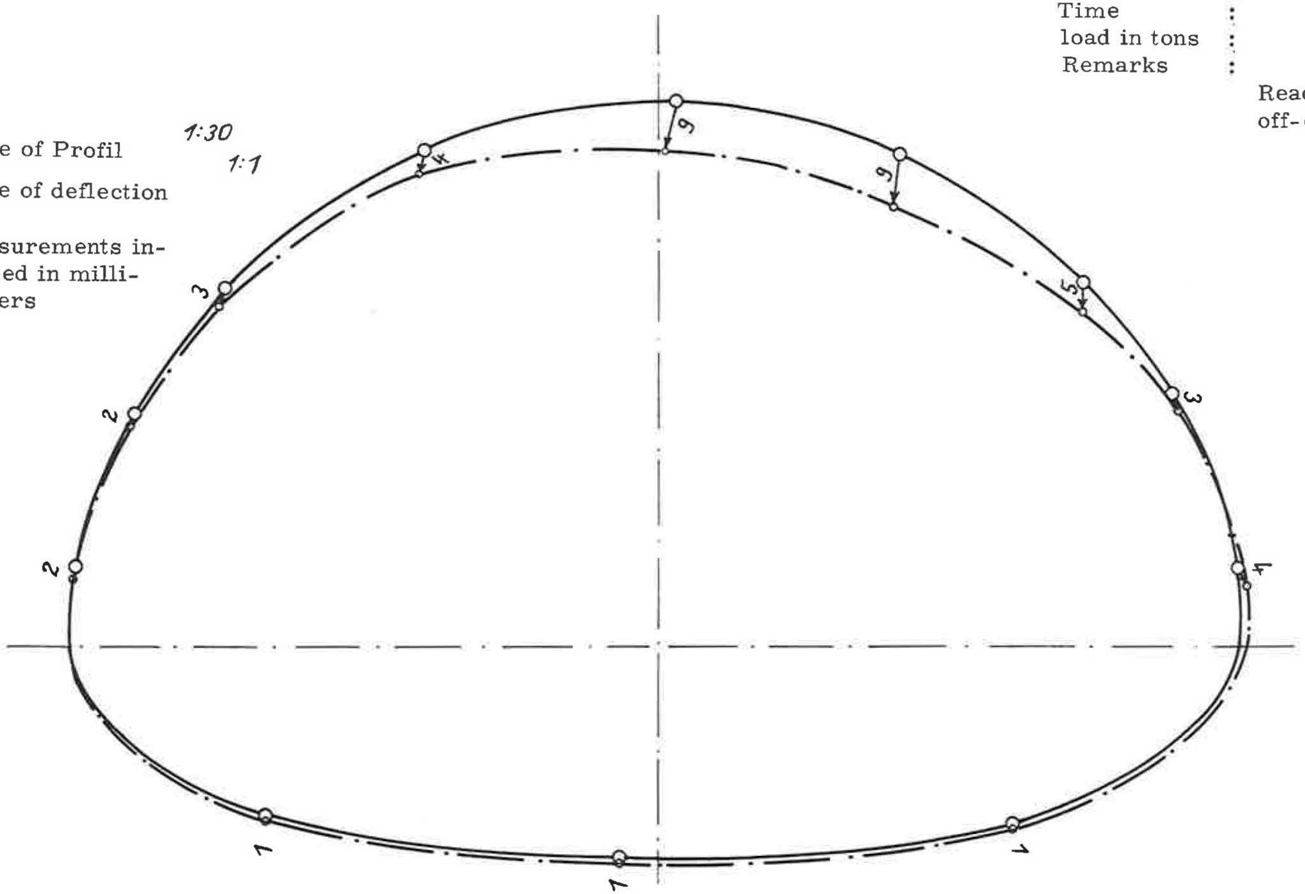


Figure 19. Live-load test, deformation measurement.

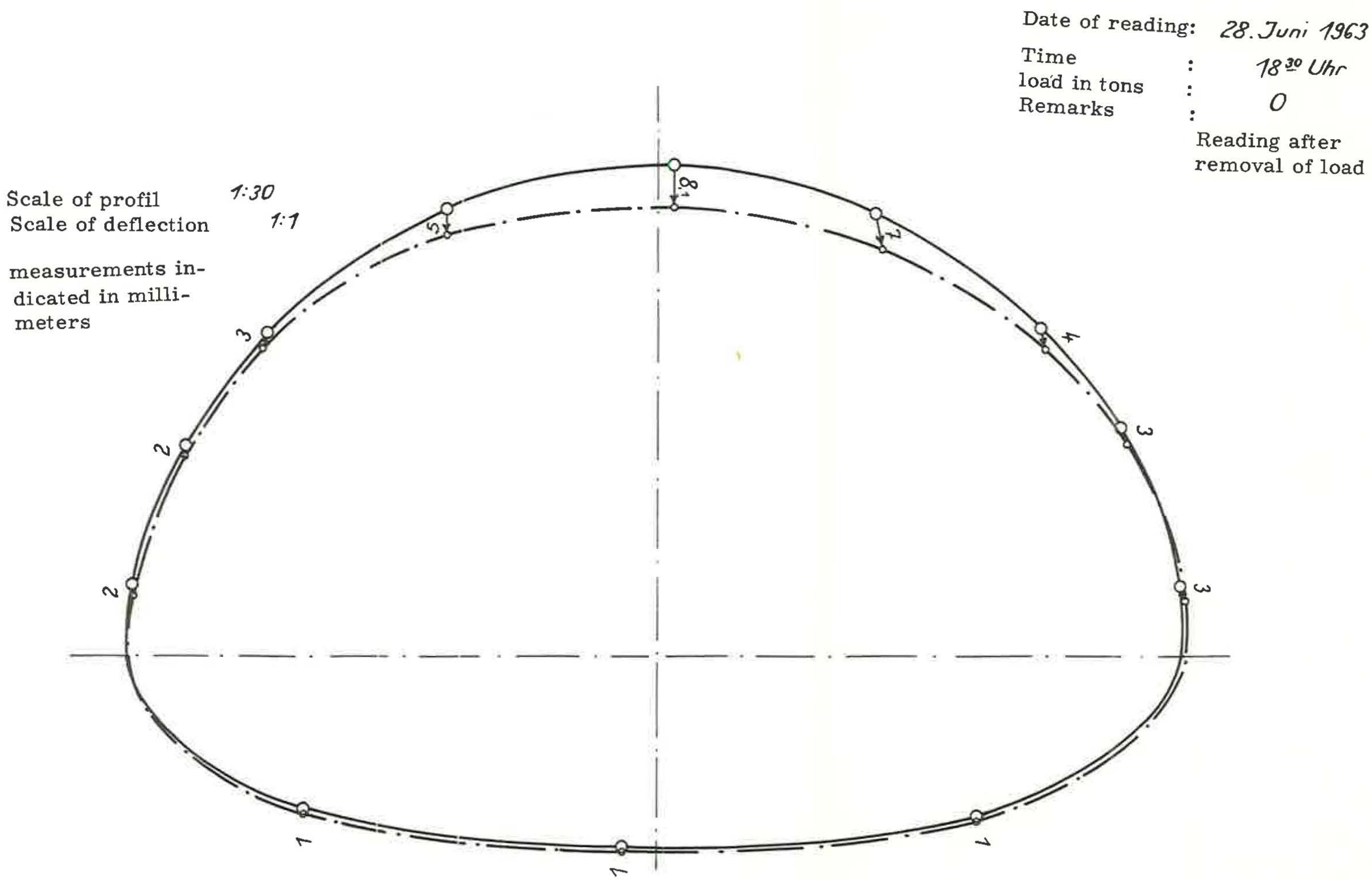


Figure 20. Live-load test, deformation measurement.

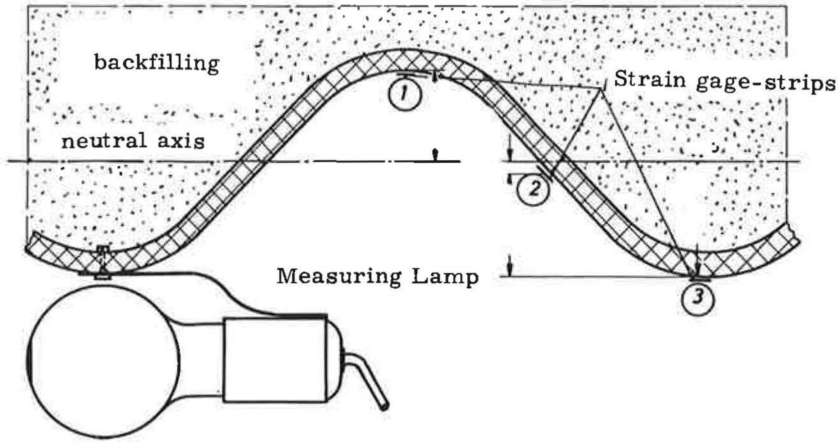


Figure 21. Positioning of strain gage strips and measuring lamp.

Point a

$$M_a = \pm 2,470 \text{ lb-in./in.}$$

Point b

$$M_b = \pm 1,367 \text{ lb-in./in.}$$

The stresses thus developed are as follows (the effect of normal force having been neglected as insignificant):

Point a

$$\begin{matrix}\text{min.}\\ \text{max.}\end{matrix}\ \sigma_a = \pm \frac{2,470}{0.0989} = \pm 24,975 \text{ psi}$$

Point b

$$\begin{matrix}\text{max.}\\ \text{min.}\end{matrix}\ \sigma_b = \pm \frac{1,367}{0.0989} = \pm 13,822 \text{ psi}$$

This rough estimate shows that the stresses in the load position "assembly" may become so large that they must be taken into account together with the loading stresses under the service load, when considering the stresses effective in the structure.

Deformation measurements are shown in Figure 9 and Figures 16 through 20. The positioning of the strain gage strips and measuring lamp for this test is shown in Figure 21.

LOADING-TO-FAILURE TEST

On June 28, 1963 preparations began for the loading-to-failure test. The slabs were removed and the pipe arch uncovered to the crest. The unloading caused a slight vertical rise of the crest of 3.47 in. The upper layers were removed for the purpose of conducting the crushing test with undisturbed and unpreloaded soil in the area of largest soil pressures, i.e., directly underneath the applied load. Before the new material was placed, three Heierli pressure cells were installed in backfill in a horizontal place above the pipe arch (Fig. 22). The center cell was placed 4 in. above the crest underneath the center of the loaded area, and the other two were installed at distances of

Figure 22. Installing Heierli pressure cells.

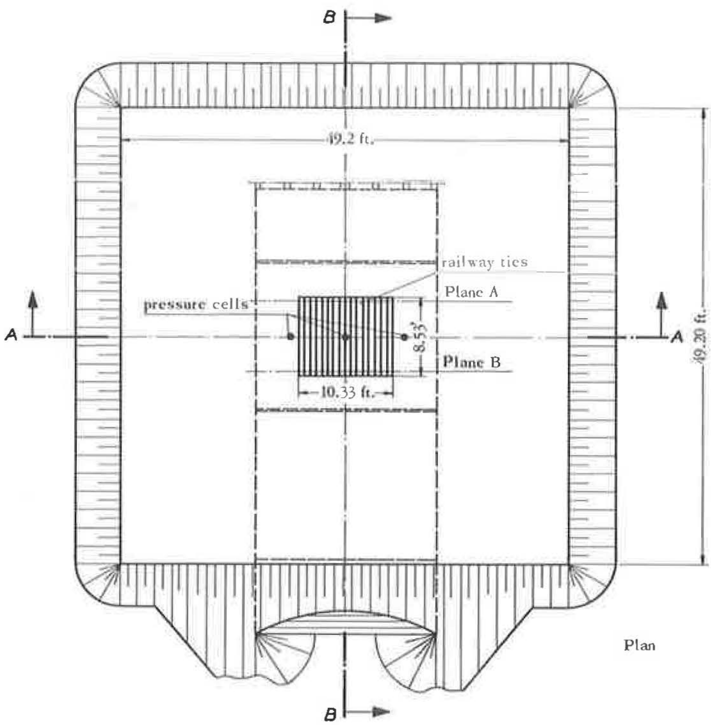
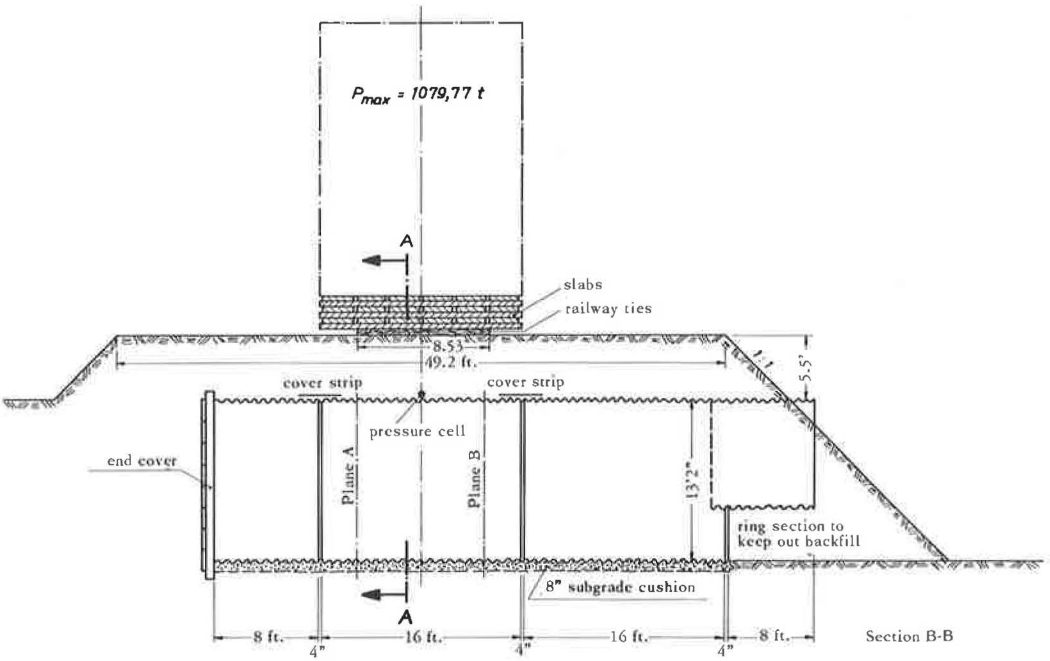


Figure 23. Setup for loading-to-failure test.



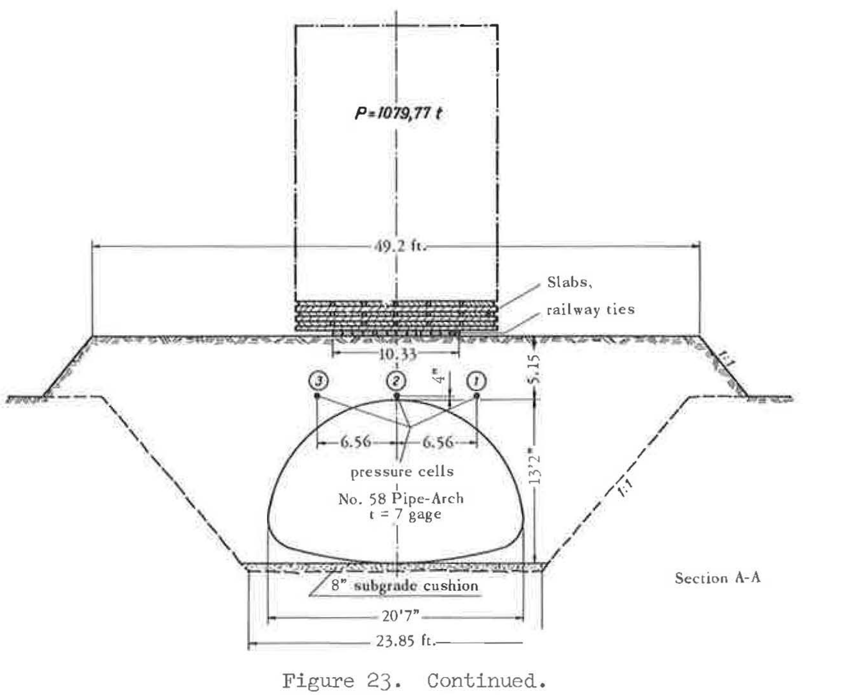


Figure 23. Continued.

6.56 ft left and right of the crest. Measurements are made by pressure gage strips incorporated in pressure cells. For the crushing test, the backfill was extended at the shoulders as a further precaution against subgrade failure. This required the addition of another ring section at the open end of the structure. After the cover height of 5.15 ft for the crushing test was reached, loading was started on July 2, 1963. For higher

Date of reading : 2. Juli 1963

Time : 8^{00} Uhr

Load in tons : 0

Remarks : Original deformation prior to loading

Plane 1

Scale of Profile 1: 30

Scale of deformation 1:5

Measurements indicated in millimeters

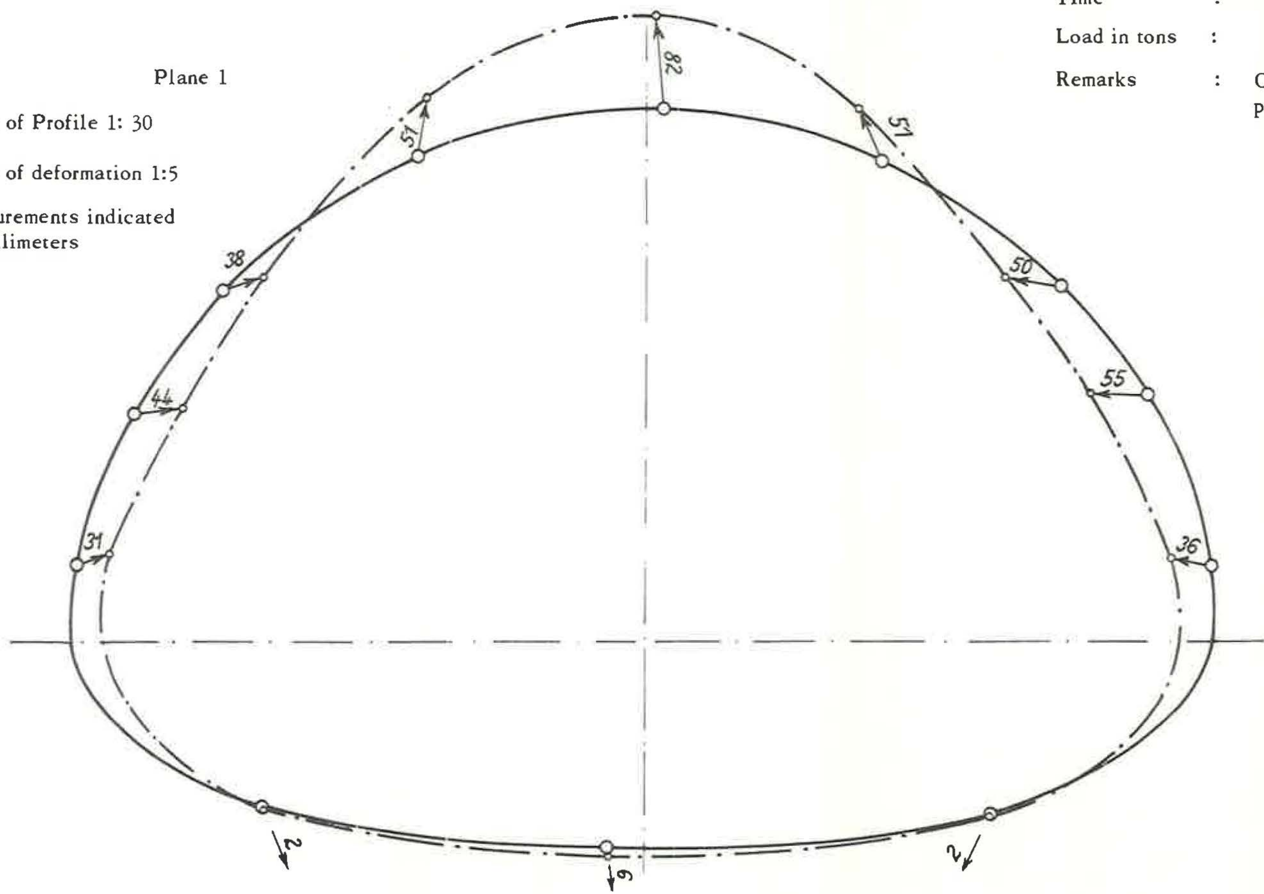


Figure 24. Loading-to-failure test, deformation measurement.

Date of reading : 3. Juli 1962
Time : 8⁰⁰ Uhr
Load in tons : 260,52 to
Remarks : Original deformation prior to loading

Plane 1

Scale of Profile 1:30

Scale of deformation 1:1

Measurements indicated in millimeters

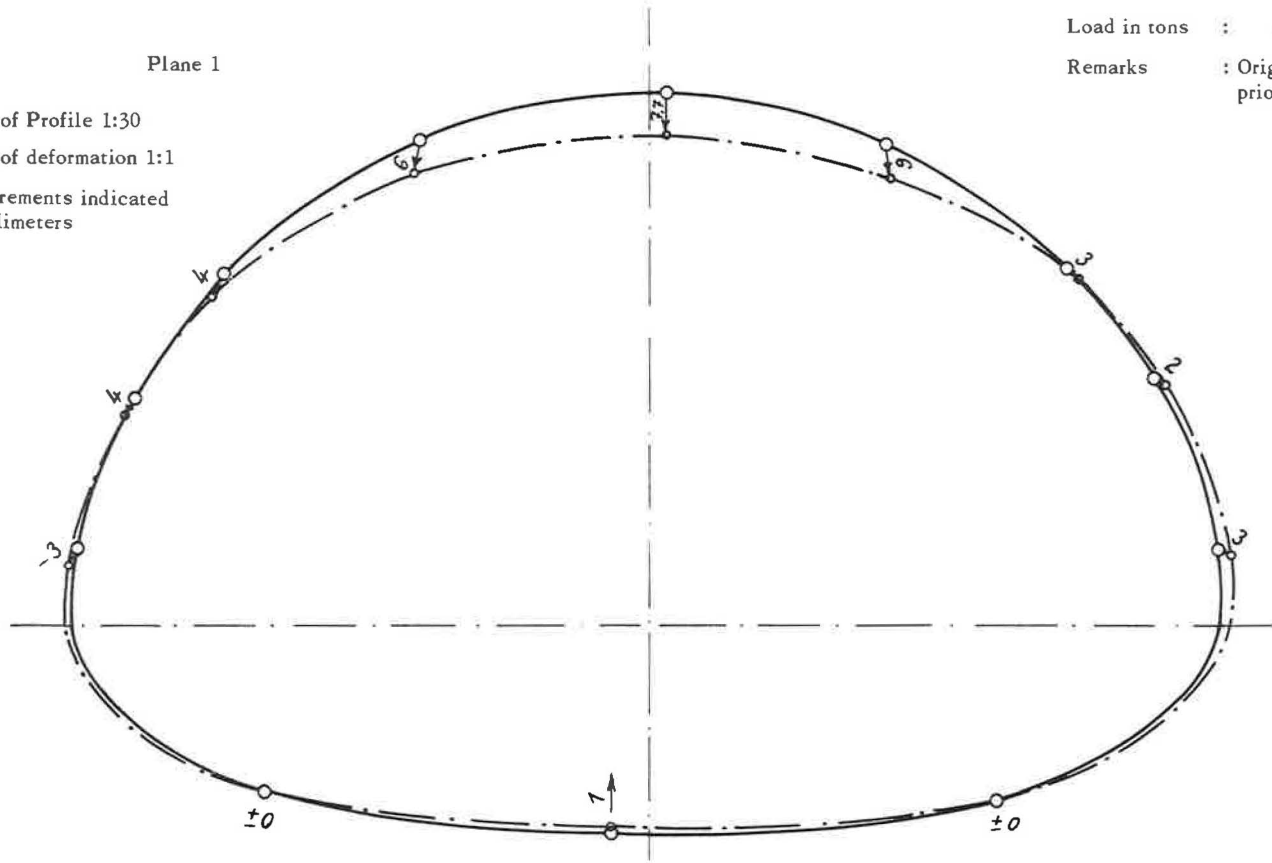


Figure 25. Loading-to-failure test, deformation measurement.

stability of the slab pile, slabs were placed crosswise for this test, using the same supporting area of railway ties as in the live-load test. The setup for the loading-to-failure test is shown in Figure 23.

At the end of the first day, a 260.52-ton load had been applied. As was the case during the live-load test, only slight deflections and strains were introduced by this load. As compared to the conditions before the application of this load (Fig. 24) the following values (Fig. 25) were noted for the most important deformations, stresses and soil pressures at P = 260.52 tons:

Plane 1—0.26-in. vertical deflection in crest;
Plane 2—0.27-in. vertical deflection in crest;

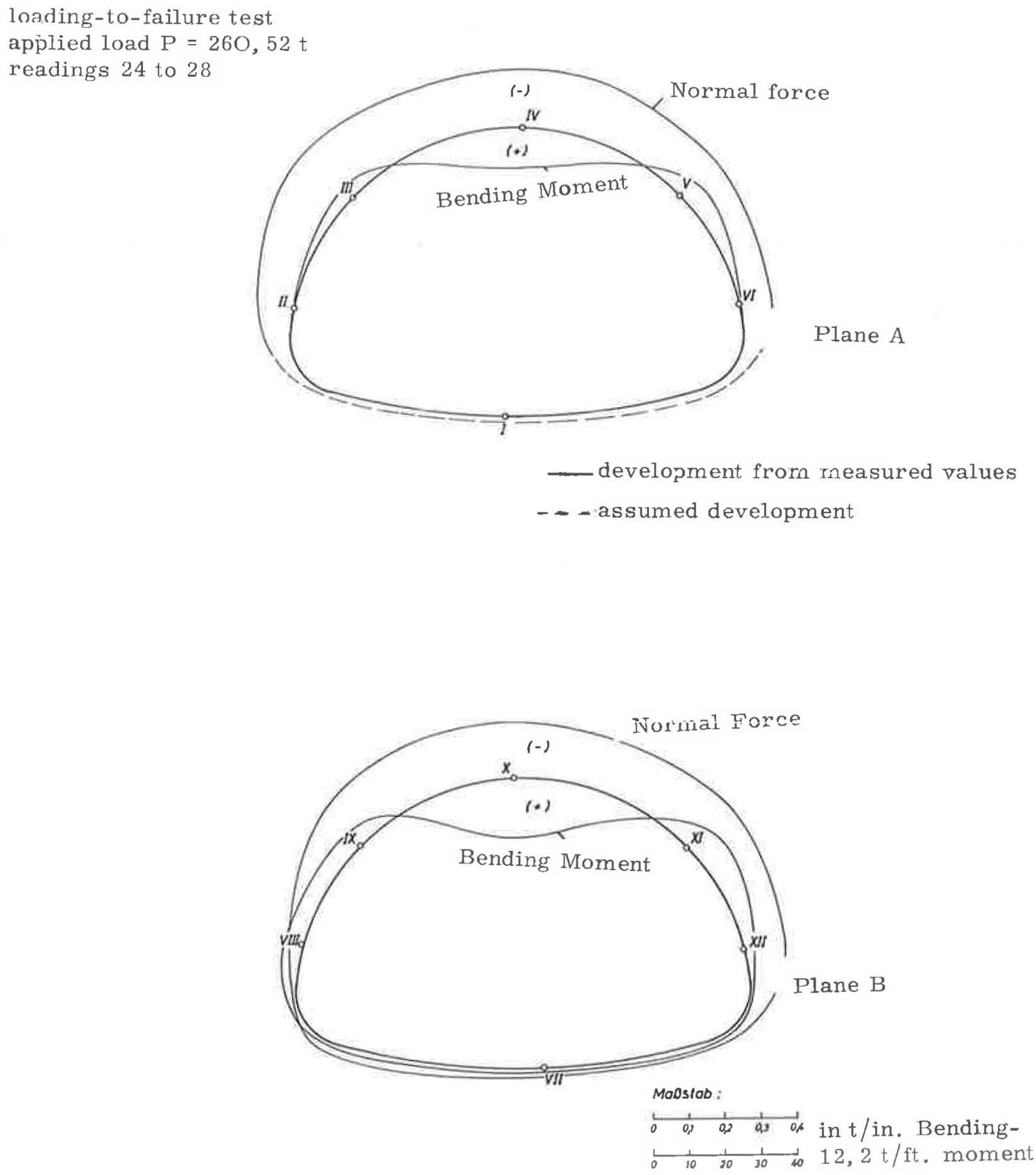


Figure 26. Development of normal forces and bending moments from reading taken.

Change of extreme fiber stresses (Fig. 26) in crest point IV = -6,159 psi = -1,309 psi; in crest point X = -6,841 psi = +28.4 psi; and
Soil pressure at 4 in. above crest—p_2 = 19.34 - 5.83 = 13.51 psi.

The 260.52-ton load was left unchanged overnight. The following morning, a reading revealed the following slight changes under the same load:

Plane 1—0.30-in. vertical deflection in crest;
Plane 2—0.31-in. vertical deflection in crest;

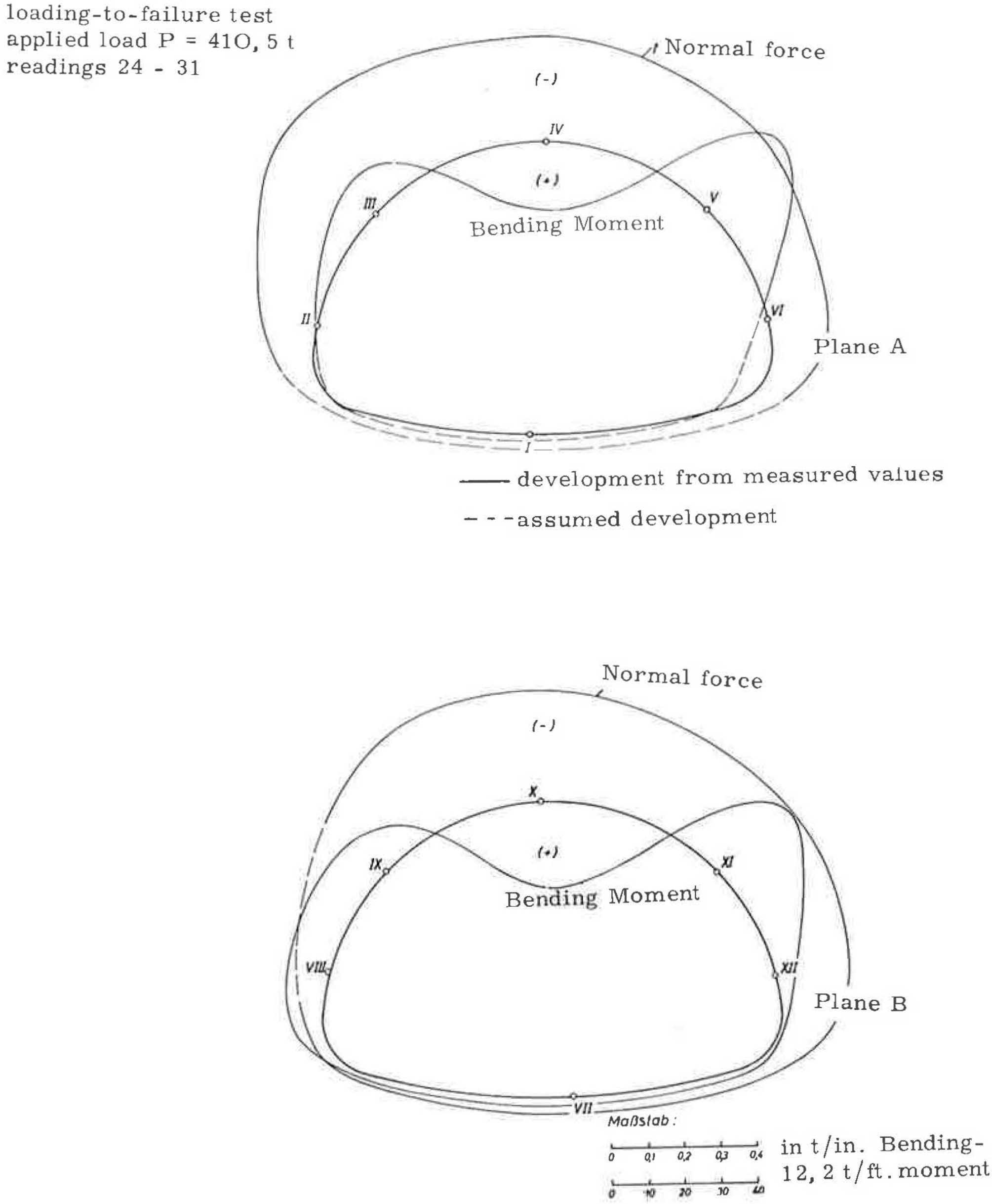


Figure 27. Development of normal forces and bending moments from reading taken.

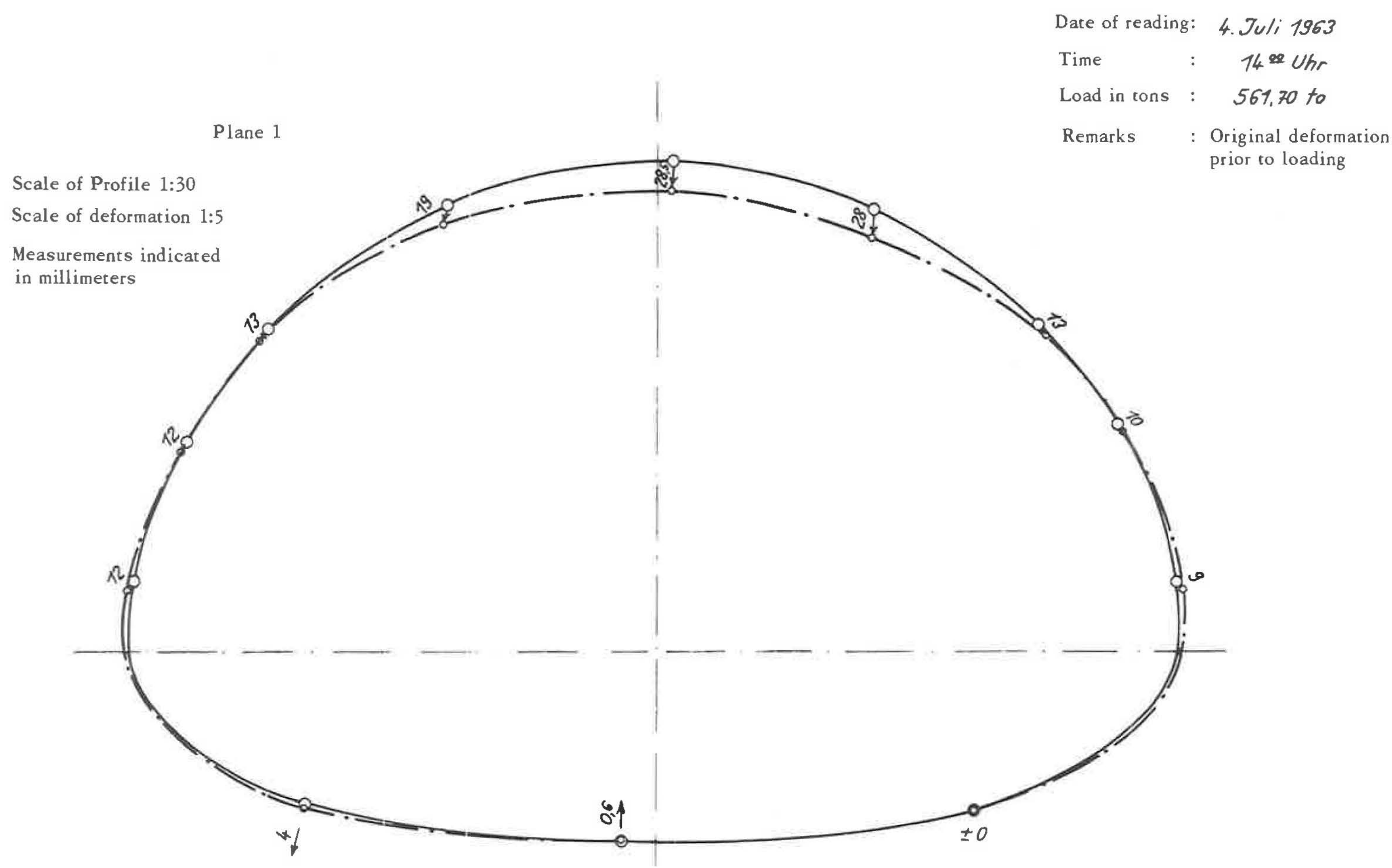


Figure 28. Loading-to-failure test, deformation measurement.

Date of reading : 4. Juli 1963
Time : 19⁵⁰ Uhr
Load in tons : 820 to
Remarks: : Original deformation prior to loading

Plane 1

Scale of Profile 1:30

Scale of deformation 1:10

Measurements indicated in millimeters

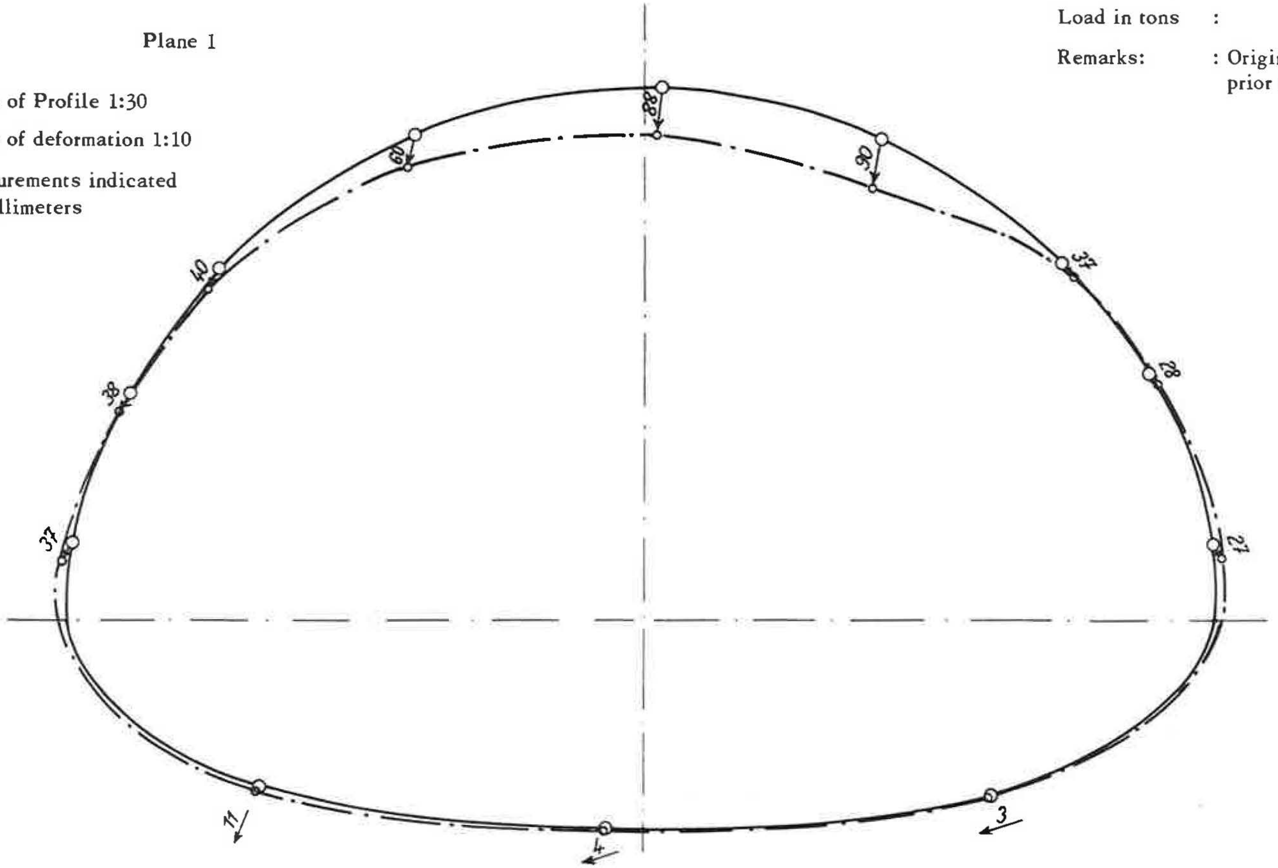


Figure 29. Loading-to-failure test, deformation measurement.

Date of reading : 5. Juli 1963
Time : 17¹⁵ Uhr
Load in tons : 1079,77 to
Remarks : Crushing test was stopped at this point

Plane 1

Scale of Profile 1:30

Scale of deformation 1:10

Measurements indicated in millimeters

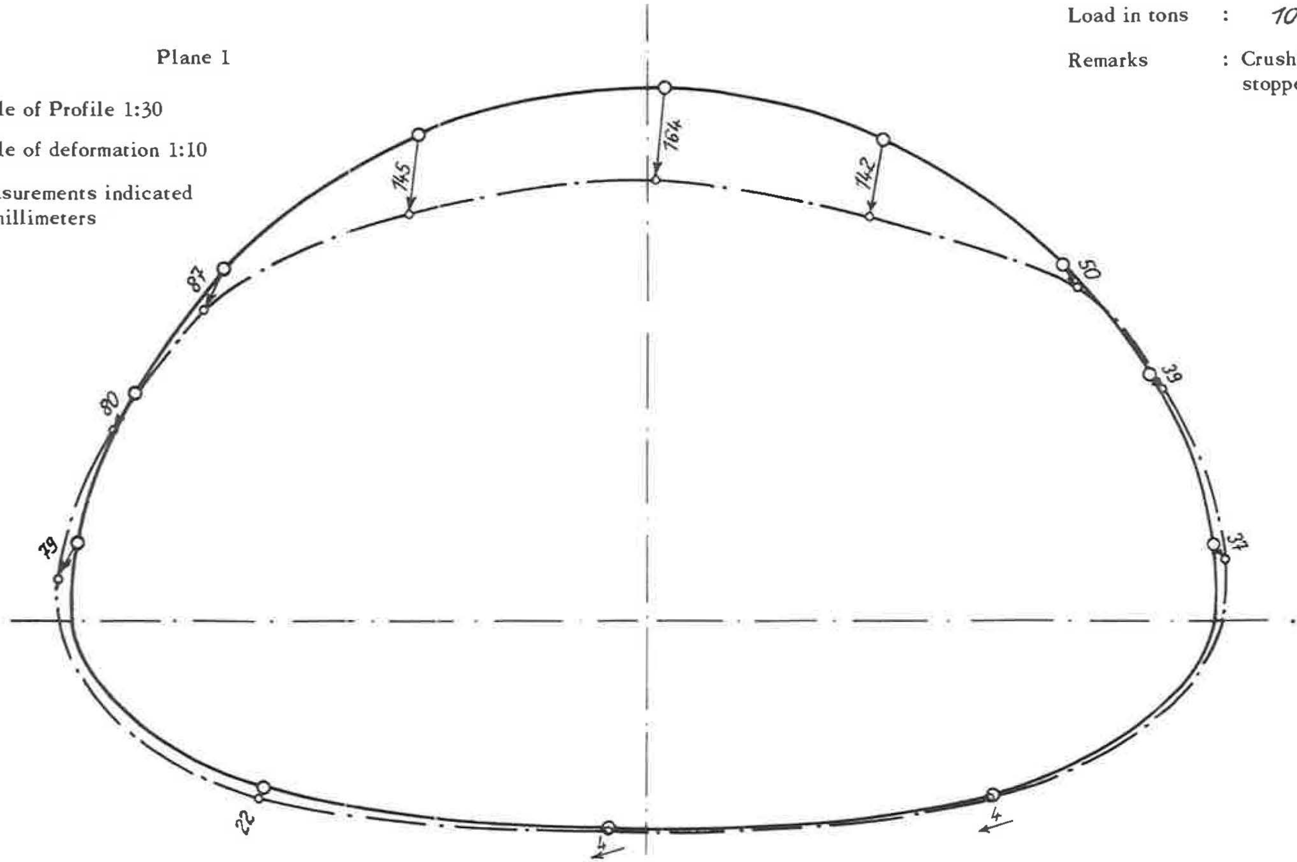


Figure 30. Loading-to-failure test, deformation measurement.

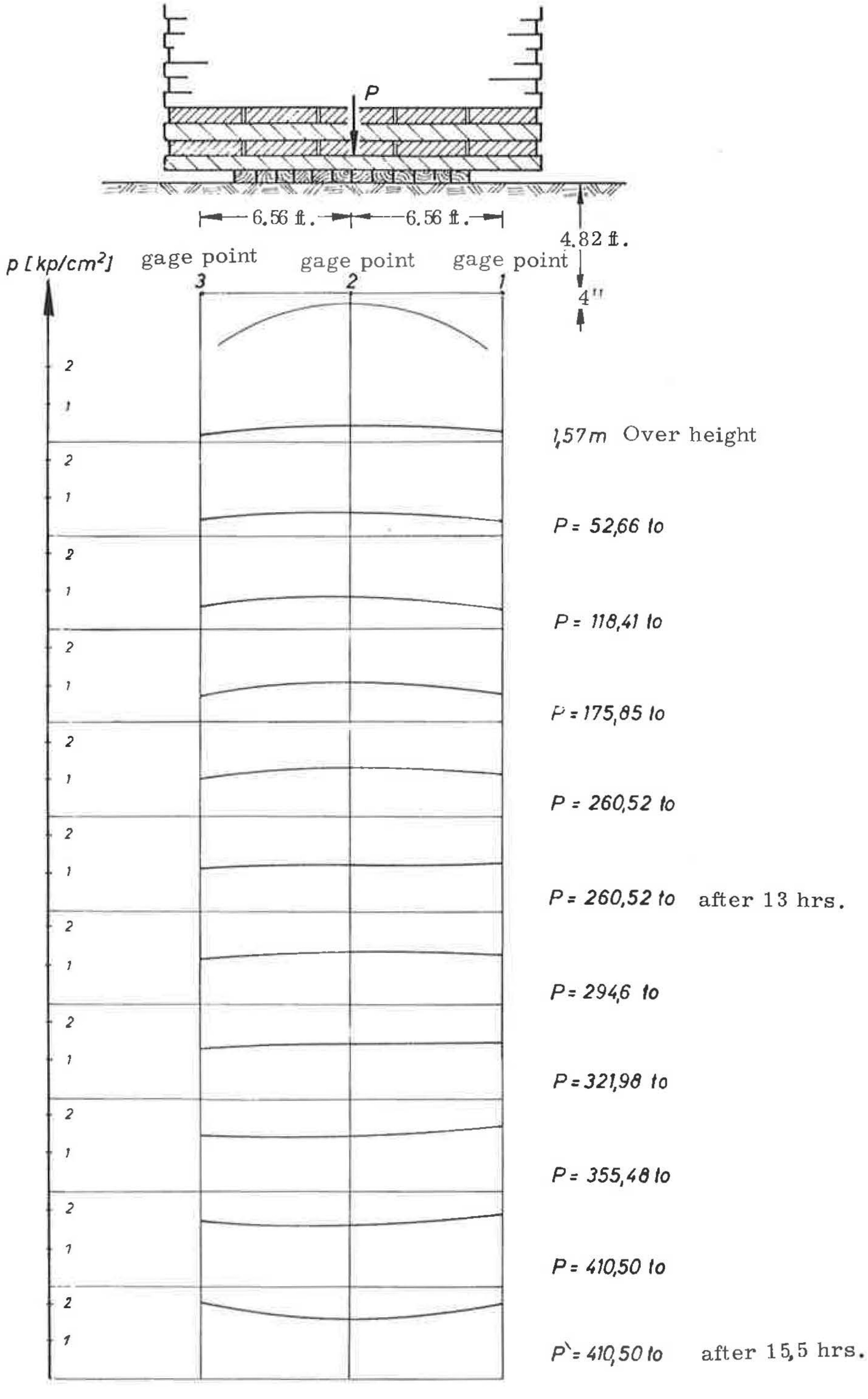

P
6.56 ft.
6.56 ft.
4.82 ft.
4"
p [kp/cm²]
gage point 3
gage point 2
gage point 1
2
1
1,57m Over height
P = 52,66 to
P = 118,41 to
P = 175,85 to
P = 260,52 to
P = 260,52 to after 13 hrs.
P = 294,6 to
P = 321,98 to
P = 355,48 to
P = 410,50 to
P` = 410,50 to after 15,5 hrs.

Figure 31.

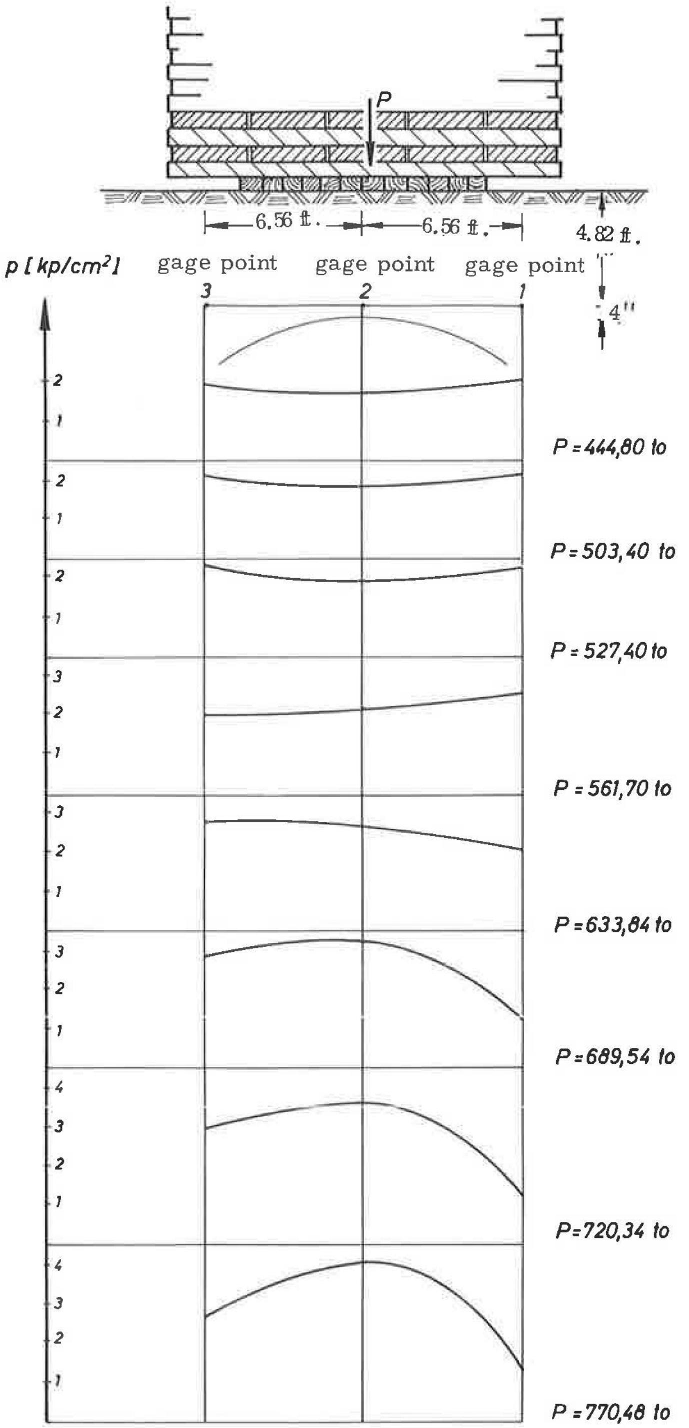

P
6.56 ft.
6.56 ft.
4.82 ft.
4"
p [kp/cm2]
gage point
3
gage point
2
gage point
1
P = 444,80 to
P = 503,40 to
P = 527,40 to
P = 561,70 to
P = 633,84 to
P = 689,54 to
P = 720,34 to
P = 770,48 to

Figure 32.

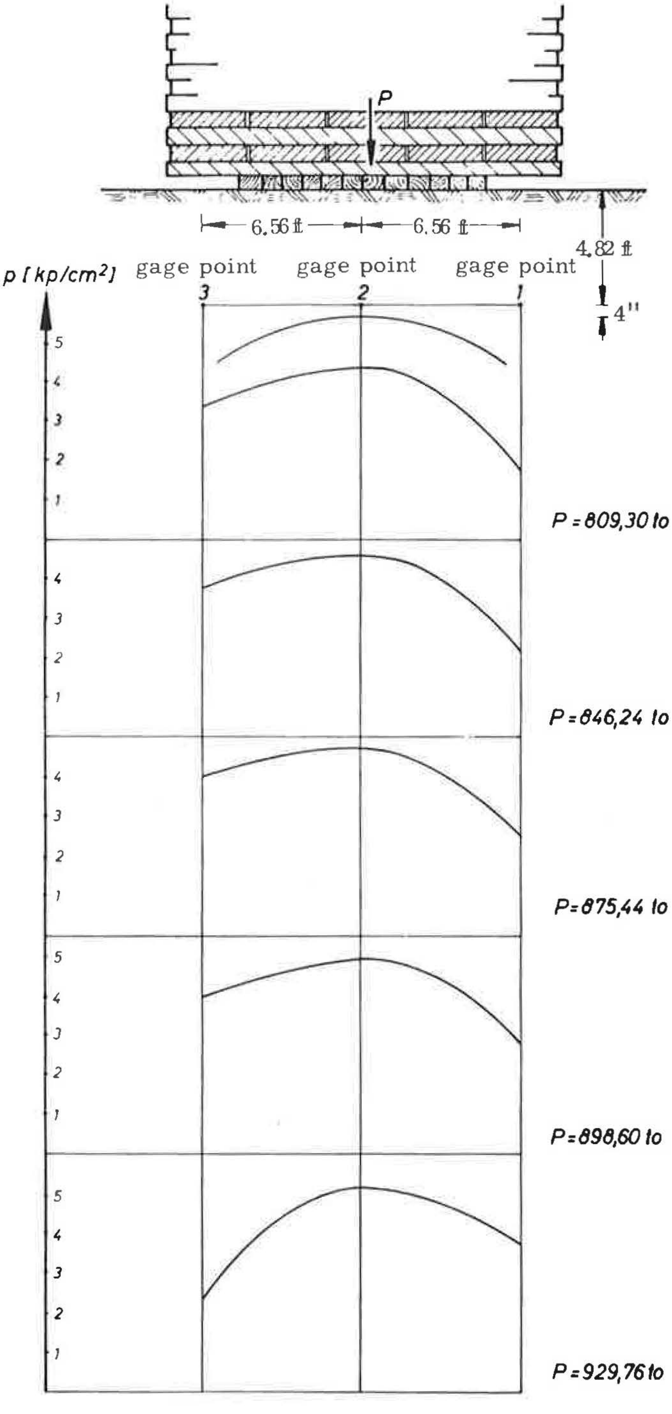

P
6.56 ft
6.56 ft
4.82 ft
4"
p [kp/cm²]
gage point
gage point
gage point
3
2
1
P = 809,30 to
P = 846,24 to
P = 875,44 to
P = 898,60 to
P = 929,76 to

Figure 33.

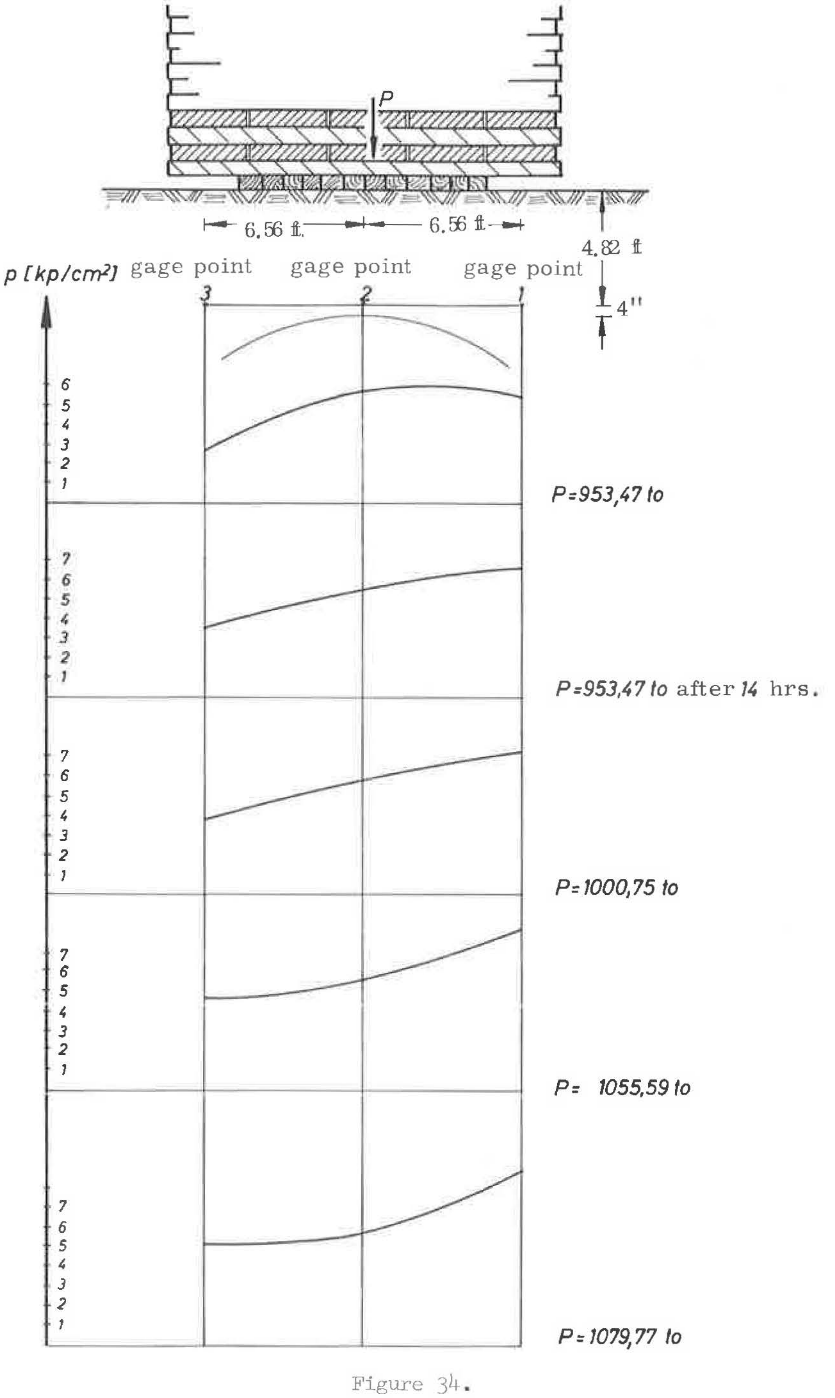


Figure 34.

Extreme fiber stresses in crest point IV = -5,021 psi = +156 psi; in crest point X = -6,600 psi = +484 psi; and

Soil pressure at 4 in. above crest—p_2 = 18.35 - 5.83 = 12.52 psi.

These changes may be ascribed to consolidation of the soil. Although reduced soil pressure was measured at the central gage point, pressures at the other point (p_1 and p_3) had increased.

On July 3, 1963, the load was increased to 410.5 tons. Measurements showed the following changes, as compared to the condition at P = 0:

Plane 1—0.59-in. vertical deflection in crest;
Plane 2—0.60-in. vertical deflection in crest;
Extreme fiber stresses (Fig. 27) in crest point IV = -10,184 psi = -3,001 psi; in crest point X = -11,734 psi = -2,205 psi; and
Soil pressure at 4 in. above crest—p_2 = 23.90 - 5.83 = 18.70 psi.

The gage pins observed by the theodolite and the measurements of soil pressure revealed a slight eccentricity of the load, which again had to be ascribed to inevitable off-center loading and nonuniform soil. As the slab pile became higher (approximately 2.46 ft/100 tons), the danger of inclination increased. Throughout the test, however, direct deformation measurements and soil pressure readings evaluated on the spot permitted an estimate on the amount of eccentricity, which could then be offset, as required, by stacking the slabs accordingly.

On July 4, 1963, the load was increased from 410.5 to 953.74 tons. From above 510 tons, deformations increased considerably (Figs. 28-30). Whereas a 0.31-in. deflection in the crest had been measured under a load of P = 260.52 tons, the deflection increased to as much as 1.12 in. under a 561.70-ton load and to 3.43 in. at 820 tons. Up to a 561.70-ton load, soil pressures in the plane 4 in. above the crest showed a larger increase at the outer measuring points 1 and 3 than at the central point 2 (Figs. 31 and 32). From 561.70 to 929.76 tons, soil pressure at the central gage point increased faster than on the sides. Soil pressures at points 1 and 3 indicated and unstable behavior of the slab pile (Figs. 33 and 34). As it was expected that the pipe arch would soon collapse and there was a danger of the high stack destroying the measuring instruments when falling down, the strain gages were removed at P = 689.54 tons, so that after that no strain readings were taken.

With P = 689.54 tons, readings were as follows:

Plane 1—2.06-in. vertical deflection in crest;
Plane 2—3.11-in. vertical deflection in crest;
Extreme fiber stresses (Fig. 35) in crest point IV = -18,874 psi = -5,291 psi; in crest point X = -23,084 psi = -7,766 psi; and
Soil pressure at 4 in. above pipe-arch crest—p_2 = 47.22 - 5.83 = 41.39 psi.

Under a load of 850 tons, two inward bulges began to develop on each side of the crest (Fig. 36). At that stage, the one on the right was about 11.8 in. and the one on the left 5.9 in. deep, as measured radially. When darkness set in, loading had to be interrupted at P = 953.74 tons. Since a collapse seemed imminent on account of the inward bulging, the pipe was watched throughout the night so that the development of a possible collapse might be studied closely. The large increase in deformations noted toward the evening, which caused the pipe arch to continue deflecting for a short while even after loading had been stopped, came to a standstill in the course of the night.

On the next morning, it was noted that the first layer of slabs was resting firmly against the soil as a result of settling of the ties and sagging of the slabs. Through this, the loaded area had increased from 8.53 × 10.33 = 88.11 sq ft to approximately 16.4 × 9.84 = 161.4 sq ft. These and the earlier consolidations may be regarded as the reason why settlements and deformations died down during the night after a period of sharp rise. When loading was continued on July 5, 1963, the influence of the enlarged loaded area was notable. Although the soil pressure at gage point 2 remained unchanged under a load increase from 1,000.75 to 1,055.79 tons, it rose on both sides. Under a load of P = 561.70 tons, an irregular increase of pressures had already been observed, particularly at the outer gage points. By the time 1,079.77 tons had been applied, this development had reached such unfavorable effects that the soil pressure

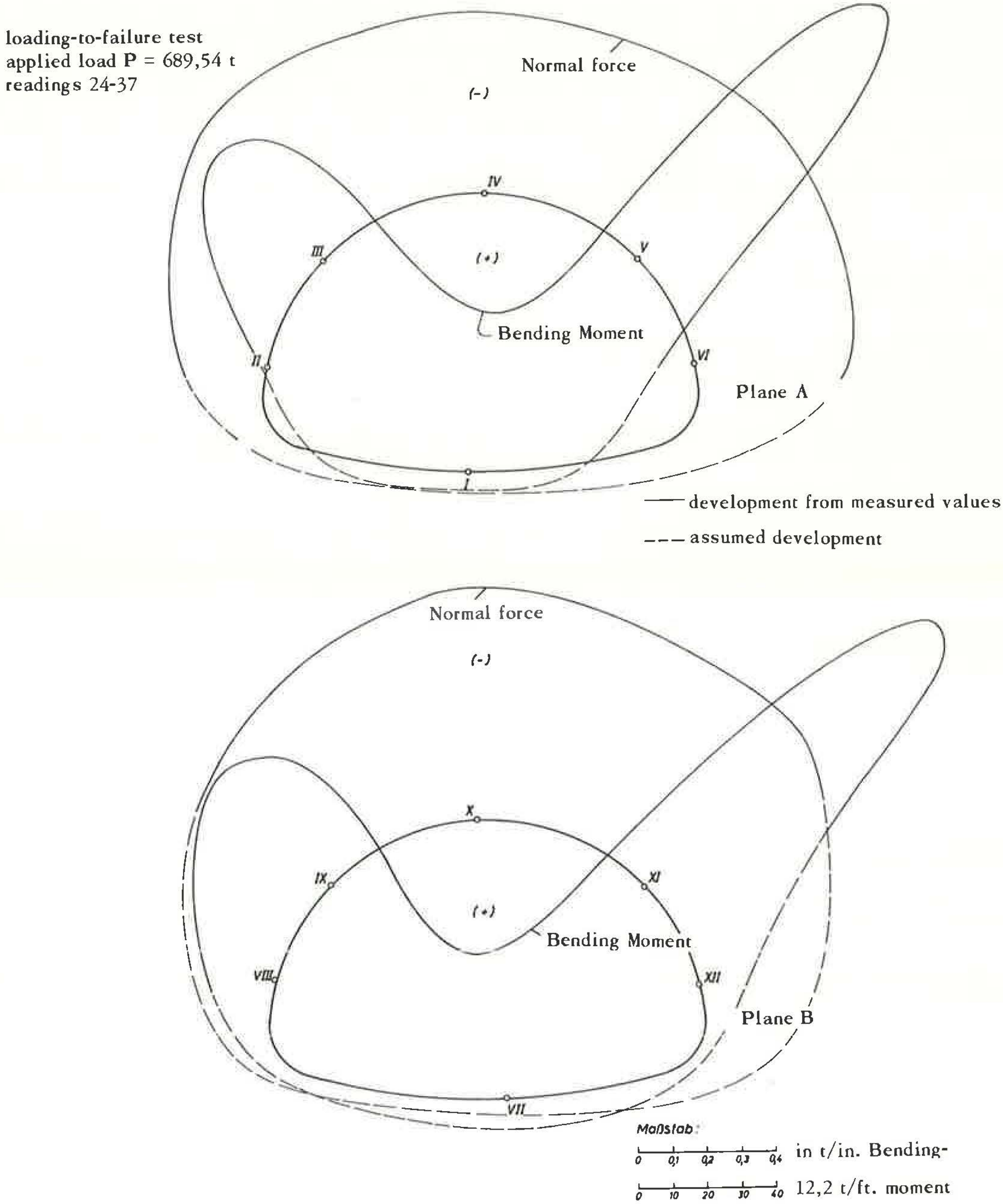


Figure 35. Development of normal forces and bending monents from reading.

at gage point 1 was nearly double that at point 3, which suggested a further loss of symmetry. Deformation measurements, however, gave no indication of imminent collapse. Even the bulge-shaped deformations did not increase much. Thus, there was a risk that the slab pile, which had reached a height of approximately 28 ft and was about 5.25 ft above the surrounding terrain, would tumble down before the utmost carrying capacity of the pipe arch could be reached. This would probably have damaged the two cranes employed for stacking the slabs (Figs. 37 and 38). The applied

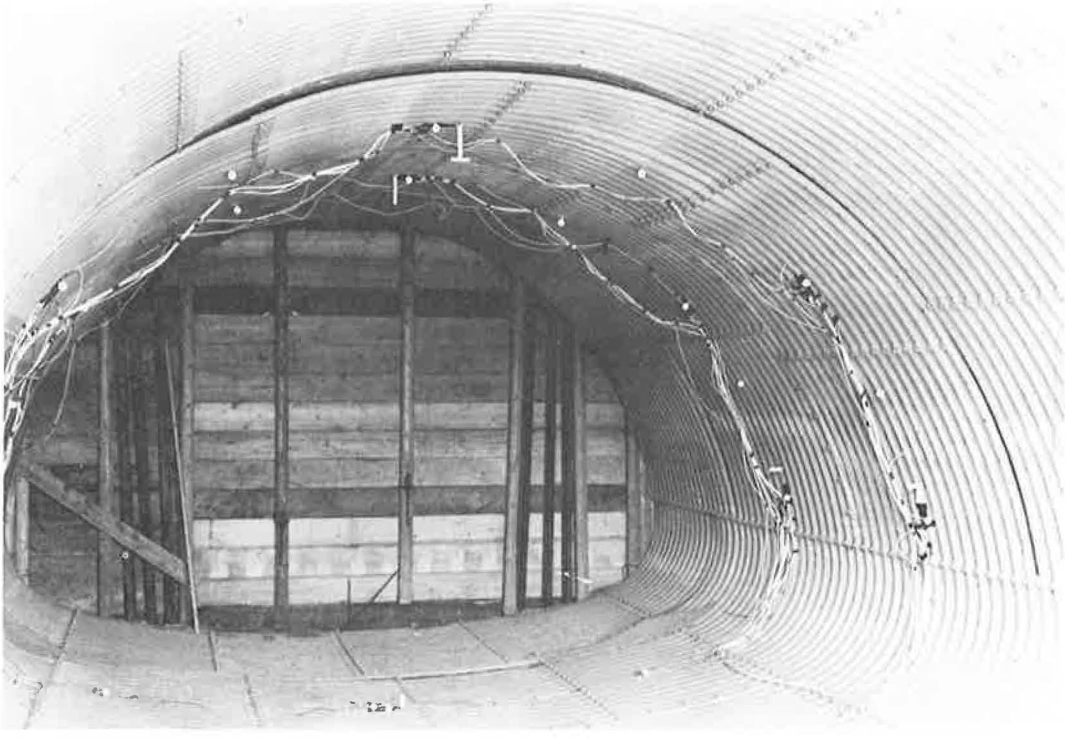


Figure 36. Inside of pipe arch with bulge-shaped deformations on either side of crest. Center section of structure clearly deflected against adjacent rings.

load of p_{max} = 1,079.77 tons and measurements taken so far seemed to give ample evidence; therefore, it was considered not necessary to continue loading until the pipe arch collapsed, which would have been dangerous under the high load.

On July 6, 1963, the slab pile was removed by the cranes and on July 7, the uncovered pipe arch was examined (Fig. 39). The bulge-shaped deformations observed from 850 tons upward were of a plastic nature (Fig. 40), as was the deflection in the crest line parallel to the axis.

Although the center section of the structure was free to move independently from the outer parts and its length of 16 ft had been so selected that it should be completely within the pressure area of the load, plastic deflection near the center was much larger than toward the ends. The bulges always followed the longitudinal seams, even where these were staggered in the two rings of the center section. Near these bulges the ring sections were bent and the plates shifted against each other. The connecting bolts were deformed to an extent that some had been sheared off.

Results of Strain Measurements

Figures 26, 27, 35, 41 and 42 show the normal forces and bending moments for the various load increments as measured by the strain gages. As was done accordingly when measuring deflection, the strains existing after backfilling were disregarded; in other words, base readings of strains were taken as loading began.

Results of Deformation Measurements

Deflection readings have been shown separately for the structure after backfilling and for the loading conditions, as was done for the live-load test. Figure 24 shows

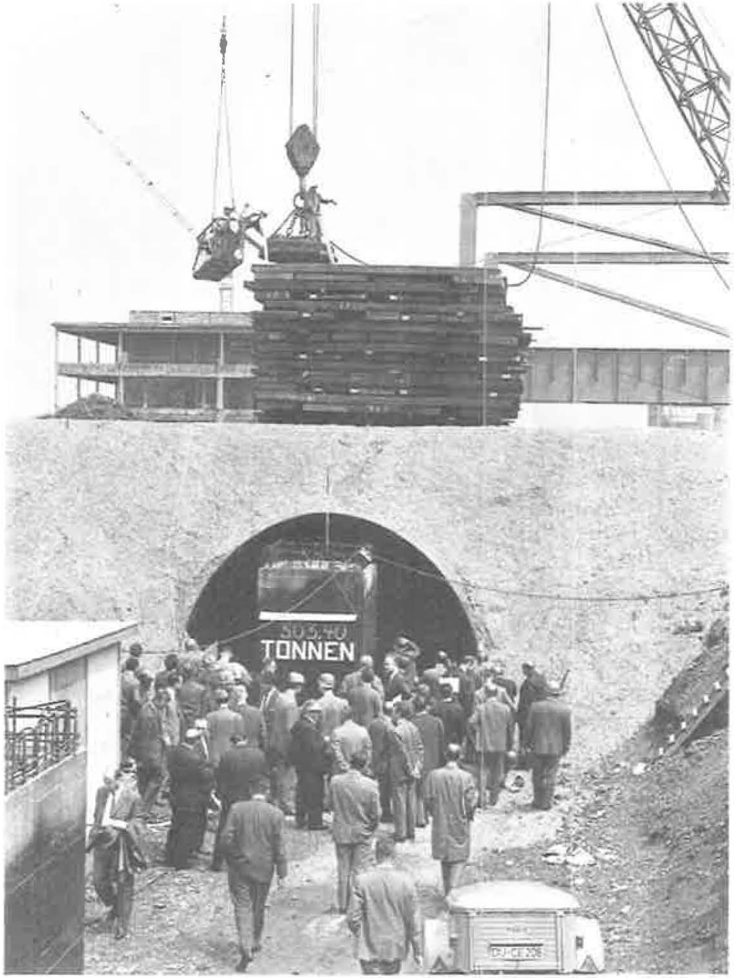


Figure 37. Loading-to-failure test, application of load by magnetic crane.

Figure 38. Loading-to-failure test with P = 1,000 tons.

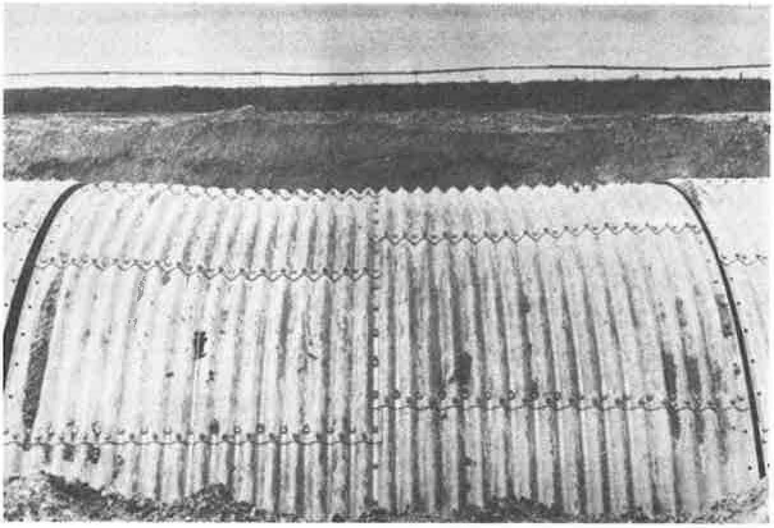


Figure 39. Test structure uncovered after maximum loading of p_{max} = 1,079.77 tons.

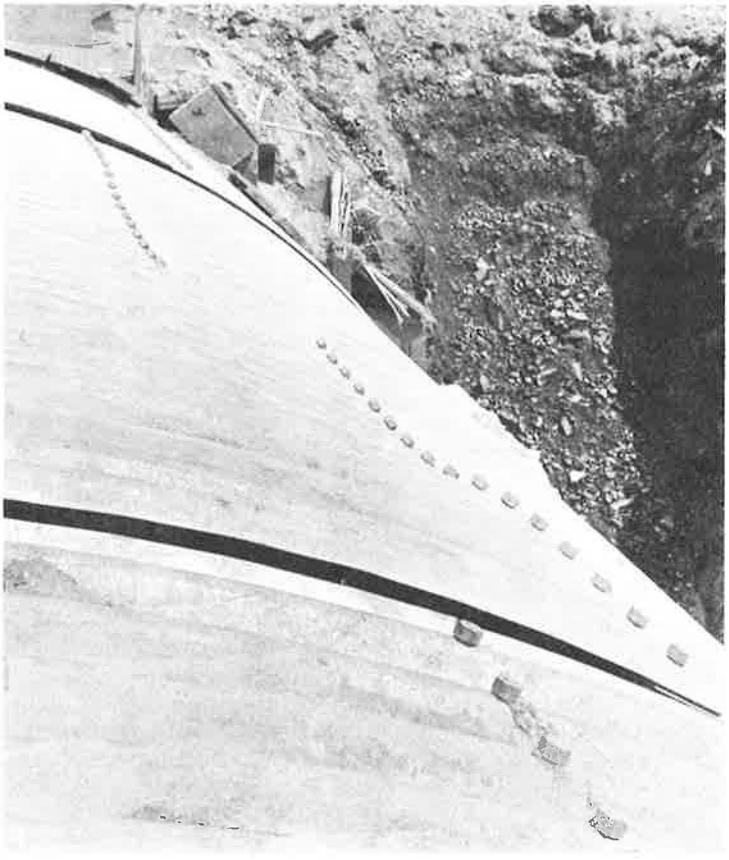


Figure 40. Plastic deformations apparent in uncovered structure.

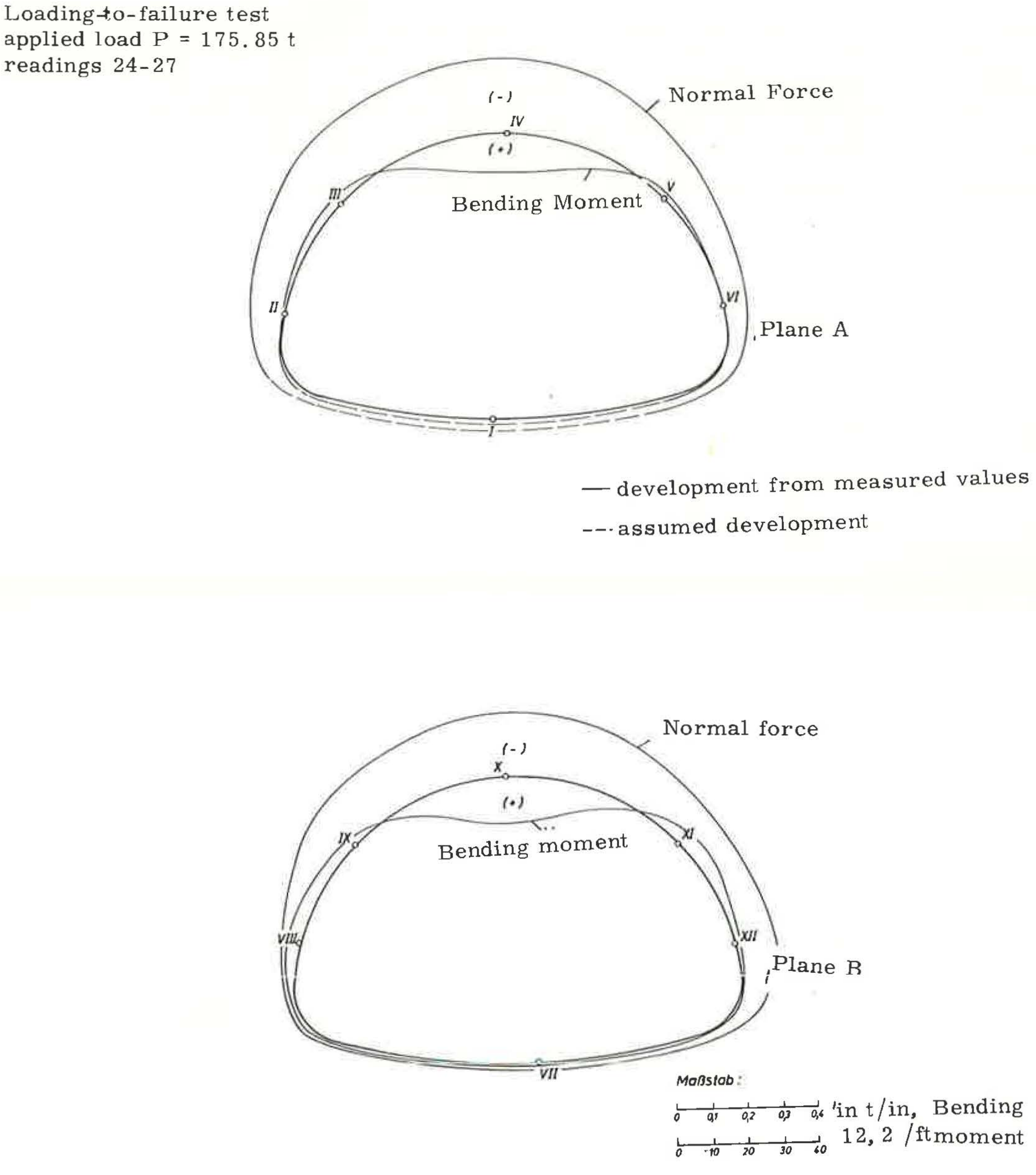


Figure 41. Development of normal forces and bending moments from readings taken.

deflections during placement of fill and new cover up to a height of 5.5 ft above center. Figures 25, 28, 29 and 30 represent the newly introduced deformations for the various load increments. As for the live-load test, these readings do not include the deflections resulting from backfilling. The actual total of deflections from the beginning of backfill placement becomes evident when superposing these deflection figures on Figure 24.

Results of Soil Pressure Measurements

Soil pressures were measured at three gage points on a plane 4 in. above the crest. One of the gage points was located in the load center directly above the pipe-arch crest,

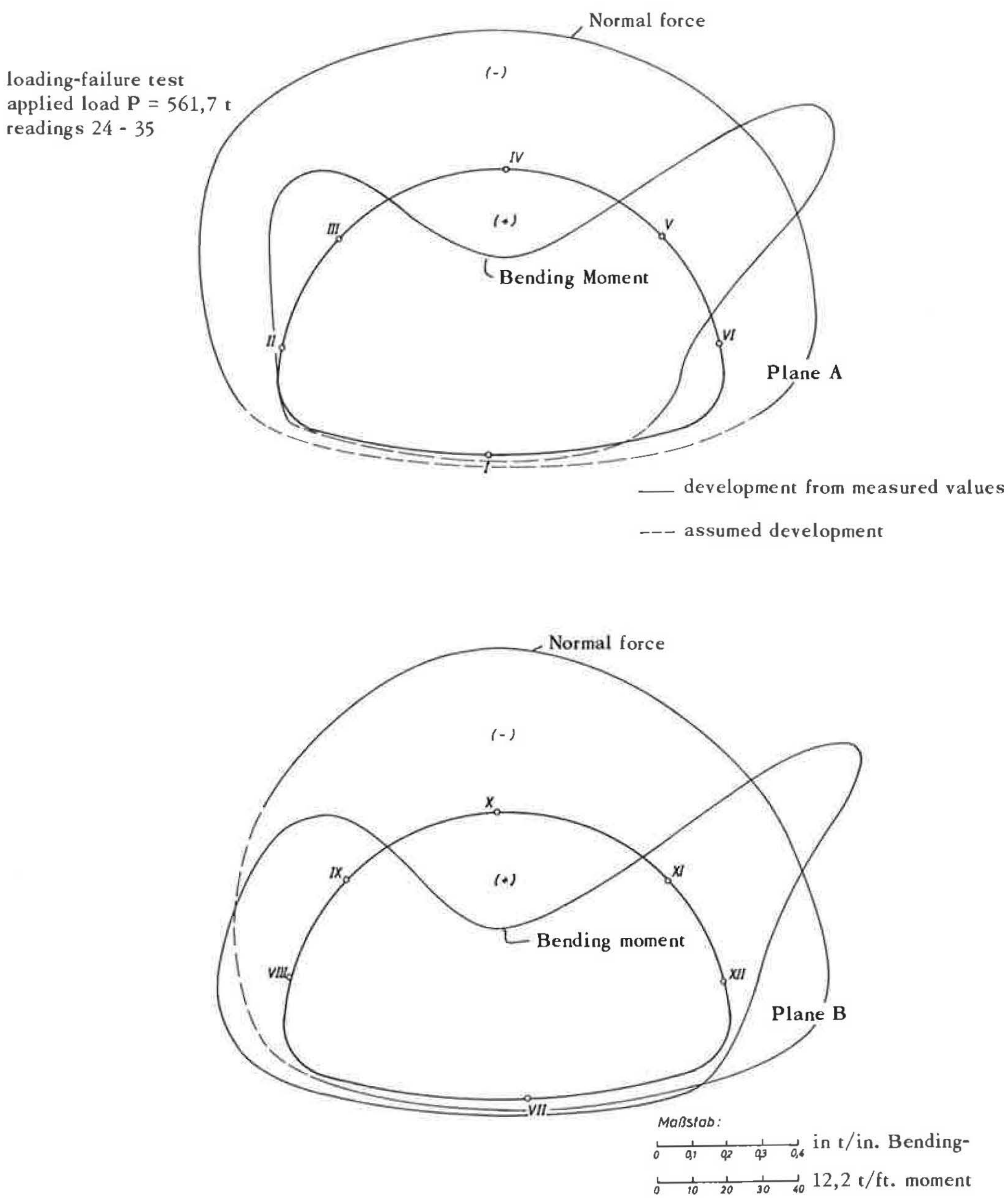


Figure 42. Development of normal forces and bending moments from reading.

and the others at a distance of 6.56 ft on either side of the center. Soil pressure readings are shown in Table 1 and have also been represented in a graph for further clarity. The pattern of soil pressures at the outer gage points, which shows higher values sometimes on the right and sometimes on the left sides, resulted from different loading according to the measured soil pressure. As soon as readings at one of the outer gage points showed higher values, more load was applied on the other side to prevent inclination of the slab pile. Values shown are metric measures.

Computation of Average Distribution of Soil Pressures. —The soil pressure distribution is assumed linear along the height h = 4.82 ft. From the determined values p_o and p_u the pressure distribution is given by:

TABLE 1

SOIL PRESSURE

Applied Load (t)	Soil Pressure (kg/sq cm)		
	Point 1	Point 2	Point 3
1.57 m[a]	0.32	0.41	0.25
52.66 = P	0.45	0.73	0.49
118.41 = P	0.65	0.97	0.63
175.85 = P	0.81	1.15	0.78
260.52 = P	1.15	1.36	1.07
260.52 = P[b]	1.24	1.29	1.17
294.60 = P	1.36	1.39	1.27
321.98 = P	1.48	1.45	1.37
355.48 = P	1.68	1.54	1.46
410.50 = P	1.92	1.68	1.76
410.50 = P[c]	1.97	1.64	1.85
410.50 = P[d]	1.98	1.63	1.95
444.80 = P	2.12	1.75	2.05
503.40 = P	2.31	1.98	2.24
527.40 = P	2.45	2.07	2.39
561.70 = P	2.67	2.23	2.05
633.84 = P	2.10	2.73	2.80
689.54 = P	1.40	3.32	2.90
720.34 = P	1.23	3.68	3.00
770.48 = P	1.48	4.17	2.64
809.30 = P	1.75	4.41	3.37
846.24 = P	2.15	4.68	3.83
875.44 = P	2.55	4.80	4.07
898.60 = P	2.83	4.98	4.00
929.76 = P	3.80	5.24	2.34
953.47 = P	5.52	5.58	2.64
953.47 = P[e]	6.97	5.58	3.63
1000.75 = P	7.59	5.88	3.88
1055.59 = P	8.63	5.88	4.77
1079.77 = P	9.29	5.92	5.34

[a]Cover height.
[b]After 13 hr.
[c]After 4.5 hr.
[d]After 15.5 hr.
[e]After 14 hr.

$$p_o = \frac{P}{F}$$

where

P = applied load,
F = loaded area 8.53 × 10.33 = 88.11 sq ft, and
p_u = measured value.

CONCLUSIONS

The test described in this report conducted on an Armco-Thyssen multi-plate pipe-arch conduit of 20-ft 7-in. span, 13-ft 2-in. rise and 7-gage wall thickness, showed the following results:

1. With a cover height of one-sixth the span = 3.44 ft and a loaded area 8.53 ft wide and 10.33 ft long = 88.11 sq ft, the pipe-arch-soil structure proved capable of carrying a load of P = 151.32 tons applied both axially and off-center showing but slight deformation (0.386 in. = 1/640 of span).
2. With a cover height of one-fourth the span and the same axial loaded area a load of 953.75 tons was applied and, with an enlarged loaded area of approximately 16.4 × 9.84 = 161.4 sq ft resulting from settlement, a load of 1,079.77 tons could be reached in this test without the pipe arch being crushed.

A comparison with the ring compression method may seem of interest in this connection. As is known, the determination of load-carrying capacity by this theory is based alone on compression in the ring and the seam strengths, as derived from actual test data on bolted seams. For 7-gage multi-plate and 4 bolts/ft the seam strength is 93,000 lb/ft (see Armco Catalog MP-1663). The maximum load is determined as follows:

Table 2 shows the determination of average pressure distribution at the outer and centrally located pressure cells. The pressure drop at the outer cells was 68 percent, that at the center cell 66 percent, the average being

$$\frac{68 + 66 + 68}{3} = 67.33\ \%.$$

On the basis of this load distribution, the loaded area above the pipe-arch crest may be determined.

TABLE 2

COMPUTATION OF AVERAGE PRESSURE DISTRIBUTION

P (to)	p_0 (kp/sq cm)	p_{u1} (kp/sq cm)	p_{u2} (kp/sq cm)	p_{u3} (kp/sq cm)	$p_{ur} = (p_1 + p_3)/2$	$\Delta p_{outer} = p_0 - p_{ur}$	$\Delta p_{center} = p_0 - p_{u2}$	$\frac{\Delta p_{outer}}{p_0}$	$\frac{\Delta p_{center}}{p_0}$
175.85	21.5	5.1	7.4	5.3	5.2	16.3	14.1	0.75	0.65
294.0	35.9	10.4	9.8	10.3	10.3	25.7	26.1	0.71	0.73
410.5	50.0	16.0	12.7	15.1	15.5	34.5	37.3	0.69	0.75
503.0	61.5	20.0	15.7	19.9	20.0	41.5	45.8	0.67	0.75
633.8	77.5	18.0	23.2	25.5	21.8	55.7	54.3	0.72	0.70
720.0	88.0	9.1	32.7	27.5	18.3	69.7	55.3	0.79	0.62
809.0	99.0	14.3	40.0	31.2	42.7	56.3	59.0	0.56	0.59
929.8	113.0	34.8	48.3	20.9	27.8	85.2	65.0	0.75	0.58
1055.0	129.0	83.1	54.7	45.2	64.2	64.8	74.0	0.50	0.57
Average value								0.68	0.66

$$F_1 = 10.33 \times 8.53 = 88.11 \text{ sq ft}$$

$$F_2 = \frac{F_1}{1 - 0.6733} = \frac{88.11}{0.3267} = 269.70 \text{ sq ft.}$$

Furthermore,

$$F_2 = (8.53 + 2x) \times (10.33 + 2x) = 269.70 \text{ sq ft}$$

following

$$x = 3.51 \text{ ft at } 4.82\text{-ft depth.}$$

At crest level or 5.15 ft below surface,

$$x' = \frac{3.51}{4.82} \times 5.15 = 3.75 \text{ ft.}$$

The total loaded length of the structure is

$$L = 8.53 + (2 \times 3.75) = 16.03 \text{ ft.}$$

This shows that the test structure of 16-ft length is completely within the loaded area.

According to the seam strength chart, the maximum load the structure can carry will be 1,273 tons (metric tons), derived as follows:

p_{max} = 93,000 × 16 × 2 × 2,976,000 lb;

less dead load of 20.56 × 16 × 5.15 × 100 = 169,414 lb — leaving for the imposed load, 2,806,586 lb, or 1,273 metric tons.

With 1,079-ton loading, this ultimate load was nearly reached in the test. The safety factor of 4 recommended for the determination of wall thickness by the ring compression method is thus fully insured for the safety of the structure against collapse.

When 850 tons had been imposed, the first signs of overloading appeared. Should

TABLE 3

AVERAGE VALUES FROM TENSION TEST

Wall Thickness	Test Taken from	Yield Stress °F (lb/in.)	Tensile Stress °B (lb/in.)	[1]Elongation (%)
1 gage	Crest	54,447	61,302	17.5
	Flank	44,694	53,252	29.5
5 gage	Crest	52,228	58,443	23.5
	Flank	47,221	55,115	35.6
7 gage	Crest	55,442	64,360	21.4
	Flank	45,870	58,600	29.7
8 gage	Crest	51,460	58,785	22.2
	Flank	49,639	61,018	30.0

ARMCO MULTI-PLATE PROFIL NR. S 32

FOR COUNTRYSIDE ROADS
STANDARD CLEARANCE PROFILE
WIDTH OF CARRIAGEWAY = 16'5"

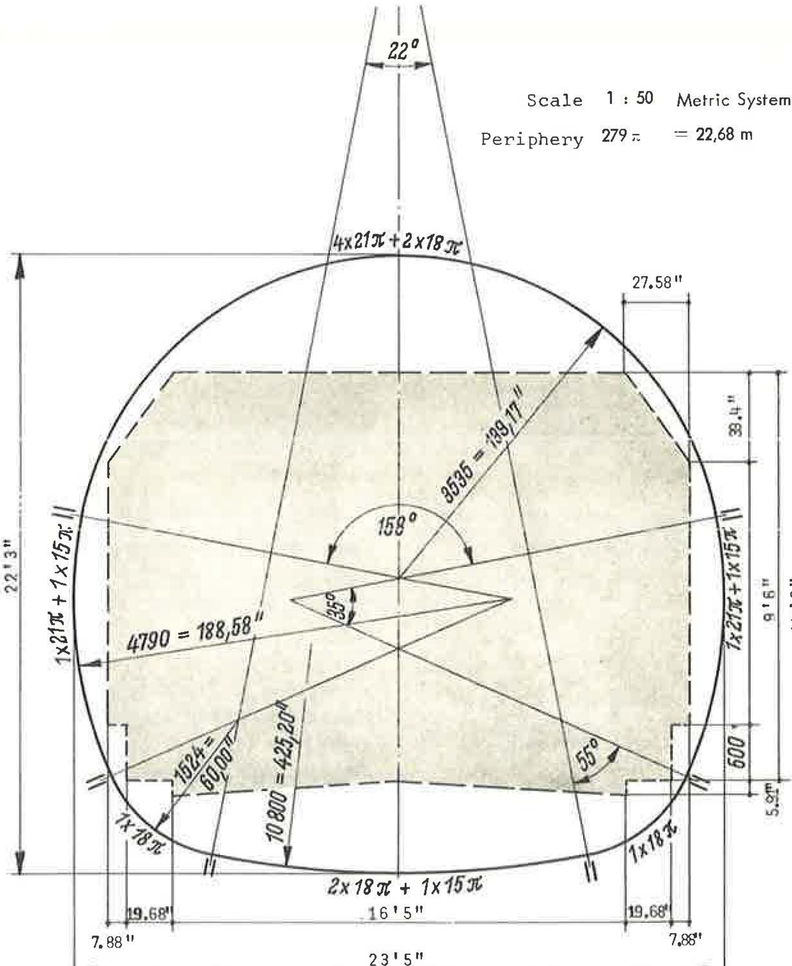


Figure 43.

Figure 44. Special profile S 32 during construction.

these be eliminated, this would leave an actual safety factor of SF = 4 - 1,273/850 = 4 - 1.5 = 2.5. Thus the loading-to-failure test proved again that the ring compression method is well suited for designing corrugated steel pipe.

In consequence of this test result, a 213-ft long king-size multi-plate pipe arch of 23.42-ft span, 22.30-ft rise and 279π circumference could be successfully installed in Germany under the Autobahn between Butzbach and Siegen. This is believed to be Europe's largest corrugated pipe to date (Table 3, Figs. 43, 44).

ACKNOWLEDGMENTS

This test was conducted under the scientific direction of Professor Dr.-Ing. Dr.-Ing. h.c. Kloeppel, Darmstadt Technical University, Germany. This report is based on the test report made by him. The computations and measuring results contained in this paper were taken from that report. All strain measurements and tension tests were conducted by the Institute of Statics and Steel Construction headed by Professor Kloeppel.

Deformation was measured by Dipl.-Ing. E. Jacobs, Essen Engineering College.

Measurements of soil pressures were made by the Weil/Rhein branch of Ernst-Mach Institute.

Tests on backfilling material were made by the Institute of Soil Mechanics headed by Professor Dr.-Ing. H. Breth, Darmstadt Technical University.

Operations were responsibly directed by Armco-Thyssen, Dinslaken.

The test setup was designed in conjunction with the German Federal Railway Authorities.

Appendix

TESTING OF MATERIALS USED

Backfilling Material

Sandy gravel was used as backfilling material for the pipe arch. Its single Proctor density at an optimum moisture content of 6.8 percent was determined to be 120 pcf. The results of the three axial pressure tests indicate a friction angle of 37.5 deg for the sandy gravel at this density.

During backfilling the compactness obtained at the 7 points was determined by the calibrated sand method. This showed an average dry density of 128 pcf, which means that by compaction of fill in 8-in. lifts with Losenhausen AT 200 surface vibrators, a compactness of 107 percent of the single Proctor density was obtained. The results of the drop-penetration test with 70 to 90 blows for 8 in. of penetration depth also indicate the good compaction of the fill.

Tension Tests on Conduit

Test Specimen. —Corrugated multi-plate sheet of different gages as per the company's delivery program, but not curved vertical to the direction of corrugations.

Material. —MU St 34-2 steel plate, cold worked by pressing the rolled shape and hot-dip galvanized consequently.

Tension Test. —Six proportional test bars from each specimen, i.e., four from the corrugation crest and two from the flank.

Discussion

M. G. SPANGLER, Research Professor of Civil Engineering, Iowa State University, Ames—This is an excellent paper; a scholarly and well-written report on a well-conceived and conducted full-scale experimental demonstration project in the field of loads and supporting strengths of underground conduits. It is a particularly noteworthy contribution in this field because it chronicles the change in shape of a pipe-arch structure acted upon by vertical loads and lateral earth pressures. Quantitative data are presented which show that the deformation of a pipe arch under vertical load follows the same general pattern as that of a circular flexible conduit; that is, the vertical dimension shortens and the horizontal dimension lengthens, thereby mobilizing the lateral support of the side columns of soil. The writer has always assumed this to be true but this is the first documentation of the facts which he has seen.

The author states that the live-load test was conducted under severest possible conditions as regards the railroads' desires for loading on the structure and considering a safety factor of 3. However, from the standpoint of structural performance of the conduit, it is the writer's opinion that the installation was unusually favorable. It is difficult to imagine an environment for a flexible conduit installation which could be more favorable with respect to deformation of the pipe, the performance characteristic most frequently in evidence when a structure of this kind gets into structural difficulty.

Flexible conduits, particularly those of larger radius, derive their ability to sustain vertical load almost wholly from the restraining influence of the soil backfill at the sides. The more strain-resistant the sidefill soil, the less will be the deflection of the conduit and vice versa. To visualize this fact, imagine a structure of the type and size used in these experiments, installed in such a way that there was no soil in contact with the sides, and therefore no lateral pressures acting on the conduit (Fig. 45). Obviously this imaginary structure could carry only the merest fraction of the vertical load which the actual structure successfully carried.

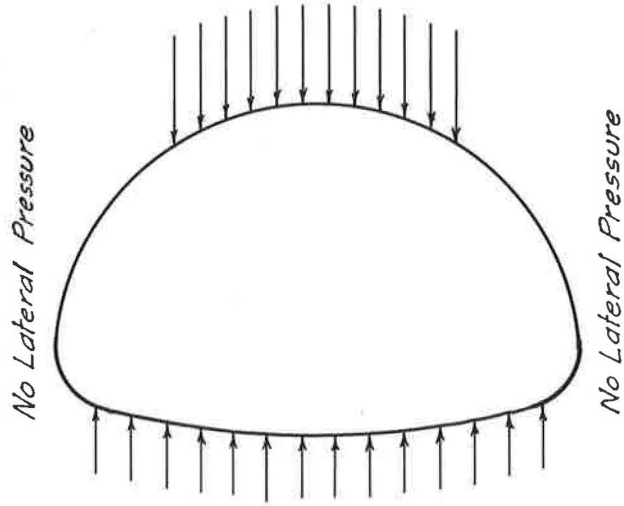


Figure 45. Imaginary pipe arch with no lateral pressure.

Now imagine further that the sidefill soil consisted of a highly compressible low-density material such as a uniform grain-size silt of high moisture content. The deflection of the structure would be nearly as great and its ability to carry vertical load nearly as limited as in the imaginary no-lateral-pressure case illustrated. These imaginary situations are cited to emphasize the fact that the structural performance of a flexible conduit is directly dependent on the strain-resistant quality of the sidefill soil, and there is a tremendous range of soil quality between this very poor imaginary material and the very excellent sandy gravel used in the experiments. The physical properties of the conduit wall—that is, the gage of metal, depth and spacing of corrugations, modulus of elasticity, etc.—are relatively minor contributors to resistance to deformation and ability to carry vertical load. The structural performance of flexible conduits cannot be predetermined without a reasonably precise statement concerning the kind, quality and extent of the side columns of soil which play such an important role in supporting the structure. It is not sufficient to say merely that the sidefill soil should be "of good quality" or "thoroughly compacted" or some similarly vague description.

The quality of soil from the standpoint of its effectiveness in minimizing deformation of flexible conduits can be expressed in terms of the "modulus of soil reaction" (3, 8), whose units are lb/sq in. It is somewhat similar to modulus of elasticity of elastic materials, except that it appears to involve a size-factor. Present knowledge, still very imperfect, indicates the following relationship:

$$E' = er$$

in which

E' = modulus of soil reaction, psi;
r = radius of conduit wall, in.; and
e = modulus of passive resistance of soil, psi/in.

The modulus of passive resistance is a quantitative expression of the relationship between strain of the soil and pressure exerted by a body pushing against it. This modulus is similar to Westergaard's (6) modulus of subgrade reaction, in his analysis of stresses in concrete pavement slabs; and to Cummings' (2) modulus of foundation, in his analysis of the stability of foundation piles against buckling under axial load.

The backfill soil used in Dr. Demmin's experiments was of extremely high quality for the purpose of minimizing deformation of the conduit. It consisted of a sandy gravel

material which was placed in lifts of 8 in. and each layer compacted with surface vibrators. Laboratory and field tests indicated an average dry density of 128 pcf or 107 percent of single Proctor density. The angle of friction was 37.50 deg; a very high-strength material. It is apparent that this backfill material is closely comparable to that placed at the sides of the classical Cullman County, Alabama (5) installation of 84-in. circular metal pipes wherein the pipe deflection was negligible. It is a kind of material which is completely unavailable in many areas, or if available, only at very high cost.

The modulus of reaction of the Cullman soil has been estimated to be in the neighborhood of 7,980 psi (4). In contrast, several installations of circular pipes have been observed in which the estimated modulus of soil reaction was less than 300 psi (4). This illustrates the wide range of sidefill soil restraint which may actually develop depending on the quality of soil and the manner of its placement and compaction. There is also evidence to indicate that even where high quality soil sidefills are provided, they must extend laterally for a considerable distance to be fully effective. A number of situations have developed in which excessive deflection of circular flexible pipes could be attributed to the fact that the side columns or berms of soil were very limited in lateral extent. A rule of thumb in this regard relative to actual field installation is to provide side columns of good quality, well-compacted soil for a distance on each side of the structure equal to at least twice its horizontal dimension.

The wide range of possible values of the modulus of soil reaction encountered in actual flexible conduit construction, accounts very largely for the wide range of performance of these structures with reference to deflection under load. A survey of 239 corrugated steel culverts (4), conducted in 1943 by a leading manufacturer of this type of structure, indicated a range in deflection from -5.0 to +12.1 percent of nominal diameter. Other observers have noted similar results, though on a less extensive scale. This characteristic of structural performance points up the need for research in this area to evaluate and identify the strain-resistant characteristics of soil materials in terms of determinable properties, such as mechanical analysis, Atterberg limits and density. Watkins (9) has contributed a great deal to our knowledge in this area by his work with the Modpares Device, but additional studies of the actual performance of structures in relation to sidefill soil environment are sorely needed. It is suggested that much value would accrue from an extensive detailed record of flexible conduit installations which would include not only the physical details of the conduits, but also facts concerning their installation, such as the character of bedding, and the manner of placement and lateral extent of the sidefills. The soil should be carefully identified in each case and its density determined. Then accurate records of conduit deflections over a period of several years would make it possible to determine empirically an appropriate value of the modulus of soil reaction for a variety of soils within a practical range of densities. The manufacturers of flexible metal pipes and pipe arches would be ideal agencies for collecting such information because of their worldwide contacts with installation of these kinds of structures.

An important phenomenon reported in the paper is the initial deformation of the structure as the sidefill soil berms were built up and compacted. During this stage of construction, the deflection of the pipe arch was opposite in direction to that caused by vertical load in later phases of embankment construction and, in effect, was a "prestressing" operation. The amount of reverse deflection was nominal in this instance and well within that which the structure could tolerate. The relatively low magnitude of this initial reverse deflection is thought to be associated with the very high strain-resistant quality of the sandy gravel sidefills. If the material had been a compacted clayey material, the reverse deflection probably would have been much greater. Instances are known in which it has been necessary to inhibit this initial reverse deflection by the installation of diagonally oriented tie rods inside the structure, or by piling sand bags or loose soil on top as the sidefills were built up, to prevent reverse curvature of the sides of the conduit and "barnroofing" of the top.

These experiments provide information which appears to conflict with the fundamental tenets of Whites' (7) Ring Compression Theory. This theoretical approach begins with the assumption that all loads on a flexible underground conduit act normal to the pipe wall and that the effective load system is similar to hydrostatic pressure acting

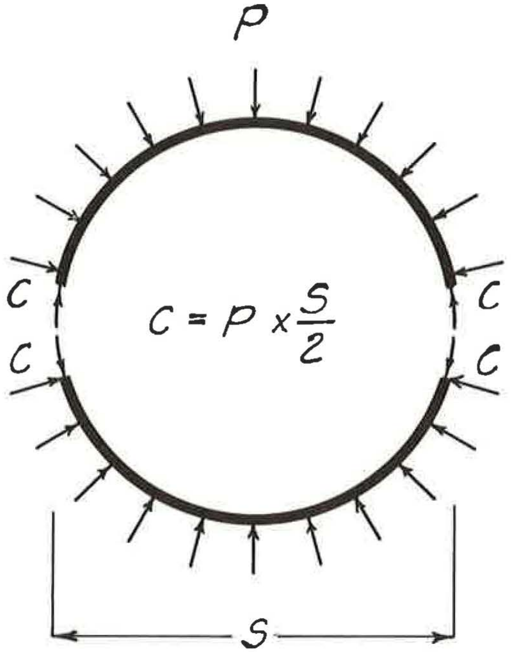


Figure 46. Basic concept of the ring compression theory.

on the outside of a cylindrical vessel. Therefore, it is postulated that the only stresses of consequence in the pipe wall are tangential compressive stresses; hence the name Ring Compression Theory. Figure 46 (1) illustrates this basic concept. Bending moment and deflection of the pipe are completely ignored in the theory.

Dr. Demmin's measurements clearly indicate that there were bending moment stresses of considerable magnitude in the experimental structure. During placement and compaction of the sidefills, the sides of the pipe arch were pushed inward and the top moved upward. This caused prestressing of the pipe wall in tension on the inside face at the sides and bottom, and on the outside face at the top and at the lower corners. At the completion of 3.44 ft of cover, prestressing was reversed to some extent, but there was a residual moment which produced a maximum outer fiber stress in the crown of nearly 40,000 psi. Graphs of the normal force and bending moment at 2 transverse planes through the structure at this load are shown in Figure 5 of the report.

As the live-load slabs were placed at the embankment surface, prestressing was counteracted to the extent that the bending moment became essentially zero at an applied load of 78.62 T, as shown in Figure 13. Then as further load was added up to 151.32 T, the bending moment increased in the opposite sense as shown in Figure 10. These bending moments, like the deflection, were probably much less in this installation than would have been the case if a more usual and less strain-resistant backfill material had been used. That the bending moments keep on increasing as loads are increased is shown by the moment diagrams in Figure 35 which were measured when the applied load was at 689.54 T. The failure to recognize bending moments and deflections and failure to relate these phenomena with the quality of the sidefill soil material constitute serious shortcomings in the Ring Compression Theory, in this writer's opinion.

In reference to the diagrams showing bending moments and normal forces around the periphery of the pipe arch: Values of these functions developed from instrument measurements are shown in solid lines, whereas dashed lines are used to indicate assumed values in regions where the instruments apparently did not yield firm information. It is noted that most of the diagrams shown assumed values in the bottom of the structure between the corners and that these assumed values are consistently relatively low.

This writer has never seen a pipe arch which has developed structural difficulty. However, he has been told by some who have observed such phenomena that there is a tendency for the bottom of the structure to bend upward near the longitudinal centerline, which would seem to indicate a fairly high positive moment in this region. This tendency is in evidence where measured values of bending moment are shown in Figures 5 and 10. However, most of the estimated values of moment are negative in direction and relatively low in magnitude.

Furthermore, the estimated normal forces on the bottom of the arch are very low in magnitude, while the measured values on the top surface are relatively high. Since action must equal reaction it is difficult to accept the estimated values as shown. At least it is suggested that here is a fertile field of needed research to determine more

accurately the actual magnitude and distribution of normal forces on the bottom of an arch and bending moment stresses in this region and in the vicinity of the bottom corners.

There is a great deal of value in demonstration projects such as this, but there are dangers associated with them also. One danger is that readers may not fully realize the favorable aspects of the demonstration and thus gain the impression that all such structures will perform equally satisfactorily. This of course is far from true, as evidenced by the fact that failures of underground conduits do occur. And all too often such failed structures are merely replaced and potential lessons which might be learned are not made available to the engineering profession.

It is this writer's contention that engineers can learn more from one failure situation, if it is thoroughly studied and the causes determined, than can be learned from a dozen or more successful installations. One difficulty in the development of knowledge in this manner is the reluctance of owners and installers of conduits to permit publication of the facts when failures occur. Typical of attitudes in this regard is that of a member of the staff of a certain state highway department. Knowing the writer's interest in underground conduits, he told of a failure of a large-size highway culvert in his state. It had been investigated and a report made to the chief engineer. When asked for a copy of the report, including the photographs which accompanied it, he hesitated, then agreed to send the report, but with the understanding that it be held confidential. He remarked, "We are not very proud of this installation." In another state a series of culverts under an interstate highway got into trouble and the writer was asked to investigate the situation, but before even going on the job, was sworn to secrecy by the chief engineer of the department. There is heartening evidence that this attitude may be changing for the better, but it has been all too prevalent in the past.

Much of our knowledge in engineering practice has resulted from the study of failures of structures and publication of the results. Early in this century the failure of the great Quebec cantilever bridge stimulated research relative to the carrying capacity of latticed steel columns, with the result that column design is now on a much more reliable basis than formerly. Later the failure of the Ft. Peck dam led to tremendous advances in the art of foundation exploration and interpretation of sub-soil materials. Still later, study of the failure of the Tacoma Narrows suspension span resulted in the development of a vast body of knowledge of aerodynamic forces on suspension bridges, and adequate design of this type of structure is much more sure than formerly.

In each of these instances, extensive and detailed studies of the causes of failure were made by teams of experts, and the results of their studies were published so that the whole engineering profession could read and profit thereby. It is this writer's plea that the same type of high-level engineering statesmanship be applied in the culvert industry so that structural distress and failures of this small, but important type of structure may be reduced to a minimum.

References

1. Armco Steel Corporation, Drainage Products Division. Ring Compression Report, 1964.
2. Cummings, A. E. The Stability of Foundation Piles Against Buckling. Proc. Highway Research Board, Vol. 18, Pt. II, pp. 57-65, 1938.
3. Spangler, M. G. The Structural Design of Flexible Pipe Culverts. Iowa Eng. Exper. Sta. Bulletin 153, Iowa State Univ., 1941.
4. Spangler, M. G. Discussion of Structural Considerations and Development of Aluminum Alloy Culvert, by A. H. Koepf. Highway Research Board Bull. 361, pp. 55-64, 1962.
5. Timmers, John H. Load Study of Flexible Pipes Under High Fills. Highway Research Board Bull. 125, pp. 10-11, 1956.
6. Westergaard, H. M. Computation of Stresses in Concrete Roads. Proc. Highway Research Board, Vol. 5, Pt. I, pp. 90-112, 1925.
7. White, H. L., and Layer, J. P. The Corrugated Metal Conduit as a Compression Ring. Proc. Highway Research Board, Vol. 39, pp. 389-397, 1960.

8. Watkins, Reynold K., and Spangler, M. G. Some Characteristics of the Modulus of Passive Resistance of Soil: A Study in Similitude. Proc. Highway Research Board, Vol. 37, pp. 576-583, 1958.
9. Watkins, R. K. Development and Use of the Modpares Device. Proc. ASCE, Vol. 90, PL 1, Jan. 1964.

J. DEMMIN, Closure—The writer is very happy that such a well-known expert on underground conduits as M. G. Spangler was prepared to discuss the paper presented. His comments are sincerely appreciated.

It certainly cannot be stressed too much that the structural performance of flexible pipe to a great extent depends on the quality of the backfill material and the way it has been compacted. It is also very true that from one failure situation one can learn more than from a dozen successful installations. However, systematic examination of the reasons for structural failure will be possible only if preceded by tests that were conducted under known conditions. The test was to contribute to the task of collecting fundamental theoretical data that might help to indentify the causes for structural failure.

Professor Spangler mentioned in his discussion that the author has never seen a pipe arch which had developed structural failure; this is true. But the author knows more than 2,000 structures installed in Germany which have never caused major troubles so far.

Being an expert of great renown, Professor Spangler will be asked to investigate all structural failure situations, and it may therefore seem understandable that, from this point of view, the ability of the test pipe to sustain vertical loads should have been qualified. In the meantime, however, the results of this experiment have been substantiated in the field, and it has become evident that structures will not collapse if installed under similar conditions as was the test pipe. These conditions normally are to be met quite easily.

Professor Spangler has indicated that the backfill soil was of extremely high quality and that everything had been done thereby to minimize deformation. It is a fact that the backfill material was selected by the Federal German Railways, and compacted in lifts with commercial vibrators, as is recommended by our company in our installation instructions. For the test, a sandy gravel was used which had been taken from a gravel pit without further processing. This material, naturally, will not be available on every jobsite at an economically justifiable price. In case material of poorer quality is used, greater deformation will develop, and the carrying capacity would be reduced accordingly. As may be recalled, however, the test showed that a twentyfold load could be applied when using good quality soil. There is ample reserve, therefore, to warrant sufficient safety even where poorer quality backfill soil is used. By this, the writer acknowledges that the quality of backfill soil must be regarded as a factor when predetermining the carrying capacity of flexible pipe. To express this quality in terms of determinable factors, it will be necessary to know the "modulus of soil reaction." This modulus of soil reaction, together with an examination of the stability of a pipe section, should provide reliable information on its structural performance. In Germany, Professor Kloeppel is conducting research work in this field.

White's Ring Compression Theory has never claimed to be a scientific basis of structural performance, and therefore a comparison between the ring compression theory and the measured bending moments does not seem appropriate. However, the ring compression theory at present provides the best approximation to the actual structural performance of flexible pipe. This is evidenced by the fact that hundreds of structures designed by this method are operating quite satisfactorily. As shown in the paper, the author calculated a maximum load of 1,273 tons for the test structure on the basis of the ring compression formula disregarding bending moments. The fact that the test had to be stopped at 1,079 tons without complete failure, shows that the ring compression formula gives astoundingly good approximations for determining load capacity.

It is certainly just as inappropriate to ascribe too much value to the measured bending moments, since the bending moment stresses already developing during assembly and prior to backfilling are so high that they could cause the steel to yield. It must also be expected, that as the sidefill berms are built up to the crest, bending moment stresses might develop in other places, which might approach the yield point of the steel. From a conventional point of view, therefore, the pipe has been "overloaded" several times even before the top cover is placed. Despite this, we know that these bending moments have not much influence on the ability of a flexible structure to carry loads. This fact will justify, disregarding the bending moments, as is the case in the ring compression theory.

Even if the assumed values of bending moments and normal forces developed on the bottom of the pipe are not based on strain gage readings, they were estimated with good reliability on the basis of deformation on the pipe invert. Unfortunately, the strain gages installed on the bottom of the pipe arch were damaged beyond use. As deformations of the bottom of the pipe arch were very low in magnitude, the corresponding bending moment would likewise be very low. The possibility of an inaccurate estimate, as indicated by Spangler, would therefore seem unlikely. Further, it was stated that the small normal forces acting in the bottom area of the pipe arch, did not conform to the relatively high values in the top of the structure. In the writer's opinion, this fact may be explained by the great frictional forces acting around the pipe periphery, which would bring about an equilibrium.

Finally, the writer would like to stress that this one large-scale experiment will naturally not answer all the questions pertaining to the determination of the load-carrying capacity of flexible pipe. Convincing evidence was provided, however, that when using good quality backfill soil which was carefully compacted, a large pipe arch was capable of carrying twenty times the load desired by the railroad authorities. This provides for a sufficient safety margin even in cases where lower quality soils are used.