Granular Earth Pressures on Steel Tunnel Lining

E. A. BARTKUS, Assistant Chief Engineer
Meissner Engineers, Inc., Chicago, Illinois, and
E. VEY, Professor of Civil Engineering, Illinois Institute of Technology

A number of field measurements was made to determine the pressures and corresponding deflections for an arch type steel lined tunnel under storage piles of crushed stone. The stone was 1½ inch material and uniform throughout.

The tunnel was constructed of ¼ inch steel plate sections, flanged on all edges inwardly. These sections were bolted together through the flanges on the inside of the tunnel, thus leaving the outside surface fairly smooth and uniform and making the tunnel quite flexible. The width of the tunnel, at the concrete base slab, was approximately 7 ft and the height at the crown 7 ft.

Goldbeck pressure cells were carefully calibrated and mounted underneath the base slab and around the outside of the tunnel during construction. Deflection gages were also mounted inside the tunnel at the section where the pressure gages were placed.

Measurements were made over a period of several months during which time the crushed stone overburden was placed on the tunnel to a maximum height of 70 ft and then removed. It was possible, therefore, to obtain measurements at various heights and configurations of overburden load.

The results indicated that the tunnel lining experienced the full overburden pressure at the crown and the pressures around the rest of the tunnel tapered off to a value approximately 0.3 times the vertical overburden at the base.

A recommended loading is given, based on the experimental data. It consists of a trapezoidal pressure distribution over the horizontal projection of the tunnel and a distribution over the vertical projection which is rectangular on the upper one-third and trapezoidal on the lower two-thirds.

● ONE phase of the general field of bulk materials handling has to do with the storage and the reclamation of material from storage.

The material in question might be sand, gravel, various sizes of screened or unscreened crushed limestone, coal, crushed, screened and unscreened ore, slag, etc. The piles might be stacked up to 120 ft in height by means of stackers, self-unloading boats, trippers of belt conveyors, grab buckets, etc., as shown in Figure 1.

One of the mechanical methods used to reclaim stored material in great volumes is by means of a belt conveyor installed in a tunnel under a storage pile (reclaiming systems up to 8,000 tons per hour are known). A number of gates and chutes suspended from the tunnel top can be opened or closed to allow the flow of material from the pile to the moving belt conveyor in the tunnel. The capacity of material flowing to the belt may be controlled by the size of the gate opening, slope of the chute, speed of the belt and the number of open gates.

As soon as the gate has been opened, material directly over the gate loses its support and slides down the chute to the belt, thus producing a cavity in the storage pile directly over it. This cavity increases in size as time goes on and finally becomes an inverted cone with its apex at the gate and its base as large as the angle of repose permits. If the material arches and stops the flow, the next gate can be opened, thus building another cavity and eventually destroying the leg of the arch at the first gate. This procedure may be repeated, providing complete control of the reclaiming process.

As a consequence of this type of reclamation the pile becomes separated into two smaller piles along both sides of the tunnel with the sides sloping at the angle of repose of the material.

Meanwhile, the original pile may be reloaded fully or partially, or with the help of a bulldozer one or both piles on the sides of the tunnel may be pushed toward the tunnel gates and reclaimed.

In the design of the tunnel the following varying load conditions have to be taken into account:

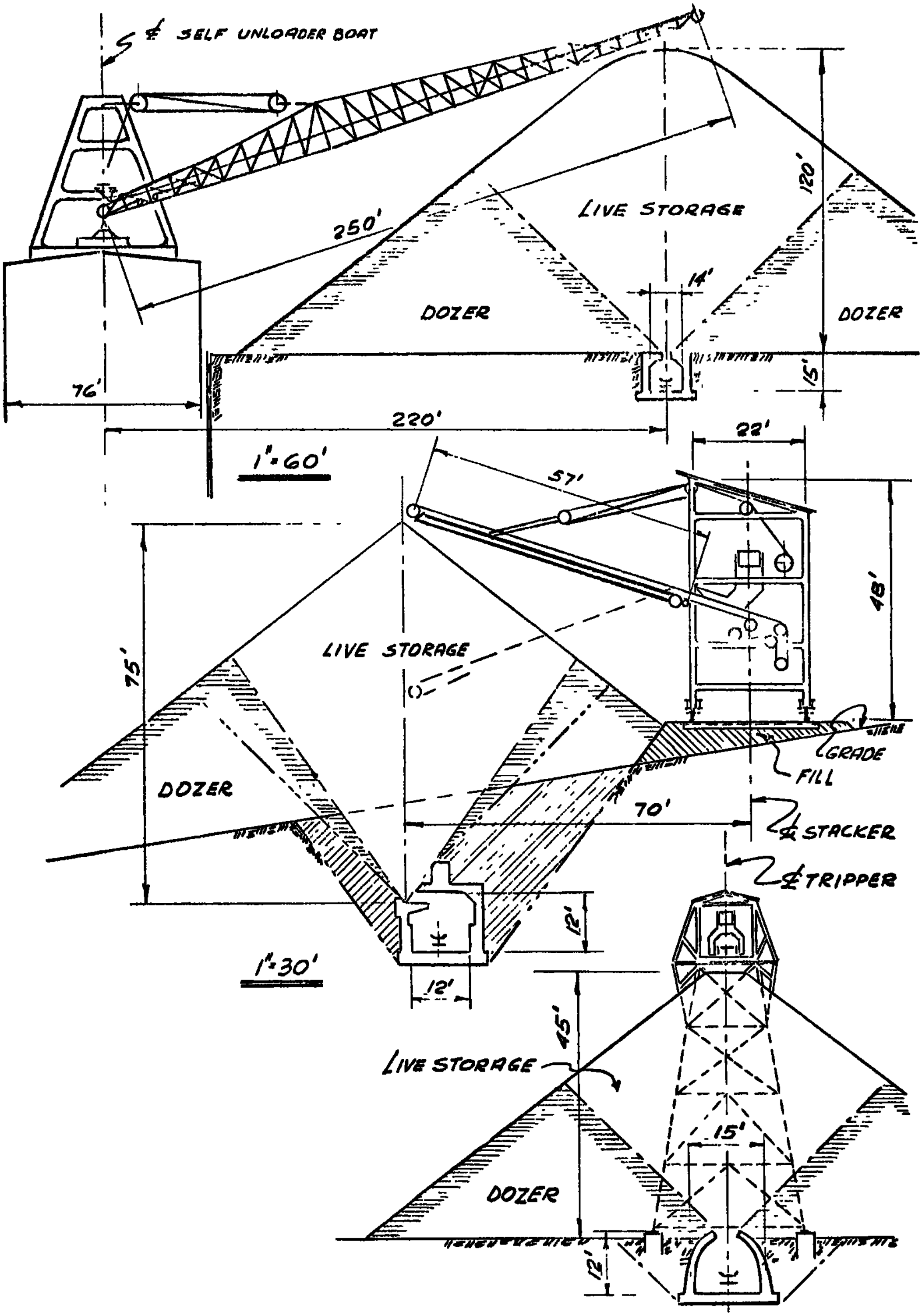


Figure 1. Sections through typical storage piles.

1. Tunnel fully loaded with either a triangular or prismatically shaped pile in cross section (full load).
2. Tunnel loaded from both sides by two triangularly shaped piles and no overburden load on the center of the tunnel.
3. Tunnel loaded from one side by triangularly shaped pile and no overburden load on the center or far side.
4. Intermediate loadings between a, b and c.

In order to design the tunnel shell it is necessary to know the pressure distribution around the tunnel under all conditions of loading. But because of the flexibility of the tunnel and the unusual configuration of some of the loadings, the use of available earth pressure formulas are questionable.

It has been the custom with some designers to take two-thirds of the pile height and consider it as a full weight on the horizontal projection of the tunnel and forty to fifty percent of that load on the vertical projection of the tunnel.

For the (2) and (3) loading conditions stated above, designers have used Culman's graphical method taking an angle of internal friction ϕ slightly larger than that of repose and fully or partially developed wall friction angle ϕ_1. An angle of sliding of 55 to 60 degrees, half developed ϕ and fully or half developed ϕ_1 have also been used.

Of the many tunnels so designed and constructed, only two failures are known to the writers, and they seem to have been the result of poor workmanship in the field rather than inaccuracy in design. However, the selection of the type of loading for which to design, the type of tunnel to be used and the questionable validity of the method of load distribution used, prompted the present experimental investigation.

An opportunity for this investigation arose through the cooperation of the consulting engineering firm of John F. Meissner Engineers, Inc. of Chicago, the Commercial Shearing and Stamping Company of Youngstown, Ohio, manufacturers of the tunnel lining

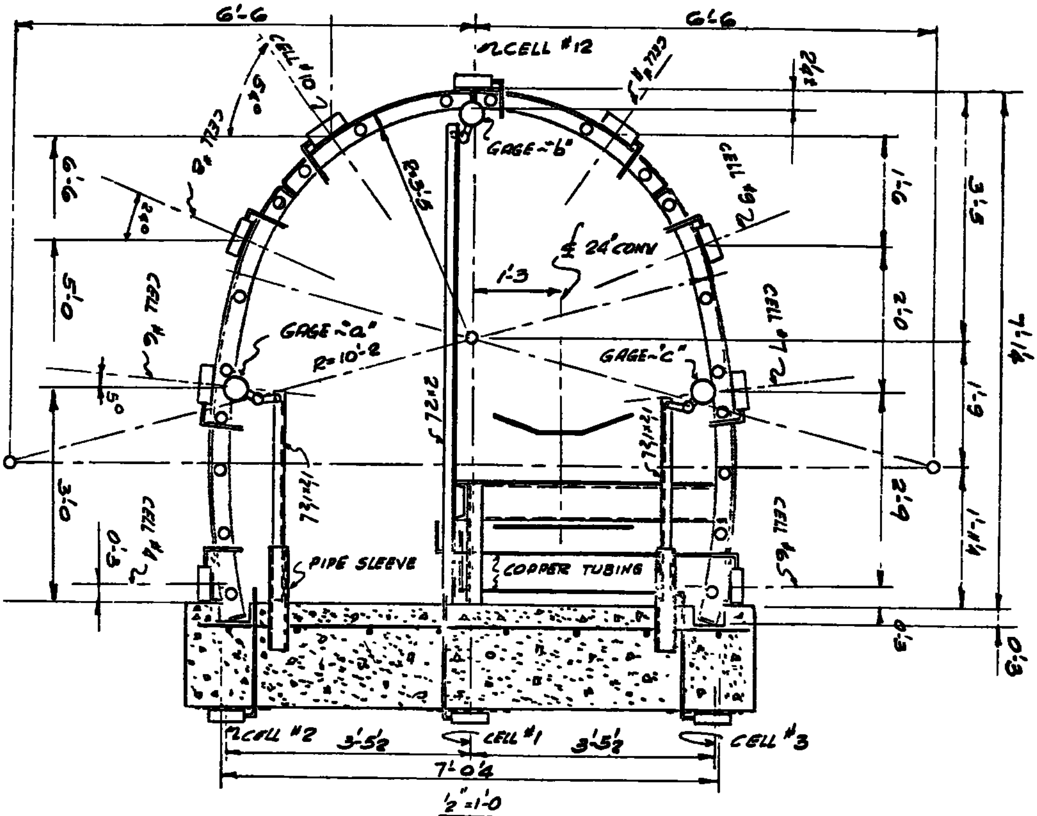


Figure 2. Arrangement of dial gages and Goldbeck pressure cells.

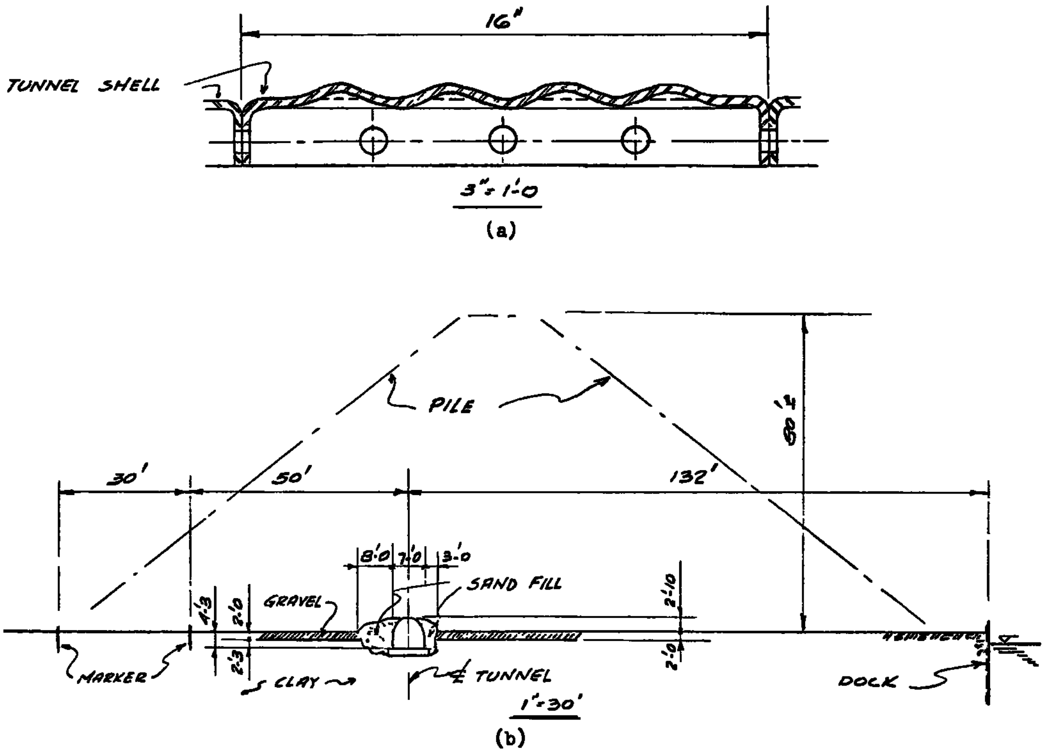


Figure 3. (a) Longitudinal section through liner plate, (b) section through tunnel and maximum size stock pile.

used, and the Marblehead Lime Company of Chicago, Illinois, who granted permission and the necessary labor to perform tests on a newly installed tunnel at their South Chicago Plant.

It was decided to try direct field measurements of pressures on the tunnel shell, together with corresponding deformations of the tunnel walls.

SITE OF TEST TUNNEL AND INSTALLATION OF GAGES

The tests were made on a new tunnel being constructed at the South Chicago Plant of the Marblehead Lime Company. The tunnel under study was to serve as a reclaiming unit for $1\frac{1}{2}$ in. crushed limestone where the storage piles over the tunnel would at times reach a maximum height of as much as 80 ft.

Figure 2 shows a section through the completed tunnel. It was constructed of $\frac{1}{4}$ in. steel liner plate manufactured by the Commercial Shearing and Stamping Company. The sections of plate used were 16 in. wide and of different lengths to fit tunnel sections of varying radii. They were corrugated in one direction and flanged on all sides with bolt holes in the flanges for connection to adjacent sections. Figure 3 (a) shows a section through a typical liner plate and Figure 3 (b) the relative proportions of the tunnel and stock pile. The springing line of the tunnel was located approximately 4 ft - 4 in. below the grade and bolted to an 18 in. reinforced concrete slab. The crown was exposed approximately 2 ft - 9 in. above the grade. The excavated area surrounding the tunnel was backfilled with sand after the tunnel was constructed, Figure 4 (a).

Goldbeck pressure cells were used for measuring pressures and these were mounted underneath the slab and around the outside of the tunnel at the locations shown in Figure 2. Cells 1, 2 and 3, underneath the slab failed to function shortly after being installed so no readings were obtained from these cells. All other cells seemed to function satisfactorily during the entire test period. All cells had been carefully calibrated and checked in the laboratory before installation.

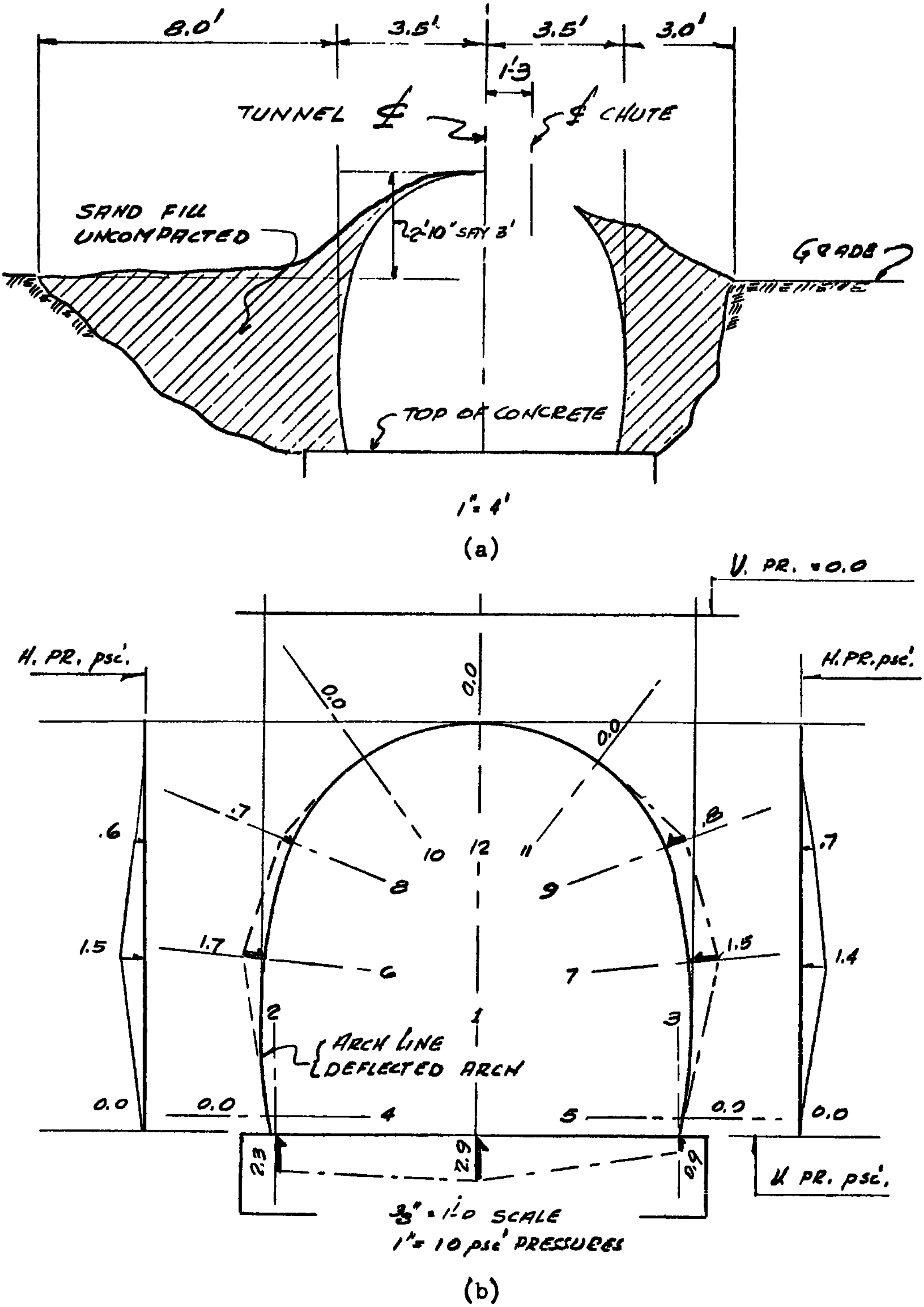
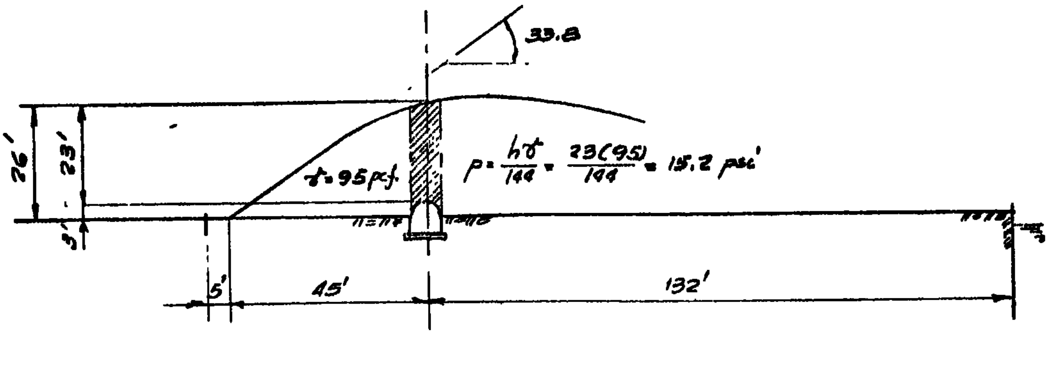
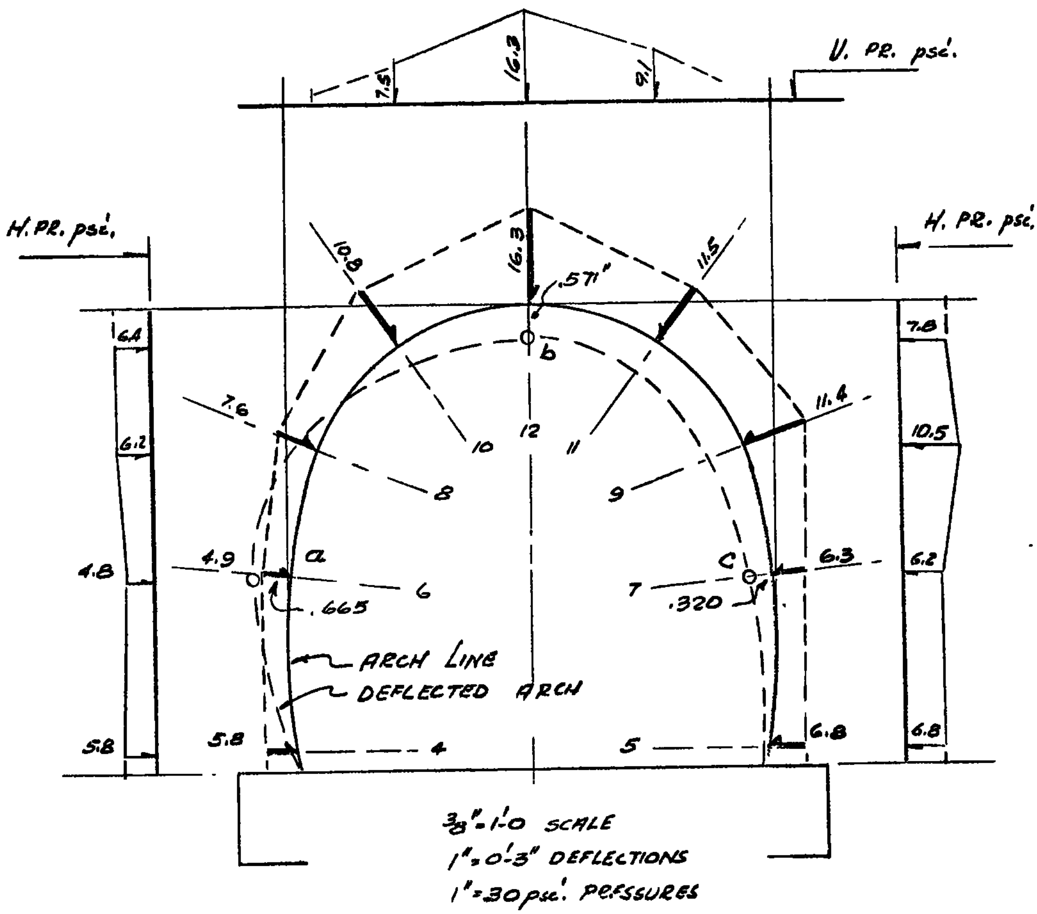


Figure 4. Field measurements 1, (a) backfilled tunnel before loading (b) gage measurements.

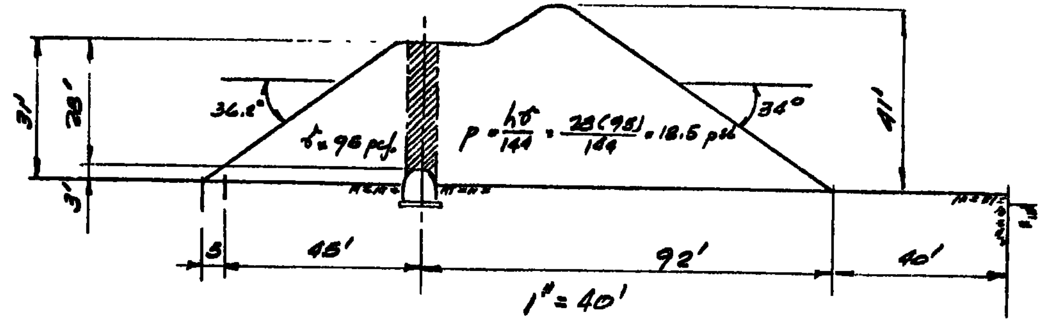


(a)

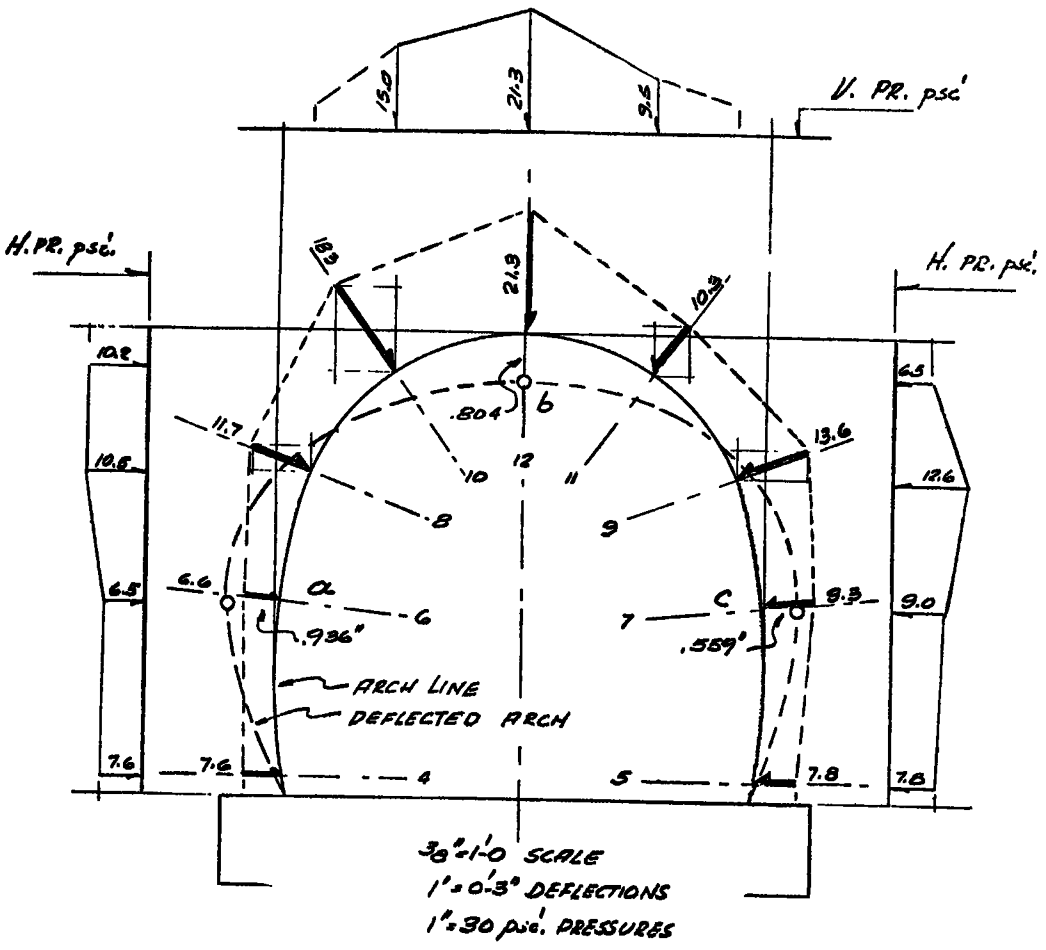


(b)

Figure 5. Field measurements 2, (a) loading diagram, (b) gage data.



(a)



(b)

Figure 6. Field measurements 3, (a) loading diagram, (b) gage data.

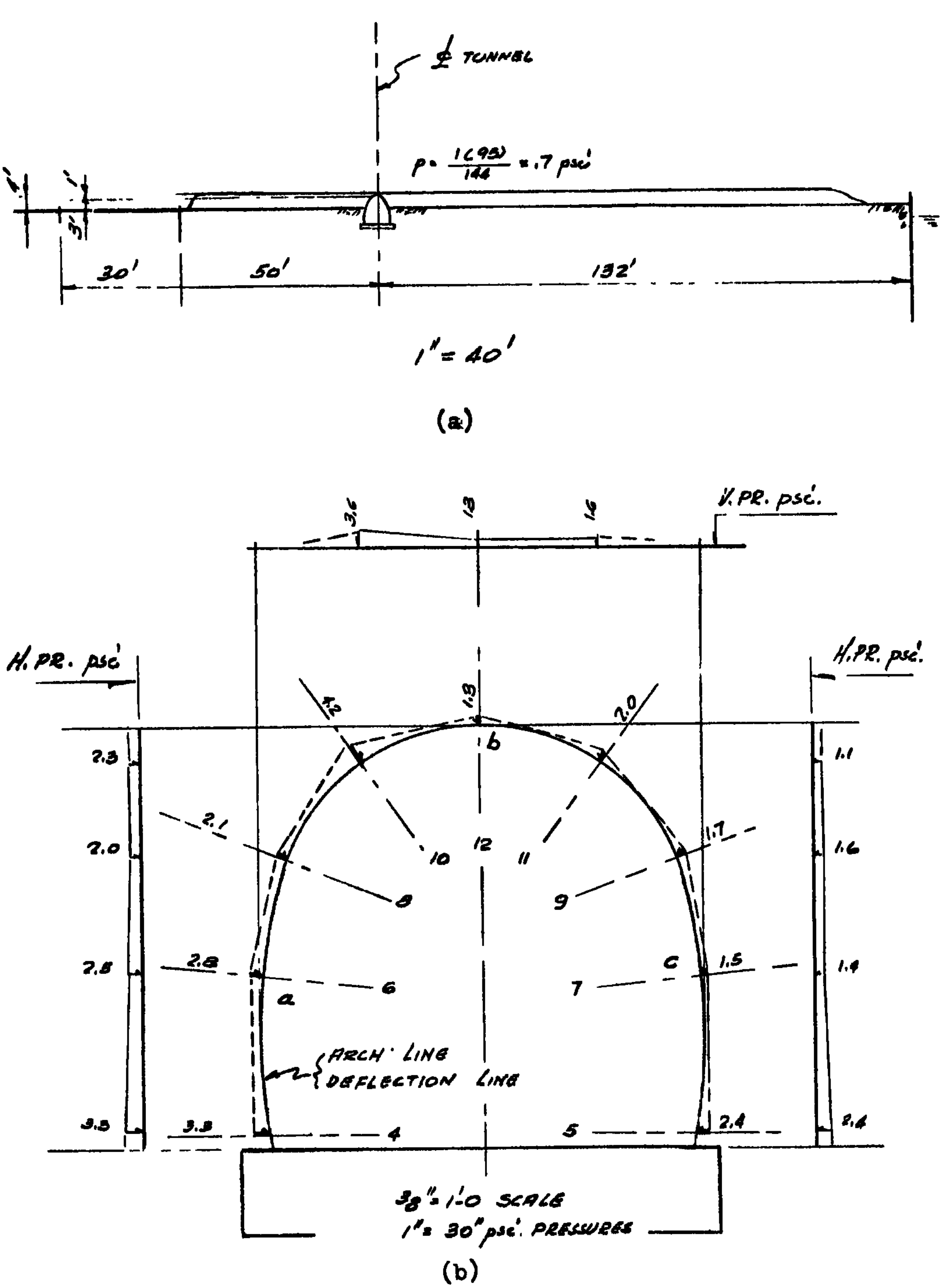


Figure 7. Field measurements 4, (a) loading diagram, (b) gage data.

In addition to the pressure cells a number of dial gages reading to one thousandth of an inch were mounted inside the tunnel at pressure gage locations 6, 7 and 12. These were used to determine how the deflections varied with pressures. The location of the dial gages are shown in Figure 2. They were mounted on steel angles cantilevered from 2½ in. pipe sleeves set in the concrete base slab.

Cells 4 through 12 were secured against slipping and tilting by means of welded back plates, clips and hooks. The diaphram of every cell was kept unobstructed, and parallel to the slope of the tunnel wall. The cells were checked before and after sand backfilling around the tunnel and no appreciable readings on the cells could be recorded after backfilling.

FIELD DATA

Readings of pressures and deflections were recorded at various stages of loading as the crushed stone pile was built up over the tunnel. The cross section of the stock pile was determined by making a survey of the pile at the gage section. Figures 4 through 12 show the plotted data. Each figure shows a cross section of the tunnel and stock pile at the gage location and the measured radial pressures are plotted around the tunnel as well as their horizontal and vertical projections. The displacements of the tunnel are also plotted from the original arch line, at an exaggerated scale for clarity.

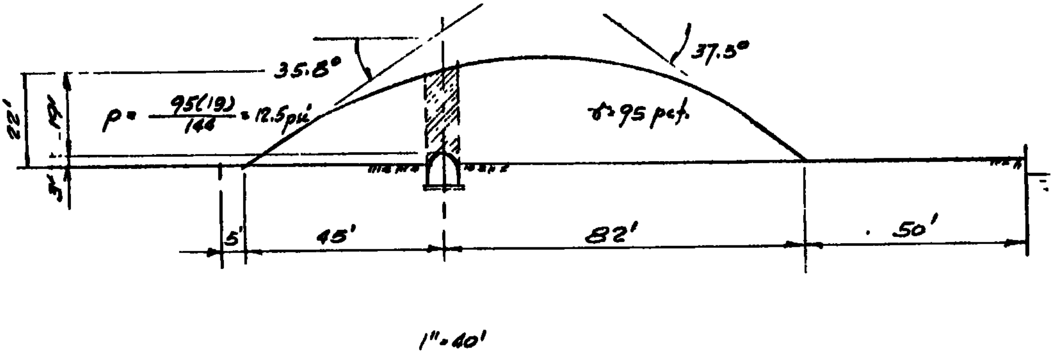
The measurements were performed in two cycles. Figures 4, 5 and 6 represent a full cycle of loading only. Figures 7 through 12 show a full cycle of loading, reclamation and complete unloading. The displacement of the tunnel in every case was measured with respect to the unloaded condition Figure 4 in the first cycle and Figure 7 in the second cycle. Figure 12 shows the final pressures and deflections of the unloaded arch before the gages were removed.

The radial pressure data plotted on Figure 4 through Figure 12 were made up of averages of second, third and fourth readings. The first readings usually were fifteen to twenty percent higher than the following ones. This could have been caused by the piston sticking to the body of the cell requiring an additional internal pressure to break this seal. The first release of air and the second and third injections or releases showed about the same pressures. These pressures were recorded for use. With an increasing number of injections within a short period of time the pressures on the cell pistons gradually decreased. The phenomenon could be due to the movement of the cell cylinder. Even a very short stroke could temporarily displace the sand behind the piston thus decreasing the pressure on the cell. The average of the few readings as explained, were corrected for zero error as previously determined in the laboratory and the results plotted in Figure 4 through Figure 12.

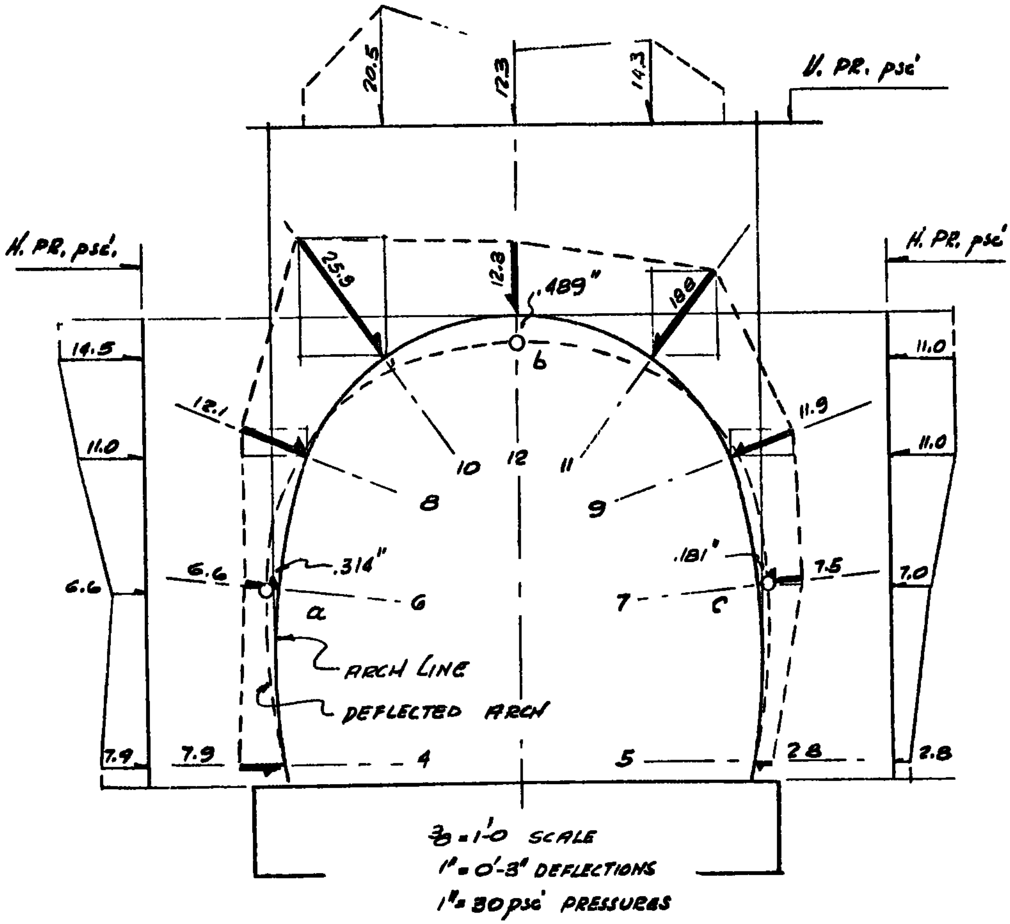
SUMMARY OF FIELD DATA

In developing general pressure distributions for use in design, two conditions were considered. The first condition was when the greatest height of stock pile was directly over the tunnel crown as represented in Figures 5, 6, 8 and 9. The second was when two stock piles of maximum height stood on both sides of the tunnel with no load on the crown as shown in Figure 11. It is apparent that for design purposes the most severe condition of loading is the first. From the nine sets of data taken, only four showed a full load and these are represented in Figures 5, 6, 8 and 9. However, in the summary the data shown in Figure 8 was omitted because some of the gage readings were so far out of line with the other measurements that they looked unreasonable.

For the first condition the measured pressures were averaged for the remaining three cases and the vertical component of the radial pressure over the full width of the tunnel computed. This pressure was then expressed as the product of a pressure factor and the overburden weight and the derived pressure diagram was approximated by a trapesoid. The lateral pressure was then computed as the horizontal component of the radial pressure on a vertical plane from the crown of the tunnel to the base and the pressure distribution approximated by another trapezoid. The vertical pressure was constant and approximately equal to 1.1 times the overburden over the middle third of the horizontal projection. Over the outside third on each side the pressure tapered to



(a)



(b)

Figure 8. Field measurements 5, (a) loading diagram, (b) gage data.

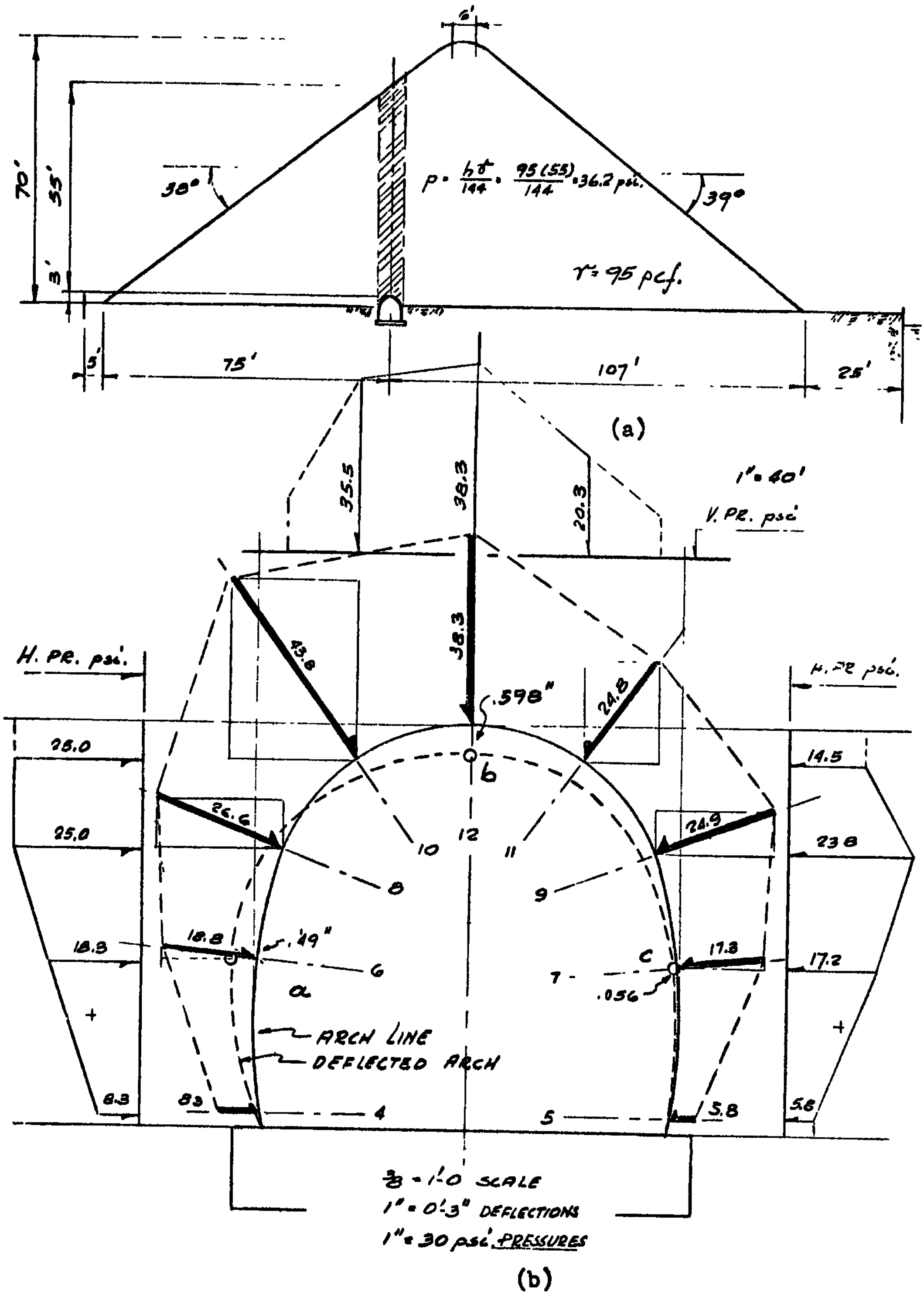


Figure 9. Field measurements 6, (a) loading diagram, (b) gage data.

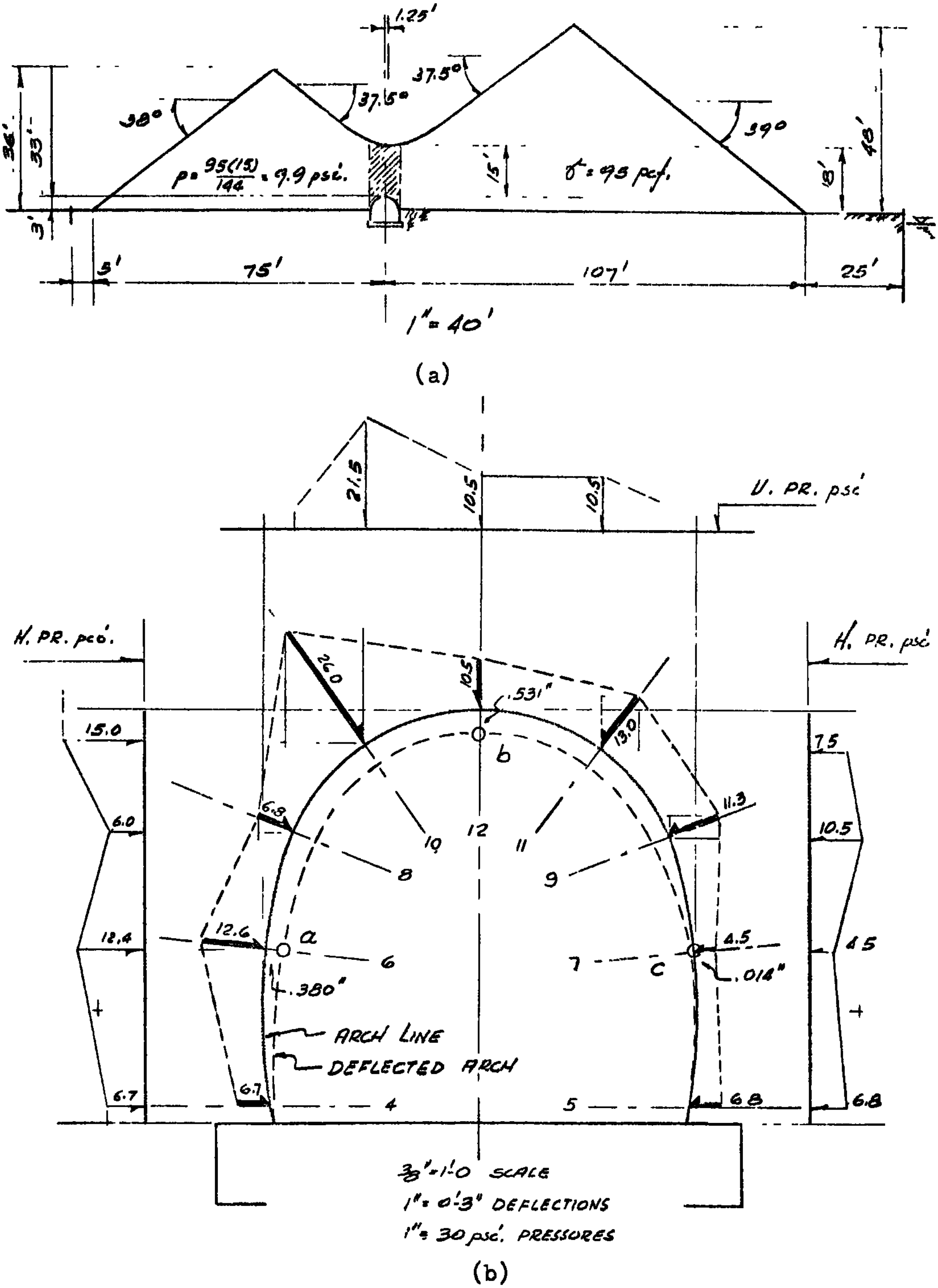


Figure 10. Field measurements 7, (a) loading diagram, (b) gage data.

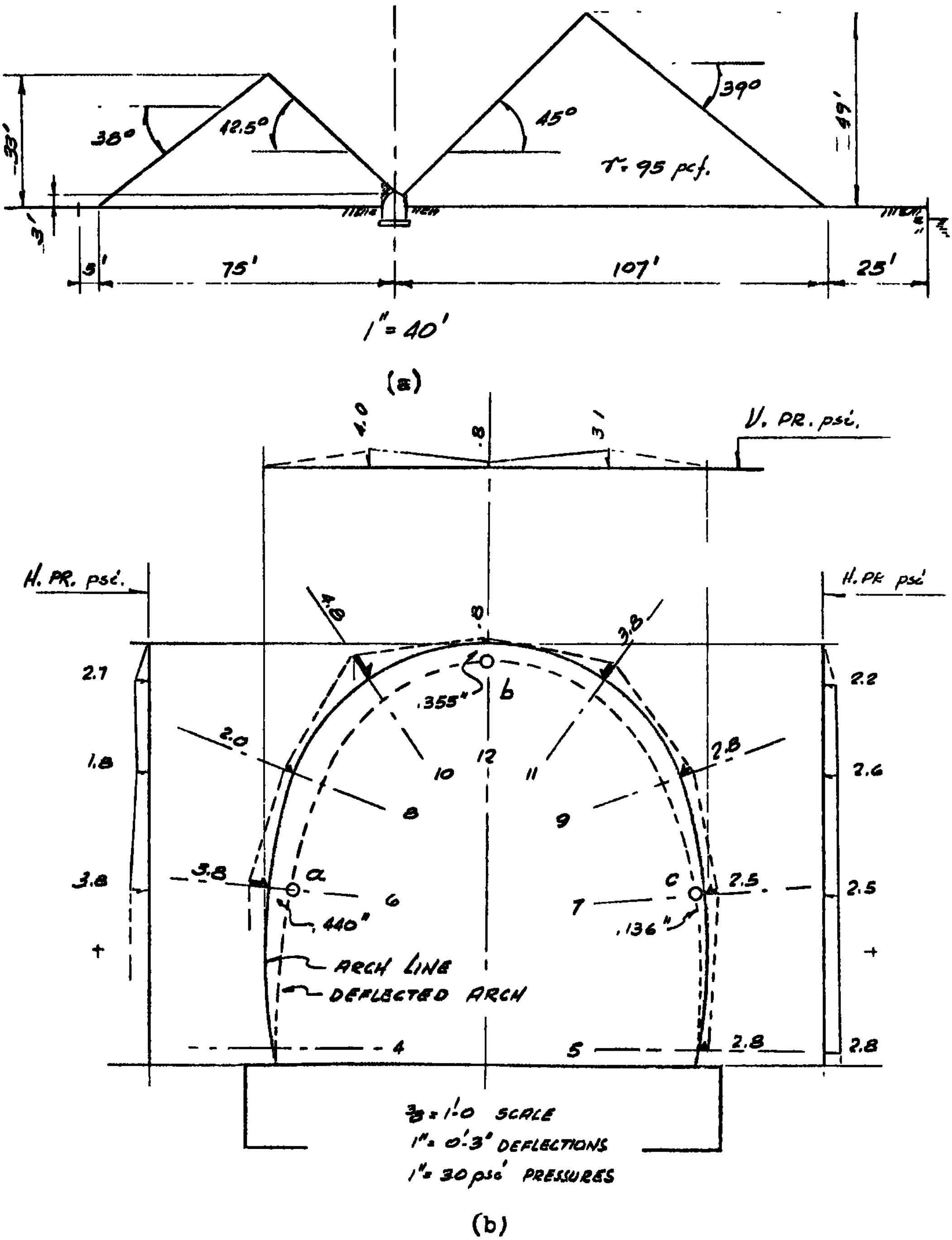


Figure 11. Field measurements 8, (a) loading diagram, (b) gage data.

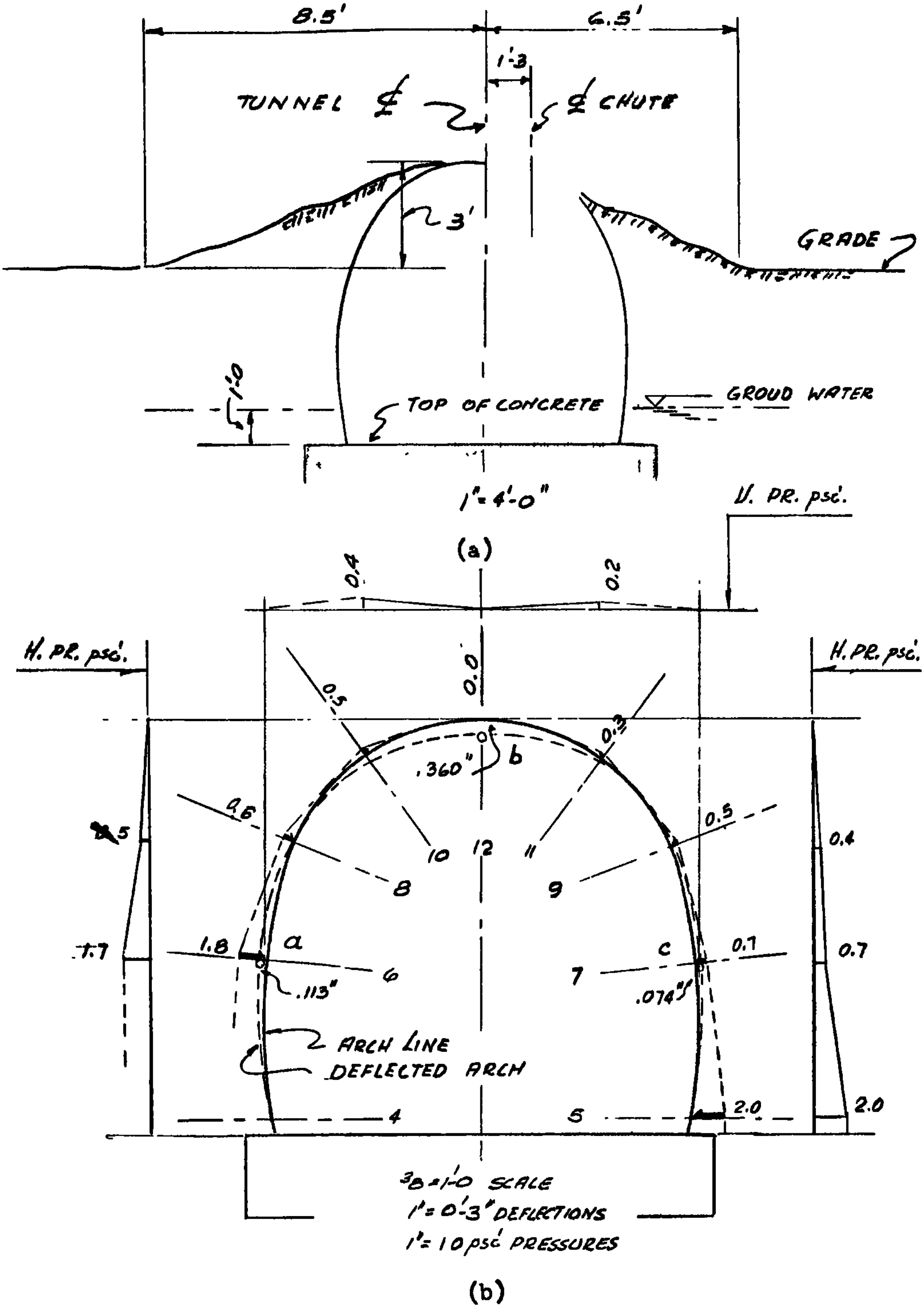


Figure 12. Field measurements 8, (a) unloaded tunnel, (b) gage data.

zero. The same procedure was followed for arriving at a lateral pressure diagram. In this case the pressure was approximately constant over the upper third to the height of the tunnel and equal to about 0.55 times the overburden (γH) over the crown. Over the lower two-thirds the pressure varied linearly from the upper value of $0.55\gamma H$ to approximately 0.12 times the full overburden pressure $\gamma(H + h)$ at the bottom.

These pressure diagrams were constructed to give the total measured force over the vertical and horizontal projections and at the same time to approximate as closely as possible the actual distribution of the three loadings. A summary of the results is shown in Table 1 and Figure 13 shows the derived pressure distribution diagrams.

The second loading condition represented in Figure 11 shows the tunnel loaded on each side with no load directly over the crown. The recorded data show comparatively small pressures. Apparently the shear resistance on planes parallel and adjacent to the slopes is large enough to resist nearly all the shear forces. This is reasonable since fracture and sliding of the outside of the pile occurred as the material was withdrawn until the final inclinations of the separated pile slopes were reached. These slopes represent final fracture planes.

Deflections in the first loading cycle were measured from the initial condition shown in Figure 4. Figure 5 shows some lateral movement of the arch toward the left. Apparently the load was built up on the tunnel from the right side due to the boom location of the self-unloader boat. A higher overburden load (Figure 6) produced a much higher deflection of the crown resulting in almost a symmetrical deflection of the sides. In this

TABLE 1

SUMMARY OF FIELD DATA

Fig. No.	Lateral Pressures Gage No. psi					Height of Pile H ft	Vert Load $H\gamma$ psi	$\frac{\text{Lat Pr}}{H\gamma}$ %	Height H + h ft	Vert Load $(H + h)\gamma$ psi	$\frac{\text{Lat Pr}}{(H + h)\gamma}$ %	
	8	10	11	9	Avg							
5	6.2	6.4	7.8	10.5	7.7	23.0	15.1	51.0	23 + 7			
6	10.5	10.2	6.5	12.6	10.0	28.0	18.6	53.7	28 + 7			
8[a]	11.0	14.5	11.0	11.0	11.9	19.0	12.5	~~95.4~~	19 + 7			
9	25.0	25.0	14.5	23.8	22.1	55.0	36.2	61.3	55 + 7			
	Gage No.					Av Fig 5, 6, 9		55.3	←			
	4	6	7	5	Avg	Av Fig 5, 6, 8, 9		~~65.3~~	a			
5	5.8	4.8	6.2	6.8	5.9				30.0	19.7	30.0	
6	7.6	6.5	9.0	7.8	7.7				35.0	23.0	33.4	
8[a]	7.9	6.6	7.0	2.8	6.1				26.0	17.1	~~35.6~~	
9	8.3	18.3	17.2	5.8	12.4				62.0	40.9	30.4	
									Avg Fig. 5, 6, 9		31.3	← b
	Vertical Pressures Gage No. psi							%	Vert Pr (data average) / $H\gamma$			
	10	11	Avg									
5	7.5	9.1	8.3			23.0	15.1	55.0				
6	15.0	9.5	12.3			28.0	18.6	51.3				
8[a]	20.5	14.9	17.7			19.0	12.5	~~119.0~~				
9	35.5	20.3	27.9			55.0	36.2	56.2				
						Avg Fig. 5, 6, 9		54.2	← c			
	Gage No. 12 psi											
5		16.3				23.0	15.1	108.0				
6		21.3				28.0	18.6	114.5				
8[a]		19.0				19.0	12.5	~~98.7~~				
9		38.3				55.0	36.2	105.5				
						Avg Fig. 5, 6, 9		109.3	← d			

[a] Figure 8 data in this summary not used.

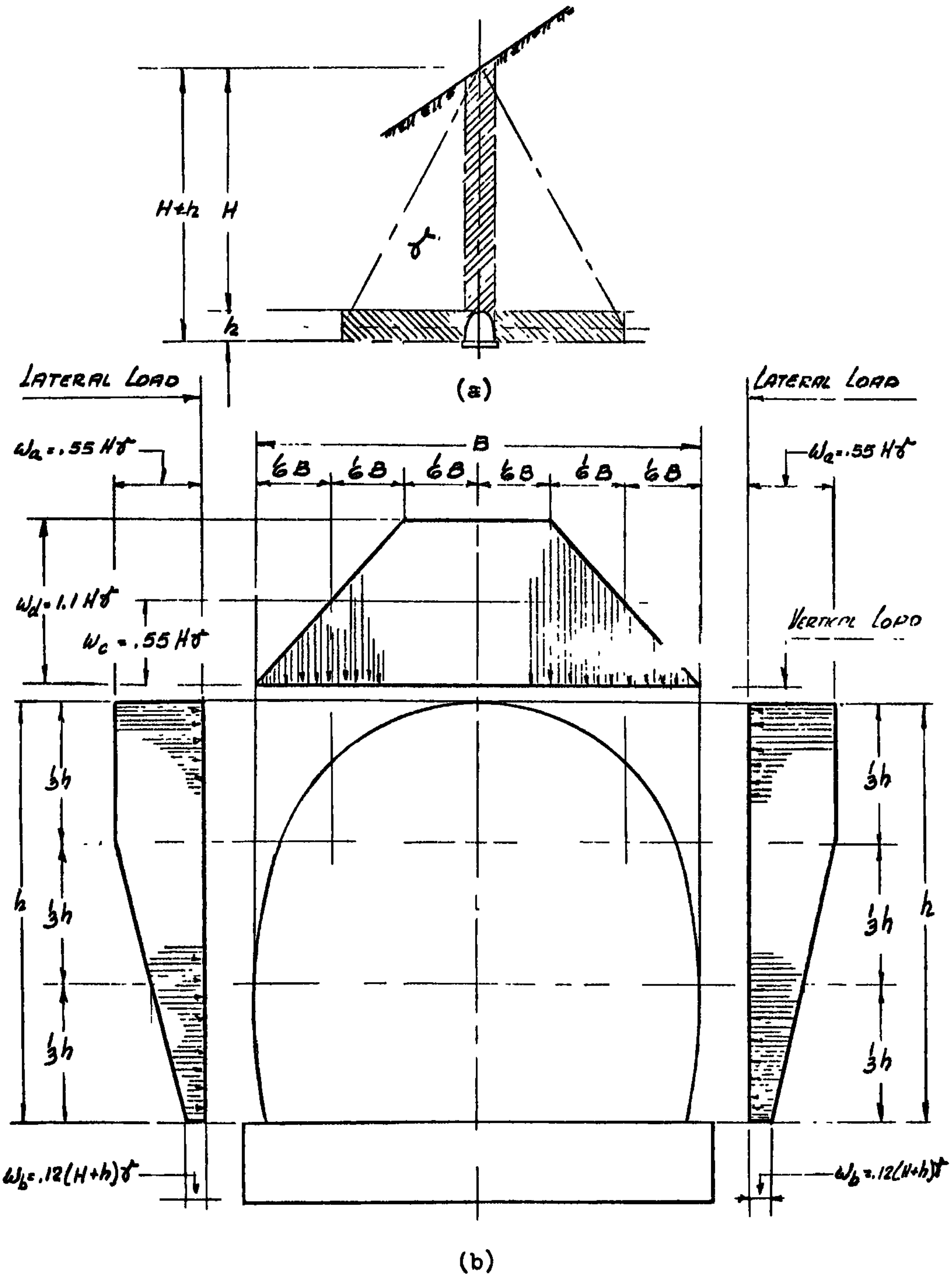


Figure 13. Recommended design loading, (a) loading diagram, (b) pressure diagram.

cycle no measurements of the deflections during the unloading of the tunnel were taken.

The second cycle started with the measurements shown in Figure 7 and this position of the arch was taken as the base for all following loading conditions. A nearly symmetrical deformation of the tunnel shell is shown in Figure 8 followed by some sidesway due to the unsymmetrical loading shown in Figure 9.

As the overburden load decreased, leaving in place two piles on the sides of the tunnel (Figures 10 and 11), the lateral deflections changed their directions and the tunnel moved inwards. At the same time the crown moved up a little, but did not reach the original position of Figure 7.

The deflection line did not reach the original tunnel line even at the complete unloading, Figure 12. This was probably caused by some permanent bending, shortening of the shell or readjustment of the bolted and painted joints.

The deflections are larger on the left side of the tunnel than on the right side. This could be due to a wider sand backfilled area on the left side, thus allowing more compression and larger deflections of the left rib of the arch.

DISCUSSION OF RESULTS

It is believed that the Goldbeck pressure cells were quite reliable and the data which they gave were quite accurate measurements of the pressures acting on the face of the cell. There are several factors, however, which may have influenced those pressures and this raises the question as to whether the values measured at any given location were the true pressures acting on the tunnel lining.

The cells were mounted on the outside surface of the tunnel which meant that the cell protruded beyond the surface of the wall approximately 2 in. This together with the rigidity of the cell and the stiffening effect of the small angles used to hold the cell in place undoubtedly had some effect on the yielding of the tunnel lining and hence on the pressure. A factor which apparently has a very pronounced effect on the measured pressure is the movement of the sensitive part of the cell face necessary to actuate the gage. It has already been pointed out that for a particular cell reading the pressure necessary to get the initial reading was abnormally high. The next three or four pressure cycles gave essentially the same value and subsequent cycles gave decreasing pressures. The first high value was attributed to sticking of the movable piston. The values which could be duplicated three or four times were taken as the true pressures, but it is obvious that the nature of the gage operation, i. e., movement of the cell face, although extremely small, against the granular fill is an undesirable feature of the gage. Any cell which undergoes a deflection whether produced by the fill or by being forced against the fill is going to be affected by such movement.

Probably the most meaningful pressure measurements are those under the heaviest overburderns, because the effect of the factors described above would be proportionately small compared to such effects as rigidity of the base slab, relative flexibility of the joints and the initial distortion of the tunnel. The latter would vary depending on where the first overburden loads were placed. The derived pressures shown in Figure 13 are therefore believed to be accurate enough for ordinary design purposes.

The average value of the pressures directly over the crown was found to be 10 percent greater than the overburden. This was based on the three loadings shown in Figures 5, 6 and 9. For the loading shown in Figure 8 the value was almost 2 percent less than overburden and if this value were included the average would be about 7 percent greater than overburden. Ordinarily the crown pressure might be expected to be less than overburden due to the shear resistance of the fill on vertical planes and to arching effects. If the tunnel had been made through an existing fill there probably would have been enough vertical movement of the overburden for this to occur. But because the fill was built up after the tunnel was in place and because the material flowed out over the crown from the unloader boom it was difficult for the static shear resistance and arching to develop. The static deflection which actually occurred took place very gradually and during the early stages of loading there were probably some small sudden deflections due to the dynamic effects of the moving stone. These deflections and corresponding dynamic pressures would tend to be sealed in by the resistance of the existing overburden. The same effect has been observed to produce abnormally high anchor tensions in anchored sheet piling as a result of compacting the fill above the anchor level.

The lateral pressures do not vary much over the depth of the tunnel for moderate overburden heights but they seem to taper off near the bottom for the greater overburdens. This is much more pronounced on the side of the tunnel which deflects inward from the original arch line. It is quite apparent that the lateral pressures are very much influenced by the way the tunnel lining deflects during loading operations. The pressures are largest where the tunnel shows the greatest outward movement and smallest where the greatest inward movement occurs. The manner in which the tunnel lining deflects depends principally on the looseness and symmetry of the initial sand fill and the side from which the tunnel is first loaded. The relative flexibility of the bolted joints also affects the deflection picture.

Results of the tests described in this report show that there are a number of variables which influence the pressures of granular earth on flexible tunnel lining of this type. As a next step in the investigation it is recommended that an attempt be made to separate the variables and evaluate each one independently, so that it may be possible to estimate their relative effects under actual field conditions.

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