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“Experiments on Stoneware Pipes and Pipe-Sewers.”

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ALTHOUGH stoneware pipes have been used in this country for upwards of 80 years for the conveyance of sewage, little appears to be known regarding their properties under actual working conditions. The determination of the crushing loads of pipes under concentrated loads has been made on several occasions, but very little work has been recorded. The earliest records are those of Bazalgette and Haywood in London in 1855-56. Not only do these two gentlemen appear to be the earliest workers, but with the exception of Mr. Baldwin Latham, M. Inst. C.E., they are the only engineers who have made any communication relative to their experiments. Experience indicates that manufacturing methods have improved since 1856, a superior pipe being the result. A need therefore exists for a complete investigation of the properties of sewer pipes.

Where the depth of trench exceeds 12-15 feet it is usual in this country either to surround the pipes with concrete or to use pipes of increased thickness, or sometimes to use both, regardless of the factor of safety which is obtained by using the pipe unprotected.

The Author set himself to ascertain when it becomes necessary to protect a pipe-line, and what knowledge is needed before the line can be designed to perform its duty unaided.

The efficiency of a pipe-line depends upon the integrity of the pipes, and of the joints, and the conditions affecting the flow of sewage in pipes.

A line of pipes may fail owing to the following causes :—

- (a) Weight of the superincumbent earth.
- (b) Impact loads in shallow trenches along roads.
- (c) Failure of the bed on which the pipe is laid.
- (d) Suspension of the pipe due to joint holes not being properly excavated.
- (e) Deterioration of the pipe material.
- (f) Deterioration of the jointing material.

The pressure on a pipe in a trench is not directly proportional to the depth, as seems to be the opinion of many engineers, for the earth itself acts more or less as a beam to support part at least of the weight of earth above the pipe. The percentage of the total weight which is transmitted to the pipe depends on the nature of the earth, the depth of trench, and the manner of returning the earth.

Failures from earth-pressure have generally been due to lack of supervision during the haunching-up process. If the returned earth is not carefully packed and properly rammed under and around its sides, the pipe, as far as its underside is concerned, is loaded with a concentrated load when the trench is refilled. To guard against this element of improper packing, it may be wiser to specify that pipes should be able to withstand a certain concentrated load than that they should be able to withstand a certain uniform load. The conditions under which a concentrated-load test is applied are more easily duplicated than with a uniform load; so that any number of concentrated-load tests may be carried out under the same conditions.

Uniform-Load Tests.—These were carried out in the following manner:—A box was made with planks (9 inches by 3 inches) securely bolted together, the internal dimensions being 3 feet 6 inches (*Fig. 1*). Into this was placed the pipe to be tested, sand being carefully packed and rammed around it, 6 inches of sand-covering being given to the pipe. A timber platform of 3 inches oak, as long and as wide as the pipe, was placed on top of the sand. The load was communicated to the pipe by a lever, the middle knife-edge of which was carried on a hardwood block resting on the centre of the platform. A hole cut in the end of the box enabled the interior of the pipe to be examined as the loading proceeded. The first crack was in all cases taken to be the point at which failure took place. This crack was without exception longitudinally top and bottom, followed almost immediately by cracks at the sides, thus dividing the pipe into four portions. On continuing the loading the pipes were able to stand pressures greater than that which produced the original failure, this being due to the portions of the pipe taking a new bed. With but few exceptions it was impossible to crush the pipe completely under test.

The Author has noticed occasionally in trenches, especially after a heavy storm of rain, a longitudinal crack in the returned earth due to insufficient packing of the material under and around the pipes, settlement, or, in other words failure, of the abutment of the pipe-arch having taken place. To test the effect of an unyielding abutment on the strength of the pipe,

Fig. 1.

pipes haunched with concrete were loaded in the machine previously described. They were able, without exception, to stand much greater loads before failure than were similar pipes embedded in earth as in an ordinary trench.

It is a condition of pipe-laying contracts that the best material must be selected to be returned first; if this be done and it be well rammed, the conditions of actual practice and of the test are as nearly alike as it is possible to obtain. It is, of course, assumed that in the trench the bed upon which the pipe rests is of a sound nature, and that bell-holes are properly excavated to avoid suspension of the pipe.

Concentrated-Load Tests.—The load at failure in this test is much less than in the

case of that of a uniform load, so that in a properly refilled trench the pipes will withstand greater loads before failure than those obtained in the concentrated-load test. The pipes were supported on two strips of wood, 2 inches by $\frac{1}{2}$ inch by 15 inches long, placed 15 inches apart on the platform of a hydraulic press (Fig. 2). The load was transmitted from the piston to the pipe

Fig. 2.

through a metal plate (18 inches by $\frac{1}{2}$ inch by $\frac{3}{8}$ inch) resting on an india-rubber pad. The pressure gauge was calibrated to read the total load in pounds direct, as well as the pressure in pounds per square inch. For pipes of small diameter the pressure from the mains only was used, while for those of larger diameter hydraulic accumulator pressure was utilized.

The results, given in Tables III and IV in the Appendix, show

that stoneware pipes are not of the same average strength for all diameters. It is not possible to obtain a uniformity in strength per lineal foot or per square foot for all diameters at the same time, for with the same strength per lineal foot a 9-inch pipe is three times as strong per square foot as a 27-inch pipe, hence a uniformity per lineal foot is not a criterion of the bearing strength; and for a uniformity in strength per square foot either the smaller pipes would have to be too thin or the larger ones too heavy to handle, but for convenience the strength per lineal foot is to be recommended. Pipes up to 12 inches in diameter may be required to withstand a uniform load of 3,000 lbs. per lineal foot, and the larger ones 4,000–5,000 lbs. per lineal foot before failure, or concentrated loads of 1,500 and 3,000 lbs. per lineal foot respectively.

The degree of burning to which the pipe is subjected greatly affects its strength. The operation of burning is at present a very unscientific one; and although it may not be possible for this to be carried out with any very great degree of refinement, owing to varying atmospheric conditions, there is much room for improvement.

Haunching of the pipes with concrete in good ground may be resorted to when the factor of safety is low. Where the ground is uncertain, owing to the presence of sand liable to "run," the pipes should in any case be haunched with concrete, and also have a bedding of the same material.

Theoretically the relation between the load and the thickness of the pipe material is given by the following equation:—

$$P = \frac{KT^2}{D + kT}$$

P = uniform load expressed in pounds per lineal foot.

T = thickness in inches.

D = internal diameter in inches.

K and k are constants.

These constants must take into account the varying manufacturing conditions, the degree of ramming of the earth under and around the pipe, i.e., the stability of the abutment, the percentage of the superincumbent earth borne by the pipe, the modulus of rupture of the material of each pipe, and the influence of the bell of the pipe. The thickness for a specific load per lineal foot may then be determined from the formula or its graphical representation. The small number of the Author's experiments unfortunately do not afford sufficient information for the determination of K and k .

It was not possible to crush the pipes of diameter greater than

12 inches by a uniform load, so that concrete around these pipes would only prevent the occurrence of possible cracks, and thereby lessen the liability to leakage. When a fire-crack previously existed at the top, bottom, or mid-way points, it always caused premature failure; whereas when the blemish was placed at 45° to the horizontal, the pipe was as strong as one which was sound. It is therefore not wise to place a pipe in a trench so that a fire-crack comes on top, as is usual, but rather at some point between the horizontal and vertical positions.

The load which, in an ordinary trench in good ground, may be expected to be transmitted to the pipe, may now be considered. This the Author has measured in actual sewer trenches, the method being as follows:—At the bottom of a trench (*Figs. 3*) were placed two oak sills, 8 inches by 4 inches in section, and 6 inches longer than the width of the trench, supported on a brick foundation. Four coiled springs of $\frac{3}{4}$ -inch square steel, 3 inches internal diameter and

Figs. 3.

16 inches high, were placed on bearing-plates (see detail, *Figs. 3*) on the sills. A platform 5 feet long, of timbers as shown, rested on the springs. On the underside of the platform, oak strips, 7 inches by $1\frac{1}{2}$ inch, acted as the bearing surface for the metal plates on the springs. To prevent the latter closing beyond the point at which deformation ceased to be proportional to the load, metal stops were used, as shown in the detail. With a 3-ton load each one was compressed 7 inches approximately, thus giving for the platform a compression of 7 inches with a 12-ton load. The springs were calibrated before and after use, as was also the platform when erected (*Fig. 4*). Deflections were measured by a telescope and graduated rod attached to iron pins let into the side of the platform. The ends of the trench were sheeted, and, to allow free movement of the platform, it was 2 inches narrower than the trench. The 1-inch space on each side was covered up with waterproof sheeting, fastened to the platform and to the timbering,

to prevent the earth from falling between the platform and the sides of the trench. The loading was carried out in 6-inch layers, each layer being rammed, viz., two men ramming to one filling in. Movements of the spring were read for each 12-inch increase of depth of earth; from the calibration curve the loads corresponding to these movements were obtained. The weight of earth per cubic foot for each 12-inch layer was measured by a cubic-foot box and spring balance. The percentage of the superincumbent load transmitted to the platform was then calculated for each

Fig. 4.

additional 12-inch layer. (See Tables VA, VB and VC in the Appendix.)

The results show that pipes laid in a trench in clay have a greater percentage of the load above transmitted to them than at the same depth in sand or gravel. The relationship between depth and the load transmitted is shown by the curves in *Fig. 5*, the percentage load transmitted varying between 80–38 per cent. in gravel, 84–28 per cent. in sand, and 97–57 per cent. in clay for trenches of from 1 to 10 feet in depth.

From the curves it appears that the cohesion rapidly increases to a depth of about 4 feet, from which point the curves take a more horizontal direction. To a certain depth the increasing ratio of lateral pressure to weight of earth, and the increasing value of the coefficient of friction, together act to lessen the load transmitted to the pipe by increasing the cohesion. As the earth reaches the state beyond which no further compression is possible the cohesion becomes constant, and the curve approaches the horizontal. It appears that the cohesion approaches the constant value at or about a depth of 10 feet below ground-level. On this assumption it may be said that after the ratio of lateral pressure to weight of earth

Fig. 5.

has reached a constant value the percentage of superincumbent load transmitted to the pipe will be constant. The percentage load transmitted depends upon the coefficient of friction of the earth or upon $\tan \phi$. The experiments show that the greatest percentage of the superincumbent earth load which is to be expected in a deep trench is 57 per cent., and that this occurs with clay filling. This was to be expected, since for wet clay $\tan \phi$ is a maximum. If therefore the sewer be designed to meet such a case, its safety is assured in good ground.

The experiments so far described are with uniform loads on the platform only, but as in roads the pipes have to carry a portion of

the vehicular traffic as well as the earth load, it is necessary to consider the effect of such loading on the pipe in trenches of various depths. If vehicular traffic be considered as a concentrated load, the percentage of such load transmitted to the pipe would obviously be greater in the shallow trench than in a deep one. It remains therefore to determine what is the minimum depth of trench which will allow the heaviest load of the district over it without damage to the pipes. In the Solihull area this load is a 10-ton steam-roller, the width of rear wheel of which is 1 foot 6 inches; the chord of contact for a 6-foot diameter wheel is 2 feet 6 inches, the load carried on each rear wheel being 4 tons, or 2,400 lbs. per square foot.

When the experimental trench was completely filled, a load equivalent to 2,400 lbs. per square foot was placed upon it, the deflection of the springs measured and the corresponding percentage of load transmitted to the platform calculated. This percentage load was always about 5 per cent. less than the percentage of earth-load transmitted at that particular depth of trench under consideration. In deducing the probable effect of a steam-roller on a pipe-sewer, the percentage of load transmitted has been taken to be the same as for the earth-load at each particular depth. Assuming that the pipes are of the strength shown in Table I (Appendix), factors of safety for live and dead loads may be obtained by dividing the breaking load per square foot by the load per square foot actually occurring in the trench under consideration. These factors of safety are tabulated in Tables VIA, VIB, and VIC (and see explanation, p. 314).

It has been assumed that the percentage load transmitted is constant for depths greater than 10 feet, and using this constant the probable loads in trenches up to 20 feet deep have been calculated.

In good ground, a factor of safety of 2 may be taken as sufficient to cover imperfections in laying, which might cause undue stress on certain pipes. Where the ground is of an uncertain nature the factor of safety is very variable, owing to the liability of the ground to move. This movement may cause disrapture of the pipe-line as distinguished from cracking from an earth-load, and is referred to later. By suitably increasing the minimum factor of safety allowable, a means is afforded of more readily deciding whether a single or double strength pipe, with or without concrete, is required, and at the same time forming a foundation about which to group future judgments and results. The Author has not been able to obtain any results on pipes of thickness $d/10$, as only those of $d/12$ have been used at Solihull.

It appears from Table VI that in a trench loaded with a live load

the net pressure decreases to a minimum at a depth of about 7 feet, from which point it increases with further increase of depth; whereas with the earth load the pressure increases with the depth continuously.

Concreting the pipes to the haunches (Tables VI*d*, *e* and *f*) increases their strength from 20 per cent. to 25 per cent.; where, therefore, this is sufficient to produce a factor of safety of 2, concrete may be resorted to. Where it is not sufficient, as shown by the Tables, the pipes must be surrounded with concrete, or a double strength pipe, with or without concrete, as the conditions may require, must be used.

The following recommendations are based upon the Tables :—

(1) *Gravel Trenches*.—9-inch pipes require a reinforcement as a protection against live load, i.e., in roads where the depth of cover is less than 2 feet and greater than 18 feet; 12-inch and 15-inch pipes when less than 4 feet and greater than 11 feet deep; pipes of greater diameter always require reinforcing against live load. With the pipes mentioned concrete to the haunches is sufficient to produce a factor of safety of 2. With the 18-inch pipe this concrete is sufficient when the depth is greater than 3 feet and less than 12 feet; for all other depths and sizes of pipes additional reinforcement is required. With a dead load all pipes at all depths up to 20 feet are safe without any concrete.

(2) *Sand and Loam Trenches*.—9-inch pipes are safe for all loads except a live load with depth of cover less than 3 feet. 12-inch and 15-inch pipes require concrete to the haunches when the depth is less than 5 feet and greater than 20 feet, and 18-inch pipes when less than 6 feet and greater than 14 feet deep; 21-inch and 24-inch pipes when less than 8 feet and greater than 12 feet deep; and 27-inch pipes always require concrete against a live load. With a dead load all pipes at all depths up to 20 feet are safe without concrete.

(3) *Clay Trenches*.—Pipes of diameter greater than 18 inches require haunching with concrete against a dead load when the depth exceeds about 14 feet. Against a live load all pipes at all depths require reinforcing. Table VI*f* shows that mere haunching is not sufficient, further reinforcement being required.

Comparing the results with practice, it has been found that 9-inch pipes having a cover less than 2 feet deep in roads which have been intersected by the sewer trench were invariably cracked, so that as far as these are concerned the experimental results are trustworthy. The Local Government Board requires all pipes having a depth of cover in roads less than 4 feet, and in fields less than 3 feet, to be surrounded with concrete. The results show that

for small pipes the requirements are too stringent, and for the larger pipes not sufficiently so. Trenches in gravel, sand, and clay have been selected as general types: for earth varying between these types further experiments are needed.

Hydrostatic Load Tests.—In the winter months the permanent water-level may be at some considerable height above the pipe in a deep trench. If pipes are not impervious the possible leakage depends on the height of the ground water above the pipe. To investigate the behaviour of pipes when under water-pressure they were loaded with an internal hydrostatic load. The pipes were loaded to destruction, so that the tensile strength at that point could also be determined. During the loading the pipes were examined for evidence of sweating, and also for determining the effect of imperfections of the glaze on their imperviousness. The machine used for testing the pipes worked on the same principle as the one used in the concentrated load test (*Fig. 2*). The general arrangement is shown in *Fig. 6*.

The pipe was placed in the machine on an indiarubber pad, the bell of the pipe being uppermost; inside the pipe and resting on an indiarubber ring in the bell was an iron drum, the function of which was to reduce the volume of water required. Water was pumped into the pipe, the air-cocks remaining open. When the air had all been expelled the pressure was increased by an accumulator in steps of 5 lbs. per square inch. The cracks at failure were straight and directly down the pipe when the material was of an open nature, but when the original clay was compact and tough the pipe fractures were irregular. The presence of a fire-crack, or of a cavity in the metal, appeared to govern failure.

The results of the tensile strength tests are given in Table VII (Appendix). In this Table specimens which failed owing to the presence of fire-cracks have not been included. There were, however, variations due to differences in the thickness of the pipe material, so that the figures may be taken to represent fair averages. It may be seen from the Table that pipes may be expected to possess a tensile strength of 600 lbs. per square inch, with an internal pressure intensity of 100 lbs. per square inch for all sizes, the pipes of smaller diameter being able to withstand much greater intensities. If the pipe be considered to be a thin cylinder, by use of the formula $pd = 2tf$,

$$\left[\begin{array}{l} \text{where } p = \text{pressure intensity in lbs. per square inch,} \\ f = \text{tensile strength in lbs. per square inch,} \\ d = \text{internal diameter of pipe in inches,} \\ t = \text{thickness of pipe in inches,} \end{array} \right]$$

the thickness required for an average pipe to possess a tensile

Fig. 6.

strength of 600 lbs. per square inch with an internal pressure

intensity of 100 lbs. per square inch may be calculated. These are shown below :—

Diameter, inches	9	12	15	18	21	24	27
Theoretical thickness, inches .	0·75	1	1·25	1·50	1·75	2·00	2·25
Actual thickness, inches . .	0·90	1	1·25	1·50	1·87	2·12	2·37

The equation $pd = 2tf$ gives a relation $t = d/12$. Pipes are obtainable in this country in two thicknesses, viz., $d/12$ and $d/10$.

Mention has been made of the possible settlement of the bed on which the pipes are laid; upon this depends the integrity of the pipe-line as a whole. It is found that pipes of 15 inches diameter of superior manufacture, jointed with Portland cement, and supported on bearings 8 feet 6 inches apart, will only deflect $\frac{1}{30}$ inch before fracture. Where the pipes are laid upon a soft bed the pressures are often more than sufficient to produce this deflection; therefore, in trenches in rock or moving ground a concrete foundation must be provided for the pipes, though where this should begin and end must depend on the judgment of the engineer.

Watertightness of a Pipe-Line.—This depends on:—(a) The porosity of the pipe; (b) The mode of joining. Engineers do not seem to be agreed as to the permissible leakage in stoneware pipe-sewers. Mr. Rudolf Hering allows 10 per cent. of the daily flow of sewage as infiltration water, whilst many engineers in this country stipulate that the sewers must under all conditions be perfectly watertight.

As ground varying from rock, through clay, to running sand was met with at Solihull, the leakages in different grounds are interesting if only for comparison with those obtained elsewhere.

Diameter.	Cubic Feet per Mile per 24 Hours.
27 inches	174
24 „	204
18 „	150
15 „	105
12 „	65
9 „	40
9 „	25

The foregoing are purely leakages from inside the pipes, and do not include those from bad joints in manholes, as these can always be located and repaired by puddling or concreting round the manhole. The leakages shown above are not allowable leakages, but such as have occurred on the works, the intention being to reduce these leakages to as small an amount as possible.

Porosity of the Pipes.—Pipemakers contend that the salt-glaze of the pipes is impervious to moisture. The Author's experiments show that the conditions of manufacture greatly affect the degree of perfection attained in the glazing. The surface of the pipe, especially in larger sizes, contains air-holes formed during the drying, and steam-holes formed during the burning processes. In the larger sizes also the surface generally contains cinders, and these being incapable of taking a glaze, allow water to enter the body of the pipe by soaking round them. The air- and steam-holes are never completely glazed, the fumes of salt apparently being unable to pass through the small passages in the material. The result of these imperfections is that the pipe may act more or less as a sieve in the presence of water under pressure.

Sewer pipes generally show signs of sweating during the first few months after being laid, due to water passing through the unglazed pores mentioned. It is not possible to obtain any quantitative results of this sweating in a laboratory experiment, but the Author has been able to ascertain, by internal inspection of many hundreds of yards of 21-inch, 24-inch, and 27-inch diameter pipe-sewers, that the pipes which show the greatest amount of sweating are those which contain imperfections in the glaze. Imperfections being more numerous in pipes of large diameter, the amount of sweating in these is proportionately greater. It is also dependent on the height of the permanent water-level above the pipe.

The following experiments were made upon pipes to test their liability to sweating:—

- (a) The absorption of water by whole pipes.
- (b) Their behaviour under internal water-pressure.
- (c) Absorption of water by the pipe material.

The object of (a) was to find the effect of large flaws in the glaze, and (b) the smaller and less easily noticeable defects.

To determine the absorption of the whole pipes, they were cleaned and dried until there was no difference between successive weighings. The pipes were then submerged in water for 48 hours, at the end of which period they were withdrawn and re-weighed when the surplus water had run off.

Average values are shown in Table VIII in the Appendix. The pipes marked (a) were from Carboniferous clays from the Ashby district, possessing a very thick glaze; (b) were from the Rowley Regis district, and were of the same make as those which had low breaking-loads in the external and internal pressure tests, i.e., those marked * in Tables IV and VII; (c) were from the Ashby district, but had a thinner glazing than (a).

As the pipes were being loaded with an internal water-load, flaws in the glaze and fire cracks were carefully watched. Pipes having a cindery surface, and also those containing air-holes, imperfectly glazed, allowed water to percolate at pressures ranging from 5–20 lbs. per square inch, or under a head of water of from 11·5 to 46·0 feet. In many cases the water did not merely ooze out, but issued as a jet. With one class of pipe under test in the trench 50 per cent. of the pipes leaked in this manner through fire-cracks under a 4-foot hydrostatic head. With the better-class pipes (a) and (c), however, no leakage was noticeable up to the point of failure under internal hydrostatic load.

There still exists, however, the possibility of some minute flaw in the glaze having escaped notice; to prevent leakage from such flaws, therefore, the material of the pipe itself should be impervious. Pieces of the pipes were dried and weighed, and immersed in water for 48 hours, and at the end of this time re-weighed, and the percentage absorption obtained. The Carboniferous rock clays of the Moira and Alfreton districts produce a stoneware having a much smaller absorption than the more open clays of Swadlincote and Rowley Regis. Average values are tabulated below:—

Locality.	Absorption.
	Per Cent. of Weight of Pipe.
Moira	0·71
Alfreton	0·40
Swadlincote	1·50
Rowley Regis	3·00

Experience indicates that, fire-cracks being absent, the amount of leakage through a pipe of material whose porosity is less than 1 per cent. is negligible.

During the progress of the works several 27-inch and 24-inch diameter pipes, on being cut, were found to be of a blue-black colour in the central portion of their thickness; sometimes this

colour extended throughout the material, and whenever it occurred it was always present in the thickest part of the pipe, viz., the collar. Such pipes also contained a greater percentage of cinders and pin-holes than the characteristically cream-coloured variety. Unglazed pieces of pipes of about 200 grammes each, of both kinds, were dried, weighed, and then soaked in water for 24 hours, and the absorption obtained; the average percentage absorption for many pieces was as follows:—

Cream-coloured specimens	2·08 per cent.
Blue-coloured specimens	2·28 „

The results show that cindery pipes allow water to pass through the metal, and that pipes of blue material have a greater absorption than the cream-coloured ones. In general, the rejection of cindery pipes also usually ensures badly burnt material being rejected. If, therefore, the engineer insists on pipes badly glazed or fire-cracked being rejected, the pipe itself may be eliminated altogether from the conditions affecting the watertightness of the pipe-line, which then entirely depends on the efficiency of the joints.

The Author, in conjunction with the pipemakers, very carefully investigated the possible causes of the blue colour and of the cindery nature of the surface, their conclusion being that it is due to the kilns being “drawn” before the metal has had time to become completely vitrified. A loaf which has been too quickly baked for the interior to be properly cooked offers a perfect analogy to the badly burnt pipe.

This is, however, only one cause of the blue colouration, for it has been found to occur in pipes which are known to have been burnt for the correct period. This is accounted for as follows:—The fires have been negligently allowed to burn low during the time the temperature of the kiln should be a maximum: the fireman heaps on coal, the draught carries showers of coal and cinders into the kiln, and these become embedded in the pipe: owing to the sudden reduction in temperature, due to the damping of the fires, thorough vitrification does not take place, and the pipe-metal has a blue colour, as explained before.

As pipes are expected to be in use for at least the period of loan, viz., 30 years, the glaze must during that period be unaffected by the flowing sewage and by the free acids and alkaline salts which it contains. To determine the durability of the glaze under the action of sand, gravel and grit, samples of pipe-metal completely glazed were subject to attrition in an attrition-tester. The rubbing media consisted of 4 lbs. of Leighton Buzzard sand and 3½ lbs. of shot,

mixed with an equal volume of water. The metal was subjected to the rubbing action for 500,000 revolutions of the machine at a speed of 30 revolutions per minute. The effect on the absorption at the end of this period was determined by the method previously described. In no case was there found to be any absorption, the metal being quite impervious before and after being subject to attrition. In the old sewers at Solihull, which have been in continuous operation for a period of 32 years, the glaze is unaffected, even where the velocity of the sewage is as high, at times, as 8 feet per second. It may be safely assumed, as far as pipe-sewers are concerned, that there is no necessity to fix an upper limit to the allowable velocity of the sewage.

Concentrated acids do not have any effect upon the glaze of the pipe, even on continued boiling. Owing to the possibility of the pipe being in places unevenly coated, unglazed stoneware was also boiled with concentrated acids. Specimens of the pipe were ground to a powder and boiled for 6 hours in a concentrated solution of hydrochloric and nitric acids, and the loss in weight obtained. The average loss for the Ashby variety of pipe was 0·13 per cent., and for the Rowley Regis variety 0·52 per cent. With a 1 per cent. solution of acids there was no effect on the pipe. As far as the Author is aware, this question of deterioration of the pipe, owing to the presence of free acids, has not arisen in actual practice, so that it is safe to assume that in urban districts at least no deterioration from this cause will occur with salt-glazed Carboniferous clay pipes.

THE INTEGRITY OF THE JOINTS.

The only remaining condition affecting the watertightness of the sewer which had yet to be considered was that of the efficiency of the joints. This depends upon:—

- (1) The condition of the trench during period of setting.
- (2) The dimensions of the joint.
- (3) The jointing material.

Under the pressure test (6 feet head), to which every part of the line at Solihull is submitted before the trenches are filled in, joints made of composition (1 of cement to 1 of sand) were not as watertight as those made of neat cement. The leakages in wet ground were on the average 50 per cent. greater when the joints were tested at the end of 12 hours than at the end of 24 hours. In dry ground it was exceptional to find any leakages in a test of 15 minutes' duration on a line 150 feet long. The percentage of leaks

is also greater where yarn is not used than where the pipe is held in position by the yarn. This material prevents the cement in the bottom of the joint from being squeezed out and an open joint being formed on top.

It is essential that the trench be kept dry until the cement has attained its initial set. The cement should also be cool, as at times trouble with the testing has occurred owing to the cement having expanded. As mentioned earlier, the pipes are all tested in lengths of about 150 feet to a pressure corresponding to a head of water 6-9 feet. (2.6-3.9 lbs. per square inch), with an air-pressure machine. Where, however, water has been available, the pipes have been tested by its means.

The efficiency of the joint also depends on its shape. Experiments were made on joints of the following dimensions:— $\frac{1}{2}$ inch by $\frac{1}{2}$ inch, $2\frac{1}{2}$ inches by $\frac{1}{2}$ inch, 3 inches by $\frac{1}{2}$ inch, $3\frac{1}{2}$ inches by $\frac{3}{4}$ inch, and 4 inches by 1 inch, the larger dimensions being the depth and the smaller the width of the joint. The joints were at the end of 24 hours put under air pressure. The deep and narrow joints leaked with pressures greater than 6 lbs. per square inch (14 feet head), but those in which the width was proportionately increased were air-tight up to 10 lbs. per square inch (23 feet head), and those of depth less than $2\frac{1}{2}$ inches always leaked under pressures greater than 3 lbs. per square inch.

Those brands of pipes whose spigot ends were only thinly glazed on the outside always made tighter joints than those well glazed, a tighter grip of the cement being obtained with the former. The deep and narrow joints failed owing to the inability of the workman to force the cement well down into the socket, the result being that the cement was porous and full of air-holes, instead of being homogeneous.

The conclusions with regard to the shape of the sockets to withstand a pressure of 20 feet head are indicated in Table IX in the Appendix. In the Author's opinion, care should be taken to ensure that the cement is well rammed down into the socket instead of depending on a fillet to prevent leakage, for the cement in the fillet setting quicker than that in the interior of the joint may cause a crack to form directly through the joint. The width of the joint should not be less than $\frac{1}{2}$ inch, and the interior of the socket and the exterior of the spigot of the pipe should be corrugated, and, as far as possible, unglazed, to allow of a better grip for the cement being obtained.

Solubility of Cement.—As the watertightness of a sewer depends to so great an extent upon the cement, the Author, in investigating

the solubility of cement, subjected fresh dry cement to the action of distilled water, and also to water from the Keuper marls of the district, containing calcium and magnesium chlorides in solution. Five grammes of cement were placed in 200 cubic centimetres of distilled water; part of the cement had dissolved and part had swelled out to about ten times its original bulk at the end of a month. The effect of the water containing salts in solution was similar to that of the distilled water, only in a degree intensified. The experiments seem to show that even when setting is complete, the cement is not impervious to the action of water. This action appears to be that of dissociation of the elements of the cement into calcium oxide, aluminium hydroxide, and silicic acid.

Regarding the nature of the calcic portion of the cement, upon which its setting properties depend, all minerals containing calcium are to a large extent attacked by even weak acids, the solvent action being, generally speaking, directly proportional to the amount of calcium present. This element also has an affinity for the acid molecule in a neutral salt, which may be in solution in the water passing over the calcium mineral. Waters containing NaCl, MgCl, MgSO₄, etc., in the solution have therefore a greater solvent action on cement than has distilled water. The experiment mentioned above is thus explained.

It has been particularly noticed in sewers of diameters greater than 21 inches, i.e., sewers into which it is possible to enter for inspection, that where the ground water causes a head to be upon the joints an efflorescence occurs around the inside of the joint. After a period varying from a few days to 3 weeks, the amount of leakage, generally speaking, diminishes, the efflorescence having apparently closed up the pores through which the water containing the dissolved cement had passed. This effect is, however, not invariably produced, as occasionally the leakage increases.

Little is at present known in regard to the action of water on cement, although experiments are being made at the Agricultural Colleges at Bozeman, Montana, and at Fort Collins, Colorado. The results obtained up to the present indicate that this action should be borne in mind when framing specifications regarding the permissible leakage in a line of pipes jointed with cement.

CONDITIONS AFFECTING THE FLOW OF SEWAGE IN PIPES.

For economical working, sewage must possess a self-cleansing flow. To design a pipe-line so that it will under all conditions of flow remain clean, requires a knowledge of the effect of velocity and

depth of flow upon the transporting power of the stream. This knowledge can only be obtained from experiments in actual sewers and from observations on the working of sewers which have been in operation for a considerable time, for in such sewers the material to be transported, the irregularities of the bed, and the variations in the velocity are those peculiar to a sewer.

The matter which sewers in urban districts may be called upon to carry falls under the following heads :—

- (1) Road detritus.
- (2) Garbage.
- (3) Excreta.
- (4) Paper and textiles.

And the manner in which it may be carried is: (1) By dragging; (2) by intermittent suspension; (3) by continuous suspension.

As far as the Author is aware, previous experiments do not offer any information regarding the transport of material in suspension in flowing sewage, while those of Dubuat and others¹ on "dragging" under ideal conditions are of little value as far as actual practice is concerned. There are many experiments on transport in channels recorded, but none in actual sewers.

As a condition precedent to the obtaining of a loan for sewerage works from the Local Government Board, sewers must be designed to carry six times the dry weather flow of sewage from the drainage area; at ordinary times, therefore, the depth of flow is only very small. In designing the sewer, the necessary gradients and diameters are obtained on the assumption that the sewer is to flow half-full or full, but as this condition may not occur until many years after the sewers come into operation, and even then only during or after an occasional storm, it is very necessary to know if the sewer will be self-cleansing with small flows, such as will occur during the first few years of operation.

The fact that the depth of flow, with a constant volume of discharge, is dependent upon the gradient and diameter of the sewer, and the uncertainty as to whether depth of flow or velocity had the greater effect upon the power of suspension of the stream, led the Author to endeavour to determine the limiting grades for efficient working of sewers of diameters used in practice.

Measurement of Velocity.—For the measurement of velocity Pitot tubes were used. The meter consisted of velocity and pressure

¹ See Baldwin Latham, "Sanitary Engineering," p. 80. 2nd Edition, London, 1878.

tubes ($\frac{1}{16}$ inch diameter barometer tubing), fixed at right angles. The nozzles were drawn to $\frac{1}{32}$ inch diameter, and the ends slightly cup-shaped. The upper ends of the tubes were connected by indiarubber pipes to the gauge-tubes, which were connected by a U tube and cock, so that, by altering the pressure above the water column, the upper levels of the water in the gauge could be brought to any required position. Measurements were made at manholes, the gauge-tubes being at the surface. The velocity and pressure-tubes were mounted on a vertical rod, resting on a bearing which allowed the instrument to set itself with the velocity-tube parallel to the current, and the pressure-tube at right angles to it. The meter was rated previous to use, the constant being approximately unity. The apparatus was home-made, and performed its duties well.

Motion by Dragging.—The following velocities of sewage along the invert of the pipe were sufficient to commence motion by dragging:—

Material.	Velocity. Feet per Second.
Fine mud	0.50
River sand	0.75
Road grit	0.80
Gravel, $\frac{1}{4}$ -inch (if disturbed)	0.60
„ „ (in mass)	1.00
„ „ to $\frac{1}{2}$ -inch	1.25
„ 1-inch	1.80
„ $1\frac{1}{4}$ -inch	2.25
„ 2-inch	3.30
Broken stone, $2\frac{1}{2}$ -inch	3.80

The material always moved with a velocity less than that of the stream, the mean velocity of the material being roughly 0.60 of that of the stream. The curve in *Fig. 7* shows the relation between the velocity of the particle and its weight in a stream of mean velocity 3 feet per second.

The depth of flow is a very important factor determining the motion by dragging. The broken stone referred to in the Table, though only just moved with a velocity of water 3.80 feet per second in a stream 1 inch deep, was easily moved in a stream 6 inches deep and a mean velocity of 2.00 feet per second.

By considering the action of water on a particle adhering to the invert of a sewer, this variation of transporting power with depth of flow may be explained as follows:—The water wave, on striking the particle, is deflected from its rounded surface; the resistance to deflection does greater work on the particle if the stream be deep than if it be shallow. For in the deep stream not only is the water

striking the particle under greater pressure due to increase of head, but its translational energy is greater; although the bottom velocity is not dependent on the depth of flow to as great an extent as is the surface velocity.

Examination of, and experiments on, the Solihull sewers show that for sewers to carry heavy road detritus (this will only occur during storm time), a mean velocity of at least 2·5 feet per second is required, and a depth of flow of at least 4 inches. It must be borne in mind that although road detritus only finds its way into the sewers during storms, it may not be discharged to the detritus chambers at the disposal-works while the stream is still swollen, especially if the works be far from the point of entry of the road detritus. There is, therefore, a danger of stoppage occurring if the

Fig. 7.

energy of the flowing stream is not sufficient to carry the material away during times of small flow.

Movement in Intermittent and Continuous Suspension.—The experiments indicate that when fine material is placed in a sewer the motion is first one of dragging, and after a period, more or less short, the particles are lifted up into the stream, the motion then being one of suspension. More material is carried in the lower layers of the stream than in the upper, the amount being proportional to the depth. The motion appears always to be from the sides to the centre, and the greatest amount of suspended matter is always carried along the central thread of the stream. The amount and size of matter carried in suspension depends upon the velocity of the stream and upon its depth of flow, though the stream of greatest velocity is not necessarily that which possesses the greatest power of suspension. Increase of depth of flow, velocity being constant,

has the effect of allowing a film to form on the glaze; the diminution in velocity due to this tends to throw material out of suspension. The power of suspension is dependent rather on the depth of flow than on the velocity, for particles are thrown to a greater height in a deep stream than in a shallow one, and as the velocity of descent of the particles is the same in both cases, the material will be carried farther in the former case, and will thus have a greater opportunity of remaining suspended.

The condition to ensure material being taken into suspension appears to be that the bottom velocity be sufficiently great; for the material to move in continuous suspension the condition is that the depth of flow be sufficiently great. The intermittent suspension movement is by increase in the density of the liquid converted more and more into that of continuous suspension, as by increasing the density of the liquid the time of descent of suspended matter is also increased.

The velocity of the suspended matter is greater than that of the stream which bears it, the velocity being approximately 1.3 times that of the mean velocity of the stream. The difference between the velocities is due to the resistance to which the filaments surrounding the suspended body are subject being different from that to which the body itself is subject. A portion of the force due to gravity, in the case of water, is used in overcoming internal friction of the filaments due to their non-parallelism. There is no such loss of energy internally in the particle, and in this case the only work which gravity has to do is to overcome the friction against the filaments surrounding the body and to affect acceleration.

Diameter of Sewer.	Velocity to Start Suspension.	Depth to Start Suspension.	Depth for Continuous Suspension.	Width of Flow.
1 Inches.	Foot per Second.	Inches.	Inches.	
9	1.50	2	3	8
12	1.20	1 $\frac{3}{8}$	2 $\frac{1}{2}$	9
15	1.00	1 $\frac{1}{2}$	1 $\frac{3}{4}$	9 $\frac{1}{2}$
18	0.98	1 $\frac{1}{8}$	1 $\frac{1}{8}$	8 $\frac{1}{2}$

The above values are averages only. For pipes of diameter greater than 18 inches the depths to ensure suspension are very small. The material carried was of size up to $\frac{1}{8}$ inch diameter in continuous suspension, and $\frac{1}{4}$ inch in intermittent suspension. Where the depth for the minimum dry weather flow of sewage is greater than the minimum depths shown in the Table, the sewer

will, as regards fine material, be self-cleansing. The Author finds that for sewers of diameter greater than 15 inches these depths of flow are not sufficient to enable large matter such as paper, garbage, etc., to be taken away without risk of stoppage. Also that a depth of flow of at least 4 inches is required for the effectual removal of road detritus, and he would recommend that a depth of flow for 9-inch to 15-inch pipes of not less than $2\frac{1}{2}$ inches, and for 18-inch to 27-inch diameter pipes $4\frac{1}{2}$ inches be obtained by suitably selecting the grade for the condition of minimum flow during dry weather. Nine-inch pipe-sewers are generally used as lateral sewers, and these, generally speaking, have only very small flows, which are frequently only intermittent, so that such sewers are rarely self-cleansing. With the larger sizes the Author finds that to obviate flushing the following gradients are the flattest at which sewers should be laid:—

12-inch diameter	1 in 200, or 0·50 per cent.
15-inch " 	1 in 300, or 0·33 "
18-inch " 	1 in 400, or 0·25 "
21-inch " 	1 in 500, or 0·20 "
24-inch " 	1 in 600, or 0·16 "
27-inch " 	1 in 750, or 0·13 "

The discharge from houses being intermittent, it rarely happens that 9-inch lateral sewers possess a depth of flow sufficient to carry all solid matter away. The effect of this irregularity of flow is, in some cases, to cause deposits, and always to permit a filmy growth of mycelial matter on the pipe. This growth causes a considerable diminution of the velocity of the sewage, and the flushing experiments, to which attention will shortly be directed, show that sewers which have been cleaned from this film have the velocities of flow in them increased from 5 per cent. to 15 per cent. Sewers which possess only intermittent flow, therefore, need to be aided in their work by periodically flushing them to restore their efficiency. There is little actual data bearing upon sewer flushing available, but the Author is indebted to Mr. H. N. Ogden for very many valuable suggestions contained in his book, "Sewer Design."

The experiments about to be described were made at Solihull to determine at what diameter and gradient sewers require assisting in their duty, and the relation between volume and period of flush, and between the gradient and diameter of sewer.

Sewers other than 9-inch laid in a district not fully developed require artificially cleansing during the first years of their working, assuming that they were designed to be self-cleansing when the

drainage area is developed. In a 9-inch lateral sewer the different house-connections discharging their sewage cause the volume and velocity of flow to be increased from the point of entry of any special connection to the junction with the main sewer. The farther a house-connection is from the dead end, therefore, the less the liability of the 9-inch sewer to become blocked. Artificial means of increasing the flow in the upper portions of the laterals is needed to remove deposits to such position that they will be carried forward by the main current.

Gentle flows are favourable to the generation of sewer-gas, owing to the decomposition of the slime on the sides of the pipe. Periodical flushing at a sufficiently small interval of time, generally 48 hours, prevents the formation of the slime. When a reduction of gradient takes place, deposit of mineral particles and the heavier organic matter may, and frequently does, occur at a distance from the point of change varying between 15 feet and 35 feet.

The simplest method of keeping 9-inch sewers cleansed would be to have all laterals flowing continuously with a self-cleansing flow artificially obtained. Against this, however, must be placed the excessive cost of water and the increased volume which would have to be dealt with at the disposal works, and also the extra cost of enlarging the mains to take the increased flow. The methods in practice are :—

- (1) Periodic hand-flushing by damming up the sewage.
- (2) The use of automatic flushing-tanks.

The former is objectionable, especially in sewers possessing discontinuous flow, as owing to the stoppage of flow a deposit may occur, which is not always removed when the valve is released. A film of decomposable matter also is deposited on the sides of the sewer, above the ordinary level of the sewage. The decomposition of this matter, as has already been pointed out, produces sewer-gas. The method in itself also is unreliable, as the flushing must depend on the whim of the workman. It is, however, the only method applicable to the flushing of main sewers; in them, however, the film may not remain very long, as the flow may be sufficient to fill the sewer when a storm occurs. The automatic siphon and tank method is the one most applicable to the flushing of 9-inch and 12-inch sewers, the apparatus being situated, whenever possible, at the heads of all laterals.

Flushing Experiments.—A manhole was used as the measuring-tank from which a known volume could be discharged into the sewer; the capacity of the manhole per foot of depth was obtained,

so that by filling it to the required depth any desired volume of water could be discharged. Those manholes which contained a flap, or other rapidly moving valve, only required the closing of the incoming sewer to fit them for the experiments. Those which did not contain a valve were fitted on the outgoing end with an asbestos-faced cover strutted against the brickwork. The cover and strut

Figs. 8.

were connected together, and by cords were easily lifted to the surface when it was desired to discharge the water.

The depths of the flushing wave were measured in the downstream manholes every 30 seconds after the valve was opened. The condition of the sewer before and after flushing, and the distance to which heavy material was carried by different volumes of water, and also the effect of the flush on the mycelial growth were noted. It was found that all filmy matter was removed from

a 9-inch pipe when the depth of flushing wave was greater than 3 inches. On the same sewer all heavy material in the invert of the manholes was carried through about 600 feet when the volume of the flush exceeded 300 gallons and the gradient was steeper than 1 in 350, or 0.286 per cent., and a 400-gallon flush on 9-inch sewer of gradient less than 1 in 100 did not have any cleansing effect at a distance from the flush-tank greater than 600 feet. Where there was an original flow of sewage in the sewer this acted as a serious impediment to the flushing wave, and the cleansing power was thereby much reduced.

The types of curves obtained are shown in *Figs. 8*. In the particular case illustrated the 100-gallon flush was almost useless. The depth of flushing wave decreases from the flush-tank end to a point down-stream, at which the depth and velocity are little affected by the discharge. Nine- and 12-inch sewers having a

Fig. 9.

continuous flow of a depth greater than one-quarter of the diameter and a mean velocity of 2.5 feet per second, or greater, require flushing at infrequent intervals. If, for any reason, these conditions be not fulfilled, the engineer-in-charge must resort to artificial cleansing of the sewers to prevent stoppage.

The secret of effective flushing appears to be that a large volume of water at high velocity be discharged into the sewer; the volume, as far as possible, should be sufficiently great to completely fill the sewer, so that the effect may be felt for a considerable distance.

The period between successive discharges from the tank for 9-inch sewers should not exceed 48 hours, and should, if possible, be 24 hours; in 12-inch, 15-inch and 18-inch sewers having continuous flows, the period may vary from 1 week to 3 weeks, and on main sewers of larger diameters than 18 inches occasional flushing only is required.

Nine-inch lateral sewers, laid at gradients flatter than 1 in 100 (1 per cent.), require automatic flushing. The volume of water required is a variable quantity depending on the gradient, but in general should not be less than 400 gallons. A discharge of this volume is then generally sufficient to cleanse a length of 600 feet of sewer. Should the main sewer be at a greater distance than this from the flush-tank, an additional tank 600 feet lower down than the previous one, or periodic hand-flushing below this length, becomes necessary to ensure that the heavy material is discharged into the main sewer. On gradients steeper than 1 in 100 a periodic flush of 200 gallons gives as good results as one of 400 gallons. The effect of gradient on the depth of flushing-wave is shown in *Fig. 9*.

SUMMARY OF CONCLUSIONS.

In what follows it is assumed that the sewer is to be laid in good ground, as in bad ground the consideration of saving in cost must not on any account be allowed to have precedence over the safety of the pipe-line.

To enable pipes to be safe in ground which may vary from sand to clay, the engineer may demand that pipes of thickness $d/12$ be capable of bearing the uniform loads shown in Table I, previous to failure. As the uniform load test is not an easy one to carry out commercially, the concentrated load test may be substituted for it; the loads in this test corresponding to those of the uniform loads are shown in Table III. To ensure pipes capable of withstanding these standard loads only being used, those badly burned or fire-cracked must be rejected. To ensure that the necessary factor of safety is obtained the engineer must insist on the correct refilling of the trench to prevent undue stress being brought upon any particular pipe or pipes. By the use of the tables of factors of safety, it may be determined when the pipe requires a reinforcement and the necessary provisions made in the estimate.

The pipe material should possess a tensile strength of 600 lbs. per square inch with an internal hydrostatic load of 100 lbs. per square inch, and with heads up to 50 feet the pipes should be free from sweating. The percentage absorption of the pipe-material should not be greater than 2 per cent. after 48 hours' immersion; and while that of the whole pipes may be very variable, according as the spigot and socket ends are thinly or thickly glazed, it should not exceed 2 per cent.

If the joints are made as recommended, and the interior of the socket and the exterior of the spigot are as far as possible unglazed

the sewer will in general be watertight. Leakages, however, may occur, no matter how carefully the joints have been made, and to cure which may at times be very difficult. It would, therefore, seem advisable to allow a leakage not exceeding 200 cubic feet per mile per 24 hours as an average for all sizes. The volume allowed as leakage should be much less in good ground and in sewers of small diameter, in which the jointer can work all round the joint. Where the ground is waterlogged, a patent-jointed pipe should be used to withstand the head of water, and also to allow the cement to attain its initial set without there being any liability of its being washed out by the water.

The irregularities in the interior of the pipe-line, due to the abutting joints and variations in the cylindricity of the pipes, greatly affect the uniform flow of the sewage. To minimize the variations as much as possible, blisters thicker than $\frac{1}{8}$ of the thickness of the pipe and greater in diameter than $\frac{1}{10}$ of the internal diameter, variations from the straight greater than $\frac{1}{16}$ inch per foot, and a difference between any two diameters at right angles greater than $\frac{1}{32}$ the nominal diameter, should cause the rejection of the pipe. No pipes of less thickness than the standard should be allowed on the works. To ensure the heavy matter being carried by dragging, and the lighter matter in suspension, a mean velocity of 2.5 feet per second and a depth of flow of at least 3 inches appear to be required. Where such conditions are not obtainable, owing to the undeveloped nature of the drainage-area, or the natural configuration of the ground, artificial means of cleansing the sewer become necessary. A volume of 400 gallons of water should be discharged every 24 hours into such sewers when they are of diameter less than 12 inches; this volume is then sufficient to cleanse a length of 600 feet. For the larger sewers damming up the sewage by penstocks and the use of the sewage itself as a flushing medium become necessary.

In conclusion, the subjects of pipe-strength, earth-loads and pipe-flow offer wide scope for further original work, and the Author believes that the experiments, while not being in any way complete, may possibly be of use to others working in the same direction.

The Paper is accompanied by ten drawings, from which the Figures in the text have been prepared.

APPENDIX.

TABLE I.—UNIFORM LOADS (AVERAGE VALUES).

Diameter, inches .	9	12	15	18	21	24	27
Thickness, inches .	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$
Load, lbs. per square foot }	3,500	2,900	2,760	2,400	2,200	2,100	1,900
Load, lbs. per lineal foot }	3,150	3,540	4,000	4,200	4,400	4,900	5,060
Total load at collapse, lbs. . . }	9,000	10,500	..	..	..	..	..
Total load at first crack, lbs. . . }	7,800	8,700	10,000	12,600	13,200	14,700	15,200

TABLE II.—UNIFORM LOADS ON PIPES HAUNCHED WITH CONCRETE (AVERAGE VALUES).

Diameter, inches	9	12	15	18
Thickness, inches	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$
Load, lbs. per square foot . .	4,230	3,680	3,360	2,970
Load, lbs. per lineal foot . .	3,700	4,300	4,900	5,200
Total load at collapse, lbs. .	12,000	..	..	..
Total load at first crack, lbs. .	9,250	10,700	12,250	15,600

TABLE III.—CONCENTRATED LOADS (AVERAGE VALUES).

Diameter, inches .	9	12	15	18	21	24	27
Length, feet . .	2.5	2.5	2.5	3.0	3.0	3.0	3.0
Thickness, inches .	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$
Total load, lbs. .	4,020	4,090	3,640	4,600	6,000	8,000	10,800
Load per lineal foot, lbs. . . . }	1 610	1,640	1,455	1,533	2,000	2,666	3,611

TABLE IV.—CONCENTRATED LOADS.

Diameter.	Thickness.	Load.	Load per Lineal Foot.	Remarks.
Inches.	Inches.	Lbs.	Lbs.	
27·50	2·50	10,000	3,333	Material cream coloured.
27·50	2·50	12,000	4,000	Failed along firecrack.
28·00	2·50	10,000	3,333	Material cream coloured.
27·00	2·50	12,000	4,000	„ blue coloured.
27·50	2·50	10,000	3,333	„ cream coloured.
27·00	2·25	11,000	3,666	„ „ „
24·50	2·10	8,000	2,666	„ „ „
24·00	2·00	8,620	2,870	Failed along firecrack.
24·50	2·25	9,400	3,133	Material cream coloured.
24·25	2·25	11,000	3,666	„ „ „
24·00	2·00	7,800	2,600	„ blue coloured.
24·50	2·00	7,000	2,333	„ „ „
21·40	1·75	6,000	2,000	„ cream coloured.
21·25	1·75	5,600	1,866	„ blue coloured.
21·00	1·90	6,300	2,100	„ cream coloured.
21·40	1·90	8,000	2,666	„ „ „
21·00	1·75	6,000	2,000	„ „ „
21·25	1·75	5,600	1,866	„ blue coloured.
18·00	1·50	4,000	1,333	„ cream coloured.
18·25	1·40	4,200	1,400	„ „ „
18·00	1·60	4,800	1,600	„ „ „
18·00	1·50	4,000	1,333	Failed along firecrack.
18·25	1·60	5,000	1,666	Material cream coloured.
18·50	1·60	5,600	1,866	„ „ „
15·00	1·25	3,750	1,500	„ „ „
15·00	1·25	3,700	1,480	„ „ „
15·10	1·40	3,000	1,200	„ „ „
15·50	1·40	4,500	1,800	„ „ „
14·90	1·25	3,250	1,300	„ „ „
15·00	1·25	3,630	1,450	„ „ „
12·25	1·10	4,750	1,900	„ „ „
11·75	1·00	3,650	1,460*	„ grey-black coloured.
12·00	1·00	3,500	1,420	Cindery material.
12·00	0·90	4,000	1,600	Material cream coloured.
12·00	1·00	4,500	1,800	„ „ „
12·25	1·00	4,120	1,650	„ „ „
9·10	0·75	4,000	1,600	„ „ „
9·00	0·75	3,600	1,440*	„ grey-black coloured.
9·10	0·90	4,500	1,800	„ cream coloured.
9·00	0·75	4,000	1,600	„ „ „
8·90	0·60	3,800	1,420	Failed along firecrack.
9·00	0·90	4,200	1,680	Material cream coloured.

TABLE VA.—TRENCH EXPERIMENTS. RESULTS IN GRAVEL, 120 LBS. PER CUBIC FOOT.

	0	1	2	3	3.5	4	5	6	7	8	9	10	11
Depth in feet	0	0.344	0.381	0.427	0.576	0.628	0.690	0.762	0.845	0.936	1.037	1.147	1.268
Scale 1, feet	0.322	0	0.022	0.046	0.049	0.052	0.062	0.072	0.083	0.091	0.101	0.110	0.121
Deflection, feet	0	0.345	0.382	0.426	0.573	0.626	0.688	0.762	0.843	0.933	1.035	1.145	1.265
Scale 2, feet	0.325	0	0.024	0.047	0.053	0.056	0.066	0.074	0.081	0.090	0.100	0.112	0.120
Deflection, feet	0	0.020	0.037	0.044	0.047	0.048	0.053	0.062	0.073	0.082	0.090	0.100	0.110
Mean deflection	0	0.021	0.037	0.045	0.048	0.053	0.062	0.073	0.082	0.091	0.100	0.111	0.120
Load transmitted in lbs.	0	1,080	1,800	2,220	2,360	2,590	3,040	3,520	3,970	4,370	4,750	5,200	5,650
Percentage of total load, lbs.	..	80	67	55	50	48	45	43.5	42	40.5	39	38.5	38

TABLE VI.—TRENCH EXPERIMENTS. RESULTS IN LOAMY SAND, 130 LBS. PER CUBIC FOOT.

	0	1	2	3	3'-5	4	5	6	7	8	9	10	11
Depth in feet	0	0.817	0.833	0.847	0.845	0.849	0.852	0.853	0.856	0.858	0.859	0.862	0.865
Scale 1, feet	0.792	0	0.015	0.055	0.053	0.057	0.060	0.061	0.064	0.066	0.067	0.070	0.073
Deflection, feet	0	0.025	0.041	0.043	0.047	0.052	0.055	0.055	0.055	0.058	0.059	0.061	0.063
Scale 2, feet	0.795	0	0.834	0.843	0.847	0.852	0.852	0.855	0.855	0.858	0.859	0.861	0.863
Deflection, feet	0	0.023	0.039	0.048	0.052	0.057	0.057	0.060	0.060	0.063	0.064	0.067	0.073
Mean deflection, feet	0	0.024	0.040	0.052	0.054	0.057	0.059	0.061	0.062	0.065	0.067	0.070	0.073
Load transmitted in lbs.	0	1,230	1,980	2,580	2,690	2,800	2,910	2,980	3,020	3,140	3,190	3,470	3,600
Percentage of total load, lbs.	..	84	68	59	56	48	42	37	35	33	30	29	28

TABLE Vc.—TRENCH EXPERIMENTS. RESULTS IN CLAY, 125 LBS. PER CUBIC FOOT.

	0	1	2	3	3·5	4	5	6	7	8	9	10	11
Depth in feet	..	..	..	..	..	..	..	..	..	..	..	..	..
Scale 1, feet	0·285	0·313	0·361	0·434	0·491	0·562	0·643	0·730	0·829	0·936	1·055	..	..
Deflection, feet	..	0·028	0·048	0·063	0·067	0·071	0·081	0·087	0·099	0·107	0·119	..	..
Scale 2, feet	0·203	0·227	0·279	0·342	0·410	0·485	0·566	0·657	0·754	0·863	0·982	..	..
Deflection, feet	..	0·024	0·052	0·063	0·068	0·075	0·081	0·091	0·097	0·109	0·119	..	..
Mean deflection, feet	..	0·026	0·050	0·063	0·067	0·073	0·081	0·089	0·098	0·108	0·119	..	..
Load transmitted in lbs.	..	1,350	2,450	3,040	3,220	3,500	3,890	4,280	4,680	5,120	5,600	..	..
Percentage of total load, lbs.	..	96·5	87·5	80	75	73	67	63	60	58	57	..	..

NOTE ON TABLES VIA-VIF.

In the following Tables the figures in the first column, namely, 3,500 lbs., 3,000 lbs., etc., are breaking loads per square foot of average pipes of each particular size, taken from Table I.

Net pressure lbs. per square foot equals load per square foot transmitted to a pipe in a trench filled to the particular depth in question.

If to this is added the equivalent of a 10-ton steam roller (as explained in text) resting on the trench, the figures are as in net earth pressure and steam roller. So that the figures 2,015, 1,760, etc., represent loads per square foot transmitted to pipe in a trench 1 foot, 2 feet, 3 feet, etc., on depth; this is the live load.

If the breaking loads be divided by dead and live loads at any particular depth of trench the factor of safety for each is obtained. The factors of safety therefore occupy the whole of the remaining space of Table.

TABLE VIA.—SHOWING RELATION OF STRENGTH OF PIPE TO TRENCH PRESSURE AS A FACTOR OF SAFETY.

Gravel.

Depth, feet . . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure, lbs. per square foot . .	96	160	197	239	269	310	350	388	420	460	500	550	640	730	820	910
Net earth pressure plus steam roller, lbs. per square foot	2,015	1,760	1,517	1,380	1,350	1,350	1,350	1,360	1,390	1,400	1,425	1,640	1,550	1,640	1,730	1,820
9 in. diam. Live 3,500 lbs..	1.6 37	2.0 22	2.3 18	2.5 15	2.6 13	2.6 11	2.6 10	2.6 9	2.5 8.3	2.5 7.6	2.5 7.0	2.4 6.4	2.3 5.5	2.1 4.8	2.0 4.2	1.9 3.8
12 in. diam. Live 3,000 lbs..	1.4 31	1.7 19	2.0 15	2.2 13	2.2 11	2.2 9.7	2.2 8.6	2.2 7.7	2.1 7.1	2.1 6.5	2.1 6.0	2.0 5.5	1.9 4.7	1.8 4.1	1.7 3.6	1.7 3.3
15 in. diam. Live 2,700 lbs..	1.2 28	1.5 17	1.8 14	2.0 12	2.0 10	2.0 8.7	2.0 7.7	2.0 6.9	1.9 6.4	1.9 5.8	1.9 5.4	1.8 4.9	1.7 4.2	1.6 3.7	1.5 3.3	1.5 3.0
18 in. diam. Live 2,400 lbs..	1.1 25	1.4 15	1.6 12	1.7 10	1.8 9	1.8 7.8	1.8 6.9	1.8 6.1	1.7 5.7	1.7 5.2	1.7 4.8	1.6 4.3	1.6 3.7	1.4 3.3	1.4 2.9	1.3 2.6
21 in. diam. Live 2,200 lbs..	1.0 23	1.2 14	1.4 11	1.6 9.5	1.6 8.2	1.6 7.1	1.6 6.3	1.6 5.7	1.6 5.2	1.6 4.8	1.6 4.4	1.5 4.0	1.4 3.4	1.3 3.0	1.3 2.7	1.2 2.4
24 in. diam. Live 2,100 lbs..	1.0 22	1.2 13	1.4 10	1.5 9.1	1.5 7.8	1.5 6.8	1.5 6.0	1.5 5.4	1.5 5.0	1.5 4.6	1.5 4.2	1.4 3.8	1.4 3.3	1.3 2.9	1.2 2.6	1.1 2.3
27 in. diam. Live 1,900 lbs..	0.9 20	1.1 12	1.2 9.6	1.4 8.3	1.4 7.0	1.4 6.1	1.4 5.4	1.4 4.9	1.4 4.5	1.3 4.2	1.3 3.8	1.3 3.4	1.2 3.0	1.1 2.6	1.1 2.3	1.0 2.1

TABLE VII.—SHOWING RELATION OF STRENGTH OF PIPE TO TRENCH PRESSURE.
Sandy Loam.

Depth, feet. . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure, lbs. per square foot. }	109	177	230	250	273	289	319	343	352	377	415	437	510	580	655	725
Net earth pressure plus steam roller, lbs. per square foot. . . . }	2,130	1,810	1,650	1,400	1,280	1,180	1,150	1,140	1,070	1,075	1,090	1,110	1,185	1,255	1,330	1,400
9 in. diam. Live 3,500 lbs..	1.7 31	1.9 20	2.1 15	2.5 14	2.7 13	3.0 12	3.1 11	3.1 10	3.2 9.8	3.3 9.3	3.2 8.4	3.1 8.0	3.0 6.8	2.8 6.0	2.6 5.5	2.5 4.8
12 in. diam. Live 3,000 lbs..	1.4 27	1.7 17	1.8 13	2.1 12	2.3 11	2.5 10	2.6 9.5	2.6 8.7	2.8 8.5	2.8 8.0	2.7 7.2	2.7 6.8	2.5 5.9	2.4 5.2	2.3 4.6	2.1 4.2
15 in. diam. Live 2,700 lbs..	1.3 25	1.5 15	1.6 12	1.9 11	2.1 10	2.3 9.8	2.3 8.5	2.3 7.9	2.5 7.7	2.8 7.0	2.5 6.5	2.4 6.2	2.3 5.3	2.1 4.7	2.0 4.1	1.9 3.7
18 in. diam. Live 2,400 lbs..	1.1 22	1.3 13	1.4 10	1.7 9	1.9 9	2.0 8	2.1 7.5	2.1 7.0	2.2 6.8	2.2 6.4	2.2 5.8	2.2 5.5	2.0 4.7	1.9 4.1	1.8 3.7	1.7 3.3
21 in. diam. Live 2,200 lbs..	1.0 20	1.2 12	1.3 9.6	1.6 8.8	1.7 8.0	1.8 7.6	1.9 6.9	1.9 6.4	2.0 6.2	2.0 5.8	2.0 5.3	2.0 5.0	1.9 4.3	1.7 3.8	1.6 3.4	1.6 3.0
24 in. diam. Live 2,100 lbs..	1.0 19	1.2 12	1.3 9.1	1.5 8.4	1.6 7.7	1.8 7.3	1.8 6.6	1.8 6.1	2.0 6.0	2.0 5.6	1.9 5.1	1.9 4.8	1.8 4.1	1.7 3.6	1.6 3.2	1.5 2.9
27 in. diam. Live 1,900 lbs..	0.9 17	1.0 10	1.1 8.3	1.3 7.6	1.5 7.0	1.6 6.6	1.7 6.0	1.7 5.5	1.7 5.4	1.7 5.0	1.7 4.6	1.7 4.3	1.6 3.7	1.5 3.3	1.4 2.9	1.3 2.6

TABLE VIC.—SHOWING RELATION OF STRENGTH OF PIPE TO TRENCH PRESSURE.

Clay.

Depth, feet. . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure, lbs. per square foot . .	121	219	300	367	418	472	525	580	640	712	782	854	1,070	1,210	1,350	1,425
Net earth pressure plus steam roller, lbs. per square foot	2,440	2,319	2,220	2,117	2,028	1,982	1,965	1,970	2,005	2,077	2,147	2,219	2,435	2,575	2,715	2,790
9 in. diam. 3,500 lbs. . .	Live 1.4 Dead 29	1.5 16	1.6 11	1.6 9.5	1.7 8.3	1.8 7.4	1.8 6.7	1.8 6.0	1.7 5.5	1.7 4.9	1.6 4.5	1.6 4.1	1.4 3.3	1.3 2.8	1.3 2.6	1.2 2.4
12 in. diam. 3,000 lbs. . .	Live 1.2 Dead 25	1.3 14	1.3 10	1.4 8.2	1.5 7.1	1.5 6.4	1.5 5.7	1.5 5.2	1.5 4.7	1.4 4.2	1.4 3.8	1.3 3.5	1.2 2.8	1.2 2.5	1.1 2.2	1.0 2.1
15 in. diam. 2,700 lbs. . .	Live 1.1 Dead 22	1.1 12	1.2 9	1.3 7.3	1.3 6.4	1.3 5.7	1.3 5.1	1.3 4.6	1.3 4.2	1.3 3.8	1.2 3.4	1.2 3.1	1.1 2.5	1.0 2.2	1.0 2.0	1.0 1.9
18 in. diam. 2,400 lbs. . .	Live 1.0 Dead 20	1.0 11	1.1 8	1.1 6.5	1.2 5.7	1.2 5.1	1.2 4.6	1.2 4.1	1.2 3.7	1.1 3.4	1.1 3.0	1.1 2.8	1.0 2.2	0.9 2.0	0.9 1.8	0.9 1.7
21 in. diam. 2,200 lbs. . .	Live 0.9 Dead 18	0.9 10	1.0 7.3	1.0 6.0	1.1 5.2	1.1 4.7	1.1 4.2	1.1 3.8	1.1 3.4	1.1 3.2	1.0 2.8	1.0 2.6	0.9 2.1	0.9 1.8	0.8 1.6	0.8 1.5
24 in. diam. 2,100 lbs. . .	Live 0.8 Dead 17	0.9 9.6	0.9 7	1.0 5.7	1.0 5.0	1.0 4.5	1.1 4.0	1.1 3.6	1.0 3.3	1.0 3.0	1.0 2.7	0.9 2.4	0.9 2.0	0.8 1.7	0.8 1.6	0.7 1.5
27 in. diam. 1,900 lbs. . .	Live 0.8 Dead 16	0.8 8.7	0.8 6.3	0.9 5.2	0.9 4.5	0.9 4.0	0.9 3.6	0.9 3.3	0.9 3.0	0.9 2.7	0.9 2.4	0.8 2.2	0.8 1.8	0.7 1.6	0.7 1.4	0.7 1.3

TABLE VII.—SHOWING RELATION OF STRENGTH OF PIPE, HAUNCHED WITH CONCRETE, TO TRENCH PRESSURE.

Gravel.

Depth, feet. . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure, lbs. per square foot . }	96	160	197	230	269	310	350	388	420	460	500	550	640	730	820	910
Net earth pressure plus steam roller, lbs. per square foot . . . }	2,015	1,760	1,517	1,380	1,350	1,350	1,350	1,360	1,390	1,400	1,425	1,460	1,550	1,640	1,730	1,820
9 in. diam. Live	1·9	2·4	2·7	3·0	3·1	3·1	3·1	3·1	3·0	3·0	2·9	2·9	2·7	2·5	2·4	2·3
4,200 lbs. Dead	43	26	21	18	15	13	12	11	10	9·1	8·4	7·7	6·5	5·7	5·1	4·7
12 in. diam. Live	1·7	2·1	2·4	2·7	2·7	2·7	2·7	2·7	2·6	2·6	2·6	2·5	2·4	2·2	2·1	2·0
3,700 lbs. Dead	38	23	19	16	14	12	10	9·5	8·8	8·0	7·4	6·7	5·8	5·0	4·5	4·0
15 in. diam. Live	1·6	1·9	2·2	2·4	2·5	2·5	2·5	2·5	2·4	2·4	2·4	2·3	2·2	2·1	2·0	1·8
3,400 lbs. Dead	35	21	17	15	12	11	9·7	8·7	8·1	7·4	6·8	6·2	5·3	4·6	4·1	3·7
18 in. diam. Live	1·4	1·7	2·0	2·2	2·2	2·2	2·2	2·2	2·1	2·1	2·1	2·0	1·9	1·8	1·7	1·6
3,000 lbs. Dead	31	19	15	13	11	9·7	8·6	7·7	7·1	6·5	6·0	5·4	4·7	4·1	3·7	3·3

TABLE VI.E.—SHOWING RELATION OF STRENGTH OF PIPE, HAUNCHED WITH CONCRETE, TO TRENCH PRESSURE.
Sandy Loam.

Depth, feet. . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure, lbs. } per square foot . }	109	177	230	250	273	289	319	343	352	377	415	437	510	580	655	725
Net earth pressure plus steam roller, } lbs. per square foot }	2,130	1,810	1,650	1,400	1,280	1,180	1,150	1,140	1,070	1,075	1,090	1,110	1,185	1,255	1,330	1,400
9 in. diam. Live	1.97	2.32	2.54	3.0	3.28	3.56	3.65	3.69	3.92	3.9	3.8	3.8	3.5	3.3	3.1	3.0
4,200 lbs. Dead	38	24	18	17	15	14	13	12	12	11	10	9.5	8.0	7.0	6.5	5.8
12 in. diam. Live	1.75	2.04	2.24	2.62	2.90	3.14	3.20	3.25	3.46	3.44	3.40	3.36	3.12	2.94	2.78	2.62
3,700 lbs. Dead	34	21	16	15	14	13	12	11	11	10	9	8.4	7.2	6.4	5.6	6.1
15 in. diam. Live	1.6	1.9	2.0	2.5	2.7	2.9	3.0	3.0	3.2	3.2	3.1	3.1	2.9	2.7	2.5	2.4
3,400 lbs. Dead	31	19	15	14	12	12	11	10	10	9	8.2	7.8	6.7	5.9	5.2	4.7
18 in. diam. Live	1.4	1.7	1.8	2.1	2.3	2.5	2.6	2.6	2.8	2.8	2.7	2.7	2.5	2.4	2.3	2.1
3,000 lbs. Dead	28	17	13	12	11	10	9.5	8.7	8.5	8.0	7.2	6.8	5.9	5.2	4.6	4.1

TABLE VI.—SHOWING RELATION OF STRENGTH OF PIPE, HAUNCHED WITH CONCRETE, TO TRENCH PRESSURE.
Clay.

Depth, in feet . .	1	2	3	4	5	6	7	8	9	10	11	12	14	16	18	20
Net pressure in lbs. } per square foot . }	121	219	300	367	418	472	525	580	640	712	782	854	1,070	1,210	1,350	1,425
Net earth pressure } plus steam roller } lbs. per square } foot }	2,440	2,319	2,220	2,117	2,028	1,982	1,965	1,970	2,005	2,077	2,147	2,219	2,435	2,575	2,715	2,790
9 in. diam. Live	1.7	1.8	1.9	2.9	2.1	2.1	2.1	2.1	2.1	2.0	1.9	1.9	1.7	1.6	1.5	1.5
4,200 lbs.. Dead	35	19	14	11	10	9.0	8.0	7.2	6.6	5.9	5.4	4.9	4.0	3.5	3.1	3.0
12 in. diam. Live	1.5	1.6	1.7	1.7	1.8	1.9	1.9	1.9	1.8	1.8	1.7	1.7	1.5	1.4	1.4	1.3
3,700 lbs.. Dead	30	17	12	10	8.9	7.9	7.0	6.4	5.8	5.2	4.7	4.3	3.1	3.0	2.7	2.6
15 in. diam. Live	1.4	1.4	1.5	1.6	1.7	1.7	1.7	1.7	1.7	1.6	1.6	1.5	1.4	1.3	1.2	1.2
3,400 lbs.. Dead	28	15	11	9.2	8.1	7.2	6.5	5.8	5.3	4.8	4.3	4.0	3.2	2.8	2.5	2.4
18 in. diam. Live	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.5	1.5	1.4	1.4	1.3	1.2	1.2	1.1	1.1
3,000 lbs.. Dead	25	14	10	8.1	7.1	6.4	5.7	5.2	4.7	4.2	3.8	3.5	2.8	2.5	2.2	2.1

TABLE VII.—HYDROSTATIC LOADS.

Diameter.	Thickness.	Length of Pipe.	Pressure.	Tensile Strength.
Inches.	Inches.	Feet.	Lbs. per Square Inch.	Lbs. per Square Inch.
9	$\frac{3}{4}$	$2\frac{1}{2}$	175	1,050
9	$\frac{3}{4}$	$2\frac{1}{2}$	175	1,050
9	$\frac{3}{4}$	$2\frac{1}{2}$	185	1,100
9	$\frac{3}{4}$	$2\frac{1}{2}$	175	1,050
9	$\frac{3}{4}$	$2\frac{1}{2}$	100	600*
12	$1\frac{1}{8}$	$2\frac{1}{2}$	160	855
12	$1\frac{1}{4}$	$2\frac{1}{2}$	190	1,000
12	1	$2\frac{1}{2}$	125	750
12	$1\frac{1}{8}$	$2\frac{1}{2}$	145	775
12	1	$2\frac{1}{2}$	120	720
15	$1\frac{3}{8}$	$2\frac{1}{2}$	125	685
15	$1\frac{1}{4}$	$2\frac{1}{2}$	120	725
15	$1\frac{3}{8}$	$2\frac{1}{2}$	130	714
15	$1\frac{1}{4}$	$2\frac{1}{2}$	120	725
15	$1\frac{3}{8}$	$2\frac{1}{2}$	150	800
18	$1\frac{1}{2}$	3	100	600
18	$1\frac{5}{8}$	3	110	620
18	$1\frac{1}{2}$	3	110	660
18	$1\frac{1}{2}$	3	110	660
21	$1\frac{3}{4}$	3	100	600
21	$1\frac{7}{8}$	3	105	580
21	$1\frac{3}{4}$	3	100	600
24	$2\frac{1}{8}$	3	110	621
24	$2\frac{1}{8}$	3	105	690
24	2	3	100	600

TABLE VIII.—ABSORPTION OF WATER BY WHOLE PIPES.

Kind.	Diameter.	Weight, Dry.	Weight, Wet.	Mean Absorption.	Mean Percentage Absorption.
	Inches.	Lbs.	Lbs.	Lbs.	
<i>a</i>	9	70·5	71·0	0·5	0·71
<i>b</i>	9	70·0	72·0	2·0	2·86
<i>c</i>	9	71·0	72·0	1·0	1·40
<i>a</i>	12	122·0	122·7	0·7	0·58
<i>b</i>	12	109·0	111·5	2·5	2·30
<i>c</i>	12	110·0	111·0	1·0	0·91
<i>a</i>	15	170·0	171·5	1·5	0·90
<i>b</i>	15	172·0	176·0	4·0	2·30
<i>c</i>	15	185·0	187·0	2·0	1·09
<i>a</i>	18	240·0	244·0	4·0	1·66
<i>b</i>	18	245·0	251·5	6·5	2·65
<i>c</i>	18	250·0	252·5	2·5	1·00
<i>a</i>	21	392·0	399·5	7·5	1·92
<i>a</i>	24	560·0	571·0	11·0	1·96
<i>a</i>	27	728·0	743·0	15·0	2·05

TABLE IX.—PROPOSED PROPORTIONS OF SOCKETS AND JOINTS.

Diameter of Pipe.	Diameter of Yarn.	Depth of Socket.	Width of Joint.	Depth of Cement.
Inches.	Inch.	Inches.	Inch.	Inches.
9	$\frac{1}{2}$	$3\frac{1}{2}$	$\frac{1}{2}$	3
12	$\frac{1}{2}$	$3\frac{1}{2}$	$\frac{1}{2}$	3
15	$\frac{3}{4}$	4	$\frac{9}{16}$	$3\frac{1}{2}$
18	$\frac{3}{4}$	4	$\frac{5}{8}$	$3\frac{1}{2}$
21	$\frac{7}{8}$	$4\frac{1}{2}$	$\frac{3}{4}$	$3\frac{3}{4}$
24	1	$4\frac{1}{2}$	$\frac{7}{8}$	$3\frac{3}{4}$
27	1	5	1	4