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Some structural aspects of the design of concrete pipelines†

by

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The Chairman (Sir Herbert Manzoni) said that the Paper described a piece of pioneering work. It was probable that less was known about the many factors that governed the strength of pipelines than about the majority of civil engineering projects, for very little research work had been done on the subject. The Authors, however, had extended the work that had been done at Iowa. Pipelines were designed by what might be described as Roman methods, involving crude strength and bulk rather than by scientific analysis; there had always been, and still were, so many difficulties and imponderables, such as the adequate packing and shaping of concrete round the pipes in the bottom of a narrow trench, or the determination of the characteristics of the back-filled earth. For these reasons, it was likely that the present methods would continue until much more basic information was made available.

70. On two previous occasions Sir Herbert had taken the Chair when Papers on this subject were presented, and on each occasion criticism had been made of the suggestions arising from careful investigation. Because of the practical difficulties, certain assumptions and seemingly uneconomic methods might have to be continued on the basis of experience, but it was much better if experience were added to a known basis of actual research and ascertained facts.

71. If nothing else were to result from the Authors' research, at least their insistence on the necessity for a new type of joint was most valuable. There was more trouble with the old-fashioned joints than with almost anything else, and the introduction of an agreed type of flexible joint would be a monument to their work.

72. Mr Clarke had spent, the Chairman believed, about 30 years on practical work and he was, therefore, able to bring to his research a knowledge of most of the practical difficulties, and the results of the Authors' work might well be that pipelines in the future could be laid more economically and with less risk of damage.

Sir George McNaughton (Chief Engineer, Ministry of Housing and Local Government) said that the normal requirements of the Ministry of Housing and Local Government in respect of concrete protection of sewers were well known to all engineers concerned with that work, for they appeared on the last page of the Ministry's Form K29, which had to be sent in each time loan sanction was required. Those requirements had been in existence for a very long time, but in recent years some people, of whom he was one, had come to the conclusion that they were probably open to challenge, particularly in regard to the usefulness of the 6-in. concrete surround, and especially when it was applied to pipes of large diameter. The challenge had been made with a good deal of hesitation. At that stage the Authors had supplied a considerable amount of evidence about the number of broken pipes they had managed to unearth, but at the same time the engineers at the Ministry knew, or thought that they knew, that

† Proc. Instn civ. Engrs, vol. 14, p. 67 (Sept. 1959).

in Britain there were many thousands of miles of sewers that were giving good service. Nevertheless, they had continued to collaborate with the Authors, who had kindly extended their original work to provide help on this very difficult question. At the same time, the Ministry had set up an informal working group to look into the matter. The working group consisted of a number of Ministry engineers, several eminent municipal engineers nominated by the Institution of Municipal Engineers, and the Authors.

74. The group had not held many meetings before it became apparent that the Authors' research was of far-reaching importance, and in fact might lead to radical alterations to, or even complete scrapping of, the Ministry's normal requirements for concrete protection. It had been necessary to bear in mind that revised requirements should not conflict with any amended Code of Practice that might be issued later. They had been forcibly reminded that a large amount of capital had been sunk in the manufacture of stoneware and concrete pipes. While it was very easy on paper to make suggestions and demand special requirements, care must be taken to see that anything that was done did not, at the very worst, put some pipe manufacturer out of business or, what was more likely, involve manufacturers in very large capital expenditure to meet a whim of the working group or some rather involved theory.

75. The group, therefore, interviewed representatives of the manufacturers. Sir George had expected very tough opposition from them and had been surprised to find that they themselves had been giving a great deal of thought to the problem, presumably as a result of their previous contacts with the Authors.

76. The working group had also to consider future costs, and it was desirable that their recommendations should result in economy in construction. Such economy would achieve a considerable saving throughout Britain, because the total mileage of sewer pipes, and other pipes laid annually, was very large indeed.

77. The group's task, as he saw it, was to translate the research which had been carried out by the Authors into a form of requirement that the busy municipal engineer could specify in a concise manner to meet the varying conditions of the subsoil. That might sound easy, but it was in fact very difficult, because the subsoil conditions could vary, especially in towns, every few yards, and it was not possible to let the contractor open up a few hundred yards and then start to consider what was to be the type of pipe or protection. It was necessary to have something which would be, if possible, of general application.

78. The group had done a good deal of work. Its deliberations were not yet finished and it would be wrong for Sir George to try to anticipate its findings, but he knew that it had in mind the acceptance of the Authors' view of the desirability of requiring flexible joints as normal practice. He thought it would accept the Authors' view that the omission of site concrete would be of great advantage, and it might accept the Authors' opinion that two, or possibly three, classes of pipe laid at different depths, with flexible joints, would approach as near as possible the ideal. It would be wrong for him to say that the group was prepared at that moment to accept a simplification of that kind; its acceptance would depend on the ability of the manufacturers to produce a satisfactory and reasonably cheap flexible joint, and pipes strong enough to support the loads they had to carry without special protection. The question of the joint was extremely interesting. They had not got very far with it yet, but the progress made was promising. Would the Authors, in their reply, deal with this matter in a little more detail than they had done in the Paper?

79. Sir George remarked that they, the "navvy engineers" on the working group, had been a great nuisance to the Authors, challenging almost every line they had written and almost every idea they had put forward. The Authors had refused to budge an inch. If in the end the working group were able to put forward something of real use and general application, the real credit would be due to the Authors.

Mr F. W. Davey (Consulting Engineer) observed that the Paper was concerned solely with cylindrical pipes, and devoted a great deal of attention to the bedding. Cylindrical pipes were the general rule in Britain and also in America, but on the Continent, and

especially in Germany, flat-bottomed pipes were much more common. They had the advantages that bedding problems were not so difficult, and that it was possible to strengthen the pipe just where the bending moments were greatest. Code or Ministry regulations should not insist that bedding requirements appropriate to cylindrical pipes be applied to all types of pipe, some of which did not need quite such onerous bedding construction.

81. In § 10, the Authors had stated that "in the absence of more precise data the surcharges given in Table 2 should be regarded as minima and, under roads, concentrated loads should be doubled for impact". That impact factor of 2 was much greater than that usually adopted for bridges, and any information justifying that larger factor was very important indeed. The natural assumption would be that in bridge design the impact factor to be adopted should be no smaller than that needed for the pipes. Indeed, it might well be thought that in bridge design a slightly bigger impact factor would be needed; first, because although it was very unlikely that the failure of a pipeline would involve loss of life, it was not inconceivable that the failure of a bridge would do so; and secondly, in the absence of experimental justification it might be thought that the cushioning effect of 4 or 5 ft of backfill would result in a smaller impact effect on a pipeline than on the deck system of a bridge. Would the Authors enlarge on that in their reply?

82. In § 20, the Authors had raised the question of the actual design formula. Mr Davey understood that the textbook formula most used was that the maximum bending moment under a two-point load was $0.159 P.d$. He had secured more than 130 routine results on two-point testing of pipes of diameters ranging from 9–54 in., which had all been made by the same process. The cube strengths were very consistent, so that it could be assumed that the modulus of rupture would be fairly consistent; but the textbook formula resulted in widely different values. By logarithmic plotting he had arrived at a purely empirical formula which fitted this particular range of results much more closely. The empirical formula—he did not advance any theoretical reason for it, for it was purely empirical—was that the maximum bending moment was $0.026 P.d^{\frac{3}{2}}$. That seemed to result in a modulus of rupture more constant than that of the textbook formula.

83. In § 27 reference had been made to placing cradles over the pipe instead of under it. Recently, Mr Davey had made some very small-scale experiments in order to see the effect of putting precast beams over the top of pipes during the backfilling. The experiments seemed to show that by varying the spacing of those precast beams it was possible to transfer, as it were, a proportion of the load from the crown to the haunches, and that by varying the spacing almost pure compression on the pipe could be achieved under any load. If that could be developed it seemed that a promising method would be to standardize as normal a pipe of quite moderate strength, and to cope with exceptionally onerous conditions by means of precast beams. That would have the additional advantage that existing pipelines, laid with pipes of moderate strength, could be strengthened easily without replacement should some later development, such as an aerodrome or arterial road, cross the pipeline. Would the Authors give their views on that suggestion?

Mr W. W. M. Milton (Civil Engineer, Balfour, Beatty & Co. Ltd) said that during the past 12 months his firm had been engaged on the construction, in flexible-jointed reinforced concrete pipes, of several miles of collecting aqueducts on hydro-electric schemes in Scotland. They had encountered certain difficulties, and Mr Milton had been reassured on reading a Paper from the Building Research Station, which proved that they had been, to some extent, right in their findings and in the measures that they had adopted.

85. Class B or Class A bedding, as the depth of fill required, had been used throughout. They had ditched the invert of the trench and had always used granular material beneath the pipe, except where the depths of fill required concrete bedding. They had, however, experienced another difficulty, mentioned by the Authors in § 17, which

related to secondary stresses in the pipe. The pipes, which had been laid in conditions rather more exacting than those applied to sewers, were pressurized and had been subjected to a pressure test. They were at an altitude of between 1,300 and 1,400 ft above sea level. During the recent summer there had been a more-than-average amount of sunshine in Scotland, but the evenings had been cold, and Mr Milton believed that temperature stresses had contributed to trouble that had been experienced with cracked collars. The Authors had recommended a flexible joint as one of the remedies for that type of trouble. Mr Milton's firm had used flexible-jointed pipes throughout, but had still experienced that trouble to a limited extent. He would welcome the Authors' further views on this question.

Mr E. H. Ely (Senior Assistant, J. D. & D. M. Watson, Consulting Engineers, Westminster) said that he had recently laid a 42-in.-dia. concrete pipe, and there had been some doubt whether the concrete bed had been satisfactorily formed. The pipe had been laid on one precast concrete block 9 in. $\times$ 9 in. $\times$ 18 in. transversely, with only about 7 in. between the outside of the pipe and the wall of the trench. The contractor had poured and vibrated the concrete bed and haunch through this gap, and Mr Ely had been worried about whether that would give a good job underneath. Would equally good, if not better, protection have been afforded by placing granular fill, instead of concrete, under the pipe and a concrete middle over it? What was the Authors' opinion of the alternative method of placing the concrete bed first, putting the pipe on the bed, and then concreting the haunch?

87. In their introduction, the Authors had illustrated a new type of flexible joint which incorporated a series of annular rubber lips for which the diameter of a normal concrete pipe socket was reduced by aluminous cement mortar. Had there been any special reason for using aluminous cement? It was notoriously treacherous material, and if it failed the joint would be of very little use.

Mr Fernando Derqui said that the subject under discussion was of great interest to him because he was closely connected with a relatively important Spanish building firm, and they had often met troubles of the kind with which the Authors had dealt. It had been a great source of satisfaction to listen to the discussion of some of the problems and it had taught him a great deal.

Mr A. J. Woodford (Senior Engineer, Sir Alexander Gibb & Partners, Consulting Engineers) noted that Fig. 25 showed two classes of bedding, B and A. It was fairly clear why with Class B the trench had to be excavated to the depth shown, but did the Authors consider it necessary in all cases for Class A to remove a minimum of 6 in. below the concrete bedding?

90. Referring to the Authors' suggestions for the revision of Codes and Standards, Mr Woodford asked for their views on the A.S.T.M. Standards for both unreinforced and reinforced pipes. The A.S.T.M. Standards included not only standards for testing and strengths, very much on the lines that the Authors had recommended, but also reinforcement styles. What were the Authors' views on the advantage of elliptical reinforcement in pipes over the more normal circular section?

Mr G. S. G. Goode (J. D. & D. M. Watson, Consulting Engineers), who had recently returned to Britain after spending 3 years in Auckland, New Zealand, said that in Auckland a volcanic granular material, scoria, was readily available and relatively cheap, whereas concrete cost at least £10 per cu. yd. The Authors need not, therefore, be surprised that considerable savings were effected by using granular material instead of concrete. Many miles of pipes had been laid in that way, and so far as he knew there had been no trouble.

92. As a resident engineer, he had been horrified to watch pipes crack after they had been laid on a concrete bed and haunch across an open bulk excavation. He wondered how often the same thing happened after the trench had been closed up.

The following contributions were received in writing.

Mr John Gaskin (Borough Engineer, Southport) believed that Southport was one of the first local authorities to use the thick-wall pipes referred to in § 57. In formulating the design for these pipes, which were supplied by Messrs Ellis & Sons of Leicester, he had been helped and advised by Mr Clarke.

94. Between July and August 1957, 118 yd of 36-in.-dia., and 232 yd of 38-in.-dia. pipe were laid. By taking out the bed peat which was just below the invert and refilling with sand in which the pipes themselves were bedded, it had been possible to dispense with all concrete protection and with the proposed timber piling under the pipes. There had been some difficulty in obtaining even support for the pipeline on the sand filling before the sand had been filled to about half way round the pipe. It would appear that the use of gravel, possibly hoggin, for filling under the pipes, in accordance with grading for Class B bedding, would have been a better proposition.

95. The 38-in.-dia. pipes were designed for 3,500 lb/sq. ft and the 36-in.-dia. pipes for 4,200 lb/sq. ft, 20% less than the 5,000 lb/sq. ft which Mr Clarke had suggested. The uniform distributed load had been taken as 500 lb/sq. ft, and the higher loading figure had corresponded with the maximum depth of the pipes. Since the pipes were all laid in open agricultural land, and since it was most unlikely that any kind of vehicle would be passing over them, these figures were thought to be adequate.

96. To prevent water draining down the length of the pipeline and possibly washing away the sand underbedding, clay-puddle walls were constructed across the full width of the trench. They had been specified at 150-ft intervals, but the Contractor for his own benefit put them in at 50-ft intervals.

97. Such calculations as were carried out tended to show that the cost of the pipe was very greatly increased by quite small increases in the loading capacity, and Mr Gaskin questioned the necessity to assume the Type H.B. loading on public roads in assessing the necessary strength of these pipes, particularly when a certain amount of arching would be expected from a properly refilled trench. Consideration should also be given to the possibility of constructing the road foundation in reinforced concrete where wide trenches occurred. The remarks about the size of concentrated loading applied particularly to shallow trenches, as would be seen in Fig. 1, and any permissible reduction in the load would probably lower the cost of these pipes.

98. The Authors' statements in § 57 were supported by Mr Gaskin's experience in Southport. The site work began rather slowly owing to lack of experience on the part of the Contractor, but was very rapidly speeded-up. The use of Stanton-Cornelius joints made it possible almost entirely to eliminate secondary loading. An additional advantage to be gained from a properly designed pipe was that the actual weight on the sub-base was less than that which would result from entirely surrounding the pipe with 6 in. of concrete. In Southport that was a very serious consideration, because some pipelines had to be supported by piling, owing to peat and running sand in the foundations. There had been no difficulty in handling the extra-heavy pipes themselves, particularly because there had been suitable machinery on the job.

99. The cost had worked out advantageously, largely because of savings through the omission of piling. Since that date the actual cost of the manufactured pipes had been reduced; the prices negotiated with the Contractor were not entirely representative of what would have obtained had the contract been let in the first case with thick-wall pipes included in the Bill, or had the Contractor had the necessary experience in laying them. Savings up to nearly £4 per yd were in fact obtained.

100. Referring to § 59, Mr Gaskin understood that the Authors would be prepared to accept a 0.01-in.-wide crack rather than the 0.005-in. crack that he had worked to at Southport.

Mr H. T. Hale (Partner, J. D. & D. M. Watson, Consulting Engineers) observed that the use of flexible joints had been growing practice for some time. However, it was interesting to see the reintroduction of the style of pipelaying shown in Class C (ordinary)

in Fig. 26, which, possibly without the tamped granular fill and relying purely on normal backfilling, had been an accepted method of construction in parts of Britain a considerable time ago.

102. The Authors' general recommendation that Class B first-class bedding should be a minimum standard of construction at which to aim, presented considerable difficulties in ground which had a high or varying water table. It was well known that a long length of sewer pipeline traversing, say, a valley introduced a natural plane of weakness which tended to alter the ground-water regime within that valley. The introduction of a granular backfilling as a bed and surround for the pipe would increase the undesirable tendency of the sewer trench to become a new water course. Further, the introduction of a circular bed underneath the pipe and the filling of this with granular material would obviously, under such conditions, attract ground-water movements under the pipeline. Therefore, a heavy granular material without fines should be selected for this bed, to prevent settlement which would result from the transmission of fines down the pipeline trench.

103. In such a case, would the Authors recommend a Class A plain concrete bed, or a Class A reinforced concrete bed as alternative construction? Obviously both styles of Class A bedding would contain the water movements to a more suitable location with respect to the pipeline, but might prove extremely costly in a long line.

104. Another accepted means of containing water movement down a pipeline was the provision at intervals of impermeable stanks across the whole of the disturbed trench area. What was the Authors' view as to the necessity for the introduction of such stanks, and what method of introduction would they recommend for Class B first-class bedding?

105. Mr Hale could not see how either of the preferred modified beddings, Class A or Class B, shown in Fig. 25, could survive if water table conditions varied from below the formation level to, say, 18 in. above the crown level of the pipeline.

106. While the methods of construction advocated by the Authors were admirable for comparatively dry conditions, further thought should be given to their application to alternating wet and dry trench conditions.

107. A further difficulty with the shaped bed in Class B first-class bedding was that little formation was left in which to place underdrains in a wet trench if they were not to be below the permanent works—a location which some engineers considered undesirable.

Mr S. A. Rossiter (Chief Civil Engineer, Merz and McLellan, Consulting Engineers) remarked that the detail of the flexible joints in Fig. 4 might often be difficult of execution and supervision in the rigours of trench work. He suggested the alternative arrangement shown in Fig. 28. The precast cradle of weak vermiculite concrete was of the same profile as the concrete bed or surround; in the latter case, and for larger pipes, the cradle was made up of three or more segments. The invert section of the cradle was long and wide enough to be set in the trench bottom accurately to level in order to receive the sockets of the pipe which could then be set in weak lime mortar on the upper face of the cradle. After the pipes had been laid, using the conventional rubber ring, the joint was completed by setting the remaining vermiculite segments in weak lime mortar around the spigot and pipe and butting against the adjoining socket, so as to provide a continuous weak cross-section at each pipe joint. Concrete bedding or surrounding was then completed around the pipe barrels. Other suitable weak materials could be used instead of vermiculite.

109. But Mr Rossiter doubted whether the technique of supporting pipe barrels was correct. Instead, should the pipes be designed for the joints to be supported? If the support under the barrel of one pipe yielded more than its neighbours, overloading of the joints and pipes at each end of it must result. On the other hand, if the support under a joint yielded, oversteering was avoided until the flexure in it and its two neighbours was fully utilized.

110. The proposal in § 27 to provide a cradle (or saddle) over the pipe appeared to

be consistent with Mr Davey's experiments using precast concrete lintels over the pipe. Those proposals could constitute an independent structure to protect the pipe from superimposed load. Thus, standardized pipes of one conventional design could be used as water carriers regardless of the depth at which they were laid or the nature of the ground, while the load-carrying structure could be dealt with according to the circumstances. The pipe could be completely covered with a few inches of selected filling material, and the saddle or lintel built on top of that. According to the circumstances Mr Davey's lintels might be suitable, or the relieving structure could be a plain or reinforced concrete arch, which in some cases could utilize the sides of the trench as adequate abutments.

FIG. 28

111. In their introduction, the Authors had mentioned the Tyton rubber ring joints commonly used with spun iron pipes. Tenders had recently been obtained for such a pipeline on terms closely comparable with a plain concrete pipeline laid a few years before. It was found that the spun iron pipeline was in fact very slightly cheaper than the concrete pipeline, part of which was surrounded, part haunched, and the rest un-haunched. It would appear, therefore, that an economic case for the extra-strength plain or reinforced concrete pipe suggested in § 57 might in some cases be difficult to justify.

112. In their introduction the Authors had said that sulphuric acid attack on pipes was invariably above the waterline, and had then referred specifically to consequent bacterial effects in sewers. Would they agree that attack below waterline might be expected where sulphuric acid occurred in an industrial effluent arising from inorganic processes?

113. The Authors had also referred to the hardness of the rubber rings used in jointing, and to the bursting pressure that those rings caused. How were those qualities to be measured and controlled?

Mr J. B. Rowntree (Metropolitan Drainage Board, Auckland, New Zealand) noted that the Authors had used an impact factor of 2 and had not varied this with depth, both points being different from American and Australian practice. Would they give the reference to the basis for this?

115. Fig. 5 implied that the manhole settled more than the sewer, but it was possible

for the reverse to be the case. Any movement in either direction of the length of pipe between the flexible joints would build up soil pressures against it and these could generate secondary difficulties. To be really free the pipe should be enclosed in a larger pipe giving it freedom of movement. An alternative which was effective, but not inexpensive, was to stop the pipe flush with the face of the manhole, cast a concrete ring round the end, and seal the joint with a waterstop, with bulb centre capable of taking up any possible movement (Fig. 29).

FIG. 29

116. Regarding the method of adding strength to pipes, Mr Rowntree had shown¹² that the costs of increasing the load coefficient from 1.9 to 3.0 for reinforced pipe of 24, 36, and 48 in. dia. were respectively 35s., 55s., and 75s. per ft by concrete bedding and 2s. 6d., 7s., and 21s. per ft by increasing the reinforcement.

Mr S. D. Mitchell (Deputy Engineer, Stirlingshire and Falkirk Water Board) believed that some engineers who had placed too much trust in the existing codes and had laid large-diameter naked pipes in accordance with them, would have been startled by the discovery that far from the assurance of a factor of safety of about 4, there was a factor of perhaps less than unity. In short, the structure they had designed to be adequate was in reality cracked, or on the point of cracking.

118. If the square of the pipe radius were taken as a function of the moment on the pipe wall, and the square of the thickness of the pipe wall were taken as a function of the moment of resistance, the graph of these values (Fig. 30) showed their wide divergence as the pipe diameter became large.

119. However, many examples could be quoted of naked stoneware pipes, and even the less strong fireclay pipes, up to say 12 in. dia., laid at depths of 10 ft or more, which had stood the test of time. And many engineers would be hoping that the revised codes would still have a ready rule for the small diameters.

Mr A. G. Norman (Chief Designer, Concrete Plants, Stanton Ironworks Co. Ltd) remarked that the Authors had drawn the attention of engineers to the urgent need for a rationalized approach to the design of underground pipelines. They had also exposed how little experimental work had been done in Britain until the past few years, whereas in America, Marston was experimenting in 1910.

121. The American term "load factor" could be confused with the current name for "factor of safety". "Bedding factor" seemed more appropriate.

122. Mr Norman noted that the views expressed in the Paper were far from final,

¹² J. B. Russell, "Concrete pipes and sewers". New Zealand Engineering, vol. 14 (Dec. 1959).

but there appeared to be some inconsistencies in the suggestions put forward. A substantial amount of the information was based upon American experience and experimental work, yet the Authors had recommended an impact factor of 2, which was higher than that used in the U.S.A. In the "Standard Specifications for Highway Bridges", adopted by the American Association of State Highway Officials, the value was given as follows:—

For culverts with a cover of 0–1 ft:

“ “ “ “ “ “ 1 ft 1 in.-2 ft:

“ “ “ “ “ “ 2 ft 1 in.-2 ft 11 in.:

impact value = 30%

 $(F_i = 1.3)^*$

impact value = 20%

 $(F_i = 1.2)^*$

impact value = 10%

 $(F_i = 1.1)^*$

* F_i added for convenience.

FIG. 30

123. Australian standards were similar, and in view of the many pipelines laid, both there and in America, how did the Authors justify a factor of 2? It should also be noted that the British Standard for Bridges recommended an impact factor of 25%.

124. Since there was some confusion regarding crushing loads on pipes, Mr Norman suggested that the term "B.S. crushing test load" (abbreviated to "B.S. load") should be used to define the conditions variously termed in the Paper as "standard crushing test", "two-edge bearing-test-load W_T and W (rubber pads)", and that "equivalent two-edge bearing load" (abbreviated to "two-edge load") be used to define laboratory strength, two-edge crushing strength, and W (two-edge).

Mr G. S. Mitchell (Chief Engineer, Concrete Pipe Dept, Trollope and Colls Ltd) observed that no branch of civil engineering had suffered so much from the neglect of thoughtful design and careful execution as the construction of concrete pipelines.

126. He agreed with the Authors' comments in § 7, for careful quality control of factory-produced concrete was imperative for any pipe manufacturer. Yet when the occasional failure occurred on the site as a result of bad design or poor workmanship, or both, the pipe manufacturer was often accused of having supplied faulty pipes.

127. With regard to the advocacy of flexible joints, probably 40–50% of the spigot-and-socket-type pipes now being supplied by the industry were provided with flexible joints of the rubber-ring type. They were not only used where specified by the engineer, but by laying contractors who often asked for them because of the speed and cleanliness in laying and jointing.

128. Extra-strength reinforced concrete pipes extending over a wide range of diameter were already being supplied to some extent to meet two-edge 0.01-in. crack loads whose required values had been calculated upon the principles enunciated in the Paper. They were being laid upon types of bedding suited to individual circumstances, and in most cases had flexible joints. However, it would be some time before a fully comprehensive range of strength classes for all diameters would be available, for this development involved very considerable modifications of the manufacturer's plant.

129. Referring to § 8, the eight 20,000-lb. wheel loads amounted in all to 72 tons, and with an impact factor of 2 became a total design load of 144 tons on eight wheels, whose points of road contact lay on the perimeter of a rectangle 9 ft by 6 ft. Despite the comment in § 66 (i) Mr Mitchell felt that while such a loading was possible its occurrence on any given public road would be rare, and that in any case the speed at which such a vehicle would travel would inevitably be low. Also, if this load were to be provided for in the design of pipelines under all public roads, on the assumption that no control could be exercised over the use by heavy transport of by-roads, as well as main arterial roads, then what was meant by an "estate road"? While this might be just as much a public road as any other, the liability of the passage of 72-ton-load units must be very remote indeed.

130. If the intention was to design for the worst possible case on every square yard of every public road in Britain, then was it assumed that every public road had, or would have in the future, a paving and surface sufficiently robust to resist damage from such heavy wheel loads?

131. With adequate paving and the moderate speeds of exceptionally heavy transport it was most probable that impact effects would be relatively low in the first instance and would rapidly disperse through the paving and the soil beneath. Therefore, the impact factor of 2 allowed for in Fig. 1 for all depths of cover, right down to 30 ft, seemed over-conservative except perhaps for very shallow depths.

132. A high proportion of the sewers constructed had a cover of the order of 8–10 ft above the top of the pipe, and it was doubtful whether any serious impact effects were likely at this or at greater depths. Fig. 31 showed how the public road loading given by Fig. 1 would be reduced if no impact factor were taken into account, and how this would operate towards economy in design.

133. The difficulty seemed to be the lack of experimental evidence, and certainly the problem required further exploration. A more realistic approach might be to consider the effect of the lighter wheel loads of commonly occurring high-speed heavy traffic. But if the very heavy B.S.153-type H.B. loading were to be officially adopted for pipeline design then no additional impact allowance should be made.

134. The field strength of a pipe was given in the Paper as $W_f = F_m W_T C_{bs}$, where F_m was the bedding factor and had the value 1.9 for Class B bedding. In § 23 it was stated that if Class B bedding (Fig. 25) were used about 50% of the field strength depended on care in construction. That statement was capable of two different interpretations. With Class B bedding properly constructed $W_f = 1.9 W_T C_{bs}$, but if the bedding were very badly done then W_f might drop to, say, $1.0 W_T C_{bs}$, when the conditions would approximate to those of Class D in Fig. 26, where the material surrounding the pipe afforded no effective support. Bad workmanship on the bedding could then result in a complete waste of money. Alternatively, it might be intended that W_f might drop to, say, $1.5 W_T C_{bs}$, which was roughly a 50% reduction in the supporting value of the bedding. Would the Authors clarify this point?

135. Mr Mitchell believed that in any case on such points as particle size, grading, amount of fines permissible, etc., the "selected material" must be specified within fairly close limits, for on many sites it would have to be imported from the nearest

available source; and since cost would be a consideration there might be a temptation to use unsuitable materials. The choice might be from a variety such as crushed stone, pea shingle, gravel, hoggin, sand, etc., and before a decision could be made considerations such as ease of compaction, bulking due to variable moisture content, and so on, would have to be taken into account.

FIG. 31.—PUBLIC-ROAD LOADING FOR A 48-IN.-DIA. PIPE

136. With Class B bedding it would be essential to withdraw all side timbers at least up to the level of the top of the pipe in order to avoid disturbance of the selected material after it had been placed and tamped into position. Clearly, the most meticulous site supervision would be necessary to ensure satisfactory results from this type of bedding.

137. Fig. 25 had evidently been drawn for a very small-diameter pipe. An additional figure showing the design to scale for, say, a 48-in.-dia. pipe would be useful, for it would give a better idea of what the respective beddings would look like for a typical main sewer.

The Authors, in reply, noted that Sir George McNaughton had mentioned cost, variable loading, flexible joints, the elimination of site concrete, and pipe strength, and had particularly asked for further details of joints. So far as cost was concerned, all the indications were mildly favourable and in some cases more than mildly, as witnessed by the remarks of Messrs Gaskin and Rowntree. If account were taken of the expected savings in maintenance, pumping, and treatment costs, the use of rational methods of design and construction should yield very considerable savings in the overall cost of the

pipeline, especially if this cost was defined as the whole of the expenditure on a pipeline throughout its life. Mr Rossiter's remarks about the relative costs of spun iron and concrete raised the question of where, and at what size did prices favour each type?

139. On the question of flexible joints the Authors had no doubt whatever that the greatest source of trouble with rigid pipes was the mortar joint. This was very much worse with salt-glazed pipes than with concrete pipes, because of the differential thermal and moisture expansion of the two materials in the joints, but the rigidity which it produced in a concrete pipeline was also a cause of fracture. Iron pipes also were fractured if rigidly jointed. Most clay bearing soils shrank and swelled at shallow depths with changes in moisture content, and pipelines laid in such soils must move with them and, if a pipeline could not bend it would break. These soil movements (or thermal or moisture changes) also tended to cause lengthening or shortening of the pipeline which, if restrained at the joints, could be fractured in tension or even occasionally in compression. Flexibility and extensibility ($\pm$) were therefore essential qualities in any pipeline. This was very obvious in mining areas. Most engineers would be familiar with the Cornelius type of rubber joint ring, which had been in use for concrete pipes all over the world for the past 30-40 years. There was a variation of it in which mortar was filled into the annular space between the ring and the outside end of the socket. That was not wise; it provided flexibility and then destroyed it. Why it had ever been done remained a mystery. Presumably, someone had been afraid that the rubber would perish or that it would not remain tight, and so had added mortar to make sure. The Authors had also tested the American "Tylox" joint, a multi-lip type which had proved satisfactory for large concrete pipes and gave reasonable flexibility and draw. For concrete pipes, therefore, flexible joints were available and it only remained to make wider use of them.

140. For spun iron there were a number of well-established joints and the new "Tyton" joint which had recently arrived from America. The Authors believed that this joint was now marketed by all the spun-iron makers in Britain and in most European countries, and it appeared to be an excellent simple joint.

141. For salt-glazed pipes the situation was not so easy, because of the obvious difficulty of large and unavoidable dimensional tolerances, but the British Ceramic Research Association had recently developed a form of rubber-ring joint for these pipes. This joint had not yet been tested at the Building Research Station*, but in a number of field tests it had behaved remarkably well. In addition to this joint, which was in its infancy and not yet available for sizes of pipe over 6 in., various other joints which employed hot or cold bituminous material were also available. These were preferable to mortar, but they were not so quick and easy to make as the rubber type. With the hot-poured material it was essential to observe the temperature recommended by the makers. If the material was poured either too hot or too cold trouble would ensue.

142. Mr Milton had mentioned cracked collars in flexibly jointed reinforced concrete pressure pipes. These were most probably caused by the addition of non-uniform thermal stresses to the ring stress caused by the hydraulic and ring pressures. Uneven temperatures in the wall caused a distortion of the pipe from the circular, and it tended to become very slightly egg-shaped. Restraint by the warmer or cooler parts of the wall then produced tensile bending stresses on one side or the other of the pipe wall at the hottest or coldest point, and with sufficient temperature difference the pipe would crack. This type of fracture could even occur when pipes were lying in a stockyard or alongside a trench. In flexible joints of the rubber-ring type the rubber in pressing outwards on the socket or collar induced a ring tensile stress in the socket or collar, as did the hydraulic pressure, so that there was a tensile prestress in the socket or collar before the temperature stresses began to operate.

* 4-in. and 6-in. joints have since been tested and proved to have satisfactory slew and draw for normal use without leakage under a pressure of 20 lb/sq. in.

143. There had been other reports from Scotland of this type of failure where the jointing material was bituminous and the radial pressure therefore small or zero. Mr Goode's trouble was probably of this type and it would normally be expected to occur before back-filling. The obvious precaution was to protect the pipes against differential temperature changes. Mr Rossiter raised the same question in another form. Ring pressure could be estimated by suitable laboratory tests and indeed this had been done. To estimate the stress in the socket was another matter however. Two methods of control were being considered; first, the insertion of reinforcement in the socket ring or collar, and secondly, the adoption of a rubber ring the size and hardness of which would just produce water-tightness under 20 lb/sq. in. test pressure and maximum angular deformation of the joint.

144. The Authors could not answer Mr Ely's question regarding the use of aluminous cement in the "Tylox" joint; that was a manufacturer's problem. They understood, however, that there was no chemical incompatibility between the cements, provided that the Portland was sufficiently hardened before the aluminous was placed.

145. Mr Rowntree's suggestion (Fig. 29) for a flexible connexion to a manhole would appear to be practicable, but it would probably be more expensive than a normal ring-type joint, and not quite so good as regards draw. It was agreed that exceptionally the pipeline might settle relative to the manhole.

146. Mr G. S. Mitchell's remarks regarding the demand for flexible joints were most encouraging. It was excellent news that contractors preferred them.

147. As might be gathered from the Paper and Sir George's remarks, the Authors would like to see the end of site concrete, which was responsible for many of the troubles which were experienced. They recalled one instance of a 72-in. pipe with an 18-in. concrete surround. This had been intended to carry a very large overload (about 80 ft of fill), but before any fill had been placed the pipe had broken transversely at each day's work joint in the site concrete. What was more remarkable was that towards the working end the cracks became increasingly narrower, thus demonstrating that the fractures had been caused by setting and thermal shrinkage which occurred when the thick site concrete surround cooled. As this cooling progressed, the crack widened.

148. Mr Davey had referred to the Authors' concentration on cylindrical pipes, and to the possibilities of using flat-bottomed pipes. No research on flat-bottomed pipes had been done at B.R.S. as yet, but, from general principles, it might be argued that the flat bottom, if thickened suitably, corresponded to the site concrete cradle, but was probably superior to it in quality and field strength. It must not be overlooked, however, that it was adding weight and, in the larger sizes, considerable weight. Whether such pipes in the larger sizes could be handled conveniently on the site, was questionable. They had not laid flat-bottomed pipes larger than about 36 in. Some engineers who had experience of the larger diameters might agree to make a contribution on this subject. The bedding factors which the Authors quoted for cylindrical pipes would not, of course, apply to flat-bottomed pipes for which, in effect, a sand box test load would be the working load, since the flat bottom was virtually the bedding. Attention should be drawn to the necessity for uniformity in the bedding even with a flat-bottomed pipe, because if the bedding were not uniform, there would be trouble. Another point was that most flat-bottomed pipe, so far as they knew, had ogee joints and there were as yet no flexible ogee joint. There was an American version, but the Authors had no experience of it. It seemed doubtful whether, in any pipe of considerable size, such a joint would permit sufficient angular movement without leaking. The normal ogee joint was not likely to remain watertight, because the usual mortar plastered round it would crack as soon as the pipe moved. These joints might be satisfactory for land drainage, but would not appear to be reliable for sewers.

149. The question of pipe strength classification was still open to debate. In reply to Mr Woodford, the A.S.T.M. had recently reduced the whole range of concrete pipes to only two specifications, one for plain pipes up to 24 in. dia. and the other for all reinforced pipes from 12 in. to 108 in. The latter included five strength classes, which was made possible and brief by the device of using a D-load as the specified test load.

This D-load multiplied by the internal diameter of the pipe in feet gave the minimum three-edge test load per foot run of pipe. That seemed to be a very useful simplification of the specification, and it would help the manufacturer when the wall thicknesses were not standardized. It would also help the designer. The Authors hoped, therefore, that this idea would find favour with the B.S.I. and its committees when the matter was dealt with in Britain. An interesting point was that all the loads depended on the outside diameter of the pipe, and not on the inside diameter, and therefore the Authors would prefer to see the "D" in the D-load as the *outside diameter* rather than the inside diameter, particularly since British pipe makers would have to be given freedom with wall thickness for some considerable time to come. Precision of internal diameter would not appear to be of real importance hydraulically, considering the broad assumptions made in estimating flow.

150. Elliptical reinforcement in circular pipes might be attractive theoretically, but in practice there was difficulty in ensuring precision in the position of the steel. During spinning the reinforcement would tend to become circular. There was also some danger of the pipes being laid the wrong way up, which would be disastrous. It was allowed for in the A.S.T.M. specifications, but the Authors had found no enthusiasm for it amongst British pipe makers.

151. In reply to Mr Gaskin, a crack width of 0.01 in. on test was considered acceptable at present. It was specified by the A.S.T.M. and no valid argument had yet been raised against it. It was the width of crack at the surface and in the region of the steel it would be much narrower. It was doubtful whether cracks of this width would occur in normal service with properly designed pipes.

152. Mr Davey's attempt to correlate modulus of rupture with a theoretical two-edge moment had the Authors' sympathy. The difficulty probably lay in the modulus which was a theoretical concept and proved very variable experimentally. The Authors would prefer test results to theory in assessing pipe strengths.

153. They agreed with Mr S. D. Mitchell's general argument—brittle pipes were very variable in crushing strength. Mr Norman's suggestions for standard nomenclature regarding pipe strength were welcome and could with advantage be adopted in future revisions of pipe standards.

154. Mr G. S. Mitchell had found ambiguity in § 23. 50% of field strength meant $W_f/2$, and since $W_f = F_m W_t C_{bs}$ and W_t and C_{bs} did not change, F_m must fall from 1.9 to approximately 1; that was, a bad bed might be better than no bed at all.

155. Mr Ely had referred to the use of precast concrete blocks under a bedded and haunched pipe. It was possible, of course, that the blocks might cause trouble if they were not taken out. The risk might be less if the early part of the placing were all from the same side of the trench. If the concrete were on the dry side and placed from both sides in the early stages, there would be a risk of cavities under the pipe. His other suggestion of putting the bed concrete in first, laying the pipe on it and then putting more concrete higher up, was commonly done but had a weakening effect because the horizontal construction joint was not likely to transmit tensile stress. In the Authors' opinion, bedding the pipe in granular material and placing a concrete cradle on top would be more reliable than either of the above methods.

156. It was interesting to note from Mr Derqui's remarks that similar trouble with pipelines was experienced in Spain.

157. Mr Woodford had asked about the necessity of under-cutting the trench for concrete cradles. In rock this would not be necessary if the concrete bed were sufficiently thick, but in soils which were liable to be uneven it was needed, because it was essential to produce a uniform foundation whatever kind of bed was to be placed on it. If there was a lack of uniformity in the foundation, there would be differential settlement. With flexible joints there was less risk, but the risk was still there, especially with relatively long unreinforced pipes which would tend to be loaded as beams. On the whole, therefore, the Authors felt that under-cutting would be a useful normal practice and would greatly help in the elimination of secondary causes of fracture.

158. The remarks of Mr Manning and Mr Goode on granular bedding were very

encouraging. The Authors were also grateful for Mr Gaskin's testimony and agreed with him that gravel, or preferably sand and gravel in fine-grained soils, would be better than sand alone. Mr Gaskin's note on puddled clay water stops was valuable information.

159. Mr Hale was perhaps under a misunderstanding. The Authors were not recommending beddings inferior to Class B. Mr Gaskin had perhaps given Mr Hale the solution to the problem of the possibility of granular bedding acting as a land drain. On the further point, experience with silts indicated that if only coarse material were used in the bed, differential settlements of the pipes was likely to occur as the foundation soil was squeezed into the voids in the bedding. The Authors did not favour the use of Class A bedding if it could be avoided, and preferred the modified Class B, which did not involve the circular shaping of the trench bottom and which could either constitute, or contain, a temporary drain.

160. Mr Rossiter's suggestion (Fig. 28) might present too much resistance to the shortening of the line. The real solution to the problem was to avoid the use of site concrete.

161. If pipes were supported on their sockets as suggested, they would probably break either as beams or in bearing; the longer the pipe the greater the risk.

162. The Authors were puzzled by Mr Davey's suggestion of cross-beams, and Mr Rossiter's similar suggestion of an arch supported on the fill alongside the pipe. Either would tend to sink with the fill and so bring the load on to the pipe. This was quite different from their suggestion of a reinforced slab, above and clear of the pipe, supported on the undisturbed sides of the trench.

163. They agreed with Mr Norman that "bedding factor" was preferable to "load factor". The latter was the Iowa term. They also agreed with all that Mr G. S. Mitchell had said about "selected material" and the need for site supervision. In

FIG. 32

accordance with Mr Mitchell's request, Fig. 32 showed a 48-in. reinforced concrete pipe with a modified Class B bed as the Authors conceived it.

164. Several speakers had referred to their dissatisfaction with the B.S.153 type HB live load and the impact factor of 2. Public road loading was the concern of the appropriate Ministries and was specified by them. It seemed logical, however, that if bridges must be designed for B.S.153 loading, sewers should be required to carry the same load. There was a difference, however, in the respective factors of safety. There was also some evidence that fast-moving heavy freight vehicles would produce loads on the pipes of the same order as the slow-moving HB train. The designation "estate road" in the Paper referred to private roads and carriageways.

165. With regard to the impact factor, they were advised by the Road Research Laboratory, which had been doing a great deal of work on the soil stresses at various depths below the surface produced by the impact of wheels on road surfaces, that a factor of not less than 2 should be adopted for the present since, although the experiments had so far only covered a depth of about 3 ft, there was no change in the factor in that depth. It should be realized, however, that the deeper the pipe, the less influential the surcharges become and therefore the less important their magnitude and the impact factor. Also it must not be overlooked that the recommended uniform surcharge of 500 lb/sq. ft might cause a higher load on the pipe than the concentrated surcharge and the higher load should always be used in design. The Authors regretted that time and space had not permitted the inclusion in the Paper of more data on live loading. Later studies suggested that it was not perhaps quite so onerous as appeared at first sight. It was hoped that the results of these studies could be published shortly.

166. In reply to Mr Rossiter, the attack on concrete pipes by H_2SO_4 contained in the effluent would naturally be below the water line. H_2SO_4 of bacterial origin always attacked above the water line because it was formed there by the action of aerobic bacteria.

167. In reply to Mr S. D. Mitchell regarding the revised codes, the Authors obviously could not commit the respective committees, but they also would be pleased if the simple rules that Mr Mitchell wanted could be included. They thought that this could be done, if it were acceptable to all concerned, and it would certainly save a great deal of office time.

168. Mr Ely's suggestion with regard to bedding and haunching that the bed concrete should be placed first and the haunching later might, on balance, be considered the lesser of two evils and preferable to the alternative of haphazard casting in one operation, since the weakening effect of the horizontal construction joint was likely to be less serious than the risk of having large voids in the concrete below the invert. It was perhaps not always sufficiently well appreciated by those concerned with the construction work that in order to fulfill its function properly a bed and haunch was called upon to sustain tensile stress, and that this stress was highest precisely at the point where the concrete might be weakest. Although the 180-degree bed and haunch commonly used in Britain was potentially capable of a higher bedding factor than the American 120-degree concrete cradle, it was not, generally speaking, possible to take advantage of this in practice because of the great uncertainty with regard to the tensile value of the bed concrete. Later work at B.R.S. had shown that by providing steel tensile reinforcement at the critical section much of the uncertainty was removed. By reinforcing in this way and thickening the bed, bedding factors of 7 or even higher had been obtained in the laboratory. By inverting the bed and haunch, and employing it as a saddle over the pipe, objections which might be raised to the introduction of reinforcement were minimized, and there were also other advantages. In this connexion it appeared from Mr Rossiter's remarks that there might have been some misapprehension regarding the suggestion put forward in § 27. The object was not to protect the pipe from the load, but to confer certain *constructional* advantages. The load still had to be carried whichever position was adopted, and it was the function of the cradle acting in conjunction with the pipe to do this. The point was, it could do so equally well in either position. It was hoped that it might be possible to publish the results of this later work shortly.