

# FIELD TEST OF 72 IN.-DIAMETER CAST-IN-PLACE NONREINFORCED CONCRETE PIPE<sup>a</sup>

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(Reviewed by the Pipeline Division)

**ABSTRACT:** Monolithic nonreinforced cast-in-place concrete pipe (CIPCP) for storm drain, sewerage, and low-head irrigation pipelines has been in existence since 1954. This is a report on a field test of 72-in. inside diameter (ID) CIPCP with less than 2 ft cover and application of live loads exceeding 2 times H20-44 standard design highway loading. The test is directed and conducted at California State University at Sacramento for California Department of Transportation. The field test confirms the analytical model for satisfactory performance of the pipe under the conditions described.

## INTRODUCTION

On September 10, 1987, a contract was awarded to the Foundation of California State University at Sacramento to design and execute full-scale field testing of a 72-in.-diameter cast-in-place concrete pipe (CIPCP). The purpose of the test program was to develop evidence as to the performance of CIPCP under minimum cover when subjected to highway loads at least as high as the standard H20-44. Soil samples were taken, and soil analyses were performed. Concrete cylinders and cores were prepared and tested. Suitability of the site and the in situ soil, a moist, firm, clayey silt over silty clay, was verified by means of a test pit. Live loads (wheel load) were certified.

The 72-in. CIPCP was chosen as being the most representative size of pipe for the product, which has a size range of 24–120 in.

The CIPCP of this study is a continuous single-stage monolithic concrete casting resulting in the manufacture and installation of nonreinforced concrete pipe used for storm, sewerage, and irrigation pipelines.

The machine used for the casting process for the 72-in.-diameter CIPCP of this study is shown in Fig. 1. The process utilized fixed and movable metal forms, the latter with mechanical tampers and internal vibrators to properly place and densify the concrete.

## RESPONSE OF CONCRETE PIPE

Because of the brittle nature of the material, successful performance of structural elements of concrete requires either a low threshold (or the ab-

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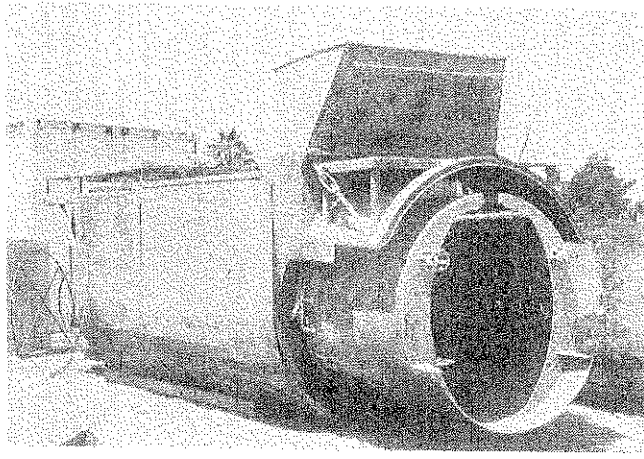
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**FIG. 1. 72-in. Diameter Movable Metal Form Casting Machine for Cast-in-Place Concrete Pipe**

sence) of tensile stress, or the transfer of large tensile stresses from the concrete to tough, ductile fiber reinforcement of which steel reinforcement bars are the most notable example. The CIPCP of this study is an example of the former; reinforced concrete pipe (RCP) is an example of the latter.

Shear forces, in-plane thrust forces, and bending moments in the pipe wall all contribute to the stress response of the structure, when a pipe structure is resisting the application of load. For rigid structures such as concrete, secondary effects due to primary deflections are quite properly neglected; the deformed structure lies well within the bounds of small-deflection theory.

Since the stresses that arise as a consequence of the wall thrust force and the wall bending moment are both parallel and track the circumferential direction of the wall, these stresses are numerically additive. The wall thrust, due to outer wall loads, always results in compression only; the bending moment always results in both tension and compression. A properly designed and constructed CIPCP will have sufficient thrust compression stress (also known in flexible-pipe theory as "ring compression") to mask, at all points within the pipe wall, most (or all) of the flexurally induced tension stress.

To achieve this end, CIPCP is placed in the ground with passively constraining sidewalls, from approximately  $25^\circ$  above spring line on each side down through invert. This trench serves as forms for the casting of the concrete. See Fig. 2 for the standard section.

Any tendency for the pipe to elongate along its horizontal diameter will excite passive soil pressures in the trench walls that will create the appropriate load conditions for the reduction of the would-be bending effects. In much the same way, an arch structure develops lateral reaction thrusts that provide the same benefit. Without adequate sidewall passive capacity, this counter moment benefit may not be relied upon. Such is the case for all pipe, RCP and CIPCP included. In the case of RCP in a trench with sidewall backfill, this passive reaction is dominated by the backfill. For CIPCP, adequate sidewall passive capacity, may be expected from vertical trench-wall faces that become the external forms for the concrete pipe. The absence of tensile distress at crown and invert would be evidence of such operative passive forces in the vicinity of spring line. The test, in part, was designed

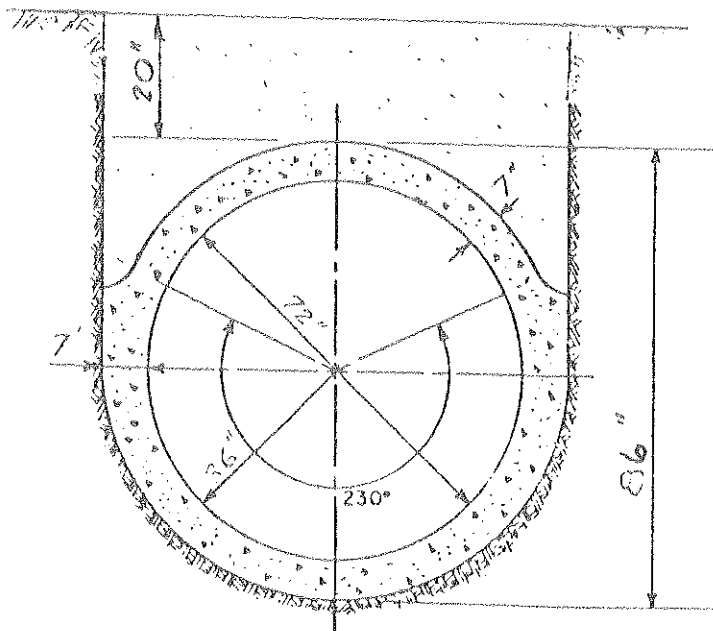


FIG. 2. Detail of Pipe Cross Section for 72-in.-Diameter CIPCP

to offer experimental evidence related to the formation and adequacy of these passive forces.

### CONSTRUCTION

Trench excavation was completed on October 21, 1987. The trench was excavated with the special U-shaped bucket designed for 72-in. cast-in-place concrete pipe construction. The native soil at the site proved ideal for the purpose, permitting vertical trench walls with smooth-cut surfaces, and a firm uniform invert. A laser, installed at the end of the trench, was used by the tracked excavator operator to maintain line and grade using a target affixed to the bucket. Approximately 120 ft of trench bottom was finish-graded for the expected more than 80 ft of test pipe.

October 23, the pipe-casting machine was placed in the trench and secured to the winch cable at the north end of the trench. An engine in the front of the casting unit (pipe machine) provided power required to tamp and vibrate the concrete and winch the unit forward, south to north.

Transit-mix concrete (6 sack, 3,000 psi, 28-day design compressive strength, 1.5–2-in. slump, water-cement ratio of 0.49, 15% fly ash, and water reducer) was fed into a hopper, tamped, and vibrated to force the concrete down and around the steel mandrel. Smooth part-circle aluminum forms were placed inside to support the top two-thirds of the pipe. These 4-ft-long sections are fed through the casting machine just before the wet concrete emerges from the extruder. A workman behind the machine finish-troweled the exposed lower portion of the pipe (invert) as it was exposed.

Concrete was delivered in five transit-mix trucks. Test cylinders were taken from the first four trucks. (The last truck's concrete was not in a significant location.) Concrete from trucks 1–4 covered stations 0–12, 12–30, 30–48, and 48–66, respectively. The test section was at station 48 and therefore, used concrete from truckload No. 3. The concrete cylinder breaks for all cylinders tested ranged from 3,830 psi to 3,900 psi at the 28-day breaks.

## CONSTRUCTION LOAD TESTS

On October 26, test-cylinder breaks were made. The 72-hr compression strengths were 2,300–2,450 psi. Attainment of 2,000 psi strength is an established guideline for starting backfilling operations. Since this was met, backfilling was started south of the test section (station 48) from stations 0–30. The purpose was to demonstrate the pipe's ability to handle construction equipment and backfill dead loads at 2,000 psi compression strength. Native soil, a silty clay from the trench, was placed in lifts and compacted with a vibrating sheeps-foot roller to 20 in. of the pipe crown to relative density of 92.2% Caltrans test standard. A temporary crossing for the test load was graded at stations 12–20. A loaded gravel truck was driven across five times and then parked directly over the pipe for 15 minutes (Fig. 3). The pipe interior surface directly under the wheels was carefully inspected before and after the load test. There was no change or damage. The measured concentrated wheel load on the surface was 15 kips. The backfilling operations and the concentrated wheel load test verify the 2,000 psi criterion as a condition for proceeding with construction after completion of concrete casting.

## INSTRUMENTATION

The strategy for the study of the performance of the pipe subjected to design loads no less than H20-44, included the following four systems of observation.

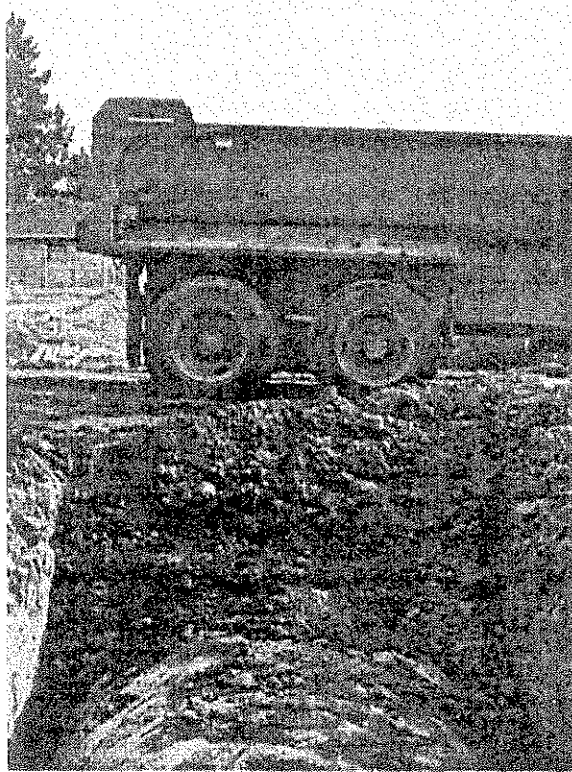


FIG. 3. Loaded Gravel Truck for 72-Hour Test

### Visual Inspection

Visual inspection of the interior of the pipe, for evidence of tension cracks, was performed after the placement and removal of the test loads. On the assumption that, in regions of maximum tension (on the interior wall of the pipe, these occur at crown and invert), cracks may form under live load and then close with the withdrawal of the live load from the pipe, a special effort was made to discover evidence of such cracking. To this end these areas were first wet down and then surface-dried. Any tension cracks that may have closed with the withdrawal of load would be expected to show evidence of such cracking due to the capillary draw of water into the crack. The wetting fluid was water-treated with a mild detergent (Fotoflo 200) for the purposes of breaking the surface tension, thereby enhancing the opportunity for the wetting fluid to be drawn into the crack, should cracks be present.

### Interface Pressure

Five pressure sensors were positioned in a plane normal to the longitudinal axis of the pipe. Two of these sensors, at 5 and 7 o'clock positions, were placed at the soil-pipe interface soon after the excavation of the trench (October 22) and prior to the placement of the concrete (October 23). The remaining three pressure sensors, at 3, 9, and 12 o'clock positions, were installed on October 27 in the soil in preformed pockets.

### Tangential Strains—Inner Pipe Wall

At 24 points around the inner pipe wall surface at approximately every 15° measured clockwise from the crown, a circumferential strain gage was glued to the pipe wall (see Fig. 4). Each gage was calibrated to read directly in microin. of strain. A harness of connecting wires lead to a multichannel switching and balancing unit and digital recording readout instrument, external to the pipe.

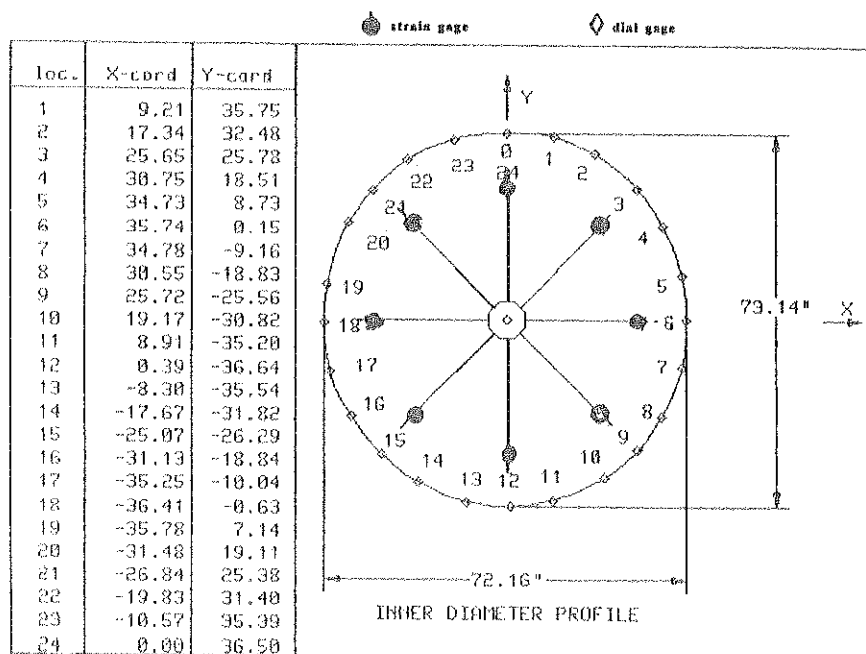


FIG. 4. Location of Strain Gages and Dial Gages

### Radial Displacements—Inner Pipe Wall

At eight points around the inner pipe wall surface, at approximately every 45° measured clockwise from the crown, a mechanical spring-loaded dial gage (see Fig. 4), with a least count of 0.0001 in., was positioned to sense the radial displacement of the inner wall of the pipe. The planes of the dial gages and strain gages were sufficiently close (less than 3 in. apart) to be considered coincident. This coincident plane will, in the future, be referred to as the plane of instrument.

The dial gages were each mounted at the end of one of eight spokes radiating from a central hub. The hub was positioned at the center of the inner-wall circle and was supported by a truss-like frame with a base of approximately 22 ft [see Figs. 5(a) and 5(b)]. Smooth tungsten carbide steel tabs were positioned and glued on the inner wall of the pipe so as to receive the sensing probe of the gage. Enough adjustment potential was built into the system of base, truss, hub, spokes, and gage so as to permit the noted and desired alignment. During the source of the test, the gages were read using a surveyor's transit telescope positioned outside the pipe.

### THE TEST

Intermittent wet weather and wet soils in the test area prevented sufficient time to set up the actual instrumented testing until June of the next year.

On June 22, 1988, the instrumented pipe was live-load tested with an off-highway Euclid RX 35 loaded with aggregate base material to that point where the front axle registered 32 kips (a more critical single-wheel loading than the standard H20-44 dual-wheel loading). The rear-axle loading registered more than twice the standard H20-44 with a maximum dual-wheel load of 35.5 kips.

The latter is the maximum load that was placed directly over the pipe (see Fig. 6). Wheel loads were measured at the site by the California Highway Patrol using standard portable scales.

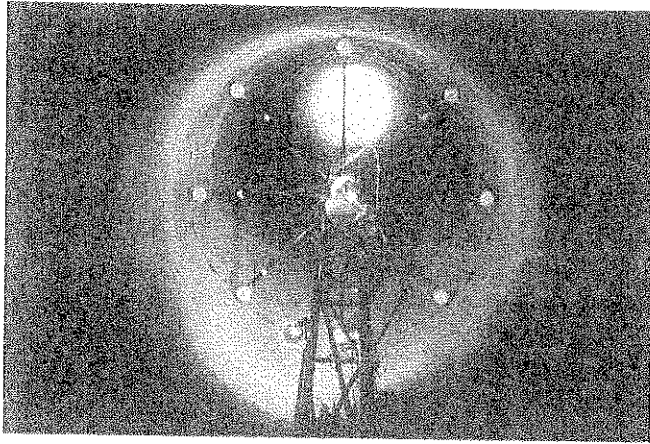
Live-load testing was started at 1:30 P.M. Three load positions were employed.

1. Front wheels on pipe centerline with right front wheel directly over the instrumented X-section.
2. Front wheels on pipe centerline and centered over the instrumented X-section.
3. Rear right dual wheels on pipe centerline and centered over the instrumented X-section.

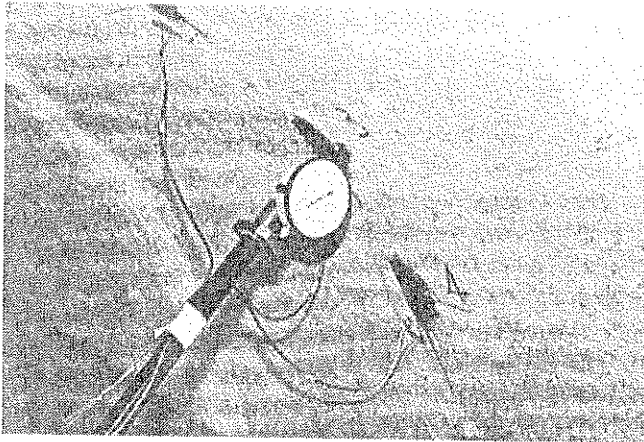
Monitoring the instrument readout during loadings clearly showed that load position 1 produced a greater stress than position 2. As previously planned, this same position was used with the heavier rear wheels as the critical test load. The truck was run back and forth several times, then parked in position 3 for approximately 30 minutes while readings and observations were made.

Visual observations and gage readings indicated no distress whatever in the pipe. Since the structure has passed proof load testing (more than twice an H20-44 wheel load, less than 2 ft of cover, unpaved road) no further tests were made.

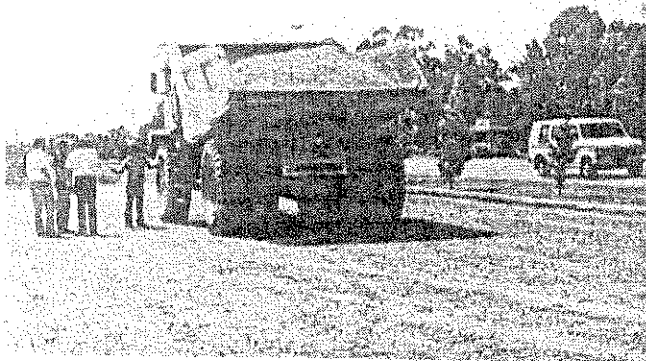
Following testing, soil borings were taken alongside the pipe to obtain data on the in situ material outside the trench. The materials were consistent



**FIG. 5(a). Dial Gages and Support Frame**



**FIG. 5(b). Detail of Dial Gages and Strain Gages**



**FIG. 6. Loaded Off Road Vehicle for Strength Test**

with the initial soil sampling in the trench area, namely clayey silt over silty clay. In-place soil densities varied from 99 pcf to 109 pcf with initial tangential modulus values from 800 to 4,200.

## EXPERIMENTAL RESULTS

The experimental results of the study offer a very compelling statement as to the capabilities of the test section, as a reasonable representative of CIPCP, to successfully resist the unusually large test wheel load of more than 35,500 lb (more than twice the 16,000 lb maximum wheel load of the H20-44 highway design load). As will be described in more detail, the visual qualitative test results, the quantitative displacement-gage measurements of radial displacement, and the quantitative strain-gage measurements of inner-wall circumferential strain all confirm the successful response of the pipe.

The pressure sensors, however, did not successfully measure the interface pressures at the five points previously noted in the section on instrumentation. It is important in experimental design to introduce redundant systems of measurement in anticipation of failure of any one system—an event that often takes place. In this study, three systems of measurement were introduced. The strain-gage system of measurement correlated with the displacement system of measurement. The pressure system of measurement failed. The sensor at 12 o'clock was the victim of an open circuit. The other four sensors just did not respond to the interface pressures known to be acting. It is believed that the long period between the time of installation of the pressure sensors (late October 1987), and the time of the load test (early June 1988), may have been responsible for the difficulty. Minor soil settlements, in part due to the rains, which included minor flooding, followed by the necessary subsequent surface regrading, are believed to have caused a disturbance to the previously secure contacts between the sensing face of the pressure gage and the material to be sensed at the interface. It is worth noting that the other instrumentation systems of mechanical displacement gages and electrical strain gages were installed only one day prior to the test.

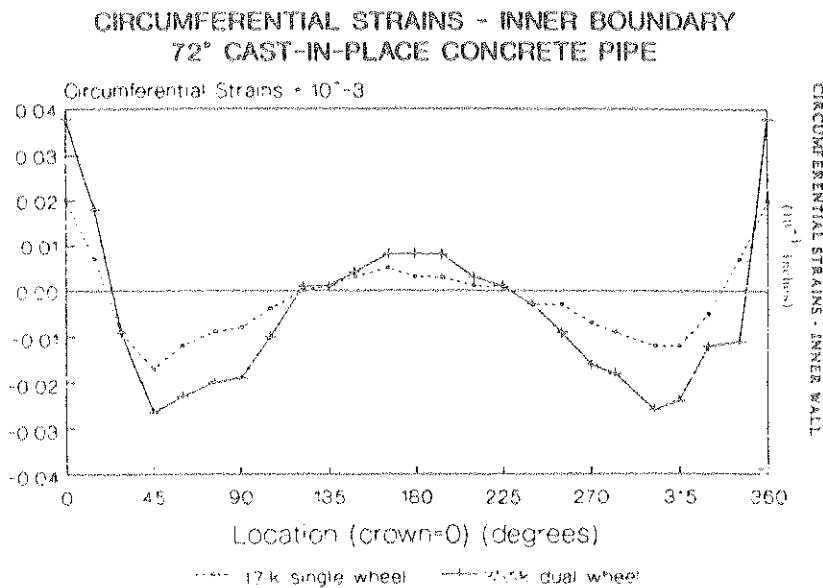
For the following, all measurements of reference are taken at the test section along the inner circumference of the pipe, measured clockwise from the crown as viewed from north to south.

Attention is now drawn to Table 1 and to Fig. 7 wherein circumferential strains are plotted against location. Note the orderliness of the unsmoothed raw strain readings. The largest tension strains occur, as expected, in the vicinities of the crown ( $0^\circ$ ) and invert ( $180^\circ$ ). At the haunch, spring line, and some distance below spring line, the expected fields of compression are noted. Also note that the strains from the 35.5-kips wheel load are approximately double those from the 17-kips wheel load. The tensile strains at the crown indicate maximum tensile stress levels in the order of 60–70 psi and 120–140 psi for the 17-kips and 35.5-kips wheel loads, respectively. These stress levels are less than those predicted by the standard industry calculations. The stresses predicted by analysis are 161 psi and 385 psi respectively for the 17-kips and 35.5-kips loads. The difference is a reflection of the conservative nature of the design.

The radial displacements are listed in Table 2 and are plotted versus location on Fig. 8. Note at first, that the displacements are very small, indeed, and that the maximum radial displacement is at the crown and of a magnitude of approximately 0.004 in. inward for the 35.5-kips wheel load,

TABLE 1. Circumferential Strains

Gage Number (1)	Location (degrees) (2)	Strain: 17k on Instrument Line			Strain: 35k on Instrument Line		
		Zero (3)	Load (4)	Strain $10^{-3}$ (in.) (5)	Zero (6)	Load (7)	Strain $10^{-3}$ (in.) (8)
24	0.00	-0.004	0.016	0.02	0	0.038	0.038
1	14.57	-0.001	0.006	0.007	-0.002	0.016	0.018
2	28.38	0	-0.009	-0.009	-0.002	-0.011	-0.009
3	45.26	-0.001	-0.018	-0.017	-0.004	-0.03	-0.026
4	60.22	-0.001	-0.013	0.012	-0.002	-0.025	-0.023
5	76.91	-0.001	-0.01	-0.009	-0.003	-0.023	-0.02
6	90.53	-0.001	-0.009	-0.008	-0.003	-0.022	-0.019
7	105.29	0	-0.004	-0.004	-0.002	-0.012	-0.01
8	121.98	0.002	0.002	0	-0.001	0	0.001
9	135.02	-0.002	-0.001	0.001	-0.002	-0.001	0.001
10	148.24	0.001	0.004	0.003	0.002	0.006	0.004
11	165.90	-0.002	0.003	0.005	-0.002	0.006	0.008
12	179.51	-0.003	0	0.003	-0.004	0.004	0.008
13	193.32	-0.001	0.002	0.003	0	0.008	0.008
14	210.20	0	0.001	0.001	-0.002	0.001	0.003
15	224.78	-0.002	-0.001	0.001	-0.005	-0.004	0.001
16	239.93	0	-0.003	-0.003	-0.004	-0.007	-0.003
17	255.27	-0.003	-0.006	-0.003	-0.004	-0.013	-0.009
18	270.23	-0.001	-0.008	-0.007	-0.003	-0.019	-0.016
19	282.49	-0.002	-0.011	-0.009	-0.003	-0.021	-0.018
20	302.67	-0.002	-0.014	-0.012	-0.003	-0.029	-0.026
21	314.94	-0.002	-0.014	-0.012	-0.003	-0.027	-0.024
22	329.51	-0.003	-0.008	-0.005	-0.002	-0.014	-0.012
23	345.43	-0.001	0.006	0.007	-0.004	-0.015	-0.011
24	360.00	-0.004	0.016	0.02	0	-0.038	0.038



Tremont Study

**FIG. 7. Plot of Circumferential Strains—Inner Boundary 72-in. CIPCP**

and one-half that for the 17.0-kips wheel load. This low level of displacement is compatible with the performance of a very stiff soil-pipe response.

A study of the response of the test of the 17.0-kip wheel load (6.5% greater than the standard H20-44) shows the total vertical diameter shortening, rigid-body motion culled out, is approximately 0.0008 in. (0.0022 inward at the crown and 0.0014 outward at the invert). This remarkably low level of deformation response points to the likelihood that it is the whole soil-structure composite that is supporting the active live load. This favorable attribute is judged to be created, in large part, by the intimate contact between the outer pipe wall and the wall of the trench at the interface where the two meet. The passive soil thrusts in the vicinity of the spring line create the counter moments in the pipe wall needed to reduce the tension otherwise created by the wall bending moments.

Attention is drawn to the radial deformation response of the pipe when subjected to the 35.5-kips wheel load (122% greater than the standard H20-44) (see Fig. 8). Note that between crown and spring line (0°–90° and 270°–360°) the patterns of radial deformation (solid lines) are such that the recorded displacements of the 17.0-kips and 35.5-kips concentrated loads track one another. Between spring line and invert, the pattern for the 35.5-kips load abruptly changes; the invert moves inward rather than outward. This pattern may be explained by a change in curvature of the pipe shell in the vicinity of the invert, with its effects being registered at 135°, 180°, and 225°. The dotted line in the figure is simply conjecture as to what radial deformations might have occurred in the absence of such a phenomenon.

Very careful and repeated visual inspection of crown and invert revealed no tension cracks. Also, note the low order to the magnitude of the numbers; a measured 0.0015 in. inward at the invert as contrasted with the conjectured 0.0040 in. outward at the invert.

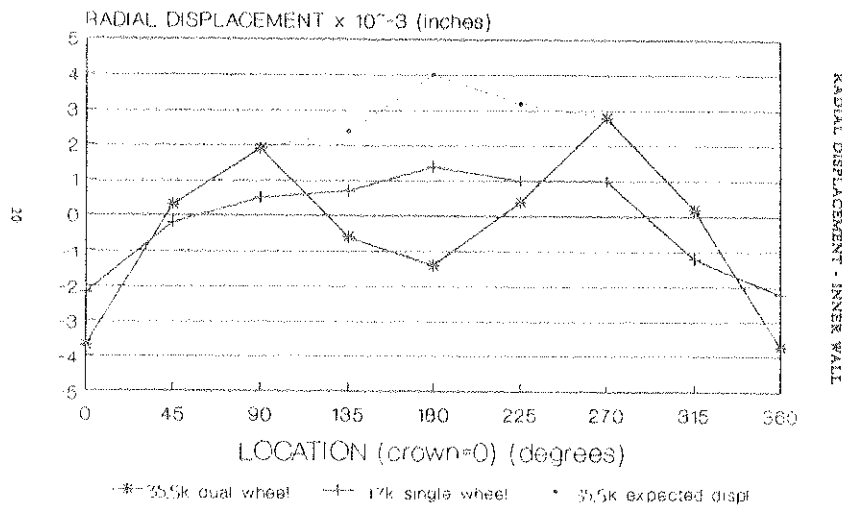
Two possible explanations for this change in curvature are offered:

1. A plastic hinge, at invert, was in the process of forming. Had a fully formed plastic hinge resulted, it is reasonable to expect tension cracks at the invert, an

TABLE 2. Radial Displacements

Gage Number (1)	Location (degrees) (2)	Deflection: 17k on Instrument Line			Deflections: 35k on Instrument Line		
		Zero (3)	Load (4)	DISPL (in.) (5)	Zero (6)	Load (7)	DISPL (in.) (8)
24	0	0.2019	0.2041	-0.0022	0.2031	0.2068	-0.0037
3	45	0.2017	0.2019	-0.0002	0.2031	0.2028	0.0003
6	90	0.2013	0.2008	0.0005	0.2022	0.2003	0.0019
9	135	0.2192	0.2185	0.0007	0.219	0.2196	-0.0006
12	180	0.2188	0.2174	0.0014	0.2175	0.2189	-0.0014
15	225	0.2181	0.2171	0.001	0.2168	0.2164	0.0004
18	270	0.2188	0.2178	0.001	0.218	0.2152	0.0028
21	315	0.2007	0.2019	-0.0012	0.2012	0.201	0.0002
24	360	0.2019	0.2041	-0.0022	0.2031	0.2068	-0.0037

## RADIAL DISPLACEMENT - INNER BOUNDARY 72" CAST-IN-PLACE CONCRETE PIPE



**FIG. 8. Plot of Radial Displacement—Inner Boundary 72-in. CIPCP**

event that did not take place. The cross-section of the CIPCP, a highly redundant structural element, has a great deal of reserve capacity beyond the formation of a fully developed plastic hinge at invert. Two hinges, say at invert and crown, transform the cross section into two stable two-hinged arches. Two more hinges, say in the vicinity of the spring line on each side of the pipe, transform the cross section into two stable three-hinged arches. The conclusion drawn is that a significant reserve capacity exists in this test pipe even after the application of the 35-kip concentrated dual-wheel load.

2. At some level of load, a threshold is crossed beyond which increasing interface shear in the vicinity of spring line exceeds the capacity of the interacting walls of concrete and soil trench. At this point, some vertical interface slippage occurs, which is resisted more competently at invert than at any point between spring line and invert. This phenomenon would result in a reversal of curvature, the event still wholly contained within the elastic response of the structure (as evidenced by the very small movement and the absence of cracks).

In either case, the conclusion drawn is that the pipe has significant reserve capacity after the application of the 35.5-kips concentrated dual-wheel load.

### ANALYTICAL PREDICTIONS

It is the purpose of this section to illustrate that the performance of CIPCP may be anticipated by calculation. The calculations that follow are standard moment and thrust calculations utilizing Paris ring coefficients (Paris 1921).

The principle of the analysis is to calculate the maximum bending moment and normal thrust force in the pipe wall, and then to algebraically add the bending tension stress to the thrust compression stress. This yields the largest magnitude of tension in the pipe wall, a value that is compared with the modulus of rupture for the concrete (appropriate for nonreinforced concrete where bending dominates the stress field). A factor of safety against failure is calculated.

The loads on the pipe are conservatively taken as line loads, the vertical

live and dead loads at the crown and the horizontal load acting at spring line. Other loads considered are the dead weight of the pipe and the water load within the pipe. This latter is included for discussion, but it does not represent the case of the field test of this report. Design values of concrete strength and geometry are used.

## LOADS

The live load on the pipe was delivered by a Euclid R35 off-road hauler. The tire size for this hauler is 1800-33, 18 in. wide with rated pressure of 65 psi.

The front axle of the hauler has two wheels; the rear axle has two sets of duals, each wheel of each dual being 5.25 in. clear distance from its companion wheel. The front wheel placed over the test section carried a load of 17.0 kips; the rear dual placed over the test section carried a two-wheel load of 35.5 kips. The wheels of each test section were oriented normal to the longitudinal axis of the pipe.

## FRONT-WHEEL ANALYSIS

### Vertical Loads

It is common practice to assume that a surface wheel will, for a short length of pipe, load the pipe with a uniform line load acting at the crown. The loading cone shown in Fig. 9 may reasonably lie within those values calculated for  $30^\circ < a < 45^\circ$ .

An impact factor of 20% increases the 17.0-kips load to 20.4-kips.

Using the conservative estimate  $a = 30^\circ$ , the live load (LL) is calculated as:

$$LL = \frac{20.4}{1.5 + (2 \times 1.67 \times \tan 30^\circ)} = 5,950 \text{ lb/ft} \dots\dots\dots (1)$$

Using Marston's Theory for earth loading,

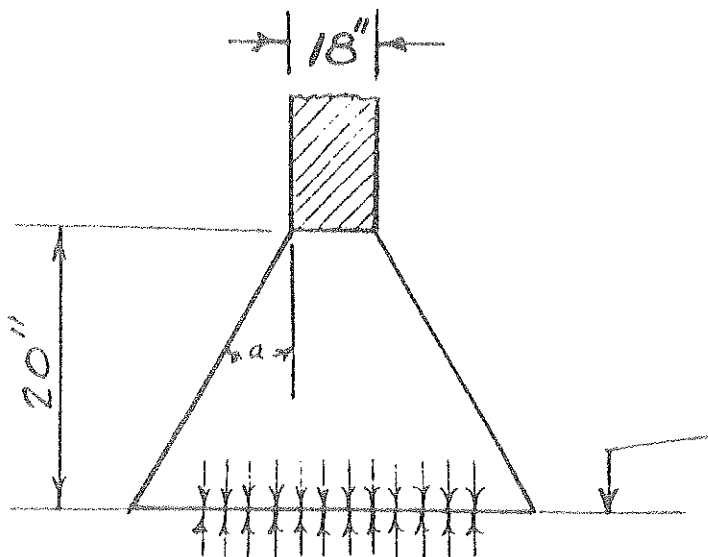


FIG. 9. Diagram of Wheel Load Distribution to 72-in. CIPCP

$$W = C_d w B^2 \dots\dots\dots (2)$$

where  $C_d$  = ditch coefficient = 0.22, for  $H/B = 1.67/7.25 = 0.23$ ;  $H$  = height above crown; and  $B$  = trench width.

The trench soil, a slightly moist, firm-to-hard, brown clayey silt has a unit weight of  $110 < w < 130$  pcf. Using the large value, the earth load is:

$$W_e = 0.22 \times 130 \times \frac{87}{12^2} = 1,500 \text{ lb/ft} \dots\dots\dots (3)$$

The dead weight of the 72-in. diameter CIPCP  $W_p = 1,790$  lb/ft.

The sum of the live and earth loads ( $V$ ) is

$$V = LL + W_e = 5,950 + 1,500 = 7,450 \text{ lb/ft} \dots\dots\dots (4)$$

**Lateral Loads**

In the construction of CIPCP, the trench walls and bottom are the forms against which the concrete is cast. Prior to the surface loading of the pipe-soil composite, the self-standing sidewalls neither lean against the pipe (active pressure) nor are they leaned upon by the concrete pipe (passive pressure). When load is placed on the pipe-soil composite (directly above the pipe) on line with the pipe's vertical centerline, the response of the pipe is a slight shortening of the vertical diameter and a correspondingly small lengthening of the horizontal diameter. The effect is to create a lateral passive pressure at the pipe-soil interface of magnitude bounded by the at-rest pressure and full passive pressure. For reasons explained immediately following, the lowest pressure (at rest) is the most conservative and adopted for this study.

Assume that the top portion of the CIPCP works as a structural arch as noted in Fig. 10. The vertical load at the crown represents the sum of the live and earth loads. The lateral thrust, a consequence of active earth pressure (Rankine coefficient  $>1$ ), is applied at spring line. From equilibrium, it is noted that the vertical reactions ( $W/2$ ) are independent of the lateral thrust ( $H$ ) and bending moment ( $M$ ), given symmetry of loads and reactions. Whatever moment  $H$  and  $M$  excite at other points (say about the crown), is reduced by a moment of opposite sense excited by the spring-line thrusts. For an arch of the geometry noted in Fig. 10, the greater the value of  $H$ , the lesser the net moment, illustrating the structural efficiency of the arch.

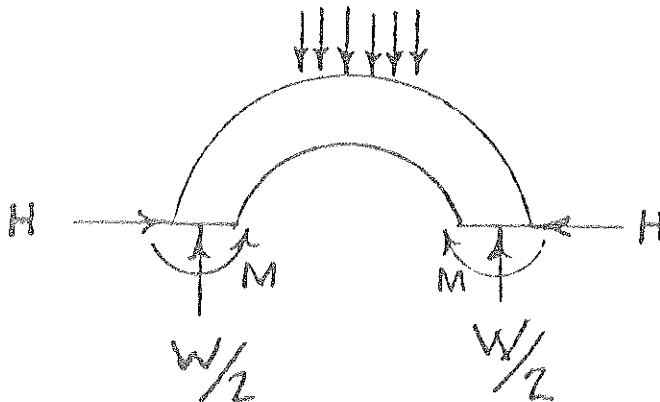


FIG. 10. Plot of Forces Acting on Top Half of 72-in. CIPCP

These same arguments apply for calculation of net moment at invert. Consistent with this, a conservative estimate of the Rankine coefficient of lateral pressure is taken equal to 1 for the purposes of this calculation. Usually a Rankine coefficient of active pressure (0.33) is used in determining the lateral forces at spring line. This is judged to be overly conservative in this prediction calculation in that it is not possible to have active pressure operating against the sidewall of the CIPCP, but undoubtedly reassuring to users.

**Analysis for Stresses**

The maximum thrust and moment of the pipe section are separately calculated and subsequently added. The conditions of loading follow.

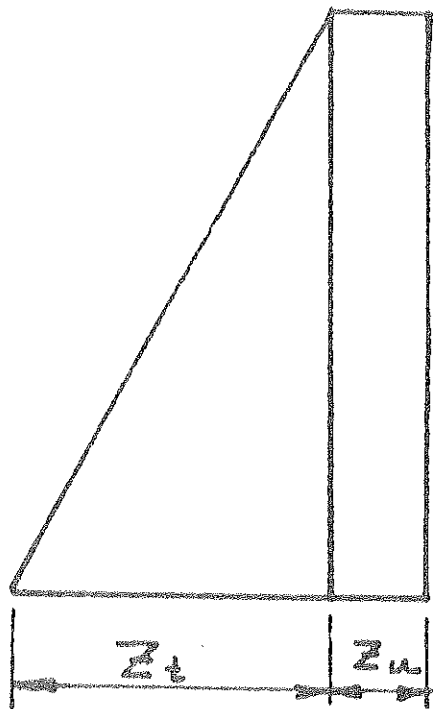
1. Vertical line loads at crown.
  - a. Live load.
  - b. Earth load above crown.

Thrust ( $T_v$ ) = 0 ..... (5)

due to symmetry of loads and geometry.

Moment ( $M_v$ ) =  $0.125VR = 0.125(7,450 \text{ lb/ft})(3.29 \text{ ft})$   
 = 3,060 ft-lb/ft ..... (6)

2. Lateral line loads at spring line (see Fig. 11), where  $T_{lu}$  = thrust in the pipe wall due to  $Z_u$  (lbs/ft of pipe length);  $M_{lu}$  = moment in the pipe wall due



**Lateral Soil Pressure Diagram**

**FIG. 11. Lateral Soil Pressure Diagram**

to  $Z_u$  (ft-lbs/ft of pipe length);  $T_u$  = thrust in the pipe wall due to  $Z_u$  (lbs/ft of pipe length);  $M_u$  = moment in the pipe wall due to  $Z_u$  (ft-lbs/ft of pipe length); and  $R$  = radius to center of pipe wall (ft).

a. Rectangular pressure distribution.

$$\begin{aligned} \text{Thrust } (T_u) &= 1.0(Z_u)(R) = 1.0(kw'H)R = 1.0(1 \times 110 \times 1.67)(3.29) \\ &= 550 \text{ lb/ft min} = 1.0(1 \times 130 \times 1.67)(3.29) = 710 \text{ lb/ft max} \dots\dots (7) \end{aligned}$$

$$\begin{aligned} \text{Moment } (M_u) &= -0.250(Z_u)R^2 = -0.250(1 \times 130 \times 1.67) \\ &\times (3.29)^2 = -590 \text{ ft-lb/ft} \dots\dots\dots (8) \end{aligned}$$

b. Triangular pressure distribution.

$$\begin{aligned} \text{Thrust } (T_u) &= 1.375(Z_u)R^2 = 1.375(kw')R^2 = 1.375(1 \times 110)(3.29)^2 \\ &= 1,640 \text{ lb/ft min} = 1.375(1 \times 130)(3.29)^2 = 1,930 \text{ lb/ft max} \dots\dots (9) \end{aligned}$$

$$\begin{aligned} \text{Moment } (M_u) &= -0.292(Z_u)R^3 = -0.292(1 \times 130)(3.29)^3 \\ &= -1,350 \text{ ft-lb/ft} \dots\dots\dots (10) \end{aligned}$$

3. Dead weight of pipe.

$$\text{Thrust } (T_p) = 0.027(W_p) = 0.027(1,790) = 50 \text{ lb/ft} \dots\dots\dots (11)$$

$$\text{Moment } (M_p) = 0.070(W_p)R = 0.070(1,790)3.29 = 410 \text{ ft-lb/ft} \dots\dots (12)$$

Totals (exclusive of water in the pipe):

$$\begin{aligned} T(\text{min}) &= \text{Thrust}(\text{min soil wt}) = 0 + 550 + 1,640 + 50 \\ &= 2,240 \text{ lb/ft} \dots\dots\dots (13) \end{aligned}$$

$$\begin{aligned} T(\text{max}) &= \text{Thrust}(\text{max soil wt}) = 0 + 710 + 1,930 + 50 \\ &= 2,690 \text{ lb/ft} \dots\dots\dots (14) \end{aligned}$$

$$M = \text{Moment} = 3,060 - 590 - 1,350 + 410 = 1,530 \text{ ft-lb/ft} \dots\dots (15)$$

Maximum stress:

$$f = \frac{-T(\text{min})}{12t} + \frac{6M}{t^2} \dots\dots\dots (16)$$

$$f = \frac{2240}{-12 \times 7} + \frac{6 \times 1530}{(7)^2} = 161 \text{ psi} \dots\dots\dots (17)$$

With a modulus of rupture for  $fc' = 3,000$  psi concrete taken at 492 psi, the factor of safety = 3.0. The 17.0-kip wheel load is predicted to cause no distress to the CIPCP, nor did it. Modulus of rupture is taken as  $9.0\sqrt{fc'}$  (*ACI Materials Journal*, 1990).

**REAR-WHEEL ANALYSIS**

In Fig. 12, note that each of the 18-in. dual wheels, spaced 5.4 in. apart, will distribute the live load a distance of 41 in. along the crown of the pipe, when the angle with the vertical is 30°. For a 35.5-kip dual-wheel load, the effective crown-line load,  $(2 \times 17.75 \times 12 \times 1,000)/41 = 10,390$ , which

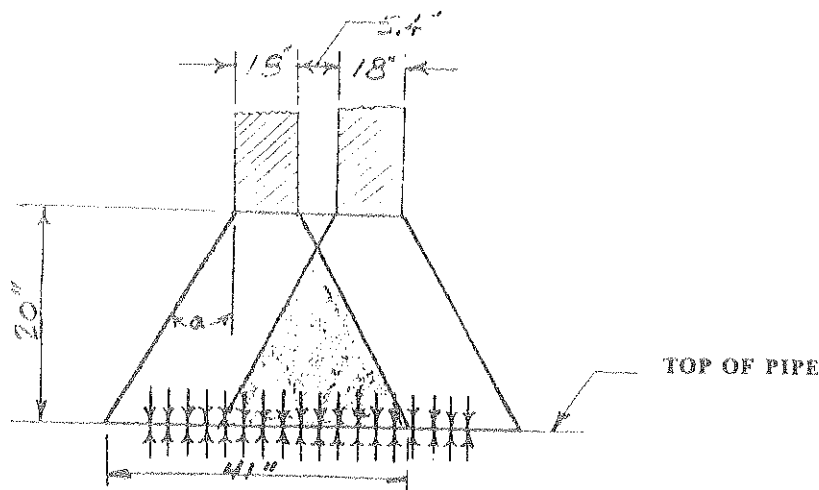


FIG. 12. Diagram of Rear Wheel Load Distribution for Field Test of 72-in. Diameter CIPCP

is greater than  $1.20 \times 35,500/5.37 = 7,930$  lb/ft. The total vertical load is (V):

$$V = LL + W = 10,390 + 1,500 = 11,890 \text{ lb/ft} \dots\dots\dots (18)$$

The moment for this heavier load is

$$M = 0.125VR = 0.125(11,890)3.29 = 4890 \text{ ft-lb/ft} \dots\dots\dots (19)$$

Including the same (as for the earlier calculation of front-wheel load) effects of lateral forces and weight of pipe, the maximum moment =  $4,890 - 590 - 1,350 + 410 = 3,360$  ft-lb/ft, resulting in a maximum tensile stress of:

$$\frac{-T}{12t} + \frac{6M}{t^2} = \frac{-2,240}{12 \times 7} + \frac{6 \times 3,360}{7^2} = 385 \text{ psi} \dots\dots\dots (20)$$

for a factor of safety based on rupture modulus = 1.2. Distress is not predicted for the CIPCP with a dual-wheel load of 35.5 kips, nor did it occur.

#### PIPE FILLED WITH WATER

As a comment, although the test pipe was not filled with water, it is interesting to note that, had the pipe been carrying a full load of water (exclusive of head), it still is predicted to perform well. The thrust and moment:

$$\text{Thrust } (T) = -0.451W = -0.451(1,765) = -800 \text{ lb/ft} \dots\dots\dots (21)$$

$$\text{Moment } (M_w) = 0.070R = 0.070(1,765)3.29 = -410 \text{ ft-lb/ft} \dots\dots (22)$$

For the case of the dual rear wheels, the maximum tensile stress is:

$$\frac{-2,240 - 800}{2 \times 7} + 6 \times \frac{3,360 + 410}{7^2} = 445 \text{ psi} \dots\dots\dots (23)$$

The safety factor based on rupture modulus is 1.1.

## PREDICTION USING $f_c'$ ACTUAL

The design calculations used for prediction are based on  $f_c' = 3,000$  psi, the 28-day design strength of the concrete specified. In actuality, the design strength of the concrete 7 months after the 28-day breaks was determined to be 4,800 psi by the CALTRANS testing laboratory. Based upon this strength, the modulus of rupture is 624 psi. Safety factors for the 17-kip and 35.5-kip loads are 3.2 and 1.5, respectively.

## CONCLUSIONS

1. The 72-in. diameter cast-in-place nonreinforced concrete pipe (CIPCP) constructed to the standard dimensions and material specifications for the product can be safely used under H20-44 highway loads with as little as 2 ft of cover, with or without pavement.
2. Normal backfilling of the pipe may be started when the minimum compressive stress of the concrete reaches 2,000 psi.
3. Performance of this product is dependent upon competent structural support from the in situ soil, which is assured by the construction-site requirements for the product. The properties of the soil at the test section met the established specifications of vertical free-standing trench walls through pipe zone and uniform, firm invert trench, for the in situ material. These practical standards for acceptable sidewall soil have been established by some 35 years of experience, and are described in specifications by American Concrete Institute and many state and local-agency standard specifications. The results verify the suitability of these standards for a normal H20-44 loading.

## ACKNOWLEDGMENTS

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## APPENDIX I. CONVERSION TO SI UNITS

<u>To convert</u>	<u>To</u>	<u>Multiply by</u>
in.	mm	25.4
ft.	m	0.3048
cu ft.	m <sup>3</sup>	.0283
lb	kg	0.4536
lb force/ft	N/m	14.59
psi	MPa	0.00689
kips	kg	453.6

## APPENDIX II. REFERENCES

- "American Concrete Institute 346-90 Specifications & 346R-90 Recommendations for cast-in-place nonreinforced concrete pipe." (N/A). *Manual of concrete practice*, American Concrete Inst., Detroit, MI.
- Paris, J. M. (1921). "Stress coefficients in large horizontal pipes." *Engineering News Record*, 8(19), 768-770.
- Soil engineering*. (N/A). Spangler and Handy Intext Educational Publishers.