Investigation of Backfill Compaction Effect on Buried Concrete Pipes

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ABSTRACT
The present study deals with the experimental investigation of buried concrete pipes. Concrete pipes are buried in loose and dense conditions of gravelly sand soil and subjected to different surface loadings to study the effects of the backfill compaction on the pipe. The experimental investigation was accomplished using full-scale precast unreinforced concrete pipes with 300 mm internal diameter tested in a laboratory soil box test facility set up for this study. Two loading platforms are used namely, uniform loading platform and patch loading platform. The wheel load was simulated through patch loading platform which have dimensions of 254 mm *508 mm, which is used by AASHTO to model the wheel load of a HS20 truck. The pipe-soil systems were loaded up to pipes collapse. Pipes were instrumented with strain gauges to measure circumferential strains, in addition to dial gauges, for measurements of the pipe vertical deflections and settlement of the loading platforms. The test results indicated that flexure governed the buried pipe behavior. Flexural cracks formed slightly before the ultimate load. A comparison of soil backfill, between a loose and dense compaction, showed that the dense backfill improve largely the pipe installation and the strength of pipe-soil system.

KEYWORDS: Buried concrete pipes, Backfill compaction, Bedding factors, Strain gages, Patch loading.

تحري تاثير حدل تربة الدفن على الانابيب الخرسانية المطمورة

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الخلاصة
الدراسة الحالية تتعلق بالتحري التجريبي للانابيب الخرسانية المطمورة. في هذه الدراسة تم دفن الانابيب في ظروف تربة مرصوصة و غير مرصوصة من الحصى الخابط و تم تسليط احمال سطحية مختلفة لدراسة تاثير حدل تربة الدفن على سلوك الانبوب. تم اجراء فحص النماذج باستخدام انابيب خرسانية غير مسلحة مسبقة الصب باحجام حقيقية و بقطر داخلي مقداره 300 ملم حيث تم فحصها مختبريا باستخدام صندوق تربة و هيكل فحص تم تصنيعهما خصيصا لهذه الدراسة. تم استخدام نوعين من منصات تسليط الاحمال و هي منصة تسليط الاحمال المنتظمة و منصة تسليط احمال الرقعة (الاحمال المركزه) . حمل اطار المركبة تم تمثيله من خلال تحميل الرقعة و التي ابعادها 254*508 ملم و المعتمدة من قبل AASHTO لتمثيل حمل اطار مركبة HS20. منظومة الانبوب-التربة تم تحميلها لغاية انهيار الانبوب. جُهزت الانابيب بمقاييس الانفعالات لقياس الانفعالات المحيطية بالاضافة الى تم تثبيت مقاييس الهطول لقياس التغييرات العمودية في قطر الانبوب بالاضافة الى مقاييس نزول منصات التحميل. نتائج الفحوص اشارت الى ان سلوك الانحناء هو الذي يحكم تصرف الانبوب و ان شقوق الانحناء تتكون قبل حمل الفشل بقليل. من مقارنة تربة الدفن بين الحدل المفكك و المرصوص تبين بان الدفن المرصوص يُحسن بشكل كبير طريقة تنصيب الانبوب و كذلك قابلية تحمل منظومة الانبوب-التربة.

الكلمات الرئيسية : الانابيب الخرسانية المطمورة ، حدل تربة الدفن ، معاملات الوسادة ، مجسات الانفعالات ، احمال الرقعة.

INTRODUCTION

Buried pipes are constructed from various materials in wide range of sizes and shapes and were primarily used for drainage applications prior to 19th century. Today buried pipe infrastructure serve many purposes, including sewer lines, drain lines, water mains, telephone and electrical conduits, highway and railway culverts, gas and liquid-petroleum lines, coal slurry lines, pedestrian and stock passes, subway tunnels, heat distribution lines and numerous other special functions [Bashir 2000].

Buried pipes are classified as either rigid or flexible. A flexible pipe is defined as one that will deflect at least 2 percent of its diameter without structural distress while rigid pipe is generally those that cannot deflect more than 2% of its diameter before failing. In the industrial market, the most common rigid pipes are clay, cast iron, unreinforced concrete and reinforced concrete pipes, etc, while flexible pipes includes PVC, Steel, Ductile iron, etc [Moser and Folkman 2008].

Concrete Pipes plays a significant role in highway construction, sewerage water disposal and canal flow. The economics of manufacturing, durability of pipe, and rigidity under a load make them an attractive choice in many situations [Haque 1998].

Today, it is well known that besides the pipe material, the installation procedures have a great effect on the performance of the pipe-soil system.

The present study is to evaluate the structural behavior of a full scale non-reinforced concrete pipe installed under earth and exposed live loadings. The pipes installed in a laboratory soil box. Gravelly sand was used as the soil. Deep burial was simulated by applying a surcharge load after the soil box had been filled with soil. The loading system is a special load frame facility.
The pipes were instrumented to record data for strain and the change in vertical diameter. The strain gages were wired to data acquisition system. Data was directly read by a computer activated system. The measured data were the applied load, surface strains at crown, invert and springlines, vertical diameter change (deflection) and loading platform settlement.

The main objective of the current work is to investigate the behavior of the concrete pipes subjected to different loading systems and its interaction with surrounding soil at different compaction efforts.

MATERIAL DESCRIPTIONS
CONCRETE PIPE

Full scale non-reinforced concrete pipes of 300 mm internal diameter, 394 mm external diameter and thickness of 47 mm were used in the present work. The length of pipes was 1000mm. The concrete compressive strength of the pipe is determined by using non-destructive tests, namely ultrasonic pulse velocity. The ultrasonic test was performed on two pipes, for each test 10 readings were taken by direct method, and the average compressive strength is about 30 MPa.

The elastic constants of concrete and steel (in case of reinforced pipe) are necessary when calculating wall thrust and bending moment. The elastic modulus values used in concrete design computations are usually estimated from empirical expressions that assume direct dependence of the elastic modulus on the strength and density of concrete [Mehta and Monteiro 2006].

According to ACI Building Code 318 [ACI Code 318M 2008], with a concrete unit weight between 1500 and 2500 kg/m3, the modulus of elasticity were determined from:

$$E_c = \gamma_c^{1.5} * 0.043\, f_c'^{1/2} \quad (1)$$

Where
E_c = static modulus of elasticity (MPa)
γ_c = concrete unit weight (kg/m^3)
f_c' = 28-day compressive strength of standard cylinders (MPa).

Thus, based on the density of concrete pipe of 2280 kg/m^3 and the compressive strength of 30 MPa, the value of modulus of elasticity according to Eq.(1) is 25634 MPa. In this study Poisson ratio is assumed equal to 0.129 [Haque 1998].

The cross sectional area A and moment of inertia I of the pipe, per unit length (namely plane pipe) are $A = 1 * t$ and $I = 1 * t^3/12$, and they equal to 47 mm^2/mm and 8651.917 mm^4/mm respectively.

BACKFILL SOIL

The soil used in this study was relatively uniform gravelly sand with maximum particle size of 25 mm, it pass through sieve #25.

According to ASTM D422, the backfill soil composed of approximately 13 % Medium Gravel, 74% Sand, and 1.17% fines passing sieve No. 200, by weight. The particle size distribution curve for backfill material is shown in **Fig.1**. The uniformity coefficient, Cu and coefficient of curvature Cc are calculated based on **Fig. 1**.

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.9}{0.125} = 15.2 \qquad (2)$$

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(0.31)^2}{0.125 * 1.9} = 0.4046 \qquad (3)$$

Where, D10, D30, and D60 are the diameters corresponding to percents finer than 10, 30, and 60 %, respectively [Das 1984]. Based on **Fig. 1** it is found that $D_{10} = 0.125$ mm, $D_{30} = 0.31$ mm and $D_{60} = 1.9$ mm.

Thus based on grain size curve and with the uniformity coefficient, Cu = 15.2 and the coefficient of curvature, Cc = 0.404, the backfill soil is classified as poorly graded sand with gravel (SP) according to unified soil classification system USCS. And it is classified as A-1-b soil class according to AASHTO. According to Iraqi standards this soil is classified as Granular base material Type D. According to ASTM C1479-07a it is meets the gradation requirements of Category I-SIDD (Standard Installation Direct Design) soil. The soil classification according the above data is summarized in **Table 1**.

BEDDING

Bedding is the first layer of embedment material around the tested pipe. It was finished before the installation of pipe. The bottom base of the soil box was directly overlain by the gravelly sand material which served as the bedding layer. Overall thickness of the bedding layer below pipe invert is constant for all tests and equal to 30 cm. Before transporting the complete instrumented pipe to the Lab the trench and bedding is completed. compacted backfill soil (Dense bedding) were used in this study, The dense bedding layer was leveled and compacted sufficiently using hand tamper.

DESCRIPTION OF EXPERIMENTAL APPARATUS

SOIL BOX

A fabricated steel box with dimensions 1.5m by 1.05m in plane and 1.3m in height was used to study the behavior of buried concrete pipe. As mentioned previously the pipes had a length of 1000 mm, so that the ends of the pipe would not touch the sidewalls of the test box in which its width equal to 1050 mm. This means that the ends of the pipe are not restrained, but are free to undergo axial (longitudinal) expansion just as they would if placed with bell and spigot connections in the field [Moore et al., 2004].

The soil box was designed to have two trench widths, a wide trench, 1.5 m wide in which the clear distance between pipe and the trench wall is 1.375 Do , namely 0.55 m , and a narrow trench, 0.8 m wide, in which the clear distance between pipe and the trench wall is 0.5 Do , namely 0.2 m. The natural ground (at bedding foundation and two sides of the trench) was simulated by soil box steel walls.

LOADING SYSTEM

The loading frame system included three essential components:

1-Reaction frame: which is constructed using steel I sections and anchored into concrete footing with clear width of 1.73 m and clear height from the ground of 2.21 m, as shown in **Fig. 2**.

3-Hydraulic jack: A hydraulic jack of 400 kg/cm^2 maximum capacity (which equivalent to 39.3 MPa or 23 tons in force unit) with a piston diameter of 87.8 mm was used to apply the load on the backfill. The piston is connected to hydraulic power supply/control unit which operate manually.

3- Loading Platform: After the tank was filled with soil a grillage consisting of three layers of steel sections was assembled on 6mm steel plates rest on top of the leveled soil surface for loading applications as will be discussed later.

MODELING OF LOADING

The load was transferred from the bottom face of the hydraulic cylinder to the top of the backfill over the pipe by a loading platform. There are two types of loading platforms depending on the type of loading namely uniformly distributed load and patch loading.
Thus two types of loading platforms were used in the present study, namely uniform load platform and patch platform. Each platform is a simulation of an actual loading type, as summarized in **Table 2**.

UNIFORM LOADING PLATFORM (EARTH FILL)

The bottom of the platform comprised of steel plates of thickness 6 mm and dimensions cover the top of leveled backfill, and then closely arranged IPE 140 steel I-beams of 1m length which approximately cover the top of the backfill and its top plates. At the top of IPE 140 beams structure there was two HP 150 beam sections installed transversely at 1/3 span of the IPE 140 beams from each end so that the load on these HP 150 beams will distributed equally to the IPE 140 beams. The hydraulic cylinder rests on a heavy beam of length 0.85m which in turn rest on the transverse HP 150 beams.

PATCH LOADING PLATFORM

A steel plate of size 508 mm * 254 mm with thickness of 30 mm was used for transferring the load from the hydraulic cylinder to the top of the backfill to represent the AASHTO HS20 truck wheel loading.

The dimensions of the platform plate is based on the AASHTO specifications, due to the HS 20, 32,000 pound and the Alternate Truck 25,000 pound design axle are carried on dual wheels. The contact area of the dual wheels with the ground is assumed to be a rectangle, with dimensions of 10 in * 20 in, [ACPA 2009].

DESCRIPTION OF EXPERIMENTAL INSTRUMENTS

The experimental instruments used in the test series of pipe included mainly two systems: the data acquisition system, the pipe deflection measurement system, and the loading system. The data acquisition system consists of strain indicator unit. The pipe deflection measurement system consists of a dial gauges. The loading system is a special load frame facility.

DATA ACQUISITION SYSTEM

Strains of pipe surface were measured using a mobile data acquisition system. The data acquisition system used in this study includes personal desktop computer, strain indicator has 8 channels and accompanying software. The measured data comprised output voltages, which were then post processed using application programs and spreadsheets to derive design variables.

In the present work the strain gauges were directly wired into the individual quarter-bridge Wheatstone circuit using the active-dummy method, where one strain gage serves as a dummy gage and one strain gage serve as active gage.

TML Strain gauges Type PL-60-11 are used in all tests. It is an electrical resistance unidirectional strain gauges metal foil gages 60mm in length designed for measuring concrete strain on the surface of a concrete structure. All gauges had a nominal resistance of 120 ohms and a nominal gauge factor of 2.22.

There was one primary section for each pipe. Four strain gages were installed in each layer (inner concrete and outer concrete) of the primary section. A total of 8 gages were installed for each pipe, which sums up a total of 16 gages for both active and dummy pipes. Uniaxial strain gauges were used to measure the circumferential strains at four separate locations around the pipe circumference: at the Crown, Invert and both Springlines. The strain gauge locations were at mid-span for patch loading and at 330 mm for uniform loadings.

DEFLECTION DIAL GAUGE

A dial gage of 1 cm reading capacity was used to measure the vertical deflection of pipe. A special stand was manufactured capable of adjusting the vertical position of dial gauge arm in addition to keeping it firmly inside the pipe as shown in **Fig. 3**. Dial gauge is installed at mid span for patch loading and at 250mm for uniform and strip loadings.

SETTLEMENT MEASUREMENT DIAL GAUGE

The settlement of the loading platforms was monitored using dial gauge of 5 cm reading capacity.

COMPACTION CONTROL OF GRANULAR FILL AND COMPACTION EQUIPMENT

In geotechnical engineering practice, it is customary to use the dry density of the compacted fill to control the field compaction operation. Accordingly, a standard Proctor density test, AASHTO T-90 or ASTM D698 (or Modified Proctor Compaction Test AASHTO T-180 or ASTM D1557) is performed on the soil and the maximum dry density of the soil determined. The target dry density to be achieved in the field is then expressed as a percentage of the maximum dry density.

In this study the compaction characteristics of the test soil were determined in accordance with Modified Proctor Test, ASHTTO T-180. Six compaction tests were performed to obtain the moisture-density relationship of the backfill soil. The maximum dry density obtained was 21.5 kN/m^3 with a corresponding optimum moisture content of 7.6% as shown in **Fig.4**.

The field compaction devices that are most commonly used in the compaction of pipe embedment materials are impact rammers and vibratory plates. In this study the mechanical compactor could not be used due to the limited space of soil box, therefore a hand tampers are used as compaction tool, namely manual compaction is used in this study. Two steel hand tampers with height of 1.35 m are used in this study, one with contact area of 200 *200 mm steel plate with thickness of 16 mm, and total weight of 14.7 kg as shown in **Fig. 5**. This tamper was used for all layers except for narrow trench between pipe and wall region. The second is smaller than first tamper; which consist of a steel plate with 200 * 100 mm contact area and thickness of 16 mm, and total weight of 10.9 kg as shown in **Fig. 5**. This tamper was used mainly with narrow trench conditions to compact the soil between pipe and trench.

FIELD DENSITY MEASUREMENT

In-situ density and moisture content of the backfill and bedding layer was monitored according to AASHTO T191-86 sand cone method. Sand passed through sieve No. 20 and retained on sieve No.50 was used in the sand cone apparatus. Density of the standard sand was determined in the laboratory and the average value was 1.53 g/cm^3. Weight and volume of the soil specimen was measured in the lab to compute the test density and then the compaction degree based on the maximum dry density of 21.5 kN/m^3 and optimum moisture content of 7.6%. The backfill compaction results for each test and bedding layer compaction are presented in **Table 3**. In the present study the terms dense soil or dense compaction are used for compaction greater than 92% and loose soil or loose compaction for compaction less than 90%.

TEST VARIABLES

Test variables included trench width, compaction degree, and backfill cover. All these variables are investigated under two types of loadings, uniform, and patch loadings.

The investigation is accomplished to select which combinations could provide the proper information. A total of 4 tests were conducted with the test variables and 2 tests for the sake of failure load analysis (bedding factor analysis) in addition to the three edge bearing test, thus a total of 7 tests were accomplished. The test variables are summarized in **Table 4**.

The descriptions of tests according to their variables are shown in **Fig. 7**.

THREE EDGE BEARING TEST, TEB

A three-edged bearing test is used to determine the strength of a rigid pipe in which this test strength is directly related to the load carrying capacity of the buried pipe. The pipe is supported at two locations along the bottom, and a vertical load is applied at the top until the pipe fails. When concrete pipe is subjected to a load, either by a testing apparatus or a field installation, this load tries to deform the pipe into an elliptical shape. During the loading process, tensile stresses develop on the inside of the pipe at the crown and invert and on the outside of the pipe at the springline, and compressive stresses develop opposite these tensile stresses. Since concrete is

strong in compression but weak in tension, cracks form in the tensile zones [WRI 2003].

The test pipes failed during the three edge bearing test at the applied vertical pressure of 73.79 bar which equivalent to a line load of 44.68 kN/m. Thus according ASTM C14M, the test pipes were belonging to Class 3.

BEDDING FACTORS ANALYSIS

As described earlier, the test pipe was subjected to loading controlled by a hydraulic jack. The load was applied until one of the following conditions was met:

1. Maximum capacity of the load cell was achieved, or
2. Invert and crown strain gages had failed, or
3. Failure of the pipe

The failure load is good criterion for the sake of comparison between tests and clearly reflects the effect of test variables on the pipe strength. It should be noted that failure load here is the load of total collapse of the pipe. Also due to unreinforced concrete pipe the crack load is slightly less than collapse load.

In this study the failure load analysis was achieved through the bedding factor analysis, in which the Three Edge Bearing TEB test result is considered as a reference quantity for comparison of buried pipe strength or loading capacity. Due to TEB test results was in kN/m units at the pipe crown, therefore the vertical stress at pipe crown level due to the failure loads of different loading platforms are calculated firstly using or approximate method and then converted from stress to equivalent line loads.

The vertical stress at the crown level due to patch loading platform is determined using an approximate method called the 2:1 method for rectangular loads. In this method the surface load on an area B x L is dispersed at a depth z over an area (B + z) x (L + z) as shown in the **Fig. 6**. The vertical stress increment under the center of the load is [USACE (EM 1110-1-1904) 1990]:

$$P_Q = \frac{Q}{(B+z)\,.\,(L+z)} \qquad (4)$$

Where Q = is the resultant of a surface rectangular load, kN.

Finally, for uniform loading platform which could be expressed as simulated earth fill; the overburden pressure due to gravity loads is expressed by the following relation:

$$P_g = \gamma_s H \qquad (5)$$

where, γs = unit weight of soil.

Thus after determination the vertical stress at the buried pipe crown due to applied surface load, it is converted to equivalent line load at the pipe crown by multiplying the stress by outside diameter of the pipe, namely 397 mm as shown in **Table 5**.

Once the pipe load has been determined, the next step in pipe failure load investigation involves defining the bedding factor. The bedding factor is defined as the ratio between the supporting strength of the buried pipe to the strength of the pipe in a three-edge bearing test [Selig and Packard 1987]. The bedding factor is determined according to the following equation [Wong et al., 2002]:

$$B_f = \frac{W_L + W_E}{TEB} \qquad (6)$$

Where B_f is the bedding factor, W_L is live load such as vehicle load, W_E is earth load (here due to 30cm or 60 cm backfill covers) and TEB is the Three Edge Bearing test strength. According to above equations the bedding factors for each test were calculated and summarized in **Table 5**.

In the present study the bedding factors are indication for quality of pipe-soil system, namely as the bedding factor values are high as the installation quality is good. Thus higher values of bedding factors are not necessary to be accompanied with higher failure loads.

Based on **Table 5**, the highest bedding factor is 2.30 for Test No.5 and lowest bedding factor is 1.26 for Test No.6, these results are expected due to Test No.5 used dense backfill while for Test No.6 uncompacted backfill was used.

For the same backfill cover and the same bedding conditions as for Tests No.1 and 2 the bedding factors are considerably different due to different compaction efforts which indicate that the pipe-soil strength considerably is affected by the backfill compaction although similar conditions of bedding or cover conditions.

ANALYSIS OF BACKFILL COMPACTION EFFECT

The comparison was accomplished for two loadings types, uniform loading and patch loading as shown in **Table 6**.

STRAIN ANALYSIS

Generally, the experimental data obtained from the tests were not the direct reading of strain, and they were only the readings of voltage change. Therefore, to obtain the readings of the strain, the experimental data needed to be transformed after tests, in this study an Excel Spreadsheets are developed for data transformation.

According to quarter-bridge Wheatstone circuit, if the resistances are R1, R2, R3 and R4 (in Ohm, Ω) and the bridge voltage is E (in Volt, V). Then, the output voltage v_o (Volts) is obtained with the following equation:

$$v_o \approx \frac{1}{4} \cdot \frac{\Delta R}{R} . E = \frac{1}{4} \cdot G_s \cdot \varepsilon . E \qquad (7)$$

Where
R1 = Strain gage resistance
ΔR_1 = Change in strain resistance
ε = strain
G_s = Gage factor

Thus the obtained is an output voltage that is proportional to a change in resistance, i.e. a change in strain. This microscopic output voltage is amplified for analog recording or digital indication of the strain.

Thus the strain is computed from the following equation

$$\varepsilon = \frac{4 * \Delta V}{G_s * E} \qquad (8)$$

Where ΔV = change in voltage (reading of the increment of voltage)

In general the strains for loose compaction were always greater than that of dense compaction which is expected conclusion as shown in **Fig. 8** to **Fig. 11**. At springlines there are relatively slight different between tests set for both loading conditions as shown in **Fig. 10** and **Fig. 11** which mean that there are small difference between compaction in both tests either loose or dense compaction. The approximately similar compaction effort at springlines of tests sets are due to the compaction by hand tamper faced difficulties at springlines region such as limited space or avoid pipe damage, thus this results are expected.
The tensile strains are relatively closed between tests set in case of uniform load but there are clear gaps in case of patch loading especially at invert and crown as shown in **Fig. 8b** and **Fig. 9b**, in which there are rapid and sharp increasing in tensile strains with increasing loadings in case of loose compaction which reflect the probability of rapid growth of cracking. .
In contrast, the compressive strains are relatively closed between tests set in case of patch loading but there are clear gaps in case of uniform loadings, in which the gap increased with load increasing as shown in **Fig. 8a** and **Fig. 9a**.

BENDING MOMENT ANALYSIS

The bending moment and the thrust force can be obtained from the experimental strain data of the test pipes. If the strains are known through the thickness, then the bending moment and thrust in the pipe wall can be computed from equations of mechanics of materials.
From the principles of strength of materials, circumferential stresses at the inside and the outside walls of the pipe due to beam bending and axial forces are expressed as follows

$$\sigma_i = \frac{P}{A} + \frac{M * c}{I} \qquad (9)$$

$$\sigma_o = \frac{P}{A} - \frac{M * c}{I} \qquad (10)$$

Where ,
σ_i , σ_o = Circumferential stresses at the inside and outside of the pipe respectively N/m^2.
P = Axial thrust per unit length of the pipe, N/m
A = Cross-sectional area per unit length of the pipe, m^2/m).
M = Bending moment per unit length of the pipe , N-m/m.
c = Distance from the neutral axis to the extreme fiber, m.

I = Moment of inertia of unit length of pipe wall (m^4/m).

According to the principles of theory of elasticity, the circumferential stresses can be related to the strain gage readings from the inner and the outside walls of the pipe as

$$\sigma_i = E_c * \varepsilon_i \qquad (11)$$

$$\sigma_o = E_c * \varepsilon_o \qquad (12)$$

where,

ε_i , ε_o = Circumferential strains at the inside and outside of the pipe wall respectively,

E_c = Young's modulus of pipe materials, N/m2.

Further, when subtracting Eq.(9) from Eq.(10), and substituting the value for circumferential stresses from Eq.(11) and Eq.(12) , the equation to calculate the bending moment is obtained as follows:

$$M = \frac{(\sigma_i - \sigma_o)}{2*c} * I = \frac{(\varepsilon_i - \varepsilon_o)}{2*c} * E_c * I \qquad (13)$$

The sign conventions are, the axial thrust is assumed to be positive in tension and the bending moment is positive when producing tension in the exterior fibers of the pipe wall.
In general the bending moments for loose compaction under similar loading were always greater than that of well compacted soil, the results are shown in **Fig. 12** and **Fig. 13**. At both springlines the bending moments coincided, this is because springline compaction (by hand tamper) is less than the rest to avoid pipe damage, thus these results are expected. At crown and invert, the bending moment of pipe with loose backfill were higher that those of well compacted backfill with gap increasing with increasing applied load.
At invert, there is sharp increase in bending moment of loose backfill in comparison with well compacted backfill under both loading conditions as shown in **Fig. 13a** and **Fig. 13b**,

DEFLECTION ANALYSIS

As expected the vertical deflection of pipe in case of loose backfill are greater than that of well compacted backfill as shown in **Fig. 14** (for Tests No.1 and No.2 there are no available data for deflection). The deflection of Test No.4 (dense compaction) at load increment before failure was 0.14 mm (0.047% as percent from internal diameter of 300mm) while for Test No.3 (loose compaction) the deflection for load increment before failure was 0.185mm. this reflect that the buried strength increased considerably with compaction effort of surround backfill.

SETTLEMENT ANALYSIS

The settlement actually decreased with increasing backfill effort, this concept clearly appears in **Fig. 15**. It clearly indicate that settlement of patch platform was much greater than the settlement of uniform platform due to small contact area of patch platform in comparison with uniform platform. **Fig. 15** indicate that the shape of settlement curve patch loading platform were sharply increased with increasing loading after loading of 19 kN in case of loose backfill while the sharp increase start after loading of 78 kN in case of well compacted backfill.

FAILURE LOAD ANALYSIS

The failure loads of dense compaction tests were higher than that of loose compaction tests, as shown in **Table 7**.

In general a single crack pattern has been observed for all pipes pipe, which appear as longitudinal and approximately straight cracks along the inner faces of invert and crown and outer faces of springlines, namely flexural cracks as shown in **Fig. 16**.

CONCLUSIONS

Based on the analysis results the following conclusions can be drawn:

1. The highest bedding factor obtained is 2.30 for dense backfill and lowest bedding factor is 1.26 for uncompacted backfill.

2.For the same backfill cover and the same bedding conditions as for the bedding factors are considerably different due to different compaction effort which indicate that the pipe-soil strength considerably affected by the backfill compaction although similar conditions of bedding or cover conditions.

3. It is found that the compaction of backfill cover of 30 cm or 60 cm over the pipe crown and also

The backfill below the springline will improve the installation of the concrete pipe and then the strength of pipe-soil system.

4. The collapse loads of pipes under uniform load with 60cm backfill cover; ranged from surface overburden pressures of 131.2 kPa (very loose backfill) to 248.2 kPa (well compacted backfill) overburden pressures which yield that installation quality can increase the strength of pipe-soil system to approximately 50% as an upper limit.

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Table (1): Soil Classifications

Standards	**ASTM D2487**	**AASHTO M145**	**Iraqi S**	**ASTM C1479-07a**
Soil Classification	SP (Poorly graded sand with gravel)	A-1-b	Class D	Gravelly Sand (Category I)

Table (2): Loading Platforms Types

No.	**Loading Platform**	**Simulation of**	**Length, m**	**Width, m**	**Contact area m2**
1	Uniform loading	Earth fill	0.8	1.05	0.840
2	Patch Platform	AASHTO HS20 wheel	0.508	.254	0.129

Table (3): Soil Densities and Compaction Degree for Tests

Test No.	**Wet Density**	**Dry Density**	**Water Content**	**Compaction**
Dense Bedding	22.15	21.18	4.60	98.51
Test_No.1	19.75	19.07	3.56	88.71
Test_No.2	20.78	19.90	4.45	92.54
Test_No.3	20.81	19.23	8.20	89.45
Test_No.4	21.07	19.92	5.75	92.65
Test_No.5	21.27	20.19	5.37	93.90
Test_No.6	19.09	18.37	3.91	85.43

Table (4): Variables of Tests

Test No.	Trench width	Cover of backfill	Backfill Compaction	Loading	Bedding compaction
1	Narrow	30 cm	Loose Compacted	Uniform loading	Compacted
2	Narrow	30 cm	Dense Compacted	Uniform loading	Compacted
3	Wide	60 cm	Loose Compacted	Patch Platform	Compacted
4	Wide	60 cm	Dense Compacted	Patch Platform	Compacted
5	Narrow	60 cm	Dense Compacted	Uniform loading	Compacted
6	Narrow	60 cm	Not Compacted (Rained Soil)	Uniform loading	Compacted
7	Three Edge Bearing Tests				

Table (5): Bedding Factors for Different Tests

Test No.	Cover above crown	Loading Type	Failure Loading		Vertical Stress at crown kN/m^2	Vertical Load kN/m	Bedding Factor, B_d
			kN	kN/m^2			
1	30 cm	Uniform	156.4	186.2	192.29	75.76	1.70
2	30 cm	Uniform	178.7	212.8	218.89	86.24	1.93
3	60 cm	Patch	134	1039.0	155.50	61.27	1.37
4	60 cm	Patch	141.5	1096.8	163.52	64.43	1.44
5	60 cm	Uniform	208.5	248.2	260.38	102.59	2.30
6	60 cm	Uniform	110.2	131.2	143.38	56.49	1.26

Table (6): Tests Used in Backfill Compaction Analysis

Set No.	Dense Compaction Tests	Loose Compaction Tests	Loading Type
1	Test No.2	Test No.1	Uniform
2	Test No.4	Test No.3	Patch

Table (7): Effect of Backfill Compaction on Failure Load

Set No.	Dense Compaction Tests		Loose Compaction Tests		Loading Type
	Test No.	Failure Load	Test No.	Failure Load	
1	Test No.2	212.8 KN/m^2	Test No.1	186.2 KN/m^2	Uniform
2	Test No.4	141.5 KN	Test No.3	134 KN	Patch
3	Test No.5	248.2 KN/m^2	Test No.6	131.2 KN/m^2	Uniform

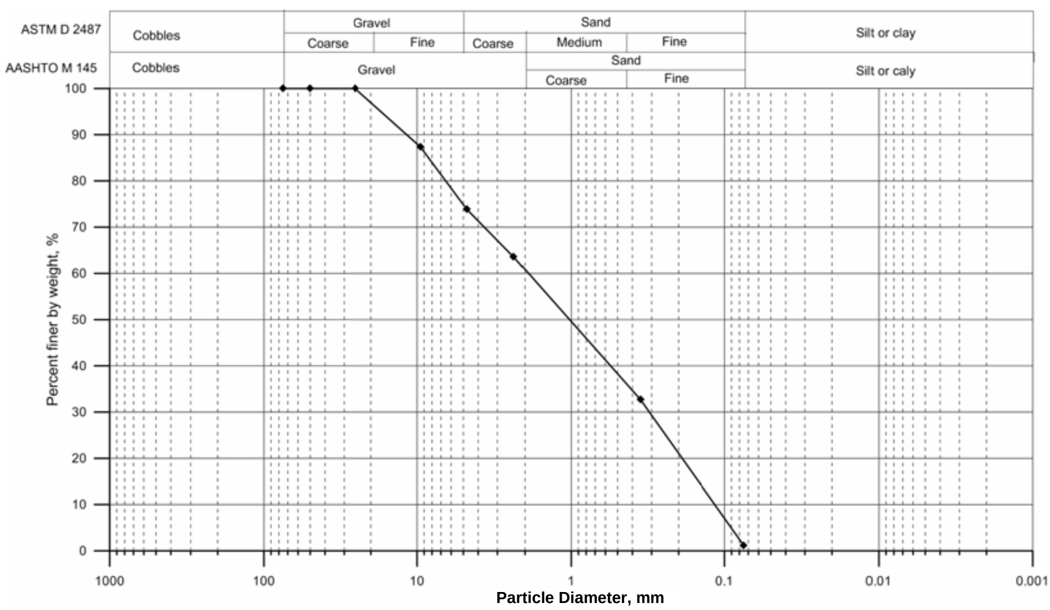


Figure (1): Grain Size Distribution For Backfill Soil

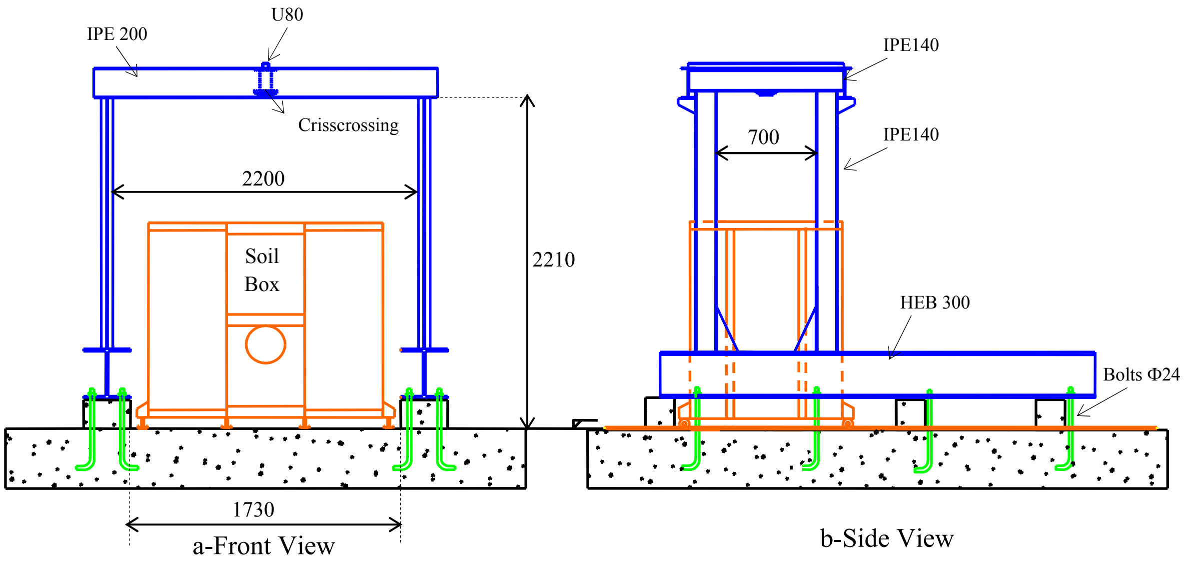


Figure (2): Reaction Frame Dimensions in Millimeters

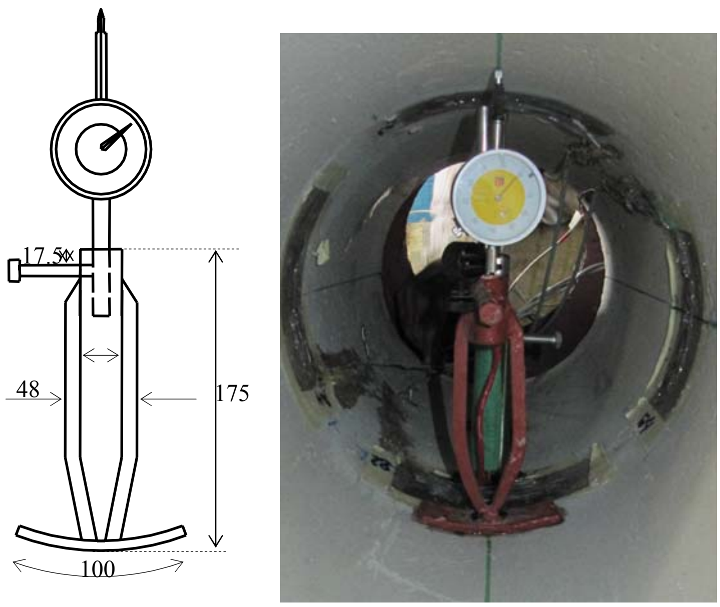


Figure (3): Deflection Dial Gauge and Its Adjustable Stand

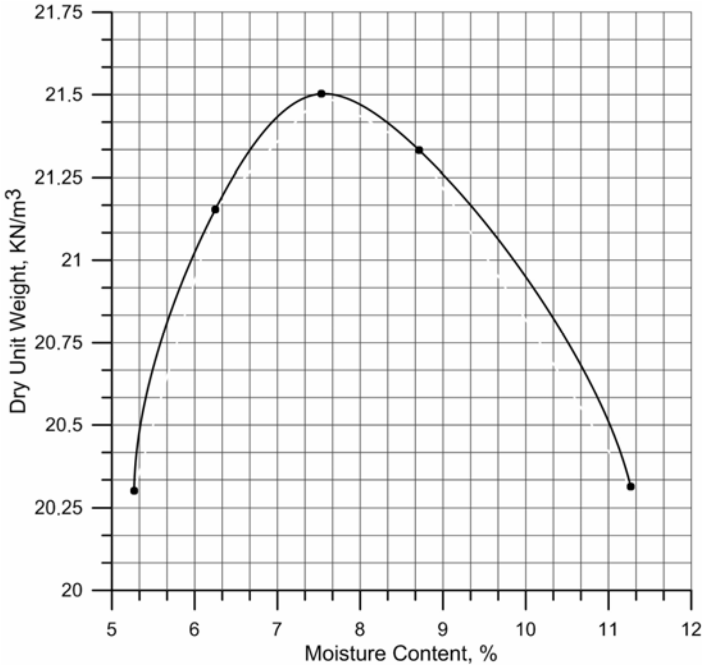


Figure (4): Typical Compaction Curve (Water Content versus Dry Unit Weight)

Figure (5): Steel Hand Tampers

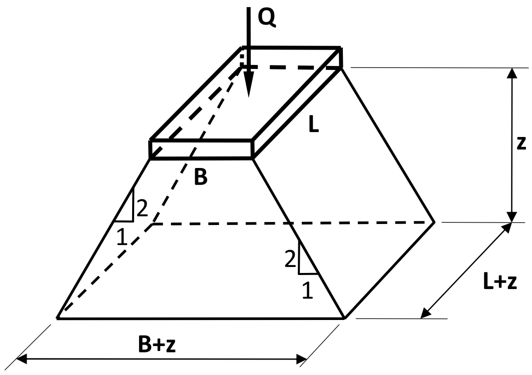


Figure (6) : Approximate Stress Distribution by the 2:1 Method

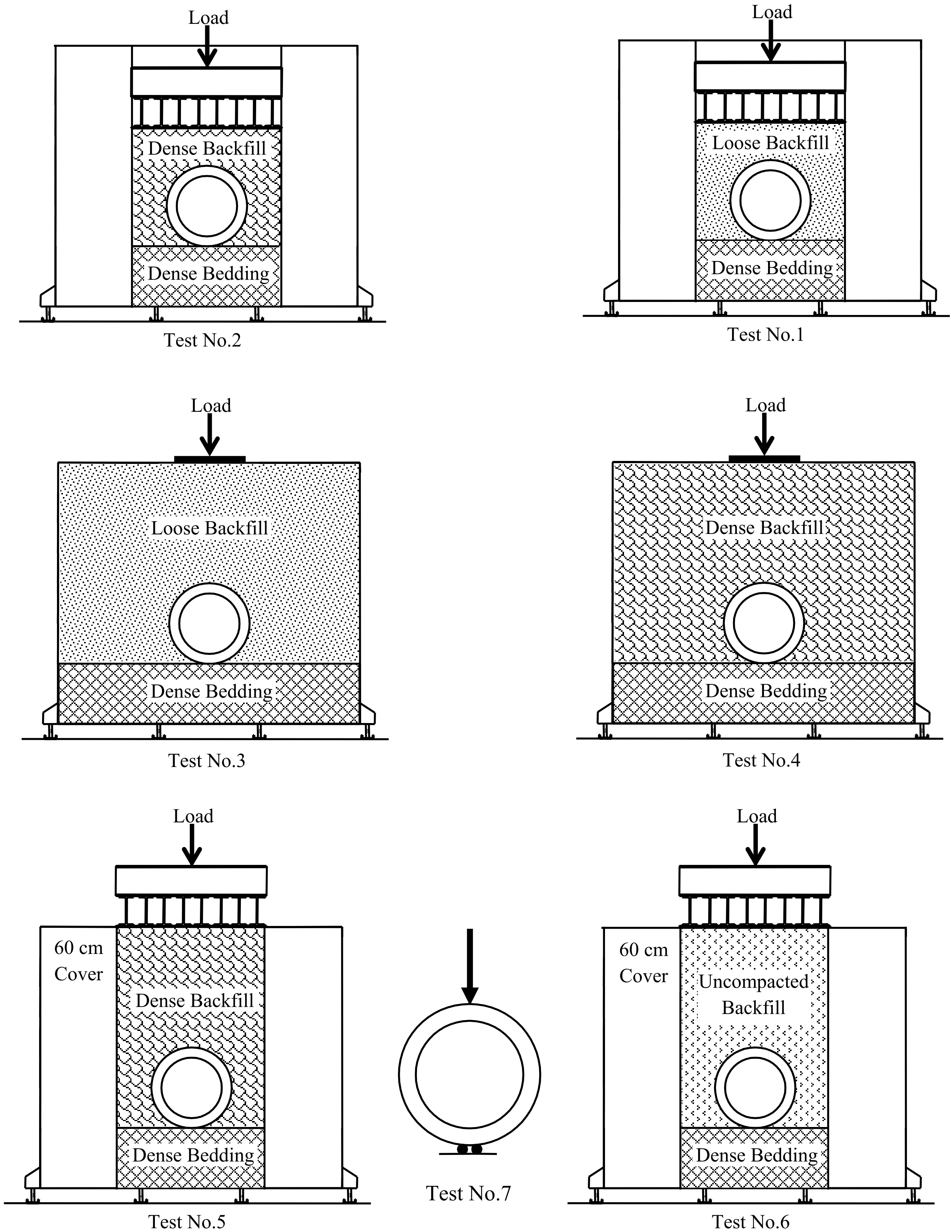
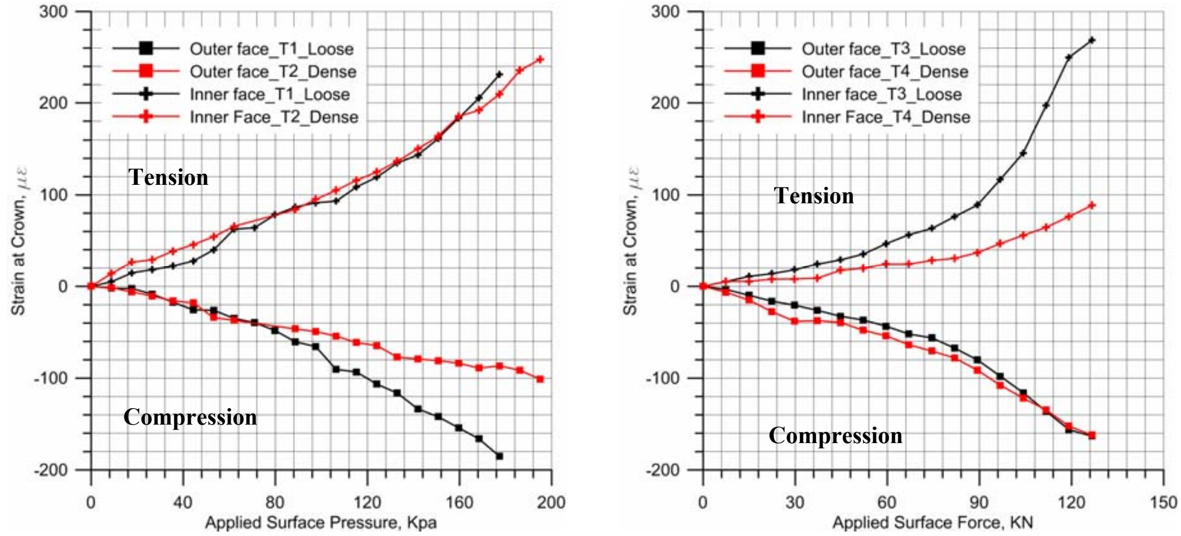
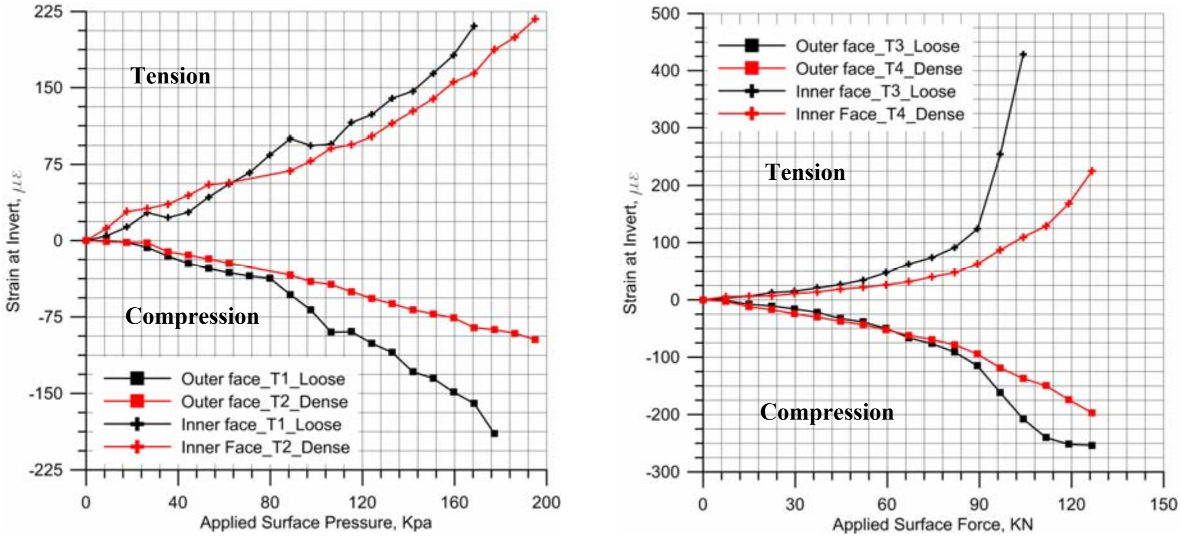


Figure (7): Description of Tests Variables



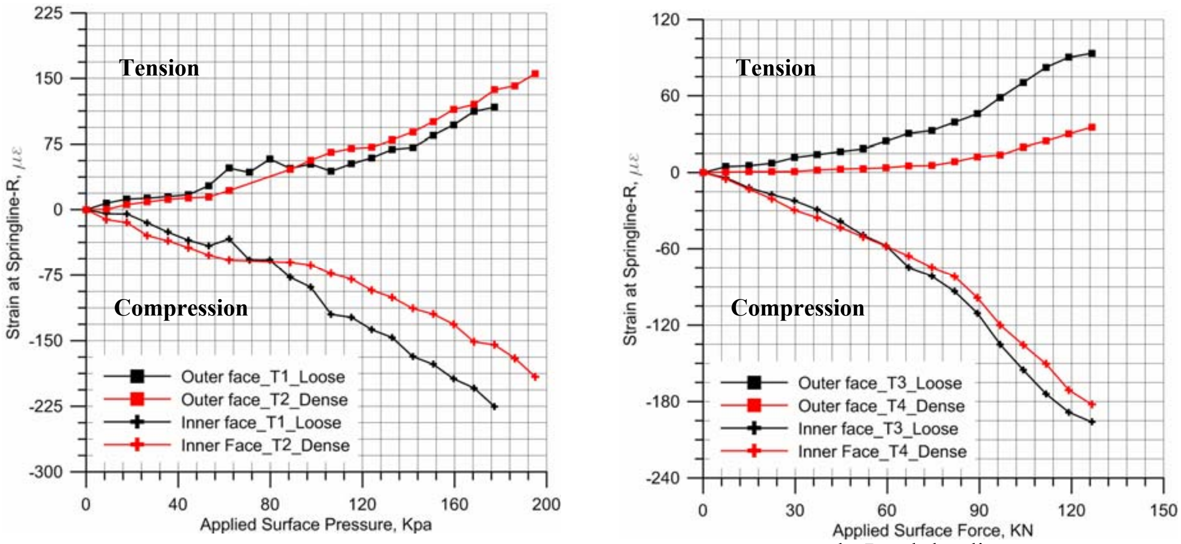
a: Uniform Loading b: Patch Loading

Figure (8): Compaction Effect on Pipes Strains at Crown



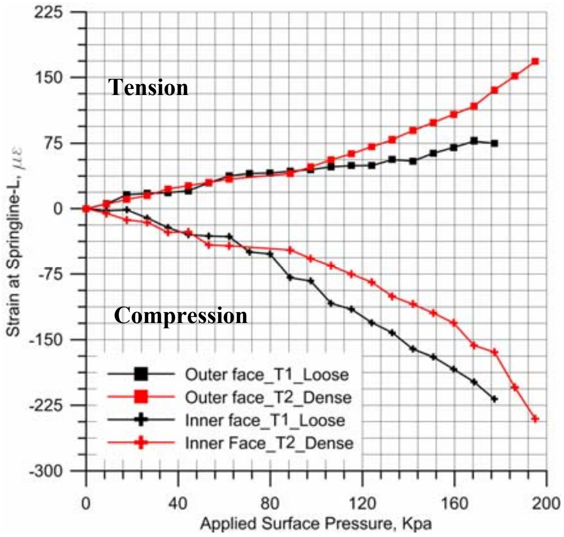
a: Uniform Loading b: Patch Loading

Figure (9): Compaction Effect on Pipes Strains at Invert

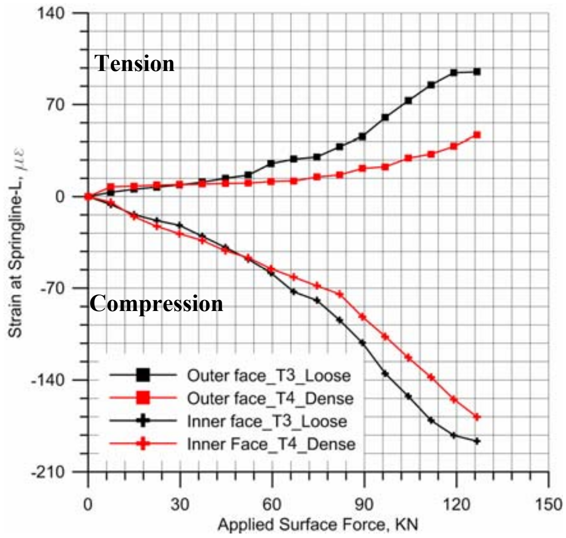


a: Uniform loading b: Patch loading

Figure (10): Compaction Effect on Pipes Strains at Right Springline

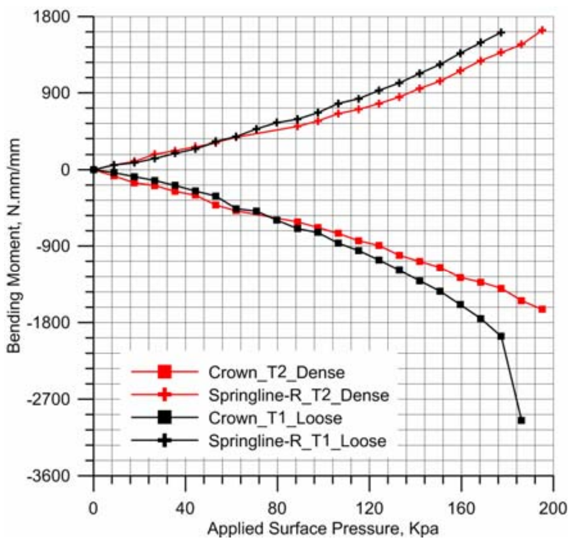


a: Uniform Loading

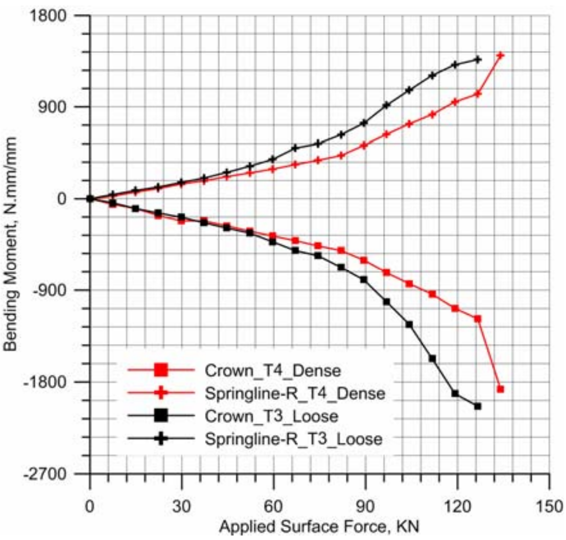


b: Patch Loading

Figure (11): Compaction Effect on Pipes Strains at Left Springline

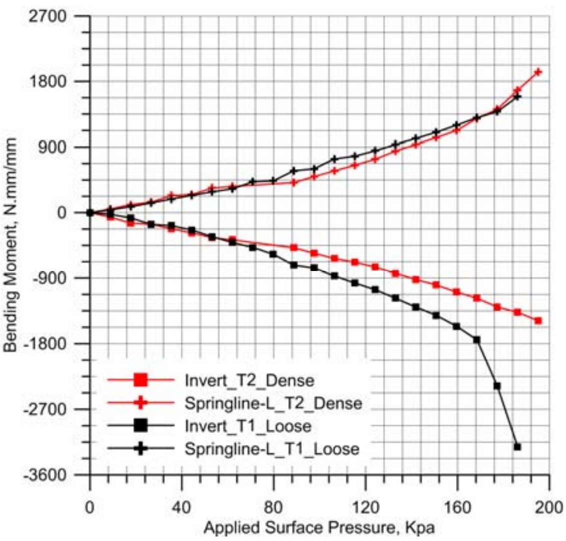


a: Uniform Loading

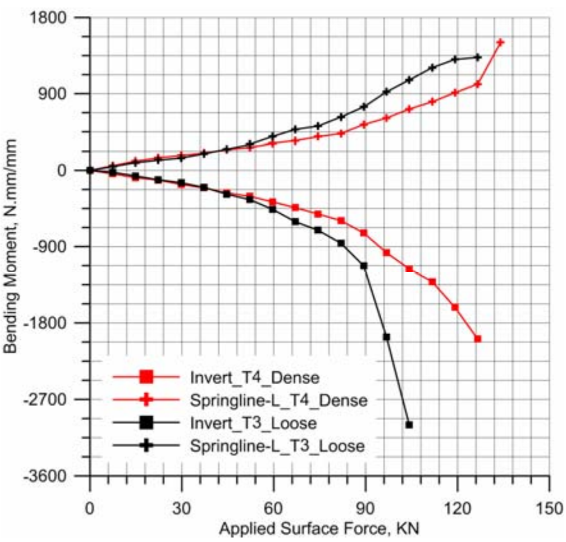


b: Patch Loading

Figure (12): Compaction Effect on Pipes Bending Moments at Crown and Right Springline



a: Uniform Loading



b: Patch Loading

Figure (13): Compaction Effect on Pipes Bending Moments at Invert and Left Springline

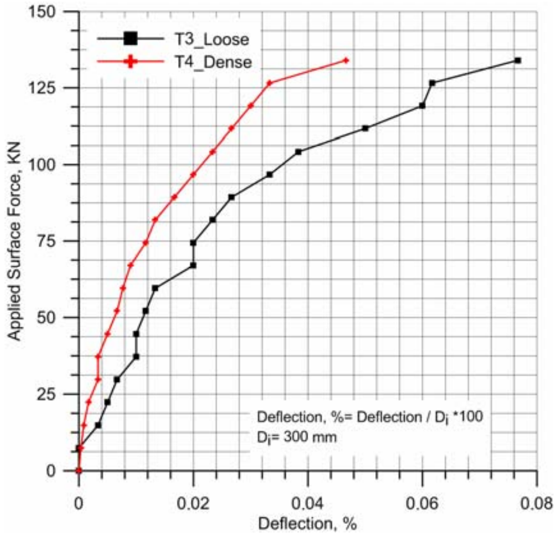
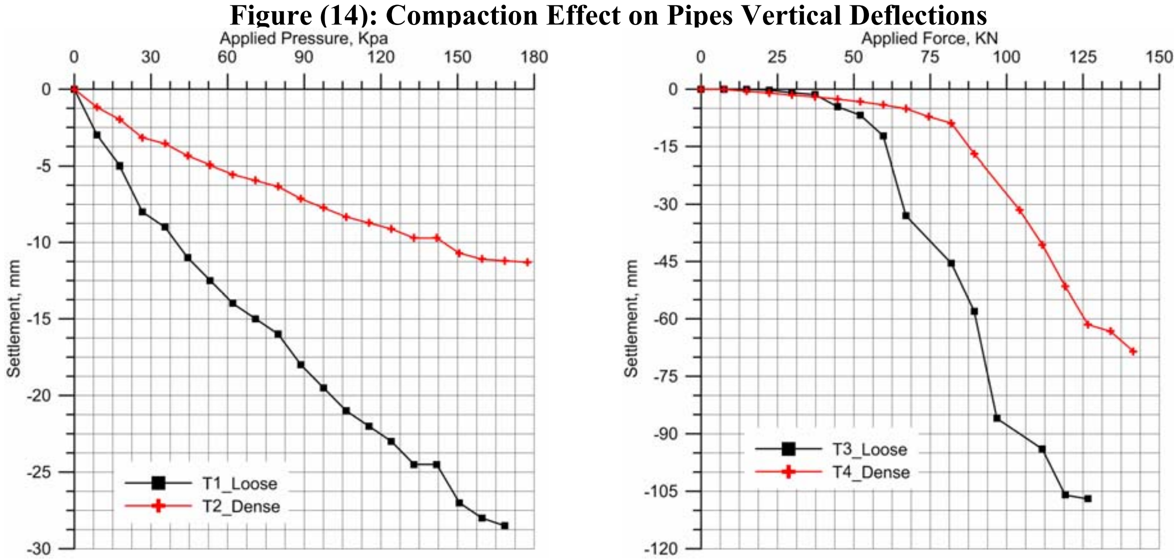


Figure (14): Compaction Effect on Pipes Vertical Deflections



a: Uniform Loading b: Patch Loading

Figure (15): Compaction Effect on Loading Platform Settlement

Figure (16): Cracking Pattern