CHAPTER 10: SUMMARY AND CONCLUSIONS

10.1 OVERVIEW OF THE THESIS

The thesis concerns the design and performance of flexible pipes, buried beneath dry sand backfill in shallow trench conditions and subjected to traffic loading. To understand the complex soil-structure interaction between pipe and soil, a comprehensive study was undertaken of twenty-two buried pipe tests on profiled, spirally-wound uPVC pipes of nominal diameters ranging between 300 and 450 mm. In terms of industry standards, the pipes would be considered to be very flexible.

The majority of the tests were conducted in a laboratory soil box facility with a dry, poorly-graded sand used as the fill material. The tests were commissioned to verify the safe cover heights for construction trafficking of unsurfaced backfills, although three tests employed a simulated protective, thin pavement layer. Half the buried pipe installations were instrumented with soil pressure cells, which provided useful information in the analyses of the performances of the flexible, plastic pipes.

A series of tests were performed on remnants of a 375 mm pipe test to verify assumed values of pipe stiffness, tensile stiffness and tensile strength. The behaviour of the soil used in the pipe installations was studied through a series of triaxial tests, designed to provide information on the steady state line of the sand. A constitutive model was determined for the sand, based largely on the state parameter concept.

The soil constitutive model was incorporated in a new element for initially two- and later, three-dimensional finite element analyses (FEA). The validity of the element was checked against laboratory triaxial tests and plate loading tests of the sand. Thereafter finite element analyses were performed for each buried pipe installation.

Initial void ratios, based on density indices derived from soil penetration testing, were assigned to the various zones of the constructed soil (bedding, surround and backfill). An initial stress state was implemented and loads were applied by enforcing incremental displacement of the rigid plate, or by applying forces to the nodes of the flexible loading plate used in the simulated pavement tests. In the two-dimensional analyses, the pipe was represented by beam-column elements with properties developed from manufacturers' specifications, Australian Standards and a limited testing program on 375 mm diameter pipe and coupons taken from the pipe.

Where the soil box was used, it was found to be of advantage to apply interface elements at the joint, with properties derived from interface shear box tests. Where the trench had been excavated in the ground, the natural soil was modelled as undrained, isotropic and homogeneous clay soil of Young's modulus, appropriate to test results on soil taken horizontally from the sidewalls.

In the course of time, a three-dimensional version of the element became available and the buried pipe analyses were re-analysed. The 3D FEA was limited to some extent by choice of elements and processing time. A 3-D interface element was not available, so analyses were undertaken with completely rough sidewall conditions (2D analyses indicated that the load-deflection response of analyses with an interface joint were slightly overestimated by analyses with a perfectly rough wall, but were significantly underestimated by FEA incorporating a perfectly smooth wall).

An 8-noded brick element was available, whereas 15-noded triangular elements were used in the 2D analyses. The pipe was modelled as a uniform thickness, elastic material of shear and flexural stiffness equivalent to the particular pipe cross-section and diameter. The pipe thickness was accordingly adjusted in the numerical model. Checks were made on the pipe properties by modelling the pipe stiffness tests.

Although some difficulty was encountered matching the large displacements of the loading plate in most of the tests, three-dimensional finite element modelling provided useful insights into the inadequacies of the laboratory-testing program. In particular, the assumption of a rigid restraint of the sidewall was shown to be unreliable. Indeed, this research has emphasised the need to more carefully consider the boundary conditions when establishing a buried pipe testing facility. An example of the care required is the recent design of a testing facility for small diameter pipes under soil loading, as described by Brachman, Moore and Rowe (2000).

10.2 SUMMARY OF FINDINGS

The major findings of the thesis have been grouped into six categories as shown in the following discussion.

10.2.1 The Buried Pipe Tests

a) Sufficient cover height is essential in protecting the pipe from trafficking of the backfill surface.

b) In terms of the response of the pipes to the measured soil pressure, 150 mm above the pipe crown, vertical diametric strain of the pipe appeared to be largely independent of pipe stiffness for the range of pipe stiffness investigated in this test series.

c) The rigidity of the sidewall boundary condition for the laboratory soil box tests was called into question after comparing the relative performance of these pipes with those pipes tested in the field, after being buried in trenches cut from stiff clay.

10.2.2 Structural Properties of the Pipe

a) The manufacturer's value of pipe stiffness for the 375 mm diameter pipe was shown to be slightly conservative.

b) Conversely, tensile testing of coupon specimens from the same pipe revealed that current recommendations for Young's modulus of the pipe material exaggerated the tensile modulus by a factor of three or more.

10.2.3 Characterization of the Sand Fill

a) The sand in the study was a poorly-graded, angular sand, with some mica and feldspar particles.

b) The critical state shear strength, ϕ'_{cv}, was estimated to be 31.5°.

c) A critical state line (CSL) was chosen based on constant volume triaxial testing; the CSL was defined by values of voids ratio, Γ, of 1.07 for a reference effective mean stress of 1 kPa, and a gradient, λ ,of 0.055.

d) The parameter, A, (Collins, Pender and Yan, 1992) needed to define the relationship between the difference in current and critical state shear strength friction angles, ($\phi' - \phi_{cv}'$), and state parameter, ξ (Been and Jefferies, 1985), was determined to be 0.98.

e) Bolton's (1986) equation for the relationship between dilation angle and the difference in peak and critical state shear strength friction angles in plane strain, was found to be applicable to triaxial compression data.

f) Davis' flow rule (1969) for triaxial conditions, when applied to the experimental data, yielded estimates of dilation angle, which were found to correlate well against both Bolton's dilation index (based on Q = 9) and state parameter.

g) The triaxial test data agreed generally with Rowe's (1962) stress-dilatancy expression and the assumption that ϕ'_f is equal to ϕ'_{cv}.

10.2.4 Constitutive Model for the Sand

a) From the laboratory soil investigation, an elasto-plastic, isotropic soil model has been developed, which incorporates the state parameter concept. Yield is defined by the Mohr-Coulomb criterion. Post-yield deformations are based on a non-associated flow rule (Davis, 1969), and are defined by the state parameter concept of Been and Jeffries (1985), and Bolton's (1986) equation for dilation of sand in plane strain. Both the rate of dilation and the effective friction angle of the soil in the model vary with the current void ratio and effective mean stress, i.e. the state parameter.

b) The soil constitutive model incorporated non-linear elasticity with both the shear and bulk modulus not directly linked. The non-linearity requires eight material constants. A further five material constants are required to implement the state parameter model.

c) A set of material constants for the soil was established, based on single element modelling of the triaxial tests on the sand. The development of dilation with increase of stress was modelled well by adoption of these constants.

d) The soil constitutive model has been incorporated into finite element program AFENA (Carter and Balaam, 1995) for two and three-dimensional analyses.

e) The application of the constitutive model has been demonstrated for the finite element analysis of a plate-loading test in the sand, confined within a drum.

10.2.5 FEA of the Buried Pipe Tests

a) Two-dimensional FEA could not adequately model the observed behaviour of the soil-pipe systems.

b) Although the 3D analyses improved the predictions of behaviour when compared with the 2D analyses, the assumption of a rigid sidewall with either a perfectly rough or perfectly smooth boundary condition could not adequately simulate the observed load-deflection behaviour of the pipes.

c) The side boundary conditions were relaxed to allow possible distortion of the wall with external loading of the backfill. The wall was assigned a flexural stiffness commensurate with the braced formwork that formed the sides of the laboratory soil box. Boundary restraints were reduced also. The predicted behaviour of the pipe-soil system then began to provide better simulation of the observed behaviour. In particular, a better match was seen with measured horizontal diametric pipe strains.

d) The assignment of an appropriate, single stiffness of the sidewall was not obvious, upon review of the FEA solutions for all laboratory tests on the pipe. Increasing or decreasing the wall stiffness by a factor of two sometimes improved the matching of the experimental data in certain cases.

e) The predicted moments that may be developed in these particularly flexible pipes are relatively insignificant, however the hoop forces in the pipe ring can be appreciable, although still remaining within the capacity of the pipe material.

10.2.6 Impacts on Flexible Buried Pipe Design

a) For a non-paved and trafficked backfill surface, it is very difficult to predict reliably the vertical soil pressure above the pipe. It has been shown that the vertical stress below the centre of a loaded rectangular area is under-predicted by the theory of elasticity.

b) The predicted distribution of vertical soil stress by FEA was non-uniform, with the bulk of the stress concentrated closer to the pipe crown.

c) Based on the FEA of the four field tests, a simple empirical expression was derived between the average applied surface pressure and the average vertical stress, 150 mm above the pipe and extending across one pipe diameter. The relationship requires the geometry of the installation and the ratio of the Young’s modulus of the backfill soil to the Young’s modulus of the natural soil forming the trench walls.

d) Based on the backanalysis of the buried pipe test data and pressure cell measurements, currently recommended values of the “modulus of soil reaction”, E′, seemed to be adequate for design purposes for poorly graded sand.

e) Good design estimates of observed vertical pipe deflection in the field tests could be made with the Iowa equation over the early stages of external loading with current recommendations for E′ and the empirically-derived average pressure above the pipe.

f) Hoeg’s theory produced more conservative design estimates, which however extended the useful range of application and provided good estimates of the lateral pipe response.

g) The AASHTO design approach gave almost identical design estimates to those of Hoeg, when the vertical arching factor was applied in the estimation of the flexural deformation component.

10.3 CONCLUDING STATEMENT

This thesis contains information on the development of a soil model, which allows dilation at a realistic rate as the soil yields (non-associated flow). The state parameter concept has been adopted in this model. The adopted elastic behavior of the soil is non-linear and depends on stress state and the small strain shear modulus of the soil, which changes with stress state.

The constitutive model was developed in a single element spreadsheet, verified against triaxial test data and subsequently adopted in AFENA.

Finite element analyses of the buried pipe installations were conducted with this soil model. The pipe properties were investigated and suitable assumptions for simulating the pipe behaviour were made. Appropriate boundary conditions were applied, based on the assumption of a rigid sidewall in the case of the laboratory soil box tests.

Two-dimensional analyses were found wanting as the applied strip loading could not represent the geometry of the loaded area. A 3D element was developed subsequently. Although 3D analyses of the buried pipe installations were an improvement on the 2D approach, the experimental data were still not adequately modelled in all cases, especially for the pipes tested in the laboratory soil box. The stiffness of the soil box was subsequently investigated. The inclusion of a non-compliant sidewall boundary in the 3D model significantly improved the agreement with the observed load-deflection behaviour of the system. It may be concluded that the original laboratory soil box testing was too severe a test, when compared with installations buried in natural soil.

The knowledge gained from the buried pipe testing program, subsequent tests on the soil and the pipe, as well as the FEA studies of the buried pipe tests, led to the preliminary recommendation of simple guidelines for the design of buried flexible pipes subjected to traffic loading. A preliminary relationship has been proposed for the determination of the average soil pressure developed above the pipe crown, which can then be used with recommended values of modulus of soil reaction for sand and available expressions for pipe deflection estimates.

10.4 RECOMMENDATIONS FOR FUTURE RESEARCH

The scope of this research has been limited chiefly by:

- the soil of the installations (poorly graded sand)
- the material of the pipes (uPVC)
- pipe diameters
- the geometry of the trenches
- the geometry of the loaded area
- inadequate sidewall support in the laboratory test series
- static loading of the backfill
- neglect of pipe deformations due to the backfilling process

The pipe diameters in the test series covered a reasonable range of commonly used pipe sizes and accordingly the trench geometries were constructed to relatively common dimensions. The dimensions of the loaded area may change with design wheel load and tyre pressure, and is worth further consideration. The material of the pipes is an important factor, but less so for short-term traffic loading than for long-term embankment loading.

The preliminary relationship proposed for the average soil pressure developed above the pipe crown should be appraised for a wide range of installation and loading geometries, as well as pipe materials and sizes, but more importantly over a wider range of granular soils.

Further investigation is needed regarding the general adequacy of laboratory soil box testing. A sidewall system needs to be developed to simulate a linear-elastic natural soil, or alternatively the sidewalls should be perfectly rigid.

Granular backfills, surrounds and bedding are highly desirable around flexible pipes. However, a shortage of sand and the economic demands on industry have sometimes resulted in excavated material for the trench being compacted as the backfill above the pipe. Consequently, partly saturated silts and clayey soil have been used for this purpose. Therefore the influence of such materials on distributions of stresses above a pipe, and hence pipe deflections, requires further investigation. The role of pore water pressures is of greater importance in fine-grained soils and should be included in the study through investigation of combinations of various levels of soil compaction and moisture contents.

As the majority of the tests concerned the influence of construction traffic loading, the number of passes of wheel loads will be moderate and it could be argued that one pass is a sufficient test for a pipe installation with an unprotected backfill. Nonetheless, a transient rolling load with associated shearing action can impact differently on soil than a static load. It may be of value to compare static loading and wheel loading from a moving truck on identical pipe installations.

Finally, it was recognized in Chapter 2 that flexible pipes deform during placement and subsequent compaction of the fill. This issue has been ignored in this thesis, with the datum for pipe deformations being taken to be the commencement of testing on the finished installation. Diametric strains were then calculated relative to the nominal diameter of the pipe. As the pipe in shallow installations during soil placement can reduce in diameter laterally and expand vertically, installation deformations deserve far more attention by researchers.

10.5 REFERENCE TO THE CHAPTER

Brachman, R. W. I., Moore, I. D. and Rowe, R. K. (2000). The design of a laboratory facility for evaluating the structural response of small-diameter buried pipes. Canadian Geotech. J, V37, No.2, pp 281-295.