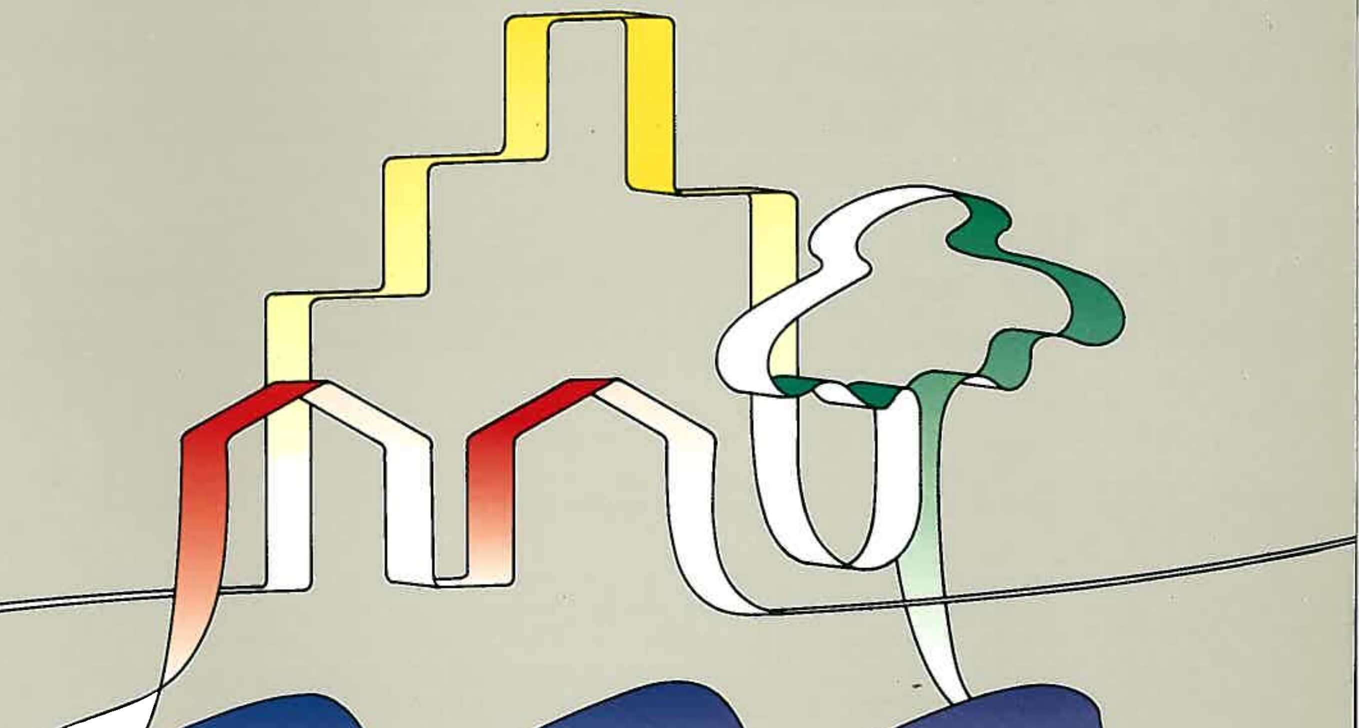
Urban Water Research Association of Australia

GLASS REINFORCED PLASTIC BORE CASING FOR LARGE DIAMETER AND DEEP BORES



Research Report No. 12

URBAN WATER RESEARCH ASSOCIATION OF AUSTRALIA

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URBAN WATER RESEARCH ASSOCIATION
OF AUSTRALIA

GLASS REINFORCED PLASTIC BORE CASING FOR LARGE DIAMETER AND DEEP BORES

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FOREWORD

This report describes UWRAA Research Project No AM-1, `Development of Glass Reinforced Bore Casing' which was undertaken over the two year period July 1987 to June 1989.

Organisational responsibility for the project was as follows:

Sponsoring Authority	:	Water Authority of Western Australia
Project Officer	:	Mr R. Lilly Water Authority of Western Australia
Principal Researcher	:	Mr R. Bowyer Water Authority of Western Australia

The project was funded jointly by the Urban Water Research Association of Australia, on the advice of the Association's Research Advisory Committee, and by the Water Authority of Western Australia.

SYNOPSIS

In Western Australia mild steel pipe used for bore casing lasts for as little as four years. On average the collapse pressure strength of steel casing is reduced through corrosion by approximately 10% per year.

Production bores used by the Water Authority of W.A. for Perth's water supply range from 250 ND by 40 metres deep costing about $40 000 to replace, to 500 ND by 1100 metres deep costing about $900 000 to replace.

The alternative materials to mild steel for bores of this size are fibre reinforced plastics (FRP) and stainless steel. Stainless steel is approximately twice as expensive to purchase as GRP (glass fibre reinforced plastic) and twice as expensive to install due to factors such as the equipment required to handle the weight and the jointing costs.

The joints on the previously available GRP casing, however, were unreliable and unsafe to use in deeper pressure grouted bores particularly of larger diameter. Many construction failures occurred for which the drilling contractor was unfairly expected to assume responsibility.

The aim of this project was to improve the reliability and capacity of these bore casing joints to enable the safe use of GRP casing to at least 400 ND and 500 metres depth.

Although at the conclusion of this successful project development of GRP casing to its full potential is not complete, the aims of developing reliability and extending the size and depth range have been met.

Tables 1 and 2 give the recommended maximum depth ratings for the newly developed GRP casing joints for pressure grouted bores using the test results from developments to December 1989.

The capacities and depths given in Tables 1 and 2 have been calculated using a load factor of 1.5.

TABLE 1

PRESSURE GROUTED CASING USING A GROUTING SHOE

DEPTH RATINGS FOR GRP CASING JOINTS AND GROUT S.G.'s

CASING NOMINAL DIAMETER	JOINT DESIGN CAPACITY TENSILE (KN)	PRESSURE (kPa)	MAXIMUM DEPTH RATING (m) GROUT AVERAGE S.G. 1.6	1.35	1.3	1.25
250	200	3970	610	1050	1220	1470
300	240	3300	510	880	1030	1240
400	320	2600	395	690	810	970
500	400	2000	310	530	620	750
600	475	1670	260	440	520	620

TABLE 2

PRESSURE GROUTED CASING WITHOUT A GROUTING SHOE

DEPTH RATINGS FOR GRP CASING JOINTS AND GROUT S.G.'s

CASING NOMINAL DIAMETER	JOINT DESIGN CAPACITY TENSILE (KN)	PRESSURE (kPa)	MAXIMUM DEPTH RATING (m) GROUT AVERAGE S.G. 1.6	1.35	1.3	1.25
All diameters	-	4000	620	1060	1230	1480

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1 INTRODUCTION

GRP is approximately twice as expensive to purchase as mild steel but cheaper overall due to it's much longer life.

Replacement of mild steel bore casing with GRP for all future large diameter deep bores would, in 10 to 20 years, save the Water Authority of W.A. approximately $2 million p.a. in replacement bore costs.

GRP bore casing to be acceptable, however, must have a reliable capacity to withstand all the stresses that may be anticipated during bore construction and operation.

These stresses are:

(a) External pressures due to strata such as swelling clays and shales, aquifer heads and construction processes such as gravel packing, backfilling, pressure grouting and bore development.

(b) Internal pressures during construction due to pressure grouting and surge development.

(c) Joint tensile stresses resulting from pressure grouting, construction and casing recovery.

The design requirements for the use of GRP bore casing vary according to whether the bore is ungrouted, pressure grouted using a grout shoe or pressure grouted without a grout shoe.

In the case of an ungrouted bore the required joint pressure and tensile capacities are limited to drawdown pressure heads and construction tensile loads respectively. The external collapse pressure capacity of the casing, however, needs to be adequate to resist soil pressures from active soils such as expanding clays and shales and surge loadings from sand packing and strata collapse.

Deep bores are normally pressure grouted to prevent contamination of aquifers from the surface or other intersected aquifers. Pressure grouting reduces the soil pressure loading on the casing but increases the required pressure capacity of the joints to the design maximum grouting pressure.

Pressure grouting is normally carried out using a grouting shoe containing a non return valve to allow the internal pressure to be released and drill string equipment to be removed immediately grouting is completed. Use of a grouting shoe, however, requires the joints to have pressure and tension capacities capable of meeting the loads from the maximum possible grouting pressure. The maximum possible pressure would normally result from a blocked or restricted grouting shoe or annulus and would be equivalent to the pump or relief valve capacities.

Pressure grouting without a grout shoe eliminates the need for the additional joint tensile capacity but increases the required joint pressure capacity further to maximise the depth capacity of the casing.

Large tension forces are only generated in the casing and joints when a constriction or blockage occurs in the grout shoe or annulus or when the casing is placed in tension during construction or in an attempt to remove the casing. With casing sizes of 250 ND and above, the least controllable of these situations will be generated by pump pressure against a blockage or constriction.

A constriction or blockage of the shoe valve or annulus can almost instantaneously force the pressure up to the capacity of the pump or pressure relief setting. It is therefore necessary to consider the pressure capacity of the equipment that would normally be used for grouting shallow bores and the maximum design pressure for deeper bores in the assessment of the required joint pressure capacity. The pumping equipment is normally direct displacement duplex pumps of pressure capacity less than 5000 kPa. However, larger pressure pumping equipment is available and a pressure limiting valve is required if higher pressure equipment is used.

The tensile strength and internal and external pressure capacities of the casing itself can be increased to any required level by increasing the structural wall thickness. The capacity of the joints to handle these pressures and stresses, however, is limited by the joint design.

The pressure and tensile strength capacities of the previous joint design were unreliable and inadequate for pressure grouted bores and particularly so for larger diameters and depths.

The aim of this project was to develope a casing joint with capacity satisfactory for reliable use with pressure grouting to at least 500 metres.

2 DEVELOPMENT AND TESTING

The objective of this research project was to develop glass reinforced plastic (GRP) bore casing suitable for use in pressure grouted large diameter and deep bores to at least 400 mm diameter and 500 metres depth.

Development and testing work has generally been carried out on 250 ND GRP casing as this is the smallest most frequently purchased size used by the Water Authority of W.A. and is therefore the most economical to use. Tests have been conducted on larger sizes as the opportunity has occurred.

The bore casing design requirements for a pressure grouted 250 ND bore of 500 metres depth meeting the minimum aims of this project are given in Table 3. These values include a load factor of 1.5.

TABLE 3

DESIGN REQUIREMENTS FOR 250ND GRP CASING

	(a)	(b)
1. Joint leak pressure capacity:	4905 kPa	2450 kPa
2. Casing maximum external pressure:	11500 kPa	11500 kPa
3. Casing joint tensile capacity:	255 kN	126 kN

(a) Cement grout S.G. 1.6.

(b) Cement/bentonite grout with pure cement tail of S.G. 1.7.

2.1 JOINT PRESSURE CAPACITY

2.1.1 General

The annulus between the bore casing and the strata of the hole is normally pressure grouted to seal off and protect the aquifers above the screened aquifer section.

The pressure required to grout a bore is equal to the difference in head pressure between the grout on the outside and the follow fluid on the inside plus pumping pressures to overcome inertia and friction heads or possibly a blockage.

In practice, the grout is often supplied in batches with delays in between which can be sufficient for settling and gelling of the cement to occur.

The pressure required to overcome friction, settling and inertia is significant and has been measured at 9% above the theoretical circulation pressure.

The highest pressure normally occurs during the final stage of the bore pressure grouting process which requires water to be pressurised inside the casing to force the last of the grout out of the casing and up the outside of the casing leaving water inside the casing and grout on the outside of the casing.

A leaking joint will allow this pressurised water to circulate through the joint and grout to remain in the inside of the casing to the level of the leaking joint causing possibly a total loss of the bore.

2.1.2 Initial Testing

To ensure that joint leakage will not occur during pressure grouting, pressure tests have been routinely carried out by the Water Authority of W.A. on GRP bore casing purchased by the Authority for use in pressure grouted bores.

The pressure capacity has been tested by pressurising mated joints internally with water until leakage or maximum design pressure occurred. This testing procedure simulates the normal condition where water is used as follow fluid to back pressure grout circulation.

The equipment used is shown in Fig. 1. and consists of two inflatable packers separated by a steel link section adequate to resist the tensile forces generated and a water pressure pump and accurate pressure gauge.

The packers are inserted into a mated joint to overlap the joint and inflated to allow the joint to be internally pressurised with water.

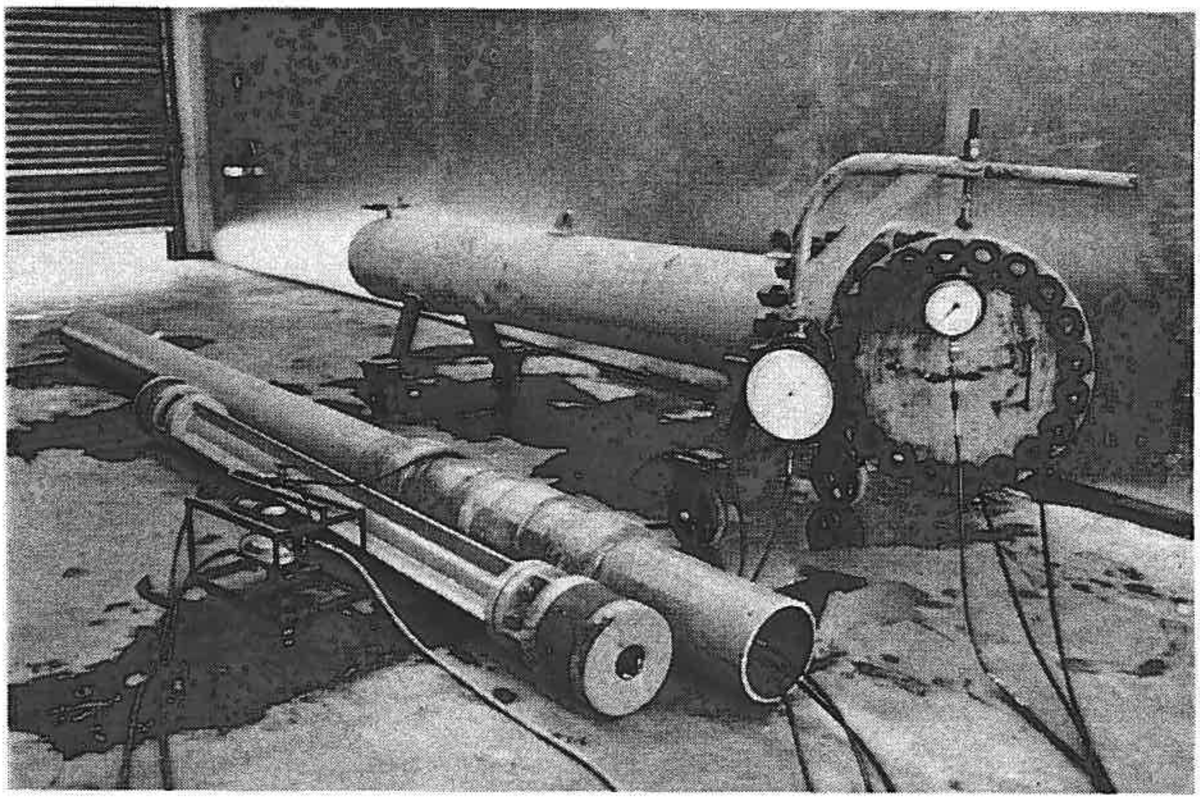

Fig. 1. Joint pressure testing equipment.

2.1.3 Test Results

In the original joint pressure tests carried out on unmodified joints, approximately 10% of all joints tested failed to reach the bore design pressure. A significant number of failures also resulted from damage to the 'O' rings from pinching during mating of the joints.

These tests established that a significant number of joints would leak without pressuring the joint and many others leaked at below design maximum grouting pressures.

Premature joint leakage was caused by:

- damage to the 'O' ring
- unevenness of the female joint sealing surface
- inadequate 'O' ring compression

The joint seal is established by an 'O' ring held in a groove on the male or spigot joint sealing against the inside surface of the female or bell joint.

The joint configuration can be seen in the photograph at Fig. 2. which shows both the male and female components of the joint and also in the drawing of the new modified joint profile at Figure 6.

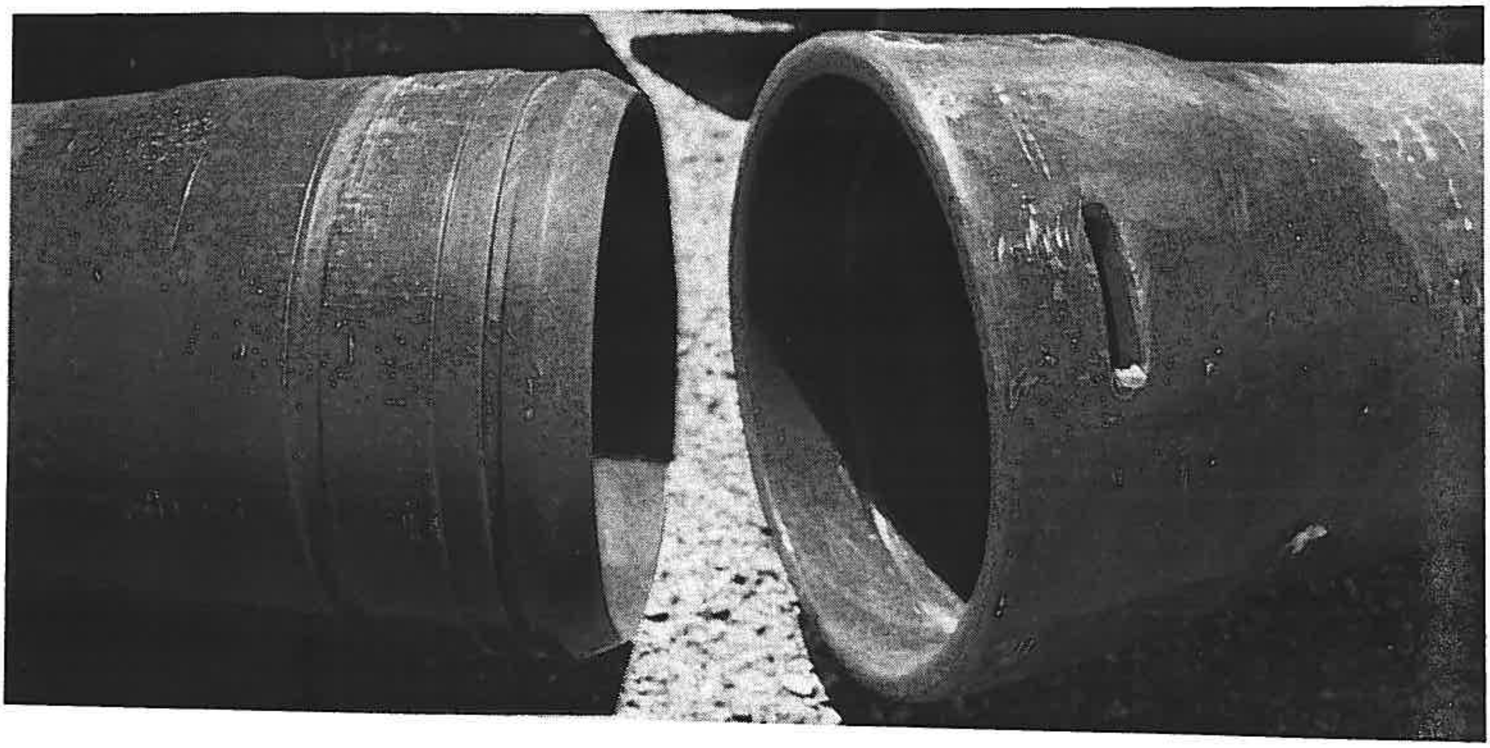

Fig. 2. Joint configuration.

2.1.4 Developments to Overcome Joint Weaknesses

2.1.4.1 Damage to 'O' rings

Large, heavy pipe joints are difficult to mate gently and therefore damage to the `O' ring being squeezed into the female joint occurred frequently. The solution was to redesign the joint so that the `O' ring would slide readily into the entrance of the female joint. This was achieved with a stepped joint. A drawing of the new joint profile is at Figure 6.

2.1.4.2 Unevenness of the female joint sealing surface

This was found to be due to poor fabrication practices such as wrappings used on the female mould or inadequate care in placing the resin-rich tissue reinforced layer. The pipe manufacturing specification (Water Authority of WA, 1990) has been modified to improve the standard of fabrication.

2.1.4.3 Inadequate 'O' ring compression

This was found to be caused by too great a tolerance between the male and female joint diameters, poor fabrication techniques such as using release wrappings on the female mould or poor machining control on the male joint. The specification has been modified to improve the standard of fabrication.

2.1.4.4 New joint test results

Pressure tests on approximately 100 new profile joints have given results of 6 000 kPa or greater with 90% of joints exceeding 7 000 kPa.

2.1.4.5 Quality control testing

As a further assurance the Water Authority now requires all joints to be pressure tested by the manufacturer to the design pressure before delivery.

2.2 CASING EXTERNAL PRESSURE CAPACITY

2.2.1 General

External pressures on the casing may result from strata such as swelling clays and shales, gravel pack or backfill, aquifer heads, pressure grouting and bore development. The maximum design pressure will depend on which of these factors apply. A minimum design pressure of 23 kPa per metre depth is recommended where gravel packing or backfill plus surge development are to be employed.

The external pressure capacity of GRP bore casing is proportional to the cube of the ratio of structural wall thickness over the structural wall OD.

However, the structural reinforcement density and wall thickness are not uniform from pipe to pipe or even along each pipe and the effective minimum structural wall thickness is very difficult to measure. A large number of collapse pressure tests are therefore required to determine the coefficients of a safe envelope curve for each pipe nominal diameter size and measured wall thickness.

2.2.2 Testing

Quality control testing of casings supplied in each contract to the Water Authority of W.A. (in accordance with ANSI/ASTM D2924-79) has allowed a series of minimum yield pressure envelope curves to be derived for 200, 250 and 300 nominal diameter casing sizes. See Figure 9. 250ND envelope curve.

A series of curves for GRP casing total wall thickness (TWT) versus depth rating has been derived from these curves. See Figure 3.

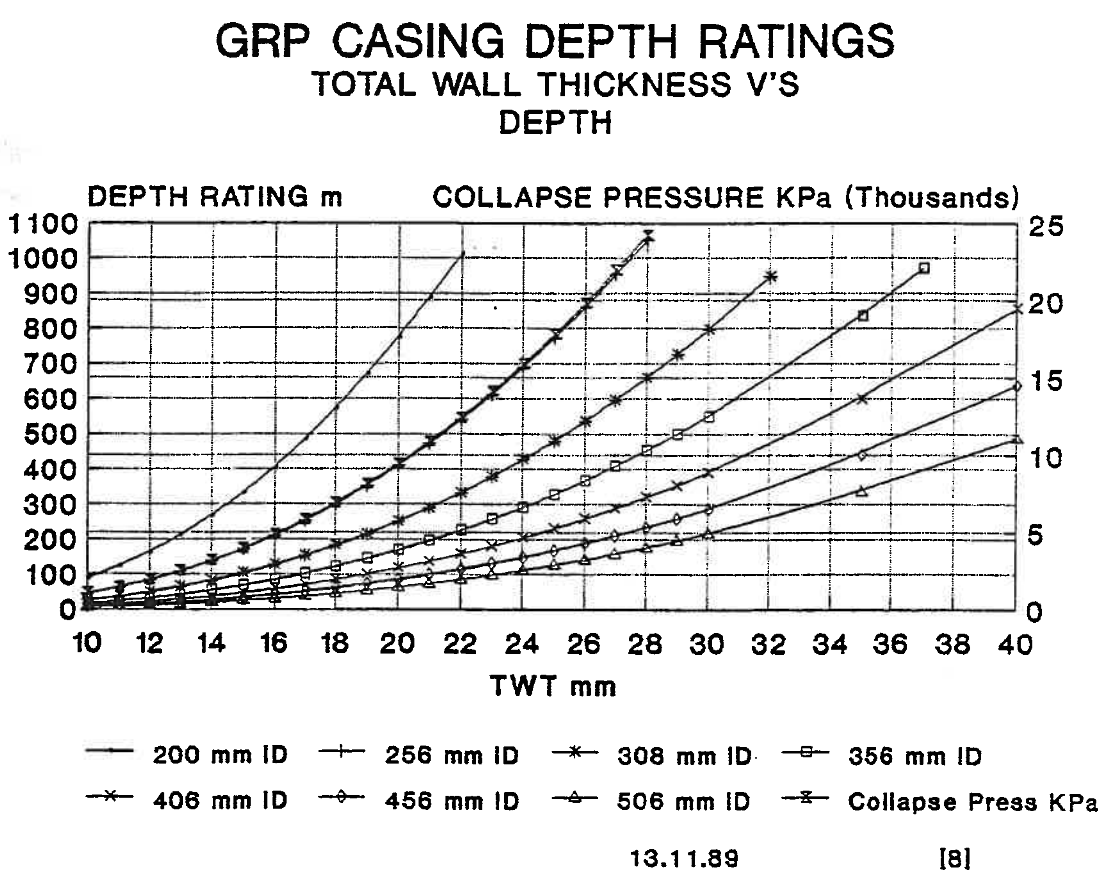

Fig. 3. G.R.P. Casing. Depth rating versus wall thickness.

2.2.3 Test Equipment and Procedure

The equipment used consists of a steel pressure vessel into which a test length of GRP casing is inserted. Two linked end sealing inflatable packers are then inserted inside the GRP test length. The whole chamber is filled with water and then the inflatable end seal packers are inflated also with water to seal the ends of the casing test length. A picture of the test chamber with packers and test piece partially inserted is at Figure 4.

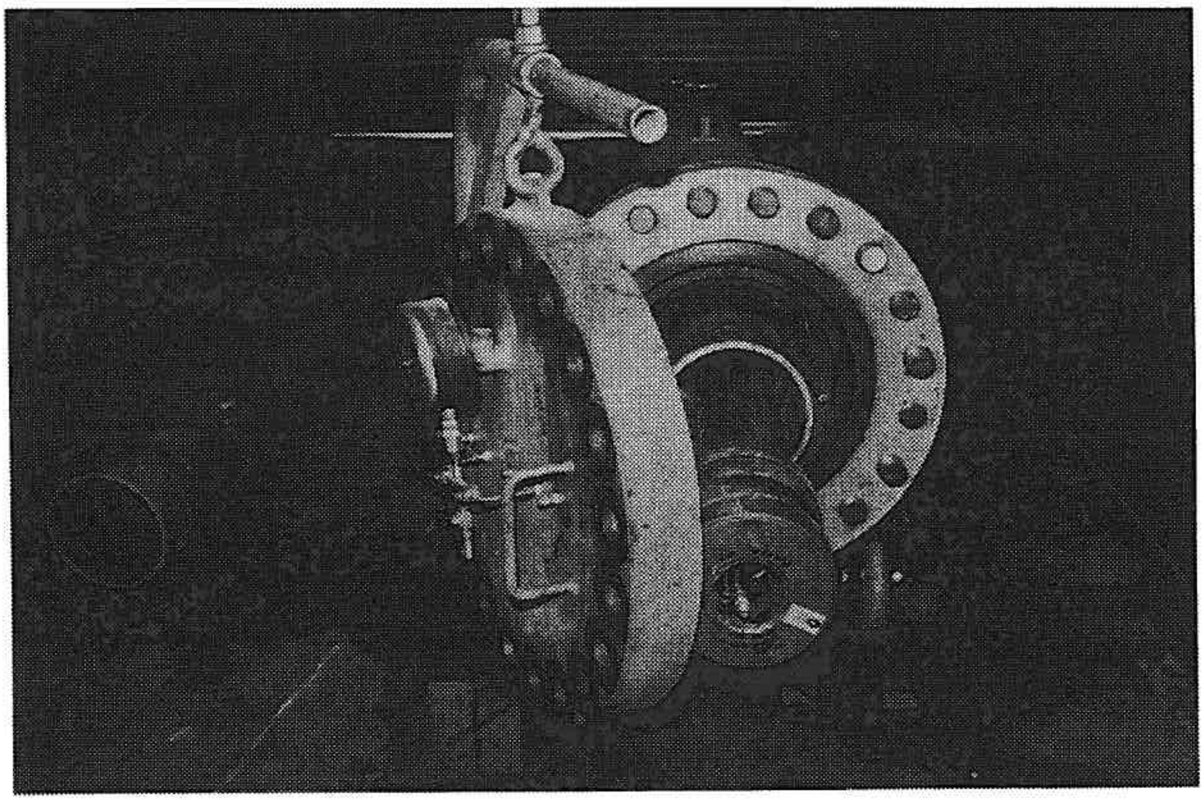

Fig. 4. External pressure testing equipment.

As the pressure on the outside of the GRP casing is increased, the quantity of water (volume change) displaced from the inside of the GRP casing is measured. The point at which External Pressure V's volume displaced becomes non-linear is taken as the yield pressure of the pipe.

2.2.4 Test Results

Forty-nine collapse pressure tests have been carried out to date by the Water

Authority on 200 ND, 250 ND and 300 ND casing. Depth capacity curves for all sizes have been calculated from these results as shown in Figure 3.
A typical pressure test result is shown in Figure 5.

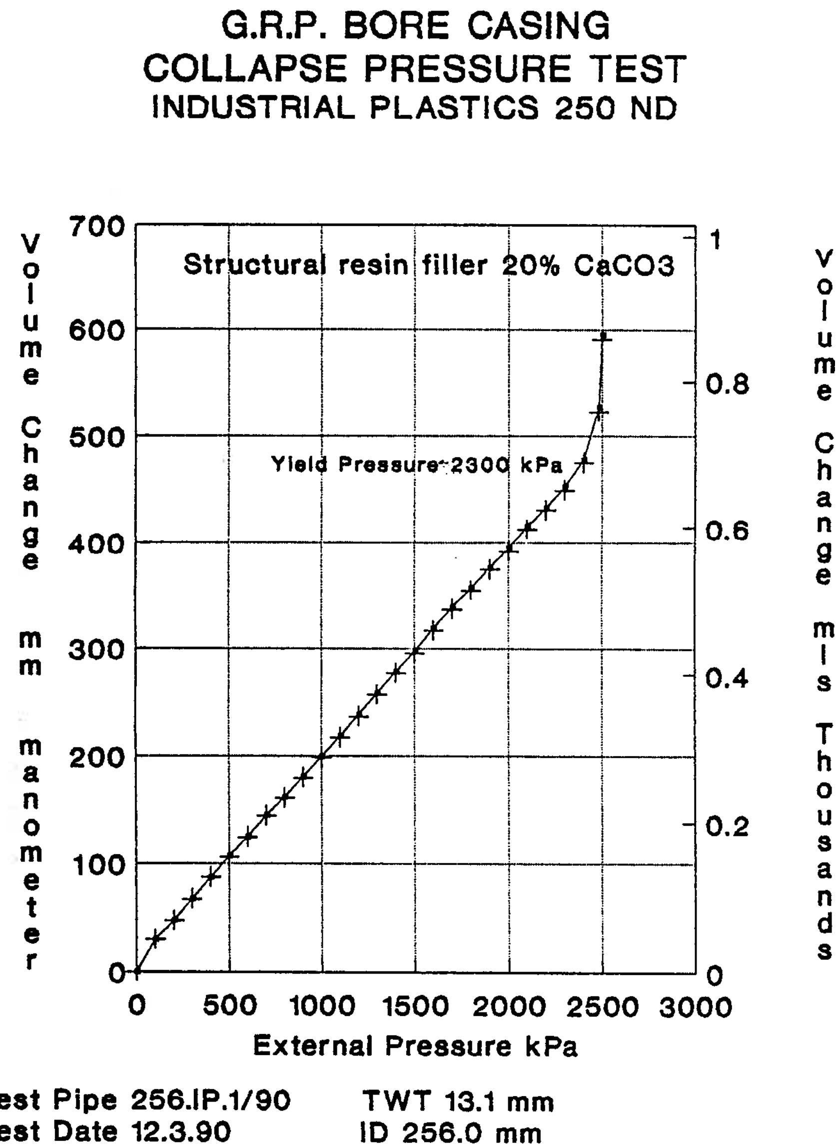

Fig.5. Collapse pressure test. External pressure v's internal volume change.

2.2.5 Utilisation of Results

With GRP casing, cost is proportional to wall thickness. For a 400 ND diameter bore 400 m deep there is a cost saving of around $5000 in reducing wall thickness towards the surface.

2.3 JOINT TENSILE STRENGTH CAPACITY

2.3.1 General

As discussed previously, the required joint tensile capacity for a grouted bore using a grout shoe is determined by the maximum pumping pressure and is equal to the internal area of the casing x maximum pump pressure x load factor of 1.5.

The maximum pumping pressure is equal to the maximum pressure required to displace the grout from inside the casing using water as the follow fluid and should allow for a blockage or constriction to occur. For deep bores the maximum design pressure is likely to be controlled by a pressure relief valve.

Utilisation of GRP casing allows the use of bentonite/cement grouts as corrosion protection is no longer a primary objective. The advantages of using bentonite/cement grouts are reduced shrinkage, better sealing characteristics and lower S.G. giving lower pumping pressures. However, pure cement grouts are still used and required in some circumstances and it is necessary to allow for the higher pressures resulting in these instances.
For example because of the low strength and slower setting time of bentonite/cement grouts it is necessary in most cases to use pure cement grout at the bottom of the bore and at other locations where high early strength is required.

The Water Authority of W.A. normally uses a plastic non-shrink bentonite/cement grout with an S.G. of less than 1.2 in combination with head and tail cement grouts of S.G. approximately 1.7. The average S.G. of this combination ranges from about 1.4 for bores of about 250 metres depth to about 1.25 for bores of about 1000 metres depth.

Bentonite/cement grout when used in deep bores also eliminates breaks in the grout column due to water separation and shrinkage of the cement.

The design minimum joint tensile capacity required for 250 ND GRP casing to 500 metres depth using cement grout of S.G. 1.6 is 255 KN.

Development and testing work was generally carried out using 250 ND GRP casing as this is the smallest size most commonly used by the Water Authority of W.A. and was therefore the most economical to use.

The new joint profile differs from the old profile by the incorporation of a step in diameter to facilitate entry and compression of the 'O' ring and the increase in diameter of the stainless steel wire rope key from 8mm to 10mm.

The new modified joint profile is shown in Figure 6.

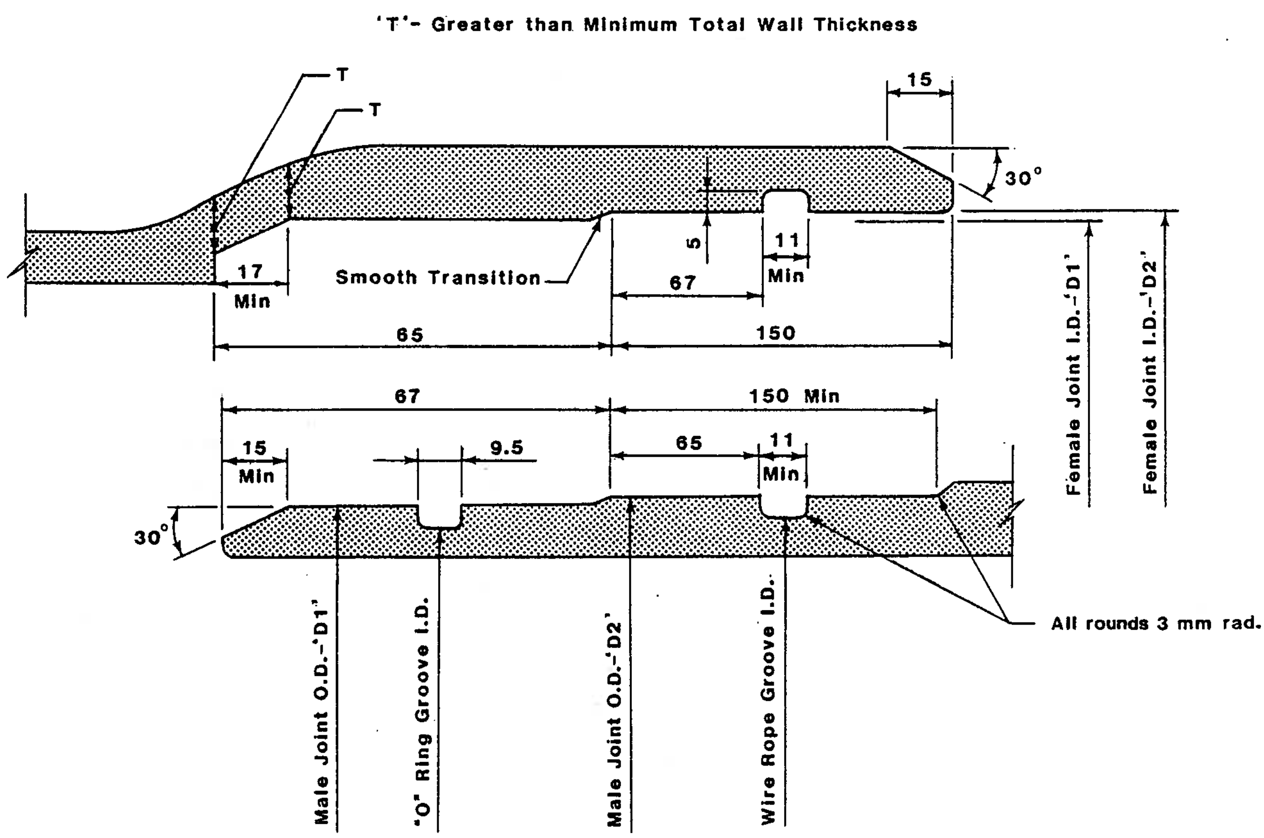

Fig. 6. New Modified Joint Profile.

A tabulation of tensile tests carried out on pipe and joint specimens for this project is contained in Table 4.

Initial tests on 250 ND casing joints at the commencement of the project were tests Nos. 2 to 5. These tests established that the existing 250 ND joint had a tensile capacity of less than 150 KN using a load factor of 1.5.

Research to increase the joint tensile strength was directed in two directions:

(a) Use of a stainless steel or alternative liner
(b) Modifications to reinforcement density and patterns using conventional glass reinforcement

2.3.2 Stainless Steel Liner Research

For bore casing, as with any water retaining structure made of GRP, it is necessary to provide a resin-rich layer to prevent contact of the structural glass reinforcement with water. Tests (Scott Bader Co. Ltd, 1980) show a reduction of 11% in bend strength of laminates after immersion in distilled water at 20°C for 300 days.

Distilled water is more detrimental to GRP than fresh water. However, physical damage to the inside surface of the bore casing wall increases exposure of the glass reinforcement and the temperature of the water may also be substantially higher than 20°C.

The incorporation of a stainless steel liner offered the potential advantages of providing a damage resistant waterproof liner in addition to increased joint tensile strength.
A 250 ND joint was fabricated incorporating a 1.6 mm thick liner in both male and female sections. The liner pipe was spiral wound stainless steel of grade 304.

The tensile testing of this joint was carried out with failure occurring at 390 KN (260 KN after application of load factor). The mode of failure was shearing of the section of GRP on the male joint between the wire rope and 'O' ring grooves.

There were no signs of failure of the bonding between the GRP and the S.S. liner and so there was considerable potential for increasing the joint strength by increasing the shear strength of the GRP between the two grooves.

Having established the anticipated improved tensile strength resulting from incorporation of a stainless steel liner, tests to determine the effect on collapse pressure of the pipe were instituted.

Problems were immediately encountered in producing pipe specimens for testing. The first two full-length spiral wound stainless steel sleeves developed ovality after grit blasting. Spiral wound SS pipe was chosen in preference to other SS pipe due to the large cost difference. Two replacement spiral wound stainless steel sleeves were manufactured and given a lighter sand blast treatment and then wound over with GRP. However, these two pipes also developed ovality due to the shrinkage pressure from the GRP pipe during curing.

External collapse pressure testing of the least oval pipe was carried out. Failure occurred at a pressure of approximately 1250 kPa (see Figure 7.) compared with a minimum expected yield pressure of 4300 kPa for GRP of this wall thickness without the S.S. liner. The S.S. liner failed and was seriously indented, however, there was no visible sign of failure in the GRP pipe.

The causes of this reduced capacity are assumed to be:

(a) The effects of ovality

(b) The effects of the S.S. liner being in compression from the curing shrinkage of the GRP pipe

(c) Imperfect bonding between the S.S. and GRP pipes

Consideration was given to alternative compressively stronger liners and purely protective linings. These included conventional SS pipe and bore screen and alternative integrated linings such as polyurethane or incorporation of "padding" to the S.S. liner to absorb shrinkage. However, these latter alternatives would not contribute to the compressive (collapse pressure) strength of the casing and all significantly increased the cost of manufacture of the casing.

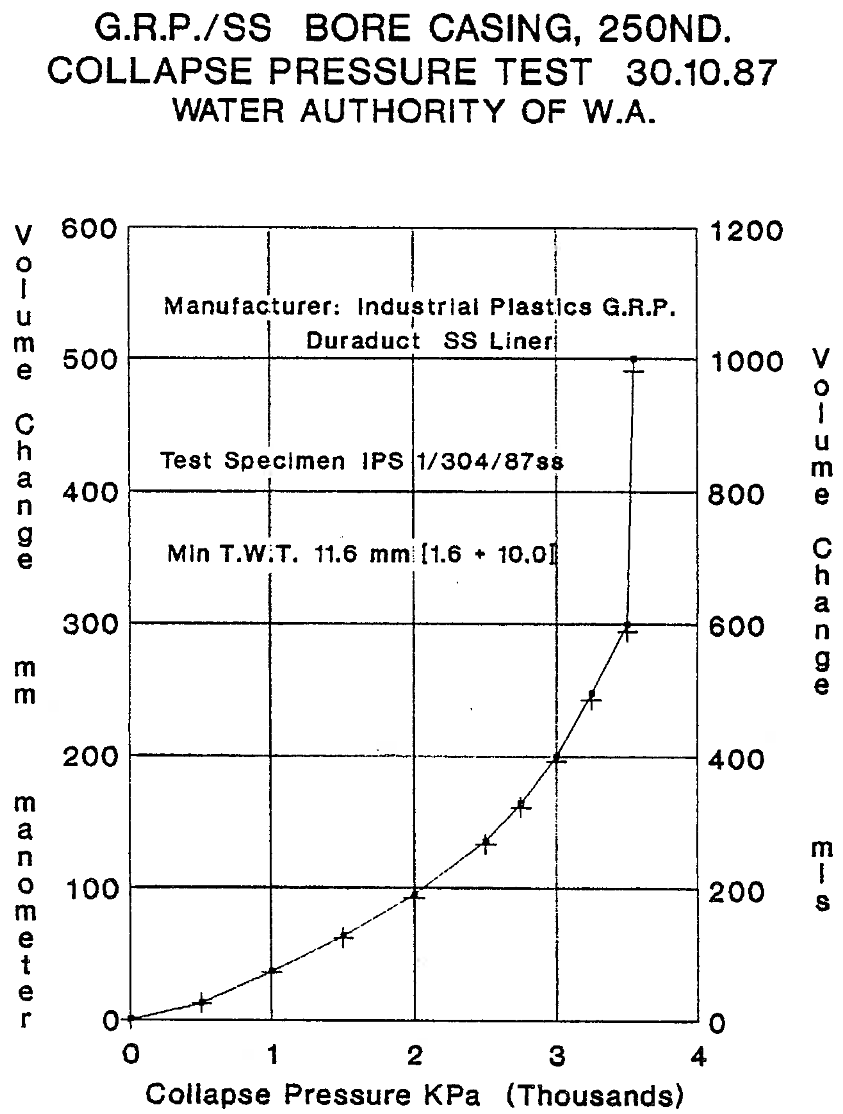

Fig. 7. External collapse pressure test on casing with stainless steel liner.

These developments were not continued to finalisation as the additional cost without significant increase in collapse pressure strength made the economics of these alternatives unattractive. In addition, the alternatives of using 'ECR' glass in place of E glass and vinyl ester resin in place of polyester resin at potentially small additional cost could almost eliminate reduction in strength due to exposure of the structural fibres to water. Evaluation of the additional costs of using vinyl ester resin and 'ECR' glass and the additional benefits is not complete.

Meanwhile, research and development into increasing joint strength by modifying reinforcement density and patterns had achieved a considerable improvement in joint tensile strength capacity.

2.3.3 Modifications to Glass Reinforcement

2.3.3.1 Standard unmodified joint tests

Joint tensile tests of unmodified joints at the commencement of this research project gave a joint tensile strength of around 220 KN (146 KN with load factor) see Table 4, Test Nos. 2-5. Failure occurred at the route of the locking groove in the male joint. The minimum tensile capacity required to meet the minimum requirements of this project is 255 KN.

2.3.3.2 First joint modification

Additional reinforcement was added to the male joint in two ways for a distance of 500 mm from the male end:

(a) Four layers of 800 gm woven cloth laid directly on top of the 450 gm backing layer before winding of the structural rovings.
This alternative was the cheapest as it minimised the additional construction time involved.

(b) Four layers of 800 gm woven cloth laid alternatively with layers of structural rovings. This alternative was tried as it reduced the change in thickness and transition to the remaining pipe for over winding and also reduced the difficulties and effects of inadequate wetting out of the woven roving cloth.

Two male joints were fabricated using alternative (a) and three using alternative (b).

There was no apparent difference in effect between the two alternative ways of adding additional reinforcement and the tensile strength of all of the modified joints was increased to about 400 KN (see Table 4 Test Nos. 6-9 and No. 11). Failure of test connections to the joint sections prevented precise results from being obtained in two tests. The female joints were not additionally reinforced and failure in two test joints (Test Nos. 9 and 11) was due to shearing through the wall of the female joint in the wire rope groove.

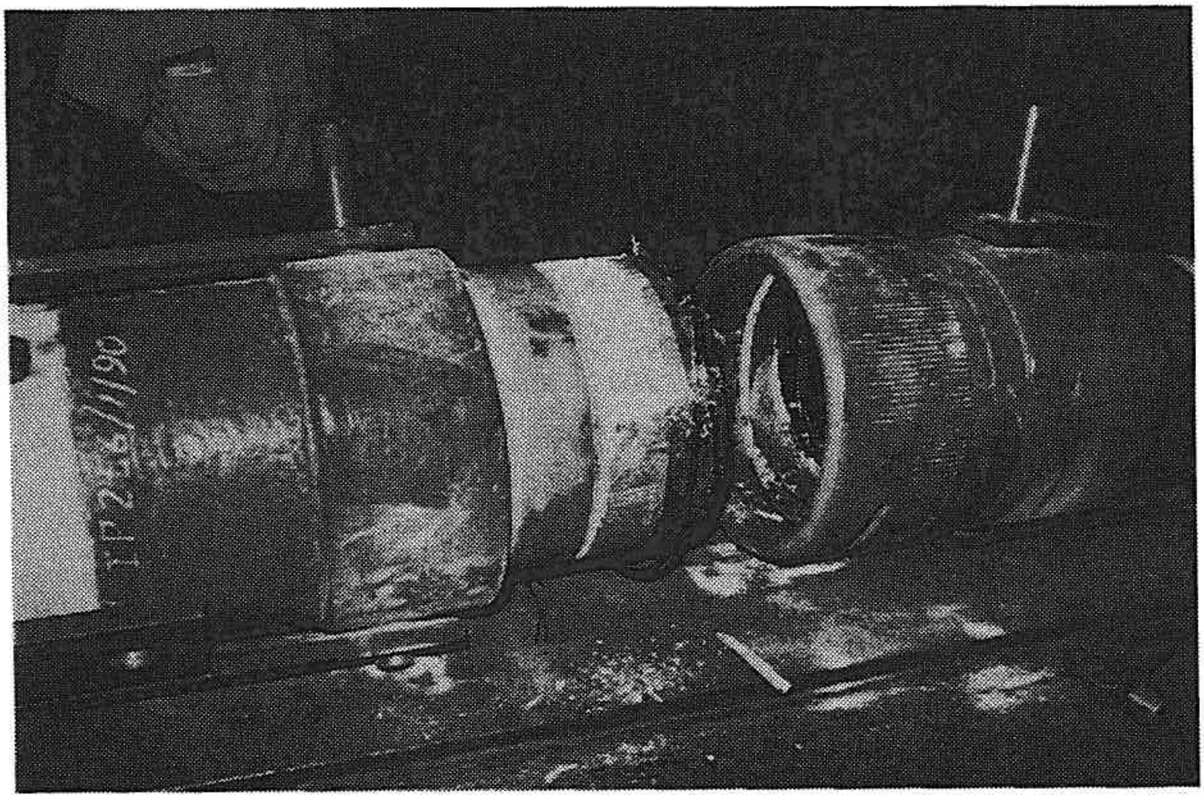

Fig. 8. Joint tensile test on 250 ND casing joint.
Failure due to shearing of section between the two grooves on male joint.

2.3.3.3 Second joint modification

The second modification was to increase the reinforcement to the male joint further to 5 layers of woven roving and to include 3 layers of woven roving in the female joint. This joint was tested to 503 KN before the test connections to the test piece failed.

This test demonstrated that the required minimum joint strength of 255 KN could be obtained by including sufficient additional glass reinforcement in the direction of the tension forces.

2.3.3.4 Third joint modification

The previous tests were conducted on joints with the difference between the male OD and the female ID carefully controlled at 1 mm. The third modification was to investigate the effect of increasing this tolerance to 2 mm. Five joints were fabricated with the male joint OD reduced from standard by 1 mm to give a 2 mm difference from the female joint ID.

The results of these tests (Test Nos 12-16) ranged from 298 KN to 300 KN, a very small range of results with failure occurring in each case due to the flattening of the wire rope and crushing of the leading edge of the male rope groove.

2.3.3.5 Fourth joint modification

Three joints were fabricated with the male joint OD reduced from standard by 0.5 mm to give a 1.5 mm difference from the female joint ID. The range of results from the testing of these joints was 433 to 491 KN with failure again occurring due to flattening of the wire rope and crushing of the leading edge of the male rope groove.

2.3.3.6 Fifth joint modification

To ensure complete wetting out of the woven roving and bonding between layers, a layer of 300gm chopped strand mat has been incorporated with each layer of woven roving.

2.3.3.7 Tests on modified 300 ND joint

With the success in achieving the required joint strength for 250 ND bore casing, the Water Authority was able to design and construct a bore using 300 ND GRP casing.

Tensile tests were carried out on two joint test pieces produced during this pipe contract. The test results obtained were 543 KN and 518 KN (Test Nos. 20 & 21, Table 4). Failure occurred due to the shearing of the section between the locking groove and the \`O' ring groove in both tests. Applying a load factor of 1.5 to the minimum test result gives maximum depth ratings of 705 metres for neat cement grout or 1700 metres for bentonite/cement grout. These results are indicative only. More tests are required to confirm these values.

2.3.3.7 Future research modifications

(a) An insert to hump the reinforcement between the two grooves on the male joint is proposed to increase the shear strength of this section.

(b) Investigations into more rigid materials to replace the stainless steel wire rope key. This would reduce the sensitivity of the joint tensile capacity to dimensional tolerance between the male OD and the female ID.

2.3.4 Joint Tensile Strength Results Summary

2.3.4.1 Stainless steel liner research

The proposed use of a stainless steel liner for increased joint tensile strength was not a practical success as it caused a reduction in the collapse pressure strength of the bore casing. Alternative liners such as polyurethane and PVC could be used but would not contribute to pipe collapse pressure strength and therefore would significantly increase costs.

Technical information to promote the use of ECR glass as a replacement for E glass in situations requiring superior chemical resistance would suggest that the use of ECR glass will reduce any additional strength deterioration following rupturing of the resin-rich layer. (Svenning Torp, Oystein Stromsodd and Magnar Onarheim, 1982).

2.3.4.2 Modifications to glass reinforcement

The use of additional woven roving reinforcement to provide increased reinforcement in the direction of the tensile joint loading is able to achieve the required joint tensile strength at minimal additional cost.

Use of ECR glass as a replacement for E glass and vinyl ester resin as replacement for polyester resin have the potential to protect the casing from strength deterioration following physical damage at minimal additional cost.

3 CONCLUSIONS

The aim of this project was to improve the reliability and capacity of GRP bore casing to enable the safe use of GRP casing to at least 400 ND and 500 metres depth.

The tensile, internal pressure and external pressure capacities of the casing itself can be readily increased to meet the required design specifications by increasing the structural wall thickness. The capacity of the casing joint, however, is limited by the joint design.

Manufacture of GRP casing joints using the new joint profile developed by this project to the Water Authority of W.A. Specification for the Manufacture of GRP Bore Casing (1990), will allow reliable use of GRP casing to the capacities and depths contained in Tables 1 & 2.

Values contained in Tables 1 & 2 are based on maximum capacities of 6 000 kPa for joint pressure test and 300 KN for joint tensile test for 250 ND casing size. These maximum load capacities although meeting the targets for this project are considered to be conservative and maximum capacities of 7 000 kPa and at least 400 KN should be able to be confirmed with the next series of tests.

GRP casing wall thickness is normally reduced with design external pressure towards the surface to minimise cost. A design utilising the maximum joint tensile capacity given in Table 1 would, however, require an adequate minimum casing wall thickness for the design maximum tensile stress.

4 ACKNOWLEDGEMENTS

Research Assistant	:	Mr K. R. Murray, Depot/Purchasing Officer Drilling Section, Water Authority of Western Australia.
ACI Fibreglass Reinforcements	:	Technical advice.
AGE Developments P/L	:	Development of inflatable packer pressure testing equipment.
Industrial Plastics Services P/L	:	Co-operation and assistance with project. Included manufacture and supply of test pieces at or below cost.
Sandvik Australia P/L	:	Supply of spiral wound stainless steel liner pipes at substantially reduced cost.
Transfield Pty Ltd	:	Technical advice and supply of some test pieces at reduced cost.

5 REFERENCES

1. Scott Bader Co. Ltd, (1980). Crystic Polyester Handbook, pp. 97-8.

2. Svenning Torp, Oystein Stromsodd, Magnar Onarheim, (1982). 37th Annual Conference, Reinforced Plastics/Composites Institute, The Society of the Plastics Industry, Inc. January 11-15, 1982. Session 9-E.

3. Water Authority of Western Australia, (1990). Specifications For The Manufacture And Supply Of Glass Reinforced Plastic (GRP) Bore Casing.

TABLE 4

TENSILE TEST RESULTS ON GRP PIPE AND JOINT SPECIMENS

(Note: OD Male 1 mm < ID Female on all Joints Unless Specified)

TEST NO.	DATE	SPECIMEN REINFORCEMENT DETAILS	MODE OF FAILURE	TEST RESULTS KN
1	28/02/86	256 ID, plain pipe, no additional reinforcement. Total wall thickness 10 mm.	Fracture of pipe along the direction of glass windings.	381
2	28/02/86	256 ID, unmodified joint, 8 mm wire rope.	Shearing through wall of male joint along wire rope groove.	227
3	28/02/86	As for Test No. 2.	As for Test No. 2.	257
4	03/03/86	As for Test No. 2.	As for Test No. 2.	236
5	03/03/86	As for Test No. 2.	As for Test No. 2.	238
6	25/09/87	256 ID, male joint reinforced with 4 layers of 800g cloth laid on 450g backing layer, 10 mm wire rope.	Failure of test connections	384
7	25/09/87	As for Test No. 6.	Shearing of band between wire rope and 'O' ring grooves on male joint.	408
8	25/09/87	256 ID, male joint, 4 layers of 800g woven roving laid alternatively with initial 4 layers of wound structural rovings. 10 mm dia wire rope key.	As for Test No. 6.	411
9	25/09/87	As for Test No. 8.	Shearing through wall of female joint along wire rope groove.	403
10	16/12/87	256 ID, male 5 layers, female 3 layers.	Failure of test connections	503

TEST NO.	DATE	SPECIMEN REINFORCEMENT DETAILS	MODE OF FAILURE	TEST RESULTS KN
11	15/03/88	As for Test No. 8.	As for Test No. 9.	400
12	18/10/88	As for Test No. 10, but male joint OD 2 mm < female joint ID.	Crushed leading edge of male rope groove. Rope flattened.	299
13	18/10/88	As for Test No. 12.	As for Test No. 12	298.5
14	18/10/88	As for Test No. 12.	As for Test No. 12	297.8
15	18/10/88	As for Test No. 12.	As for Test No. 12.	300.3
16	18/10/88	As for Test No. 12.	As for Test No. 12	298.8
17	22/12/88	256 ID, male joint 5 layers, female 3 layers. Male OD 1.5 mm < female ID.	As for Test No. 12.	321.5
18	22/12/88	As for Test No. 17	As for Test No. 12.	491
19	22/12/88	As for Test No. 17.	As for Test No. 7.	468
20	02/05/88	308 ID, male 5 layers, female 3 layers.	As for Test No. 7.	543
21	27/06/89	As for Test No. 20.	As for Test No. 7.	517.5
22	20/11/89	As for Test No. 20.	As for Test No. 7.	528
23	29/10/87	As for Test No. 2 with 1.6 mm thick spiral wound S.S. pipe liner & 8 mm key.	As for Test No. 7	390

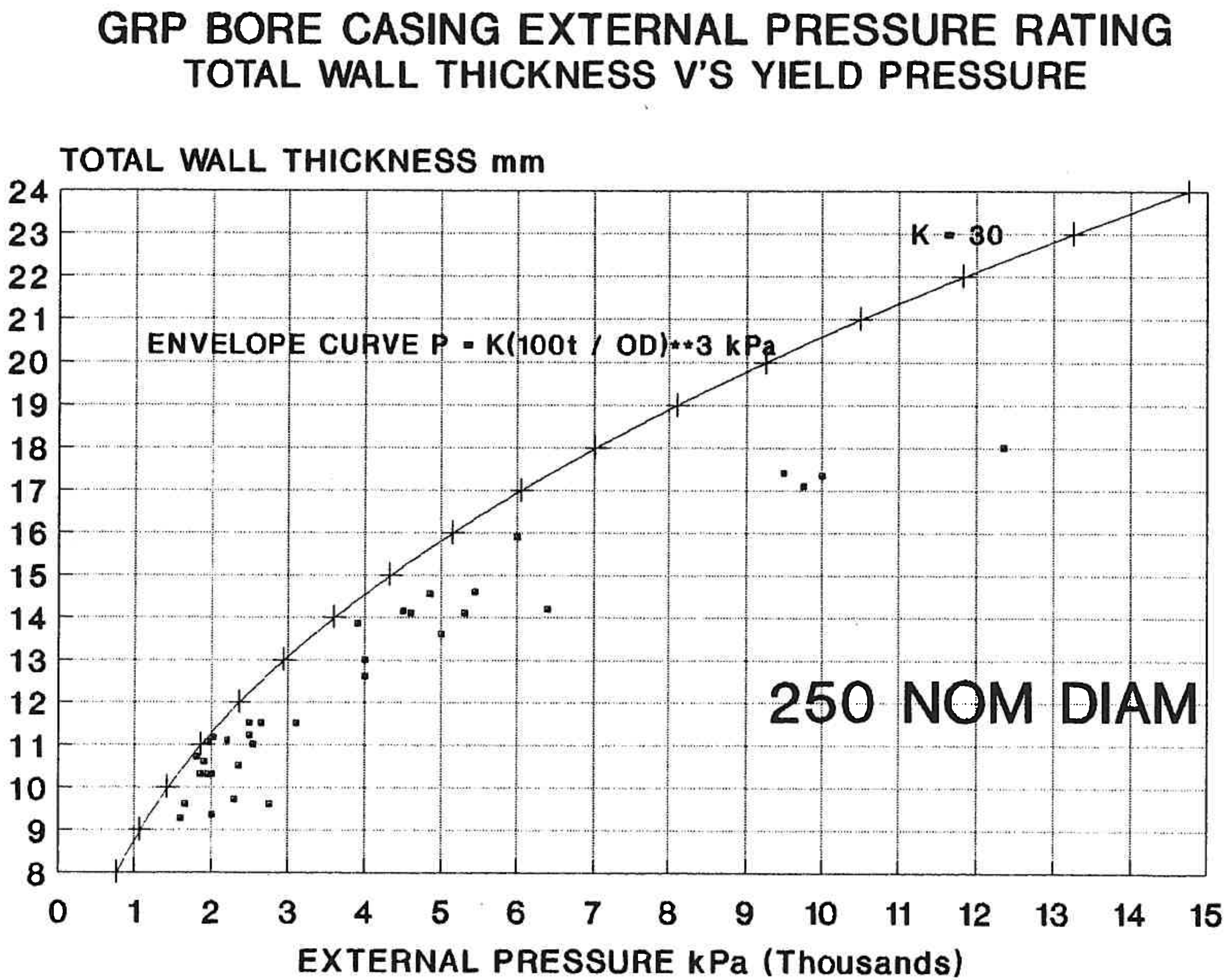

Figure 9. External Pressure V's TWT Envelope Curve for 250ND