Tibito-Casablanca Pipeline Investigation

Report prepared for

Empreso de Acueducto y Alcantarillado de Bogotá

by

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November, 1993

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Executive Summary

Jason Consultants were invited by EAAB to undertake an overall review of the TIbito-Casablanca PCCP aqueduct, to recommend a rehabilitation strategy and propose solutions for the work.

We visited the site of the pipeline, discussed the issues with EAAB staff and the pipe manufacturer and examined the various documents on the work and the problems that had arisen. Subsequently, we have made a review of PCCP lines and their performance in the USA and elsewhere.

General findings concerning PCCP pipe

1. A significant number of failures have occurred in PCCP lines in USA and elsewhere.
2. Several major US authorities have suspended or banned the further use of PCCP.
3. AWWA Specification C301-64 has been substantially amended in the light of experience.
4. No proven methods of investigation and monitoring exist which will locate all impending pipe failures with certainty.
5. Failure of pipe occurs due to breakage of prestressing wire.
6. The causes of failure and the physical circumstances leading to it are diverse.

Main findings concerning the Tibito-Casablanca line

1. There is a good deal of conflicting opinion as to the causes of failure.
2. In our opinion failure probably results from one or a combination of:
   - aggressive soil conditions
   - questionable quality of mortar cover
   - pipes operating close to or beyond the limit of design strength.
3. Further investigation is unlikely to add to the solution of the problem.
4. We are confident that if no action is taken further failures will occur.

Recommendations for a program of rehabilitation

1. EAAB should undertake a program of rehabilitation to replace or line the pipe in a way which will essentially provide a new pipe.

2. Ultimately we believe it will be necessary to replace or line the whole of the pipe. However, it should be possible to carry out the work in stages.

3. Any detailed program of rehabilitation must take into account:
   - the operational consequences of the failure of a major supply aqueduct,
   - the consequences of failure and its impact on property and people.

4. There appear to be 31 km where the likelihood or consequences of failure require action to be taken in the immediate future. On operational grounds it will probably

the necessary to renovate or replace the 4 km. in fields close to Tibitoc reservoir. For the final 20 km between the river crossing in Avenida Boyaca and the Casablanca reservoir, the necessity is not so great.

Proposed solutions

A steel pipe placed in situ seems to offer the best technical and economic solution. Installing a stand-alone steel pipeline within the existing pipe will allow operations at the full operational head with no further problems, even if the PCCP fails.

There are two possible options:

<u>First option</u>:	Using 1825 mm i.d. steel tube sections welded in situ has some technical advantages if the additional loss of cross section can be tolerated.
<u>Second option</u>	Scrolled steel pipes offer the advantage of maximising the diameters to 1875 mm, but involve longitudinal welding in situ.

Relining using sprayed ferrocrete offers a third option and may warrant further consideration. However, the lack of experience of this method and the likelihood of higher costs make it less attractive than the steel liner option.

To provide a fully detailed estimate and work program will require us to prepare with the client an agreed schedule of working to meet operational needs.

Included are provisional estimates which we believe will allow the client to make some initial evaluation of his options. We estimate that the cost of installing 1825 mm or 1875 mm steel pipe liner complete will be US $ 770 to **$800 per metre.**

Once EAAB has determined a basic strategy, we recommend a technical solution be fully developed and designed. This can then be put out to contract.

<u>Jason Consultants</u>

1. TIBITO- CASABLANCA PIPELINE INVESTIGATION

1.1 Introduction

Empreso de Acueducto y Alcantarillado de Bogotá (EAAB) invited Jason Consultants, on 27th September 1993, to visit the Tibito-Casablanca pipeline to inspect the line, to review the studies undertaken up till that date and to make recommendations for the rehabilitation of the pipeline. The work was organised and carried out through the Colombian consulting group Salgado Melendez y Asociados.

James Thomson, Senior Consultant with Jason Consultants, visited Bogota from 13th to 18th October inclusive. A site inspection was made of the line of the pipe between Tibito and Casablanca. A visit was made to the American Pipe and Construction factory to see the production plant and some of the pipe sections retained from earlier failures. Several visits were also made to the site during the period 15th to 18th October while two sections of pipe were removed and replaced. These works afforded the opportunity to carry out an inspection of the pipe interior. The remaining time was spent in discussions with engineers and managers of EAAB, plus a review of the numerous reports and correspondence relating to the pipeline and its failures.

The excellent co-operation and good understanding of the issue by the engineers and managers of EAAB made possible a good overall review in this short period.

1.2 Description of pipeline

The Tibito-Casablanca pipeline runs from the Tibito treatment plant to the reservoir at Casablanca. The length is understood to be 53 km and the diameter 2000 mm. The installation work was started in 1968 and completed in 1972. The pipes were made locally by American Pipe and Construction, a subsidiary of Ameron Pipe of California. The pipe is prestressed concrete cylinder pipe (PCCP), fabricated in accordance with AWWA C301-64.

From the reservoir at Tibito the pipeline descends to fields and continues across open farmland, crossing the Devsaca River until it hits the Altogrande road around 5.8 km. Housing and leisure facilities are under construction and it is understood that there is a strong possibility of this area being developed further.

Between 5.8 km and 13.2 km, the line is located in the western verge. At around 13.2 km it turns west, crossing the 60" line then turning south into the central reservation of the Autopista North. For several kilometres the line is located in a wide central median. As the road nears the city and urban development increases, the main is located in the western verge.

At Street 129, some 30 km along the length, the main turns west and is located under the carriageway of this street in a busy commercial and residential area. After some 2 km in Street 129, the line turns south, passing close between some apartment blocks and then running into Avenida Boyaca along the eastern carriageway of the two-lane road. The line is carried across the river as a bridge and continues along the Avenida for a considerable distance before turning west and crossing a recently developed commercial/industrial area from which it rises up the hill to the reservoir at Casablanca.

1.3 History of failures

Eleven failures have occurred on the line, the first in 1978 and the last in 1989. We understand the operating head was initially reduced by 30 metres in 1983 and by a further 20 metres at the beginning of 1988.

The failures, with details of their location and pipes, are reported in the EAAB summary sheet attached as Appendix 1.

A number of reports, commissioned by EAAB and American Pipe, have reviewed these failures in detail. In January 1984, a report by Colombian consulting groups EI, CNEC and SMA provides a useful summary of the work done up to that time. We refer to the contents of these reports in the appropriate sections of this document.

A number of facts are worth noting about the failures:

- Three of the defective pipes (Nos. 8, 9 and 11) were detected prior to rupture.
- All failures have occurred between Km 4.1 and Km 32.8 from Tibito.
- The majority of failures, nine in number, have occurred in the section between Km 13 and Km 28.2.
- All 11 failures appear to have occurred in non-paved areas.
- All ruptures were in the upper half of the pipe, with 10 out of 11 occurring in the upper eastern quadrant.
- All ruptures are reported to have taken the form of a rectangular window.

2. PRESTRESSED CONCRETE CYLINDER PIPE

2.1 Background

Prestressed concrete cylinder pipe (PCCP) was first introduced in the USA in 1942, after several years of research. About 28 000 km of this type of pipe have been manufactured and installed in the USA. An unknown but significant length has also been installed in other parts of the world.

PCCP has been produced in 16" to 252" internal diameters and for pressures up to 27.5 bar.

2.2 Specifications

The first manufacturing standard (C301-64) was published by the American Water Works Association (AWWA) in 1964. This standard was significantly revised in 1992.
AWWA Specification C301 allows for two types of fabrication:

- Steel cylinder lined with a concrete core
- Steel cylinder embedded in a concrete core.

The Tibito-Casablanca pipeline is of the second type.

Under C301, alternative design methods are permitted and are known as Appendix A and Appendix B. The Tibito-Casablanca line is understood to have been designed under Appendix B.

2.3 Specification changes

As a result of experience and extensive research, a number of modifications have been introduced into the design, manufacture and installation of PCCP. These changes have mainly come into operation as a result of experience gained in the 1970s.

<u>Main changes in manufacture</u>

Mortar coating
- Increased mortar thickness
- Additional cement-rich slurry coat prior to wire wrapping
- Increased minimum moisture content of the mortar by 10%

  [Current research is to make coatings denser and with lower permeability.]

Wire
- Eliminated 8 gauge wire
- Excluded use of Class IV wire
- Additional testing for ductility, tensile strength, torsion and splits

Cylinder steel
- Eliminated Grade E cold rolled
- Eliminated 18 gauge
- Improved welding and QC

Aggregate and cements
- New tests and quality limits are being proposed

<u>Main changes in design</u>

A Unified Design Procedure is being introduced to supersede the present alternative design methods.

<u>Notes</u>
Edmonds [1] compares theoretical results devised from Appendix A with actual test results for

pressure-strain performance and shows that test results fall well below theoretical performance. An example quoted is that pipe designed for 150 psi working pressure was found to provide an actual working pressure of 103 psi.

A second standard, C304, is currently being developed, which we understand will replace C301.

Installation of PCCP pipe is covered by *AWWA Manual M9 for Concrete Pressure Pipe*. This manual is presently under revision.

3 CAUSES OF PCCP FAILURE

3.1 Known causes of failure

It has been stated that PCCP failures always result from breakage of the prestress wire, normally because of corrosion or degradation of strength. The cause of this breakage varies from pipe to pipe, but often arises from a sequence or combination of events.

The mechanisms of failure in wire that have been documented include:

- Mechanical failure
- Galvanic corrosion
- Stress corrosion cracking
- Hydrogen embrittlement

The predominant form of failure reported has been stress corrosion cracking.

Four conditions that lead to premature failure have been identified as:

(1) Internal corrosion
This normally arises from hydrogen sulphide attack in sewer force mains. Internal corrosion failure has occurred on water mains with leaking cylinders.

(2) External corrosion
This occurs where the mortar coating does not provide an effective barrier. Aggressive constituents in the soil or ground water which can reduce the localised passivity of the prestressing wire include chlorides, sulphates and pH..

(3) Detrimental strain ageing of the wire
This occurs when wire is drawn in a manner that causes free nitrogen or carbon to migrate to dislocations in the structure and to lock in at those nodes. Characteristics are brittleness, low ductility with generally high tensile strength. Drawing too quickly and at high temperatures (600 to 700 degrees F) can be the cause. Overheating of wire during drawing can also lead to susceptibility to corrosion pitting, hydrogen embrittlement and poor torsional ductility.

(4) Hydrogen embrittlement
A number of theories exist on how hydrogen embrittles steel.. However, it has been established that hydrogen embrittlement is caused by factors external to the wire. Causes for the source of hydrogen can be electrochemical corrosion or excessive electrical potentials. Susceptibility of wire to hydrogen embrittlement is directly related to the ultimate tensile strength of the wire and, to a lesser degree, to the applied stress on the wire. It is for this reason that Class IV wire is no longer used in this application.

Investigations of failures in the USA [2] have determined the following conditions that have contributed to failure:

- Leaks in the steel cylinder
- Delamination of the mortar coating
- Chloride-induced corrosion
- Disrupted and porous coating
- Thin and inadequate cover
- Coating cracks
- Dented cylinders
- Damage during handling and installation
- Chemical attack
- Settlement
- Surge overloading
- External overloading

During our investigation it was drawn to our attention that the greatest number of failures had occurred under non-paved areas. At this point we do not have any comparative figures to show the percentage of failures in paved as opposed to non-paved areas. However, it is thought that where pipe is below paved areas the moisture content of the soil remains constant, whereas below non-paved areas (fields, verges and open areas) seasonal drying and wetting are likely.

The Bureau of Reclamation has a current moratorium on the use of PCCP until the results of a joint investigation with AWWA are complete. Since 1987, the Washington Suburban Sanitary Commission has discontinued the use of PCCP.

3.2 Current research

The US Bureau of Reclamation and the AWWA are presently undertaking a joint study, *Performance of Prestressed Concrete Pipe*. Appendix 2 outlines the content of the proposed final report due late in 1993.

This report will include:

- The results of a pipe inventory survey of lines greater than 24" which has been distributed to 500 water user organisations.

- Work undertaken on developing non-destructive assessment methods.

- Emphasis and research on prestressing wire failure mechanisms for design & protection..

3.3 Tibito-Casablanca pipeline

The numerous reports made by the various investigating bodies generally agree that failure of the pipelines occurred as a direct result of corrosion of the wire reinforcement which maintained the pipe barrel in compression under normal operating conditions. The matter of variance or contention between the various authorities is what initiated the corrosion mechanism. Various hypotheses have been proffered, but with little apparent common grounding. However, in the majority of scenarios it is the presence of moisture that promotes and/or maintains the corrosion process and this leads to a requirement for penetration of the mortar coating by some means or other.

We discuss the main parameters in the following sections.

3.3.1 <u>Corrosive ground conditions</u>

Ground conditions around the pipe generally consist of clay overlain by silt or fill material. In deeper sections there is evidence of sand or other granular material.

The natural corrosivity of a ground is generally measured in terms of its pH or resistivity. Respective values of less than 5.0 and 3000 ohm-cm are generally accepted in the UK as being corrosive to concrete.

The various reports show little correlation between each other, with particular criticism of results prepared by the University, particularly with respect to pH values. The University data from 520 samples provide pH values varying from approximately 4-7 of which the majority are less than 5, indicating an aggressive soil. These values are disputed by the other investigators, although one of the failure sites investigated by Failure Analysis Associates quoted a pH level of 4.3. All other quoted values at the failure sites are in excess of 5.5.

Resistivity values generally only give a coarse indication of soil corrosivity. The University results generally suggest a moderately aggressive soil, but do not approach the 1000 ohm cm level quoted as aggressive by the other investigators. Significant resistivity levels were only determined at depths in excess of 4.8 m.

Sulphate and chloride levels in the soil do not indicate likely potential protection although some sources indicate potential attack from soils in areas C,D,E.

3.3.2 <u>Pipe construction</u>

<u>General</u>

The pipe elements include the concrete core, steel cylinder, prestressed wires and the mortar covering to the wires. There seems to be no doubt or disagreement about the performance of the core or steel cylinder. The main source of concern relates to the performance of wire and mortar and that the performance of the wire is directly influenced by the resistance of the mortar to prevent permeation of water to and corrosion of the steel wires.

Mortar coating

Corrosive attack by an aggressive soil will lead to the removal or passivation of the lime content of the cement, so increasing the permeability of the mortar coating and allowing attack of the steel. In areas where low pH is present, attack on the mortar would therefore be expected.

Physical variations in the mortar quality were evident, both as the cover provided to the steel and in the strength of the mortar itself. Covers seem to vary between 20-37 mm. One investigator commented that 20 mm minimum would be acceptable. Rates of attack will obviously be directly related to the thickness of the mortar provided.

Investigation of mortar fragments tested in the laboratory indicated that these samples were of the specified strength. However, visual inspection of some samples revealed sandy inclusions and permeable construction. There is some evidence to substantiate this finding with reference to voids in the mortar. Of more concern were reports of both incorrect mix proportions with an excessively high aggregate cement ratio and chloride concentrations adjacent to the steel well in excess of the maximum allowable limits. The source of the chloride was unknown.

In extraction tests carried out on mortar samples it was found that the pH and alkalinity levels were both low in relation to anticipated levels. Consequently the degree of corrosion protection likely to be provided by the mortar covering would be reduced.

Cracking of the mortar could be initiated by mishandling, pipe settlement, operational methods, third party damage or a combination of these. Cracking of such structures incorporating thin wire gauge steel should be limited to 50 microns to ensure autogeneous healing wherever the possibility exists. Cracks in excess of this magnitude will allow ingress of water where it is present and precipitate corrosion of the steel.

All these factors will obviously lead to a higher rate of loss of strength and breakdown of integrity allowing premature attack of the reinforcement steel. Authoritative evidence relating to the mix design of both the cement mortar and concrete core is not available, nor is there any available data as to quality control of the materials used or the actual manufacture of the pipes.

Steel reinforcement

Investigation of the steel wires and testing of samples broadly concluded that there was general corrosion of the steel which led to loss of cross section, increased stress and ultimately failure of the wires, which varied from 4-6 mm in diameter. Failures were partly ductile but most samples exhibited a brittle failure mode.
In general it was found that the metallurgical crystallography did not indicate any serious physical inherent fault in the steel other than the presence of corrosion. In general it was found that the test strengths were within the specified limits. In one instance, despite the strength attainment, the chemical composition of the steel was such that the sulphur content was high.

There are various opinions, contradictory in several cases, that brittle failure was initiated due to the production of hydrogen in the corrosion process with diffusion of hydrogen atoms into the steel, causing embrittlement and brittle failure. This mechanism was totally refuted by one of the investigators despite the promotion of this theory by others.

Various investigators worked to determine the influence of stray AC and DC currents which might induce a voltage in the steel reinforcement causing corrosion by the formation of a voltaic cell.

The methods used to investigate differential potentials between the pipe and the surface, and from surface to surface locations along the pipeline, are of dubious accuracy. Whilst various hypotheses were proposed, the methods employed were limited and therefore the validity of the results are questionable. One investigator went as far as saying that the results related more to cylinder electrical continuity than to the condition of the wire, since it is difficult to validate continuity of the reinforcing wires. Despite this fact, some of the investigators could not explain the second and fourth failures. Corrosion caused by induced currents was proposed as a possible cause, but not necessarily the primary cause.

3.3.3 Pipe design and operation

The pipeline elements were designed to meet the requirements of AWWA -301-64 Appendix B. Design checks carried out on the pipeline indicated that the theoretical rupture pressure of the pipes was very close to the actual working pressures, and only slightly in excess of it.

The minimum level of design would be the value of the pipe test pressure which would be expected to be 1.5 times the working pressure excluding surge, but working pressure plus surge should not exceed the design or rupture pressure. No information has been provided on the original specification for the required working or design pressure, but it is understood that working pressures have been reduced as a result of the problems currently being experienced. On this basis it could be considered that the pipeline has been under-designed based on a statement made by one of the investigators that design pressures should be 2½ - 3½ times the working pressure. Although this seems excessive, some safety factor over and above the pipe test pressure would have been required.

It is believed that in the early operation of the pipeline surge pressures were prevalent but their level of magnitude is unknown. If the pressures experienced were in excess of the pipe design, there is every likelihood that the prestress capacity of the steel would have been exceeded and the additional strain on the steel and pipe could have led to cracking of the mortar protection. This is particularly likely in the relationship between the design and working stresses of the pipeline discussed above. This would, of course, relate to acceleration of breakdown of the mortar coating and corrosive attack of the steel reinforcement.

3.4 General discussion of the Tibito-Casablanca line

The evidence suggests that the failures on the Tibito-Casablanca line result from brittle fracture of the prestressing wire. The mechanisms leading to brittle fracture are most likely to be stress corrosion cracking or hydrogen embrittlement.

However, the causes which created the condition for failure are less obvious. The parameters that could cause or have an influence on the possible failure of an individual pipe are extensive and many of the individual factors are interdependent. Therefore it is very difficult, particularly in the light of the large amount of available information, some of it contradictory, to say with any certainty that one particular combination of events will lead to failure of a pipe. However, the most likely scenario is a combination of aggressive ground conditions related to questionable quality of cement mortar, with pipes operating very close to the limit of their design strength.

4 PCCP PROBLEMS AND ATTRIBUTED CAUSES

4.1 History of failures

The experience of other owners in the performance of PCCP lines is valuable in comparing the experiences on the Tibito-Casablanca with similar pipelines.

James Clift [3] provides a perspective on PCCP pipe performance. Figure 1, taken from his paper, shows for USA by year of manufacture the number of failures which total 118. Clearly something went wrong in the 1970s. It also becomes apparent that one company, Interpace, made a major contribution to the problem. Interpace took over the Lock-Joint company and produced about 50% of the total pipe made. This company also had 70% of the problems. Figure 2 shows the Interpace failure profile together with three significant events in the history. This can be compared with the failure profile for the remaining 9 companies shown in Figure 3.

Some of these failures only involved the replacement of one or two pipes, others required the rehabilitation of the whole line.

We have some reservations about the above figures as they may not fully reflect the whole picture. Price's paper [2] reports 300 ruptures being recorded . As there are many sensitive areas for both owners and producers, information is not readily available. Several clients and producers are locked in legal proceedings.

	9 COMPANIES	INTERSPACE	TOTAL
Pipe supplied, Million feet	51.0	45.0	96.0
Time period	47 years (1943-1990)	37 years (1943-1981	--
Total number of projects	17,400	11,500	28,900
Projects with problems	38	80	118

Figure 1 Performance record

PROJECTS WITH PROBLEMS

Figure 2 Interspace - 45 million feet - 11,500 projects

PROJECTS WITH PROBLEMS

Figure 3 9 companies - 51 million feet - 17,400 projects

4.2 Authorities experiencing problems

The following are some of the authorities that have experienced failures:

Bureau of Reclamation	Jordan Aqueduct 2.3 miles 66" Central Arizona, 252"
San Francisco Water Department	14 800 ft of 66"
Washington Suburban Sanitary Commission	Some thousands of feet of 72" and 84"
Fairfax County, Virginia	36"
Lake Michigan	Power station intake. 5 miles 36"
Penellis County, Florida	
San Diego County Water Department	13,000ft -various diameters rehabilitated
San Diego City	
Seattle Water Department	23 miles of 54", 60" and 66"
New Jersey	108 " line
Texas	108" line

PCCP pipe failures are also known to have occurred in:

Spain,	Madrid Western System 17.230 km 1600 mm i.d. (2 pipes failed)
Mexico	
Brazil	Sao Paulo and Tapacura

4.3 General discussion

In an overall review Clift [3] states that for 42 projects with failures:

7 cases of internal corrosion have been recorded
25 cases attributed to external corrosion
3 caused by excessive Cathodic protection
6 caused by manufacturing faults
3 due to thin mortar coating
3 due to defective cylinders

For pipe manufactured after 1982 in USA no failures are recorded.

From published material and direct communication with authorities experiencing problems, we have assembled the following information on the apparent causes of failure.

<u>San Diego Water Department</u>

This authority has rehabilitated around 4000 metres of PCCP. The pipe was installed in the 1960s and rehabilitation began in the 1980s. The cause of failure is reported as being due to a combination of soil aggressiveness, poor workmanship and chloride attack leading to failure of the prestressed wire.

Lake Michigan - Power Station Intake

The owner replaced this line of about 8 km of 30". He experienced 5 ruptures during the first six years of operation. A detailed investigation indicated that failure probably stemmed from (a) failures associated with cylinder dents, resulting in *kinking* of the steel, and (b) failures associated with mortar coating defects.

Seattle Water Department - Tolt Pipeline

A water line of about 34 km in diameters 54", 60" and 66". Failures are attributed to hydrogen embrittlement.

San Francisco Water Department [4]

The San Andreas Pipeline has a section of 4.5 km of 66" PCCP. This pipe, laid in 1980 and 1981, experienced failure in 1990. Corrosion of the prestressing wire is thought to have been induced by stray currents. We have a separate report that suggests an over-designed adjacent cathodic protection system may have been the problem.

Madrid Water Department [5]

The Western System consists of 17.23 km of 1600 mm PCCP. Two failures occurred in the line — in the upper part of the pipe and close to joints. The conclusion is that cracking occurred in the 7 mm wire when under tension due to corrosion produced by a humid environment. The information suggests the concrete coating was porous, water leakage from joints contributed to the wet conditions and the steel used was not the most suitable.

We are also aware that of the 80 failures that have occurred with Interpace pipe a great number have been attributed to manufacturing faults. Interpace manufactured its own Class IV wire from about 1970. This type of wire is no longer permitted. Other problems seem to have arisen from sub-standard mortar coating and substandard aggregates which affect the creep and strength properties of the concrete.

5 METHODS OF INVESTIGATING AND MONITORING PIPE

5.1 General overview

Corrosion evaluation techniques that have evolved over the last 20 years include:

- Internal inspection and sounding
- Potential survey techniques
- Soil resistivity surveys
- Chemical analyses of the environment

These techniques will provide some information about the condition of the pipeline and the corrosivity of the environment. None of them provides, separately or in combination, a satisfactory method of determining the integrity of the pipeline. At best they may enable some, but not all, suspect areas to be determined, which can then be more closely inspected and evaluated.

Internal inspection

Price and Bianchetti [2] [6] report that internal inspection can detect horizontal cracking at the area of the spring line. In the USA a number of incipient failures have been detected in this manner. Cracks in the invert and crown from external loads were found not to be significant. Cracks will only occur when the wires are broken. Failure of some wires is not sufficient to induce internal cracking.

In San Diego, hammers are used to sound the PCCP during internal inspection. The authority claims that it is possible to detect *hollow* areas of coating in this way.

Sophisticated internal scanners are now available which use ultrasonic, magnetic and calliper techniques. The Ultrascan system from Preussag is one example, but there are several others. However, in the light of our experience, we do not consider that such systems will be suitable for use in a concrete cylinder pipe. However, we do believe that it will be possible to collect reliable data on the state of the prestressing wire.

Potential surveys

Surveys of potential have been used widely but success, as reported, has been variable. At best it is described as a screening method. There are many conditions where meaningful information may not be obtained, e.g., where the line is below a pavement, where the soil is saturated or where there is a high water table. Certain soils create difficulties; clays or soils high in iron for instance. Scrap steel in the ground can produce misleading information.

Soil resistivity surveys

Soil resistivity surveys have their greatest value in the initial evaluation of a new installation. However, survey results can be misleading where the pipe is surrounded by mixed backfill and is not representative of local soil conditions. It has also to be recognised that the surrounding soils are not necessarily an essential element in the corrosion process. Poor quality porous mortar coatings or damaged coatings will themselves provide sufficient electrolyte for the corrosion process. Numerous instances of corrosion have been noted where soil resistivities have exceeded 100,000 ohm cm.

External inspections

External inspections are costly and can only be carried out after other methods are used to pinpoint possible problem areas. We suggest that the recent sample pipes removed from the line should be allowed to dry thoroughly and left for about three weeks before carrying out sounding tests. Experience indicates that coating defects often come to light only then.

5.2 Research into investigation and monitoring

Condition assessment research is being undertaken under the joint AWWA/Bureau of Reclamation program , *Performance of Prestressed Concrete Pipe*.

The research is focused on two approaches:

- An impact-echo method for inspecting pipelines internally.
- A fibre-optic hydrophone system for continually monitoring pipelines for distress.

The results of this research have yet to be published but are unlikely to be available in commercial form for some time.

5.3 Investigations on Tibito- Casablanca Line

The investigators have employed numerous methods for examining the interactive conditions which could influence the performance of the pipeline.

These have included investigation of:

- Virtually all the applicable parameters related to the physical condition of the soil.
- The physical and mechanical condition of the mortar coating and steel wire.
- The potential for current induction in the pipe causing corrosion.

5.3.1 Soil condition

No details of the testing methods are provided in determining the chemical parameters of the soil conditions, although it we would assume these were broadly similar. Even so, results obtained by the researchers vary widely, especially in relation to the particularly critical pH results.

Methods adopted for determining soil resistivity also appear generally the same, essentially using a four-electrode Wenner method with valuations of resistivity taken at various depths. Again, there were discrepancies between the results.

5.3.2 Mortar coating

The results of investigations into the mortar coating were generally confirmatory; strengths and extent of carbonation were of similar orders. Also generally determined were the existence of low pH and alkalinity values, and incidence of elevated chloride levels in the mortar, which indicated reduced corrosion resistance.

5.3.3 Steel

Metallurgical examination by microscopy was carried out by several investigators and all determined the incidence of brittle failures. However, their conclusions about the cause of the embrittlement were not so precise. One researcher stated that there was no susceptibility to hydrogen embrittlement.

One of the major concerns and points of discussion within the various reports related to the susceptibility of corrosion caused by induced currents and the remote detection of corrosion within the steel by measurement of differential potentials in the reinforcement steel. While most researchers indicated the possibility that induced corrosion occurring either from an AC or DC source, they could provide little justification of the true cause or the extent of damage it might produce. Indeed discussion of the literature revealed dissension between other researchers about its occurrence, its cause and its extent, rather than providing categorical proof of its incidence.

If induced currents were being generated, the degree of corrosion produced would depend on the integrity of electrical continuity within each pipe length and also between adjacent pipe lengths. There would be no direct continuity between the reinforcement steel across the pipe joint but there would be continuity across the joint via the steel cylinder. However there is no evidence of corrosion occurring in the cylinder.

There was significant discussion on the measurement of electrical continuity and the potential difference between the pipe and adjacent soil giving rise to corrosion. Two methods were discussed. One method measures the potential differences between a fixed point and other surface locations along the length of the pipeline. The second attaches an electrode to a metal component on the pipeline and measures potential at the surface along the length of the pipeline. Both these methods attempt to interpret electrical continuity (or lack of it) in the reinforcement of the pipeline to determine the location of critical points of corrosion along its length.

Little success is claimed in the operation of these techniques. Whilst some researchers promote the use of these techniques, they tend to undermine the validity of their proposals by scheduling a comprehensive detailed list of operational limitations, or provisos. A true indication of electrical continuity could only be interpreted with any degree of accuracy if a direct electrode connection had been made to the reinforcing steel of each individual pipe. Obviously this would have been prohibitively expensive, and destructive, if implemented.

5.4 General discussion of Tibito-Casablanca pipeline investigation

Considering all the research that has been carried out by the various parties, it is disappointing that so much work has been piecemeal and, we suspect, not always objective. It would have been preferable if a clear rationale or strategy had been developed for the investigations. Certainly the great many parameters involved has complicated the problem excessively.

Apart from the visual inspection of the pipeline, no results are presently available to us for the tests carried out on the two pipes removed for examination.

The evidence offered on ground conditions suggests to us that most problems could be identified by sampling and analysing soil around and at the level of the pipe. However, as has been determined, distinct anomalies relate to the quality of manufacture of the pipeline. This will provide a secondary issue to the problem, which could override or overtly influence the results of such an investigation.

Consequently, it may be best to defer further investigations and concentrate on the rehabilitation of the pipeline. The pattern of bursts appears random, although the preponderance of these occur in the region of chainage 20,500 - 29,000; 6 of the 11 failures have occurred here. Four years have lapsed since the last failure. It is statistically likely that another failure may be imminent.

No evidence has been made available on the internal condition of the existing pipe. The water analysis would suggest that the water source is of high quality. However, the low consistent values of alkalinity and conductivity would suggest it could be a highly corrosive soft water. Any rnovation solution must consider the condition of the existing internal lining or the provision of cement mortar lining.

6 RECOMMENDED REHABILITATION APPROACH

6.1 The likelihood of failure

Any strategy for dealing with the problems of the Tibito-Casablanca pipeline must address two major considerations:

- The consequences of failure of a major aqueduct supplying Bogota with water.
- The potential damage to property and persons as a result of failure.

Replacement or renovation solutions must consider the likelihood of failure and also its consequences. If the failure were to occur in the fields near Tibito this would be unfortunate but not disastrous. The cross links that have been inserted into the network would probably allow the water supply to Bogota to be maintained. On the other hand, a failure occurring in the section along 129th Street could cause a major disaster, with potential for loss of life and considerable disruption of the supply

The potential consequences of failures on this line make untenable the option of doing nothing now and repairing damage as and where it occurs. In our opinion it is going to be extremely difficult to find investigatory methods which will reliably predict the location of defective pipes.

The evidence we have on this line and the experiences we have reported on others lead us to state with certainty that further failures will occur if no action is taken.

We therefore strongly recommend that EABB put in hand measures to rehabilitate or replace the present pipeline either totally or in staged sections.

6.2 Possible strategies

Three possible strategies can be adopted:

1. Identify individual suspect pipes and remove them
2. Replace the pipe
3. Line the pipe

6.2.1 Identifying suspect sections

In the foregoing sections we have discussed the difficulties of accurately pinpointing the problem lengths which would need to be replaced. We see no foolproof method of monitoring and inspecting the line to identify all those areas which are liable to failure. A number of experienced investigators support this view.

Considerable expense could be incurred in further investigation, which may still not identify clearly the reasons for failure and will not identify suspect pipe sections with any certainty.

6.2.2 Replacing the pipe

Two broad possibilities for replacing the existing pipe or sections of it based on excavating a trench would be to:

1. Replace the existing pipe by excavating down to it, removing the old pipe and replacing it with a new pipe.

2 Install a new line adjacent to the old line which is left in position, possibly then utilising the existing pipe as a relief or secondary line.

Alternative 1 will probably be more expensive and disruptive than Alternative 2. Certainly many sections are in verges and medians where there appears to be sufficient room to install a parallel line. However there will be other sections where space is tight for the installation of a parallel line

6.2.3 Lining the pipe

Essentially the task will be to create a pipe within a pipe which will have both the hydraulic and structural characteristics needed to provide a long-term solution. This option offers a feasible and economic solution.

Advantages

Compared to any open cut solution there will be considerably reduced disruption.
Lining work will have a much lower public profile.
Work can done more quickly.
The lining pipe can be designed to be a stand-alone solution so further failures of the PCCP will be of no consequence.
A double pipe will add strength and make illegal tappings almost impossible.

Disadvantages

Some loss of capacity will result from the reduced cross-sectional area.
Sections of the line will need to be put out of service.

Whatever method of rehabilitation is adopted, an implicit requirement must be that the line will be capable of operation at the full head rather than at the present reduced pressure.

7 ALTERNATIVE RENOVATION METHODS

In this section we consider alternative methods of renovating the Tibito-Casablanca pipeline with a suitable lining. Any lining method has to fulfil structural, hydraulic, operational and pipe life needs.

Five basic approaches

(1) Slip lining
Sections of pipe are pulled or pushed into the old pipe and the annular space between the new and old pipe is grouted .

(2) Modified slip lining
Sections of pipe are reduced in some manner before being pulled or pushed into place and then reverted to their original diameter.

(3) Soft lining
A hose of fibreglass or polyester is treated with thermic resins and placed using pressure to form a close fit against the old pipe. By use of heat or ultra-violet light the resins react and set hard to form a hard structural liner.

(4) Segment or sectional lining
Sections of pipe are carried down the old pipe and placed in position.

(5) Sprayed coatings
Using reinforcing steel and high strength sprayed mortars a reinforced concrete internal liner is formed.

7.1 Slip lining

An access pit is opened up and some of the existing pipes removed to allow new sections of pipe to be jacked or pulled inside the old pipe. The liner pipes can be made from a variety of materials. Once in position the remaining annular space is grouted. This is a simple and cost-effective solution which will allow the insertion of a steel or a fibreglass pipe with the structural and hydraulic characteristics needed.

However, although the horizontal profile of the Tibito-Casablanca pipe is for the most part in long straight lengths there are many vertical variations in the profile. These vertical changes in level will make it difficult to insert a pipe which approaches the existing diameter. It would only be possible to install a much smaller diameter pipe of, say, 54" or 60". Considerable cross sectional area and carrying capacity would therefore be lost.

7.2 Modified slip lining

Over the last few years techniques have been developed which allow a PE pipe to be temporarily reduced in diameter so that it can be placed in its reduced form inside the old pipe. This PE pipe is then restored to its original diameter to form a close-fit liner pipe inside the old pipe. Pipe joints are normally made in advance by thermal welding.

There are two approaches to modified Slip lining:

- Diameter reduction by drawing through a die or passing through rollers
- Diameter reduction by folding and forming the pipe into a "U" profile which is reverted after its positioning.

These techniques have been widely accepted in the water and gas industry and undoubtedly offer an effective and economic solution. However, although systems for larger diameters have been introduced, pipe as large as 78" has not yet been lined in this way. It should also be noted that much of the larger diameter lining that is now being undertaken is to provide a sealing lining rather than a structural stand-alone solution; the old pipe continues to provide the structural strength. A structural PE liner for this size of pipe would need to be of great thickness and would be difficult to reduce in diameter.

Examples of modified Slip lining systems are Swagelining, Rolldown and U Liner.

7.3 Soft lining systems

A soft hose, factory fabricated, is brought to site and treated with appropriate two-part thermal resins. Often this hose is inverted prior to insertion. The hose is placed in the pipe by pressure (usually water under a head) and reverts as it is forced in hard up against the wall of the old pipe. Once in position, the resins are catalysed by the use of heat, e.g., by heating the water in the line. This causes the resins to set hard to form a pipe structure. It is possible to design the wall thickness to provide a required structural strength.

Soft lining systems have been used for the rehabilitation of pipelines for nearly 20 years. Although initially developed for use in sewers, the technique is widely applied to pressure lines. The range of diameters that have been successfully lined include pipes larger than 78".

One reservation is the use of certain resins in a potable water supply situation, although a number of the companies concerned have been developing formulations which try to meet authorities' needs. Some systems have received limited approvals in some countries.

Certainly soft lining systems would appear to offer a possible solution and would have some advantages compared to other solutions.

Main advantages

Virtually no loss of cross-sectional area
Reduced access to the line requirements
Rapid installation

Main disadvantages

Unproven technology for potable water
Need to use imported and proprietary technology
Reservations about the structural performance in a PCCP
Expensive

The expense of soft lining systems is likely to be a determining factor compared with the alternatives. A budget price for an inversion lining of a 78" pipe would be of the order of $1200 to 1500 per metre.

The leading company in soft lining is the Insituform Group who originated the method. Several other companies offer similar systems. These include Paltem, KM Liner and In-Liner.

7.4 Sectional or segment linings

The concept here is to erect an in-situ lining along the pipe. Obviously this technique requires man-access for placement of the sections. Individual sections can be either in pipe form or can be segments which form a pipe.

7.4.1 <u>Segment lining</u>

Fibreglass, precast concrete and steel sections are well-established for lining sewers. Because of the problems of providing a lining capable of withstanding high operating pressures, the use segment lining in pressure lines is minimal. It is theoretically possible that a system similar to a tunnel lining system, albeit lighter, could be used to form a lining. The segments could be purpose-made of concrete or steel with bolted joints and a pressure lining could be created with the use of suitable sealant between joints. Special sectional pieces could be fabricated to follow the changing vertical profile.

We have not pursued this option as we believe the problems of providing a pressure lining and the cost will make segment lining a non-starter.

7.4.2 <u>Pipe section lining</u>

An alternative is to take sections of pipe and transport them down the line to where they are required, place them in position, make joints between sections and grout up the annular space. If properly designed this approach has advantages:

Advantages

All materials and labour skills are available locally.
A tighter fitting liner can be installed with less loss of cross section than with slip lining.
Purpose-made sections can be made to match the existing profile.
A new pipe section of the required strength can be provided.

Disadvantages

A greater number of joints with more potential for leaks than with slip lining.
Joints have to be made in confined areas.
Some loss of cross section.

This approach has been widely used on the rehabilitation of pressure water lines, where either steel or GRP pipes have been employed. In the former case steel pipes are fabricated, carried into position and jointing is by in-situ welding. The length of individual sections that can be conveniently transported and placed will depend on a number of factors, including equipment available. The longer the sections the less the number of joints that have to be welded in the line. However, practical considerations of handling, access excavations and transportation will all be factors that need to be taken into consideration.

In the case of the Tibito-Casablanca aqueduct, we consider that the largest pipe that can be conveniently inserted would be a standard diameter of 1825 mm i.d.. It may be possible to use pipe sections of 3 metres. Shorter sections may have to be used depending on the problem of passing them through horizontal and vertical changes in line and level. A check should be meade before starting work on the rehabilitation of any particular section.

In the case of GRP pipes a low-profile pressure joint is used. We have obtained a budget price for the supply of 1800 DN GRP liner pipes in 2 m lengths with collar joint to suit 14 bar pressure. The cost of the pipe ex-works is more than $900 per metre. With placing and back grouting we estimate the all-in placed cost is unlikely to be less than $1200 per metre.

7.4.3 Scrolled steel pipe

A variation on carrying complete steel pipes into the line is for a cut to be made along the crown line and the pipe to be scrolled to reduce the diameter for easier transport. Having transported the section to its final position, the pipe is unscrolled and the joints, including the crown seam, are welded in-situ.

Main advantages

All material and labour skills are available locally.
A closer fit can be obtained than with sections thus reducing loss of cross section.
Purpose-made sections can be used to match the existing profile.

Main disadvantages

Labour-intensive solution.
Increased number of site-welded joints, including a longitudinal seam.
Slower than slip lining or pipe section lining.

The scrolled solution using steel pipe has been successfully used in similar situations.

7.5 Sprayed-on linings

Ferrocement is sprayed-on lining which forms a reinforced internal liner in both sewers and large pressure lines. The technique is only suited to pipes which allow man access. A ferrocement lining requires an appropriate steel reinforcement to be fixed to the wall of an existing pipe. The high strength mortar is then sprayed around the steel to form a "reinforced" concrete lining wall of suitable thickness. The mortar is mixed remotely at ground level and transported by pressure line to the point of application.

The action of a true ferrocement lining is to minimise surface cracking to 20 micron or less under maximum loading condition. This is achieved by limiting the working stress in the steel and for a fine mesh structure to be placed very near to the surface of the completed lining. The action of this construction is to ensure that cracking is maintained at the minimum level to prevent ingress of water, which could bring about corrosion of the steel. Precast linings can also be manufactured from this material, but are not applicable to pressure situations.

The system was devised initially for the renovation of sewers where multiple layers of wire gauge steel reinforcement were used to both provide crack control and the load bearing members. The system was later developed to take account of larger diameter pipelines with higher loadings, including water pipelines with high internal pressures. In these high load situations conventional steel reinforcement is provided to resist the internal pressure, whilst the wire gauge mesh is solely to ensure crack control.

Although we have not undertaken a detailed design, we estimate that the thickness of the lining will be not less than 75 mm with substantial main reinforcement. Thus the rehabilitated internal diameter will be 1850 mm..

The potable water will need more careful anlalysis, as it may be necessary to use a polymer enriched surface coating to protect the interior surface.

The system is adaptable to large diameter and high stress situations, although its use in this context has been fairly limited worldwide.

Main advantages

- Materials are readily available.
- Steel can be prefabricated to ease installation.
- Changing profiles can be easily accommodated.
- No lining to the pipe is required (but see note above).
- Only limited access points are needed.

Main Disadvantages

- Will require importation of some specialist equipment and personnel.
- Some loss of cross section.

Because the basic components require no specialised equipment, the system can be relatively economical. Depending on the load requirement and the design, the cost could range from $800 - $1200 per metre for this size of pipe and the amount of work to be done.

8 RECOMMENDED OPTIONS

8.1 Recommendations

From the alternative options discussed in the previous section we propose that the rehabilitation be undertaken by provision of a structural lining. Consideration should firstly be given to a steel liner which is installed either as pipe sections or in scrolled form. Of the other alternatives we consider a sprayed-on ferrocrete liner would provide a possible alternative to a steel liner.

As no recognised design procedures exist for this situation, assessing the method/approach to be adopted presents a little difficulty. Consequently in the design calculations offered as a preliminary proposal, the lining has been designed as a steel pipe to be installed in a trench with modifications and assumptions made to account for the lining of an existing pipe.

The existing pipe will continue to be attacked from whatever influences are currently causing corrosion problems within the pipeline. It has been assumed that the existing pipe will continue to provide a high degree of support to the new lining, which will be installed and then backgrouted to provide a "composite" structure. In the short to medium term it is most likely that the existing pipe will continue to carry external backfill and surcharge loads. In the long term deterioration may occur such that the lining will be required to carry all external loads as well as the internal pressure. On this basis the lining has been designed to withstand all loads. It has been assumed that the high degree of support and intimate contact with the surrounding ground will continue to be afforded to the pipe lining throughout its life.

A more difficult question is the extent of the pipe to be rehabilitated and the priority of individual sections.

<u>Priority sections</u>

From statistical and historical points of view the section of nearly 16 km between Km 13 to Km 29 appears to be a priority. The section between Km 13 back to Km 5.8 (where the pipeline leaves the road) is probably a second priority. One failure has already occurred in what was a field area around 4.1 km. As this area is now being developed, any failure here is going to endanger to life and property. It would therefore appear desirable to reline at least some length back from the road towards Tiberio plant.

If emphasis is placed on consequences of failure, the sections along Street 129 and into Avenida Boyaca are priorities. One failure has already occurred in Avenida Boyaca.

However the section beyond the river, further south along Avenida Boyaca to Casablanca, has no history of failures to date and less aggressive soil conditions are evident. Experience would indicate that lines under paved areas are generally less prone to failure, which may also mitigate the need for early renovation. We understand that EAAB do not propose to operate the last 20 km approaching Casablanca at pressures greater than 70 psi and also propose to install a 48" line which will provide an alternative supply aqueduct to the western part of the city. It can be argued that consideration of remedial work on this 20 km section could be deferred until such time as the 48" line is installed. It may then be possible to undertake a thorough internal inspection of this section of the line including the use of "impact-echo" techniques to evaluate the condition of the prestressing wire.

The 31 km lying between Km 4 and Km 35, either for statistical reasons or because of the severe consequences of failure, should be relined as soon as is feasible. The remaining 20 km from the river towards Casblanca can be deferred until the opportunity for a detailed investigation will allow a better judgement to be made on the state of the line. There is an initial section of about 4 km from the reservoir and through fields which is likely to prove troublesome when operating pressures are increased. This could be included in the initial contract or could be planned as a second stage operation, depending on availability of funds and operational considerations. For operational reasons, EAAB may wish to lay a parallel line from the reservoir through the fields.

We recommend that a detailed plan be drawn up with EAAB which would take into account the above factors as well as operational considerations.

8.2 Theoretical calculations

Outline design calculations for a steel liner are attached as Appendix 3. These have been based on the most rigorous operational scenario with a maximum working pressure of 13 bar and associated test pressure of 19.5 bar.

These pressures have been adopted from the documentation supplied and represent the most onerous condition. No account has been taken of any surge induced or air inclusion transient pressures as no indication of the level of such pressures can be determined without the execution of a detailed surge analysis.

The preliminary design calculation indicates that a steel wall thickness of 13 mm will suffice, working within a limiting steel wall stress of 275 N/nm^2 and a maximum deflection of 2%.

8.3 Operational considerations

Under the flow conditions calculated (See Appendix 3), the velocity in the pipe varies between 1.59 and 1.89 m/sec. For this diameter the velocity is not excessive and could approach 2.5 - 3 m/sec without problems in respect of abrasive action. It will be necessary to renew valves and other apparatus at the time of renovation. Valves should be selected such that they can be closed smoothly within the operational requirements to restrict the introduction of surge or other transient pressures.

8.4 Installation considerations

It will be essential to thoroughly clean and check the internal pipe before commencing any renovation works. Particular attention should be paid to variations in internal diameter. Any reductions in diameter will need to be accommodated in the liner pipe. The treatment of the line at valves, vents and tees will need special measures and purpose-made sections. It should be possible to connect most fittings such as valves by the provision of fabricated flanged pieces. These will need to be precisely fabricated or made with a joint which provides a degree of adjustment. This work will be carried out in open excavations, so should not prove unduly difficult. For washouts, special steel sections incorporating a washout connection will have to be prepared. [See our later comment on the advisability of replacing or rehabilitating fittings at the time of the line renovation].

To install a sectional or scrolled steel tube lining will require access to be made at a number of points along the line. Obviously the flows along the sections to be rehabilitated will have to be diverted. It will be necessary to draw up a detailed program based on the physical characteristics of the pipes, location of bypasses, access points and fittings. It may be necessary to install some additional cross links in the system in advance of rehabilitation. We estimate that on average about 3 access points per kilometre will be required. The actual individual lengths that can be lined from any access point will vary considerably.

Tubes can be factory-fabricated and prepared ready for installation. The steel tubes should be cut in the factory, using a suitable cutting tool to provide a smooth profile. The cut edges can be prepared for in-situ welding at the same time. If a scrolled solution is to be used, these edges should be protected on 'folding' of the pipe.

A purpose-built power carrier can be developed which will carry either type of sections along the line to where they are to be installed. At changes in direction purpose-built sections can be made up. The carrier must be capable of operating with minimum clearance and be easily removable from under the section after it is in position.

For scrolled pipe, a jacking device should be incorporated into the carrier for reverting the sections to their original diameter. The pipe should be unfolded in the existing pipe to ensure proper alignment of the pipe edges. A jacking device will be necessary to reshape the pipe and restrain it in position to allow tack welds to be made to ensure the pipe's stability prior to full welding of the longitudinal and circumferential joints.

Preferably, the longitudinal joints should be staggered with the joint being outside the 11 o'clock to 1 o'clock arc at the roof of the pipe.

Welding for pipe sections (circumferential) or scrolled (circumferential and seam) could be carried out using automatic equipment. On completion the welds must be tested by X-ray or ultrasonic means to ensure that a correct joint has been achieved.

The annular space should be grouted in lengths 30-50 m, with provision for removal of air from the annulus. This could be achieved from the end of the pipeline or by drilling through the steel lining and inserting grout through these grout holes around the circumference. It will be necessary to suitably plug these holes on completion. Attention should be paid to possible flotation of the steel tube during grouting.

The most suitable form of protective lining to the steel pipe needs to be checked out. The analysis of the water suggests that sprayed mortar lining of the type presently being used may not be advisable and that an epoxy lining would be more suitable.

We estimate that a single team lining this system with pipe sections should be capable of installing 200 to 250 metres per week of lining complete. Using scrolled sections we consider that progress of 150 to 200 metres a week will be possible. In either case this will obviously depend on the efficiency of the management and the workers as well as the specialist equipment. This operation is essentially a logistics challenge.

American Pipe have submitted proposals for installation of a scrolled pipe liner. Their proposals are based on experience of similar installations in the USA. They have recently revised their original proposals submitted in 1989. Broadly, their proposals appear sensible.

8.5 Estimated cost of renovation

Without a detailed agreement on the length of work to be undertaken and the sequence of sections, we are only able at this stage to put forward an outline budget cost for the works. We are also currently awaiting additional information on steel tube prices in Colombia. At this time, therefore, we are putting forward preliminary cost estimates until we can obtain the additional information required to produce a more detailed costing.

Lining Using Steel Tube Sections

Items	Outline of costs	US $
1	Preparation of access shafts and removal of pipe sections Assuming 3 shafts per km and a cost per shaft of $20 000 gives a total of $60 000 or $60/metre	60.00
2	Provision of 1825 i.d. steel pipe with 13 mm wall thickness. Weight of pipe 580 kg per metre. At $1 per kg, the price per metre is	580.00
3	Transport to site and installation of sections, welding in position per metre	50.00
4	Grout up annular space between new and old pipe per metre	30.00
5	Line steel pipe with mortar lining or epoxy resin, allow	50.00
	Total cost of installation per metre	770.00

NB. These budget prices allow for overhead and profit, but not for taxes.

Lining Using Scrolled Steel Sections

Items	Outline of costs	US $
1	Preparation of access shafts and removal of pipe sections Assuming 3 shafts per km and a cost per shaft of $20 000 gives a total of $60 000 or $60/metre	60.00
2	Fabrication of 1875 i.d. steel pipe with 13 mm wall thickness. Weight of pipe 600 kg per metre. At $1 per kg, the price per metre is	600.00
3	Transport to site and installation of sections, welding in position per metre	60.00
4	Grout up annular space between new and old pipe per metre	25.00
5	Line steel pipe with mortar lining or epoxy resin, allow	50.00
	Total cost of installation per metre	795.00

NB. These budget prices allow for overhead and profit, but not for taxes.

We have both the original and revised estimate of American Pipe. We find it difficult to see from the information provided how the second estimate was updated compared to the earlier one. The original estimate looks low particularly for the fabrication of the steel tubes.

Although our estimate appears to be higher than that of American pipe, this is mainly due to our use of a 13 mm wall pipe compared to 7/16". A closer consideration of the design, taking into account all circumstances, may allow the use of a pipe slightly thinner than 13 mm.

Apart from this variation, the American Pipe estimate compares reasonably closely with our own.

The decision to rehabilitate the line by using steel tube section or scrolled sections essentially hinges on the acceptability of losing a further 50 mm internal diameter. If a reduced diameter of 1825 mm is acceptable, then we believe that the elimination of the site welding of the longitudinal seam would be a definite advantage.

8.6 Renovation of pipeline appurtenances

Although these lie outside our brief, it would be logical that any proposed rehabilitation of the pipe should also take the opportunity to renovate or replace valves, vents, washouts and other appurtenances.

APPENDIX 1

EMPRESA DE ACUEDUCTO Y ALCANTARRILLADO DE BOGOTA
GERENCIA DE OPERACIONES

COMITE
LINEA DE CONDUCCION TIBITO-CASABLANCA

Tuberia PCCP D=78" (2.00 metros)

Relacion de roturas en la linea

Rotura No.	Dias entre Roturas	Fecha	Abscisa	Clase PSI \Tubo	Cota Terreno Metros	Plano No.	Tubos Retirados	Tiempo Repar. Horas	Sector [Localizacion]
1		11/01/78	K20+700	190 3126	2577	52	UNO	82	Autopista Norte Entrada Club Millos
2	662	07/11/79	K24+900	|190 3726	2583	61	UNO	76	Autopista Norte Calle 183
3	859	15/03/82	K4+130	170 741	2586	14	UNO	68	Autopista Norte Reten del Intra
4	407	27/04/83	K28+166	180 4196	2582	68	UNO	48	Autopista Norte Al Norte de la 134
5	241	28/12/83	K13+075	170 2022	2589	34	UNO	31	Autopista Norte Sector la Caro
6	527	09/06/85	K23+149	190 3475	2578	57	DOS	28	Autopista Norte Cerca Club de Rancho
7	861	19/10/87	K2+800	150 0411K	2575	85	DOS	28	Avenida Boyaco Calle 98
8		02/04/88	K16+624	150 2540	2583	43	SOLDA DURA	4 *	Aut. Norte al Sur Ceram. Mogio. por Inspecc.
9		04/04/88	K18+442	150 2798	2583	47	DOS	14 *	Autopista Norte Cerca al Buda por Inspecc.
10	587	05/06/89	K20+585	190 3110	2577	51	UNO	12 *	Autopista Norte (CL.200) Al Norte de la. Falla
11		06/20/89	K24+544	190 3675	2577	60	UNO	10 *	Autopista Norte por Investigacion

(1) El numero del tubo es aproximado.
(2) Las fallas 8, 9 y 11 fueron detectadas por investigacion.
* Tiempo aproximado de los trabajos sin incluir servicio

[TIBSCAB]

APPENDIX 2

FINAL REPORT OUTLINE
PERFORMANCE OF PRESTRESSED CONCRETE PIPE

I. Introduction
   A. Background - Performance Survey
   B. Research Required
   C. Research Priorities
   D. Research Undertaken

II. Research Performed
   A. Condition Assessment
      1. Review of Theory and Available Methods
         a. Nondestructive techniques
         b. Destructive methods
         c. Novel techniques and methods
      2. Techniques and Methods Empirically Evaluated
      3. Results of the Empirical Evaluations
      4. Economics of Methods and Techniques
      5. Promising Novel Techniques and Methods
   B. Rehabilitation
      1. Mechanical and Structural Requirements for Rehabilitation Schemes
      2. Rehabilitation Schemes Empirically Evaluated
      3. Efficacy of Rehabilitation Schemes
      4. Economics of Evaluated Rehabilitation Schemes
      5. Limitations of Rehabilitation Schemes
   C. Corrosion Control
      1. Available Methods
      2. Methods for Distressed Pipe
      3. Methods Applicable to Pipe Repairs
      4. Methods Evaluated Empirically
      5. Efficacy of Methods
      6. Economics of Evaluated Methods
      7. Limitations and Risks of Methods

III. Summary and Conclusions
   A. Literature and Industry Survey
   B. Laboratory Evaluations
   C. Field Applications
   D. Conclusions

IV. Recommendations

RECLAMATION/AWWARF COLLABORATIVE RESEARCH
PERFORMANCE OF PRESTRESSED CONCRETE PIPE
Fourth Quarterly Progress Report
January, February, and March, 1992

WORK ACCOMPLISHED

Condition Assessment Research

1. Pipe Inventory Survey - Work continued on processing information that was gathered from pipe inventory questionnaires distributed through Reclamation to water users. A data base was set up and the data were input. The questionnaire was tailored by AWWARF for distribution to other water using entities within the pipe industry. Pipe classes were listed relative to AWWA standards and a glossary of pipe types was included to facilitate data gathering. The glossary provided a brief description of each pipe type along with a typical range of diameters and pressure heads. The questionnaire limited responses to buried water transmission lines measuring 24 inches or larger in diameter and 1/2 mile or more in length. AWWARF plans to distribute the questionnaire to approximately 500 water user organizations next quarter. Completed responses will be due by June 1, 1992.

2. Nondestructive Assessment - Work continued in the development of a continuous monitoring technique which utilizes a system of hydrophones (underwater microphones) that listen for the sound generated by the fracture of a prestressing wire(s). Three newly designed hydrophones were successfully installed in the Agua Fria River siphon of the Central Arizona Project by the wet-tapping process. Field tests were conducted to evaluate their performance. Preliminary testing indicated a significant improvement over the original design in that the overall sensitivity was increased.

Reclamation personnel met with staff members of the Naval Research Laboratory to solicit their expertise in underwater acoustics. Several promising suggestions were made which may prove beneficial in the development of future systems.

3. Central Arizona Project Investigations - Work continued on redrafting the report pertaining to the cause(s) and extent of distress on six Central Arizona Project siphons.

Repair and Rehabilitation Research

1. Tendon repairs - Data from a storage module retrieved from an instrumented tendon repair on the Central Arizona Project were examined. The repair was performed and instrumented while the siphon remained in service, allowing water deliveries to continue, as tendons were wrapped over mortar coating and broken and corroded prestressing wire. Examination of the stored data revealed that load monitoring instrumentation had been damaged and that valid data were no longer being obtained. Although precautions were taken to protect the instrumentation from construction activities, load cells likely were damaged during shotcreting of tendon anchorage assemblies. Insitu testing was terminated and the data acquisition equipment retrieved.

In pursuit of the timely release of information, two reports pertaining to tendon repairs are planned in the current scope of work. The first report is to provide information relative to repairs performed after siphon dewatering and is scheduled for release in June 1992. The second report is to provide information relative to repairs conducted while the siphons remained in service and is scheduled for release in May 1993. To evaluate the longevity of the in service repair (i.e. the effect of wrapping over wire that may to continue to corrode and mortar coating that may disbond), monitoring was planned for a two year period. However, the recent discovery of damaged instrumentation has rendered this no longer possible. The two reports will therefore be combined into one report with a revised release date of March 1993. An earlier release date is not feasible as additional research is still needed to ascertain whether the implementation of future tendon repairs should be governed by the location of the fractured prestressing wire.

Design and Protection Research

1. Prestressing Wire Failure Mechanisms - Work was conducted on all five of the research phases as follows:

Phase I: Determination of the Threshold Concentration of Solute (Internal) Hydrogen that will Produce Delayed Wire Fracture as a Function of Sustained Loading

The analytical equipment used to determine the concentration of hydrogen in wire samples was repaired and calibrated. Calibration standards were analyzed with all values falling within the specified limits. Dummy samples were analyzed to determine the residual hydrogen content with consistent and reasonable results.

Phase II: Identification of the Rate of Hydrogen Absorption as a Function of Environmental pH and Cathodic Charging

Work in this phase was conducted, but problems were experienced with the hydrogen analytical equipment and delivery of some of the testing equipment (i.e. potentiostats etc.). Problems with the hydrogen analytical equipment were resolved and the test equipment is currently scheduled for arrival in May.

Phase III: Evaluation of Wire Fracture Toughness Parameters

In addition to sharp crack fracture toughness evaluation (i.e. developing methods to produce sharp tipped cracks of controlled size so that the stress concentration effects produced by blunt notches in the wire may be determined), a literature search was conducted to obtain further information on fracture toughness parameters. The search revealed useful information pertaining to the notch sensitivity of a material. Such information will be utilized for future research in this phase.

Phase IV: Analysis of the Effect of Complex Stress State on Fracture Loads and Fractographic Features

A fixture was produced that was to be used in conjunction with a torsion machine to apply complex stress states (biaxial and triaxial) to prestressing wire. When an exemplar specimen was tested, however, the magnitude of the torsional force did not register on the testing machine. Testing was therefore terminated and repair of the torsion machine pursued.

Phase V: Determination of Time Dependent Mechanical Properties as well as Wire Relaxation and Rupture Potential

Calibration of equipment that will be used to determine the stress-relaxation rate of prestressing wire under sustained loads was initiated.

OVERVIEW OF WORK FOR NEXT QUARTER

Condition Assessment Research

1. Pipe Inventory - The pipe inventory questionnaire will be distributed by AWWARF to the water users. Incoming data will be input and compiled in the data base.

2. Nondestructive Assessment - Preparations are under way for field testing to obtain wire break data utilizing the new hydrophones. The testing should provide the additional information needed for improved classification and more accurate determination of wire break locations. Specifications are also being prepared for acquisition of an acoustic monitoring system (AMS) for the Agua Fria River Siphon. This system will replace the existing equipment with a complete, integrated system that will provide extensive signal processing capabilities and continuous monitoring of an array of hydrophones. The system is projected to be completed in early 1994.

3. Jordan Aqueduct Failure Investigations Report - Management review of the Jordan Aqueduct report is ongoing. The report likely will be reviewed and published next quarter.

Repair and Rehabilitation Research

1. Tendon Repairs - Work will be initiated on the second of three tests to check the validity and assumptions of the Reclamation finite element model. Model predictions have demanded that prestressing wire and mortar coating be completely removed from the pipe perimeter prior to performing a tendon repair, if wire from the distressed pipe has fractured near springline. It is anticipated that this testing may ultimately lead to an in-service tendon repair which may be performed under any circumstance regardless of wire fracturing location. Instrumentation procurement and development of a plan to strategically locate instrumentation on exemplar pipe will be pursued.

Design and Protection Research

1. Prestressing Wire Failure Mechanisms - Work will continue on all five of the research phases as follows:

Phase I: Determination of the Threshold Concentration of Solute (Internal) Hydrogen that will Produce Delayed Wire Fracture as a Function of Sustained Loading

An alternative method of determining the diffusible hydrogen content of charged wires will be evaluated. The method is based on the Welding Institute's International Standard for measuring the diffusible hydrogen in test welds; namely, ISO 3690. Round robin testing at various laboratories confirmed that the method provides consistent and reproducible results of high accuracy. The advantages of this method in current investigations is that it allows the determination of the diffusible hydrogen without having to store the samples at low temperatures and duplicate samples can be subjected to mechanical testing immediately. Results can be confirmed by cross checking the hydrogen contents of selected samples against those determined by the recently calibrated analytical equipment.

Phase II: Identification of the Rate of Hydrogen Absorption as a Function of Environmental pH and Cathodic Charging

The first set of tests will be initiated as uncharged samples will be loaded and subjected to various potentials and solutions. Uncharged samples will be loaded to 75 and 90 percent of the minimum specified tensile strength and subjected to potentials of 500, 900, and 1200 mV with reference to a saturated copper/copper sulfate electrode. The wire will be suspended in solutions containing pH's of 7, 9, and 12. All tests will be reproduced in triplicate and a total of 54 samples will be tested.

Phase III: Evaluation of Wire Fracture Toughness Parameters

Work will continue in the development of methods to produce sharp tipped cracks of controlled size and the testing of such samples. Test methods to determine the notch sensitivity of prestressing wire will also be examined. The influence of other parameters such as the effects of temperature will be researched.

Phase IV: Analysis of the Effect of Complex Stress State on Fracture Loads and Fractographic Features

Repair of the torsional testing machine will be pursued. In addition, consideration will also be given to a method whereby a measured torque can be simultaneously applied to a sample under tension. Such data would be very useful in evaluating the tendency of the prestressing wire to behave in brittle fashion under triaxial stress conditions. The apparatus, if developed, would be used in addition or lieu of the fixture requiring the torsion machine.

Phase V: Determination of Time Dependent Mechanical Properties as well as Wire Relaxation and Rupture Potential

Stress rupture testing will begin as wire will be subjected to high loads for extended periods of time to determine the rate of stress-relaxation.

PROBLEMS ENCOUNTERED

Condition Assessment Research

None.

Repair and Rehabilitation Research

Tendon Repairs - The Arizona Projects Office has experienced problems in contracting with Miller Pipeline, Inc. to perform excavations around exemplar Central Arizona Project pipe. Miller was to demonstrate their techniques for removing a minimal amount of backfill material beneath springline. Although one such technique has already been implemented on several Central Arizona Project repairs, alternative methods which promote market competition and require less material removal are being sought. Apparently, there is a problem pertaining to a liability issue. It is unknown at this time whether or not the demonstration will take place.

Design and Protection Research

Prestressing Wire Failure Mechanisms Research - Problems were experienced in receiving some of the testing equipment for the phase II research thereby causing a delay in the initiation of tests. Since the equipment is scheduled to arrive in May, the minor delay will not affect the targeted completion date. Problems were also experienced in phase IV research with the break down of the torsion machine. At this time, the cost of repairing the machine is unknown. If machine repair is too costly or parts cannot be obtained, alternative testing methods will be pursued.

QUARTERLY RESEARCH (WORK) PROGRESS REPORT QUARTER 2 FY 92

PROJECT NUMBER: NM056, AWWA1 FY ALLOCATION: $204,525

TITLE: Performance of Prestressed Concrete Pipe

RESPONSIBLE CODE: D-3730

PRINCIPAL INVESTIGATOR: Michael T. Peabody, Harry K. Uyeda

TASK (FROM PROJECT PLAN)	ORIGINAL PROPOSED TIME FRAME (MM/YY-MM/YY)	REVISED TIME FRAME (MM/YY-MM/YY)	PERCENT COMPLETED (QUARTER/TOTAL)	COMMENTS (INCLUDING MAJOR ACCOMPLISHMENTS, TECHNICAL OR ADMINISTRATIVE PROBLEMS ENCOUNTERED)
Pipe inventory questionnaire	3/91-10/92		(5/45)	Data being gathered from questionnaire
Nondestructive testing	5/91-5/93		(5/55)	Hydrophone work continues
Jordan Aqueduct failure investigations report	1/91-1/92	6/92	(0/95)	Report is undergoing management review
CAP PCP investigations report	1/91-6/92		(40/50)	Report is being completly revised
CAP PCP tendon repair report-pipe dewatered	1/91-6/92		(0/50)	Testing and monitoring completed
CAP PCP tendon repair report-pipe in service	5/91-5/93		(5/40)	Test data retrieved and reviewed
PCP repair techniques report	5/91-9/92		(0/100)	Report issued but external release being held up
PCP pipe strength report	1/91-2/93		(0/40)	Data reduction for 6 month test completed
PCP soil-cement/soil interaction report	1/91-9/92		(0/20)	Testing and analysis completed, report draft required
Prestressing wire failure mechanisms	5/91-8/93		(5/15)	Work continues on phases I, II, III, IV and V
Computer model comparisons study	6/91-5/93		(0/5)	SPIDA finite element model obtained

FY Costs+Obligations to date: $202,977

Anticipated FY funding excess (+) or deficiency (-): $0

APPENDIX 3

DESIGN CALCULATIONS

Structural design of pipeline using steel pipes

Assume pipe wall thickness of 13mm on 1820mm OD

$$\text{Stiffness} = EI/D^3$$

$$= \frac{200 \times 10^9 \times 13^3}{12 \times (1820 - 13)^3}$$

$$= \frac{200 \times 10^9 \times 13^3}{12 \times (1807)^3}$$

$$= \underline{6206 \text{ N/m}^2}$$

Consider maximum loading of 4m

Backfill + heavy traffic loading = 101 kN/m^2

1. Check buckling stability

Timoshenko buckling equation

$$P_{cr} = \frac{24EI/D^3}{(1-m^2)}$$

$$EI/D^3 = \frac{P_{cr}\,(1-m^2)}{24}$$

EI/D^3 = pipe stiffness and the stiffness required to provide a factor of safety of 1.5 is

$$EI/D^3 = \frac{P(1-m^2)}{16}$$

where P = external design load (backfill only)
m = Poisson's ratio = 0.3

Q $$EI/D^3 = \frac{80\,(1-0.3^2)}{16} \times 1000 \text{ N/m}^2$$

$$= 4550 \text{ N/m}^2$$

Stiffness provided is 6206 N/m^2 Q pipe OK

2. Check deflection

Consider pipe unrestrained by existing conditions.

Maximum external design loading pressure = 101 kN/m^2

$$\frac{a}{D} = \frac{K (D_L P_b + P_S) \quad x \; D_R}{8EI/D^3 + 0.061E^1}$$

where
a = deflection of pipe vertical diameter
D = pipe diameter
D_L = deflection by factor = 1
D_R = rerounding factor (not applicable for d > 2.5m)
P_b = vertical pressure due to backfill
P_S = vertical pressure due to surcharge
EI/D^3 = pipe stiffness
E^1 = modulus of passive resistance of surround
K = bedding constant say 0.083

$$\frac{a}{D} = \frac{0.083 \times 101.1}{(8 \times 6.206)+(0.061 \times 15000)}$$

$$= \frac{8.39}{49.65 + 915}$$

$$= \underline{0.9\% < 2\%} \qquad \text{Q OK}$$

3. Check wall stress

Combined stress = stress due to pressure + stress due to bending

$$= P_i (R_i/t) + 6AP_v (R/t)^2$$

where
P_i = internal pressure
P_v = external vertical pressure (backfill + surcharge)
R = pipe mean radius
R_i = internal radius
t = pipe wall thickness
A = $0.250 - \dfrac{0.0145}{(8EI/D^3)/E^1 + 0.06}$

$$= \frac{P_i D_i}{2t} + 6\left[0.25 - \frac{0.0145}{\frac{8\ EI/D^3}{E^1} + 0.06}\right] P \left[\frac{D}{2t}\right]^2$$

$$= \frac{1.3 \times 1794}{2 \times 13} + 6\left[0.25 - \frac{0.0145}{\frac{8 \times 0.006206}{13} + 0.06}\right] \frac{101}{1000} \left[\frac{1807}{2 \times 13}\right]^2$$

$= \quad 89.7 + 6(0.0228)\ 487.9$

$= \quad 89.7 + 66.7$

$= \quad 156.4 < 275$ Q OK

For test pressure

$P_i = 1.95$

and stress $= 134.55 + 66.7$

$= 201.25\ N/mm^2$

Hydraulic design of pipeline

It is very difficult to assess the full capability of the existing system without a detailed hydraulic analysis being carried out. Adopting a very simplistic approach and using information on the hydraulic profile detailed in Drawing No. 5600-DRM-001 the following calculation has been prepared. This assumes a constant hydraulic gradient between the tanks and calculates the likely discharge through the pipe.

The calculation has been based on a lining pipe 1820mm OD, 13mm wall thickness and 25mm minimum internal cement mortar lining. The calculation has been carried out assuming draw off from the maximum level of the tank at Tibito filling to the higher level at Casablanca.

Hydraulic calculation

$$Q^{1.85} = \frac{a\ C^{1.85}\ D^{4.87}}{1.2 \times 10^{10}\ L}$$

where Q = discharge (l/sec)

a = head loss (m)

C = Hazen Williams C value

D = pipe diameter (mm)

L = pipe length (m)

1. Higher level tank at Tibito

With draw off from tank full (2698.84 level)

$$Q^{1.85} = \frac{61.34 \times 120^{1.85} \times 1744^{4.87}}{1.2 \times 10^{10} \times 51971}$$

$$= \frac{61.34 \times 7022 \times 6.114 \times 10^{15}}{1.2 \times 10^{10} \times 51971}$$

$$= 4.223 \times 10^{6}$$

$$\mathbf{Q = 3.81\ cumec}$$

This discharge does not take account of any draw off along the route of the pipeline.

Repeating the calculation over a shorter length to the first draw off point at 27 850 produces a discharge of 4.51 cumec. Basing on a 1875 mm steel liner, the flow would increase to 5.42 and 6.42 cumecs respectively which we understand would meet EAAB's requirements.

References

1. Edmunds, Robert C., *Pipeline design lessons of recent prestressed concrete pressure pipe failures.*

2. Price, Robert E., *The investigation, cause and prevention of PCCP failures.*

3. Clift, James S., *PCCP - A perpective on performance.*

4. Laderman, Michael R., & Patel, Suresh, *Large diameter pressure pipeline trenchless rehabilitation.*

5. Peris, Manuel Garcia, *Corrosion problems in prestressed plate-lined concrete piping.*

6. Bianchetti, Ronald L., *Corrosion and corrosion control of prestressed concrete cylinder pipes — a review.*