Engineering Failure Analysis 33 (2013) 388–397

Contents lists available at SciVerse ScienceDirect

Engineering Failure Analysis

journal homepage: www.elsevier.com/locate/engfailanal

Loss of a 30″ directional crossing due to pipeline collapse during pullback

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ARTICLE INFO

Article history:
Received 13 April 2013
Received in revised form 6 June 2013
Accepted 10 June 2013
Available online 19 June 2013

Keywords:
HDD crossing
Pipe collapse
Tunnel loss
Failure during pullback

ABSTRACT

This article concerns the loss of a Horizontal Directional Drilling crossing due to a failure that took place during pullback of the pipeline. The more than 500 m long drilling was meant to lodge a 30″ pipeline that crossed a large river. The failure caused the total loss of the crossing, and the necessity to repeat the entire process in another, less favorable location. Root causes involved a difficult soil composition, and constraints at a populated bank that led to engineering solutions not proved before on a pipe of such diameter. Discontinuities on the tunnel and the use of a sacrifice pipe at the front end of the pipe column, which was related to the need for more pipes than originally calculated because of a series of problems at the beginning of the perforation, were also defined as root causes. Lessons learned are highlighted.

1. Introduction

Horizontal Directional Drilling (HDD) is used for installing utility lines, ranging from large-size pipeline river crossings to small-diameter cable conduits, with no need of a trench [1]. Directional drilling has inherent capabilities. Massive river crossings, high profile locations and large diameter projects have been possible due to innovation. Drill operators and manufacturers are finding new and creative ways of tackling tough projects and difficult situations [2,3].

This is a technology that not only reduces costs but results in a much lesser environmental foot prints, to such an extent that open trench crossings are in many places forbidden by local regulations [4]. HDD relies on a tunnel that is made with a cutting tool in front of a pipe string, called reamer [5]. The tunnel is dag and successively enlarged by rotating reamers of increasing diameter, which detach soil particles with a combination of drilling fluid (also called mud), that flows under high pressure through directional nozzles, and cutting tools.

HDD applications have evolved from technologies originally developed in the construction of oil and gas wells. Horizontal wells can be very long; world record belongs to a Qatar oil well with a hole of more than 12.000 m. The introduction of new techniques has allowed existing constraints to be successfully overcome. This is achieved through sound engineering principles and optimization during the Field Development Plan [6]. In order to provide guidelines to ensure public safety and protection of existing underground facilities, different aspects of the design and construction of an HDD have been addressed by the industry. In 1995, the Pipeline Research Committee at the American Gas Association published a manual titled Installation of Pipelines by Horizontal Directional Drilling, an Engineering Design Guide [7].

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http://dx.doi.org/10.1016/j.engfailanal.2013.06.001

Guidelines are not intended to be a step-by-step procedure manual but rather a collection of fundamental elements of the HDD process. For example, minimum vertical and horizontal clearances with existing features are important for an HDD in urban soil, in order to minimize the chance of damage to existing infrastructure. Constructors must assure that all reasonable steps have been taken to ensure success in the process. In doing so, new ideas for the improvement of this technique are continually being developed. As is the case with other technologies, there is a driving force towards the deployment of larger diameter pipelines. Also, as costs and environmental concerns arise, HDD crossings are now also being made in conditions not previously considered suitable.

There are a few threats to the success of an HDD, ranging from not being able to successfully complete the tunnel, to damaging the pipe once it is pulled through the finished tunnel. Once any tract of the pipe is within the tunnel, it is almost impossible to be pulled out. Therefore, any damage to the pipe leads to the total loss of the HDD and a new place to do it should be envisaged. That is why there is a series of design and control procedures to avoid pipe damage.

During the final installation of the pipeline into the bore or tunnel (“pull back”), the pipe tract is subjected to traction, but also to bending and lateral compression, since the tunnel is never perfectly straight and continuous in diameter. Loads upon the pipe are carefully determined and measured during the operation, and limits have been carefully determined. State of the art calculation methods allow defining carriage force in actual HDD installations section by section [8]. There are currently several design practices for calculating tensile loading during the installation of steel and polyethylene pipe using HDD. A sensitivity analysis of the models reveal that while the relative influence of the various parameters is a function of the length of pipe within the bore, the predicted pulling load is very sensitive to mud weight and mud drag [9]. Modifications to the calculation methods for pulling loads are periodically proposed, based on documentation from completed HDD projects. Research is currently underway to develop models that better capture the physical reality in the borehole during an HDD installation.

Axial force transfer is an issue in deviated wells and tunnels, where friction and buckling phenomenon take place. To reduce unpredictable lock-up situations it is important to apply relatively high axial loads. On the other hand, the general criterion is that once the pipe exceeds conventional buckling criteria, axial force cannot be transferred by the pipe anymore. Recent research shows that, even though buckling criteria are exceeded, axial force transfer could be still good if the pipe is in rotation [10]. But this introduces technical difficulties, especially in HDD jobs with long strings of large diameter pipe.

Application of quality management has improved reliability in the area of trenchless technology in general and HDD in particular. The development of load cell technology electronic records of installation loads have become a powerful tool for implementing quality control programs. The records provide installation rates, the average pull load per unit length of pipe, the maximum load exerted on the pipe, as well as the entire load history of the installation. Radio transmitting capability provides real-time load cell data that are used to adjust drilling practices to match the level of installation effort to the parameters of the given installation, thereby maximizing productivity and reliability [11].

2. The failed HDD crossing

Fig. 1 shows a sketch of the designed HDD for the crossing of a gas pipeline under a large river. The Directional Drilling described here consists in a 6 stage process. Stage 1 involved drilling a 12″ diameter pilot hole along the proposed design centerline. In stages 2–5, the pilot hole was progressively enlarged to the desired diameter of 42″, to accommodate the pipeline. The pilot hole was drilled with a surface-launched rig with an inclined carriage, adjusted at an angle of about 17 ° for entrance and 8–12° for ground exit.

An important aspect in this case is the characteristics of the soil. As frequently is the case in crossings involving large rivers, gravel of different sizes comprises the upper layer of soil. Loose gravel and sand make it difficult to start a tunnel at a low entrance angle, as required for the HDD, and to maintain the tunnel clean in subsequent stages. Initial design, therefore, included digging two large holes at each side of the HDD, down to stable clay soil. However, the depth of the gravel layer (more than 10 m), its looseness and the limitations in space restricted this solution. As sketched in Fig. 1, the dimensions required for the hole at the south bank and the restrictions in space due to houses in the nearby village required that the hole had to

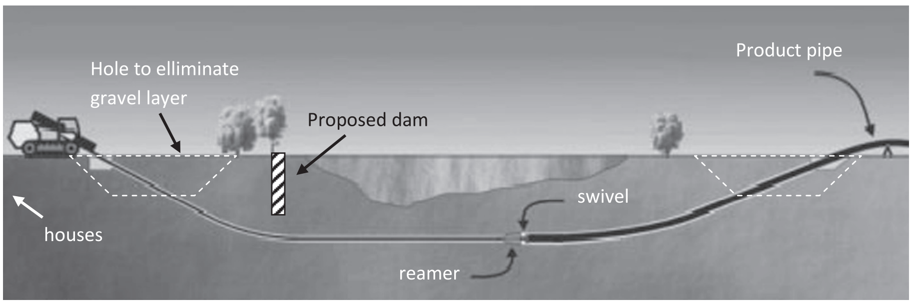

Fig. 1. Sketch of the initially proposed HDD, at the stage of pullback of the product pipe.

be dag too close to the river bank. At that time of the year, the water level in the river was closed to maximum. In order to avoid risk of a bank rupture and consequent flood, a deep concrete dam was required. Design was therefore changed; preliminary tests showed that the gravel layer could be safely bored at an adequate angle without digging the initial holes.

Fig. 2a shows a sketch in elevation of the actual tunnel, once it was finished. The horizontal scale is highly reduced. Drilling started in the south margin of the river (left side in Fig. 2). When actual digging started, the bad conditions of the subsurface layer of gravel caused the deviation of the drilling head. This first attempt had to be aborted; boring was restarted about 20 m backwards, away from the river. The position of the first attempt of initiating the tunnel is highlighted in Fig. 2b. This initial mishap increased the necessary length of pipeline along the tunnel, so that two extra tubes were added, one at each end of the pipe string. Fig. 2c shows a detail of the area in the north margin where the pipeline pullback would begin, once the tunnel was finished (right).

The boring operation was assisted by bentonite sludge (mud) injection in order to add strength to the drill head, condition the tunnel, refrigerate the tool and, by the gelifying properties of the sludge, extract the pieces of terrain that were being drilled towards the trench. The progress of the pilot hole was monitored by an electromagnetic down-hole navigational system. In order to maintain tunnel roundness, a centering device was used for subsequent drilling stages. As an example, Fig. 3 shows the reamer used for the last drilling stage and the centering device at the exit on the northern margin.

Once the tunnel had the required diameter, the pipe string was attached to the reamer, with a swivel to ensure that the rotation (torque) applied to the reamer is not transmitted to the pipeline. In order to predict the occurrence of buckling or other inconveniences, simultaneous torque-drag-buckling measurements were made. In this particular tunnel there was a risk of water from the river getting into the tunnel, or vice versa, drilling sludge being poured into the river. Real-time assessment calculations, along with mud flow and pressure, were made.

Circled in Fig. 2b and c are the boundaries between the surface gravel layer and the underlying clay soil. Despite all precautions, after the tunnel was completed it was apparent that a change in slope was produced at the interlayer in the south bank (Fig. 2b). This bending of the pipeline is called ‘‘dog leg’’. As discussed before, this could lead to higher than expected

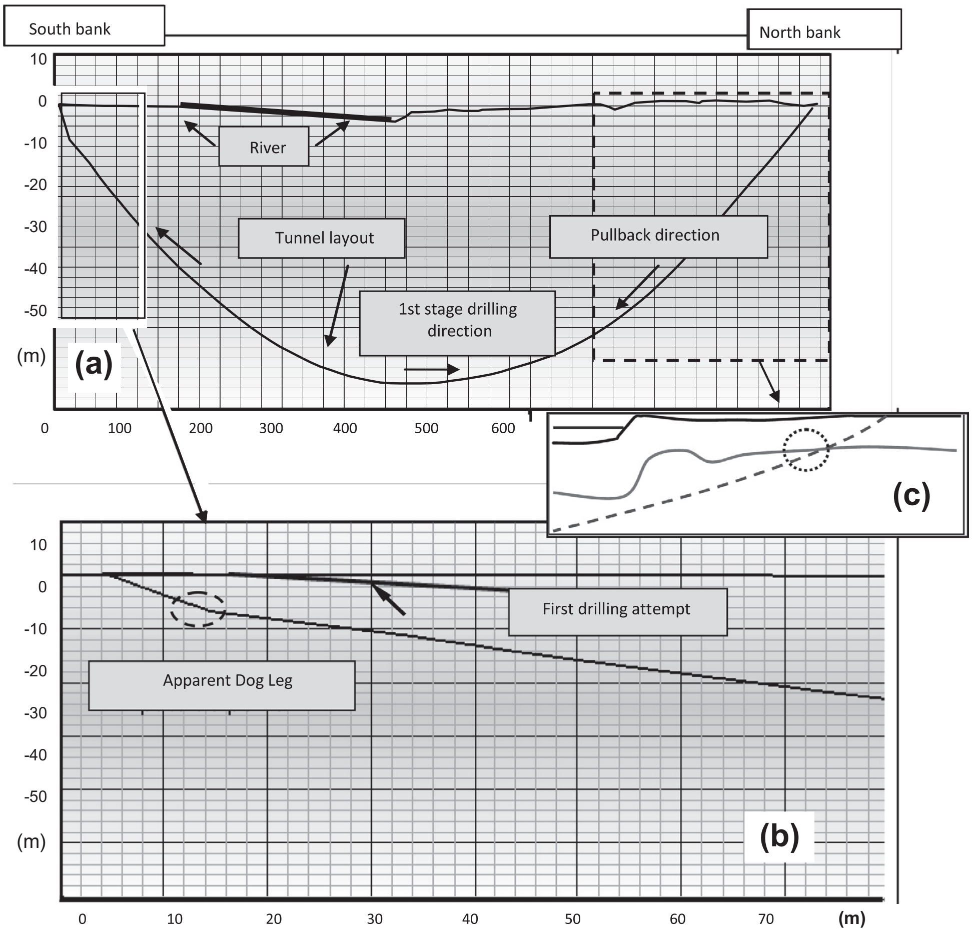

Fig. 2. (a) Sketch in elevation of the HDD tunnel under the river (scales in m), (b) detail at south bank, (c) detail at north bank.

Fig. 3. Detail of the last drilling stage of the tunnel.

stressed in the pipe when being pulled along the tunnel. No apparent dog leg was found in the northern bank, from where the pipe pull back would start (Fig. 2c).

The product pipeline was built with 30″ diameter API 5L X65 double submerged arc welded (DSAW) pipes [12] 8.14 mm thick, with a three-layer coating. The design analyses of installation loads and stresses are based on the product pipe being installed along the designed path using the best management practices (BMPs) of the HDD industry [5]. For increased strength and reliability, pipe thickness at the crossing was increased 50%, up to 11.9 mm. The length of the pipeline involved in the crossing was more than 500 m. While the tunnel was being finished, the pipe string was sealed in both ends and hydrostatically tested. Water was left into the pipe string during pull back. For this pipe diameter, the addition of water into the product pipe is required to control buoyancy. Buoyancy considerably affects pull loads and stresses during the installation procedure.

3. Initial analysis of failure during pipe pullback

After analyzing the geometry of the tunnel, maximum expected stress levels at the pipeline were calculated. Then it was decided to proceed to pipe pullback. Fig. 4a shows the pipe string hanging from raisers in the north bank, waiting for pullback. When the pipe string was about to be inserted on the tunnel, one of the sliders broke and the pipe string fell from a height of about 3 m (Fig. 4b). This incident caused a minor collapse of the tunnel entrance, see Fig. 5. Preventively, a cleaning gauge was passed from the southern margin, to ensure there was no internal collapse on the tunnel. Since the gauge found collapsed parts, it was re-inserted into the tunnel, this time from the northern margin. Special attention was kept to go through the gravel layer by reaching more than 13 m depth (see detail in Fig. 2c). The gauge was left in this position until it was removed the next day, through the same side it had been inserted the last time (north bank).

After this, pullback activities restarted normally: pipe advance was kept at about 3 min per each 12 m pipe, torque values were under 35.000 ft lb (well below the maximum, 60.000 ft lb). Pull load values were between 150 and 200 k lb during most of the drilling process. When the reamer was about 40 m from reaching the southern end of the tunnel, a peak of 332 klb was recorded. Operation was kept going, but by the time the next pipe got into the northern end, slug flow pressure doubled. Pullback continued anyway.

After only two more meters were pulled, the anchoring of the pulling machine failed again, due to the larger load and the poor stiffness of the soil layer, and the pullback operation had to be stopped. An attempt was made to continue operation the next day, but it was not possible to move the pipeline anymore.

An emergency failure analysis was carried out. The geo-referenced data strongly suggested that the apparent dogleg near the end of the tunnel at the south bank could have been responsible for the problem (see Fig. 2b). This position was also compatible with the position of the reamer at the time of increased axial load and mud pressure. Although not serious increase of torque was detected at this location, it was concluded that buckling of the pipe could probably have started when the pipe head reached the dogleg.

It was decided to make an excavation in order to uncover pulling rod up to the pipe head. Fig. 6 shows the undesired event: collapse by lateral buckling readily evident at the front end of the pipe string. Collapse started in the first pipe, near the head joint, between diameter reinforcements in the head. This is the least stiff zone to external pressure. Only two pipes and the beginning of a third one could be further uncovered without compromising the stability of the trench. All these pipes had collapsed by lateral buckling, see Fig. 7.

Fig. 4. (a) Pipe string at north bank waiting for pullback. (b) Failure of pipe raiser at beginning of pullback.

Fig. 5. Minor collapse of the tunnel entrance.

The proximity of an in-service gas pipeline, risks of erosion and flood from the river were practical limitations that discouraged to continue digging in order to recover more pipes for further analysis. Only one pipe and a portion of the second one were extracted. No damage was observed on the teeth and other parts of the reamer that was heading the pipeline. By comparing the positions of the beginning and the end of the pipe string, it was verified that the pipe string underwent a 90° rotation under the tunnel.

If the buckling collapse of the pipeline had indeed been initiated at the "dog leg" near the south end, (detail in Fig. 2b), then the HDD could have been recovered by digging until reaching the "dog leg". A further attempt was made to determine the extension of the collapse in the pipe length. A gauge (a "caliper pig") was introduced inside the pipe from the end at the opposite bank (north), and it got stuck in a position near the deepest point of the tunnel. The depth of the stuck point (about 58 m) made it not possible to dig a trench in order to visually evaluate the nature of the problem. At this moment, the HDD crossing had to be abandoned, and restarted in a new location.

Fig. 6. The undesired event: collapse by lateral buckling seen at the front end of the pipe string.

Fig. 7. Digging at the south bank confirm extended lateral buckling of the pipe.

4. Root Cause Analysis

As part of the quality assurance procedures, the constructor and associated specialists carried out a Root Cause Analysis. When performing a Root Cause Analysis, it is necessary to look at more than just the immediately visible causes, which are often the proximate causes. Root causes are multiple factors (events, conditions or exceeded barriers) that contributed to or created the proximate causes and subsequent undesired outcome. If eliminated or modified, the undesired outcome would have prevented. Typically multiple root causes contribute to an undesired outcome [13–15].

4.1. Field assessments and review of construction data

The first step in the RCA was to check construction data. The sketch in Fig. 8 shows the evolution of the recorded parameters in the drill report. Indicated in vertical axes are measured traction load, mud pressure and torque during pullback of the pipe string into the tunnel, which took place from right to left in the sketch. Southern river bank is on the left, horizontal distances are shown in meters in the horizontal axis. The dotted vertical line (1) indicates the position where the cleaning pig had been left for a whole day, near the northern bank of the river. It is worth noticing that it corresponds to the position were a slug pressure peak was recorded and with a relative load peak. Position (2) indicates the starting site of the buckling collapse of the pipe, according to the evidence given by the internal inspection tool (pig). Position (4), near the south end of the HDD, depicts the possible ‘‘dog leg’’ where the pipe string got stuck.

Analysis of the geo-referenced data also showed inconsistencies in the electromagnetic navigational system reports. Further research evidenced that this was due to changes made to the records to avoid negative position values, after all the works were moved backwards, following the failed first attempt of drilling. This correction was not properly made and the error lead to the apparent ‘‘dog leg’’ presented in Fig. 2b. However, the real reports were then analyzed and showed there was no such defect on that position of the tunnel. However, the presence of less severe discontinuities in the clay-gravel interface could not be discarded. It would be in this local deviation of the tunnel where the string of already collapsed pipes probably clogged, when only a few meters were left to reach the surface.

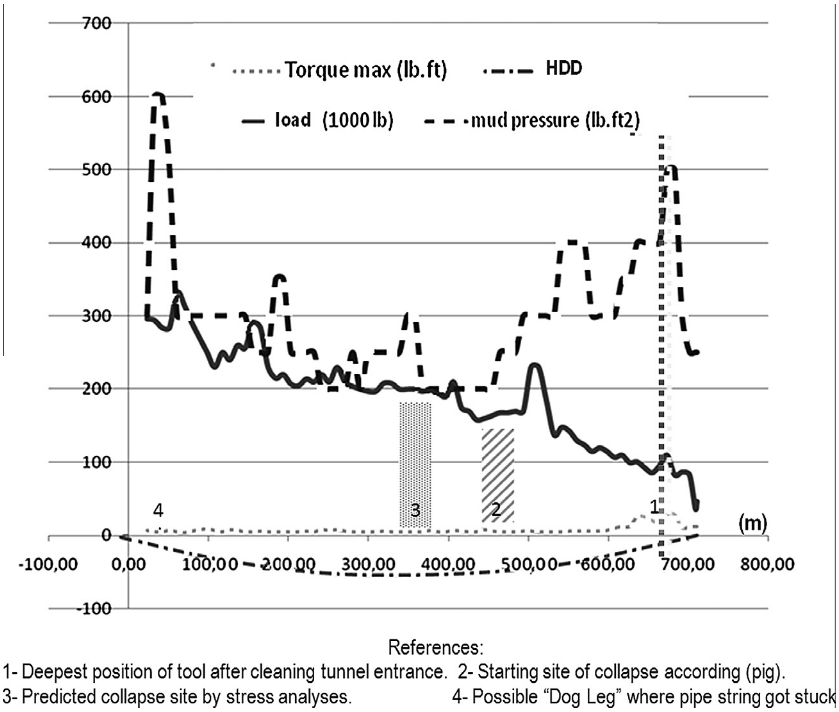

Fig. 8. Measured traction load, mud pressure and torque during pullback of pipe string into tunnel. Southern river bank is on the left, horizontal distances in meters.

Nominal diameter of the tunnel was 42″ (1.0 m), while the major diameter of the deformed pipe was over 46″ (1.2 m). It was not possible that the deformed pipe had travelled nearly the whole length of the tunnel and that the parameters recorded had not showed any significant increments. However, as it was already mentioned, only the last drilling stage used a centering device. This could have lead to an ovalization of the tunnel, since the axis of the following stage would rest in the lower point of the hole drilled by the previous tool, as shown in Fig. 9. The estimated major diameter (height) of the tunnel is 54″ (1.4 m). Under these conditions, it is possible that the pipe collapse took place early on the pullback process and then rotated until it was accommodated on the major diameter of the pipeline to keep traveling with no further resistance to pullback.

An analysis of Annular Solids was based upon the size of the respective reamers, the reaming penetration rates and the drilling fluid flow rates included in the reaming logs. Annular Solids percentages greater than 20% would indicated areas within the reamed hole in which hole cleaning may not be optimal. This was not detected.

The beginning of the buckling collapse of a cylindrical pipe under the combined effects of outer pressure, traction and bending is dependent on certain critical stress conditions. For a certain pipe diameter, these critical conditions depend upon thickness and pipe strength. Compliance with pipe standards was checked. Earlier investigations found out that two pipes of less thickness had been attached to the pipe string. When boring was restarted, adding 20 m to the length of the HDD, two extra tubes were added to the pipe string. No further 12 mm thick pipes were available at the site. Considering that the pipes at the ends of the string were only for the pullback process and would be discarded when connecting the pipe string to the service pipeline, it was decided to use the pipes that were available at the site, which were 8.14 mm thick. In order to make sure that these two pipes would be cut off after pull back, it was decided to weld one pipe to each side of the string.

It was clear at this point that the thinner pipes could have been a large part of the problem. Once initiated in a thin pipe, the collapse could probably have propagated into the rest of the string, as the pipe moved. Models of stresses applied to the pipe string were carried out, in order to verify the influence of pipe thickness and the location in the tunnel at which the original buckling collapse of the pipe took place. Two approaches were taken: analytical and numerical.

4.2. Stress analyses

The design analyses of installation loads and stresses were reassessed, using the management practices (BMPs) of the HDD industry [5]. The installation stresses on the product pipe were analyzed in general accordance with the API RP 2A – WSD [16]. Installation Stress Analyses were conducted for the two pipe thicknesses, considering the pilot hole survey data, to calculate the axial tension and bending stresses, the external hoop stress and the combination results of these stresses at each joint.

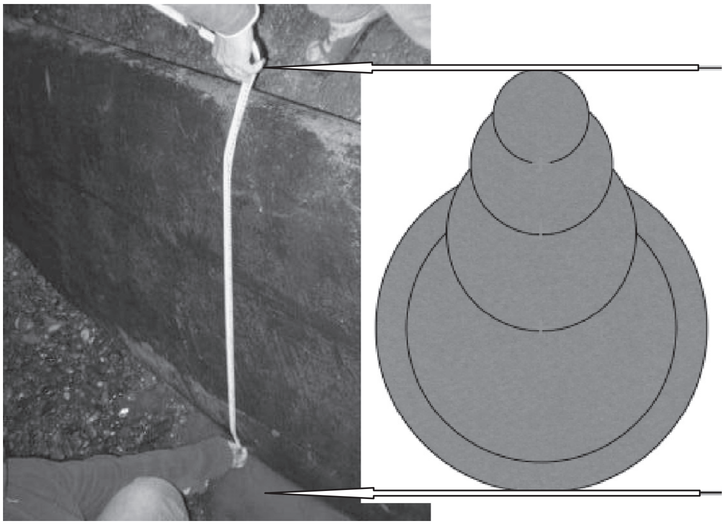

Fig. 9. Width of collapsed pipe (1.2 m) is less than estimated height of the tunnel (1.4 m).

Drilling fluid weights in the reamed hole of 9.5 lb/gal and 12 lb/gal were considered; the as-built pilot hole survey data was analyzed. External hoop stress violations and combined stress violations occurred for joints 10 through 60, when considering a 8.14 mm thick pipe. This is almost the entire pipe length, except for the first 120 m from the south end. Stress violations for the 12 mm thick product pipe were found only when assuming 12 lb/gal drilling fluid, for joints 34 and 35. This corresponds to a length of 10–20 m, to the north of the lowest point below the river. This section is indicated as point 3 in the sketch of Fig. 8.

These analyses are based on considering an open HDD hole throughout the pullback process. If the gravel sections of the hole collapsed during pullback, then the induced stresses on the pipe would have been greater than analyzed.

Little information is available in the literature on the numerical modeling of HDD. However, simulation of the propagation of collapse has been thoroughly undergone for underwater pipe in gas and oil offshore platforms [17]. When collapse occurs in a particular section of the pipe, this may be restricted to that part or propagate along the pipeline. In subsea pipelines, required pressure for the onset of collapse in steel pipes is frequently lower than the pressure required for its propagation. Intermediate elements called arrestors are frequently placed on the pipe to prevent propagation of the collapse.

In order to study the influence of the length of the thinner pipe, the model [18] was applied with the geometries involved in the HDD. Fig. 10 shows that the collapse external pressure drops as the length of the thinner pipe increases and stabilizes for thin tracts longer than 6 m. The formula for this limit pressure is obtained from buckling theory [19]:

$$P^{cri} = \frac{(n^2 - 1)Et^3}{12(1 - \mu^2)R^3} \tag{1}$$

where n = 1/2 the amount of plastic hinges (2 in this case); E = elasticity modulus; t = thickness; R = external radius; μ = Poisson's ratio.

A Finite Element Model (FEM) was carried out using Shell type elements, in order to reproduce the onset of buckling in the front end of the pipe during pullback [20]. The model includes geometric and material nonlinearity (plasticity), and contact conditions in the inner wall surface of the pipeline. A 1.6% initial ovalization was introduced, to force onset of buckling. The model was fixed on one end; on the other end all rotations and translations (except axial) were restricted, outer pressures are 5.33×10^5 for t = 8.14 mm and 16.9×10^5 for t = 11.9 mm. Fig. 11 shows results from the numerical model, with local bending and post-collapse propagation. Insert in Fig. 11 compares the numerical prediction with the actual pattern in the pipe. A thickness reduction of 32% implies an elastically determined reduction in collapse pressure of 68%.

The evidences observed at the crossing site and mechanical models allowed defining the collapse of the pipe by the combined effect of external pressure and bending and traction loads in a localized pipe bending, as the immediate cause of the incident. Although the tunnel does not present physical evidence (the tunnel and the pipe had to be abandoned), it is considered that the entire length of the pipe, to the south (left) of point (3) indicated in Fig. 8, collapsed.

The reduction in thickness of the first pipe in the column lead to a stress state twice and a half more critical than for the rest of the column. Results of the numerical modeling also show that the influence of the thinner pipe in the collapse pressure, if the thin tube is more than 6 m long, is such that the strength of the pipe string is similar to that of the whole pipe string made with thin pipe. This is the present case (each pipe is about 12 m long).

Stress analyses were also used to analyze the influence of other construction parameters. The addition of water into the product pipe is the standard method that contractors typically use to control buoyancy of the product pipe during the

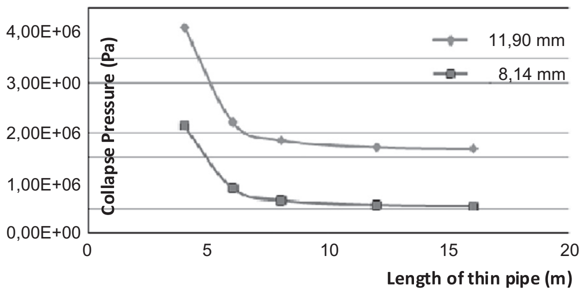

Fig. 10. Variation of collapse pressure with length of thinner pipe, according to model by Dvorkin et al. [17].

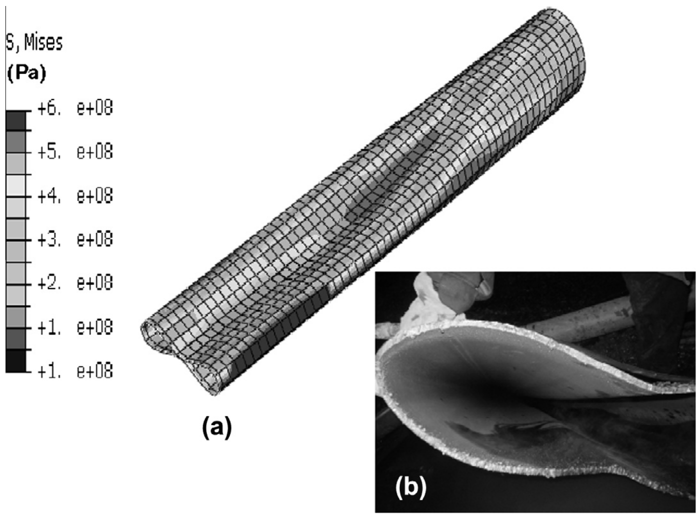

Fig. 11. (a) Numerical model with local bending and post-collapse propagation and (b) actual pattern in the pipe.

installation procedure. It was verified that the effective pipe weight would have been larger without buoyancy control. The higher effective pipe weights for the empty product pipe results in considerably higher pullback forces. The HDD design calculations for the 30″ diameter 8.14 mm steel pipe indicate installation stress violations for the product pipe being installed while empty, when considering typical drilling fluid weights (9.5 lb/gal and 12 lb/gal). This is primarily a result of the product pipe being pulled through the HDD hole empty while having hydrostatic head pressure creating external hoop stress on the product pipe.

5. Discussion of results and conclusions

The failure caused the total loss of the crossing, and the necessity to repeat the entire process in another, less favorable location. Root causes for this failure involve a difficult soil composition, and constraints at the populated south river bank that led to engineering solutions not proved before on a pipe of such diameter.

After evaluating the construction management practices with regard to the pilot hole geometry, reaming pass penetration rates and pump volumes, and the torque and pull load readings during pullback, no data was found to substantiate that the construction practices were not in line with the HDD industry standard of care. The subsurface conditions along the proposed alignment and profile design were difficult to drill, maintain an open hole, and eventually pull the pipe. Stress Analyses showed no stress violations resulting from the combined effects of the axial tension stress, the longitudinal bending stress and the external hoop stress, provided the whole pipe string was made of the specified 12 mm thick pipe.

It is concluded that the thinner walled (8.14 mm thick.) leading pipe at the front of the string collapsed during the pullback process, since in this case combined stresses surpassed critical buckling conditions. Once the initial collapse of the thinner leading pipe occurred downhole, the failure propagated longitudinally to the thicker walled pipe and extended to the point at which the pigging mechanism would no longer advance through the 30-inch-diameter section of pipe. The collapse was not immediately detected by an increase over the permissible pull back force, mud pressure and torque, because the lubrication and mud conditions were very good.

Two previous incidents had also a definite influence in the chain of events that led to the failure, both related to the difficult surface layer of gravel. First, the need to restart the pilot bore, which led to the addition of one pipe at each end of the string. Second, the failure of a tube hanger made the pipe front to impact the earth so close to the entrance of the tunnel that a small collapse of the gravel occurred. This then led to loose gravel or other discontinuities being left in the tunnel, that led to the increase in stresses in the pipe.

These are fortuitous accidents, but there are two stages at which quick decisions were made, in the heat of the process, that did not follow prescribed engineering procedures. The first and more obvious is the use of two sacrifice pipes, which was related to the need for more pipes than originally calculated. The decisions to use the available thinner pipes, and then to place one in each end of the string, were made ignoring the criticality of this condition.

The second failed decision, not so obvious but also important, was to pass the reamer along a short length of the tunnel, after the small collapse at the entrance. It is clear now and should have been clear then, that any debris pushed by the reamer would accumulate at the site where the reamer would be stopped. Once launched, the reaming should always be pushed all the way through the other side.

Both decisions were made in order to keep a timetable. As is many times the case, engineers are pressed within their companies to reduce costs and times, but it is their duty to avoid conditions that could violate standards or proven procedures.

Acknowledgments

This research work was partly funded by CONICET (Consejo Nacional de Investigaciones Científicas y Técnicas de la Republica Argentina), Universidad Nacional de Mar del Plata, Argentina (Grant No. 15/G316) and by Agencia Nacional de Promoción Científica (Grant No. PICT 0582), Argentina. Authors thank Gie S.A. Integrity of Assets for allowing the use of proprietary information.

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