Chapter 12

Horizontal Directional Drilling

Introduction

The Horizontal Directional Drilling (HDD) Industry has experienced so much growth in the past two decades that HDD has become commonplace as a method of installation. One source reported that the number of units in use increased by more than a hundred-fold in the decade following 1984. This growth has been driven by the benefits offered to utility owners (such as the elimination of traffic disruption and minimal surface damage) and by the ingenuity of contractors in developing this technology. To date, HDD pipe engineering has focused on installation techniques, and rightfully so. In many cases, the pipe experiences its maximum lifetime loads during the pullback operation.

The purpose of this chapter is to acquaint the reader with some of the important considerations in selecting the proper polyethylene pipe. Proper selection of pipe involves consideration not only of installation design factors such as pullback force limits and collapse resistance, but also of the long-term performance of the pipe once installed in the bore-hole. The information herein is not all-inclusive; there may be parameters not discussed that will have significant bearing on the proper engineering of an application and the pipe selection. For specific projects, the reader is advised to consult with a qualified engineer to evaluate the project and prepare a specification including design recommendations and pipe selection. The reader many find additional design and installation information in ASTM F1962, “Standard Guide for Use of Maxi- Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings,” and in the ASCE Manual of Practice 108, “Pipeline Design for Installation by Directional Drilling.”

Background

Some of the earliest uses of large diameter polyethylene pipe in directional drilling were for river crossings. These are major engineering projects requiring thoughtful design, installation, and construction, while offering the owner the security of deep river bed cover with minimum environmental damage or exposure, and no disruption of river traffic. Polyethylene pipe is suited for these installations because of its scratch tolerance and the fused joining system which gives a zero-leak-rate joint with design tensile capacity equal to that of the pipe.

To date, directional drillers have installed polyethylene pipe for gas, water, and sewer mains; electrical conduits; and a variety of chemical lines. These projects involved not only river crossings but also highway crossings and right-of-ways through developed areas so as not to disturb streets, driveways, and business entrances.

Polyethylene Pipe for Horizontal Directional Drilling

This chapter gives information on the pipe selection and design process. It is not intended to be a primer on directional drilling. The reader seeking such information can refer to the references of this chapter. Suggested documents are the "Mini-Horizontal Directional Drilling Manual" and the "Horizontal Directional Drilling Good Practices Guidelines" published by the North American Society for Trenchless Technology (NASTT).

Horizontal Directional Drilling Process

Knowledge of the directional drilling process by the reader is assumed, but some review may be of value in establishing common terminology. Briefly, the HDD process begins with boring a small, horizontal hole (pilot hole) under the crossing obstacle (i.e. a highway) with a continuous string of steel drill rod. When the bore head and rod emerge on the opposite side of the crossing, a special cutter, called a back reamer, is attached and pulled back through the pilot hole. The reamer bores out the pilot hole so that the pipe can be pulled through. The pipe is usually pulled through from the side of the crossing opposite the drill rig.

Pilot Hole

Pilot hole reaming is the key to a successful directional drilling project. It is as important to an HDD pipeline as backfill placement is to an open-cut pipeline. Properly trained crews can make the difference between a successful and an unsuccessful drilling program for a utility. Several institutions provide operator-training programs, one of which is Michigan State University's Center for Underground Infrastructure Research and Education (CUIRE). Drilling the pilot hole establishes the path of the drill rod ("drill-path") and subsequently the location of the PE pipe. Typically, the bore-head is tracked electronically so as to guide the hole to a pre-designed configuration. One of the key considerations in the design of the drill-path is creating as large a radius of curvature as possible within the limits of the right-of-way, thus minimizing curvature. Curvature induces bending stresses and increases the pullback load due to the capstan effect. The capstan effect is the increase in frictional drag when pulling the pipe around a curve due to a component of the pulling force acting normal to the curvature. Higher tensile stresses reduce the pipe's collapse resistance. The drill-path normally has curvature along its vertical profile. Curvature requirements are dependent on site geometry (crossing length, required depth to provide safe cover, staging site location, etc.) But, the degree of curvature is limited by the bending radius of the drill rod and the pipe. More often, the permitted bending radius of the drill rod controls the curvature and thus significant bending stresses do not occur in the pipe. The designer should minimize the number of curves and maximize their radii of curvature in the right-of-way by carefully choosing the entry and exit points. The driller should also attempt to minimize extraneous curvature due to undulations (dog-legs) from frequent over-correcting alignment or from differences in the soil strata or cobbles.

Pilot Hole Reaming

The REAMING operation consists of using an appropriate tool to open the pilot hole to a slightly larger diameter than the carrier pipeline. The percentage oversize depends on many variables including soil types, soil stability, depth, drilling mud, borehole hydrostatic pressure, etc. Normal over-sizing may be from 120% to 150% of the carrier pipe diameter. While the over-sizing is necessary for insertion, it means that the inserted pipe will have to sustain vertical earth pressures without significant side support from the surrounding soil.

Prior to pullback, a final reaming pass is normally made using the same sized reamer as will be used when the pipe is pulled back (swab pass). The swab pass cleans the borehole, removes remaining fine gravels or clay clumps and can compact the borehole walls.

Drilling Mud

Usually a "drilling mud" such as fluid bentonite clay is injected into the bore during cutting and reaming to stabilize the hole and remove soil cuttings. Drilling mud can be made from clay or polymers. The primary clay for drilling mud is sodium montmorillonite (bentonite). Properly ground and refined bentonite is added to fresh water to produce a "mud." The mud reduces drilling torque, and gives stability and support to the bored hole. The fluid must have sufficient gel strength to keep cuttings suspended for transport, to form a filter cake on the borehole wall that contains the water within the drilling fluid, and to provide lubrication between the pipe and the borehole on pullback. Drilling fluids are designed to match the soil and cutter. They are monitored throughout the process to make sure the bore stays open, pumps are not overworked, and fluid circulation throughout the borehole is maintained. Loss of circulation could cause a locking up and possibly overstressing of the pipe during pullback.

Drilling muds are thixotropic and thus thicken when left undisturbed after pullback. However, unless cementitious agents are added, the thickened mud is no stiffer than very soft clay. Drilling mud provides little to no soil side-support for the pipe.

Pullback

The pullback operation involves pulling the entire pipeline length in one segment (usually) back through the drilling mud along the reamed-hole pathway. Proper pipe handling, cradling, bending minimization, surface inspection, and fusion welding procedures need to be followed. Axial tension force readings, constant insertion velocity, mud flow circulation/exit rates, and footage length installed should be recorded. The pullback speed ranges usually between 1 to 2 feet per minute.

Mini-Horizontal Directional Drilling

The Industry distinguishes between mini-HDD and conventional HDD, which is sometimes referred to as maxi-HDD. Mini-HDD rigs can typically handle pipes up to 10" or 12" and are used primarily for utility construction in urban areas, whereas HDD rigs are typically capable of handling pipes as large as 48". These machines have significantly larger pullback forces ranging up to several hundred thousand pounds.

General Guidelines

The designer will achieve the most efficient design for an application by consulting with an experienced contractor and a qualified engineer. Here are some general considerations that may help particularly in regard to site location for PE pipes:

1. Select the crossing route to keep it to the shortest reasonable distance.

2. Find routes and sites where the pipeline can be constructed in one continuous length; or at least in long multiple segments fused together during insertion.

3. Although compound curves have been done, try to use as straight a drill path as possible.

4. Avoid entry and exit elevation differences in excess of 50 feet; both points should be as close as possible to the same elevation.

5. Locate all buried structures and utilities within 10 feet of the drill-path for mini-HDD applications and within 25 feet of the drill-path for maxi-HDD applications. Crossing lines are typically exposed for exact location.

6. Observe and avoid above-ground structures, such as power lines, which might limit the height available for construction equipment.

7. The HDD process takes very little working space versus other methods. However, actual site space varies somewhat depending upon the crossing distance, pipe diameter, and soil type.

8. Long crossings with large diameter pipe need bigger, more powerful equipment and drill rig.

9. As pipe diameter increases, large volumes of drilling fluids must be pumped, requiring more/larger pumps and mud-cleaning and storage equipment.

10. Space requirements for maxi-HDD rigs can range from a 100 feet wide by 150 feet long entry plot for a 1000 ft crossing up to 200 feet wide by 300 feet long area for a crossing of 3000 or more feet.

11. On the pipe side of the crossing, sufficient temporary space should be rented to allow fusing and joining the polyethylene carrier pipe in a continuous string beginning about 75 feet beyond the exit point with a width of 35 to 50 feet, depending on the pipe diameter. Space requirements for coiled pipe are considerably less. Larger pipe sizes require larger and heavier construction equipment which needs more maneuvering room (though use of polyethylene minimizes this). The initial pipe side "exit" location should be about 50′ W x 100′ L for most crossings, up to 100′ W x 150′ L for equipment needed in large diameter crossings.

12. Obtain "as-built" drawings based on the final course followed by the reamer and the installed pipeline. The gravity forces may have caused the reamer to go slightly deeper than the pilot hole, and the buoyant pipe may be resting on the crown of the reamed hole. The as-built drawings are essential to know the exact pipeline location and to avoid future third party damage.

Safety

Safety is a primary consideration for every directionally drilled project. While this chapter does not cover safety, there are several manuals that discuss safety including the manufacturer's Operator's Manual for the drilling rig and the Equipment Manufacturer's Institute (EMI) Safety Manual: *Directional Drilling Tracking Equipment.*

Geotechnical Investigation

Before any serious thought is given to the pipe design or installation, the designer will normally conduct a comprehensive geotechnical study to identify soil formations at the potential bore sites. The purpose of the investigation is not only to determine if directional drilling is feasible, but to establish the most efficient way to accomplish it. With this information the best crossing route can be determined, drilling tools and procedures selected, and the pipe designed. The extent of the geotechnical investigation often depends on the pipe diameter, bore length and the nature of the crossing. Refer to ASTM F1962 and ASCE MOP 108 for additional information.

During the survey, the geotechnical consultant will identify a number of relevant items including the following:

a. Soil identification to locate rock, rock inclusions, gravelly soils, loose deposits, discontinuities and hardpan.

b. Soil strength and stability characteristics

c. Groundwater

(Supplemental geotechnical data may be obtained from existing records, e.g. recent nearby bridge constructions, other pipeline/cable crossings in the area.)

For long crossings, borings are typically taken at 700 ft intervals. For short crossings (1000 ft or less), as few as three borings may suffice. The borings should be near the drill-path to give accurate soil data, but sufficiently far from the borehole to avoid pressurized mud from following natural ground fissures and rupturing to the ground surface through the soil-test bore hole. A rule-of -thumb is to take borings at least 30 ft to either side of bore path. Although these are good general rules, the number, depth and location of boreholes is best determined by the geotechnical engineer.

Geotechnical Data For River Crossings

River crossings require additional information such as a study to identify river bed, river bed depth, stability (lateral as well as scour), and river width. Typically, pipes are installed to a depth of at least 20 ft below the expected future river bottom, considering scour. Soil borings for geotechnical investigation are generally conducted to 40 ft below river bottom.

Summary

The best conducted projects are handled by a team approach with the design engineer, bidding contractors and geotechnical engineer participating prior to the preparation of contract documents. The geotechnical investigation is usually the first step in the boring project. Once the geotechnical investigation is completed, a determination can be made whether HDD can be used. At that time, design of both the HDPE pipe and the installation can begin. The preceding paragraphs represent general guidance and considerations for planning and designing an HDD polyethylene pipeline project. These overall topics can be very detailed in nature.

Individual HDD contractors and consultant engineering firms should be contacted and utilized in the planning and design stage. Common sense along with a rational in-depth analysis of all pertinent considerations should prevail. Care should be given in evaluating and selecting an HDD contractor based upon successful projects, qualifications, experience and diligence. A team effort, strategic partnership and risk-sharing may be indicated.

Product Design: DR Selection

After completion of the geotechnical investigation and determination that HDD is feasible, the designer turns attention to selecting the proper pipe. The proper pipe must satisfy all hydraulic requirements of the line including flow capacity, working pressure rating, and surge or vacuum capacity. These considerations have to be met regardless of the method of installation. Design of the pipe for hydraulic considerations can be found elsewhere such as in AWWA C906 or the pipe manufacturer's literature and will not be addressed in this chapter. For HDD applications, in addition to the hydraulic requirements, the pipe must be able to withstand (1) pullback loads which include tensile pull forces, external hydrostatic pressure, and tensile bending stresses, and (2) external service loads (post-installation soil, groundwater, and surcharge loads occurring over the life of the pipeline). Often the load the pipe sees during installation such as the combined pulling force and external pressure will be the largest load experienced by the pipe during its life. The remainder of this document will discuss the DR selection based on pullback and external service loads. (Polyethylene pipe is classified by DR. The DR is the "dimension ratio" and equals the pipe's outer diameter divided by the minimum wall thickness.)

While this chapter gives guidelines to assist the designer, the designer assumes all responsibility for determining the appropriateness and applicability of the equations and parameters given in this chapter for any specific application. Directional drilling is an evolving technology, and industry-wide design protocols are still developing. Proper design requires considerable professional judgment beyond the scope of this chapter. The designer is advised to consult ASTM F 1962 when preparing an HDD design.

Normally, the designer starts the DR selection process by determining the DR requirement for the internal pressure (or other hydraulic requirements). The designer will then determine if this DR is sufficient to withstand earth, live, and groundwater service loads. If so, then the installation (pullback) forces are considered. Ultimately, the designer chooses a DR that will satisfy all three requirements: the pressure, the service loads, and the pullback load.

Although there can be some pipe wall stresses generated by the combination of internal pressurization and wall bending or localized bearing, generally internal pressure and external service load stresses are treated as independent. This is permissible primarily since PE is a ductile material and failure is usually driven by the average stress rather than local maximums. There is a high safety factor applied to the internal pressure, and internal pressurization significantly reduces stresses due to external loads by re-rounding. (One exception to this is internal vacuum, which must be combined with the external pressure.)

Figure 1 Borehole Deformation

Design Considerations for Net External Loads

This and the following sections will discuss external buried loads that occur on directionally drilled pipes. One important factor in determining what load reaches the pipe is the condition of the borehole, i.e. whether it stays round and open or collapses. This will depend in great part on the type of ground, the boring techniques, and the presence of slurry (drilling mud and cutting mixture). If the borehole does not deform (stays round) after drilling, earth loads are arched around the borehole and little soil pressure is transmitted to the pipe. The pressure acting on the pipe is the hydrostatic pressure due to the slurry or any groundwater present. The slurry itself may act to keep the borehole open. If the borehole collapses or deforms substantially, earth pressure will be applied to the pipe. The resulting pressure could exceed the slurry pressure unless considerable tunnel arching occurs above the borehole. Where no tunnel arching occurs, the applied external pressure is equal to the combined earth, groundwater, and live-load pressure. For river crossings, in unconsolidated river bed soils, little arching is anticipated . The applied pressure likely equals the geostatic stress (sometimes called the prism load). In consolidated soils, arching above the borehole may occur, and the applied pressure will likely be less than the geostatic stress, even after total collapse of the borehole crown onto the pipe. If the soil deposit is a stiff clay, cemented, or partially lithified, the borehole may stay open with little or no deformation. In this case, the applied pressure is likely to be just the slurry head or groundwater head.

In addition to the overt external pressures such as slurry head and groundwater, internal vacuum in the pipe results in an increase in external pressure due to the removal of atmospheric pressure from inside the pipe. On the other hand, a positive internal pressure in the pipe may mediate the external pressure. The following equations can be used to establish the net external pressure or, as it is sometimes called, the differential pressure between the inside and outside of the pipe.

Depending on the borehole condition, the net external pressure is defined by either Eq. 1 (deformed/collapsed borehole) or Eq. 2 (open borehole):

(1) $$P_N = P_E + P_{GW} + P_{SUR} - P_I$$

(2) $$P_N = P_{MUD} - P_I$$

WHERE

P_N = Net external pressure, psi

P_E = External pressure due to earth pressure, psi

P_{GW} = Groundwater pressure (including the height of river water), psi

P_{SUR} = Surcharge and live loads, psi

P_I = Internal pressure, psi (negative in the event of vacuum)

P_{MUD} = Hydrostatic pressure of drilling slurry or groundwater pressure, if slurry can carry shear stress, psi

(Earth, ground water, and surcharge pressures used in Eq. 1 are discussed in a following section of this chapter.)

(3) $$P_{MUD} = \frac{g_{MUD} H_B}{144 \dfrac{in^2}{ft^2}}$$

WHERE

g_{MUD} = Unit weight of slurry (drilling mud and cuttings), pcf

H_B = Elevation difference between lowest point in borehole and entry or exit pit, ft

(144 is included for units conversion.)

When calculating the net external pressure, the designer will give careful consideration to enumerating all applied loads and their duration. In fact, most pipelines go through operational cycles that include (1) unpressurized or being drained, (2) operating at working pressure, (3) flooding, (4) shutdowns, and (5) vacuum and peak pressure events. As each of these cases could result in a different net external pressure, the designer will consider all phases of the line's life to establish the design cases.

In addition to determining the load, careful consideration must be given to the duration of each load. PE pipe is viscoelastic, that is, it reacts to load with time-dependent properties. For instance, an HDD conduit resists constant groundwater and soil pressure with its long-term stiffness. On the other hand, an HDD force-main may be subjected to a sudden vacuum resulting from water hammer. When a vacuum occurs, the net external pressure equals the sum of the external pressure plus the vacuum. Since surge is instantaneous, it is resisted by the pipe's short-term stiffness, which can be four times higher than the long-term stiffness.

For pressure lines, consideration should be given to the time the line sits unpressurized after construction. This may be several months. Most directionally drilled lines that contain fluid will have a static head, which will remain in the line once filled. This head may be subtracted from the external pressure due to earth/ groundwater load. The designer should keep in mind that the external load also may vary with time, for example, flooding.

Earth and Groundwater Pressure

Earth loads can reach the pipe when the borehole deforms and contacts the pipe. The amount of soil load transmitted to the pipe will depend on the extent of deformation and the relative stiffness between the pipe and the soil. Earth loading may not be uniform. Due to this complexity, there is not a simple equation for relating earth load to height of cover. Groundwater loading will occur whether the hole deforms or not; the only question is whether or not the slurry head is higher and thus may in fact control design. Thus, what loads reach the pipe will depend on the stability of the borehole.

The designer may wish to consult a geotechnical engineer for assistance in determining earth and groundwater loads, as the loads reaching the pipe depend on detailed knowledge of the soil.

Stable Borehole - Groundwater Pressure Only

A borehole is called stable if it remains round and deforms little after drilling. For instance, drilling in competent rock will typically result in a stable borehole. Stable boreholes may occur in some soils where the slurry exerts sufficient pressure to maintain a round and open hole. Since the deformations around the hole are small, soil pressures transmitted to the pipe are negligible. The external load applied to the pipe consists only of the hydrostatic pressure due to the slurry or the groundwater, if present. Equation 4 gives the hydrostatic pressure due to groundwater or drilling slurry. Standing surface water should be added to the groundwater.

(4)
$$P_{GW} = \frac{g_W H_W}{144\frac{in^2}{ft^2}}$$

WHERE

P_{GW} = Hydrostatic fluid pressure due to ground and surface water, psi

g_W = Unit weight of water, pcf

H_W = Height to free water surface above pipe, ft (144 is included for correct units conversion.)

Borehole Deforms/Collapse With Arching Mobilized

When the crown of the hole deforms sufficiently to place soil above the hole in the plastic state, arching is mobilized. In this state, hole deformation is limited. If no soil touches the pipe, there is no earth load on the pipe. However, when deformation is sufficient to transmit load to the pipe, it becomes the designer's chore to determine how much earth load is applied to the pipe. At the time of this writing, there have been no published reports giving calculation methods for finding earth load on directionally drilled pipes. Based on the successful performance of directionally drilled PE pipes, it is reasonable to assume that some amount of arching occurs in many applications. The designer of HDD pipes may gain some knowledge from the approaches developed for determining earth pressure on auger bored pipes and on jacked pipes. It is suggested that the designer become familiar with all of the assumptions used with these methods.

O'Rourke et. al. published an equation for determining the earth pressure on auger bored pipes assuming a borehole approximately 10% larger than the pipe. In this model, arching occurs above the pipe similar to that in a tunnel where zones of loosened soil fall onto the pipe. The volume of the cavity is eventually filled with soil that is slightly less dense than the insitu soil, but still capable of transmitting soil load. This method of load calculation gives a minimal loading. The method published here is more conservative. It is based on trench type arching as opposed to tunnel arching and is used by Stein to calculate loads on jacked pipe. In Stein's model, the maximum earth load (effective stress) is found using the modified form of Terzhaghi's equation given by Eq. 6. External groundwater pressure must be added to the effective earth pressure. Stein and O'Rourke's methods should only be considered where the depth of cover is sufficient to develop arching (typically exceeding five (5) pipe diameters), dynamic loads such as traffic loads are insignificant, the soil has sufficient internal friction to transmit arching, and conditions are confirmed by a geotechnical engineer.

Using the equations given in Stein, the external pressure is given below:

(5)
$$P_{EV} = \frac{K g_{SE} H_C}{144 \frac{in^2}{ft^2}}$$

(6)

$$k = \frac{1 - \exp\left(-2\frac{KH_C}{B}\tan\left(\frac{d}{2}\right)\right)}{2\frac{KH_C}{B}\tan\left(\frac{d}{2}\right)}$$

WHERE

P_{EV} = external earth pressure, psi

g_{SE} = effective soil weight, pcf

H_C = depth of cover, ft

k = arching factor

B = "silo" width, ft

d = angle of wall friction, degrees (For HDD, d = f)

f = angle of internal friction, degrees

K = earth pressure coefficient given by:

$$K = \tan^2\left(45 - \frac{f}{2}\right)$$

The "silo" width should be estimated based on the application. It varies between the pipe diameter and the borehole diameter. A conservative approach is to assume the silo width equals the borehole diameter. (The effective soil weight is the dry unit weight of the soil for soil above the groundwater level, it is the saturated unit weight less the weight of water for soil below the groundwater level.)

Borehole Collapse with Prism Load

In the event that arching in the soil above the pipe breaks down, considerable earth loading may occur on the pipe. In the event that arching does not occur, the upper limit on the load is the weight of the soil prism ($P_E = g_{SE}H_C$) above the pipe. The prism load is most likely to develop in shallow applications subjected to live loads, boreholes in unconsolidated sediments such as in some river crossings, and holes subjected to dynamic loads. The "prism" load is given by Eq. 7.

(7)

$$P_E = \frac{g_{SE}H_C}{144\frac{in^2}{ft^2}}$$

WHERE
P_E = earth pressure on pipe, psi
g_{SE} = effective weight of soil, pcf
H_C = soil height above pipe crown, ft
(Note: 144 is included for units conversion.)

Combination of Earth and Groundwater Pressure

Where groundwater is present in the soil formation, its pressure must be accounted for in the external load term. For instance, in a river crossing one can assume with reasonable confidence that the directionally drilled pipe is subjected to the earth pressure from the sediments above it combined with the water pressure.

Case 1 Water level at or below ground surface

(8)
$$P_E + P_{GW} = \frac{g_B H_W + g_D \left(H_C - H_W\right) + g_W H_W}{144 \frac{in^2}{ft^2}}$$

Case 2 Water level at or above ground surface (i.e. pipe in river bottom)

(9)
$$P_E + P_{GW} = \frac{g_B H_C + g_W H_W}{144 \frac{in^2}{ft^2}}$$

WHERE
H_W = Height of Ground water above pipe springline, ft
H_C = height of cover, ft
g_B = buoyant weight of soil, pcf
g_W = weight of water, pcf
g_D = dry unit weight of soil, pcf

Live Loads

Wheel loads from trucks or other vehicles are significant for pipe at shallow depths whether they are installed by open cut trenching or directional drilling. The wheel load applied to the pipe depends on the vehicle weight, the tire pressure and size, vehicle speed, surface smoothness, pavement and distance from the pipe to the point of loading. In order to develop proper soil structure interaction, pipe subject to vehicular loading should be installed at least 18” or one pipe diameter (whichever is larger) under the road surface. Generally, HDD pipes are always installed at a deeper depth so as to prevent frac-outs from occurring during the boring.

For pipes installed under rigid pavement and subjected to H20 loadings, Table 1 gives the vertical earth pressure at the pipe crown as determined by AISI [3]. Live loads under flexible pavement and unpaved roads can be calculated. (See Spangler and Handy in references.)

TABLE 1
H20 Loading Under Rigid Pavement (AISI)

Height of Cover (ft)	(ft) Load (psf)
1	1800
2	800
3	600
4	400
5	250
6	200
7	175
8	100

The live-load pressure can be obtained from Table 1 by selecting the load based on the height of cover and converting the load to units of “psi” by dividing the load in “psf” by 144.

Performance Limits

Hydrostatic Buckling or Collapse

Ring Deformation

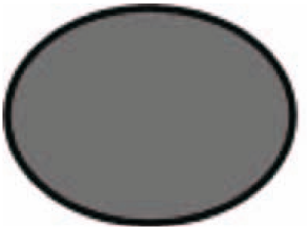

Figure 2 Performance Limits of HDD Pipe Subjected to Service Loads

Performance Limits of HDD Installed Pipe

The design process normally consists of calculating the loads applied to the pipe, selecting a trial pipe DR, then calculating the safety factor for the trial DR. If the safety factor is adequate, the design is sufficient. If not, the designer selects a lower DR and repeats the process. The safety factor is established for each performance limit of the pipe by taking the ratio of the pipe’s ultimate strength or resistance to the applied load.

External pressure from earth load, groundwater, vacuum and live load applied to the HDD pipe produces (1) a compressive ring thrust in the pipe wall and (2) ring bending deflection. The performance limit of unsupported PE pipe subjected to compressive thrust is ring buckling (collapse). The performance limit of a PE pipe subjected to ring bending (a result of non-uniform external load, i.e. earth load) is ring deflection. See Figure 2.

Time-Dependent Behavior

Both performance limits are proportional to the apparent modulus of elasticity of the PE material. For viscoelastic materials like PE, the modulus of elasticity is a time-dependent property, that is, its value changes with time under load. A newly applied load increment will cause a decrease in apparent stiffness over time. Unloading will result in rebounding or an apparent gain in stiffness. These changes occur because the molecular structure rearranges itself under load. The result is a higher resistance to short term loading than to long-term loading. Careful consideration must be given to the duration and frequency of each load, so that the performance limit associated with that load can be calculated using PE material properties representative of that time period. The same effects occur with the pipe's tensile strength. For instance, during pullback, the pipe's tensile yield strength decreases with pulling time, so the safe (allowable) pulling stress is a function of time.

For viscoelastic materials, the ratio of the applied stress to strain is called the apparent modulus of elasticity, because the ratio varies with load rate. Typical values for the apparent modulus of elasticity at 73°F (23°C) are presented in Table 2. Consult the manufacturer for specific applications.

TABLE 2
Apparent Modulus of Elasticity and Safe Pull Tensile Stress @ 73º F

Typical Apparent Modulus of Elasticity			Typical Safe Pull Stress		
Duration	**HDPE**	**MDPE**	**Duration**	**HDPE**	**MDPE**
Short-term	110,000 psi (800 MPa)	87,000 psi (600 MPa)	30 min	1,300 psi (9.0 MPa)	1,000 psi (6.9 MPa)
10 hours	57,500 psi (400 MPa)	43,500 psi (300 MPa)	60 min	1,200 psi (8.3 MPa)	900 psi (6.02 MPa)
100 hours	51,200 psi (300 MPa)	36,200 psi (250 MPa)	12 hours	1,150 psi (7.9 MPa)	850 psi (5.9 MPa)
50 years	28,200 psi (200 MPa)	21,700 psi (150 MPa)	24 hours	1,100 psi (7.6 MPa)	800 psi (5.5 MPa)

Ring Deflection (Ovalization)

Non-uniform pressure acting on the pipe's circumference such as earth load causes bending deflection of the pipe ring. Normally, the deflected shape is an oval. Ovalization may exist in non-rerounded coiled pipe and to a lesser degree in straight lengths that have been stacked, but the primary sources of bending deflection of directionally drilled pipes is earth load. Slight ovalization may also occur during pullback if the pipe is pulled around a curved path in the borehole. Ovalization reduces the pipe's hydrostatic collapse resistance and creates tensile bending stresses in the pipe wall. It is normal and expected for buried PE pipes to undergo ovalization. Proper design and installation will limit ovalization (or as it is often called "ring deflection") to prescribed values so that it has no adverse effect on the pipe.

Ring Deflection Due to Earth Load

As discussed previously, insitu soil characteristics and borehole stability determine to great extent the earth load applied to directionally drilled pipes. Methods for calculating estimated earth loads, when they occur, are given in the previous section on "Earth and Groundwater Pressure."

Since earth load is non-uniform, the pipe will undergo ring deflection, i.e. a decrease in vertical diameter and an increase in horizontal diameter. The designer can check to see if the selected pipe is stiff enough to limit deflection and provide an adequate safety factor against buckling. (Buckling is discussed in a later section of this chapter.)

The soil surrounding the pipe may contribute to resisting the pipe's deflection. Formulas used for entrenched pipe, such as Spangler's Iowa Formula, are likely not applicable as the HDD installation is different from installing pipe in a trench where the embedment can be controlled. In an HDD installation, the annular space surrounding the pipe contains a mixture of drilling mud and cuttings. The mixture's consistency or stiffness determines how much resistance it contributes. Consistency (or stiffness) depends on several factors including soil density, grain size and the presence of groundwater. Researchers have excavated pipe installed by HDD and observed some tendency of the annular space soil to return to the condition of the undisturbed native soil. See Knight (2001) and Ariaratnam (2001). It is important to note that the researched installations were located above groundwater, where excess water in the mud-cuttings slurry can drain. While there may be consolidation and strengthening of the annular space soil particularly above the groundwater level, it may be weeks or even months before significant resistance to pipe deflection develops. Until further research establishes the soil's contribution to resisting deflection, one option is to ignore any soil resistance and to use Equation 10 which is derived from ring deflection equations published by Watkins and Anderson (1995). (Coincidentally, Equation 10 gives the same deflection as the Iowa Formula with an E′ of zero.)

(10) $$\frac{y}{D} = \frac{\dfrac{0.0125P_E}{E}}{12\,(DR\text{ -}1)^3}$$

WHERE
y = ring deflection, in
D = pipe diameter, in
P_E = Earth pressure, psi
DR = Pipe Dimension Ratio
E = modulus of elasticity, psi
* To obtain ring deflection in percent, multiply y/D by 100.

Ring Deflection Limits (Ovality Limits)

Ovalization or ring deflection (in percent) is limited by the pipe wall strain, the pipe's hydraulic capacity, and the pipe's geometric stability. Jansen observed that for PE, pressure-rated pipe, subjected to soil pressure only, "no upper limit from a practical design point of view seems to exist for the bending strain." On the other hand, pressurized pipes are subject to strains from both soil induced deflection and internal pressure. The combined strain may produce a high, localized outer-fiber tensile stress. However, as the internal pressure is increased, the pipe tends to re-round and the bending strain is reduced. Due to this potential for combined strain (bending and hoop tensile), it is conservative to limit deflection of pressure pipes to less than non-pressure pipes. In lieu of an exact calculation for allowable deflection limits, the limits in Table 3 can be used.

TABLE 3
Design Deflection Limits of Buried Polyehtylene Pipe, Long Term, %*

DR or SDR	21	17	15.5	13.5	11	9	7.3
Deflection Limit (% y/D) Non-Pressure Applications	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Deflection Limit (%y/D) Pressure Applications	7.5	6.0	6.0	6.0	5.0	4.0	3.0

* Deflection limits for pressure applications are equal to 1.5 times the short-term deflection limits given in Table X2.1 of ASTM F-714.

Design deflections are for use in selecting DR and for field quality control. (Field measured deflections exceeding the design deflection do not necessarily indicate unstable or over-strained pipe. In this case, an engineering analysis of such pipe should be performed before acceptance.)

Unconstrained Buckling

Uniform external pressure applied to the pipe either from earth and live load, groundwater, or the drilling slurry creates a ring compressive hoop stress in the pipe's wall. If the external pressure is increased to a point where the hoop stress reaches a critical value, there is a sudden and large inward deformation of the pipe wall, called buckling. Constraining the pipe by embedding it in soil or cementitious grout will increase the pipe's buckling strength and allow it to withstand higher external pressure than if unconstrained. However, as noted in a previous section it is not likely that pipes installed below the groundwater level will acquire significant support from the surrounding mud-cuttings mixture and for pipe above groundwater support may take considerable time to develop. Therefore, until further research is available it is conservative to assume no constraint from the soil. The following equation, known as Levy's equation, may be used to determine the allowable external pressure (or negative internal pressure) for unconstrained pipe.

(11)

$$P_{UA} = \frac{2E}{(1-m^2)} \left(\frac{1}{DR-1}\right)^3 \frac{f_O}{N}$$

WHERE

P_{ua} = Allowable unconstrained pressure, psi

E = Modulus of elasticity (apparent), psi

m = Poisson's Ratio

Long-term loading - 0.45

Short-term loading - 0.35

DR = Dimension ratio (Do/t)

f_o = ovality compensation factor (see figure 3)

N = Safety factor, generally 2.0 or higher

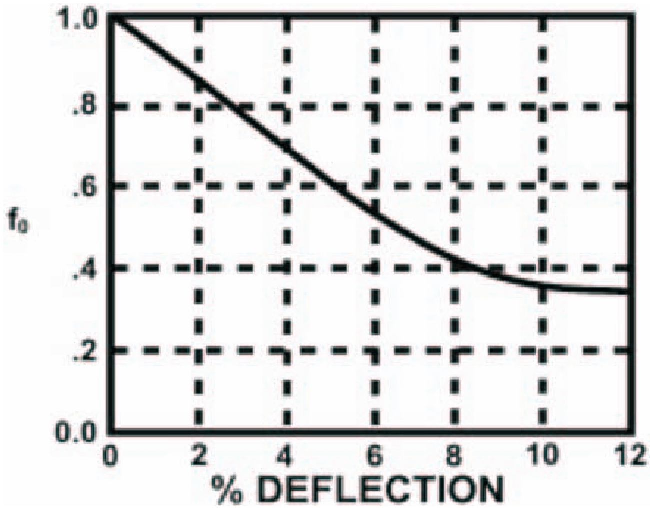

Figure 3 Ovality Compensation Factor

Note that the modulus of elasticity and Poisson's ratio are a function of the duration of the anticipated load. If the safety factor in Levy's equation is set equal to one, the equation gives the critical buckling pressure for the pipe. Table 4 gives values of the critical buckling (collapse) pressure for different DR's of PE pipe. For design purposes, the designer must reduce the values by a safety factor and by ovality compensation. When using this table for determining pipe's resistance to buckling during pullback, an additional reduction for tensile stresses is required, which is discussed in a later section of this chapter. When selecting a modulus to use in Equation 11 consideration should be given to internal pressurization of the line. When the pressure in the pipe exceeds the external pressure due to earth and live load, groundwater and/or slurry, the stress in the pipe wall reverses from compressive to tensile stress and collapse will not occur.

TABLE 4
Critical Buckling (Collapse) Pressure for unconstrained HDPE Pipe*@ 73º F

Service Pipe Life	Units	7.3	9	11	13.5	15.5	17	21
Short term	psi	1003	490	251	128	82	61	31
	ft H_2O	2316	1131	579	297	190	141	72
	in Hg	2045	999	512	262	168	125	64
100 hrs	psi	488	238	122	62	40	30	15
	ft H2O	1126	550	282	144	92	69	35
	in Hg	995	486	249	127	82	61	31
50 yrs	psi	283	138	71	36	23	17	9
	ft H2O	653	319	163	84	54	40	20
	in Hg	577	282	144	74	47	35	18

(Table does not include ovality compensation or safety factor.)

* Full Vacuum is 14.7 psi, 34 ft water, 30 in Hg.
* Axial Tension during pullback reduces collapse strength.

Multipliers for Temperature Rerating

60°F (16°C)	73.4°F (23°C)	100°F (38°C)	120°F (49°C)
1.08	1.00	0.78	0.63

Wall Compressive Stress

The compressive stress in the wall of a directionally drilled PE pipe rarely controls design and it is normally not checked. However, it is included here because in some special cases such as directional drilling at very deep depths such as in landfills it may control design.

The earth pressure applied to a buried pipe creates a compressive thrust stress in the pipe wall. When the pipe is pressurized, the stress is reduced due to the internal pressure creating tensile thrust stresses. The net stress can be positive or negative depending on the depth of cover. Buried pressure lines may be subject to net compressive stress when shut down or when experiencing vacuum. These are usually short-term conditions and are not typically considered significant for design, since the short-term design stress of polyolefins is considerably higher than the long-term design stress. Pipes with large depths of cover and operating at low pressures may have net compressive stresses in the pipe wall. The following equation can be used to determine the net compressive stress:

(12)

$$S_c = \frac{P_S D_O}{288t} - \frac{PD}{2t}$$

WHERE

S_C = Compressive wall stress, psi

P_S = Earth load pressures, psf

D_O = Pipe outside diameter, in

t = Wall thickness, in

P = (Positive) internal pressure, psi

D = Mean diameter, D_O-t, in

The compressive wall stress should be kept less than the allowable compressive stress of the material. For PE3408 HDPE pipe grade resins, 1000 psi is a safe allowable stress.

EXAMPLE CALCULATIONS An example calculation for selecting the DR for an HDD pipe is given in Appendix A.

Installation Design Considerations

After determining the DR required for long-term service, the designer must determine if this DR is sufficient for installation. Since installation forces are so significant, a lower DR (stronger pipe) may be required.

During pullback the pipe is subjected to axial tensile forces caused by the frictional drag between the pipe and the borehole or slurry, the frictional drag on the ground surface, the capstan effect around drill-path bends, and hydrokinetic drag. In addition, the pipe may be subjected to external hoop pressures due to net external fluid head and bending stresses. The pipe's collapse resistance to external pressure given in Equation 2 is reduced by the axial pulling force. Furthermore, the drill path curvature may be limited by the pipe's bending radius. (Torsional forces occur but are usually negligible when back-reamer swivels are properly designed.) Considerable judgment is required to predict the pullback force because of the complex interaction between pipe and soil. Sources for information include experienced drillers and engineers, programs such as DRILLPATH (1) and publications such as ASTM F1962, and the Pipeline Research Council (PRCI) Manual, Installation of Pipelines by Horizontal Directional Drilling, and Engineering Design Guide. Typically, pullback force calculations are approximations that depend on considerable experience and judgment.

The pullback formulas given herein and in DRILLPATH and ASTM F1962 are based on essentially an "ideal" borehole. The ideal borehole behaves like a rigid tunnel with gradual curvature, smooth alignment (no dog-legs), no borehole collapses, nearly complete cuttings removal, and good slurry circulation. The ideal borehole may be approached with proper drilling techniques that achieve a clean bore fully reamed to its final size before pullback. The closer the bore is to ideal; the more likely the calculated pullback force will match the actual.

Because of the large number of variables involved and the sensitivity of pullback forces to installation techniques, the formulas presented in this document are for guidelines only and are given only to familiarize the designer with the interaction that occurs during pullback. Pullback values obtained should be considered only as qualitative values and used only for preliminary estimates. The designer is advised to consult with an experienced driller or with an engineer familiar with calculating these forces. The following discussion assumes that the entry and exit pits of the bore are on the same, or close to the same, elevation.

Pullback Force

Large HDD rigs can exert between 100,000 lbs. to 500,000 lbs. pull force. The majority of this power is applied to the cutting face of the reamer device/tool, which precedes the pipeline segment into the borehole. It is difficult to predict what portion of the total pullback force is actually transmitted to the pipeline being inserted.

The pulling force which overcomes the combined frictional drag, capstan effect, and hydrokinetic drag, is applied to the pull-head and first joint of HDPE pipe. The axial tensile stress grows in intensity over the length of the pull. The duration of the pullload is longest at the pull-nose. The tail end of the pipe segment has zero applied tensile stress for zero time. The incremental time duration of stress intensity along the length of the pipeline from nose to tail causes a varying degree of recoverable elastic strain and viscoelastic stretch per foot of length along the pipe.

The DR must be selected so that the tensile stress in the pipe wall due to the pullback force, does not exceed the permitted tensile stress for the pipe material. Increasing the pipe wall thickness will allow for a greater total pull-force. Even though the thicker wall increases the weight per foot of the pipe, the pullback force within the bore itself is not significantly affected by the increased weight. Hence, thicker wall pipe generally reduces stress. The designer should carefully check all proposed DR's.

Frictional Drag Resistance

Pipe resistance to pullback in the borehole depends primarily on the frictional force created between the pipe and the borehole or the pipe and the ground surface in the entry area, the frictional drag between pipe and drilling slurry, the capstan effect at bends, and the weight of the pipe. Equation 13 gives the frictional resistance or required pulling force for pipe pulled in straight, level bores or across level ground.

(13) $$F_P = mW_B L$$

WHERE

F_P = pulling force, lbs

m = coefficient of friction between pipe and slurry (typically 0.25) or between pipe and ground (typically 0.40)

w_B = net downward (or upward) force on pipe, lb/ft

L = length, ft

When a slurry is present, W_B is the upward buoyant force of the pipe and its contents. Filling the pipe with fluid significantly reduces the buoyancy force and thus the pulling force. Polyethylene pipe has a density near that of water. If the pipe is installed "dry" (empty) using a closed nose-pull head, the pipe will want to "float" on the crown of the borehole leading to the sidewall loading and frictional drag through the buoyancy-per-foot force and the wetted soil to pipe coefficient of friction. Most major pullbacks are done "wet". That is, the pipeline is filled with water as it starts to descend into the bore (past the breakover point). Water is added through a hose or small pipe inserted into the pullback pipe. (See the calculation examples.)

Note: The buoyant force pushing the empty pipe to the borehole crown will cause the PE pipe to "rub" the borehole crown. During pullback, the moving drill mud lubricates the contact zone. If the drilling stops, the pipe stops, or the mud flow stops, the pipe - slightly ring deflected by the buoyant force - can push up and squeeze out the lubricating mud. The resultant "start-up" friction is measurably increased. The pulling load to loosen the PE pipe from being "stuck" in the now decanted (moist) mud can be very high. This situation is best avoided by using thicker (lower DR) pipes, doing "wet" pulls, and stopping the pull only when removing drill rods.

Capstan Force

For curves in the borehole, the force can be factored into horizontal and vertical components. Huey et al.[3] shows an additional frictional force that occurs in steel pipe due to the pressure required by the borehole to keep the steel pipe curved. For bores with a radius of curvature similar to that used for steel pipe, these forces are insignificant for PE pipe. For very tight bends, it may be prudent to consider them. In addition to this force, the capstan effect increases frictional resistance when pulling along a curved path. As the pipe is pulled around a curve or bend creating an angle q, there is a compounding of the forces due to the direction of the pulling vectors. The pulling force, F_C, due to the capstan effect is given in Eq. 14. Equations 13 and 14 are applied recursively to the pipe for each section along the pullback distance as shown in Figure 4. This method is credited to Larry Slavin, Outside Plant Consulting Services, Inc. Rockaway, N.J.

(14) $$F_c = e^{mq}(mW_B L)$$

WHERE

e = Natural logarithm base (e=2.71828)

m = coefficient of friction

q = angle of bend in pipe, radians

w_B = weight of pipe or buoyant force on pipe, lbs/ft

L = Length of pull, ft

$$F_1 = \exp(m_g a)(m_g W_p (L_1 + L_2 + L_3 + L_4))$$

$$F_2 = \exp(m_b a)(F_1 + m_b W_b L_2 + W_b H - m_g W_p L_2 \exp(m_g a))$$

$$F_3 = F_2 + m_b W_b L_3 - \exp(m_b a)(m_g W_p L_3 \exp(m_g a))$$

$$F_4 = \exp(m_b b)(F_3 + m_b W_b L_4 - W_b H - \exp(m_b a)(m_g W_p L_4 \exp(m_g a)))$$

WHERE

H = Depth of bore (ft)

Fi = Pull Force on pipe at Point i (lb)

Li = Horizontal distance of Pull from point to point (ft)

m = Coeff. of friction (ground (g) and borehole (b))

w = Pipe weight (p) and Buoyant pipe weight (lb/ft)

a, b = Entry and Exit angles (radians)

Figure 4 Estimated Pullback Force Calculation

Hydrokinetic Force

During pulling, pipe movement is resisted by the drag force of the drilling fluid. This hydrokinetic force is difficult to estimate and depends on the drilling slurry, slurry flow rate pipe pullback rate, and borehole and pipe sizes. Typically, the hydrokinetic pressure is estimated to be in the 30 to 60 kPa (4 to 8 psi) range.

(15) $$F_{HK} = p \frac{\pi}{8} (D_H^{\ 2} - OD^2)$$

WHERE

F_{HK} = hydrokinetic force, lbs

p = hydrokinetic pressure, psi

D_H = borehole diameter, in

OD = pipe outside diameter, in

ASCE MOP 108 suggests a different method for calculating the hydrokinetic drag force. It suggests multiplying the external surface area of the pipe by a fluid drag coefficient of 0.025 lb/in^2 after Puckett (2003). The total pull back force, F_T, then is the combined pullback force, F_P, plus the hydrokinetic force, F_{HK}. For the example shown in Figure 4, F_P equals F_4.

Tensile Stress During Pullback

The maximum outer fiber tensile stress should not exceed the safe pull stress. The maximum outer fiber tensile stress is obtained by taking the sum of the tensile stress in the pipe due to the pullback force, the hydrokinetic pulling force, and the tensile bending stress due to pipe curvature. During pullback it is advisable to monitor the pulling force and to use a "weak link" (such as a pipe of higher DR) mechanical break- away connector or other failsafe method to prevent over-stressing the pipe.

The tensile stress occurring in the pipe wall during pullback is given by Eq. 16.

(16)

$$s_t = \frac{F_T}{\pi t\,(D_{OD} - t)} + \frac{E_T D_{OD}}{2R}$$

WHERE

s_T = Axial tensile stress, psi

F_T = Total pulling force, lbs

t = Minimum wall thickness, in

D_{OD} = Outer diameter of pipe, in

E_T = Time-dependent tensile modulus, psi

R = Minimum radius of curvature in bore path, in

The axial tensile stress due to the pulling force should not exceed the pipe's safe pull load. As discussed in a previous section, the tensile strength of PE pipe is load-rate sensitive. Time under load is an important consideration in selecting the appropriate tensile strength to use in calculating the safe pull load. During pullback, the pulling force is not continually applied to the pipe, as the driller must stop pulling after extracting each drill rod in order to remove the rod from the drill string. The net result is that the pipe moves the length of the drill rod and then stops until the extracted rod is removed. Pullback is an incremental (discrete) process rather than a continuous process. The pipe is not subjected to a constant tensile force and thus may relax some between pulls. A one-hour modulus value might be safe for design, however, a 12-hour value will normally minimize "stretching" of the pipeline. Table 5 and Table 6 give safe pull loads for HDPE pipes based on a 12-hour value. Allowable safe pullback values for gas pipe are given in ASTM F-1807, "Practice for Determining Allowable Tensile Load for Polyethylene (PE) Gas Pipe during Pull-In Installation".

After pullback, pipe may take several hours (typically equal to the duration of the pull) to recover from the axial strain. When pulled from the reamed borehole, the pull-nose should be pulled out about 3% longer than the total length of the pull. The elastic strain will recover immediately and the viscoelastic stretch will "remember" its original length and recover overnight. One does not want to come back in the morning to discover the pull-nose sucked back below the borehole exit level due to stretch recovery and thermal-contraction to an equilibrium temperature. In the worst case, the driller may want to pull out about 4% extra length (40 feet per 1000 feet) to insure the pull-nose remains extended beyond the borehole exit.

TABLE 5
Safe Pull Load @ 12 hours for HDPE Pipes (Iron Pipe Size)

IPS Size	DR = Nom. OD	9 Lbs.	11 Lbs.	13.5 Lbs.	17 Lbs.
1.25	1.660	983	823	683	551
1.5	1.900	1288	1078	895	722
2	2.375	2013	1684	1398	1128
3	3.500	4371	3658	3035	2450
4	4.500	7226	6046	5018	4050
6	6.625	15661	13105	10876	8779
8	8.625	26544	22212	18434	14880
10	10.750	41235	34505	28636	23115
12	12.750	58006	48538	40282	32516
14	14.000	69937	58522	48568	39204
16	16.000	91347	76437	63435	51205
18	18.000	115611	96741	80285	64806
20	20.000	142729	119433	99118	80008
22	22.000	172703	144514	119932	96809
24	24.000	205530	171983	142729	115211
26	26.000	241213	201841	167509	135213
28	28.000	279750	234088	194271	156815
30	30.000	321141	268724	223015	180017
32	32.000	N.A.	305748	253741	204819
34	34.000	N.A.	345161	286450	231222
36	36.000	N.A.	386962	321141	259224
42	42.000	N.A.	N.A.	437109	352833
48	48.000	N.A.	N.A.	N.A.	460843
54	54.000	N.A.	N.A.	N.A.	N.A.

TABLE 6
Safe Pull Load @ 12 hours for HDPE Pipes (Ductile Iron Pipe Size)

DIPS Size	DR = Nom. OD	9 bs.	11 Lbs.	13.5 Lbs.	17 Lbs.
4	4.800	8221	6879	5709	4608
6	6.900	16988	14215	11797	9523
8	9.050	29225	24455	20295	16382
10	11.100	43964	36788	30531	24644
12	13.200	62173	52025	43176	34851
14	15.300	83529	69895	58006	46822
16	17.400	108032	90399	75022	60558
18	19.500	135682	113536	94224	76057
20	21.600	166480	139306	115611	93321
24	25.800	237516	198748	164942	133141
30	32.000	N.A.	305748	253741	204819
36	38.300	N.A.	N.A.	363487	293406
42	44.500	N.A.	N.A.	N.A.	396087
48	50.800	N.A.	N.A.	N.A.	516177

External Pressure During Installation

During pullback it is reasonable to assume that the borehole remains stable and open and that the borehole is full of drilling slurry. The net external pressure due to fluid in the borehole, then, is the slurry head, P_{MUD}. This head can be offset by pulling the pipe with an open nose or filling the pipe with water for the pullback. However, this may not always be possible, for instance when installing electrical conduit. In addition to the fluid head in the borehole, there are also dynamic sources of external pressure:

1. If the pulling end of the pipe is capped, a plunger action occurs during pulling which creates a mild surge pressure. The pressure is difficult to calculate. The pipe will resist such an instantaneous pressure with its relatively high short-term modulus. If care is taken to pull the pipe smoothly at a constant speed, this calculation can be ignored. If the pipe nose is left open, this surge is eliminated.

2. External pressure will also be produced by the frictional resistance of the drilling mud flow. Some pressure is needed to pump drilling mud from the reamer tool into the borehole, then into the pipe annulus, and along the pipe length while conveying reamed soil debris to the mud recovery pit. An estimate of this short term hydrokinetic pressure may be calculated using annular flow pressure loss formulas borrowed from the oil well drilling industry. This external pressure is dependent upon specific drilling mud properties, flow rates, annular opening, and hole configuration. This is a short-term installation condition. Thus, HDPE pipe's short-term external differential pressure capabilities are compared to the

actual short-term total external pressure during this installation condition. Under normal conditions, the annular-flow back pressure component is less than 4-8 psi.

In consideration of the dynamic or hydrokinetic pressure, P_{HK}, the designer will add additional external pressure to the slurry head:

(17) $$P_N = P_{MUD} + P_{HK} - P_I$$

Where the terms have been defined previously.

Resistance to External Collapse Pressure During Pullback Installation

The allowable external buckling pressure equation, Eq.11, with the appropriate time-dependent modulus value can be used to calculate the pipe's resistance to the external pressure, P_N, given by Eq.17 during pullback. The following reductions in strength should be taken:

1. The tensile pulling force reduces the buckling resistance. This can be accounted for by an additional reduction factor, f_R. The pulling load in the pipe creates a hoop strain as described by Poisson's ratio. The hoop strain reduces the buckling resistance. Multiply Eq.11 by the reduction factor, f_R to obtain the allowable external buckling pressure during pullback.

(18) $$F_R = \sqrt{(5.57 - (r + 1.09)^2)} - 1.09$$

(19) $$r = \frac{s_T}{2S}$$

WHERE

s_T = calculated tensile stress during pullback (psi)

s = safe pull stress (psi)

Since the pullback time is typically several hours, a modulus value consistent with the pullback time can be selected from Table 2.

Bending Stress

HDD river crossings incorporate radii-of-curvature, which allow the HDPE pipe to cold bend within its elastic limit. These bends are so long in radius as to be well within the flexural bending capability of polyethylene pipe. PE3408 of SDR 11 can be cold bent to 25 times its nominal OD (example: for a 12" SDR 11 HDPE pipe, the radius of curvature could be from infinity down to the minimum of 25 feet, i.e., a 50-foot diameter circle). Because the drill stem and reaming rod are less flexible, normally polyethylene can bend easily to whatever radius the borehole steel drilling and reaming shafts can bend because these radii are many times the pipe

OD. However, in order to minimize the effect of ovaling some manufacturers limit the radius of curvature to a minimum of 40 to 50 times the pipe diameter. As in a previous section, the tensile stress due to bending is included in the calculations.

Thermal Stresses and Strains

HDD pipeline crossings are considered to be fully restrained in the axial direction by the friction of the surrounding soil. This is generally accepted to be the case, though, based on uncased borings through many soil types with the progressive sedimentation and borehole reformation over a few hours to several months. This assumption is valid for the vast majority of soil conditions, although it may not be completely true for each and every project. During pipe installation, the moving pipeline is not axially restrained by the oversize borehole. However, the native soil tends to sediment and embed the pipeline when installation velocity and mud flow are stopped, thus allowing the soil to grip the pipeline and prevent forward progress or removal. Under such unfortunate stoppage conditions, many pipelines may become stuck within minutes to only a few hours.

The degree to which the pipeline will be restrained after completed installation is in large part a function of the sub-surface soil conditions and behavior, and the soil pressure at the depth of installation. Although the longitudinal displacement due to thermal expansion or contraction is minimal, the possibility of its displacement should be recognized. The polyethylene pipe should be cut to length only after it is in thermal equilibrium with the surrounding soil (usually overnight). In this way the "installed" versus "operating" temperature difference is dropped to nearly zero, and the pipe will have assumed its natural length at the existing soil/water temperature. Additionally, the thermal inertia of the pipe and soil will oppose any brief temperature changes from the flow stream. Seasonal temperature changes happen so slowly that actual thermally induced stresses are usually insignificant within polyethylene for design purposes.

Torsion Stress

A typical value for torsional shear stress is 50% of the tensile strength. Divide the transmitted torque by the wall area to get the torsional shear stress intensity. During the pullback and reaming procedure, a swivel is typically used to separate the rotating cutting head assembly from the pipeline pull segment. Swivels are not 100% efficient and some minor percent of torsion will be transmitted to the pipeline. For thick wall HDPE pipes of SDR 17, 15.5, 11, 9 and 7, this torsion is not significant and usually does not merit a detailed engineering analysis.

EXAMPLE CALCULATIONS Example Calculations are given in Appendix A.

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APPENDIX A

Design Calculation Example for Service Loads (Post-Installation)

Example 1

A 6″ IPS DR 11 HDPE pipe is being pulled under a railroad track. The minimum depth under the track is 10 ft. Determine the safety factor against buckling.

GIVEN PARAMETERS

OD = 6.625 in

Nominal Pipe OD

DR = 11 Pipe

Dimension Ratio

H = 10 ft.

Max. Borehole Depth

g_s = 120 lbf/ft^3

Unit Weight of Soil

P_{Live} = 1,100 lbf/ft^2

E-80 Live Load

PE Material Parameters

Wheel loading from train will be applied for several minutes without relaxation. Repetitive trains crossing may accumulate. A conservative choice for the apparent modulus is the 1000-hour modulus.

E_{mid} = 43,700 psi

μ= 0.45 Long-Term Poisson's Ration

Soil and Live Load Pressure on Pipe (Assuming that the earth load equals the prism load is perhaps too conservative except for a calculation involving dynamic surface loading.)

$P = (g_s H + P_{Live})\ 1\ ft^2/144\ in^2$

P = 15.97 psi

Ring Deflection resulting from soil and live load pressures assuming no side support is given by equation 10.

$$\%\frac{y}{D} = \frac{0.0125P}{\frac{E_{mid}}{12(DR-1)^3}}$$

% y/D = 5.482 Percent deflection from soil loads

Determine critical unconstrained buckling pressure based on deflection from loading and safety factor using Eq. 11

f_o = 0.56 Ovality compensation factor for 5.5% ovality from Figure 3

$$P_{UC} = \frac{2E_{mid}}{(1-m^2)}\left(\frac{1}{DR-1}\right)^3 f_O$$

P_{UC} = 61.37 psi

Critical unconstrained buckling pressure (no safety factor)

$$SF_{cr} = \frac{P_{UC}}{P}$$

SF_{cr} = 3.84 Safety factor against buckling

Example 2

A 6" IPS DR 13.5 HDPE pipe is being pulled under a small river for use as an electrical duct. At its lowest point, the pipe will be 18 feet below the river surface. Assume the slurry weight is equal to 75 Ib/cu.ft. The duct is empty during the pull. Calculate a) the maximum pulling force and b) the safety factor against buckling for the pipe. Assume that the pipe's ovality is 3% and that the pulling time will not exceed 10 hours.

Solution

Calculate the safe pull strength or allowable tensile load.

OD = 6.625in. - Pipe outside diameter

DR = 13.5 - Pipe dimension ratio

T_{allow} = 1150 psi - Typical safe pull stress for HDPE for 12-hour pull duration

$$F_s = \pi T_{allow}\ OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2}\right)$$

$F_S = 1.088 \times 10^4$ lbf

Safe pull strength for 6" IPS DR 13.5 HDPE pipe assuming 10-hour maximum pull duration

Step 1

Determine the critical buckling pressure during Installation for the pipe (include tensile reduction factor assuming the frictional drag during pull results in 1000 psi longitudinal pipe stress)

E = 57,500 ps - Apparent modulus of elasticity (for 10 hours at 73 degrees F)

μ = 0.45 - Poisson's ratio (long term value)

fo = 0.76 - Ovality compensation factor (for 3% ovality)

R = 0.435 - Tensile ratio (based on assumed 1000 psi pull stress calculation)

$$f_R = \sqrt{5.57 - (r + 1.09)^2} - 1.09$$

$f_R = 0.71$

Tensile Reduction Factor

$$Pcr = \frac{2E}{(1-\mu^2)}\left(\frac{1}{DR - 1}\right)^3 \cdot f_O \cdot f_R$$

$P_{CR} = 39.90$

Critical unconstrained buckling pressure for DR 13.5 pipe without safety factor

Step 2

Determine expected loads on pipe (assume only static drilling fluid head acting on pipe, and borehole intact with no soil loading)

g_{slurry} = 75 lbf/ft^3, drilling fluid weight

H = 18 ft, Maximum bore depth

$$P_{slurry} = Hg_{slurry}\left(\frac{1ft^2}{144in^2}\right)$$

P_{slurry} = 9.36 psi
Total static drilling fluid head pressure if drilled from surface

Step 3
Determine the resulting safety factor against critical buckling during installation

$$SF_{CR} = \frac{P_{CR}}{P_{slurry}}$$

SF_{CR} = 4.25

Safety factor against critical buckling during pull

Example 3
Determine the safety factor for long-term performance for the communication duct in Example 2. Assume there are 10 feet of riverbed deposits above the borehole having a saturated unit weight of 110 Ib/ft^3. (18 feet deep, 3% initial ovality)

Solution

Step 1
Determine the pipe soil load (Warning: Requires input of ovality compensation in step 4.

E long = 28,200 psi - Long-term apparent modulus

g_W = 62.4 lbf/ft.^{3} - Unit weight of water

H = 18 ft Max. - Borehole depth

g_S = 110 lbf/ft.^{3} - Saturated unit weight of sediments

GW = 18 ft - Groundwater height

C = 10ft. - Height of soil cover

OD = 6.625 in - Nominal pipe OD

DR = 13. - Pipe dimension ratio

μ = 0.4 - Long-term Poisson's ratio

$$P_{soil} = (g_S - g_W)\,C\,\left(\frac{1ft^2}{144in^2}\right)$$

P_{soil} = 3.30 psi

Prism load on pipe from 10' of saturated cover (including buoyant force on submerged soil)

Step 2

Calculate the ring deflection resulting from soil loads assuming no side support.

$$\%(y/D) = \frac{0.0125 \times P_{soil} \times 100}{[\frac{E_{long}}{12\ (DR - 1)^3}]}$$

% (y/D) = 3.43 Percent deflection from soil loads

t = OD/DR t =0.491 in

Step 3

Determine the long-term hydrostatic loads on the pipe

$$P_W = (\frac{GW}{2.31\ \text{ft/psi}}) + P_{soil}$$

P_W = 11.09

External pressure due to groundwater head

$$g_{slurry} = 75\ \text{lb/cu.ft.}^3$$

Unit weight of drilling fluid

$$P_{slurry} = g_{slurry}\ H\ (\frac{1\text{ft}^2}{144\text{in}^2})$$

P_{slurry} = 9.37 psi

External pressure due to slurry head

$$P_W > P_{slurry}$$

Therefor use PW for buckling load

Step 4

Determine critical unconstrained buckling pressure based on deflection from loading

f_o = 0. 64 5% Ovality Compensation based on 3% initial ovality and 2% deflection

$$P_{UC} = \frac{2E_{long}}{(1 - m^2)}\ (\frac{1}{DR - 1})^3\ f_O$$

P_{UC} = 23.17 psi

Critical unconstrained buckling pressure (no safety factor)

SF_{CR} = 2.08

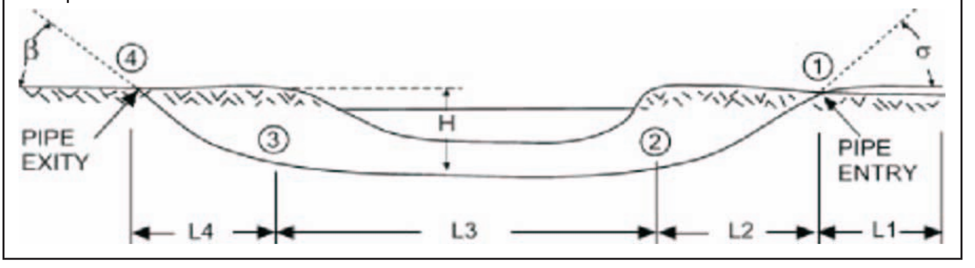
$$SF_{CR} = \frac{P_{UC}}{P_W} \qquad SF_{CR} = 2.08$$

Safety Factor against buckling pressure of highest load (slurry)

APPENDIX B

Design Calculations Example for Pullback Force

Example 1



Find the estimated force required to pull back pipe for the above theoretical river crossing using Slavin's Method. Determine the safety factor against collapse. Assume the HDPE pipe is 35 ft deep and approximately 870 ft long with a 10 deg. entry angle and a 15 deg. exit angle. Actual pullback force will vary depending on backreamer size, selection, and use; bore hole staying open; soil conditions; lubrication with bentonite; driller expertise; and other application circumstances.

PIPE PROPERTIES

Outside Diameter

OD = 24 in - Long-term Modulus - E_{long} = 28,250 psi

Standard Dimension Ratio

DR = 11 - 24 hr Modulus - E_{24hr} = 56,500 psi

Minimum wall thickness

t = 2.182 in - Poisson's ratio (long term) - μ = 0.45 - Safe Pull Stress (24 hr) - s_{pb} = 1,100 psi

PATH PROFILE

H = 35 ft Depth of bore

g_{in} = 10 deg Pipe entry angle

g_{ex} = 15 deg Pipe exit angle

L_1 = 100 ft Pipe drag on surface (This value starts at total length of pull, approximately 870 ft. then decreases with time. Assume 100 ft remaining at end of pull)

L_{cross} = 870 ft

PATH LENGTH (DETERMINE L2 AND L4)

Average Radius of Curvature for Path at Pipe Entry g_{in} is given in radians

$$R_{avgin} = 2H/g_{in}^{\,2}$$

$R_{avgin} = 2.298 \times 10^3$ ft

Average Radius of Curvature for Path at Pipe Exit

$$R_{agex} = 2H/g_{ex}^{\,2}$$

$R_{agex} = 1.021 \times 10^3$ ft

Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Entry

$$L_2 = 2H/g_{in}$$

$L_2 = 401.07$ ft

Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Exit

WHERE

L_2 & L_4 = horizontal transition distance at bore exit & entry respectively.

DETERMINE AXIAL BENDING STRESS

$R = R_{avgex}$ - Min. Radius for Drill path

$R = 1.021 \times 10^3$ ft

OD = 24 in

Radius of curvature should exceed 40 times the pipe outside diameter to prevent ring collapse.

r = 40 OD

r = 80 ft Okay

R > r

Bending strain

$e_a = OD/2R$

$e_a = 9.79 \times 10^{-4}$ in/in

WHERE

e_a = bending strain, in/in

OD = outside diameter of pipe, in

R = minimum radius of curvature, ft

Bending stress

$S_a = E_{24hr}e_a$

sa = 55.32 psi

WHERE

S_a = bending stress, psi

FIND PULLING FORCE

Weight of Empty Pipe

$P_w = 3.61 \times 10^{-2}$ lbf/in^3

$g_a = 0.95$

$g_b = 1.5$

$w_a = \pi OD2\ (DR-1/DR2) r_w\ g_a\ 12$ in/ft

$w_a = 61.54$ lbf/ft

Net Upward Buoyant Force on Empty Pipe Surrounded by Mud Slurry

$W_b = \pi(OD^2/4)\ r_w\ g_b - w_a$

$w_b = 232.41$ lbf/ft

WHERE

r_w = density of water, lb/in3

g_a = specific gravity of the pipe material

g_b = specific gravity of the mud slurry

w_a = weight of empty pipe, lbf/ft

w_b = net upward buoyant force on empty pipe surrounded by mud slurry

DETERMINE PULLBACK FORCE ACTING ON PIPE

See figure:

$L_1 = 100$ ft - $v_a = 0.4$

$L_2 = 401.07$ft - $v_b = 0.25$

$L_3 = 200$ft - $\sigma = g_{in}$ - $\sigma = 10$ deg

$L_4 = 267.38$ - $\beta = g_{ex}$ - $\beta = 15$ deg

$L_3 = L_{cross} - L_2 - L_4$ - $L_3 = 201.549$ft

$T_A = \exp(v_a\ \sigma)\ [v_a\ w_a\ (L_1 + L_2 + L_3 + L_4)]$

$T_A = 2.561 \times 104$ lbf

$T_B = \exp(v_b\ \sigma)\ (T_A + v_b\ [w_b]\ L2 + w_b\ H - v_a\ w_a\ L_2 \exp(v_b\ \sigma))$

$T_B = 4.853 \times 104$ lbf

$T_C = TB + v_b\ [wb]\ L_3 - \exp(v_b\ \sigma)\ (v_a\ w_a\ L_3.\exp(v_a\ \sigma))$

$T_C = 5.468 \times 104$ lbf

$T_D = \exp(v_b\ \sigma)\ [T_C + v_b\ [w_b]\ L_4 - w_b\ H - \exp(v_b\ \sigma)\ (v_a\ w_a\ L_4 \exp(v_b\ \sigma))]$

$T_D = 5.841 \times 104$ lbf

WHERE

T_A = pull force on pipe at point A, lbf

T_B = pull force on pipe at point B. lbf

T_C = pull force on pipe at point C, lbf

TD = pull force on pipe at point D, lbf

L1 = pipe on surface, ft

L2 = horizontal distance to achieve desired depth, ft

L3 = additional distance traversed at desired depth, ft

L4 = horizontal distance to rise to surface, ft

v_a = coefficient of friction applicable at the surface before the pipe enters bore hole

v_b = coefficient of friction applicable within the lubricated bore hole or after the (wet) pipe exits

σ = bore hole angle at pipe entry, radians

β = bore hole angle at pipe exit, radians

(refer to figure at start of this appendix)

HYDROKINETIC PRESSURE

ΔP = 10 psi

Dh = 1.5 OD

Dh = 36in

$\Delta T = \Delta P\ (\pi/8)\ (Dh^2 - OD^2)$

ΔT = 2.82 x 103lbf

WHERE:

ΔT = pulling force increment, lbf

ΔP = hydrokinetic pressure, psi

Dh = back reamed hole diameter, in

Compare Axial Tensile Stress with Allowable Tensile Stress During Pullback of 1,100 psi:

Average Axial Stress Acting on Pipe Cross-section at Points A, B. C, D

$$s_1 = \left(Ti + \Delta T\right) \quad \left(\frac{1}{\pi OD^2}\right)\ \left(\frac{DR^2}{DR - 1}\right)$$

s1 = 190.13 psi <1,100 psi OK

s2 = 343.40 psi <1,100 psi OK

s3 = 384.55 psi <1,100 psi OK

s4 = 409.48 psi <1,100 psi OK

WHERE

Ti = TA, TB, TC, TD (lbf)

si = corresponding stress, psi

Breakaway links should be set so that pullback force applied to pipe does not exceed 1,100 psi stress.

ID = OD - 2t

Fb = $s_{pb}\ (\pi/4)(OD^2 - ID^2)$

Fb = 1.64 x 105 lbf

DETERMINE SAFETY FACTOR AGAINST RING COLLAPSE DURING PULLBACK

External Hydraulic Load

External static head pressure

P_{ha} = (1.5) (62.4 lbf/ft3) (H)

Pha = 22.75 psi

Combine static head with hydrokinetic pressure

$P_{effa} = P_{ha} + \Delta P$

P_{effa} = 32.75 psi

CRITICAL COLLAPSE PRESSURE

Resistance to external hydraulic load during pullback

f_o = 0.76 Ovality compensation factor (for 3% ovality)

$r = S_4/2S_{Pb}$

r = 0.186

Tensile ratio (based on 1,100 psi pull stress calculation)

Tensile reduction factor

P_{CR} = 96.41 psi

SAFETY FACTOR AGAINST COLLAPSE

SF = Pcr/Pha

F = 4.238

WHERE

P_{ha} = applied efflective pressure due to head of water of drilling

P_{cr} = calculated critical buckling pressure due to head of water of drilling fluid, psi

SF = Safety Factor