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DESIGN OF BURIED PIPELINE SUBJECTED TO LARGE FAULT MOVEMENTS

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ABSTRACT

The responses of the buried pipeline subjected to large fault movements that resulted in compressional stresses were investigated using a finite element approach. Since the pipeline was expected to experience large deformations and high stresses near the fault crossing, plastic pipe elements and non-linear soil springs were used in the analysis model. The deformed shape of the pipe and the stress and strain distributions along the pipeline and around the circumference of the pipe were calculated and examined. It is interesting to note that although the maximum axial member force occurs at the fault crossing location, the maximum strain, which is a more critical design parameter than the axial member force in the assessment of pipe break potential, occur some distance away from the fault break point.

KEYWORDS

Fault movement, non-linear, plastic deformation, pipe-break, buried pipe, compressional strain, reverse-slip, fault crossing, finite element, circumference

INTRODUCTION

The pipeline transmitting oil from remote sources to target locations can be hundreds of miles long, covers many geographical regions and often crosses many active faults. Since a break of the pipeline at one location can render the entire pipeline functionless, design of the pipeline to accommodate large fault movements is an important task in the pipeline design.

For pipelines that are above the ground and flexibly supported, the fault movements can often be accommodated by the flexibility of the pipeline. However, when the pipeline is required to be buried in the ground for security, environmental or other reasons, the pipeline is confined by the surrounding soil to a varying degree depending on the rigidity of the soil. Previous studies have shown that the firmer the surrounding soil, the shorter the segment of the pipeline to absorb the fault movements, the higher the strain is induced in the pipeline. Many buried pipelines around active fault areas have indeed been damaged in recent earthquakes. Ariman (1983) in his "review of buckling and rupture failures in pipelines due to large ground deformations" found that the 1972 Maragua earthquake caused surfacial displacement along four prominent strike-slip faults through the downtown area of the city and nearly all water mains crossing the fault ruptured. Youd (1973) examined damages of the 1971 San Fernando earthquake and found that some ductile steel pipes that could withstand the ground shaking were unable to resist large permanent fault movements. McCaffrey and O'Rourke (1983) investigated the same earthquake and came to the same conclusion that the highest concentration of pipeline damage occurred within the zones of permanent fault displacement. It appears that the key to a successful design of the buried pipeline to resist large fault movements is to provide adequate flexibility to the pipeline such that the relative movements across the fault can be accommodated over a long segment of the pipe without introducing excessive strain around the fault break location.

For buried pipeline subjected to fault movements that result in tensile stresses in the pipeline, the Newmark-Hall procedure and the Kennedy *et al* procedure are commonly used in the analysis to assess the resulting stress and strain in the pipe. Since the procedures assume that the pipeline behaves like a cable, they are not applicable to pipelines subjected to compressional fault movements such as those associated with reverse-slip faults. This restriction was clearly described in their papers (Newmark 1975; Kennedy 1977). Wang *et al.* (1985) refined the procedure to better satisfy the equilibrium condition at the fault crossing location and extended the applicability of the improved procedure to both tension and compression conditions. Recently, Wang (1995) examined the responses of the buried pipeline using a non-linear cantilever beam theory for the large deflection segment and a beam on elastic foundation theory for the small deflection section of the pipeline. In Wang's approach, the deformed shape in the large deflection segment is assumed to be similar to a cantilever deflection instead of a constant curvature deflection as assumed by the Kennedy approach. In this paper, the deformed shape of the buried pipeline subjected to large fault movements that resulted in compressional stresses were investigated.

COMPUTER ANALYSIS MODEL

Since the above mentioned procedures were not applicable to buried pipes subjected to compression cases, a finite element approach was employed. The finite element approach utilizes equilibrium equations with given boundary conditions and can provide accurate results without any deformation assumption. A few finite element analysis programs are commercially available. Hart *et. al.* (1995) used PIPLIN (1991) to perform fault crossing resistant design for the All American oil transmission pipeline crossing the San Andreas fault. In this paper, the computer code ANSYS (1994) was employed for the analysis.

Modeling of Pipeline

In this finite element approach, the pipeline was modeled by a series of pipe elements. Since large deformations and high stress gradients are expected near the fault break location, small pipe elements were used for pipeline near the fault and large pipe elements were used for pipe sections remote from the fault. Since the pipeline was expected to experience large deformations and high stresses near the fault crossing and small deformations at remote locations, these pipe elements must possess both elastic and plastic deformation capabilities. Therefore, plastic pipe elements were used to represent the pipe.

Soil-Pipeline Interaction

For a buried pipeline, the rigidity of the surrounding soil restricts the free movement of the pipe, and vise versa. This is due to the soil-pipeline interaction effect. Since the interaction effect in general will induce more strain in the pipe for a prescribed fault movement than its unrestricted counterpart, the fault crossing resistant analysis without considering soil-pipe interaction generally will not lead to acceptable results.

To simulate the constraints of the surrounding soil, the rigidity of the soil was represented by series of equivalent springs attached to the pipe in the longitudinal and the transverse directions. The non-linear characteristics of the soil around the pipe were represented by non-linear springs. This is necessary to simulate the yield condition of the soil for large lateral displacements and the slip condition between the pipe and the soil for large axial displacements.

Longitudinal Restraint

For granular soil, the soil friction force increases linearly with displacement until it reaches its maximum value. The maximum force, tu, can be expressed as follow (ASCE 1984):

$$tu = 0.5 * \pi D\gamma H(1+K_0) \tan \delta$$

where:
K_0 = coefficient of soil pressure at rest
H = depth from the ground surface to center of the pipeline
D = external pipe diameter
γ = effective unit weight of soil
δ = interface angle of friction between soil and pipeline

Horizontal Restraint

For granular soil, the horizontal soil resistance is a non-linear function of the pipe movement. However, for all design purposes, the resistant force can be assumed to increase linearly with displacement until it reaches the maximum value. The maximum force, pu, can be expressed as follow:

$$pu = \gamma H N_{qh} D$$

where:
γ = effective unit weight of soil
H = depth from the ground surface to center of the pipeline
D = external pipe diameter
N_{qh} = horizontal bearing capacity factor

Vertical Restraint

The vertical resistance of soil to the movement of buried pipe depends not only on the type of soil but also on the direction of the pipe movement. For a shallowly buried pipeline, the resistance to the downward movement is much greater than that to the upward movement and should be properly taken into account in the analysis.

For granular soil, the upward ultimate resistance can be determined from the following equation:

$$qu = H \gamma Nqv D$$

where:
γ = effective unit weight of soil
H = depth from the ground surface to center of the pipeline
D = external pipe diameter
Nqv = vertical bearing capacity factor

For granular soil, the downward ultimate resistance can be determined from the following equation:

$$qu = c\, Nc\, B + \gamma H\, Nq\, B + 0.5 * \gamma B^2\, Nr$$

where:
γ = effective unit weight of soil

c = soil undrain shear strength
B = projected width of contact area of pipeline with soil
Nc, Nq, Nr = bearing capacity factors for horizontal strip footings, vertically loaded in the downward direction (ASCE 1984)

DESCRIPTION OF PROBLEM

A steel pipe with a diameter of 36 inches (914 mm) and a wall thickness of 0.56 inches (14mm) was buried 4 feet (1.2m) in the ground. The pipeline crossed the fault at an angle of 70 degrees. The maximum fault movements were estimated by geotechnical consultants to be 5 feet (1.5m) horizontal and 1.4 feet (0.4m) axially in compression.

COMPUTER MODEL and ANALYSIS RESULTS

Following the above description, a computer model was developed. Since the model and the expected deformed shape with respect to the fault crossing point are symmetric, only one-half of the pipeline was necessary to be analyzed and one-half of the design fault movements were used as input.

Using the model and the input given above, several analyses were performed using the computer program ANSYS. The calculated soil springs were varied to cover a range of possible soil conditions. Since the analysis involves large deformation, small displacement increments are necessary to correctly keep track of the intermediate pipeline deformed position, the load-response path. This is important to encourage convergence and is critical to arrive a correct result because plasticity is a non-conservative, path-dependent phenomenon. Due to the geometry and material non-linearity, small elements used for the long pipeline etc., the computer runs were very time consuming and the use of symmetry turned out to be very helpful in reducing the computer execution time.

The deformations of the pipe along the pipe center line and the stress and strain distributions around the circumference of the pipe were generated in the vicinity of the fault crossing and at the end of the pipe. The strain near the intersection of the fault and the pipeline was used to compare against the maximum strain limit and the deformations at the end of the pipe was used to assure that sufficient length of the pipe was included in the analysis.

The deformed position of the pipeline was calculated. Due to symmetry, the pipe at the intersection point was deformed 2.5 feet (0.76M), and reduced parabolically. A plot of the pipe deformed shape is shown in Fig. 1.

A close examination of the computer output in the area close to the fault reveal that the maximum axial member force was about 1850 kips (8140 KN) and occurred at the fault crossing location as expected. At this location, the strain due to bending moment is zero and the axial strain is about 0.001 along the entire circumference. However, the maximum strain, which is a more critical design parameter than the axial member force in the assessment of pipe break potential, occurred some distance away from the fault break point. For a case analyzed, the maximum equivalent plastic strain was about 0.017 and occurred at approximately 20 feet (6 m) from the fault intersection point.

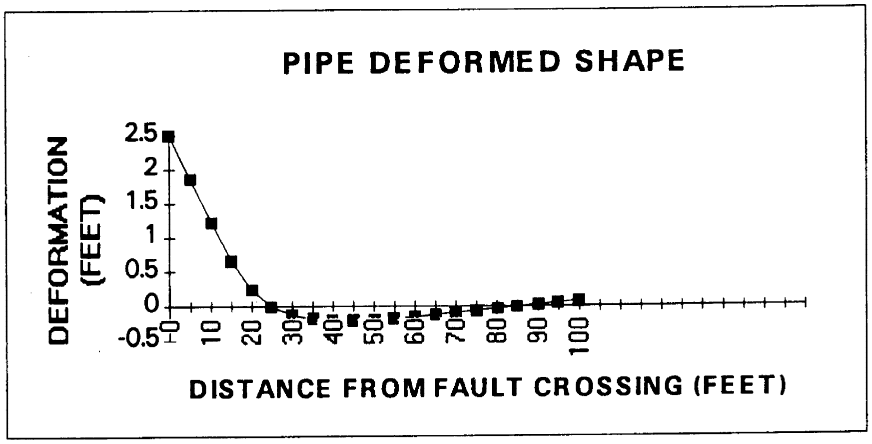

Fig. 1 Pipeline Deformed Shape

DISCUSSION AND CONCLUSION

Modern x-grade pipeline steels generally can accommodate a strain level of 5% or more and a local strain of 15% or more without rupture. An acceptable tensile strain ranging from 2% to 6% was generally suggested depending on degree of quality control and extent of weld inspection (Kenndy 1977). However, tests have shown that pipes can take substantially less strain under compression than under tension. Langner (1974) recommended a formula for the lower bound compressive strain associated with incipient wrinkling under pure bending condition;

$$\varepsilon = \frac{50}{(D/t)}\%$$

Where D and t are diameter and thickness of the pipe.

Boukamp et. al. (1973) performed experiments on large diameter pipes under combined loadings and found that there were reserve deformation capability in the pipe cross section beyond the incipient wrinkling strain. Therefore, compressive strains substantially higher than the incipient wrinkling strain were generally used. Hart et. al. (1995) used a compressive strain of 0.0125 for a diameter to thickness ratio of 107. Considering the favorable diameter-to-thickness ratio (D/t=64), the 0.017 strain is considered acceptable.

For pipelines in environmentally sensitive regions, or when a lower compressive strain is desired, ways to improve the pipeline performance including epoxy coating the pipe to reduce friction etc. can be found in ASCE Committee report (1984).

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