Protection of Pipelines and Buried Structures Using EPS Geofoam

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ABSTRACT: Expanded Polystyrene (EPS) geofoam is a lightweight, compressible material that can be used to protect buried infrastructure in areas with high to moderate seismicity. This paper summarizes recent research conducted at the University of Utah regarding the seismic design and construction of EPS geosystems to improve the seismic resiliency of pipelines and buried structures, particularly at normal fault crossings. It discusses the development and verification of an EPS cover/backfill system to protect buried pipelines and other structures from potential rupture caused by permanent ground deformation (e.g., tectonic faulting, subsidence, liquefaction, land sliding, etc. Full-scale experiments and numerical modeling show that a light-weight cover system constructed, in part, with EPS block offers significant benefits in protecting buried pipelines from the damaging effects of offset caused by permanent ground deformation. The prototype EPS cover system significantly reduced the vertical uplift force and stresses imposed on the buried pipe system as it was subjected to uplift through the EPS cover system.

INTRODUCTION

Expanded polystyrene (EPS) geofoam is a cellular geo-synthetic used in both above and below ground in many civil engineering applications. Its primary uses are: ultra-lightweight embankment and backfill, noise and vibration damping and thermal insulation of pavements (Horvath, 1995). EPS has two main advantages over traditional earthen materials when used in underground applications. The first is its very low mass density, which typically ranges from about 10 to 40 kg/m^3. This advantage can be used to reduce the vertical and horizontal loads on buried structures, utilities and retaining walls, etc. For example, the lightweight nature of geofoam was used to reduce soil pressures on a buried culvert (Kentucky DOT, 2004) and to decrease lateral earth pressure against a buried wall (Negussey and Sun, 1996). The second advantage is its relatively uniform compressibility, which can be used as a "compressible inclusion" in geosystems undergoing static, monotonic and cyclic loadings (Horvath, 2005). Upon application of external stress, the geofoam

compresses; such compression can be used to reduce static and dynamic loadings exerted against buried infrastructure. Examples of phenomena that cause external loadings that may induce such compression include: static and dynamic earth pressures (Bathurst et. al., 2007a,b), swell and frost-heave pressures, settlement of support soils, tectonic faulting, liquefaction, landslides and traffic loads (Zou et al., 2000).

In regards to the use of geofoam with buried pipelines, Yoshizaka and Sakanoue (2003) report a 33 to 60 percent reduction in the lateral soil-pipe forces when geofoam was used as lightweight trench backfill cover for pipe undergoing horizontal displacement. More, recently, Choo et al. (2007) have explored the use of geofoam as a light-weight cover system for buried steel pipelines subjected to vertical fault offset. They used centrifuge testing of scaled models to evaluate the benefits of geofoam in reducing pipeline stresses undergoing vertical offset.

Except for the study of Choo et al. (2007), relatively little research has been done to investigate the effects of vertical offset caused by normal faulting on buried steel pipelines (MCEER, 1999). In contrast, considerable work has been done for the case of rigid, steel pipelines undergoing horizontal offset from strike-slip faults, liquefaction-induced lateral spread and slope failure. For example, Newmark and Hall (1975) and Kennedy et al. (1979) developed solutions for the stresses and strains imposed on buried, steel pipeline undergoing permanent ground deformation. However, these solutions were for horizontal and axial displacement only. For normal and high angle reverse faulting, where the relative offset is mostly vertical, ASCE (1984), MCEER (1999) and ASCE-ALA (2001-2005) recommend using the Newmark and Hall (1975), Kennedy et al. (1979) and finite element (FEM) approaches for evaluating vertical movements. Others, like ASCE TCLLE (1984), MCEER (1999) and ASCE-ALA (2005) recommend using Trautmann and O'Rourke (1985) type soil-pipe springs in the evaluations.

This paper summarizes the development and testing of a light-weight EPS cover system for protecting a ductile, high-strength steel, natural gas transmission pipeline from the potential damaging effects of offset resulting from vertical. This research was done to support the design and construction of 0.6-m diameter, high-pressure natural gas transmission line that was constructed across the Salt Lake Valley, Utah by a regional natural gas utility. The alignment of the pipeline crossed the Salt Lake City segment of the Wasatch fault, which is a normal fault with an expected vertical displacement of about 2 to 3 m during a single faulting event. The thickness of pipeline cover for this project varied from about 2 to 3 m, and the pipeline was to be constructed underneath the centerline of an asphalt-paved, 4-lane highway in an urban area. For the design event, the pipeline near the fault trace on the down-dropped side (i.e., foot wall) of the fault was expected to be subjected to relative upward movement through the trench cover system as the ground moved downward relative to the adjacent segment of the pipeline that was supported on the hanging wall side of the fault. The design concept was to use the light-weight and compressible nature of EPS geofoam to evaluate and construct a cover system that would be effective in reducing the vertical, bending and shear forces induced on the pipe system from the designed offset event. Because of the relatively shallow burial of the pipeline in this area, the design concept was to place EPS block atop the pipe in

the trench excavation in an area that crossed the fault trace and the hanging wall side of the fault. The system would be constructed to create an EPS-filled, vertical "slot" within the trench that would allow relative movement during fault rupture (Fig. 1) and significantly decrease the bending stress in the pipe. Key to the design concept was the assumption that any buckling damage to the pavement from this "controlled" uplift was acceptable post-earthquake performance. In addition, it was pointed out that a segment of the pipeline near the fault trace may be partially exposed following the earthquake, but would remain undamaged from the uplift. These strategies and performance goals were discussed with the gas utility and the Utah Department of Transportation (UDOT) and were deemed acceptable. Finally, because EPS was to be placed under the roadway, the cover system design had to support live traffic loads without overstressing the EPS or underlying pipe.
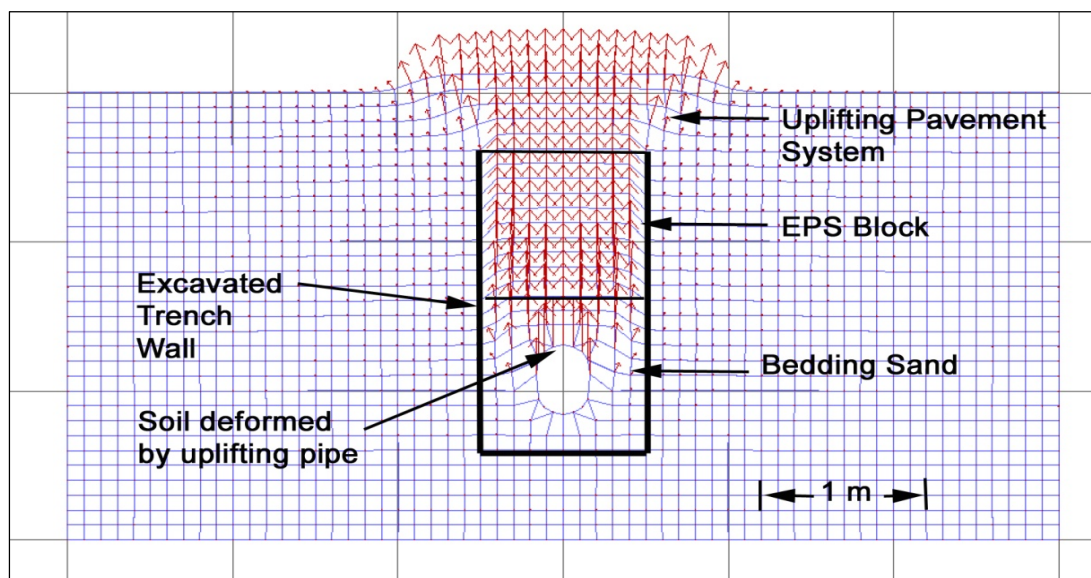

Fig. 1. Cross-sectional view of displacement vectors from modeling of EPS cover system showing the development of a controlled "slot" failure in the light-weight cover system.

EXPERIMENTAL EVALUATIONS

Because of the uniqueness of the proposed system, experimental and analytical programs were developed to support the design and construction (Lingwall, 2011). Field-scale testing was done to calibrate the numerical modeling approach and to aid in developing the required soil spring properties for modeling. The methodology and assumptions used were similar to those of Trautmann and O'Rourke (1985), expect the pipe and cover system evaluated herein consisted of larger-scale tests than those performed by these earlier researchers. The field trench tests, described below, were constructed, as reasonably as possible, to obtain plane strain conditions (i.e., 2D uniform cross-section with negligible end effects.) The uplift imposed on the pipe by the rigging of a high-capacity (i.e., 890-kN) crane was similar to that encountered by a steel pipeline exposed to vertical offset (uplift) on the down-dropped side of a fault through a trench cover system.

Two field pipe uplift tests were performed on: (1) geofoam and (2) soil covers, respectively (Lingwall, 2011). The uplift test configuration for the geofoam cover system is shown in Fig. 2. In the second test, the geofoam block and capping concrete slab were replaced with compacted backfill soil and uplift of the pipe was repeated. This latter test was conducted as a baseline so that the results of the EPS cover system could be directly compared with conventional practice of using compacted soil as trench backfill. From the comparison of the uplift force versus displacement curves, the uplift efficiency of the geofoam cover system could be estimated.

The soils encountered at the test site consisted of (from ground surface to bottom of trench, respectively): layer (1) 0.92 m of backfill soil consisting of a sandy fill that included some cobbles and a significant amount of low plasticity clay, and layer (2) medium stiff, brown, medium plastic, silty clay with sandy interbeds. This layer extended below the bottom of the trench excavations. Groundwater was not encountered in the trenches during construction and testing.

The moisture content of the unsaturated soils varied with depth and ranged from 13.4% in layer 1 to 18% at the base of layer 2. Soil properties obtained from material and laboratory testing of bulk samples are shown in Table 1. The geofoam block used in the cover for the first uplift test was EPS 29 (i.e., density of 29 kg/m^3) (Fig. 2) (Table 2). A total of four blocks were used, and the blocks were cut into two pairs with approximately the same dimensions. The first pair measured 1219 x 914 x 914 mm and the second pair measured 1067 x 914 x 914 mm. Slots to accommodate the crane rigging between the blocks were cut on-site during installation.

Table 1. Soil Properties for Uplift Tests

Material	Unit Weight	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Apparent Cohesion	Friction Angle
--------	kN/m3	%	%	%	%	kPa	deg
Road base	19.5	4.5	0	0	0	0	43
Fill (layer 1)	18.0	15.9	24.4	20.4	4	35.9	28
Clay (layer 2)	17.6	22.8	37.4	21.4	16	60	23
Interbeds (layer 2)	18.3	13.4	0	0	0	35.9	27
Bedding Sand	15.5	13.7	0	0	0	0	32

Table 2. Physical Properties for EPS 29

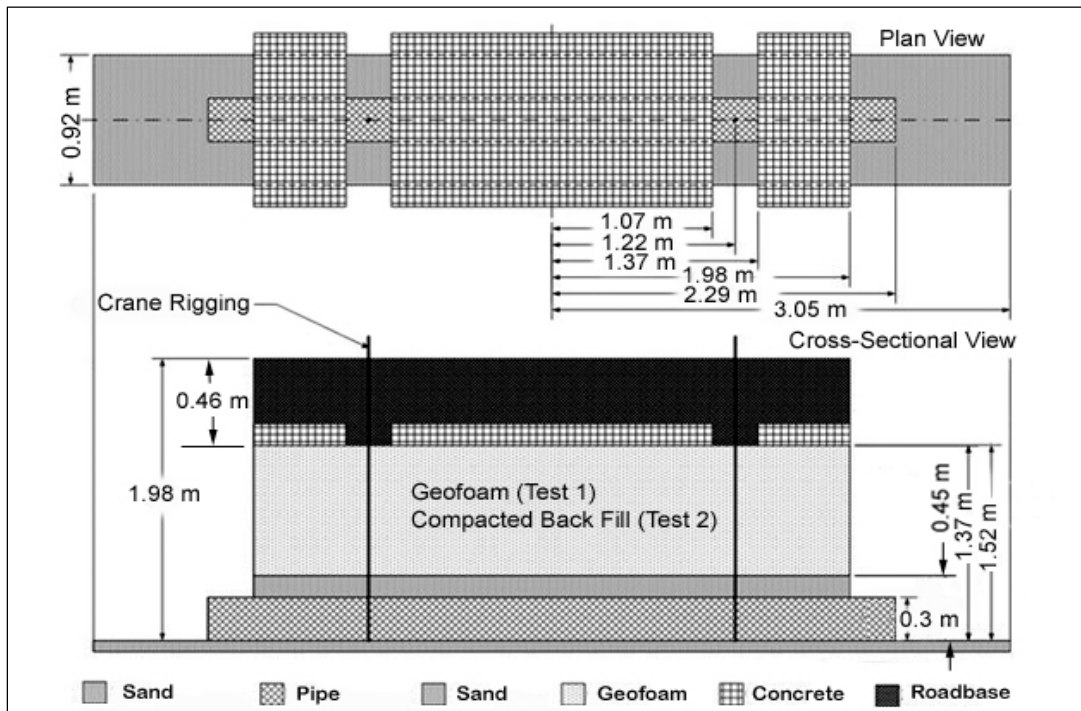|	
Density (kg/m^3)	29
Compressive Resistance at 1% axial strain (kPa)	75
Elastic Modulus (kPa)	7500
Flexural Strength (kPa)	345
Compressive Resistance at 5% axial strain (kPa)	170
Compressive Resistance at 10% axial strain (kPa)	200

Fig. 2 Configuration of uplift test for EPS geofoam light-weight cover system.

In constructing each uplift test, the respective trenches were over-excavated to a depth of 0.15 m and bedding sand was placed beneath the pipe (Fig. 2). The pipe was fitted with crane rigging and placed atop the bedding sand. Subsequently, additional lifts of bedding sand were placed along the sides of the pipe until it was covered with 0.15 m of loosely-placed sand. The pipe used for the tests was a 6.1-m length of steel grade X42 with an outer diameter of 324 mm and wall thickness of 6.5 mm. Above the pipe and bedding sand, geofoam was then placed for the first test followed by a 0.15-m thick reinforced concrete slab which represents a "load distribution slab" commonly used in EPS roadway applications. Subsequently, 0.31 m of granular road base, which consisted of an angular, well graded material with non-plastic fines, was placed and compacted atop the concrete slab to the ground surface (Fig. 3 left) For the second test, the geofoam and load distribution slab were omitted and replaced with compacted soil from the original trench excavations to depth of 0.46 m below the ground surface. Subsequently, 0.46 m of road base was compacted atop the backfilled soil.

During each test, the following measurements were made versus elapsed time using a multi-channel data-acquisition system: (1) total force required to uplift the pipe segment, (2) vertical displacement of the pipe at multiple locations, (3) vertical normal stresses that developed within the cover system above the pipe. A tension load cell was placed in-line with the crane rigging to measure the uplift resisting forces. Linear velocity displacement transducers (LVDTs) were placed at both ends of the pipe segment to measure the uplift (i.e., vertical) displacement. Total pressure earth cells were embedded in the cover system(s) above the pipe to measure the vertical stress that developed from uplift. In addition, pressure film was also used to measure

the contact stress that developed at the pipe/geofoam contact point. In addition, photography was performed for additional documentation, interpretation and analysis (Lingwall, 2011).

Fig. 3. Pre-test (left) and post-test (right) of pipe uplift test of EPS cover system.

RESULTS AND INTERPRETATION

The force, displacement, and vertical stress data gathered were reduced and interpreted. The soil backfill system had a peak force of 520 kN at a displacement of 70 mm. The geofoam cover system had a peak uplift force of 136 kN at a displacement of 188 mm. Thus, the EPS cover system reduced the peak uplift force by 520/136 = 3.8, or by approximately a factor of 4 (Fig. 4). Upon initial application of the uplift force by the crane, the geofoam cover had a relatively stiff response due (Fig. 4) to the relatively high Young's modulus of EPS 29 (Table 2). However, after about 40 kN of uplift force, the vertical stiffness dramatically reduced due to initiation of upward sliding of the EPS block as a relatively stiff elastic body.

The vertical stiffness or spring constant of the system per unit length of pipe is denoted by the symbol K_v, which is the slope of the force displacement curve shown in Fig 4. Although the behavior shown in this figure is not linear, ASCE (1984) and ALA (2005) guidelines for pipe design define K_v as the secant of the force-displacement curve taken at its peak value, F_v, which is 520 and 136 kN for the clay and geofoam cover systems, respectively:

$$K_v = F_v \,/\, \delta_v \,/\, L \qquad\qquad (1)$$

where: δ_v is the displacement corresponding to F_v and L is the length of the pipe where the interaction is occurring ($L = 4.58$ m in this case, see Fig. 2). Values of K_v are tabulated in Table 3. Thus based on this approach, the vertical stiffness of the geofoam cover system in uplift is approximately 10 percent of that measured for the

corresponding soil system. This marked reduction in stiffness is due to the smaller force required for uplift (136 vs. 520 kN, respectively), and just as importantly, the larger displacement required to develop the peak resistance (188 vs. 70 mm, respectively) achieved by the geofoam system.
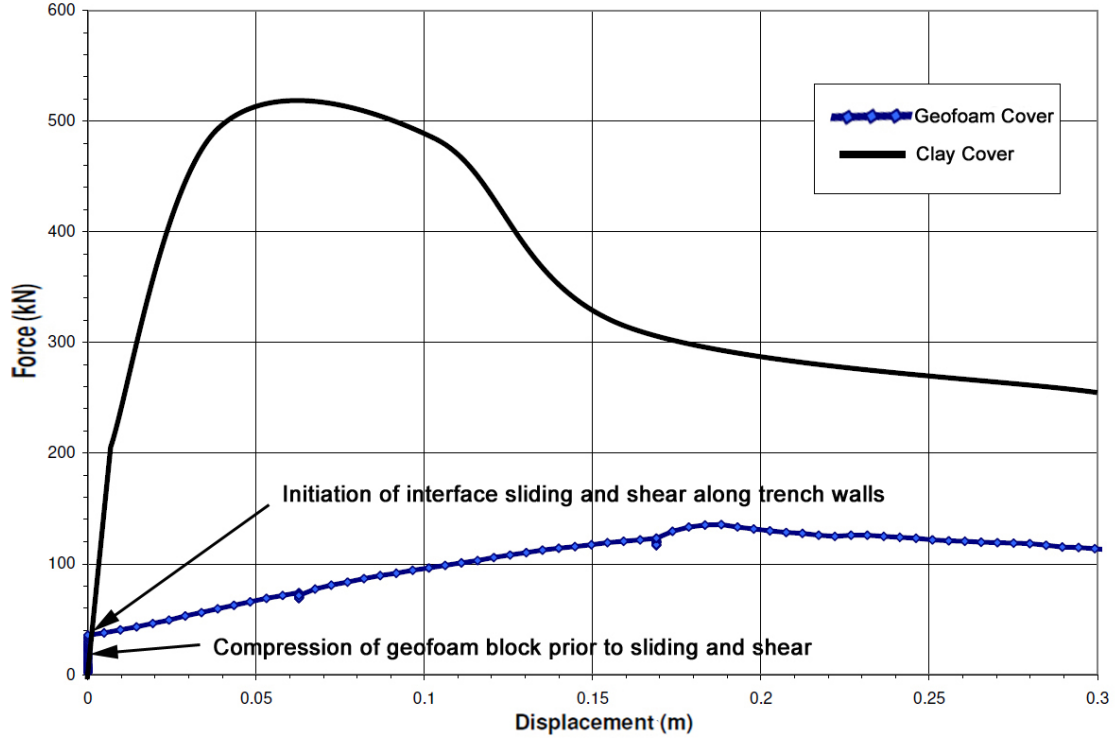

Fig. 4. Comparison of uplift force versus displacement results for geofoam and soil covers.

Another useful way to interpret the force-displacement curve is to perform a normalization described by Trautman et al. (1985) (Eqs. 2 and 3), which has been done for these tests (Fig. 5). This normalization accounts for the effects that pipe length, diameter and overburden stress have on the force-displacement behavior. Obviously, the overburden stress, or mass of the overlying cover components, is much less for the geofoam system. However, one can remove this effect by applying Eqs. 2 and 3. This allows for a better comparison of how the geofoam has affected the uplift behavior in terms of creating a "slot" failure mechanism when compared with a more general shear failure observed in the soil cover system. To perform the normalization, the peak of the normalized force curve is called the uplift factor, N_v, which is similar in concept to the bearing capacity factor, N, used for downward loading in tradition bearing capacity theory; except in this case the direction of loading is upward and the medium is not semi-infinite in that direction. In addition, the peak dimensionless displacement is called z_u:

$$N_v = \frac{F_v}{\gamma HDL} \tag{2}$$

$$z_u = \frac{\delta_v}{D} \qquad\qquad (3)$$

where: γ is the unit weight of the material in the cover, H is the height of the effective cover, D is the outside diameter of the pipe. (For systems with several layers in the cover, as was the case with these tests, the γH term in Eq. 2 can be replaced the initial vertical stress imposed by the cover system at the top of the pipe.).

Values of N_v and z_u have also been tabulated in Table 3. The lower N_v achieved by the geofoam system compared with the soil system (6.5 vs. 10.5, respectively) indicates that the "slot" failure mechanism achieved by the geofoam system is improving the efficiency of the system in uplift when compared with the more extensive shear failure developed in the soil cover system.

Table 3. Comparison of Vertical Dimensionless Factors and Stiffness Values

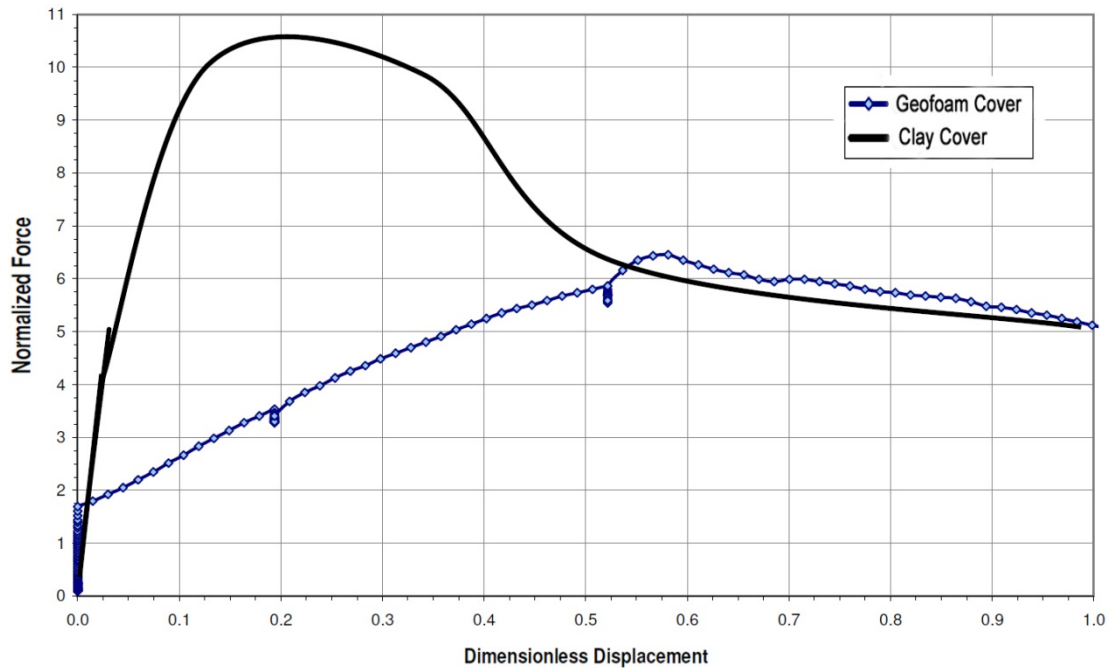|	EPS Geofoam Test	Soil Backfill Test	ASCE - ALA Recommended Values for Clay Trench Fill
K_v (kN/m/m)	158	1622	1260
N_v (dimensionless)	6.5	10.5	10
z_u (dimensionless)	0.58	0.20	0.32

Fig. 5. Comparison of uplift force versus displacement curves for geofoam and soil cover.

The field test results presented in Figs. 4 and 5 do not incorporate the construction of an asphalt pavement atop the cover. Additional uplift resistance will be encountered if pavement systems are present atop the cover system. Lingwall (2011) performed additional laboratory testing and numerical analyses to quantify this

additional resistance for both geofoam and soil cover systems. This was done to further support the project design; however, a detailed description of these evaluations is beyond the length of this paper.

CONCLUSIONS

The consequences of permanent ground displacement pose a significant threat to buried utilities and infrastructure. In the case of potential vertical offset of shallowly buried, ductile steel pipeline, this threat can be significantly reduced by the construction of a light-weight cover system using EPS geofoam placed atop the pipeline. This can be done in such a fashion so that the pipeline can undergo controlled, relative uplift across the vertical offset feature in a manner that minimizes the bending and shear stresses that develop in the pipeline.

To verify the design concept, two full-scale uplift tests were performed to quantify the uplift resistance and to support future design efforts for the construction of a natural gas transmission line over a normal fault. One of the field tests used EPS geofoam block as lightweight backfill in the cover system to explore its potential use as a countermeasure on the foot wall (i.e., down dropped side) of a normal fault. The second field test was a conventionally constructed cover that used soil backfill. The results from the second test provided a baseline comparison of the two cover systems. This comparison showed that the geofoam cover system developed about four times less uplift force than that of the corresponding soil backfill. In addition, the force-displacement behavior of the geofoam system was more desirable; it showed that about 2.75 times more displacement was required to mobilize the peak uplift resistance. When the uplift stiffness, or vertical spring constant, was calculated for these systems, the geofoam exhibited a value that was about 10 times less stiff. Both of these factors contributed to the significantly better performance of the geofoam system and it was implemented in the construction of the aforementioned and subsequent projects by the regional gas line utility at normal fault crossings.

ACKNOWLEDGMENTS

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