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INNOVATIVE RETROFIT OF THREE LARGE DIAMETER LANDSLIDE CROSSING PIPELINES USING EARTHQUAKE RESISTANT PIPE

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Abstract

The evaluation, analysis and retrofit of three seismically vulnerable large diameter pipelines are presented. The pipelines are located within a landslide with the potential for very large sliding deformations that can cause failure of the existing pipelines, and result in substantial direct and indirect damages, including direct damage losses, loss of service, and potential flooding of the surrounding area.

The existing pipelines, which range from 1.5 meter to 1.8 meter in diameter, are made from prestressed concrete cylinder pipe (PCCP) which is highly vulnerable to failure when subjected to ground deformation. In order to mitigate the seismic risk of failure, multiple conceptual alternatives were studied that included a range of very different strategies, including consideration of different alignments, and special pipeline materials, joints and construction techniques.

The preliminary evaluation and analysis of the conceptual alternatives led to the selection of the “compression pipe” alternative which incorporates flexible joints that allow joint deformation in multiple directions. During the final design phase of the project, detailed 3D nonlinear soil-pipe and pipe-soil-pipe interaction analyses of the selected alternative were performed using the ABAQUS finite element software in order to verify the adequacy of the proposed solution. The analysis model was also used to refine the design and optimize the pipeline’s seismic performance.

Keywords: pipeline, infrastructure, seismic, landslide, soil-structure interaction

1. Introduction

The Santa Clara Valley Water District (District) initiated the Penitencia Delivery Main and Penitencia Force Main Seismic Retrofit Project (Project) to improve the seismic resilience of three critical water supply/delivery pipelines that serve the District's Penitencia Water Treatment Plant (WTP) in San Jose, California. The most important objective of the project was to retrofit the existing pipelines to protect the life safety of nearby residents and the Noble Elementary school.

The 66-inch Penitencia Force Main (PFM), 60-inch Penitencia Delivery Main (PDM), and 72-inch South Bay Aqueduct (SBA) cross from a stable geologic zone onto the slow-moving Penitencia Creek Landslide (Landslide) near the Penitencia WTP. The Landslide is actively creeping and is susceptible to large seismic deformations. The Project will replace approximately 850 feet of the existing SBA, PDM, and PFM pipelines with Earthquake Resistant Ductile Iron Pipe (ERDIP) at the Landslide toe (Fig. 1). ERDIP has been used in Japan for critical infrastructure for the past 50 years and has a proven track record of no failures through several large earthquakes.

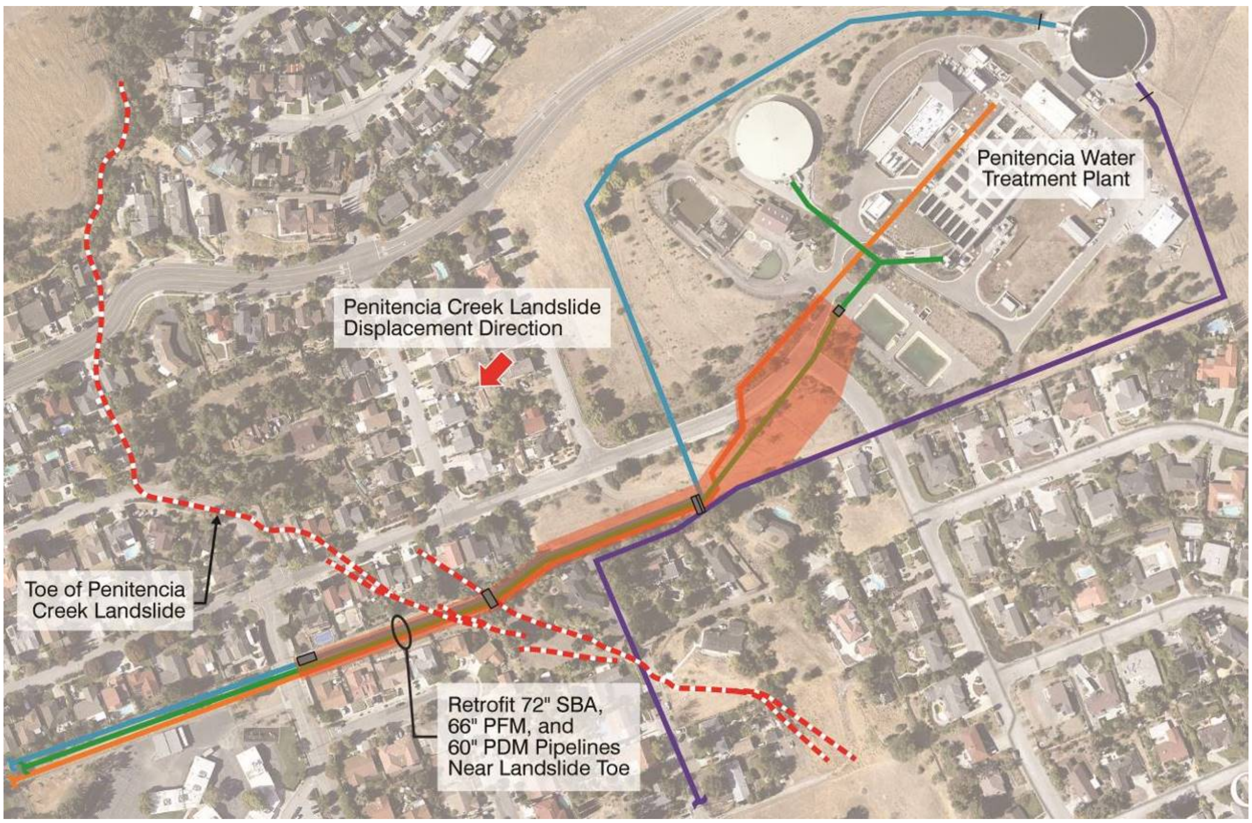

Fig. 1 – The Landslide toe and 72-, 66-, and 60-inch pipelines intersect in a densely populated area. The most important project objective is to retrofit the existing pipelines to protect the life-safety of nearby residents and the Noble Elementary School.

2. Design for Large Landslide Displacement

The project team evaluated the landslide and seismic hazards (Lettis and CEG, 2014) to establish the magnitude of the future landslide displacements in the Project area, the direction of landslide movement, and to define the location of the landslide toe. The seismic hazards evaluation was based on a 1,000-year earthquake event on the nearby Hayward and Calaveras faults and other seismic sources at greater distances. The key findings from the evaluation for landslide displacement were:

- The creep-related movement at the toe is estimated to be approximately 20 inches, and the combined maximum displacement (creep and seismic) is estimated to be 9.4 feet. The 9.4 feet of deformation is likely to occur over a 60- to 65-foot wide zone of primary deformation, with a primary/secondary zone of deformation extending 45 feet farther west and 90 to 95 feet farther east. The deformation zones and other key results of the evaluation are shown in Fig. 2.
- The azimuth of the Landslide slip direction is 220 degrees ± 20 degrees relative to true north (Lettis and CEG, 2014).
- The dip of the basal shear plane is moderately defined, ranging between 0 and 30 degrees with dip direction to the northeast, which can cause uplift of the pipelines and vault structures. The basal landslide shear plane is the interface between the moving landslide mass and the stable valley floor. The dip angle is the inclination of the primary landslide shear plane measured from a horizontal plane and perpendicular to the strike of the Landslide toe.

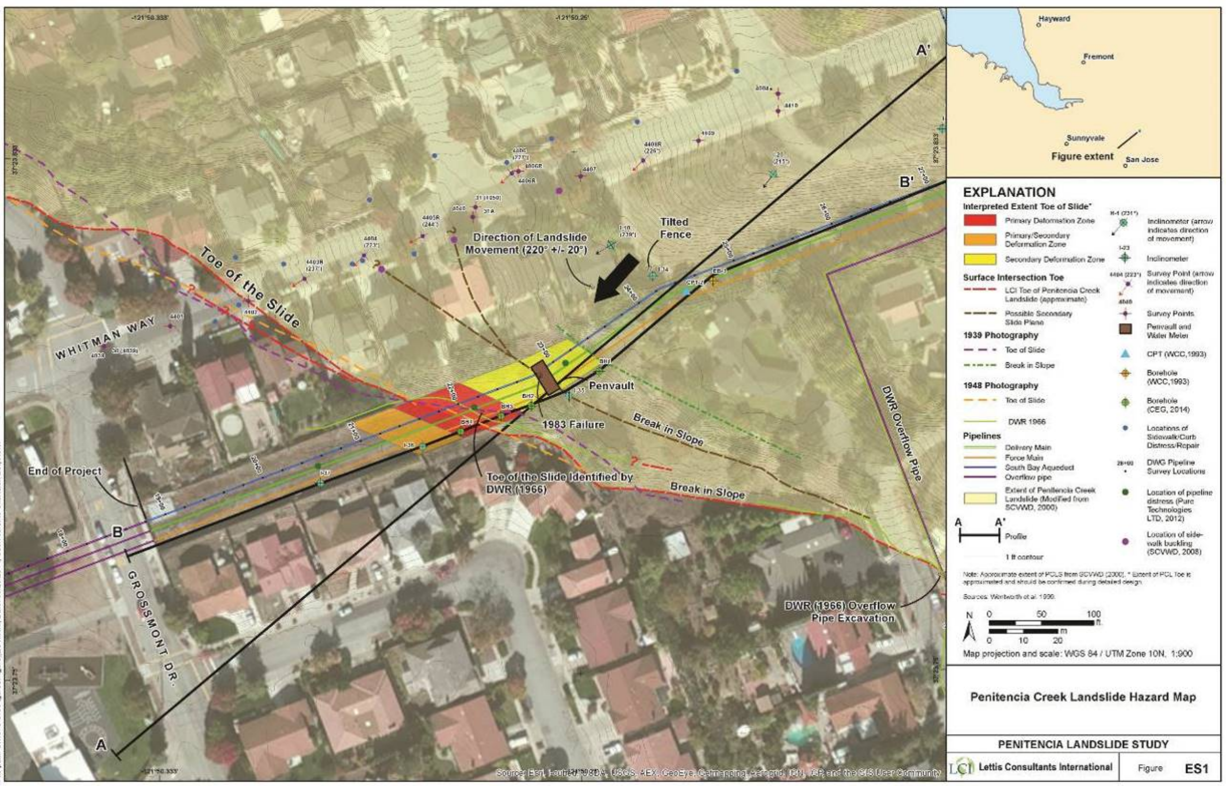

Fig. 2 – Penitencia Creek Landslide Hazard Map.

Based on the dip of the landslide toe, the slip direction, and the pipeline intersection angles (40 to 50 degrees), the PDM, PFM, and SBA pipelines are expected to experience a combination of compression and right-lateral (southeast-directed) shear, as well as possible uplift and tilting within the deformation zones as a result of landslide displacement.

3. Conceptual Alternative Evaluation

In the planning phase of the project, six conceptual alternatives were developed as potential solutions to the problems and deficiencies identified during the problem definition:

- Alternative 1—Compression Pipe (ERDIP)
- Alternative 2—Expansion Loop

- Alternative 3—Displacement Vault
- Alternative 4—Pipeline Realignment
- Alternative 5—Rubber or Metallic Bellows
- Alternative 6—No Capital Improvement

The six alternatives were evaluated and scored on nine different criteria. The alternatives were then ranked from 1 to 6 based on a pairwise evaluation methodology. The two top ranking alternatives, Alternative 1 and Alternative 2, were then analyzed in more detail for their seismic performance, and Alternative 1 (Compression Pipe) was selected as the best alternative overall.

4. Full Scale Joint Testing

The ERDIP manufacturer (Kubota Corporation) performed a full-scale test of the 60-inch diameter pipeline at their test facility in Japan in August of 2014. The purpose of the test was to verify the maximum deflection angle, maximum moment, and develop the joint model spring properties (i.e. the relationship between deflection angle and moment applied to the joint) for the 60-inch pipeline. The test was designed to stress the pipe to failure; understanding when and how the compression pipe would fail was considered a key outcome of this test.

The test results showed the 60-inch pipeline exceeded the published limits for the maximum deflection angle and maximum moment (i.e. the pipe performed better than the published limits). The "failure" of the 60-inch pipe was initiated by the failure of the plug weld between the pipe spigot and the welded metal spigot restraint ring that is intended to prevent joint pullout.

Fig. 3 – Full-scale tests of the joint performance of: (a) 60-inch diameter ERDIP performed in 2014 (left), (b) 72-inch S Type collar joint performed in September 2015 (right).

The weld failed gradually along the circumference, starting at the bottom of the joint, which was subjected to tension. This resulted in a shear failure within the restraint ring that progressed around the pipe circumference. As loading continued beyond the rotation limit of the joint, the pipe spigot at the top side of the joint (under compression) moved into the barrel of the adjacent pipe resulting in increased flexibility and rotation capacity. Overall, the joint performed well up to and beyond its rotation capacity, maintaining residual restraint capacity after initial restraint ring failure. The joint did not fail catastrophically. Fig. 3a shows the 60-inch pipeline test and Fig. 4 shows the results of the test.

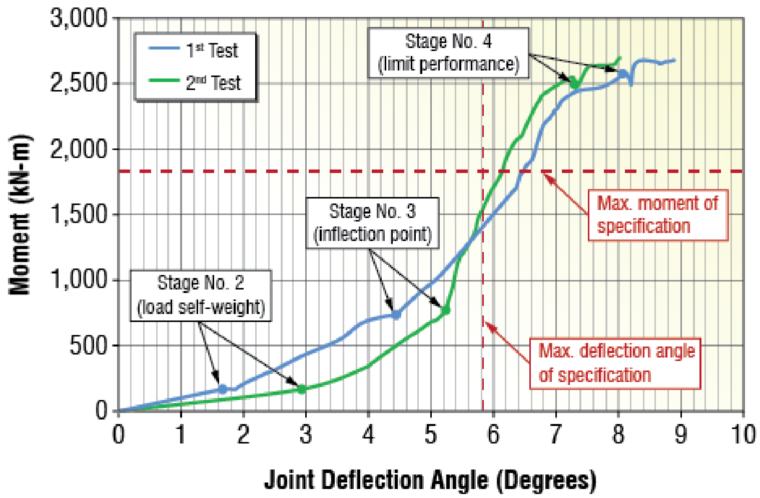

Fig. 4 – The results from two full-scale tests of the 60-inch diameter ERDIP are shown above. The pipe joint failed at least 25% more than the design deflection angle and 35% more than the design movement.

The ERDIP manufacturer (Kubota Corporation) also performed a full-scale test of the 72-inch S-type collar joint at their test facility in Japan in September of 2015 (Fig. 3b). Similar to the pipeline test, the purpose of the collar test was to verify the maximum deflection angle, maximum moment, and develop the joint model spring properties (i.e. the relationship between deflection angle and moment applied to the joint) for the 72-inch collar. The results of the collar test results were favorable and showed the following:

1. The 72-inch S-type collar joint was able to withstand the full-capacity of the testing facility (3716 kN-m), which is 53% bigger than maximum moment (2,428kN-m) and 12% bigger than limit moment (3,300kN-m) recommended for design.
2. The collar joints can deflect about 6.65° [which is about 40% higher than the maximum recommended design angle (4.67°), and about 20% higher than the limit angle (5.54°) when subjected to the limit moment (3,300kN-m).
3. According to the result of rubber gasket compression rate, the joint is not expected to leak during or after the joint is subjected to the limit moment (3,300kN-m).

3. Finite Element Modeling

3.1 2D Finite Element Soil-Spring Model Calibration

Because the three pipes at the Penitencia site have the same alignment and cross the landslide toe at about the same angle, only one pipe was modeled in the three-dimensional analysis. While no contact is expected between the pipes under the loading considered in the study, the vicinity of the pipes is expected to affect the behavior – stiffness and strength – of the surrounding soil. A fully-coupled three-dimensional continuous model that would capture this behavior along with the landslide deformations, however, was not computationally feasible, and a simplified model with line elements and discrete springs was used in the analysis.

The strength and stiffness characteristics of the soil springs in the model were calculated using the equations found in the American Lifelines Alliance Guidelines for the Design of Buried Steel Pipe (ALA, 2001). The ALA document, however, limits the applicability of its equations to buried pipelines in uniform soil conditions. Therefore, a two-dimensional plane-strain analysis, was performed to determine the necessary modification to the ALA equations to account for the presence of the other pipes in the vicinity. Please note that the plane-strain model is able to capture the horizontal and vertical response of the soil, but does not capture the response in the longitudinal direction of the pipe.

The first objective was to develop a 2D Plane-strain model in Abaqus with a single embedded pipe, subject the pipe to vertical or horizontal loads and compare the soil behavior (stiffness and strength) to that predicted by the ALA equations. The second objective was to model three pipes in the same soil medium, subjected them to the same lateral displacements, and compare the soil resistance to three times that of a single pipe, as would be used if the soil behavior was independent of the nearby pipes. An efficiency factor for each loading direction was thus determined, as shown in Table 1. These factors were then applied to the strength values of the soil springs in the 3D model.

Table 1 – Pipeline Efficiency Factors

Loading Direction	**Efficiency Factor**
Lateral (symmetric)	0.65
Vertical Uplift	0.83
Vertical Bearing	1.00

The finite-element model consists of three main components: the pipe, the soil, and the loading. The domain for each configuration is shown in Fig. 5. The difference between the two configurations is only geometric, all material properties are the same. The pipe was modeled using a homogeneous material with a circular section with an outer diameter of 6 feet (72-inch) and a thickness of 1 inch. Even though an elastic steel material was used (E=2900ksi), all the nodes in its mesh were constrained move together, allowing no deformation in the pipe, as this was not a variable of interest in the study. The pipe was considered, essentially, rigid.

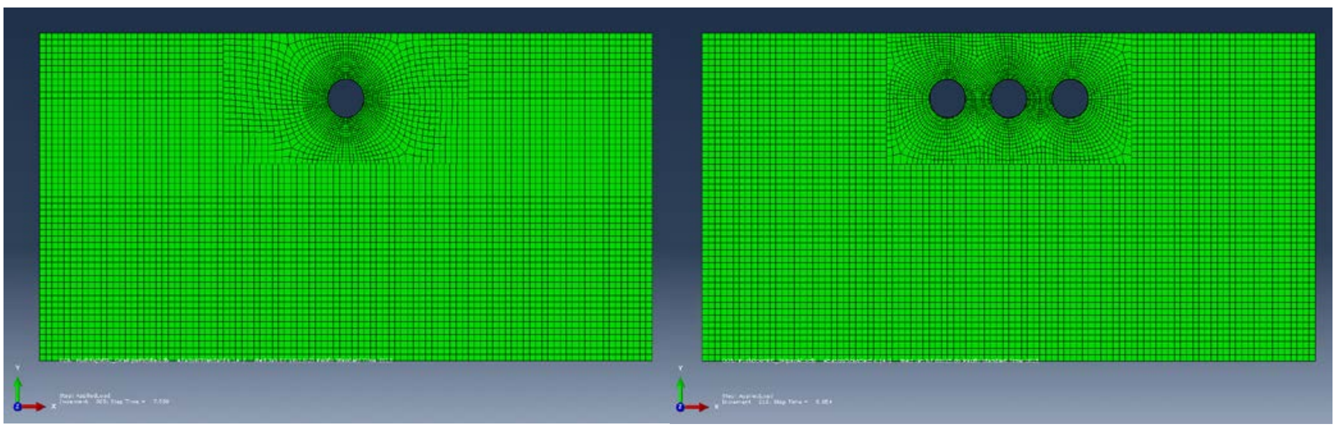

Fig. 5 – 2D-Analysis Finite-Element Mesh -- 1 Pipe & 3 Pipes

The soil material properties were defined using the mechanical properties prescribed by the geotechnical engineer. Two different soil models were evaluated -- one using a Mohr-Coulomb (MC) material and the other using a Drucker-Prager (DP) material. While the MC input parameters were taken equal to those listed above. The dilation angle was selected to improve convergence. The DP input parameters were calculated to yield the same results as an MC material in a plane-strain analysis.

The boundary conditions on the model consisted of pinned constraints along the horizontal bottom boundary of the soil and vertical rollers along the vertical boundaries. Both pipe configurations were subjected to three different lateral displacement-controlled loading conditions, after the geostatic gravity load:

- Horizontal Lateral (symmetric left or right)
- Vertical Downward (Bearing)
- Vertical Upward (Uplift)

The lateral load was applied as imposed displacements to the pipe nodes. For the case of three pipes, the pipes were constrained to displace simultaneously, in phase – this is the type of loading expected during a landslide, unlike the inertial response due to earthquakes. Fig. 6 shows the model response at the largest deformation level. In each case, the pipes were translated to a maximum displacement of 60 inches, or soil failure, whichever happened first. At each displacement increment, the total force required to impose the displacement was recorded. The resulting force-deformation response was compared to that provided by the ALA equations.

The three-pipe efficiency-modification factor for each loading condition was determined in two steps. In the first step, a scale factor was estimated such that amplifying the one-pipe response by this factor would yield a response similar to that of the three-pipe case. If the soil behavior was independent of the neighboring pipes, this scale factor would equal to three. Consequently, the efficiency factor is computed as the scale factor divided by three. These efficiency factors are shown in Table 1.

The study showed that the soil resistance in the lateral and uplift directions are reduced by the presence of the nearby pipes, while the bearing resistance is not. This is mainly because, by definition, the bearing resistance bears against the infinite medium.

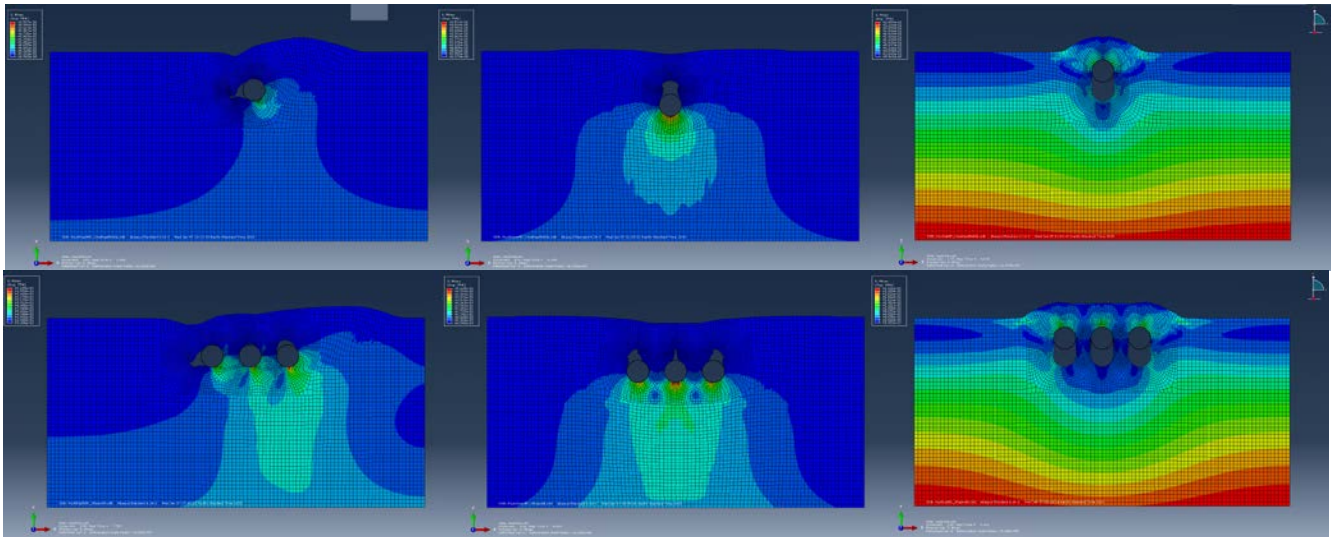

Fig. 6 – Deformed Configuration of 2D Plane Strain Analysis

3.2 3D Finite Element Landslide-Soil-Pipe Interaction Modeling

The ERDIP dissipates the large Landslide deformations by absorbing the movement through axial expansion/compression and rotation at each pipe joint. This project used a combination of ERDIP with non-standard pipe lengths and collar fittings in the primary deformation zone. Fig. 7 shows details of the ERDIP joint and collar joint. The collar joints allow substantially more axial deformation and twice the rotation deformation as a regular ERDIP compression pipe joint.

The project team created a model of the soil-pipe interaction and pipe joint behavior using Abaqus software. The model was used to design the pipeline system for the large deformation at the Landslide toe. The Abaqus model helped the project team investigate the following:

- The large deformations (geometric nonlinearity) of the soil and pipe.
- The soil-pipe and pipe-soil-pipe interactions.
- ERDIP joint biaxial interactions.
- Interactions between the SBA, PDM, and PFM pipelines.

The following three landslide modeling scenarios were used to describe the style of deformation at the Landslide toe and capture the range of possible deformations at the toe (Lettis, 2014):

- Scenario 1 – 30-degree dipping shear plane accommodating the total displacement.
- Scenario 2 – 2-degree dipping shear plane accommodating the total displacement.
- Scenario 3 – 50 percent displacement on both the 2-degree and 30-degree dipping toes.

Scenarios 1 and 2 assume a knife-edge failure mechanism at the toe, representing "worst case" scenarios, while Scenario 3, which is defined as the average of the first two scenarios, reflects distributed deformation at the toe which is judged to be more likely, but generally causes smaller demands compared to Scenarios 1 and 2. For each of the above scenarios, three different azimuth orientations of movement were checked (200, 210, and 240 degrees), resulting in a total of nine different Landslide cases. The 210° orientation was selected because the uplift is maximized at this azimuth, while the 200° and 240° azimuths maximize the horizontal oblique component of slip.

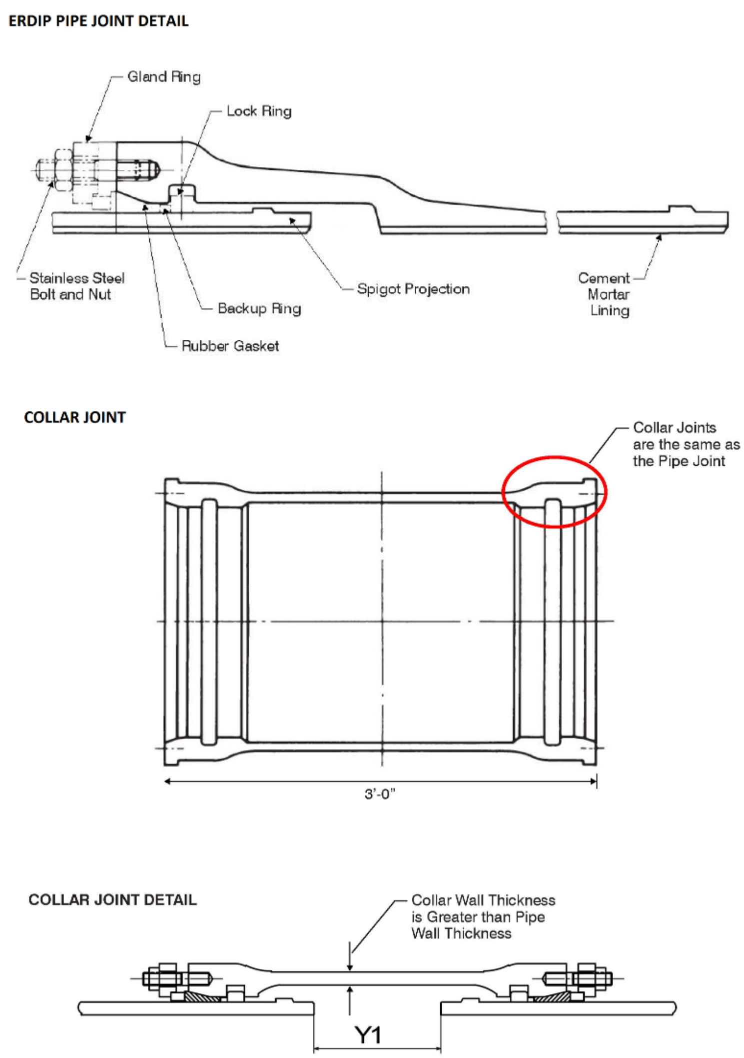

Fig. 7 – ERDIP pipe and collar joint cross-section.

The Abaqus model considered the nonlinear behavior of the ERDIP joints and collars using recommended modeling properties from the ERDIP manufacturer and properties established during the full scale testing. This included modeling the behavior of each joint in six degrees-of-freedom (or orientations) including axial, bending, shear, and torsion actions. The model directly or indirectly simulated complex joint behavior such as sliding and locking behavior, biaxial interaction, and axial-bending interaction especially for collar joints. The soil behavior was modeled using nonlinear (yielding) soil springs connecting the pipe elements to the ground. The strength and stiffness characteristics of the soil springs were set based on soil parameters obtained from soil testing (CE&G 2014) and following the American Lifelines Alliance Guidelines for the Design of Buried Steel Pipe (ALA, 2001).

Based on numerous analytical studies, and incorporating the most up-to-date soil properties (CalGeo, 2015), two design options began to emerge as the most viable solution. For each for the two options, variations of the joint spacings were considered leading to sub-options 1A, 1B, 2A, 2B.

- **Option 1 - Joint Only Design:** This option consisted of regular ERDIP joints with pipe joints spaced at various distances, depending on the level of rotation demand. This alternative used very short spacing in the high-deformation zone and pipe joints spaced a longer distances elsewhere. The project team considered several alternatives with different joint spacing for this alternative.
- **Option 2 - Joint and Collar Design:** This option is generally similar to Option 1 with the exception of using collar joints in areas of very high demand. The primary advantage of this option is the collars provided increased performance, in the form of increased axial and rotational deformation capacities, and reduced the need for very short pipe lengths.

Fig. 8, Fig. 9, and Fig. 10 show illustrations of Options 1A and 2B. It should be noted there were other options included in the analysis and Table 2 summarizes the modeling results for all of the options considered. The modeling considered five different trench backfill materials, including: native soil material, an idealized "soft soil", and three hybrid mixtures of a lightweight shale aggregate and native soil (100% lightweight, 75% lightweight, and 55% lightweight). The table compares the demand to capacity ratio (DCR) values of each alternative and trench backfill. The design options with a DCR greater than 100% have demands that exceed system capacity and these alternatives are not recommended. The alternatives with a DCR less than 100% are favorable because the demand demands are less than the system capacity.

Table 2 – Modeling Results

Design Option	100% LW	75% LW	55% LW	Native	Soft Soil
Option 1A: Joints @ 8.3'	NA	NA	NA	107%	95%
Option 1B: Joints @ 6.5'	NA	NA	NA	88%	77%
Option 2A: 5-Collar	90%	93%	99%	103%	93%
Option 2B: 9-Collar	78%	79%	81%	84%	87%

Notes:
1. LW = Lightweight Material
2. Values larger than 100% indicate overstress. Smaller values indicate favorable performance.

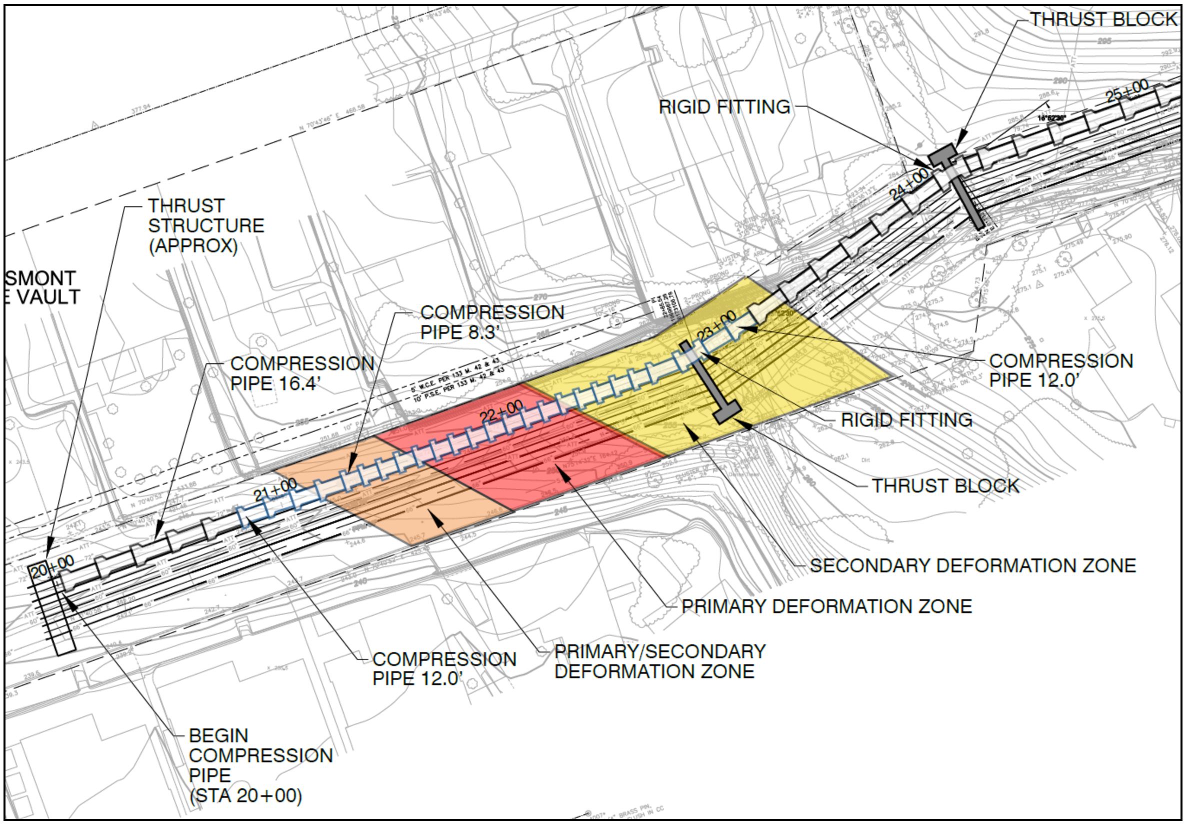

Fig. 8 – Illustration of Option 1A "Joint Only Design" consisting of ERDIP joints spaced at 8.33 feet

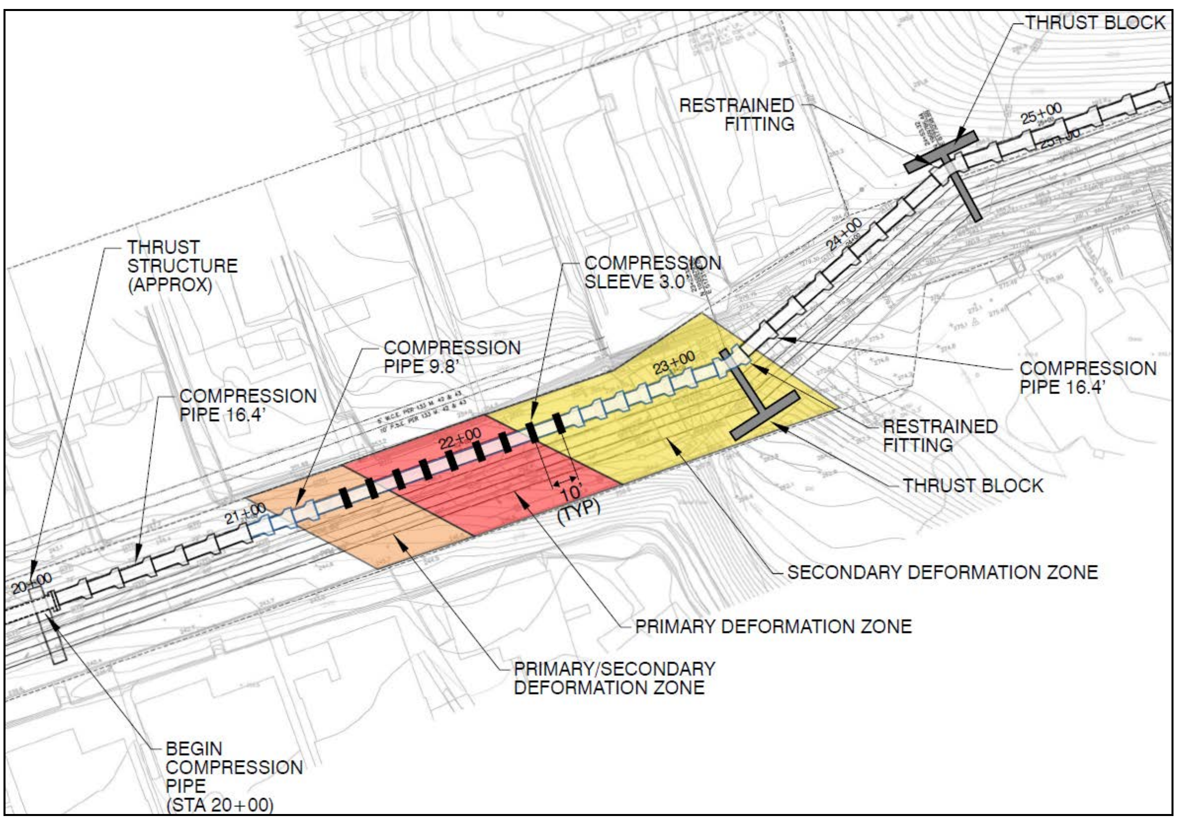

Fig. 9 – Illustration of Option 2B "Joint and Collar Design" consisting of consecutive collar joints spaced at approximately 10 feet on center.

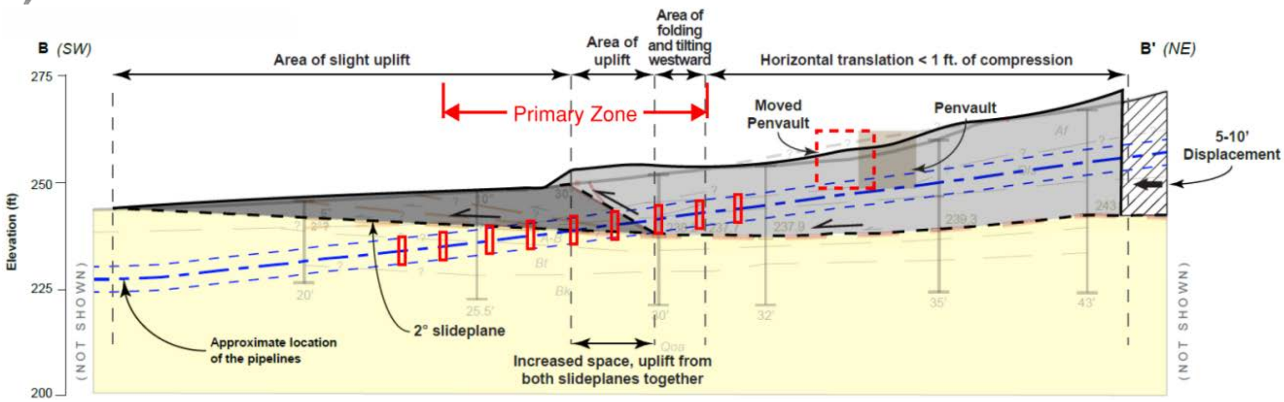

Fig. 10 – Collar joint locations under Option 2B – Section View

The project team recommended Option 2B-N (nine-collar option with native backfill) as the preferred option because it provided the best performance and was less sensitive to backfill soil properties than other options. The other alternatives offered slightly reduced performance and the following drawbacks:

1. Option 1B-N (6.5-foot joint spacing with native backfill) offered good performance; however, it required very short pipe segments.
2. Option 2A-S (five-collar option with special backfill) was the next preferred option.
3. Option 1A-S (8.33-foot joint spacing with special backfill) appeared to be the least desirable option, though it provided acceptable performance.

The final Project drawings reflected the 2B-N design option, with a slight modification which increase the number of collars from 9 to 10. The final analysis model results and deflected shape are shown Fig. 11 in for the most severe load case. Fig. 12 shows the maximum axial and rotation deformations in the ERDIP and collar joints.

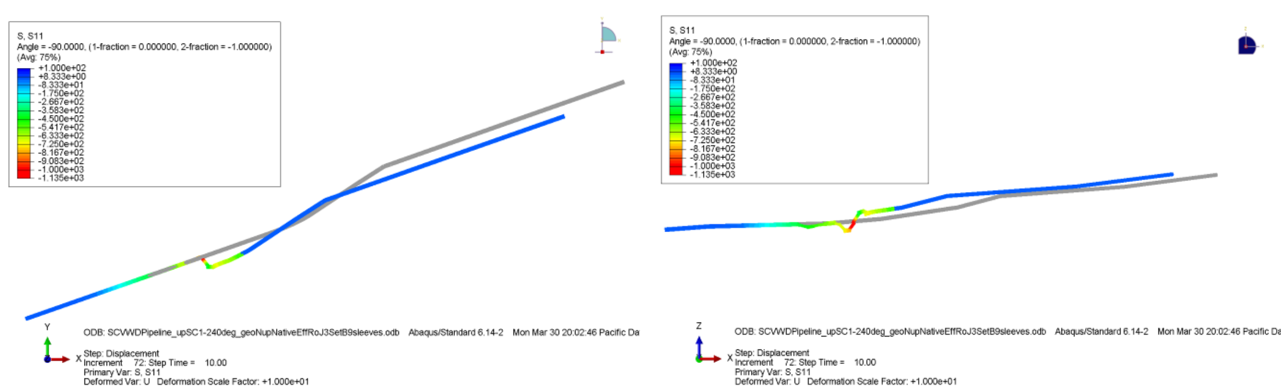

Fig. 11 – Option 2B-N Deformed Shape under Scenario 1 – 240º, Scale Factor = 10
Plan View (left) and Section View (right)

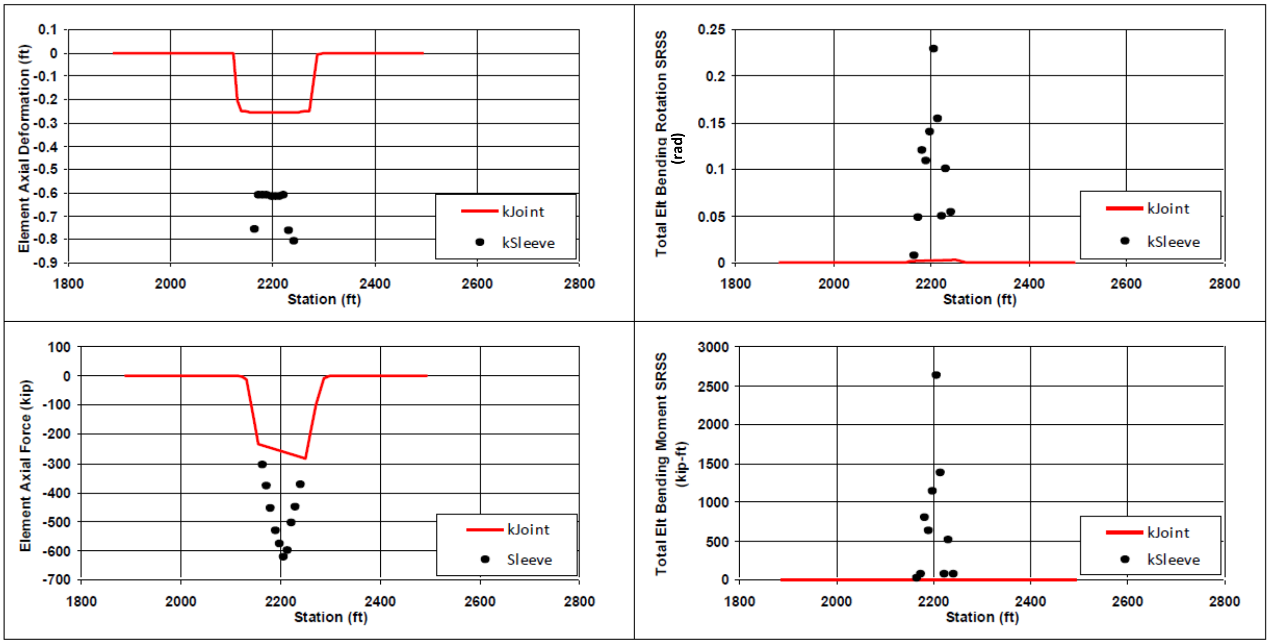

Fig. 12 – Option 2B-N Joint Biaxial Deformations and Forces under Scenario 1 – 240º
kJoint=Regular ERDIP Joint, kSleeve=Collar ERDIP Joint

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