Prepared for

Sonoma County Water Agency
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Santa Rosa, California, 95406

DRAFT

Santa Rosa Aqueduct
Rodgers Creek Fault Crossing

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MMW595
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Sonoma County Water Agency
Santa Rosa, California

This report was prepared under the
supervision and direction of the undersigned.

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1. INTRODUCTION

MMI Engineering (MMI) is pleased to submit this draft report to Sonoma County Water Agency (SCWA or Agency). This work was performed under contract number TW10/11-090, dated May 24, 2011.

The purpose of this project is to support the Agency in performing seismic design of the 36-inch Santa Rosa aqueduct to reliably transmit water across the Rodgers Creek fault. The aqueduct crosses the fault along its alignment on Sonoma Avenue between Doyle Park Drive and Alderbrook Lane in the city of Santa Rosa. The project vicinity map is shown in Figure 1.

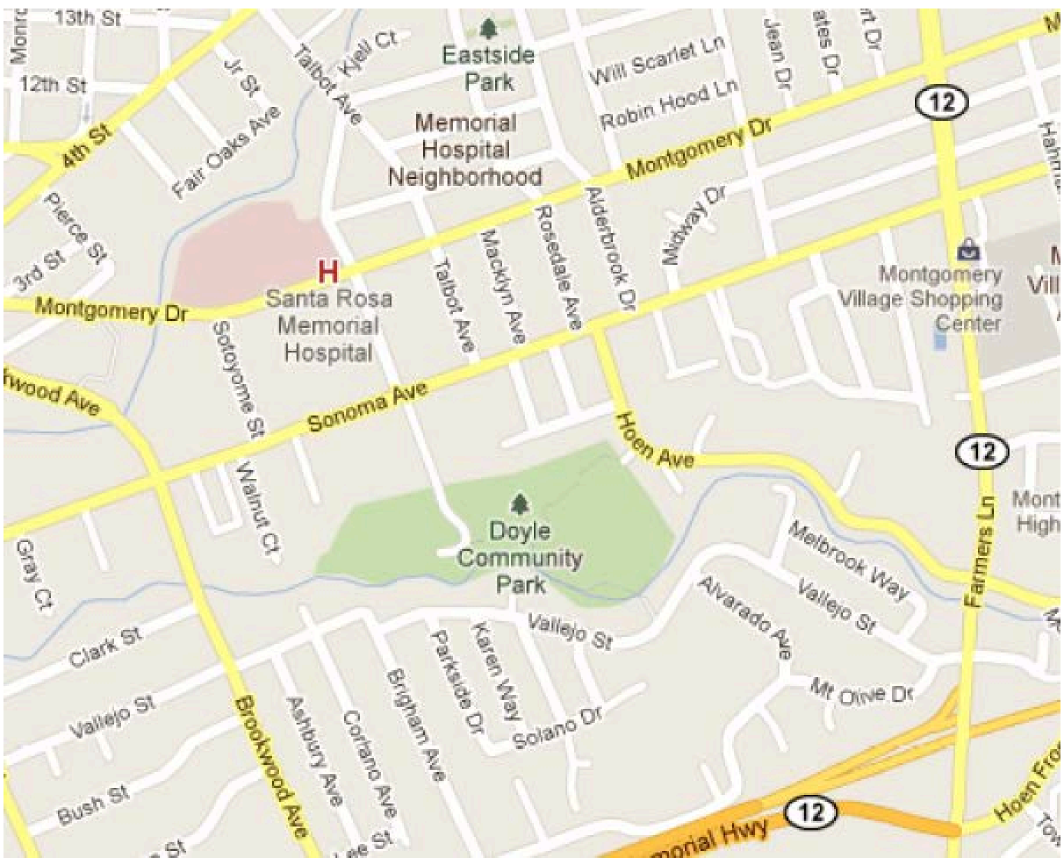


Figure 1: Project Vicinity Map (Ref. Google Maps)

2. TECHNICAL BACKGROUND

2.1 Response of Buried Pipelines to Earthquakes

Pipeline damage has been widely observed in most past earthquakes (O'Rourke and Liu, 1999) with consequences such as the inability to fight fire following the 1906 San Francisco Earthquake, restricted ability to fight fire in the Marina District during the 1989 Loma Prieta Earthquake and uncontrolled release of water in the 1994 Northridge Earthquake.

Damage to buried pipelines is a function of both the imposed ground deformation as well as the type of pipeline construction. Large permanent ground deformation (PGD) such as ground failure caused by fault rupture, landslides, liquefaction, lateral spread and seismic settlement are the primary causes of damage to buried pipelines. Transient strain and ground curvature resulting from seismic wave propagation can also impact buried pipelines but to a significantly lesser degree.

The following sections provide a brief description of pipeline response to earthquakes.

2.2 Pipeline Response to Fault Rupture

Faults are discontinuities in the bedrock that present a plane of weakness across which earthquake rupture occurs. Buried pipelines that cross active faults are subjected to abrupt deformation in a narrow zone during fault rupture that extends to the ground surface.

Faults are classified as either strike-slip, reverse or normal faults, depending upon the direction of movement of one side of the fault relative to the other. The Rodgers Creek Fault, which crosses the Santa Rosa aqueduct, is a right-lateral strike-slip fault. For this fault, the predominant movement is strike-slip with the west side moving northwards. Figure 2 shows an example of strike-slip movement on the San Andreas Fault during the 1906 earthquake. During this earthquake one side of the fault moved approximately ten feet relative to the other side (Graymer et al., 2006) as shown in the figure.

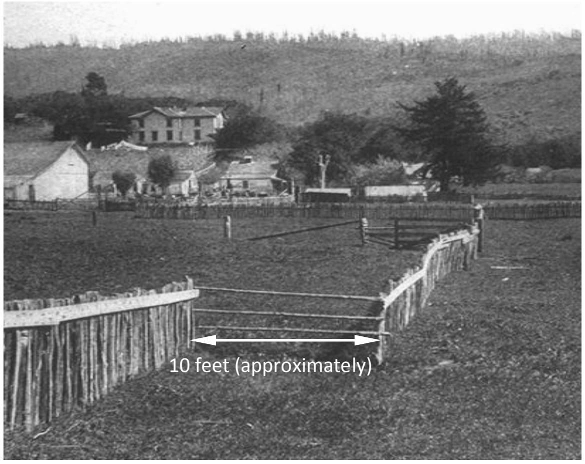


Figure 2: Strike Slip Fault Displacement during the 1906 Earthquake (NOAA)

Response of pipelines to the fault displacement is dependent on two main factors; the type of joints in the pipeline and the angle of crossing. Depending on the type of joints, pipelines can be classified as either segmented or continuous.

Segmented pipelines have joint capacity which is significantly less than the pipeline strength. Such pipelines behave as discrete sections of pipe connected together across weak joints (Figure 3). Generally speaking, pipelines with bell and spigot connections are classified as segmented pipelines. Such connections have limited expansion and contraction capacity and less than five degree rotational capacity. Large ground displacements are accommodated through concentrated deformations across joints, which due to their limited capacity fail by pull-out at the joint, crushing of bell or excessive rotation at the joint.

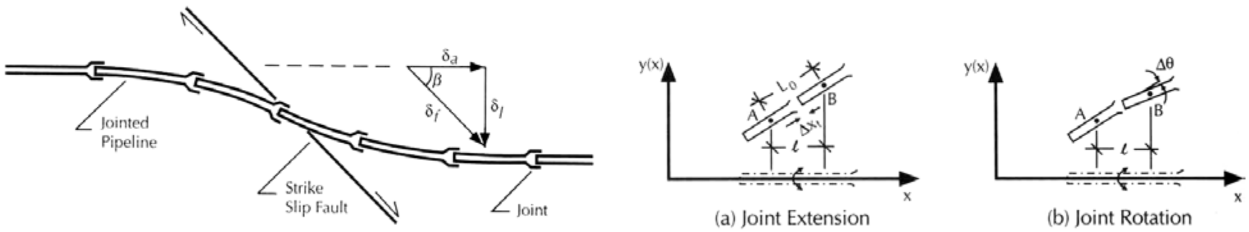


Figure 3: Response of Segmented Pipeline to Fault Rupture (O'Rourke and Liu, 1999)

Continuous pipelines have a joint capacity that is equal to or more than the pipeline strength. Such pipelines can accommodate imposed displacements by uniformly distributed deformation across the length of the pipeline. Steel pipelines with welded joints fall in this category and generally have higher ductility. These types of pipelines can resist large ground displacement through bending or axial tension and can accommodate significant ground deformation without catastrophic failure. Damage mechanisms for continuous pipelines include tensile failure, buckling of a large length of the pipeline, local buckling or wrinkling and failure at the welded joints.

The angle of crossing is the other key parameter that influences the pipeline response to fault rupture displacement. For continuous pipelines, depending upon the angle of crossing, the pipeline could be subjected to a combination of tension, bending or compression as shown in Figure 4. For pipelines constructed with ductile materials, if the crossing angle is such that the fault movement causes the pipeline to resist the imposed deformation in tension and if the joints have sufficient strength to accommodate the tensile loads the pipeline can withstand the fault displacement without rupture, albeit with severe distortion due to plastic deformation. Angle of crossing that places the pipeline in compression typically results in buckling, which is generally unacceptable.

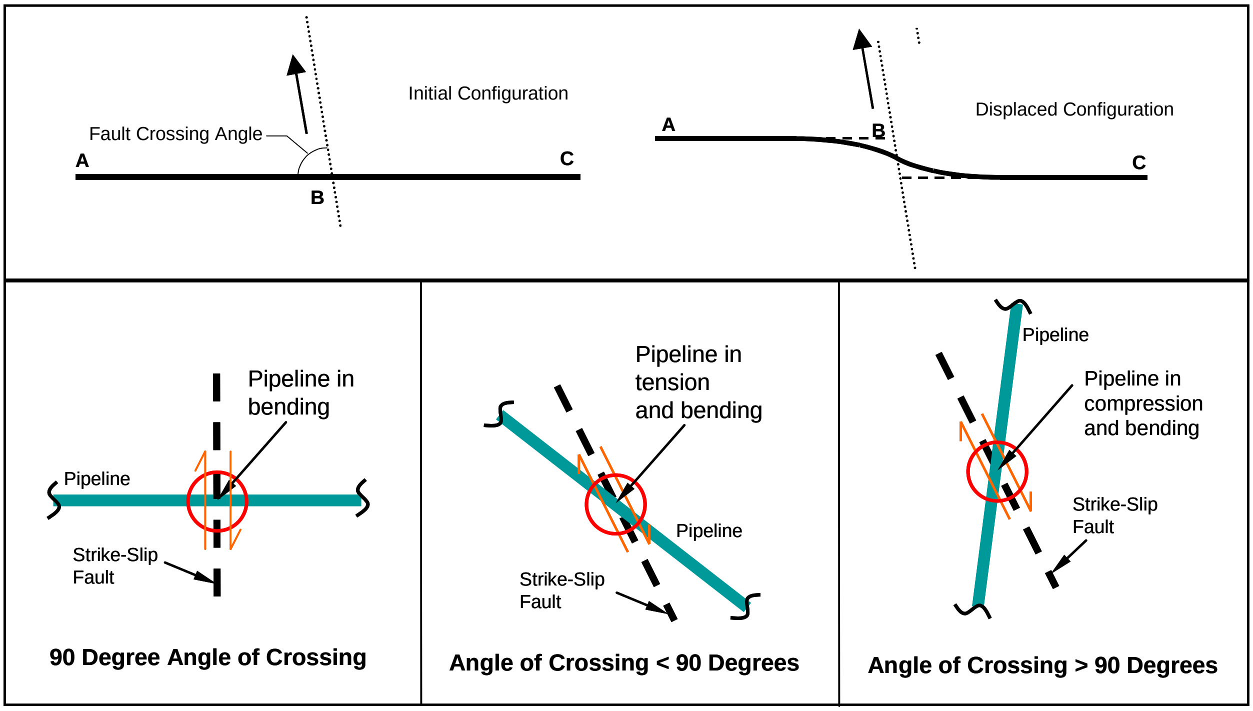


Figure 4: Response of Continuous Pipelines to Fault Rupture Displacement

2.3 Pipeline Response to Transient Ground Deformations

Transient ground strains result from spatial incoherency of ground motion, wave passage effects and general variation in site amplification across the site (Bolt et al., 2004).

During an earthquake, seismic waves propagate from the rupturing fault in the form of body waves that include both compression and shear waves. Compression waves cause axial compressive and tensile strains in the ground in a radial direction away from the hypocenter (the location at depth where the fault rupture first originates). Shear waves induce shear strains in the ground perpendicular to these radial lines. When the compression waves and shear waves are reflected by interaction with the ground surface, surface waves (Love waves and Rayleigh waves) are generated. Surface waves can also be generated by surface fault displacement. Except at very large distances from the epicenter, the magnitude of surface waves is significantly less than body waves.

A pipeline buried in soil when subjected to seismic waves incurs longitudinal and bending strains as it conforms to the associated ground strains. In most cases, these strains are relatively small, and pipelines in good condition typically accommodate these strains without damage. Bending strains are a function of ground curvature, the maximum value of which is the second derivative of transverse displacement. Bending strains are typically ignored as they are substantially smaller than longitudinal strains. Propagating seismic waves also give rise to hoop membrane strains and shearing strains but these strains are even smaller and are also generally neglected.

Compared to the deformation induced in the pipeline due to fault rupture, which is in the order of several feet, the transient ground strains are considered secondary and ignored.

2.4 Analysis Methodology

For buried pipelines, the axial component of fault movement is resisted by friction forces at the soil-pipeline interface. The resulting longitudinal strain in the pipeline at the fault crossing is a function of soil resistance. Higher soil resistance can concentrate large strains in a smaller length of the pipeline.

For a given pipeline axial force, there is a length of pipeline required to develop opposing soil frictional forces. Beyond this length, the pipeline is not affected by the fault movement and can be considered anchored.

Within the zone of large PGD the pipeline accommodates the ground deformation by failing the adjacent soil. The failure surface for lateral soil restraint is the shape of a logarithmic spiral as shown in Figure 5. Such a failure surface has also been observed in laboratory tests (Audibert and Nyman, 1977). Generally loose granular backfills (sand or gravel) that offer less resistance to pipe movement are preferred over dense or cohesive backfill materials (clay or silty clay) within the fault zone.

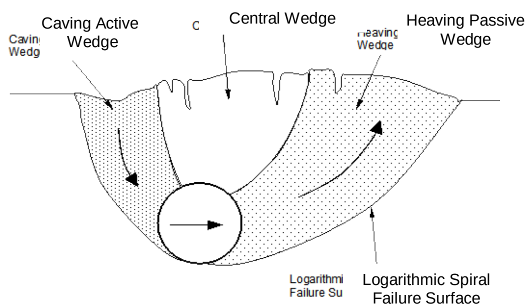


Figure 5: Logarithmic Spiral Failure of Soil (Audibert and Nyman, 1977)

2.4.1 Modeling Approach

Nonlinear finite element analysis for the Santa Rosa aqueduct was performed using the general purpose finite element analysis program ANSYS®. The analyses included consideration of material and geometric nonlinearity of the pipeline and surrounding soil.

The buried sections of pipelines were modeled as one dimensional special pipe element (PIPE20) in ANSYS. The PIPE20 element has eight integration points along its circumference. It can account for internal pressure and is capable of large displacement and plastic deformation.

In the finite element model, the three-dimensional soil restraint to the pipeline was represented by a series of discrete springs with load-deformation characteristics denoted by t-x, p-y, and q-z curves. These springs represent the nonlinear, stress-dependent behavior of soils in the axial, lateral, and vertical directions, respectively and were computed based on soil properties along the pipeline alignment. Figure 6 shows a schematic of the pipe-spring model used for the analysis. The finite element model of the Santa Rosa aqueduct is shown in Figure 7

In ANSYS, the soil springs were modeled using the COMBIN39 element. COMBIN39 is a unidirectional element with nonlinear generalized load-deflection capability and can be used for one, two, or three-dimensional applications. Pipe-soil interaction is modeled using a uniaxial tension-compression element with up to three translational degrees of freedom at each node. The element is defined by two node points and a generalized piecewise-linear force-deflection curve. The COMBIN39 element is nonlinear and requires an iterative solution.

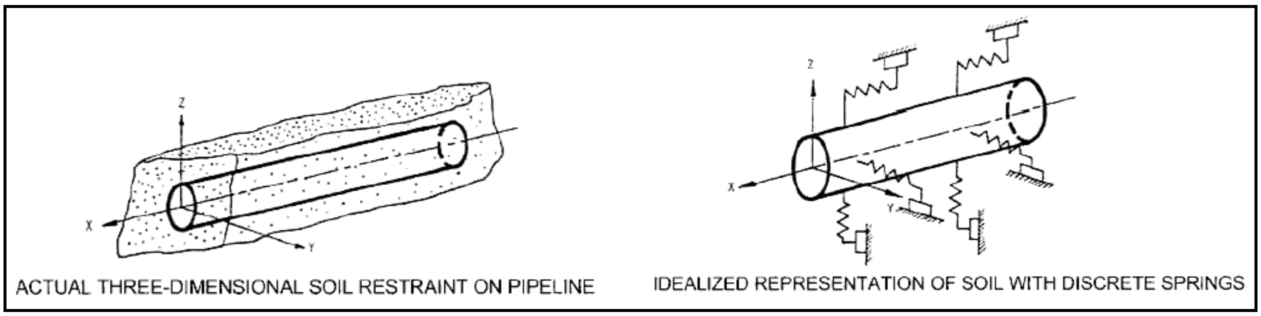


Figure 6: Schematic of Analysis Model (PRCI, 2004)

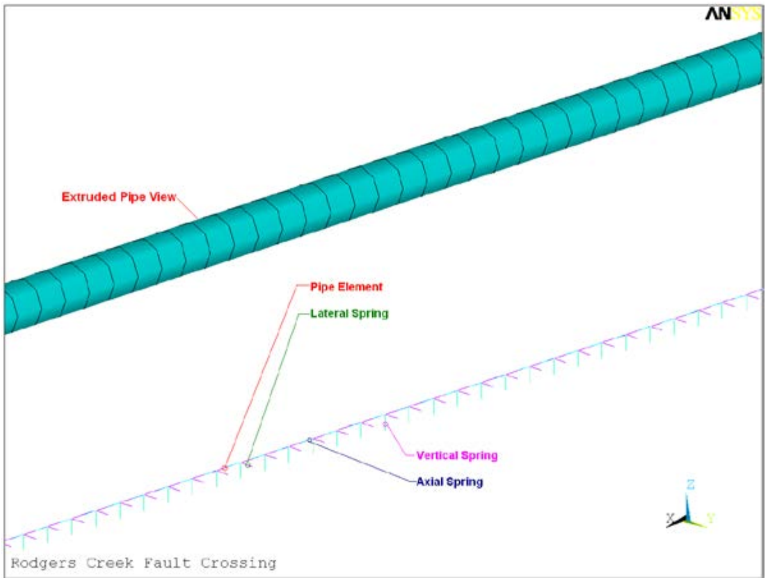
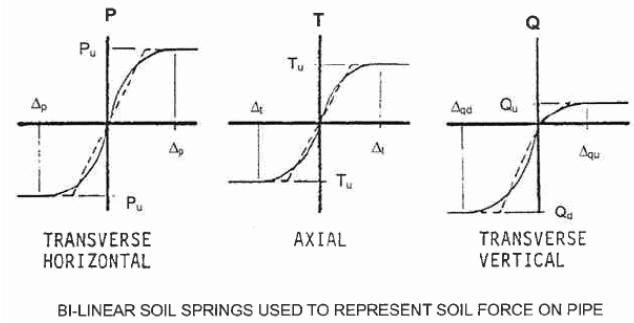


Figure 7: Finite Element Model of Santa Rosa Aqueduct

2.4.2 Soil Springs

Bilinear soil springs were developed as a function of pipeline diameter, depth of burial and soil properties. At each node of the pipeline model, soil springs were specified in the longitudinal, transverse, vertical up and vertical down direction. The soil springs were based on approaches provided in ALA, 2001, PRCI, 2004 and PRCI 2009. These approaches are similar to those contained in earlier ASCE guidelines (ASCE, 1984) and have recently been validated through large scale tests (O'Rourke, 1999). Figure 8 shows the formulation used for computing soil springs for the pipelines.

Soil springs were computed for each node of the finite element model. At each node the longitudinal, lateral, vertical up and vertical down springs were computed using the best estimate of soil properties obtained from the geotechnical/fault investigations (Appendix A) and summarized in Table 1. Uncertainties associated with spring formulations (Figure 8) and soil properties were incorporated by assuming 50% and 200% of the best estimated spring values.



$$T_u = \pi D\alpha c + \pi DH\gamma\left(\frac{1+K_0}{2}\right)\tan(\delta)$$

$$P_u = N_{ch}cD + N_{qh}\gamma HD \le Q_d$$

$$Q_u = N_{cv}cD + N_{qv}\gamma HD$$

$$Q_d = N_c cD + N_q\gamma HD + N_\gamma\gamma\frac{D^2}{2}$$

Tu = axial soil spring, Pu = lateral soil spring, Qu = vertical uplift spring, Qd = vertical bearing soil spring, D = pipe outside diameter, c = soil cohesion for backfill, H = depth to pipe centerline, γ = effective unit weight of soil, K_o = coefficient of pressure at rest, α = adhesion factor, δ = interface angle of friction for pipe and soil = $f\phi$, ϕ = internal friction angle for soil backfill, f = coating dependent friction factor at soil-pipe interface, N_{ch} = horizontal bearing capacity factor for clay, N_{qh} = horizontal bearing capacity factor for sand, N_{cv} = vertical bearing capacity factor for clay, N_{qv} = vertical bearing capacity factor for sand, N_c, N_q, N_γ = bearing capacity factors

Figure 8: Soil Spring Formulation (PRCI, 2004)

3. FAULT INVESTIGATION

3.1 <u>Historic Background</u>

The Rodgers Creek fault is one of the primary active faults in the San Francisco Bay Area and a major component of the San Andreas Fault system, connecting the Hayward fault on the south to the Maacama fault on the north. The fault is considered capable of generating an earthquake large enough to result in surface rupture.

Historically, the Rodgers Creek fault has been seismically quiescent. The only moderate to large earthquakes located near the fault were the October 1, 1969 M 5.6 and 5.7 earthquakes near Santa Rosa, and the Mare Island event of 1898 (Wong and Bott, 1995). Based on historical accounts for the region, the 1898 earthquake is interpreted as an event with magnitude between 6.2 and 6.7 that occurred along the southernmost section of the Rodgers Creek fault (Toppozada and others, 1992). Other than these events, there is no evidence of a large surface rupturing earthquake on the fault in historical records. This implies that the most recent large earthquake likely occurred before 1824, when the first historical records started being kept at a Franciscan mission built in Sonoma, and possibly before 1776, when a mission and presidio were first established in San Francisco (Hecker and others, 2005).

Results of fault trenching on the Rodgers Creek fault document the occurrence of three surface-rupturing earthquakes between about A.D. 1000 and 1776 (Schwartz et. al, 1992). Research studies indicate a recurrence interval of between 131 to 370 years (preferred value of 230 years), calculated from geologic data and regional earthquake models. Radiocarbon dating from faulted alluvial sequence at the site indicates that the most recent large earthquake occurred no earlier than A.D. 1690 and most likely occurred after A.D. 1715 (Hecker and others, 2005). Therefore, the elapsed time since the most recent large earthquake on the Rodgers Creek fault is more than 187 years and less than 321 years. The U.S. Geological Survey estimates that there is a 15.2% probability of a M7.0 rupture of the Rodger's Creek fault in the next 30 years.

3.2 Fault Location

Within the Santa Rosa Plain, there are no well-defined geomorphic features and the fault has been mapped as a discontinuous zone of sub-parallel strands from San Pablo Bay to Geyerville. The Alquist-Priolo Special Studies Zone map (CDMG, 1974) shows the Rodgers Creek fault as a single concealed fault trace inferred to connect the Healdsburg fault to the north with the Rodgers Creek fault to the south.

Precise location of the fault to the degree possible is required to design the pipeline against surface fault rupture hazard. In addition to the location of the fault, the crossing angle of the fault relative to the pipeline, width of fault zone and the estimate of rupture displacement are also required. To define these design parameters, fault investigations consisting of the following activities were performed:

- Review of existing published and unpublished literature: This included review of previous mapping studies, previous trenching investigations, review of 1942-vintage stereo aerial photographs, discussions with United States Geological Survey (USGS) researchers, review of LiDAR (light detection and ranging) and obtaining, processing and interpretation of 2-D seismic data collected by USGS along Sonoma Avenue.

- Field reconnaissance: This included reconnaissance by a Certified Engineering Geologist and a Senior Staff Geologist to evaluate evidence of aseismic creep and collection of information on surface deposits and site geotechnical conditions.

- Subsurface Exploration: Due to lack of appropriate trenching location and thickness of young Holocene alluvium at the fault crossing location, fault trenching was not considered a practical option. As a result, 12 Cone Penetrometer Tests (CPTs) ranging from 40 to 70 feet depth and one 40-foot deep boring was performed to track fault related offset in horizontal soil layers.

- Laboratory Testing: Laboratory testing to obtain geotechnical properties of subsurface soils was performed on seven representative soil samples.

Details of fault investigation study are included in Appendix A. The investigations identified five different fault strands that merge into a single vertical strand at depth as shown in Figure 9. These fault strands are labeled A through E as shown in Figure 10. Based on the fault investigations performed for the project, it is concluded that the main trace of the fault is constrained within about 300 feet (90 m), in the vicinity of Talbot and Macklyn Avenues. Within this zone two possible fault traces (‘C’ and ‘D’) are interpreted that bound the shear zone. An additional fault, trace ‘E’ centered between Rosedale and Alderbrook Avenues, is also inferred on the basis of these investigations. The zone of possible faulting centered on fault E is 200 feet (60 m) wide. The fault strands A and B are judged to be not active based on the lack of evidence of active faulting in previous trench logs.

The crossing angle between the pipeline and the fault strands is estimated to be 89° ± 3°. The fault configuration represents a flower type structure as shown in Figure 11.

Figure 12 shows surface manifestation of rupture from a fault exhibiting flower type structure.

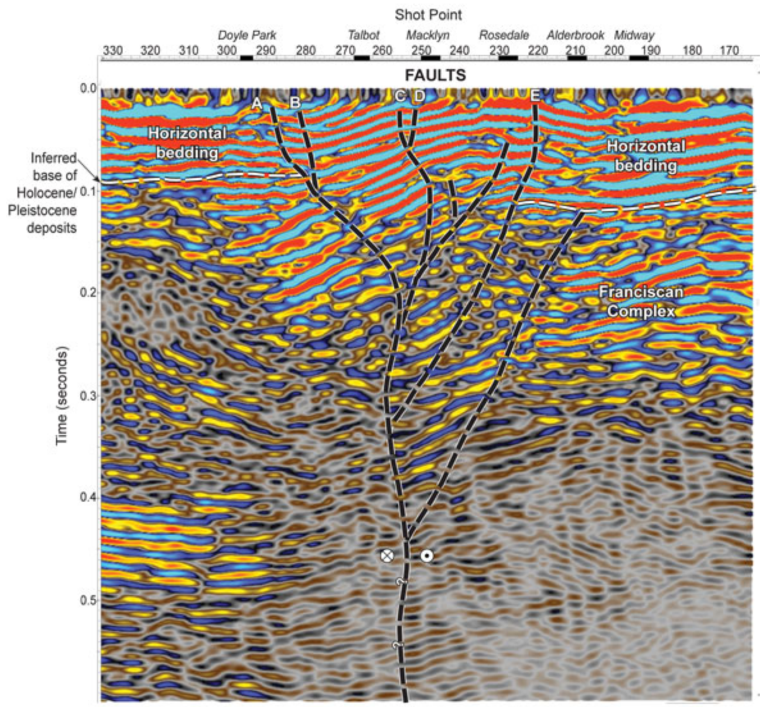


Figure 9: Interpreted 2-D Seismic Line Showing Possible Fault Structure Along Sonoma Avenue Across Rodgers Creek Fault

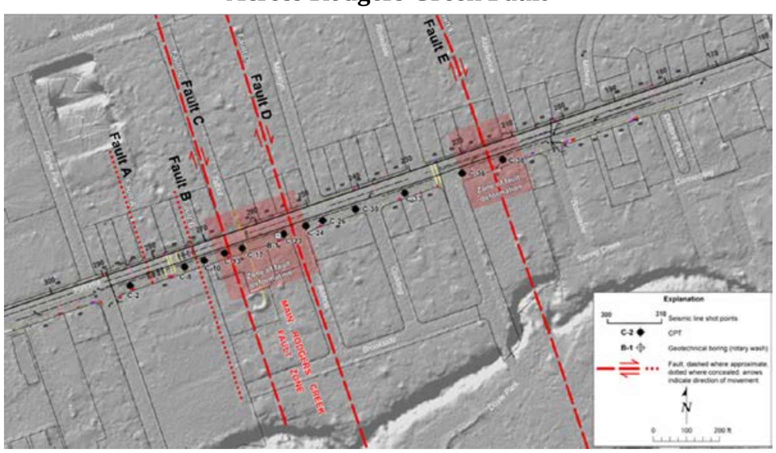


Figure 10: Possible Fault Location at Sonoma Avenue in Santa Rosa, California

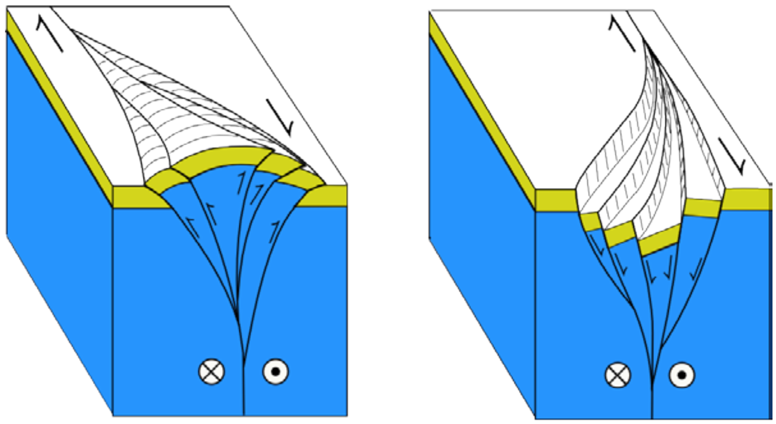


Figure 11: Configuration of Flower Structure

Figure 12: Surface Manifestation of Rupture Across a Fault Exhibiting Flower Structure (1992 Landers Earthquake)

3.3 Magnitude and Displacement Estimate

Current research suggests that the Rodgers Creek fault is capable of generating an earthquake as large as M_w 7.0 based primarily on length of the fault. An upper bound of M_w 7.3 has also been considered by the Working Group on California Earthquake Probabilities (WGCEP, 2003; 2008) based on the assumption of a combined rupture of Hayward and Rodgers Creek fault. An event considered possible but with a significantly lower probability than the Rodgers Creek fault only rupture because of the approximately 4 km wide step over between the two faults beneath San Pablo Bay. As a result, M_w 7.0 has been adopted as a design earthquake magnitude for this project.

Empirical relations developed by Wells and Coppersmith (1994) suggest an average subsurface displacement of 3.1 ± 1.8 feet for an M_w 7.0 earthquake. Based on the published geologic rate of 9 ± 1 mm/year (WGEP, 2008) and an inferred average fault creep rate of 4.9 ± 0.6 mm/year (Funning and others, 2007), horizontal displacement of 3.4 ± 0.6 feet and 4 to 12 inches of vertical displacement is possible.

3.4 Design Parameters

Parameters for design are summarized in Table 1. The table shows the magnitude of estimated rupture displacement, crossing angle and width of deformation zone. The table also shows soil material properties to be used in developing soil springs for numerical model of the pipeline.

Figure 13 and Figure 14 show a best estimate of the width of deformation zone and distribution of strain across each fault strand. Such estimates have significant uncertainty associated with them and therefore, a conservative interpretation with knife edge type offset was also considered.

For numerical analysis, four cased for fault offset were considered as follows:

- Fault C – Knife Edge: This case assumes that 100% of the estimated displacement occurs as a knife edge at Fault C.

- Fault C and D – Distributed: This case assumes that 100% of the estimated displacement occurs at the main fault strands C and D and that the displacement is distributed across a 300-foot wide fault zone as shown in Figure 13.

- Fault C, D and E – Distributed: This case assumes that 80% of the estimated displacement occurs at the main fault strands C and D and distributed across a 300-foot wide fault zone as shown in Figure 13 and 20% of the estimated displacement occurs at the fault strand E and distributed across a 200-foot wide zone as shown in Figure 14. This case represents an earlier interpretation of displacement distribution across the fault strands, which was later revised to 70% and 30% as shown in Table 1. Results presented in this report show that knife edge displacement result in highest strains in the pipe and therefore, additional analyses with 70% and 30% distribution were not performed.

- Fault E – Knife Edge: This case assumes that 100% of the estimated displacement occurs as a knife edge at Fault E.

Figure 15 shows the deformed shape of the pipeline with knife edge and distributed fault displacements. Initial analyses were performed with only the lateral fault displacement to study the overall impact of variability in soil stiffness, fault rupture characteristics and steel properties. For the alternative considered most viable, additional analyses were performed using both the estimated horizontal and vertical component of fault displacement using the best estimated properties. The final recommended design was also checked using a conservative set of properties.

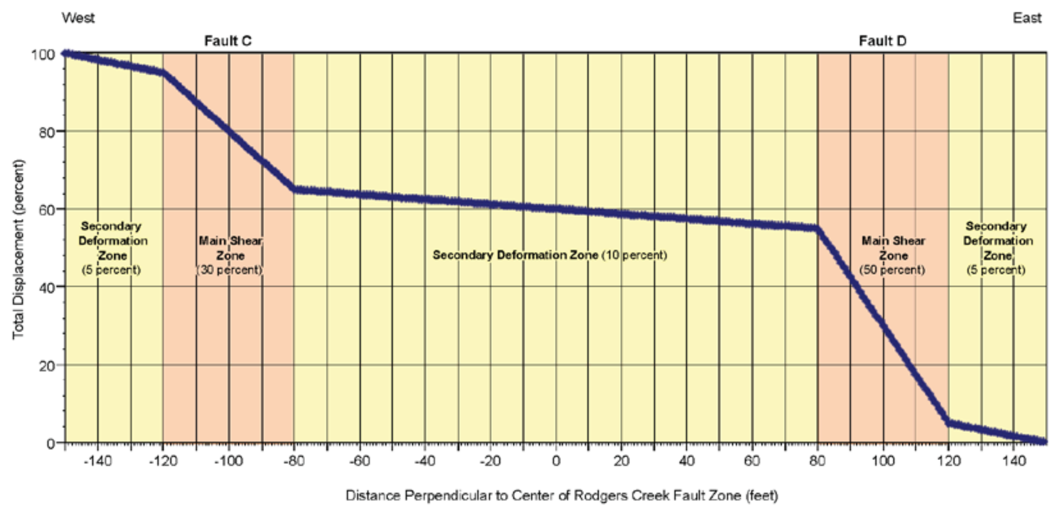


Figure 13: Width and Distribution of Deformation Across Fault Strands C and D

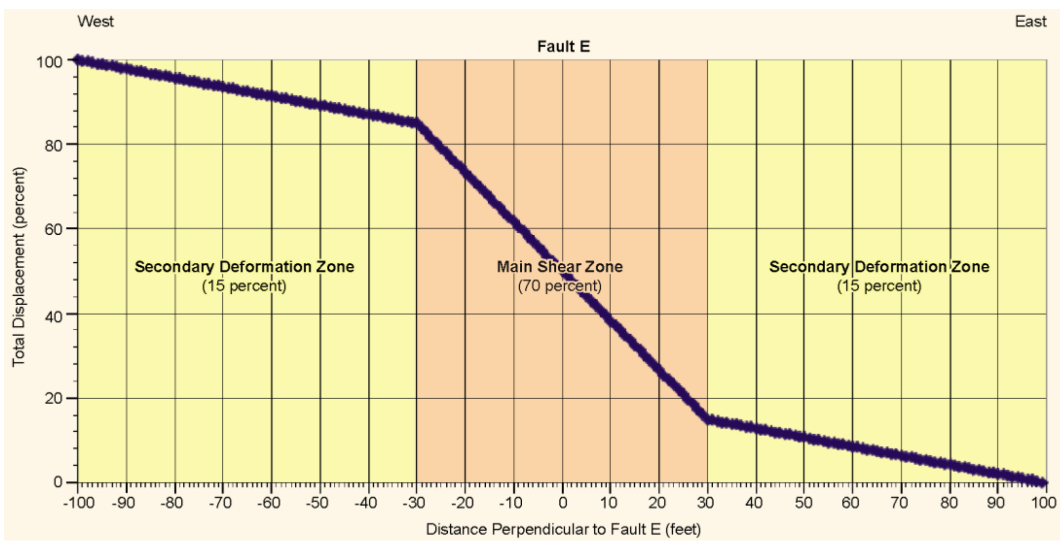


Figure 14: Width and Distribution of Deformation Across Fault Strand E

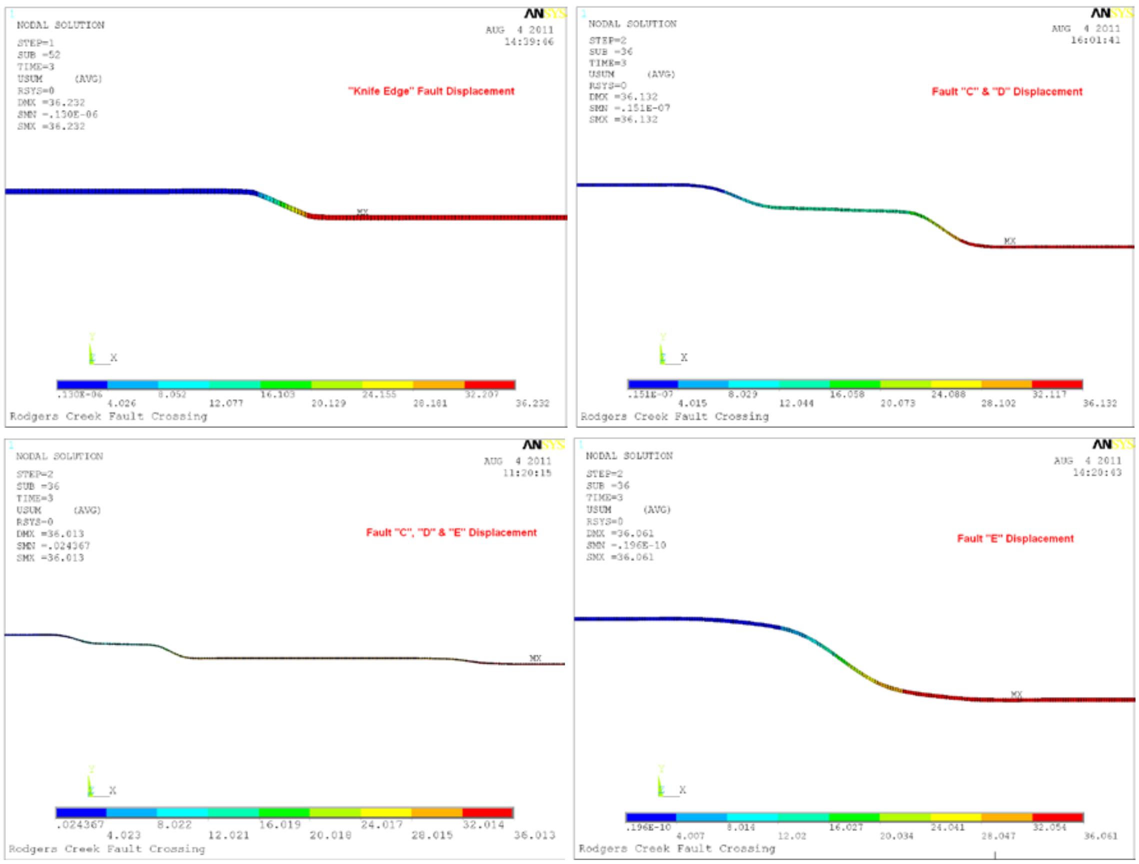


Figure 15: Application of Fault Displacement in the Finite Element Model

Table 1: Rodgers Creek Fault Design Parameter

Fault Trace	Magnitude Estimate	Estimated coseismic right-lateral slip	Estimated vertical component of slip	Primary offset width (strain distribution)	Crossing angle of pipe to strike of fault	Native materials properties (based on laboratory test data)
A and B	Mw 7.0 ± 0.2 Design Earthquake: Mw 7.0	N/A Trench data does not show active faulting	N/A	N/A	89° ± 3° (tension)	See Attachment 1, Appendix C
C and D		3.1 ± 1.8 feet	4 to 12 inches	300 feet (80% on primary traces = 1.0 to 3.9 feet; 20% distributed = 0.3 to 1.0 feet)		CLAYEY SAND (SC) to fat CLAY (CH): Dry density = 84 psf; Initial water content 11 to 35%; Apparent cohesion 0.4 ksf; Apparent friction angle 22°
E		Unknown, conservatively same as main fault	Unknown, conservatively same as main fault	200 feet (70% on primary trace = 0.9 to 3.4 feet 30% distributed = 0.4 to 1.5 feet)		

4. DESIGN AND PERFORMANCE CRITERIA

4.1 Allowable Tensile Strain

When strained in longitudinal tension, a well-designed steel pipeline is very ductile and is capable of mobilizing large strains associated with significant yielding before rupture. Strain levels associated with rupture are generally greater than 10% to 15%. Pipeline failure at lower strain levels may occur as a result of stress concentrations arising from weld discontinuities and non-uniformity in the pipe wall thickness, yield point variability, post-yield hardening and backfill soil properties. Accordingly, it is common practice to limit the maximum tensile strains well below the rupture level of the steel. A limiting tensile strain of 3% to 5% is generally recommended for modern pipelines and can be achieved with good quality control of pipe fabrication and backfill placement (ALA, 2005, PRCI, 2004).

Some of the reasons for limiting the allowable tensile strains (3% to 5%) significantly below the values that can be achieved in tensile strength tests (on the order of 20%) are: (1) the tensile tests are performed on small coupons from the body of the pipe or factory fabricated welds, (2) the most critical location for high pipe tensile strains is the heat-affected zone of lower ductility at the field fabricated girth welds, and (3) the boundary conditions for the tensile test specimens are not representative of the actual field conditions. This approach results in prudent conservatism given the large uncertainties associated with seismic design.

For the design of new pipelines, an allowable tensile strain of 4% is generally considered achievable without excessively burdensome requirements on weld quality and inspection (PRCI, 2004).

4.2 Allowable Compressive Strain

Flexural compressive strains of the same order of magnitude as tensile strains will generally not result in rupture, although consideration should be given to the potential for wrinkling due to compressive bending strains. Tests of large diameter steel Grade

X-60 pipe with a diameter to thickness ratio of about 80 (Bouwkamp and Stephen, 1973) demonstrate that large diameter pipe under pressure can mobilize significant additional flexural strain after the onset of wrinkling (up to 20 times the curvature associated with initial wrinkling). More recent tests at the University of Alberta, Canada (Mohareb et al., 1994, Yoosef-Ghodsi et al., 1994) have also demonstrated the ability of pipe with diameter to thickness ratios of 50 to 65 to withstand flexural strains 20 to 30 times greater than those associated with the first visible signs of pipe wrinkling.

Based on investigations on pipelines subjected to bending and axial compressive load resulting from differential thermal conditions, Mohareb et al. (1994) proposed several relationships for compressive bending strain based on different assumptions for acceptable pipeline limit states. These limit states included the strain associated with peak moment capacity of the pipe, post peak moment capacity strains related to maintaining 95% of the peak moment capacity, 15% ovalization of the pipe cross section, and 8% tensile hoop strain in the buckle formed after initial wrinkling of the pipe wall. A limit state associated with 15% ovalization of the pipe cross section is considered acceptable for pipeline performance at fault crossings and is recommended in the ALA and PRCI guidelines. For the 36-inch diameter pipe with 0.75-inch wall thickness an allowable compressive strain of 3.7% is computed using the formula 1.76t/D (where t is the wall thickness and D is the pipeline diameter) based on recommendations in ALA, 2005.

5. DESIGN ALTERNATIVES

The general approach for the fault crossing design is to accommodate the fault rupture displacement through bending and stretching of the pipeline. Several feet of fault displacement will result in the pipeline to deform well into the plastic range. In order to achieve this, the deforming pipe needs to fail the adjacent soil as shown in Figure 5 and should have joints that have sufficient strength to allow large deformation of the pipeline. Furthermore, it is important that within the zone of excessive deformation the pipeline does not have any sharp bends, hard points or strain risers.

Preliminary analysis showed that approximately 2,500 feet of new pipeline will likely be needed to for the fault crossing design of the Santa Rosa aqueduct. In order to maintain a straight run of the pipeline without sharp vertical or horizontal bends, the existing utility maps for the pipeline alignment were reviewed. The plan and profile of the existing pipeline together with the existing utilities are shown in Plate 1, Plate 2 and Plate 3. Considering these constraints, two design alternatives are considered as described below:

5.1 Design Alternative 1 – Deep Design

In order to avoid existing utilities and achieve a straight length of approximately 2,500 feet of pipeline centered on the identified fault traces, Design Alternative 1 (referred to as the deep design) considers that the pipeline is buried below the existing utilities as shown in Plate 1, Plate 2 and Plate 3. Nonlinear finite element analyses were performed to design the pipeline. Several preliminary analyses were performed to finalize the design. For the final design shown in Figure 16, 72 parametric studies were performed to study the effects of uncertainties in soil stiffness, steel yield strength, fault crossing angle and the distribution of fault rupture across each fault strand. The results of the analyses are summarized in Table 2.

Features of the design include:

- The new pipeline extends from Station[1] 678 to Station 703 for a total length of 2,500 feet. The total length of the pipeline is such that the strain and axial force at the tie-in points with the existing pipeline is less than the joint capacity of the existing pipe.

- A 1-inch wall thickness of the pipeline is required within the fault zone to keep tensile and compressive strains within the allowable limits. Analyses were performed for pipe thickness less than 1-inch, which yielded strains higher than the allowable values.

[1] All Station references in this report refer to stationing for the existing pipe

- Lining for the new pipeline is required to be fusion bonded epoxy. Brittle material such as mortar is not recommended within the zone of large deformation and pieces of lining would break and can jam downstream valves.

- Low friction coating is required between Station 685+50 and Station 697. Fusion bonded epoxy coating wrapped with low friction wrap to allow pipeline to slip relative to the surrounding backfill. Beyond these stations a cement mortar coating is recommended to gradually transfer the pipeline strains to the surrounding material.

- The pipeline is located within the right of way of a major street. Standard pipeline trench required for carrying traffic loads is used for the design.

- Pipeline yield strength should be limited to less than 48 ksi and a minimum 20% elongation.

- Quality control of field and shop welding equivalent to what is required for high pressure gas pipelines is required such that the joints are stronger than the pipe barrel and allow the pipe barrel to develop the required plastic strain.

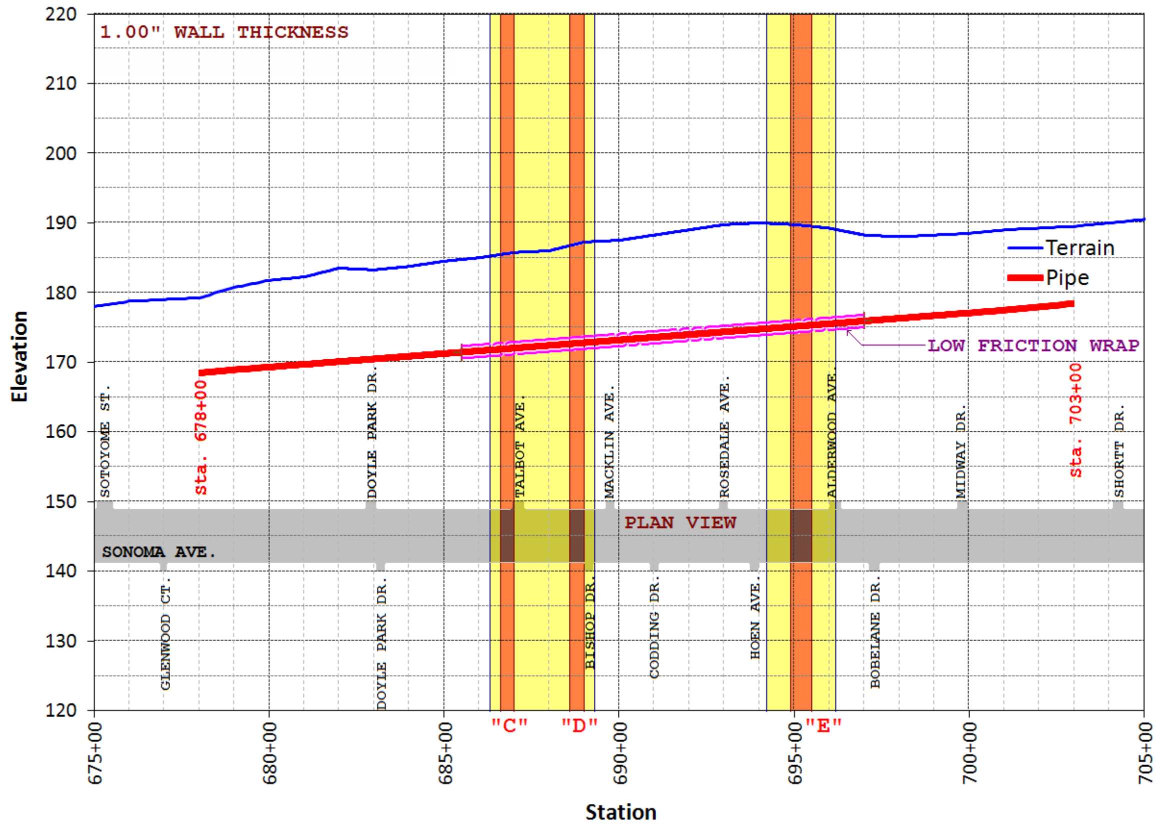


Figure 16: Design Alternative 1 (Deep Design)

Table 2: Design Alternative 1 Summary of Results

Soil Stiffness	Fault Trace	Steel Yield Strength Fy = 36ksi			Steel Yield Strength Fy = 48ksi		
		84 degree	89 degree	94 degree	84 degree	89 degree	94 degree
50% Soil Springs	Fault C Knife Edge	1.05% (T)	0.98% (T)	0.83% (T)	0.49% (T)	0.35% (T)	0.47% (T)
		0.88% (C)	0.73% (C)	0.39% (C)	0.43% (C)	0.27% (C)	0.21% (C)
	Fault C and D Distributed	0.10% (T)	0.13% (T)	0.18% (T)	0.11% (T)	0.13% (T)	0.15% (T)
		0.18% (C)	0.13% (C)	0.11% (C)	0.16% (C)	0.14% (C)	0.10% (C)
	Fault C, D and E Distributed	0.08% (T)	0.11% (T)	0.14% (T)	0.08% (T)	0.11% (T)	0.13% (T)
		0.14% (C)	0.12% (C)	0.09% (C)	0.14% (C)	0.12% (C)	0.08% (C)
	Fault E Knife Edge	0.66% (T)	0.67% (T)	0.56% (T)	0.29% (T)	0.29% (T)	0.33% (T)
		1.10% (C)	0.59% (C)	0.26% (C)	0.44% (C)	0.26% (C)	0.17% (C)
100% Soil Springs	Fault C Knife Edge	2.02% (T)	2.00% (T)	1.95% (T)	1.87% (T)	1.68% (T)	1.43% (T)
		2.00% (C)	1.56% (C)	0.96% (C)	1.48% (C)	1.32% (C)	0.84% (C)
	Fault C and D Distributed	0.17% (T)	0.24% (T)	0.32% (T)	0.15% (T)	0.18% (T)	0.21% (T)
		0.30% (C)	0.21% (C)	0.15% (C)	0.21% (C)	0.17% (C)	0.15% (C)
	Fault C, D and E Distributed	0.12% (T)	0.15% (T)	0.21% (T)	0.12% (T)	0.14% (T)	0.17% (T)
		0.19% (C)	0.15% (C)	0.12% (C)	0.17% (C)	0.15% (C)	0.12% (C)
	Fault E Knife Edge	1.86% (T)	1.75% (T)	1.77% (T)	1.48% (T)	1.39% (T)	1.30% (T)
		1.96% (C)	1.59% (C)	0.87% (C)	1.73% (C)	1.23% (C)	0.85% (C)
200% Soil Springs	Fault C Knife Edge	3.35% (T)	2.99% (T)	3.18% (T)	2.90% (T)	2.92% (T)	2.95% (T)
		3.06% (C)	2.51% (C)	1.63% (C)	2.63% (C)	2.24% (C)	1.46% (C)
	Fault C and D Distributed	0.25% (T)	0.34% (T)	0.47% (T)	0.22% (T)	0.28% (T)	0.33% (T)
		0.41% (C)	0.28% (C)	0.21% (C)	0.31% (C)	0.24% (C)	0.17% (C)
	Fault C, D and E Distributed	0.17% (T)	0.24% (T)	0.32% (T)	0.15% (T)	0.18% (T)	0.22% (T)
		0.29% (C)	0.20% (C)	0.15% (C)	0.21% (C)	0.17% (C)	0.15% (C)
	Fault E Knife Edge	3.35% (T)	2.81% (T)	3.08% (T)	2.80% (T)	2.72% (T)	2.53% (T)
		3.48% (C)	2.67% (C)	1.72% (C)	2.99% (C)	2.30% (C)	1.42% (C)

5.2 Design Alternative 2 – Shallow Design

The Design Alternative 1 requires 1-inch pipe thickness predominantly because of the significant depth of burial (more than 10-feet deep). To reduce pipe thickness a shallow design was considered. However, due to the presence of sewer near Doyle Park Drive a straight run of pipeline to completely transfer the pipeline strain to the surrounding soil cannot be provided. Design Alternative 2 considers a shorter length of pipeline with two 90 degree elbows on either side to connect to the existing pipeline as shown in Figure 17.

Features of the design include:

- The new pipeline extends from Station 684 to Station 700 for a total length of 1,600 feet. The length of the pipeline is constrained by the sewer at Doyle Park Drive. Because of the shorter length of the pipeline, significantly large axial force in the pipeline needs to be accommodated. This is partly achieved by placing 150-foot section of pipe in controlled low strength material (CLSM). However, approximately 23 kip force still needs to be transferred to the tie-in. The analysis results show that the load can be resisted through bending of the pipe laterals between the two 90 degree elbows.

- By reducing the depth of burial of the pipeline, a 0.75-inch wall thickness of the pipeline can be used within the fault zone to keep tensile and compressive strains within the allowable limits. Analyses were also performed for pipe thickness of 0.5 inches, which yielded strains higher than the allowable values.

- Lining for the new pipeline is required to be fusion bonded epoxy. Brittle material such as mortar is not recommended within the zone of large deformation and pieces of lining would break and can jam downstream valves.

- Low friction coating is required between Station 685+50 and Station 697. Fusion bonded epoxy coating wrapped with low friction wrap to allow pipeline to slip relative to the surrounding backfill. Beyond these stations a cement mortar coating is recommended to gradually transfer the pipeline strains to the surrounding material.

- The pipeline is located within the right of way of a major street. Standard pipeline trench required for carrying traffic loads is used for the design.
- Pipeline yield strength should be limited to less than 48 ksi and a minimum 20% elongation.
- Quality control of field and shop welding equivalent to what is required for high pressure gas pipelines is required such that the joints are stronger than the pipe barrel and allow the pipe barrel to develop the required plastic strain.

5.3 Design Alternative 3 – Shallow Design, 45 Degree Elbows

A third design alternative in which the 90 degree elbows for Design Alternative 2 were replaced with 45 degree elbows was also considered to improve the hydraulic performance of the pipeline. The results of analyses for this case showed significant axial force in the pipeline at the tie-in locations, which were considered unacceptable and the design concept was not developed further. Subsequent discussions with the Agency suggested that the 90 degree elbows were acceptable for the hydraulic performance of the pipeline.

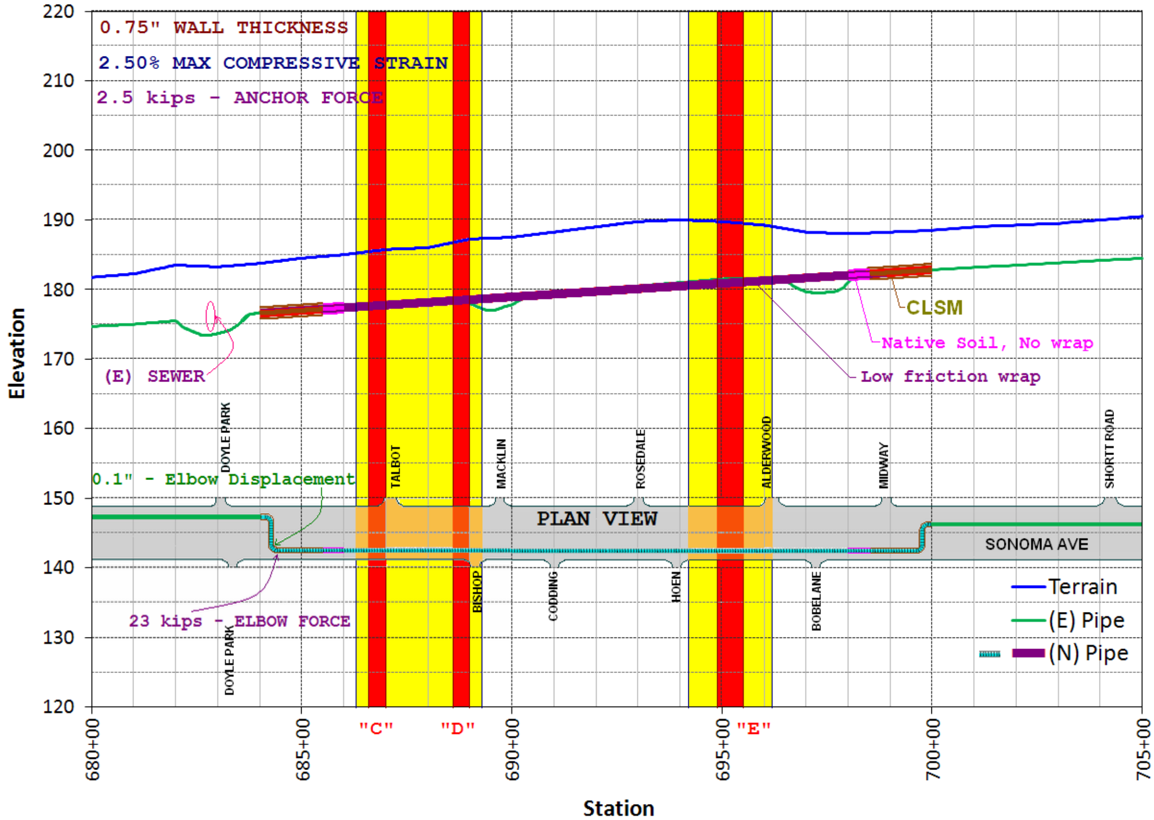


Figure 17: Design Alternative 2 (Shallow Design)

Table 3: Design Alternative 2 Summary of Results

Soil Stiffness	Fault Trace	Steel Yield Strength Fy = 36ksi			Steel Yield Strength Fy = 48ksi		
		84 degree	89 degree	94 degree	84 degree	89 degree	94 degree
50% Soil Springs	Fault C Knife Edge	0.18% (T)	0.25% (T)	0.27% (T)	0.15% (T)	0.17% (T)	0.19% (T)
		0.34% (C)	0.19% (C)	0.13% (C)	0.20% (C)	0.16% (C)	0.12% (C)
	Fault C and D Distributed	0.08% (T)	0.11% (T)	0.13% (T)	0.08% (T)	0.11% (T)	0.13% (T)
		0.14% (C)	0.11% (C)	0.08% (C)	0.14% (C)	0.11% (C)	0.08% (C)
	Fault C, D and E Distributed	0.08% (T)	0.07% (T)	0.11% (T)	0.07% (T)	0.09% (T)	0.11% (T)
		0.12% (C)	0.12% (C)	0.07% (C)	0.12% (C)	0.10% (C)	0.07% (C)
	Fault E Knife Edge	0.21% (T)	0.38% (T)	0.31% (T)	0.17% (T)	0.18% (T)	0.20% (T)
		0.37% (C)	0.25% (C)	0.15% (C)	0.23% (C)	0.16% (C)	0.14% (C)
100% Soil Springs	Fault C Knife Edge	0.98% (T)	1.04% (T)	0.76% (T)	0.46% (T)	0.60% (T)	0.42% (T)
		1.11% (C)	0.93% (C)	0.41% (C)	0.49% (C)	0.55% (C)	0.22% (C)
	Fault C and D Distributed	0.13% (T)	0.15% (T)	0.21% (T)	0.12% (T)	0.14% (T)	0.17% (T)
		0.22% (C)	0.15% (C)	0.12% (C)	0.17% (C)	0.15% (C)	0.12% (C)
	Fault C, D and E Distributed	0.10% (T)	0.12% (T)	0.15% (T)	0.09% (T)	0.12% (T)	0.14% (T)
		0.15% (C)	0.12% (C)	0.10% (C)	0.15% (C)	0.12% (C)	0.10% (C)
	Fault E Knife Edge	1.20% (T)	1.21% (T)	0.89% (T)	0.73% (T)	0.92% (T)	0.50% (T)
		1.27% (C)	1.02% (C)	0.60% (C)	0.76% (C)	0.78% (C)	0.24% (C)
200% Soil Springs	Fault C Knife Edge	1.97% (T)	2.03% (T)	1.80% (T)	1.84% (T)	1.78% (T)	1.53% (T)
		2.00% (C)	1.75% (C)	0.96% (C)	1.86% (C)	1.46% (C)	0.87% (C)
	Fault C and D Distributed	0.20% (T)	0.39% (T)	0.36% (T)	0.17% (T)	0.20% (T)	0.26% (T)
		0.34% (C)	0.35% (C)	0.18% (C)	0.24% (C)	0.19% (C)	0.16% (C)
	Fault C, D and E Distributed	0.14% (T)	0.17% (T)	0.24% (T)	0.13% (T)	0.15% (T)	0.18% (T)
		0.24% (C)	0.16% (C)	0.13% (C)	0.18% (C)	0.16% (C)	0.13% (C)
	Fault E Knife Edge	2.34% (T)	2.19% (T)	1.86% (T)	2.02% (T)	1.98% (T)	1.70% (T)
		2.50% (C)	1.92% (C)	1.19% (C)	2.18% (C)	1.74% (C)	1.15% (C)

6. CONCLUSIONS AND RECOMMENDATIONS

Three different design alternatives were considered that included the following:

- Design Alternative 1 – This design includes a 1-inch thick pipeline with a straight run of pipeline between Stations 608+00 and 703+00. The depth below ground surface for the pipeline ranges from approximately 10 to 15 feet

- Design Alternative 2 – This design includes a 0.75-inch thick pipeline with two 90degree elbows on either end of the pipeline at Stations 686+00 and 700+00. The depth below ground surface for the pipeline ranges from approximately 7 to 10 feet

- Design Alternative 3 – This design is similar to Alternative 2 but instead of 90 degrees elbows it includes 45 degree elbows.

The results of the analyses show that both Design Alternatives 1 and 2 are viable designs. However, Alternative 2 is preferable because of shallower burial depth and lesser pipe thickness compared to Alternative 1. Furthermore, the maximum strain in the pipeline are lower are lower for Alternative 2 compared to Alternative 1 (as shown in Table 2 and Table 3) due to lower resistance from soil in case of shallower burial depth.

The results shown in Table 2 and Table 3 are based on the horizontal component of the fault rupture only. Additional analyses were performed for Design Alternative 2 to study the effect of the vertical component of the fault rupture. As shown in Table 1, the vertical displacements are estimated to range from 10% to 30% of horizontal displacement. Best estimate of vertical displacement was estimated to be 20% of the horizontal displacement. The additional analyses were performed for knife edge displacement on fault strand C and fault strand E using best estimate properties as follows:

- Analysis Case 1: This analysis case was performed with knife edge displacement on fault strand C using 100% soil spring values and fault crossing angle of 84 degrees. A horizontal displacement of 3.1 feet was applied together with vertical displacement of 7.1 inches (20% of the horizontal displacement value). Maximum compressive and tensile strain values for this analysis case were computed to be 1.1% and 0.7%, respectively.

- Analysis Case 2: This analysis case was performed with knife edge displacement on fault strand E using 100% soil spring values and fault crossing angle of 84 degrees. A horizontal displacement of 3.1 feet was applied together with vertical displacement of 7.1 inches (20% of the horizontal displacement value). Maximum compressive and tensile strain values for this analysis case were computed to be 1.13% and 1.0%, respectively.

- Analysis Case 3: This analysis case is similar to Analysis Case 2 except that 200% soil spring values were used instead. The results show maximum compressive and tensile strains of 2.4% and 2.0%, respectively.

Results from analysis cases 1, 2 and 3 show that the inclusion of vertical displacement has a favorable effect on strains with a slight reduction compared the case with no vertical displacement.

Maximum tensile and compressive strains from analyses case 2 and 3 are plotted as a function of incrementally increasing fault displacement (horizontal to vertical displacement component in the ratio of 1:0.2) in Figure 18. The results show that for the best estimate soil springs (100%) the tensile and compressive strains remain below the allowable limits for horizontal displacements more than 6-feet whereas for the more conservative spring values (200%) the compressive strains reach the allowable limit for a horizontal displacement of approximately 5-feet. On the basis of these results, Design Alternative 2 is recommended for design.

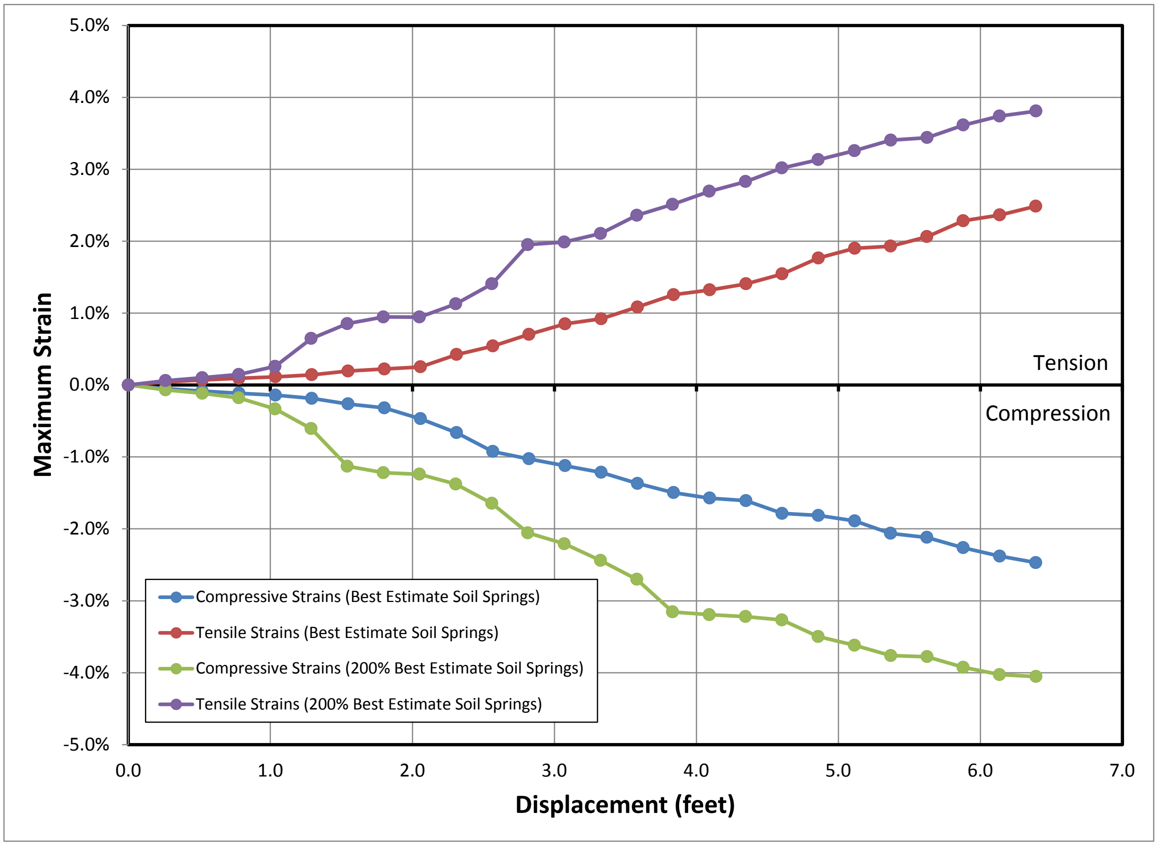


Figure 18: Maximum Strains as a Function of Increasing Displacement (Horizontal to Vertical Fault Rupture Ratio of 1:0.2) – Knife Edge on Fault Strand E

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PLATES

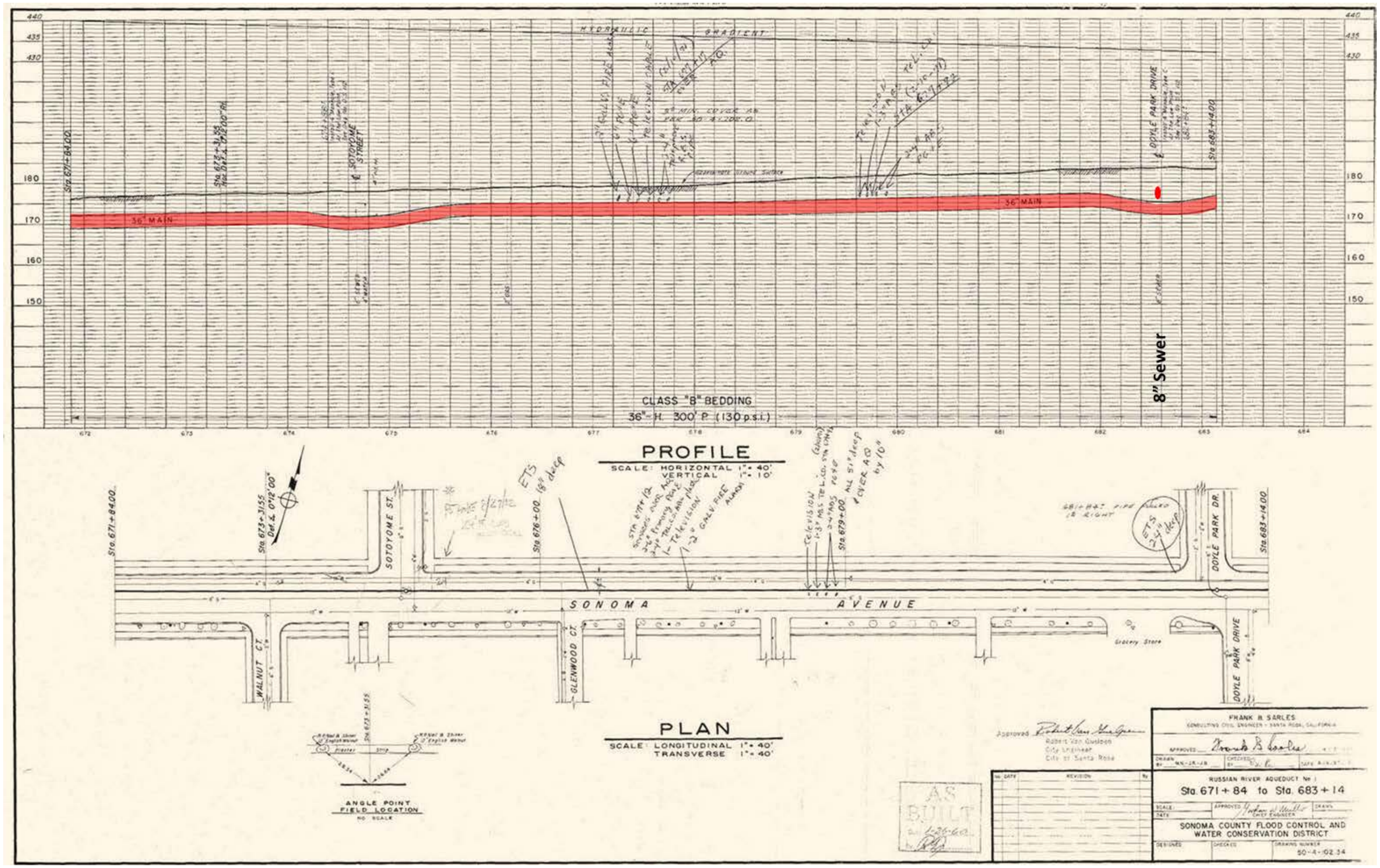

Plate 1: Plan and Profile of Existing Pipeline (1 of 3)

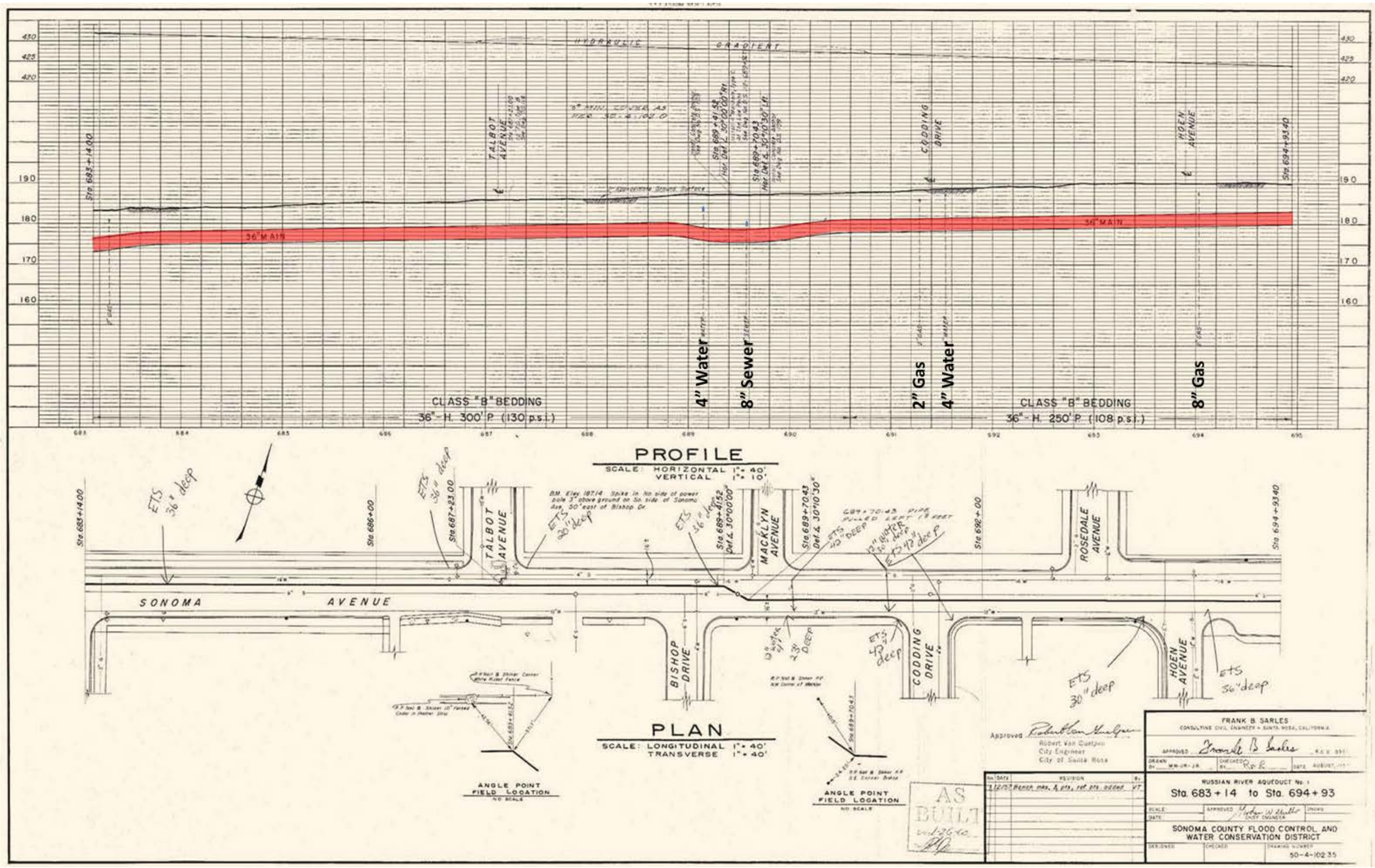

Plate 2: Plan and Profile of Existing Pipeline (2 of 3)

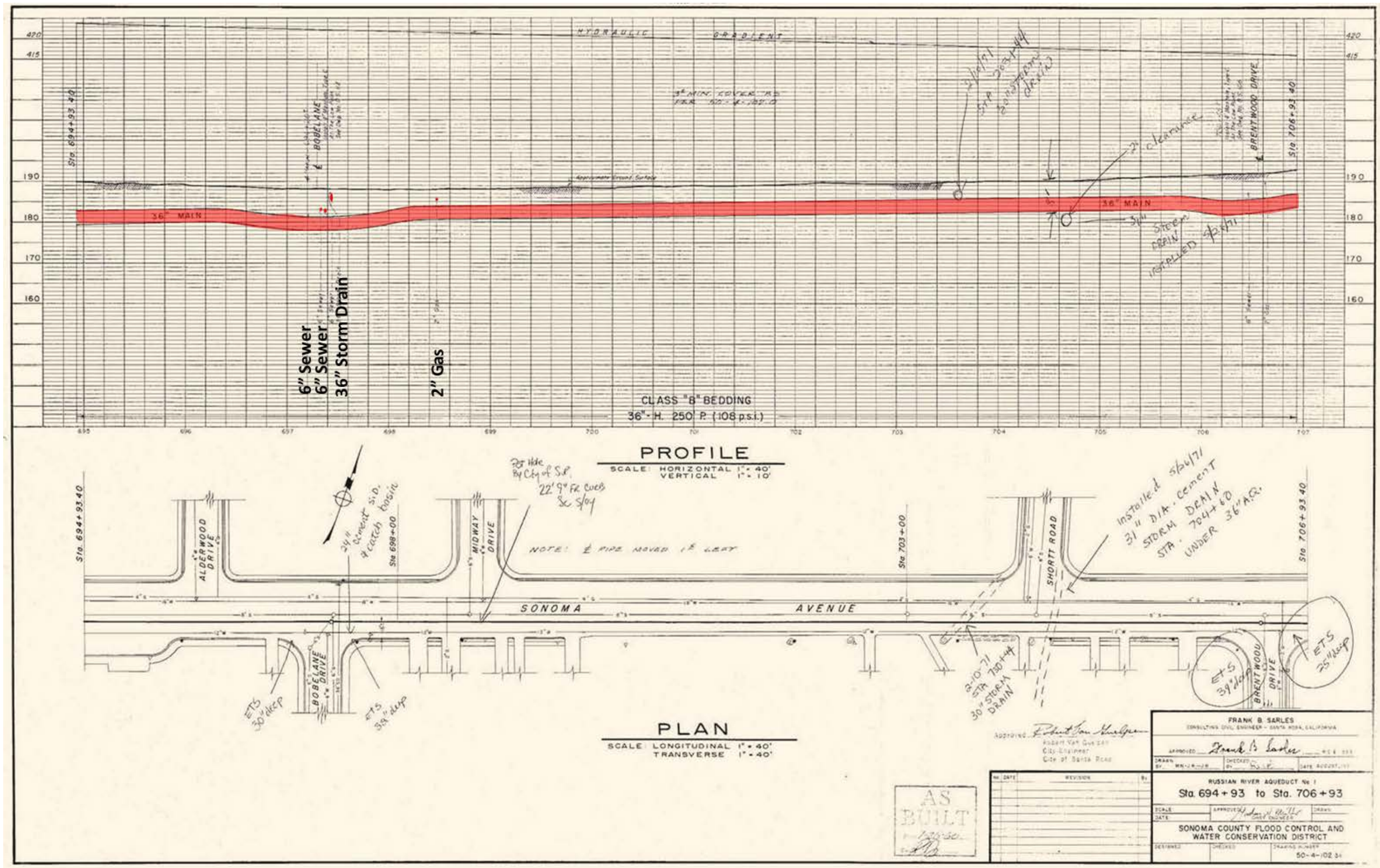

Plate 3: Plan and Profile of Existing Pipeline (3 of 3)