Seismic Response of Trestle-Supported Pipes at the Dumbarton Strait

John Eidinger[1], Luke Cheng[2], Stephanie Wong[3], Mark Ketchum[4]

ABSTRACT

The Bay Division Pipelines 1 and 2 are 60-inch and a 66-inch diameter pipelines that cross the Dumbarton Strait. These pipelines are two key links in the overall Hetch Hetchy Aqueduct owned by the San Francisco Public Utilities Commission. The pipelines are supported above ground on nineteen hundred wooden trestles, spanning across 19,000 feet of young bay muds; and go under the bay at two locations for a distance of about 3,400 feet. We are concerned how these pipes might perform during earthquakes that might occur during their remaining lifetime, considering the effects of local soil amplifications, the response of piles in the young bay mud, the stresses in the pipelines due to inertial as well as hydrodynamic loads. The long term effects of corrosion are considered, as well as a long term maintenance program that addresses ongoing wood deterioration, rust of the steel, and the ability of the pipe to withstand some overload. Considering all these factors, a long term maintenance program is developed that will keep the pipes reliable for their remaining service life.

Introduction

The SFPUC's Hetch Hetchy water transmission system was first put into service in 1934, and has continuously undergone improvements ever since. The modern Hetch Hetchy system includes components inherited by the SFPUC from the Spring Valley water Company. Today, the system is composed of a 167-mile long gravity-driven network of dams, reservoirs, tunnels, pump stations, aqueducts and pipelines that collect Tuolumne River runoff near Yosemite, as well as in local Bay Area watersheds in Alameda and San Mateo counties.

Construction of Bay Division Pipeline 1 (BDPL 1) at the Dumbarton Strait started on May 18, 1923. On September 12, 1925, BDPL 1 was placed in partial service, with water coming from the diversions in Sunol from the Alameda Creek watershed; full service started May 21, 1926. On March 3, 1930, San Francisco bought the Spring Valley Water Company. Construction of BDPL 2 started August 24, 1934 and was completed June 22, 1936. Today, the bridge, caisson and trestle portions of BDPL 1 and 2 in the Dumbarton Strait area extend from the Newark Valve house to Ravenswood Valve

[1] Principal, G&E Engineering Systems, Oakland, CA, eidinger@earthlink.net
[2] Senior Engineer, San Francisco Public Utilities Commission, lcheng@sfwater.org
[3] Project Engineer, San Francisco Public Utilities Commission, sawong@sfwater.org
[4] Principal, OPAC Engineers, San Francisco, mketchum@opacengineers.com

House, over a total distance of about 26,300 feet. Between the Newark and Ravenswood Valve Houses, BDPL 1 and 2 are supported as follows, from east to west (see Figures 1 and 2 for location and main features):

- Trestle section from Newark Valve House to Newark Slough, a distance of about 10,200 feet (East Trestle)
- Submarine section underneath Newark Slough, a distance of about 400 feet
- Trestle Section from Newark Slough to Dumbarton Valve House, a distance of about 6,300 feet (Center Trestle)
- Submarine section underneath San Francisco Bay, from Dumbarton Valve House to the Caisson, a distance of about 2,800 feet.
- Bridge section from Caisson to the west shore, a distance of about 3,900 feet.
- Trestle section from the west shore to the Ravenswood Valve House, a distance of about 2,700 feet.

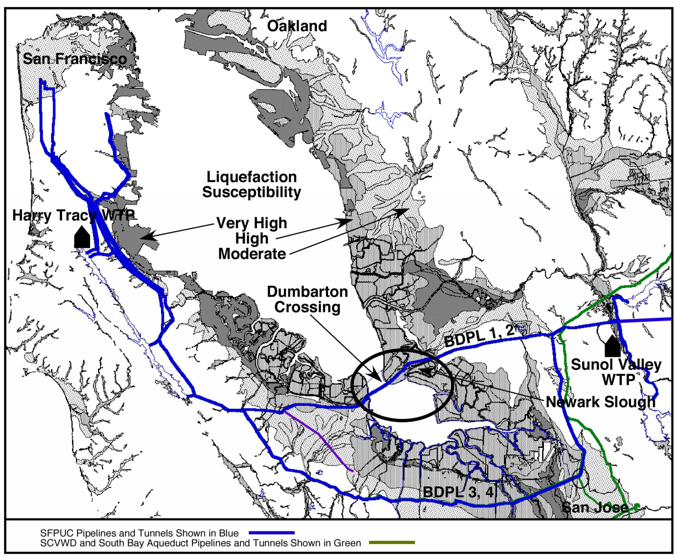

Figure 1. The Bay Division Pipelines 1 and 2 Crossing at Dumbarton

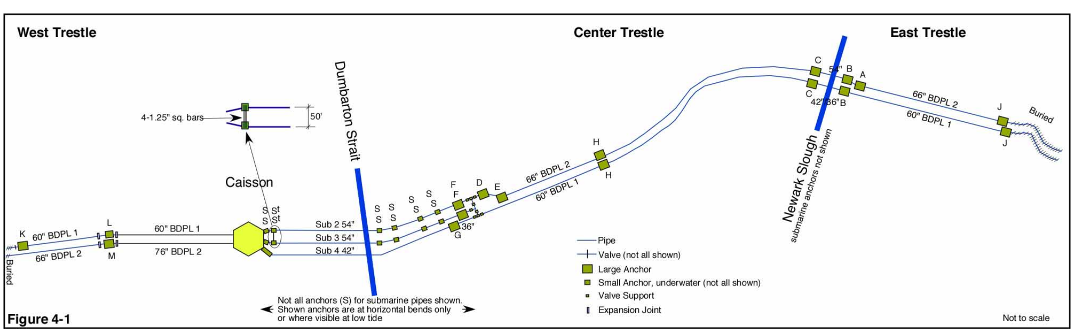

Figure 2. Plan (Not to Scale)

Figure 3 shows a portion of the BDPL 1 and 2 pipelines along the center trestle section, looking east. Figure 4 shows a typical wood trestle support, which is repeated about every 20 feet along the length of the above ground pipes. Figure 5 shows a common wood trestle support.

Figure 3. BDPL 1 and 2, Center Trestle, Looking Eastwards

Given the vintage of the pipes, we started out thinking that they would be severely overloaded under earthquake loading conditions. However, we were surprised that due to the style of construction, the pipes should be able to survive with essentially elastic stresses, at least if we can maintain the pipes in their original design configuration.

Seismic Evaluation of Above Ground Portion

As our long term plan is to construct a new pipeline parallel to these two, but completely under the bay, these BDPL 1 and 2 pipes need to be reliable only until such time that this new pipe can be built, possibly 10 to 15 years into the future. Accordingly, we decided to evaluate how these pipes might perform for an earthquake with a mean return interval of 125 years. We performed a series of deterministic and probabilistic analyses, considering attenuation of ground motion from either the nearby Hayward and San Andreas faults, and considering the soil amplification of these motions through the deep soil sediments under the Bay, as well as the thin layer of young bay muds at the surface. Figure 6 shows the resulting free field response spectra, with locations A1, A2/3 and A4 representing the western end of the pipes (Ravenswood), central locations and eastern end of the pipes (Newark). The "surface" spectra correspond to the free field motions at the top of the young bay mud layer, and the "within" motions represent the motions at the point of fixity within the soil for the wood piles that support the pipe.

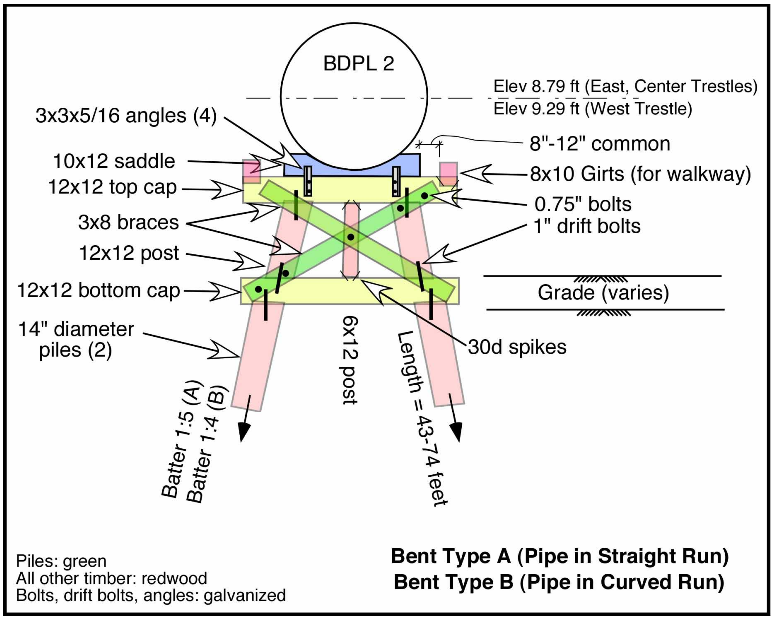

Figure 4. Typical Pipe Support

For the above ground portions of the pipeline, failure modes could be either stress overload of the pipe above the ground, or inertial overload of a series of wood trestle supports. We evaluated the non-linear capacity of the trestle supports with consideration of nonlinear pile-soil response. For example, for a transverse 30 kip load applied by the pipe to the trestle support, the loads in the piles are shown in Figure 7. As can be seen , the resulting stresses in the piles are not too high, always under 1 ksi or so. Since the pipe-to-trestle support connection uses a slider, it is essentially not possible for the pipe to generate forces more than 30 kips, and thus the trestle supports should perform well, as long as there is not much wood deterioration.

Figure 5. Typical Pipe Support

Given that the BDPL 2 pipe is able to slide over the wood, and given the nonlinear trestle support response, it was decided to evaluate pipe stresses and displacements using a completely nonlinear analysis. A set of six spectra-compatible time histories was developed, three representative of events on the San Andreas fault, and three on the Hayward fault. The time histories were adjusted to account for fault normal, fault parallel and vertical direction effects. Ten different analyses cases were run to consider the range of likely pipe-to-wood friction values, and the various time histories. Table 1 summarizes the results. The key findings are as follows: maximum longitudinal pipe stresses are under 9 ksi (elastic); and maximum movement of the pipe as it slides over the wood trestles is about 2 to 4 inches (acceptable). The results for BDPL 1 would be just as good, except that BDPL 1 was bolted to the trestle supports at many locations as part of a wood deterioration upgrade about 10 years ago; accordingly, it is planned to remove these bolts in order to improve the seismic capacity of the BDPL 1 pipe.

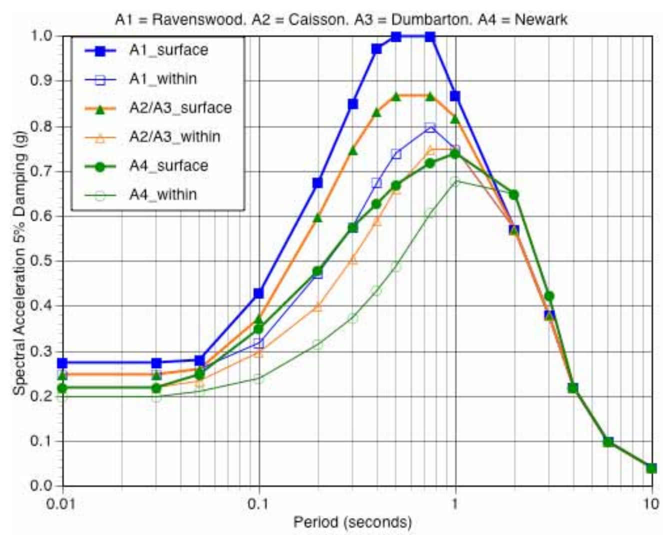

Figure 6. Horizontal Spectra, 125 Year Earthquake

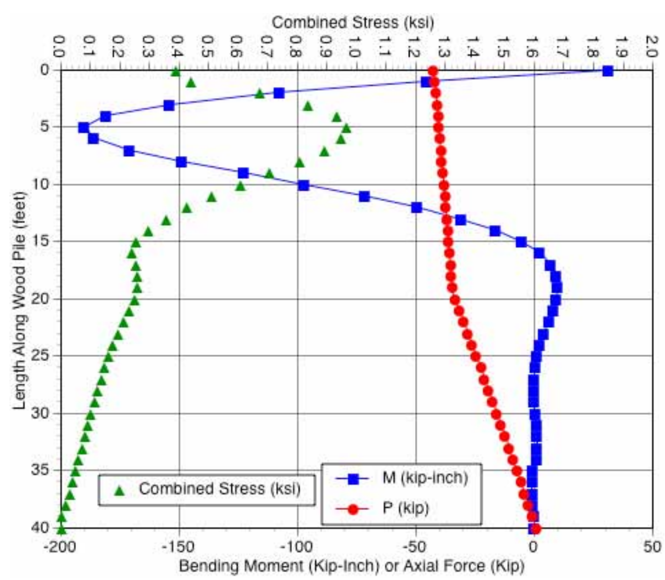

Figure 7. Pile Loads and Stress, Bent A Transverse Load of 30 Kips

Table 1. Summary of Nonlinear Analysis Cases

Analysis Case	Friction	Time History	Pipe Stress, ksi	Longitudinal Movement, inch	Transverse Movement, inch	Vertical Uplift, inch
P1.4	0.4	1	8.3	0.30	1.58	0.76
P1.5	0.4	2	7.7	0.23	1.56	0.67
P1.6	0.4	3	7.6	0.27	2.03	0.57
P1.7	0.4	J	8.3	0.32	1.50	0.54
P1.8	0.4	L	7.2	0.26	1.63	0.60
P1.9	0.4	Y	7.7	0.33	1.43	0.74
P2.1	0.2	1	8.4	0.36	2.22	0.76
P2.2	0.2	2	8.4	0.36	2.26	0.67
P2.3	0.2	3	8.8	0.32	4.10	0.57
P3.1	0.5	1	8.4	0.31	1.48	0.76
P3.2	0.5	2	8.0	0.26	1.66	0.67
P3.3	0.5	3	7.9	0.28	1.88	0.57

Seismic Evaluation of Below Water Portions

BDPL 1 and 2 cross the Newark Slough as two underwater pipes and Dumbarton Strait as three underwater pipes. Due to the soft soil conditions, construction of the pipes was a challenge. Figure 8 shows the lowering of BDPL 1 into a below grade trench under the water from a temporary work trestle. Figure 9 shows the profile of the installed pipe.

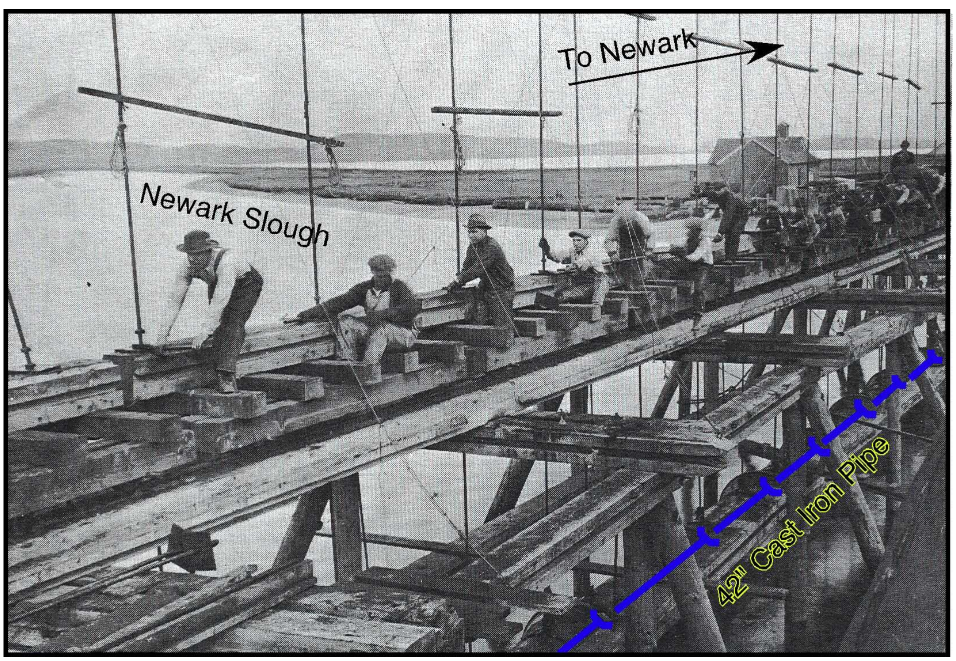

Figure 8. BDPL 1, Installation of 42" CI Pipe, 1924

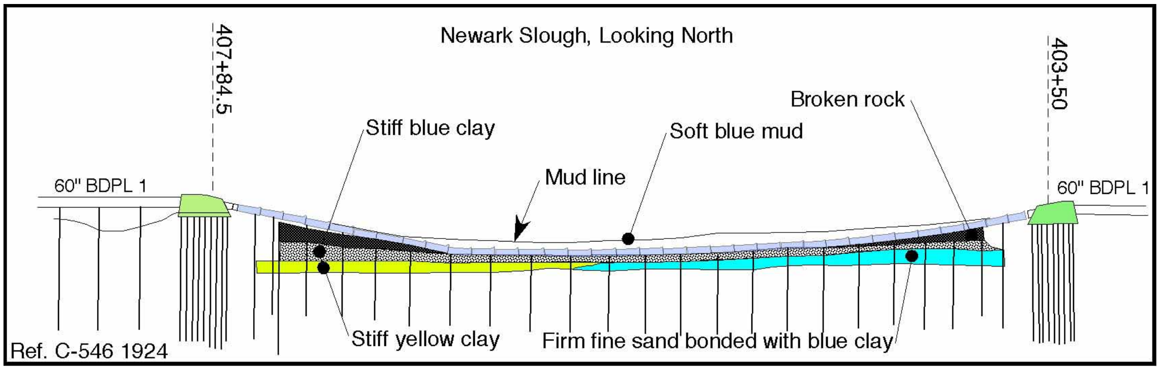

Figure 10. BDPL 1 at Newark Slough

We examined the style of installation of each of the five crossings. The BDPL 2 underwater crossings use continuous welded (or bolted flanged) pipe resting on piles underwater, while the BDPL 1 crossings use ball jointed pipes carefully laid into trenches. The pipes do not rest on the young bay muds. We quantified the stresses in the pipe due to ground shaking (they are small); the potential for flotation due to liquefaction (they can't); the possibility of underwater landslide (highly unlikely). For 4 of the 5 underwater crossings (2 at Newark Slough and 2 at Dumbarton Strait), the existing underwater construction appears to be highly reliable to withstand the 125-year earthquake. For the fifth pipe, available construction documentation was not entirely available, and a joint failure in the 1930s could not be explained (possibly dragged by a ship anchor, but possible due to other effects), so it could not be assured as being reliable. Due to hydraulic requirements, losing one of the five underwater pipelines in an earthquake would not be ideal, but the vast majority of the hydraulic capacity of the system would be retained in any case.

Hydrodynamic Loads

As reported by Schussler (1906), a time-varying hydrodynamic water pressure occurred in the Alameda pipeline in the 1906 earthquake. Schussler reported that a pressure relief valve, located at the east shore near the modern Dumbarton Valve House, repeatedly opened and closed during the 1906 earthquake. Today, the Alameda pipeline no longer exists, but the BDPL 1 and 2 pipelines are in the same alignment as the old Alameda pipeline.

While we do not have records as to what the pressure setting was for that valve to open, it would be reasonable to assume that it would have been set to open at a water pressure somewhat less than what would have been needed to yield the pipe in the hoop direction. The Alameda Pipeline was regularly exposed to a hydrostatic head of 185 feet and possibly sometimes subject to water hammer due to the several pump stations them in service along that pipeline. A review of old Spring Valley Water Company drawings indicates that the Alameda Pipeline was steel with wall thickness of 0.1495 inches (9 gage) near Niles reservoir, and 0.1793 inches (7 gage) near sea level. Assuming 7 gage wall thickness, then 185 feet of hydrostatic head results in a hoop stress of 8,000 psi.

The valve was located essentially at sea level. At the time of the 1906 earthquake, the pipe was flowing at its normal rate of about 16 MGD (3.5 ft/sec). Assuming a pipe length of seven miles, then the head loss at the Dumbarton Strait would have been about 1.6 feet/thousand feet (assuming C=100), or 59 feet, making the actual pressure in the pipe at the Dumbarton Valve House about 126 feet (54 psi). Assuming a pressure relief set point of about 108 psi, then the combined hydrostatic plus hydrodynamic pressure wave must have had multiple peaks of more than 108 psi as it passed by the pressure relief valve.

Thus, it is reasonable to assume that similar hydrodynamic pressure loads will occur in BDPL 1 and BDPL 2 in future earthquakes. The concern about hydrodynamic loads for BDPL 1 and 2 covers the following issues:

- Will the extra transient pressure cause the main pipe to fail in hoop tension resulting in a major break?
- Will the extra transient pressure cause sections of the pipe which have undergone loss of steel (corrosion, with resultant pitting) develop pin holes and small leaks?
- What are the transient forces on the pipe caused by the hydrodynamic loads that might induce larger reaction thrusts on the wood trestle piles and anchor blocks?

In order to answer these questions, one needs to estimate the actual hydrodynamic transient pressures due to the earthquake. This type of analysis is not typically addressed in earthquake engineering textbooks. Textbooks on transient loading in water pipelines are primarily concerned about effects due to rapid valve closure, and do not address earthquake loading. However, the earthquake loading case can be considered similar to the valve closing case, with the exception that instead of the activating event being the rapid closure of a valve, resulting in a single stoppage of water velocity, the activating event is the time-varying changes in velocity imposed on the pipeline by the earthquake.

For the transverse lateral and vertical directions, the time varying motion of the earthquake simply pushes the water within the pipe walls along with the pipe. Most commonly, the mass of the water is assumed to be rigidly fixed to the pipe walls. This is a reasonably assumption, as the bulk modulus of water is high enough such that it does not vibrate much relative to the pipe in the pipe's transverse directions.

For the longitudinal case, the axial movement of the pipe probably does not cause much force into the water, as the water behaves essentially independently of the pipe (albeit there are some friction and laminar flow forces between the pipe and water, but these are assumed to be negligible for purposes of earthquake movement of the water). The seismic inertial analyses previously described do not include the water mass in the direction of the pipe. However, wherever there is a change in direction of the pipeline, or a reducer in the pipeline, axial movement of the pipe will impart a corresponding axial movement of the water, setting up a hydrodynamic transient.

Given these issues, we developed a structural water model to quantify the hydrodynamic loading for a typical stretch of BDPL 2.

The BDPL 1 and 2 pipelines extend from the anchor blocks just west of the Newark Valves to the anchor block to the Ravenswood Valve House, a pipeline distance of about 26,459 feet. The sharpest "bend" along this reach of pipelines is near the Newark valves, where the pipes undergo a 42 degree bend over a distance of a few hundred feet. The average bend at the Newark Slough is about 7 degrees. The average bend under the Dumbarton Strait is about 3 degrees. There is a 90 degree bend where the pipes turn upward inside the Caisson, and again where they exit the Caisson and go onto the pipe bridge. There is a reverse bend in the pipelines in the Center trestle, being 52 degrees over a distance of 1,560 feet, and 18 degrees over a distance of 720 feet.

Given these distances and bends it was decided to model the BDPL 2 66-inch diameter pipe from the bend near the Newark Valve House to the 90 degree bend at the Caisson, a distance of about 20,000 feet. Intermediate bends and reducers were ignored.

The bulk modulus is a measure of water's compressibility, or simply the change in pressure that causes a change in volume. It can also be thought of in typical structural engineering terms as the "stiffness" of water along the length of the pipe as a pressure pulse is applied to it. A dynamic model of the water – pipe system was developed. The model includes water mass and water stiffness to match the desired sonic velocity of water of 2,825 feet/sec. The model was run using three different time history ground motions assuming one fixed end (90 degree bend) and one free end or two fixed ends. It was found that the peak forces always occur at the fixed end, and that the peak forces do not vary much between the fix-free and fix-fix assumptions. Thus, analyses used for pipe evalaution were run assuming two closed ends (representing a worst case condition of a straight pipe with ninety-degree bends at each end).

A total of eighteen different time histories were applied to the model. The time histories represent actually recorded ground motions as follows: Kobe KJM motion (1995 Kobe earthquake) in the fault normal (FN), fault parallel (FP) and vertical (V) directions; Lucerne LCN motions (1992 Landers earthquake), FN, FP and V; Corralitos (LCS) motions (1989 Loma Prieta earthquake), FN, FP, V; Capitola (CAP) motions (1989 Loma Prieta earthquake), FN, FP, V; Izmit (IZT) motions (1999 Kocaeli Turkey earthquake), FN, FP, V; Duzce (DZC) motions (1999 Duzce Turkey earthquake). In total, the eighteen time histories have PGA values ranging from 0.142g to 0.843g, and represent a range of higher frequency moderate energy spectra to lower frequency high energy spectra.

For design purposes, it is useful to normalize water hammer forces into pressures, and to establish a simple rule to predict the pressures given the input response spectra. We performed a best fit regression to correlate the PGA (or spectral acceleration) versus the computed peak hydrodynamic force. We found that the best correlation occurs using the long period (T=2.4 second) response spectra value. The following simplified model can be used to approximate the peak transient hydrodynamic pressure for the trestle supported pipe:

$$p_h = 853 * SA_{T=2.4\,\mathrm{sec};\xi=5\%}\text{, psi}$$

For example, if a response spectra has Spectral Acceleration = 0.1g at T=2.4 seconds and 5% damping, then the peak hydrodynamic pressure at an elbow would be 85.3 psi. The peak hydrodynamic pressure spatially along the pipe would be typically 50% to 80% of the peak value at a 90 degree bend.

When we add this hydrodynamic force to the seismic (plus thermal and dead weight) load cases, we find that it would challenge the BDPL 1 pipe only where its wall thickness has been degraded due to corrosion. In other words, some pin-hole leaks might occur due to the earthquake motions.

Conclusions

This paper describes the BDPL 1 and 2 pipelines from Newark to Ravenswood valve houses. The portion of the pipelines in the Caisson and on the Pipe Bridge are not covered in this paper. The following are the key observations and conclusions:

- A 125-year return period earthquake is used for evaluation the pipelines. This corresponds to a Peak Ground Acceleration of about 0.25g to 0.30g along the length of the pipelines. By ensuring that the pipes can withstand this level of earthquake, there is reasonable assurance that the pipes can be reliably kept in service for the next ten to fifteen years.
- Maximum longitudinal pipe stress for the wood-trestle supported BDPL 2 pipe is in the range of 8 to 10 ksi, assuming all supports are in reasonably good state of repair. This stress is well within allowables, and the pipe should remain in service. Concurrently, the pipe might slide up to a few inches side-to-side atop the wood trestle supports (acceptable) and up to an inch longitudinally (acceptable). Assuming little wood deterioration, the wood trestle supports should suffer little if any damage.
- Similar analyses have been performed for BDPL 1, assuming unbolted supports as were originally installed in 1924. The BDPL 1 stresses are somewhat lower than those in BDPL 2. The BDPL 1 stresses are low enough to not challenge the weak riveted girth joints. Maintenance work is needed to restore the current (2005) bolt configuration to the original (1924), to allow the pipe to slide on its wood supports.
- Hydrodynamic loading will possibly cause some pin hole leaks in BDPL 1.
- Four of the five underwater pipelines are shown to be reliable in earthquakes. The reliability of BDPL 1 under Dumbarton Strait remains uncertain.

Reference

Schussler, H., The Water Supply of San Francisco, California, Before, During and After the Earthquake of April 18, 1906, and the Subsequent Conflagration, July 23, 1906.