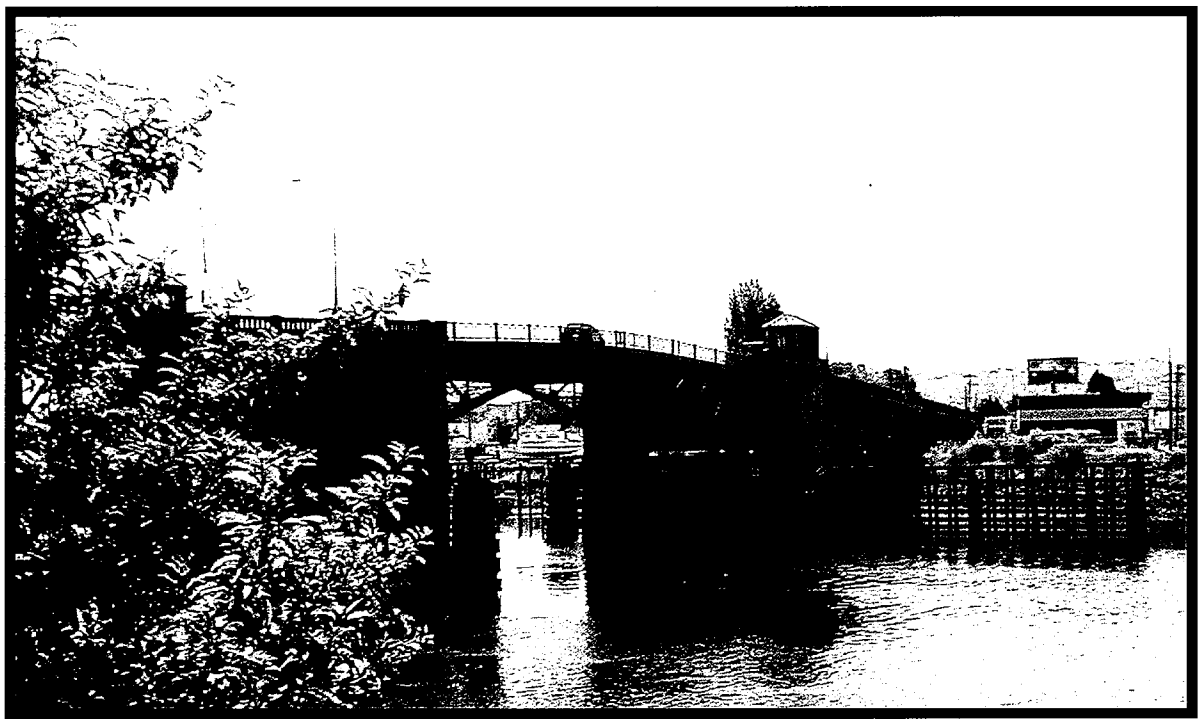
Final Report
Seismic Study of 14th Avenue South Bridge
Prepared for
King County Department of Transportation



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EXECUTIVE SUMMARY

The 14th Avenue South Bridge (also known as the South Park Bridge) is similar in many aspects to other aging bascule bridges that are located in difficult site soil conditions with high seismic hazard exposure. The unique aspect of the South Park Bridge is that it may be replaced in the near future, say within 10 to 20 years. Because of this short but uncertain remaining life of the bridge, a risk assessment and cost analysis for seismic retrofit was considered necessary. Based on these considerations, King County contracted with Imbsen and Associates, Inc. (Imbsen) from Sacramento, California to conduct this special investigation to establish appropriate seismic retrofit requirements that would provide adequate seismic safety and minimize the risk exposure to the public. In addition, Imbsen teamed with PanGEO Inc., a geotechnical firm, from Seattle, Washington to complete this investigation.

This report summarizes our findings and recommendations of a seismic vulnerability study and alternative seismic retrofit options for the existing South Park Bridge that crosses the Duwamish Waterway. The primary objective of the study is to assist King County in establishing an acceptable seismic retrofit strategy based on risk-cost-benefit considerations.

Description of the Bridge

The South Park Bridge as shown in Figure i was constructed during the 1930-1931 period. The bridge is composed of the following components:

1. A 190 foot main span – the Scherzer Rolling Lift double leaf bascule movable span resting on two concrete bascule piers;
2. Two steel truss approaches (86.5 feet and 77.25 feet) on each side of the main span;
3. Twelve concrete slab approaches spans (11 spans of 20 feet and 1 span of 18 feet-6 inches) on each side;
4. Retained fills (120 feet long) at each end.

The overall length of the structure is 1,285 feet including the retained fills. The roadway width is 38 feet with 6 foot wide sidewalks on each side. The roadway grade is 5 percent for the approach spans and 3.65 percent of the bascule span. The bascule span can be opened and closed with the mechanical and electrical equipment of the bridge. During the operation, the motors in the mechanical room drive the pinions and track girders to roll back and forth on the racks and the tracks, respectively. The operation is controlled by the bridge tender who resides in the north operator house.

Imbsen conducted a field investigation in July of 2001 to review the as-built condition of the bridge and its approaches. In addition, a follow-up field investigation was conducted after the February 28, 2001 Nisqually Earthquake on March 12, 2001 to assess the damage.

The description of the bridge, modification and repairs, existing conditions and post Nisqually Earthquake field investigation are presented in Chapter 1 of this report.

Seismic Evaluation Criteria

In this study, two levels of earthquakes are adopted for evaluation based on a 10% seismic risk assuming the bridge has 10 or 20 years of remaining service life. In addition, an earthquake level that has a 10% change exceedance in 50 years is considered when alternative retrofit schemes are investigated. The seismic evaluation criteria are presented in Chapter 2.

Response Spectrum Curves

Figure ii shows the response spectrum curves corresponding to the following three levels of earthquakes:

1. Level 1: Earthquake of 95 year return period (95 year earthquake); 10% change of exceedance in 10 years,
2. Level 2: Earthquake of 190 year return period (190 year earthquake): 10% chance of exceedance in 20 years,
3. Level 3: Earthquake of 475 year return period (475 year earthquake): 10% chance of exceedance in 50 years.

The site specific response spectrum curves of the first two levels of earthquakes were developed by our subconsultant PanGEO, Inc. They represent magnitudes of approximately 6 to 6.5 and 6.5 to 7 occurring in the Seattle area. The curve of a 475 year earthquake is obtained from AASHTO assuming a site coefficient of 1.2. This curve

represents an earthquake magnitude of approximately 7.0 to 7.5 occurring in the Seattle area with a corresponding Peak Ground Acceleration of 0.33g.

Seismic Performance Criteria

A "no collapse" criteria has been established for the structure. For the retrofit design, the seismic performance criteria of the retrofitted bridge is summarized in the following table:

Ground Motion at Site	"No Collapse" Level	Damage Level	Retrofit Strategy
Level 1 Earthquake "95 year earthquake"	Limited (Highway) / Limited (Shipping)	Repairable Damage	1
Level 2 Earthquake "190 year earthquake"	Closed (Highway)/ Closed (Shipping)	Major Damage (Replace Bridge)	1
Level 3 Earthquake "475 year earthquake"	Closed (Highway)/ Closed (Shipping)	Major Damage (Replace Bridge)	2

Where the "No Collapse" level of limited (highway)/limited (shipping) shall mean:

1. Closed to emergency vehicles immediately after the earthquake, open to emergency vehicles after inspection,
2. Closed to normal vehicles immediately after the earthquake, open to emergency vehicles after 2-3 months of repair,
3. No ability to raise and lower the bridge to permit passage of shipping immediately after the earthquake, able to raise and lower the bridge to permit passage of shipping after 2-3 months of repair.

The "No Collapse" level of closed (highway) / closed (shipping) shall mean:

1. Closed to emergency vehicles permanently,

2. Closed to normal vehicles permanently,
3. No ability to raise and lower the bridge to permit passage of shipping permanently.

Repairable damage level shall mean:

1. Damage to structural components can be repaired within the period of 2-3 months,
2. Damage to mechanical and electrical components can be repaired within the period of 2-3 months.

The level of major damage (replace bridge) shall mean:

1. Damage to some structural and mechanical components that can not be repaired or are not economical to be repaired,
2. A replacement bridge is needed to restore the functionality of the bridge.

Liquefaction Considerations

The liquefaction potential of the soil underlying the bridge alignment was analyzed by PanGEO, Inc. Their study concluded that liquefaction might occur in the top 15 feet of soil. For the evaluation of the bascule span, analyses are conducted for soil conditions with liquefaction and without liquefaction. See Appendix C, which contains the geotechnical report. Comparisons are made to this study and previous studies by Shannon and Wilson.

Seismic Evaluation

Response spectrum evaluations were conducted for the Level 1 earthquake (95 year earthquake) and the Level 2 earthquake (190 year earthquake) using GTSTRUDL on the structural segments of the bridge:

1. The bascule span has a very low probability of being in the opened position during an earthquake, therefore, analysis was conducted only in the closed position. A three dimensional model was used for the analysis with appropriate pile foundation spring constants.
2. The steel truss approach spans were modeled with the substructure elements. The superstructure was treated as concentrated masses.
3. The concrete approaches are separated with expansion joints every three spans. A three-dimensional model was used for the analysis of a typical three continuous span model.

The seismic evaluation is presented in detail in Chapters 3, 4 and 5 for the bascule span, steel truss approach spans and concrete approaches, respectively. A summary of the vulnerability assessment of the bridge subjected to different levels of earthquakes is shown in Table i. Appropriate ratios were used to scale the seismic forces due to the Level 2 earthquake (190 year earthquake) to obtain forces due to the Level 3 earthquake (475 year earthquake).

Seismic Retrofit Strategies

The following two alternative strategies of seismic retrofit were considered in this study:

Strategy Level I	The bridge will not collapse and will be repairable after the 95 year earthquake
	The bridge will not collapse under the 190 year earthquake.
Strategy Level II	The bridge will not collapse under the 475 year earthquake.

The strength and ductility concept was utilized in the retrofit strategies to prevent collapse. The strength approach requires retrofitting of inadequate bridge components to transfer the loads through the entire system (i.e., superstructure, bearings, pier and foundation, etc.). In addition, restrainer elements are proposed for the track girder and counterweight to prevent movement when the bascule bridge is in the closed position during a seismic event. Figure iii shows the locations of retrofit items considered. The details for the retrofit are presented in Chapter 6.

Retrofit Construction Cost

Retrofit costs for this report only cover costs for the structural portion of the project. Estimates have not been included for engineering designs, permitting, potential environmental mitigation, related electrical and mechanical work, and inspection expenses.

As mentioned above, two strategy levels for seismic retrofitting are considered and approximate structural costs were determined as follows:

	Total Cost (1)
Strategy Level I	$1,689,000
Strategy Level II	$2,876,000

Note 1: Includes 10% mobilization and 30% contingency for structural retrofit work only.

Recommendations

It is recommended that a seismic retrofit program of the South Park Bridge be considered. Either strategy I or II of seismic retrofit can be adopted depending on the funding available. A review of the Nisqually earthquake data should be considered in this evaluation process. After the retrofit, the bridge will be able to provide an additional 10 or 20 years of service life with seismic risk similar to the risk of newly designed bridges. A replacement program is also a reasonable option if funding is available.

Table i Damage State of Bridge Subject to Different Levels of Earthquakes

Item	Subitem	95 Years Earthquakes	190 Years Earthquakes	475 Years Earthquakes
Bascule Span Superstructure	Truss Members	o.k.	o.k.	o.k.
	Vertical Frames	Frame 0 severely damaged	Frame 0 may collapse	Frame 0 collapses
	Top Laterals	Some top laterals damaged	Some top laterals severely damaged	Some top laterals severely damaged
	Bottom Laterals	o.k.	o.k.	o.k.
	Counterweight	Needs restrainers	Needs restrainers	Needs restrainers
	Tracks	Needs restrainers	Needs restrainers	Needs restrainers
	Fascia Walls	Severely damaged	May collapse	Collapses
	Operator's House	o.k.	Damaged	Severely damaged
Bascule Span Substructure	Pile Foundations	o.k.	o.k.	Damaged
	Footings	o.k.	Damaged	Severely damaged
	East & West Pier Walls	Damaged	Severely damaged	May collapse
	North & South Pier Walls	o.k.	Damaged	Severely damaged
Truss Approach Superstructure	Truss Members	o.k.	o.k.	o.k.
	Vertical Frames	Severely damaged near bascule	May collapse near bascule	Collapses near bascule
	Top Laterals	o.k.	o.k.	o.k.
	Bottom Laterals	Not continuous	Not continuous	Not continuous
	Supports	Damaged	Severely damaged	May cause collapse
Truss Approach Substructure	Pier 1 Pile Foundation	Damaged	Severely damaged	Severely damaged
	Pier 1 Footing	Damaged	May be unstable and cause collapse	Unstable and causes collapse
	Pier 1 Columns	Damaged	Severely damaged	Collapses
	Pier 2 Pile Foundation	Damaged	Severely damaged	Severely damaged
	Pier 2 Footing	Damaged	May be unstable and cause collapsed	Unstable and causes collapse
	Pier 2 Columns	Damaged	Severely damaged	Collapses
Concrete Approach Superstructure	Cap Beams	o.k.	o.k.	o.k.
	Columns	Severely damaged	May collapse	Collapses
Concrete Approach Substructure	Pile Foundations	Damaged	Severely damaged	Severely damaged
	Footings	Damaged	Severely damaged	Severely damaged

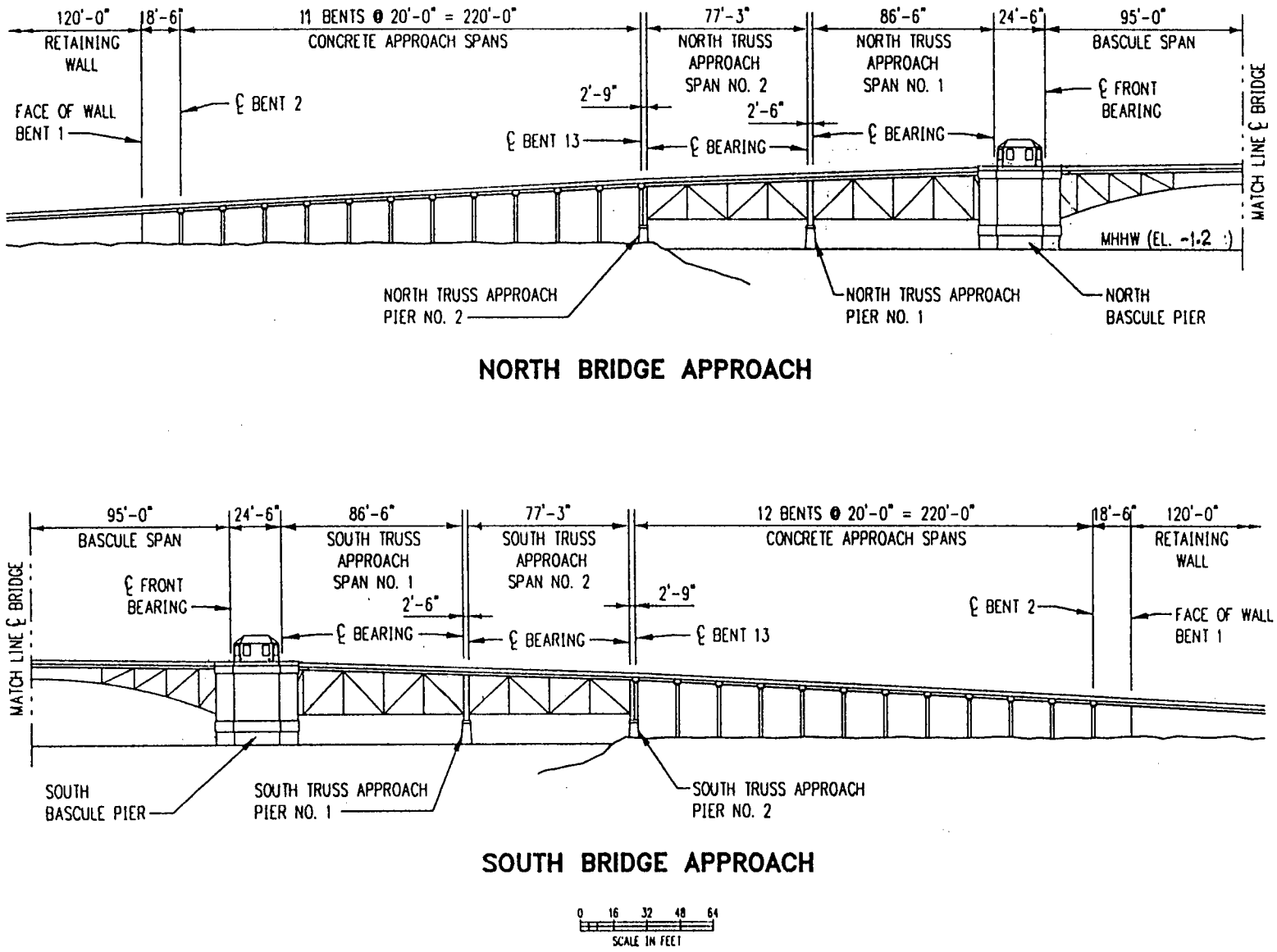


Figure i Bridge Elevation View

Period (sec)	Sa/g		
	AASHTO	190 Years	95 Years
0.0000	0.825	0.200	0.150
0.0525	0.825	0.250	0.188
0.0750	0.825	0.300	0.225
0.0975	0.825	0.350	0.263
0.1200	0.825	0.400	0.300
0.2000	0.825	0.400	0.300
0.2974	0.825	0.400	0.300
0.3326	0.825	0.400	0.300
0.4000	0.825	0.400	0.300
0.4300	0.825	0.400	0.300
0.5200	0.735	0.400	0.300
0.6000	0.668	0.347	0.260
0.8000	0.551	0.260	0.195
1.0000	0.475	0.210	0.158
1.2500	0.409	0.166	0.125
1.5000	0.363	0.139	0.104
2.0000	0.299	0.100	0.075
3.0000	0.228	0.046	0.035
5.0000	0.162	0.017	0.013

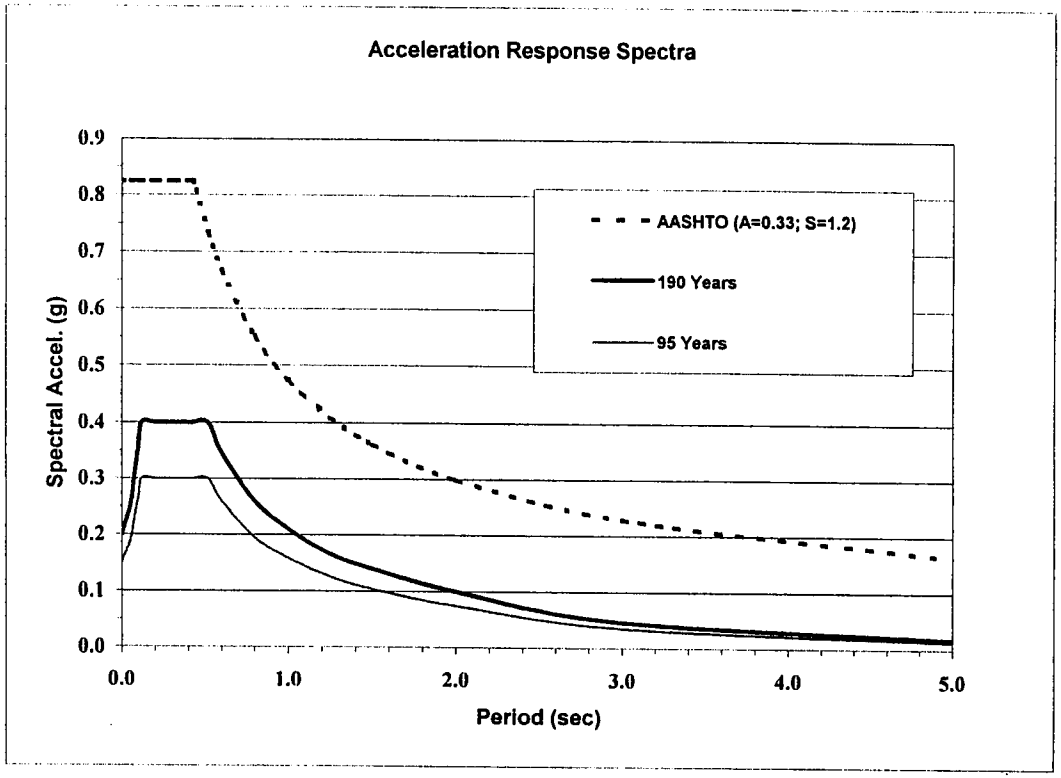


Figure ii Acceleration Response Spectra

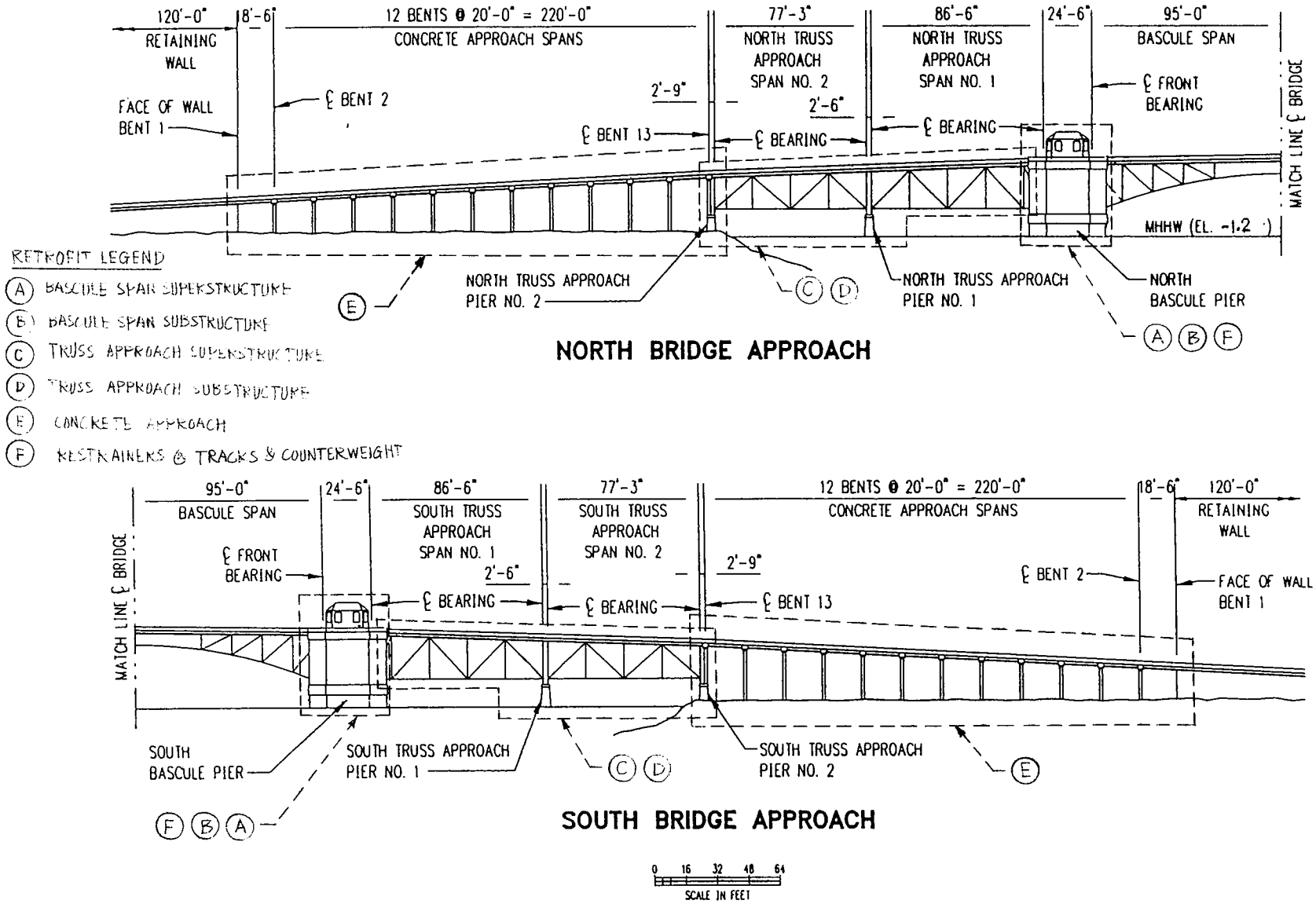


Figure iii Locations of Retrofit Items

1. INTRODUCTION

This report summarizes findings and recommendations of a seismic vulnerability study and the alternative seismic retrofit options for the existing 14th/16th Avenue South Bridge (also known as the South Park Bridge) that crosses the Duwamish Waterway (see Figure 1.1 for bridge vicinity map). The primary objective of the study is to assist King County in establishing an acceptable seismic retrofit strategy based on risk-cost-benefit considerations.

1.1 Scope of Work

The scope of work for this study includes:

1. Review existing literature and conduct field investigations.
2. Establish a seismic hazard environment at project site and develop evaluation criteria.
3. Perform geotechnical seismic evaluations and foundation analyses.
4. Perform seismic vulnerability evaluations of the existing bridge.
5. Develop alternative seismic retrofit strategies and construction cost estimates
6. Perform cost, benefit and risk assessments
7. Prepare preliminary recommendations and a draft report for King County to review and comment on. The draft report was dated April 2, 2001.
8. Prepare the final report and recommendations. Comments from King County were received on June 29, 2001 and incorporated in this final report.

9. Present the final report with recommendations to County, the City of Tukwila, and the City of Seattle Officials, and provide services to advice the County Council if needed. Presentation occurred on May 17, 2001.
10. Provide project administration and coordination throughout the project.

1.2 Background

The 14th/16th Avenue South Bridge crosses the Duwamish Waterway in the South Park area of the Seattle metropolitan area. The four-lane bridge presently accommodates an average daily traffic volume of approximately 24,000 vehicles. The speed limit for the bridge is 35 miles per hour. Maximum vertical navigational clearance for the bridge is limited to 32 feet at Mean Higher High Water (MHHW) level. On average there are three to five bridge openings per day to accommodate waterway traffic.

Inspection and maintenance reports reveal that the bridge is obsolete for traffic safety, functionality, and maintenance perspectives as well as deficient for seismic considerations. Currently, effort is underway for an environmental impact assessment for a new replacement bridge. However, the schedule for the completion of the new bridge has not been finalized. Therefore, the existing bridge may remain in service for many years. During this period of time, the seismic vulnerability of the existing bridge becomes a potential public safety issue. On the other hand, it may not be to the best interest of the public to invest and upgrade the seismic resistance of the existing 70 year old bridge to the current AASHTO standards for new bridges for no collapse. The dilemma is where is the balance point between seismic safety and judicious public policy.

Based on these considerations, King County contracted with Imbsen and Associates, Inc. (IMBSEN) to conduct this special investigation to establish the appropriate seismic retrofit requirement that would provide adequate seismic safety and minimize the risk exposure to the public. In addition, PanGEO, Inc., provided geotechnical services as a subconsultant to IMBSEN.

1.3 Description of the Bridge

The existing 14th/16th Avenue South Bridge as shown in Figure 1.2 and Photo 1.1 was constructed during the 1930-1931 period. The bridge is composed of the following components:

1. A 190 foot main span - the Scherzer Rolling Lift double leaf bascule movable span resting on two concrete bascule piers;
2. Two steel truss approach spans (86.5 feet and 77.25 feet) on each side of the main span;
3. Twelve concrete slab approach spans (11 spans of 20 feet and 1 span of 18 feet-6 inches) on each side; and
4. Retained fills (120 feet long) at each end.

The overall length of the structure is 1,285 feet including the retained fills. The roadway width is 38 feet with 6 foot wide sidewalks on each side. The roadway grade is 5 percent for the approach spans and 3.65 percent for the bascule span.

Bascule Span

Photo 1.2 is a side view of the bascule span. Photo 1.3 shows the roadway and sidewalk of the bascule span. Photos 1.4 and 1.5 are underneath views of the bascule span during an opening operation. Figures 1.3 and 1.4 show half of the bascule span with a portion of the steel truss approach. The double leaf bascule movable span has a center-to-center distance between the front bearings of 190 feet with length of roll equal to 18 foot for each leaf. The depth of each leaf varies from 23 feet at the front bearing to 6 feet - 5 inches at the center break. Each leaf supports 119 feet and 6 inches of roadway with a 24 feet - 6 inches section extending from the front bearing to the anchorage girder, and a 95-foot cantilevered truss section extending from the front bearing to the center break. The truss section is divided into six panels of 15 feet - 5 inches long.

Each cantilevered truss portion of the movable span is balanced by a concrete and steel counterweight of approximately 940,000 pounds. Trusses are of riveted steel framing common to the era of construction with center-to-center spacing between trusses of 36 feet. The floor system consists of open steel grating, steel channel deck joists, and concrete sidewalks supported on steel stringers and floor beams. Barrier posts and railings are steel.

Frame 0 is the vertical frame above the front bearing, which transfers the gravity of the superstructure to the bearings as shown in Figures 1.3 to 1.5. This frame lacks a lateral load path to transfer the lateral earthquake force from the deck level down to the bearings.

The substructure of the bascule span consists of two concrete bascule piers founded on timber piles. Figures 1.6 to 1.10 show the bascule pier and the pile foundation. The bascule pier is mainly composed of two 11 foot thick unreinforced concrete walls (or blocks), one 3 foot thick reinforced concrete wall and one 4 foot thick reinforced concrete wall resting on a 6 foot thick footing supported by 315, 12 inch diameter timber piles. There is a 15-foot thick concrete seal underneath the footing. The bascule pier also supports the steel truss approach and houses the operating machinery, electrical equipment and operator control room.

There are two operator houses on the bridge as shown in Photo 1.1. The bridge tender and the control center reside in the north operator house. The south operator house is not in use anymore. Levels one and two of the operator house are enclosed by 12 inch reinforced concrete walls. The operator level is enclosed by brick walls. The fascia wall on the opposite side of the operator house is a 12 inch lightly reinforced concrete wall. This wall is approximately 24 feet high and is not braced at the top.

Steel Truss Approaches

The steel truss approach is shown in Photos 1.6 to 1.8 and Figure 1.11. The steel truss approach span adjacent to the bascule span has a length of 86 feet - 6 inches. The truss is divided into four panels of 19 feet - 2 inches long and one panel of 9 feet - 8 inches long. The steel truss approach span, adjacent to the concrete approach spans, has a length of 77 feet - 3 inches and is divided into four panels of 19 feet - 3¾ inches. The trusses are of riveted steel construction with center-to-center spacing between the trusses of 42 feet - 10 inches. The floor system consists of a concrete deck and sidewalks supported on steel stringers and floor beams. The barriers are concrete.

The superstructure of the steel truss spans are supported by two concrete piers (Piers 1 and 2) founded on timber piles as well as by the bascule pier. Photos 1.7 and 1.8 and Figures 1.12 to 1.15 depict these two piers. Pier 1 consists of two unreinforced concrete columns connected by a cap and a spandrel beam. Each column seats on a 3.5 foot thick footing supported by 27, 12 inch diameter timber piles. There is a 5-foot thick concrete seal under the footing. Pier 2 is composed of three unreinforced concrete columns connected by a cap and a 1-foot thick reinforced concrete wall. A 2.5-foot thick footing supports each column. The outside footing is founded on 13, 12 inch diameter timber piles. The middle footing is supported by 7 timber piles. There is no seal under these footings. Pier 2 also supports 3 concrete columns of the concrete approach

Concrete Approaches

The concrete approaches are illustrated in Photos 1.9 and 1.10 and Figure 1.16. Each concrete slab approach has twelve spans with a typical span length of 20 feet and an overall length of 240 feet - 7 inches. An expansion joint is constructed, every three spans, in the slab above the cap beam. The reinforced cap beam has a cross section of 36 inches by 36 inches supported by 3 concrete columns. A typical column has a section of 22 inches by 22 inches. It is reinforced with eight longitudinal ¾ in square bars and ¼ in square tie bars spaced at 8 inches on center. Each column is supported by a 2 foot - 4 inch thick concrete footing founded on 4, 12 inch diameter timber piles.

Retained Fills

Retained fills of 120 foot long at the end of each concrete approach are depicted in Photo 1.11.

Mechanical and Electrical Components

The bascule span can be opened and closed with the mechanical and electrical equipment of the bridge. During the operation, the motors on the mechanical floor (see Photo 1.12) drive the pinions (see Photo 1.13) and the track girders (see Photo 1.14) to roll back and forth on the racks and the tracks, respectively. The operation is controlled by the bridge tender residing on the operator level in the north operator house (see Photo 1.15).

The bridge is also equipped with traffic signals and traffic gates in the approaches for traffic control during the operation of the bascule span.

1.4 Modifications and Repairs

The 16th Avenue South Bridge was opened to traffic in March 1931 with a timber and asphalt plank deck on the bascule span. Over the year, several modifications and repairs were performed. Those modifications that are important to the seismic evaluation are summarized below:

1. Asphalt surfacing was added in 1944;
2. Open steel grating and concrete sidewalks were installed in 1950 to replace the timber and asphalt plank deck;
3. Repairs were made in 1957 to Pier 2 of the south approach and additional pier repairs were performed in 1958 and 1970. These repairs included patching of spalled concrete areas.
4. Some footings of the concrete approach were underpinned in 1970's.

5. The bascule piers were transversely post-tensioned in 1982 to limit further outward movement of pier walls due to cracking of concrete near bascule rack supports. Prior to this, lugs on the track supports were being ground down to allow segmental track girders to move without binding on the track supports.
6. The timber fender for each bascule pier was also repaired in 1982.
7. The bascule center lock device was modified in 1985 to reduce bouncing of the bascule leafs due to live loading.
8. The bridge was last painted in 1977.
9. Four expansion joints of the approach spans were repaired by footer in 1994.
10. Seismic repairs following Nisqually Earthquake done in March 2001 – crack injection, approach shoring and guide track alignment.

1.5 Existing Conditions

The superstructures of the bascule span and steel truss approaches are in good condition. However, severe cracks and deterioration exist in the concrete portion of the bridge. Deterioration of foundation piles in the concrete approaches was found and foundation underpinning was carried out in 1970's. Photos 1.16 to 1.19 show the cracks and deterioration of the north and south bascule piers. Vertical, horizontal and diagonal cracks and severe concrete spalling exist in the concrete walls of both bascule piers. Cracks, concrete spalling and deterioration also exist in Pier 1 and Pier 2. A detail description of the existing condition of the bascule piers, Pier 1 and Pier 2 is given in state references six and fifteen. Photo 1.20 shows cracks in the fascia wall on the opposite side of the operator house. The concrete approaches are in poor condition and in continued repair. Cracks, concrete spalling, exposured reinforcement and deterioration can be seen

in the concrete slab, cap beams and columns. Photo 1.21 depicts cracks in a cap beam and a column. Concrete spalling in the slab and the cap beam can also be seen in the photo. Photo 1.22 shows concrete spalling and exposure of reinforcement in a longitudinal beam. Photo 1.23 illustrates concrete spalling, cracks and repair in the slab and the column. Photo 1.24 shows concrete spalling and deterioration in the slab and cracks in the retaining wall. Photo 1.25 shows misalignment of the concrete railing due to relative movement of the two adjacent approach spans. The timber piles of Pier 2 of the steel truss approach are subjected to wet and dry cycles when water elevation changes from Mean Lower Low Water (MLLW) level of -12.98 feet to MHHW level of –1.2 feet, which may cause severe deterioration to develop in the piles over the years.

1.6 Post Earthquake Field Investigation

On February 28, 2001 the Nisqually Earthquake measuring 6.8 magnitude occurred 30 miles southwest of Seattle, Washington. Initial field investigations by Mr. Paul Grant, IMBSEN's geotechnical subconsultant, indicated that the bridge sustained minor structural damage during the earthquake as follows:

- Pounding (spalling) of concrete railing at the concrete and steel truss approach (south approaches)
- Incipient plastic hinging at top and to a lesser degree bottoms of concrete columns on the south concrete approaches (Bents 5 through 12)
- South abutment settled approximately 1 inch
- Liquefaction occurred (sand boil) adjacent to column 3E on south concrete approaches.

The estimated ground motion at the bridge site was between 0.10g to 0.15g at the surface. The Mr. Grant's field investigation report is attached in Appendix D. The date of observations were February 28, March 2 and March 5, 2001.

A post earthquake field investigation at the bridge site was conducted by Mr. Robert Schamber and Dr. Majid Sarraf of IMBSEN and Mr. Paul Grant on March 12, 2001. These observations were noted:

Mechanical Components

Observations and measurements taken of the pinion and rack indicate the bascule spans shifted during the earthquake.

- Northeast pinion is offset approximately ¼ inch from the rack (see Photo 1.26). The offset becomes the largest as the pinion reaches near the end of the rack. This was evident as the bridge was opening and closing. Offset may have been present prior to the earthquake.
- Distance between the counterweight edge and steel rack frame measured as 38 inches northwest side and 37 inches northeast side.
- Gears appear to be properly aligned in the machinery room where the motors, gears and shafts are located (see Photo 1.27).
- Southwest pinion is offset approximately 1 inch from the rack (see Photo 1.28). Offset has been magnified during the earthquake.
- Distance between the counterweight edge and steel rack frame measured as 36-1/2 inches southwest side and 36-1/2 inches southeast side.

South Approach

The majority of damage was observed on the south approach concrete spans.

- Impacting occurred between the steel and concrete approaches as observed on the concrete railings (see Photo 1.29). This appears to be due to relative longitudinal movement.
- Settlement and possibly lateral movement occurred in the south approach fill, witnessed by depression of the roadway surface (locally about 2") and the formation of transverse tension cracks in the pavement with openings less than 1" (See Photo 1.30).
- Hairline cracks observed at the top of columns indicating beginning of plastic hinging. King County is currently monitoring the crack openings (see Photo 1.31). Cracks were observed around the perimeter of the top of the column on many bent columns.
- Shorter columns adjacent the steel approaches exhibit the greatest crack damage and should be repaired (see Photo 1.32). Cracks at the bottom were more pronounced than those at the top. Additionally, there was moderate tilting of the columns towards the north direction which confirms longitudinal movement of the spans and formation of plastic hinges in the columns.
- Truss bearings at Pier 1 indicate bridge moved transversely during the earthquake.

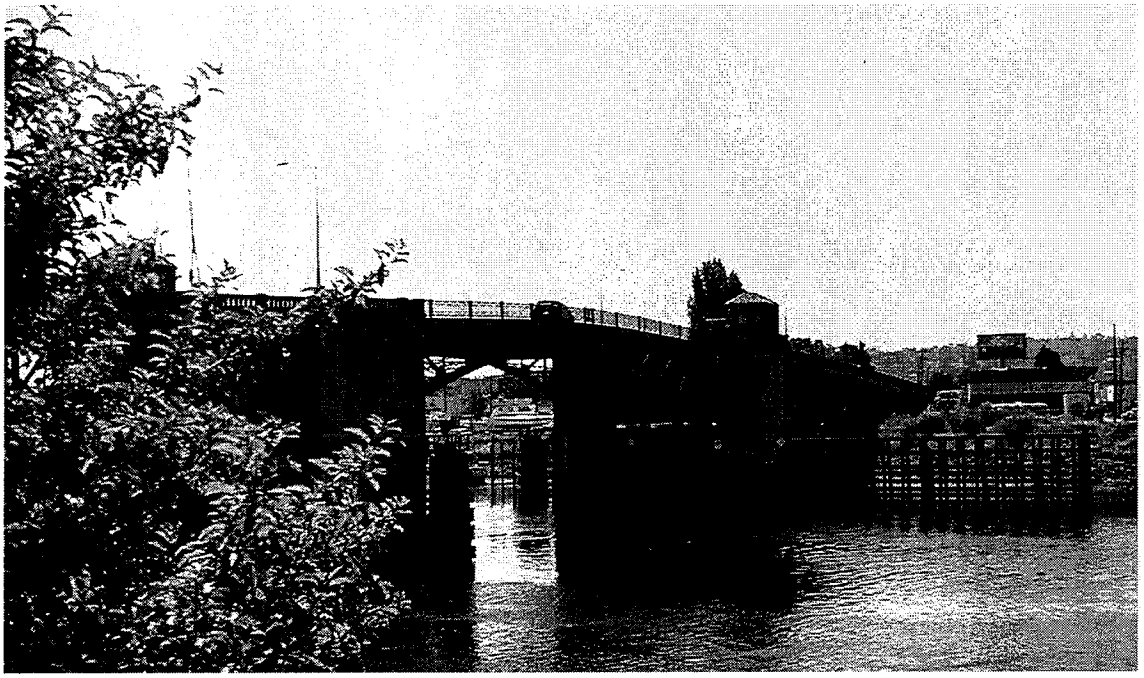


Photo 1.1 – Side View of Bridge

Photo 1.2 – Side View of Bascule Span

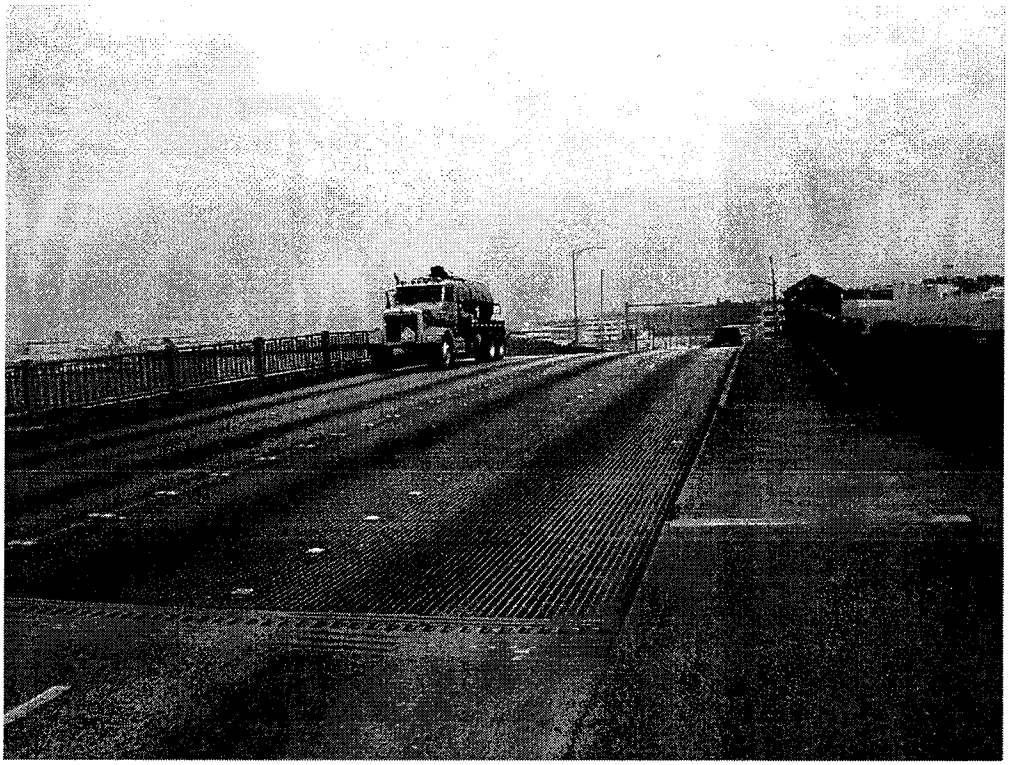


Photo 1.3 – View of Roadway and Side Walk of Bascule Span

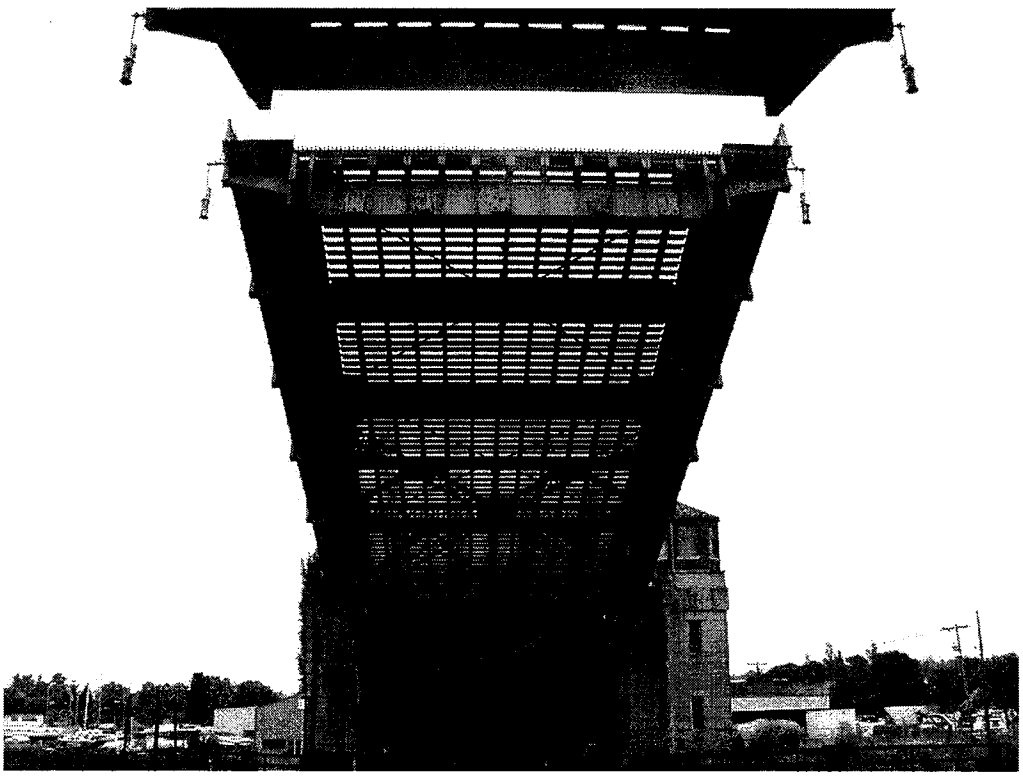


Photo 1.4 – Underneath View of Bascule Span During Opening Position

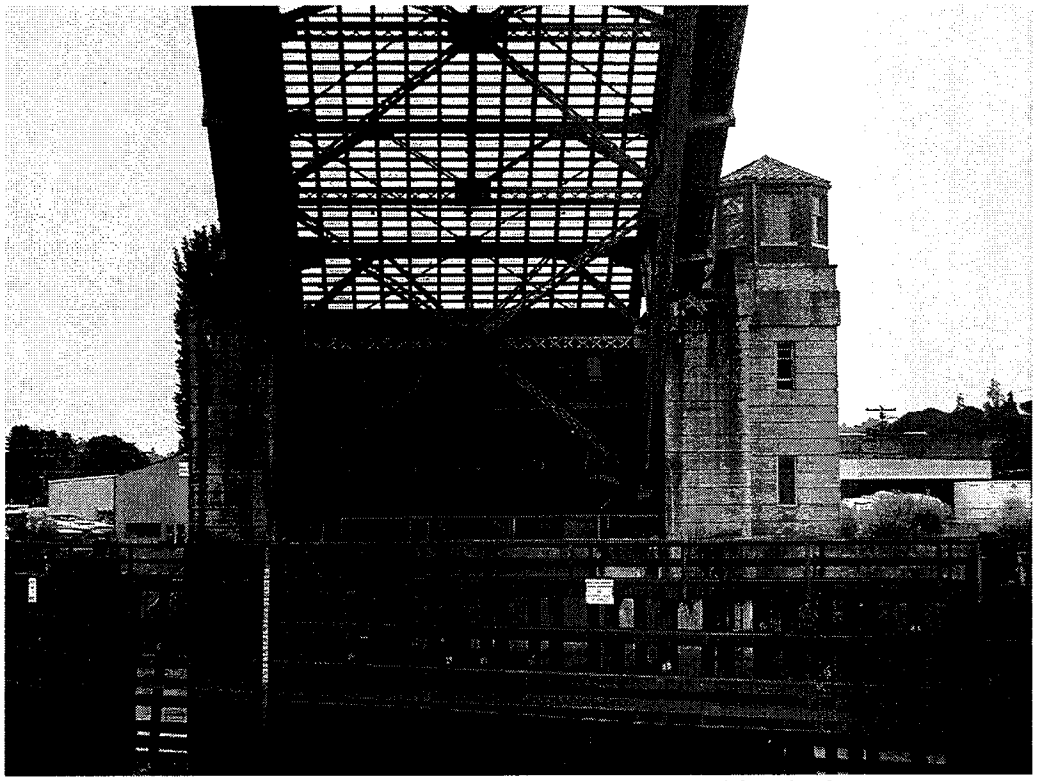


Photo 1.5 – Close View of Bascule when Bridge is Open

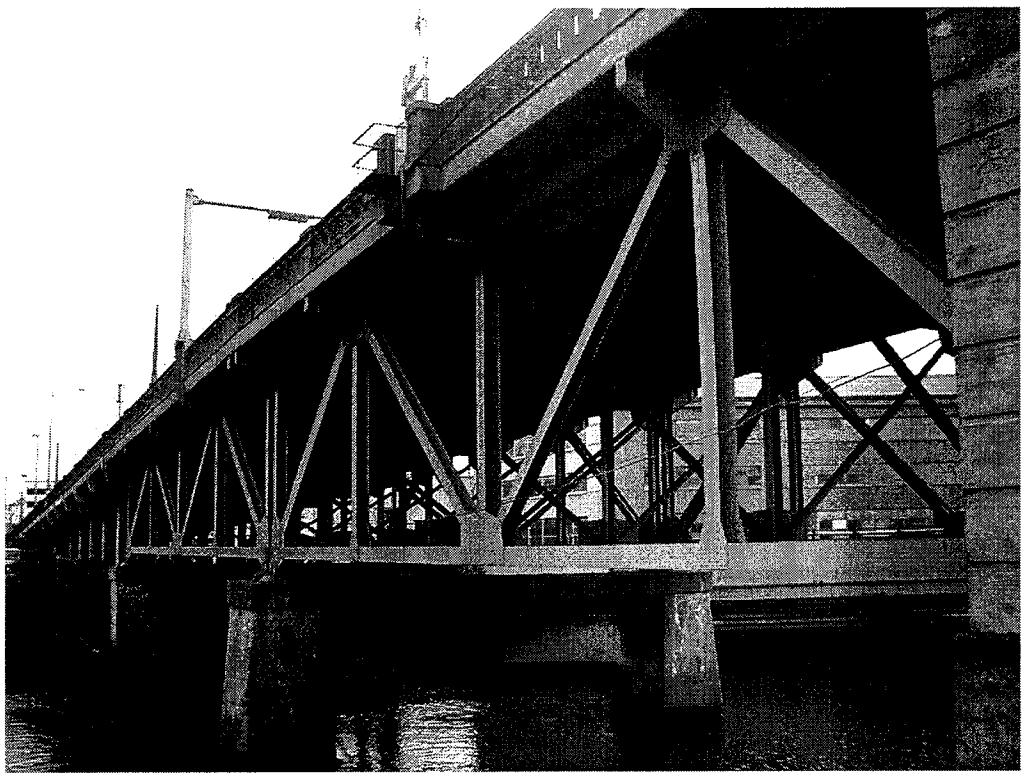


Photo 1.6 – Side View of Deck Truss Approach

Photo 1.7 – Pier 1 of Deck Truss Approach

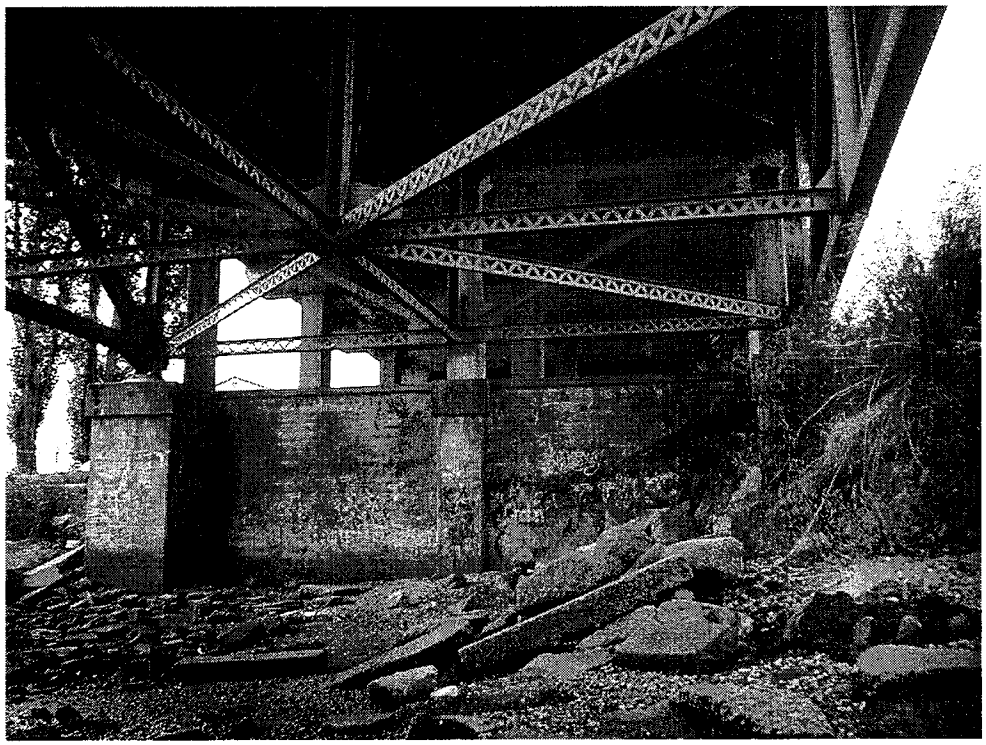


Photo 1.8 – Pier 2 of Deck Truss Approach

Photo 1.9 – Side View of Concrete Approach

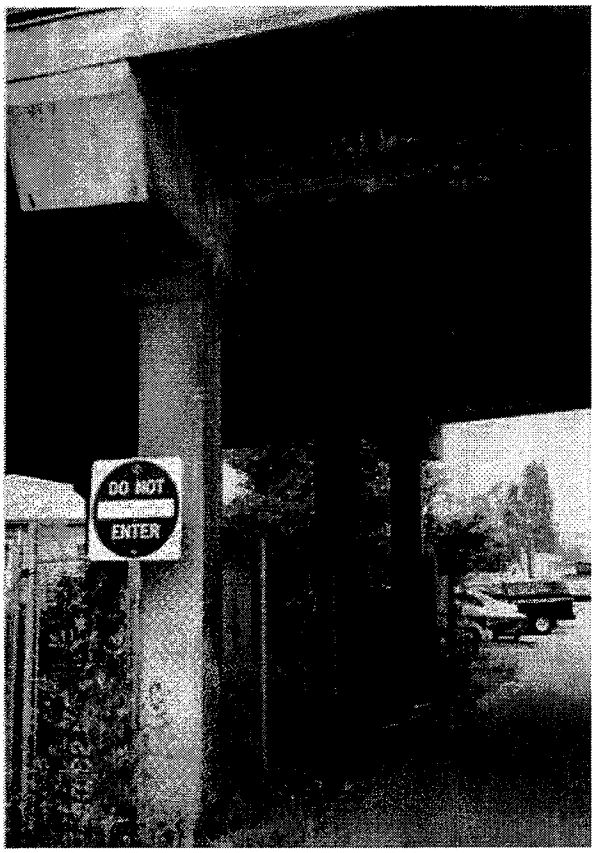


Photo 1.10 – Typical Bent of Concrete Approach

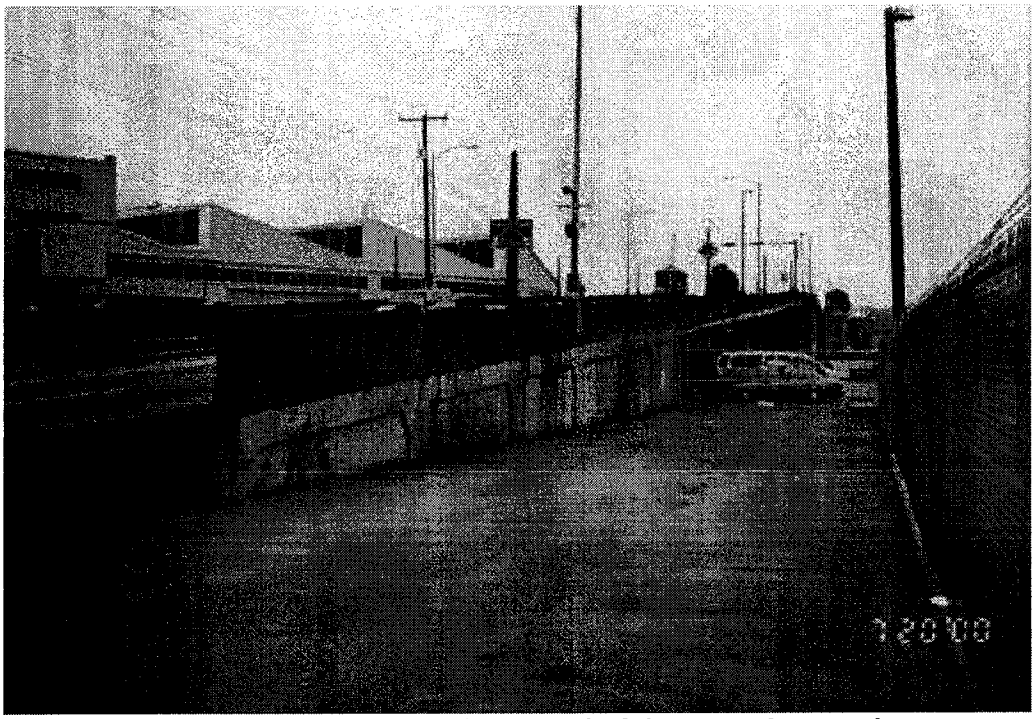


Photo 1.11 – Retained Fills at End of Concrete Approach

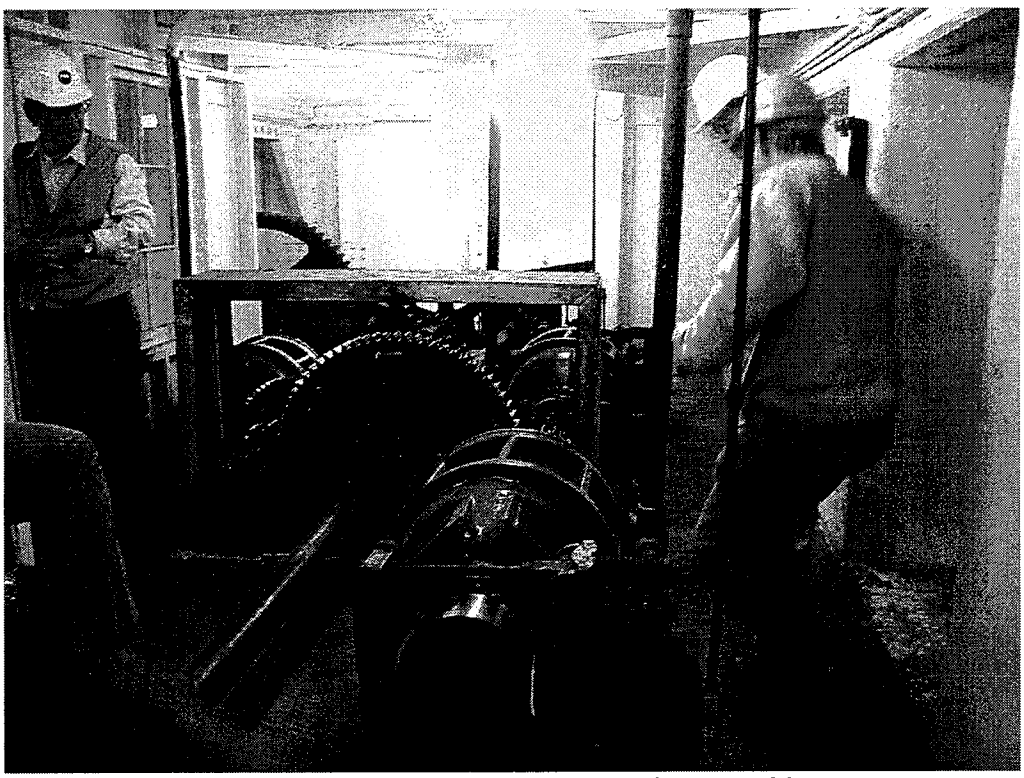


Photo 1.12 – Motors and Gears on Floor Machinery

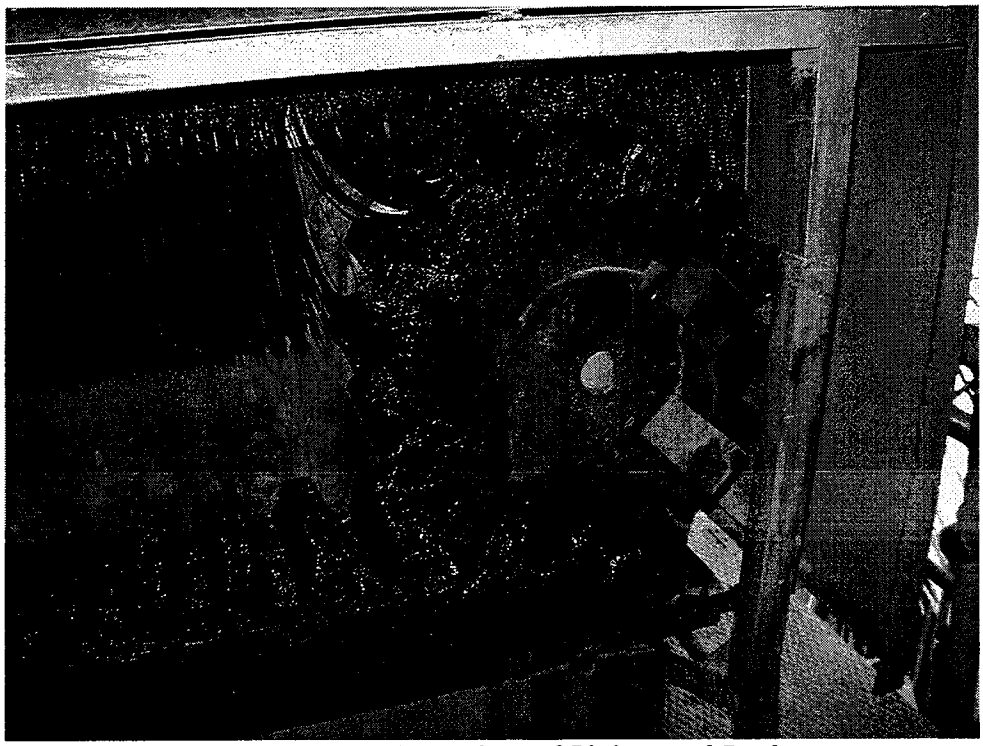


Photo 1.13 – Close View of Pinion and Rack

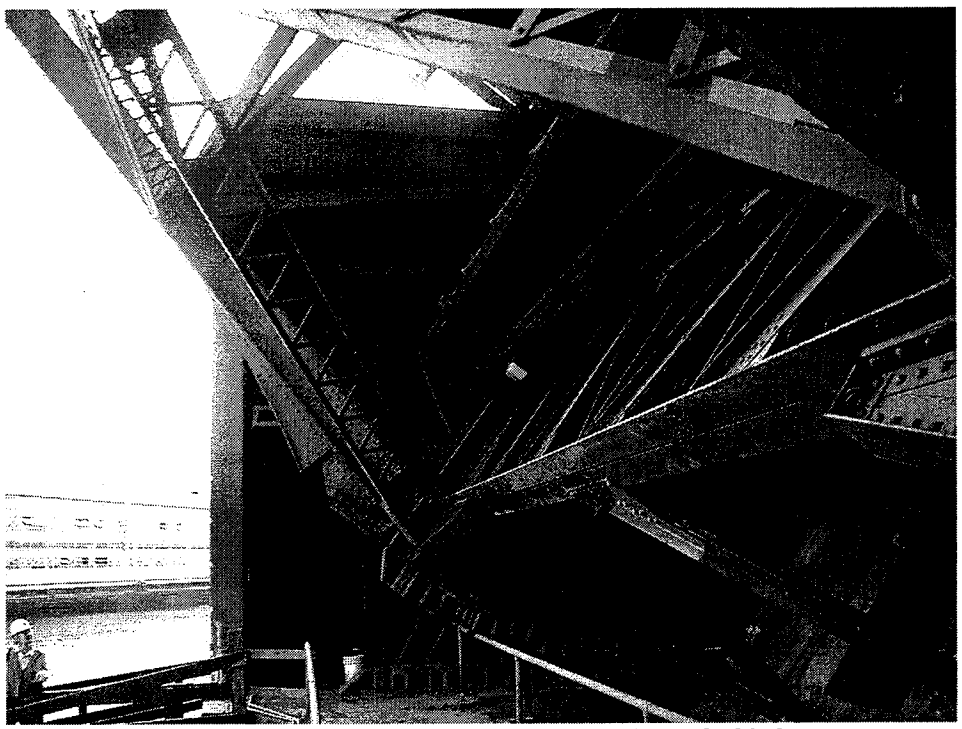


Photo 1.14 – Side View of Track and Track Girder

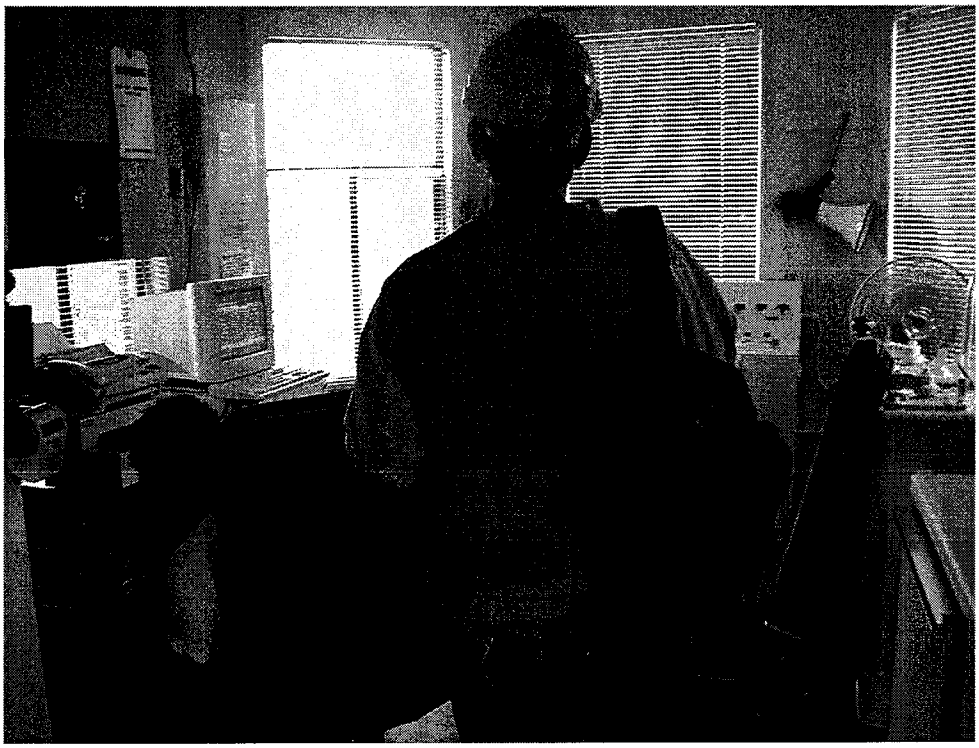


Photo 1.15 – Operator Floor

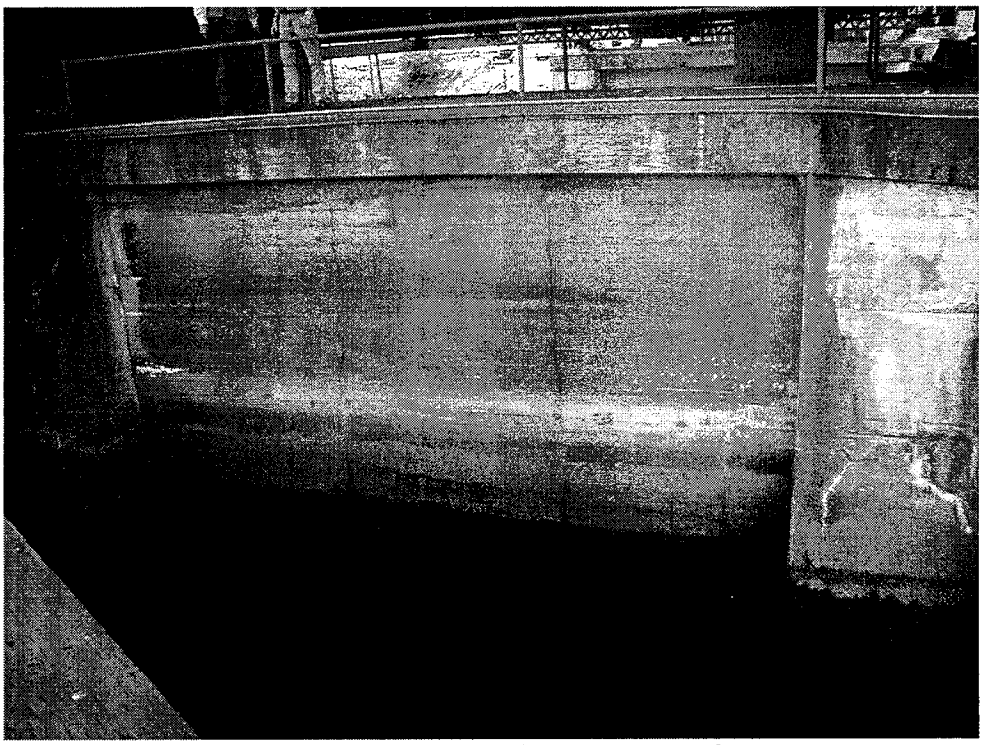
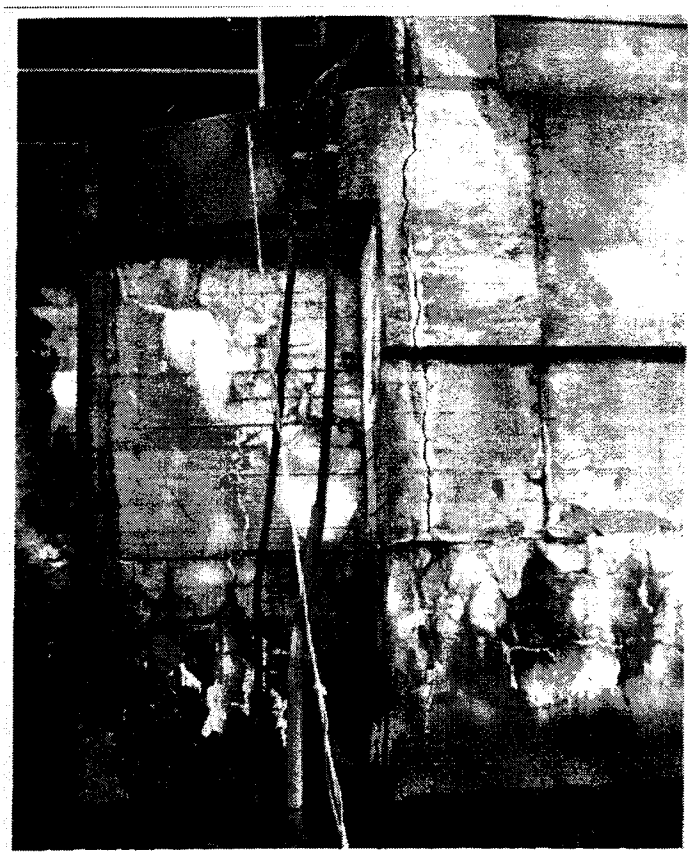
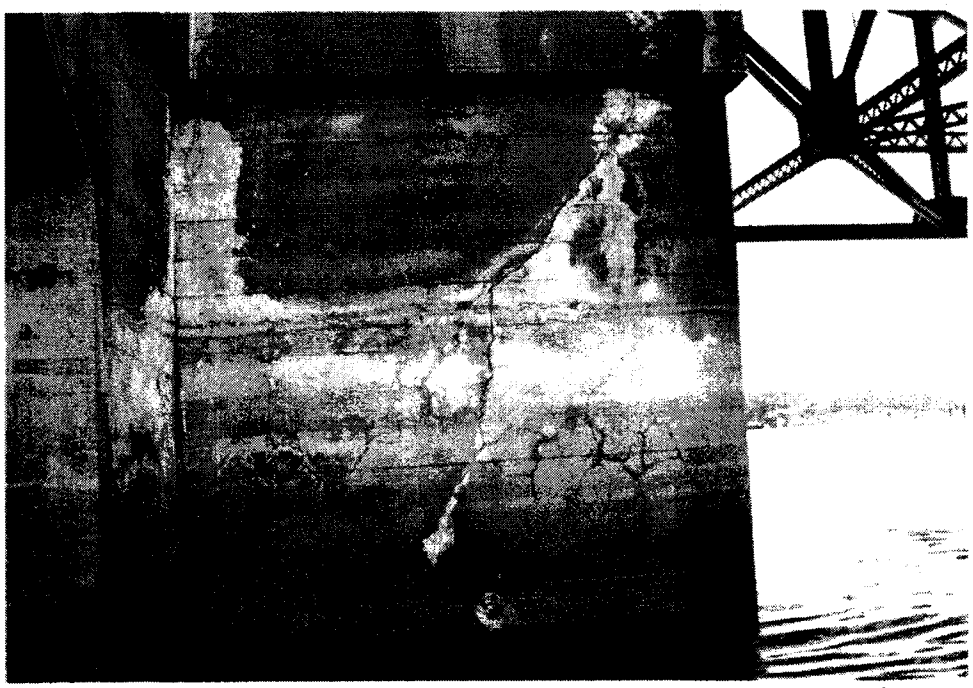


Photo 1.16 – Cracks in South Bascule



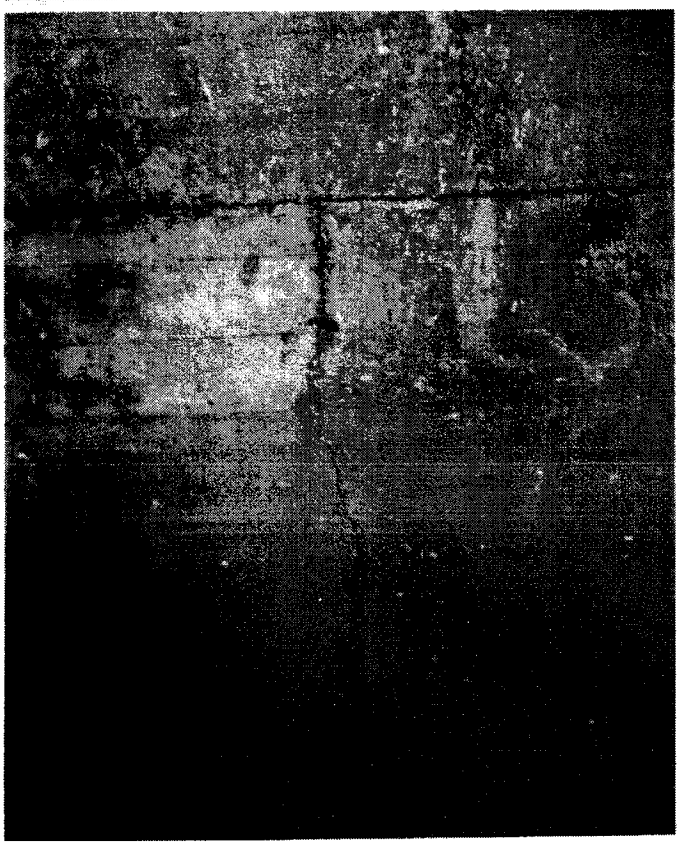
North Bascule, South Face- Extensive vertical cracking between STA 0+40 & 0+50. Also note the hairline cracking with efflorescence.

Photo 1.17 – Cracks in South Face of North Bascule



North Bascule, East Face – Diagonal and hairline cracking along the east-west wall at STA 0+35 to 0+39.

Photo 1.18 – Cracks in East Face of North Bascale



North Bascule, East Face of Interior Pit – Horizontal and Vertical Cracking.

Photo 1.19 – Cracks in North Bascale

Photo 1.20 – Crack in Fascia Wall of Bascule Pier

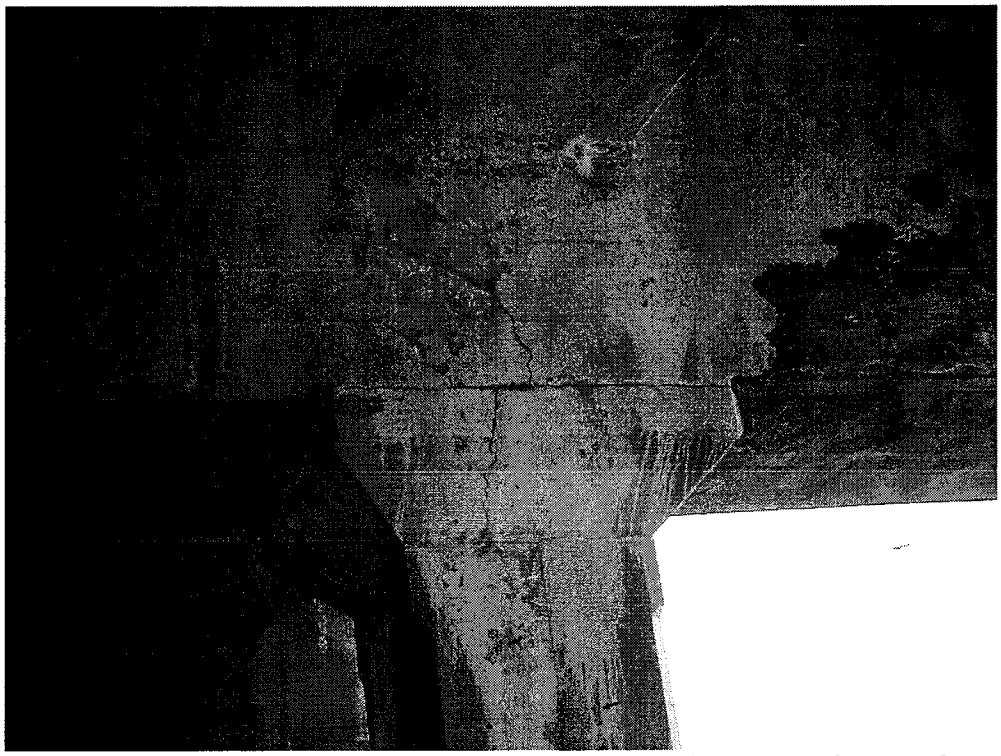


Photo 1.21 – Cracks in Cap Beam and Column of Concrete Approach

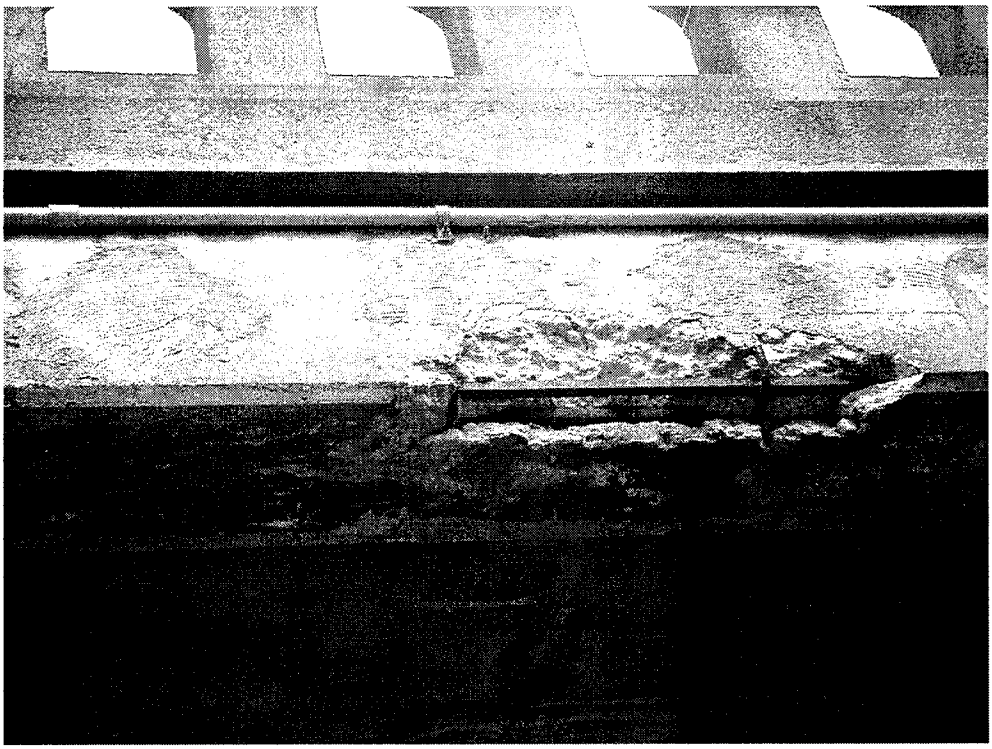


Photo 1.22 – Concrete Spalling and Exposure of Reinforcement in Concrete Appraoch

Photo 1.23 – Concrete Spalling, Cracks and Repair in Concrete Approach

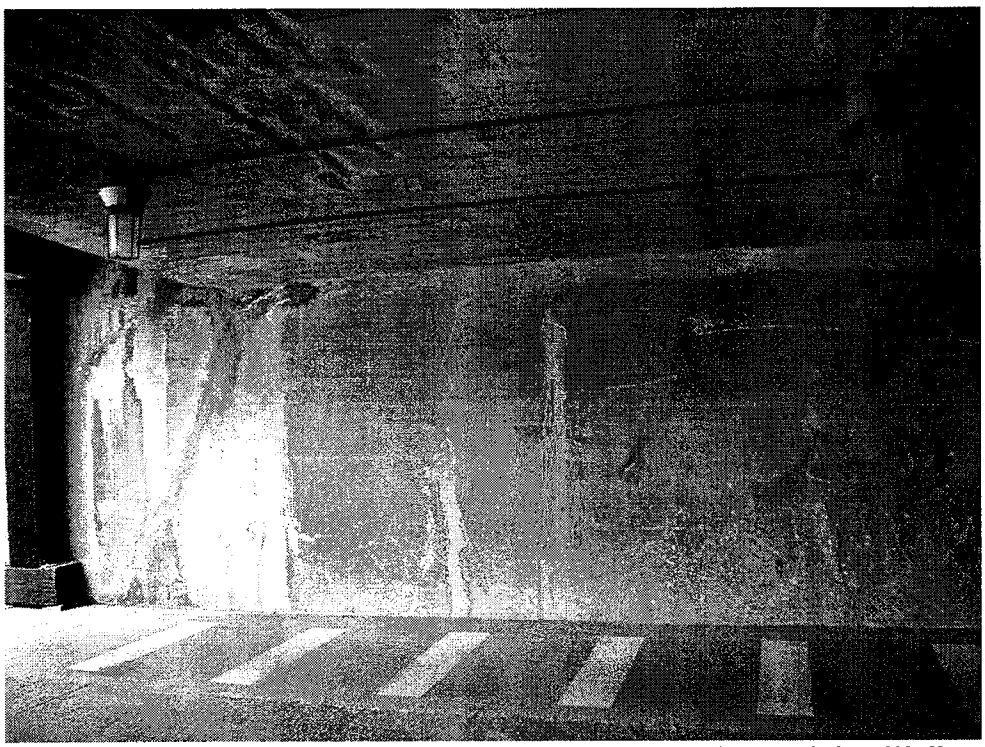


Photo 1.24 – Concrete Spalling in Slab and Cracks in Retaining Wall

Photo 1.25 – Misalignment of Concrete Railing at Approach Spans (Prior to Earthquake)

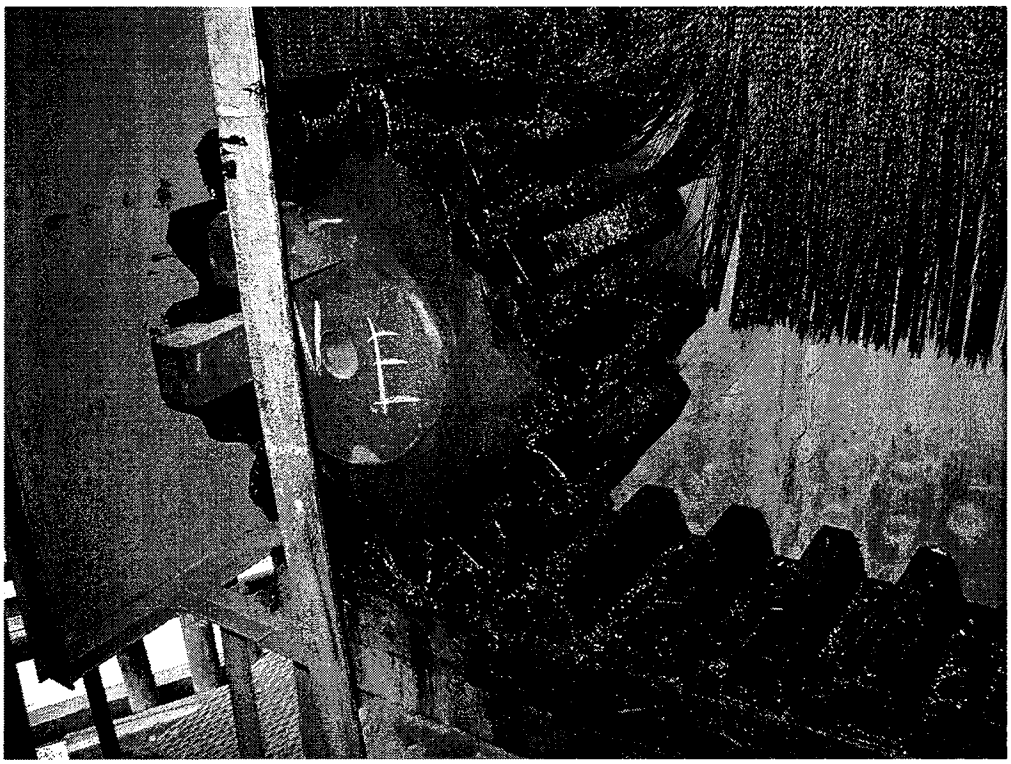


Photo 1.26 – Northeast Pinion Offset of ¼ inch (Post Earthquake Investigation by Imbsen)

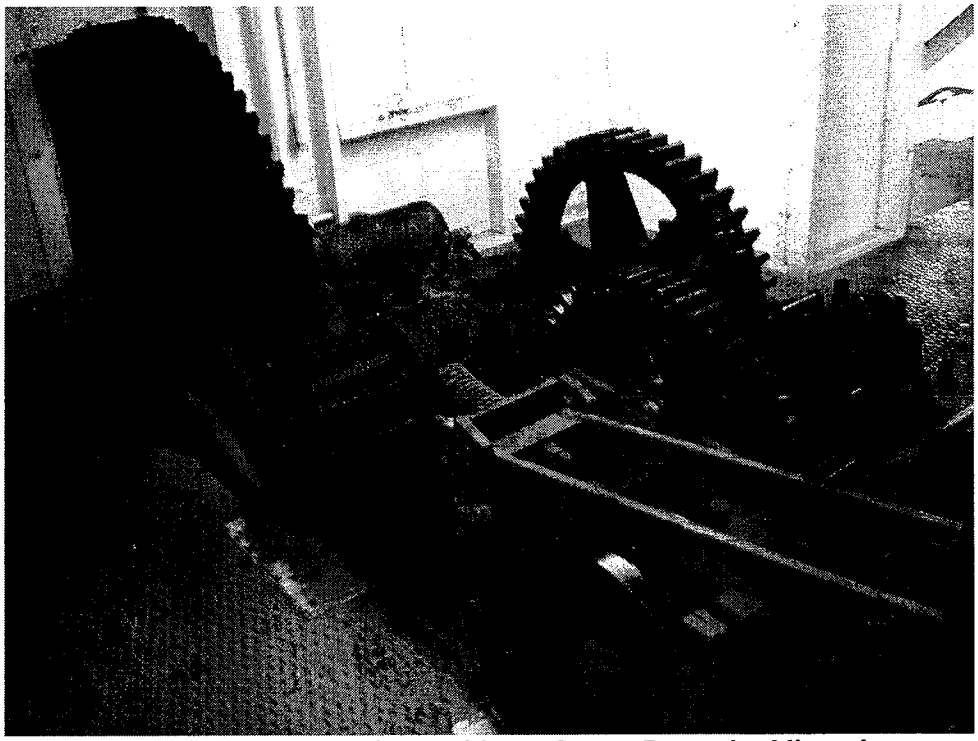


Photo 1.27 – Gears in Machinery Room Properly Aligned (Post Earthquake Investigation by Imbsen)

Photo 1.28 – Southwest Pinion Offset of 1 inch (Post Earthquake Investigation by Imbsen)

Photo 1.29 – Impacting Between Concrete and Steel Truss – South Approaches (Post Earthquake Investigation by Imbsen)

Photo 1.30 – Roadway Settlement – Southside (Post Earthquake Investigation by Imbsen)

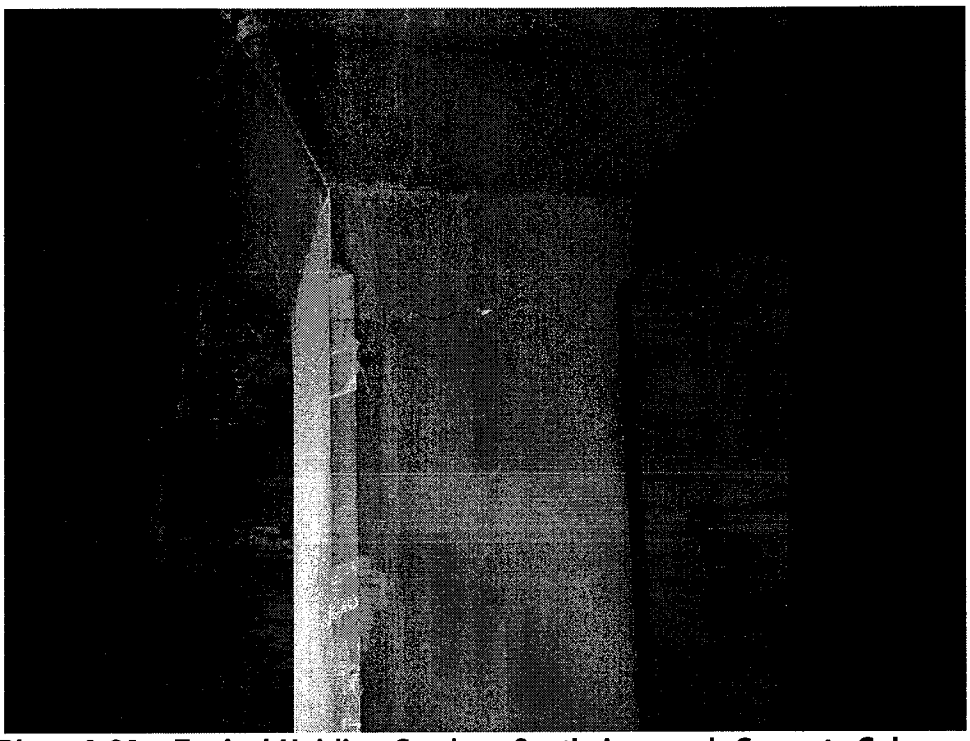


Photo 1.31 – Typical Hairline Cracks – South Approach Concrete Columns (Post Earthquake Investigation by Imbsen)

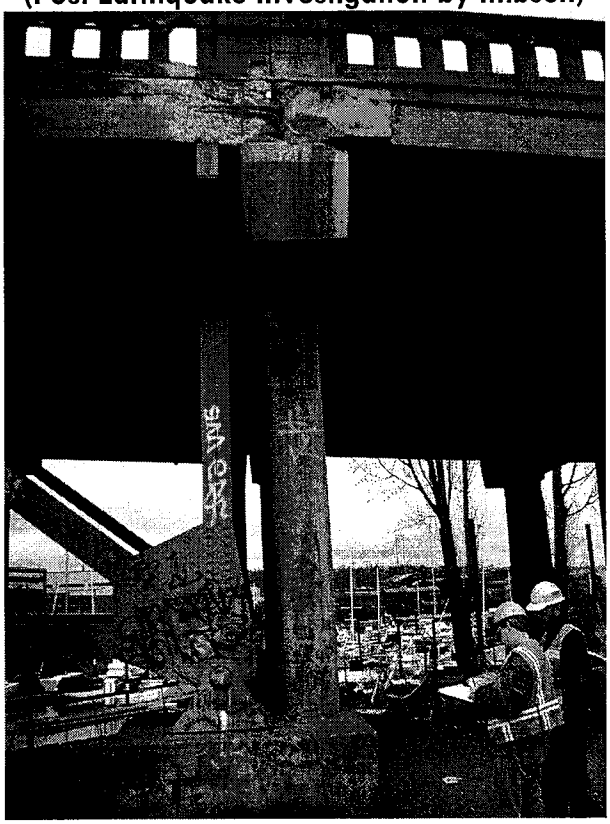
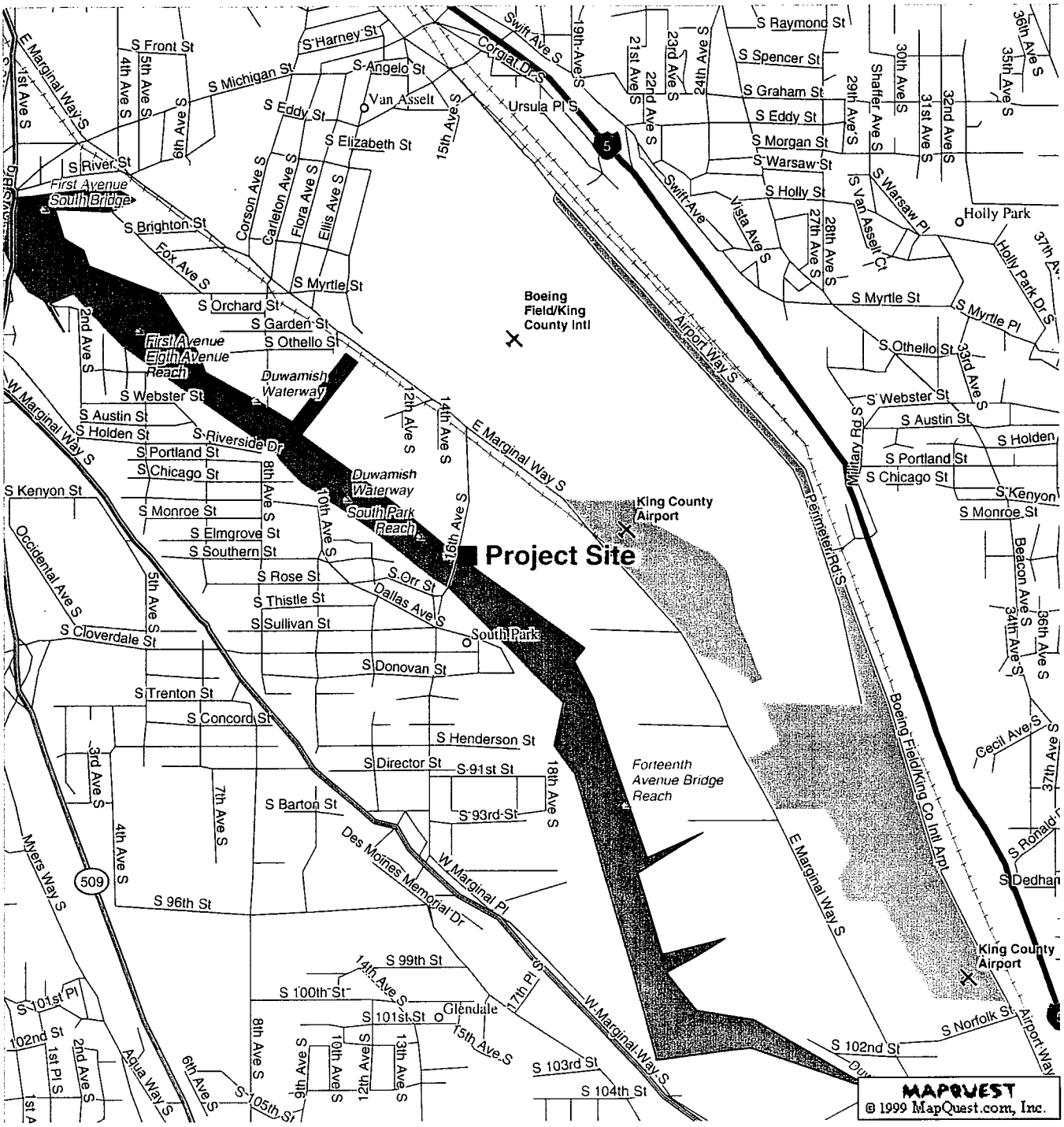


Photo 1.32 – Concrete Columns Adjacent Truss Approach (South) (Post Earthquake Investigation by Imbsen)

Photo 1.33 – Spalling on Railing – South Approach
(Post Earthquake Investigation by Imbsen)

Figure 1.1 - Vicinity Map



Water
Park
Urban area
Military land
Airport
Golf course
Shopping Center
Cemetery
Hospital, university
County border
Ferry
Limited access road
Primary road
Other road
Unpaved or private road
Railroad
Geographic feature

Business Mapping

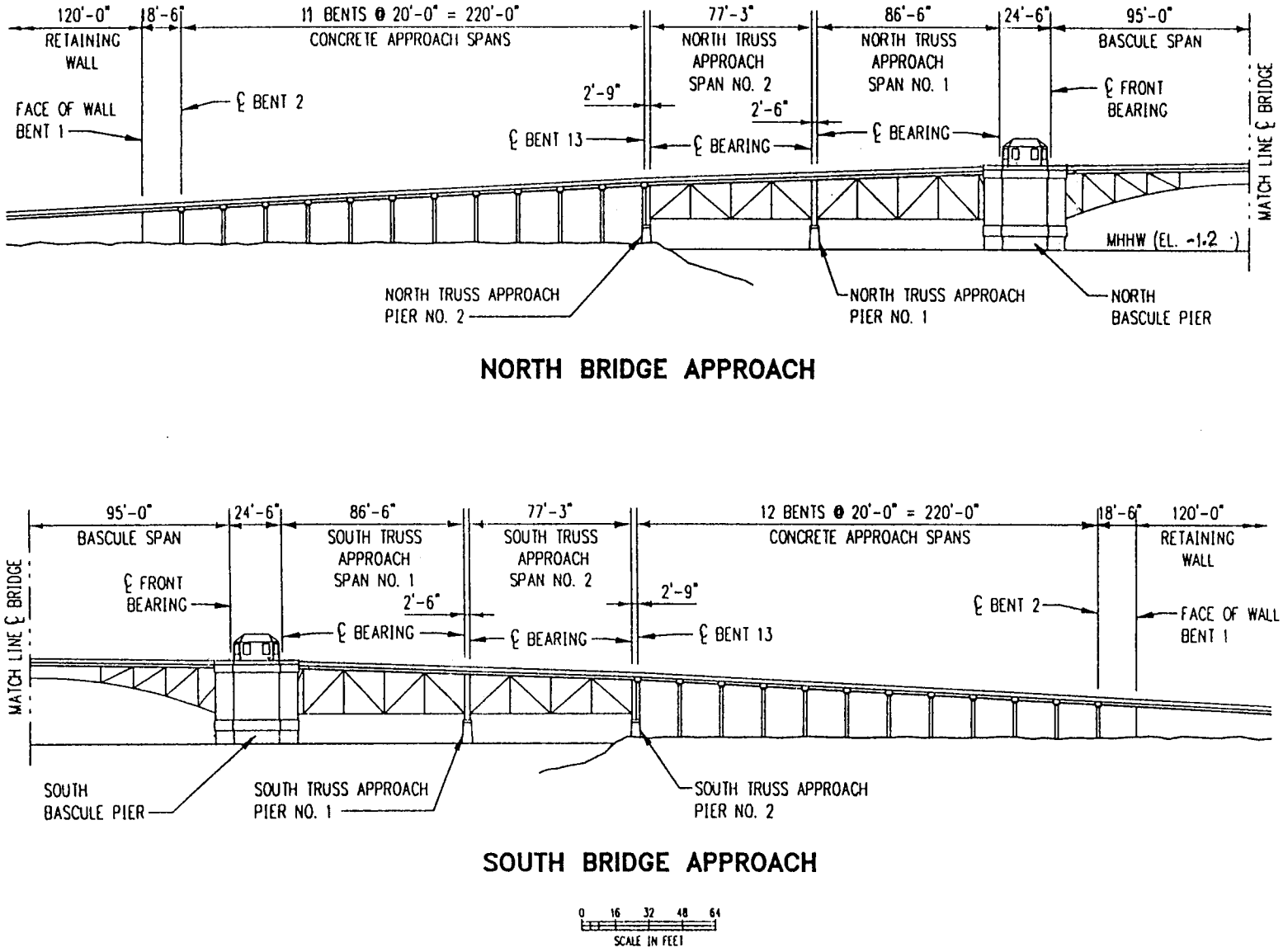


Figure 1.2 Bridge Elevation View

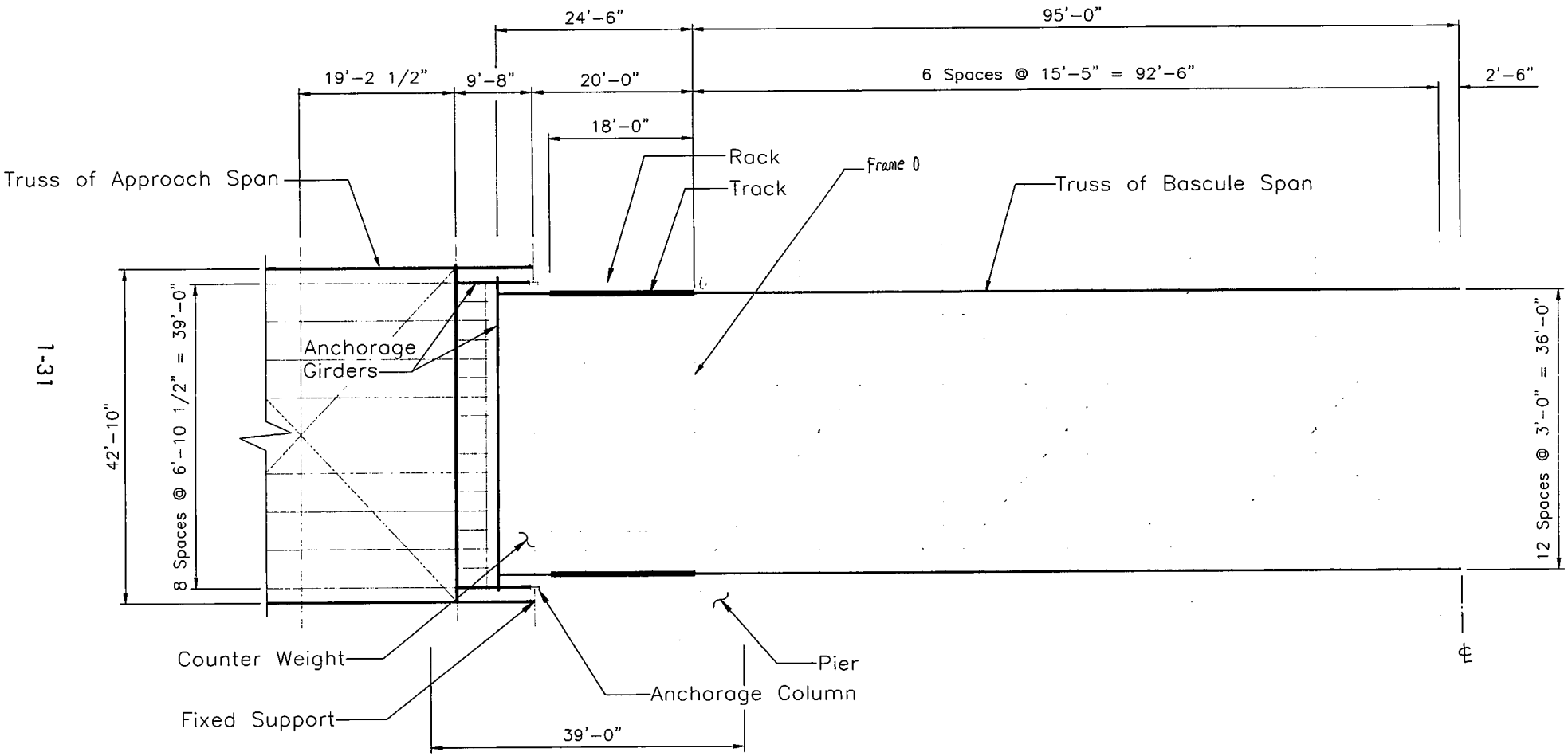


Figure 1.3 Framing Plan of Bascule Span

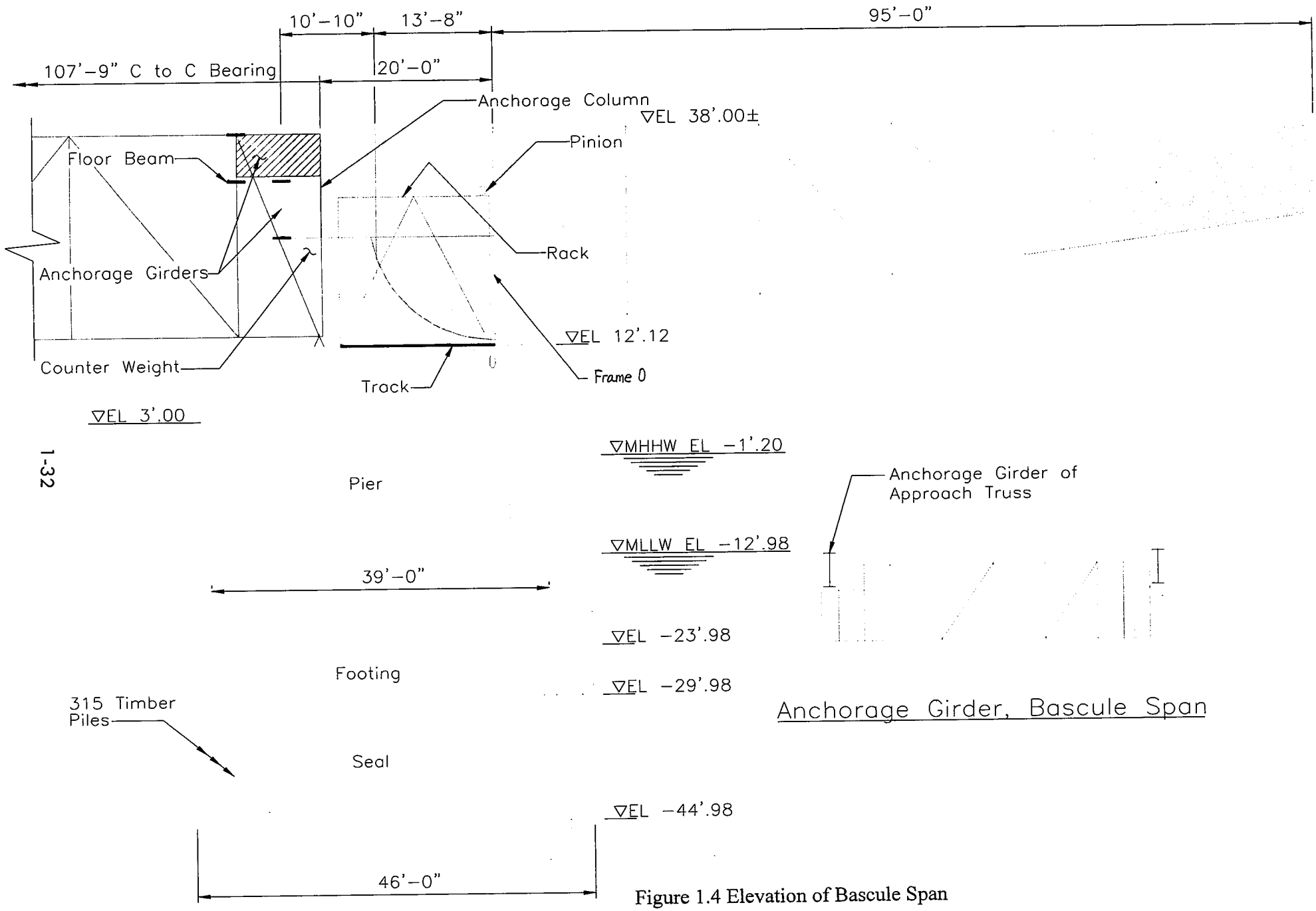
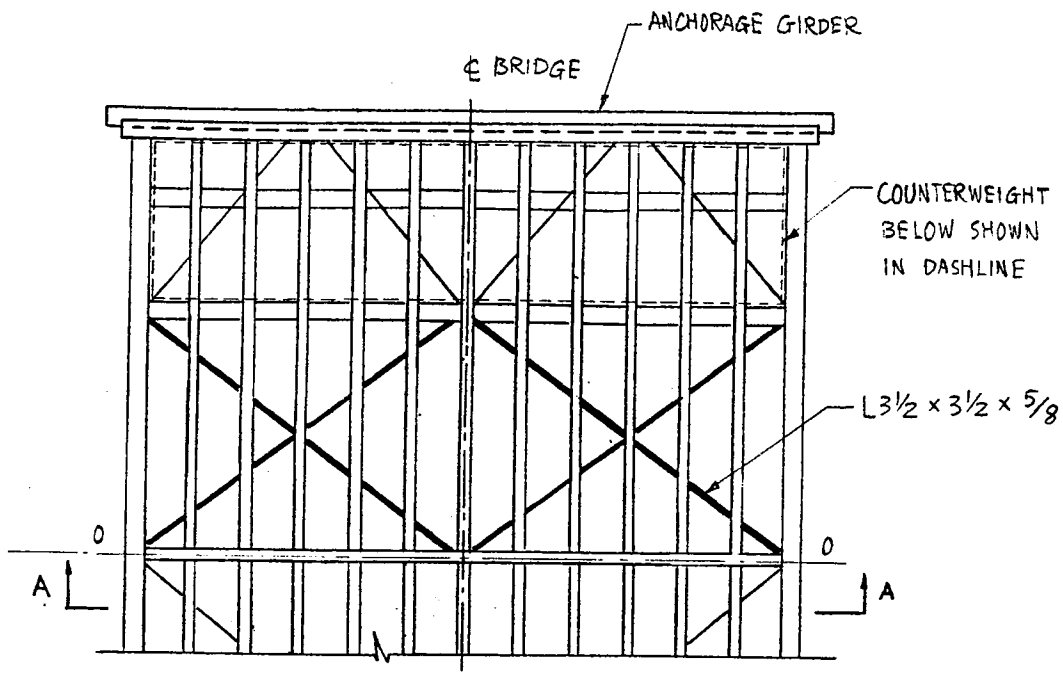
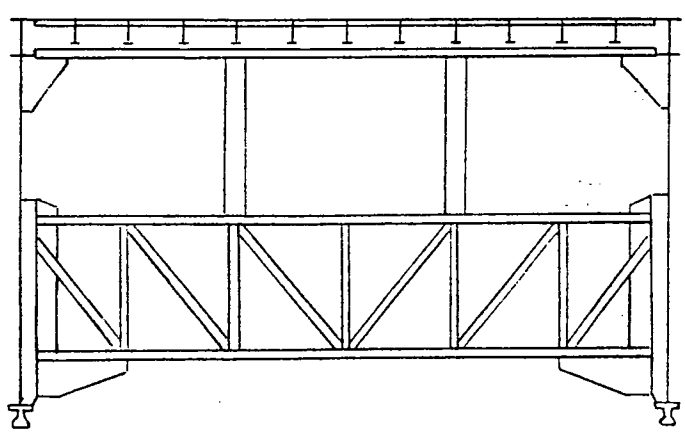


Figure 1.4 Elevation of Bascule Span



PARTIAL PLAN



SECTION A-A
(FRAME 0)

Figure 1.5 Partial Plan and Frame 0 of Bascule Span

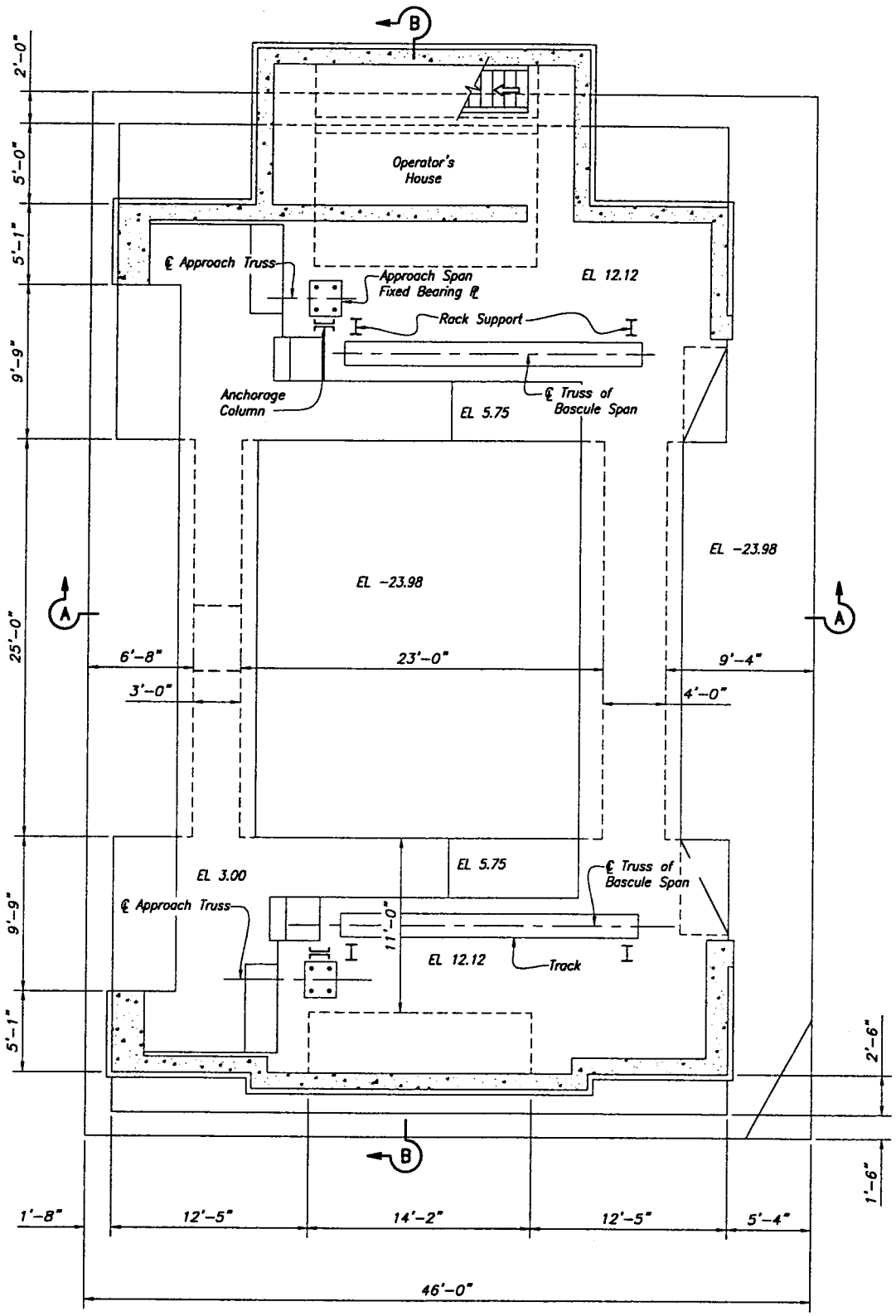


Figure 1.6 Plan of Bascule Pier

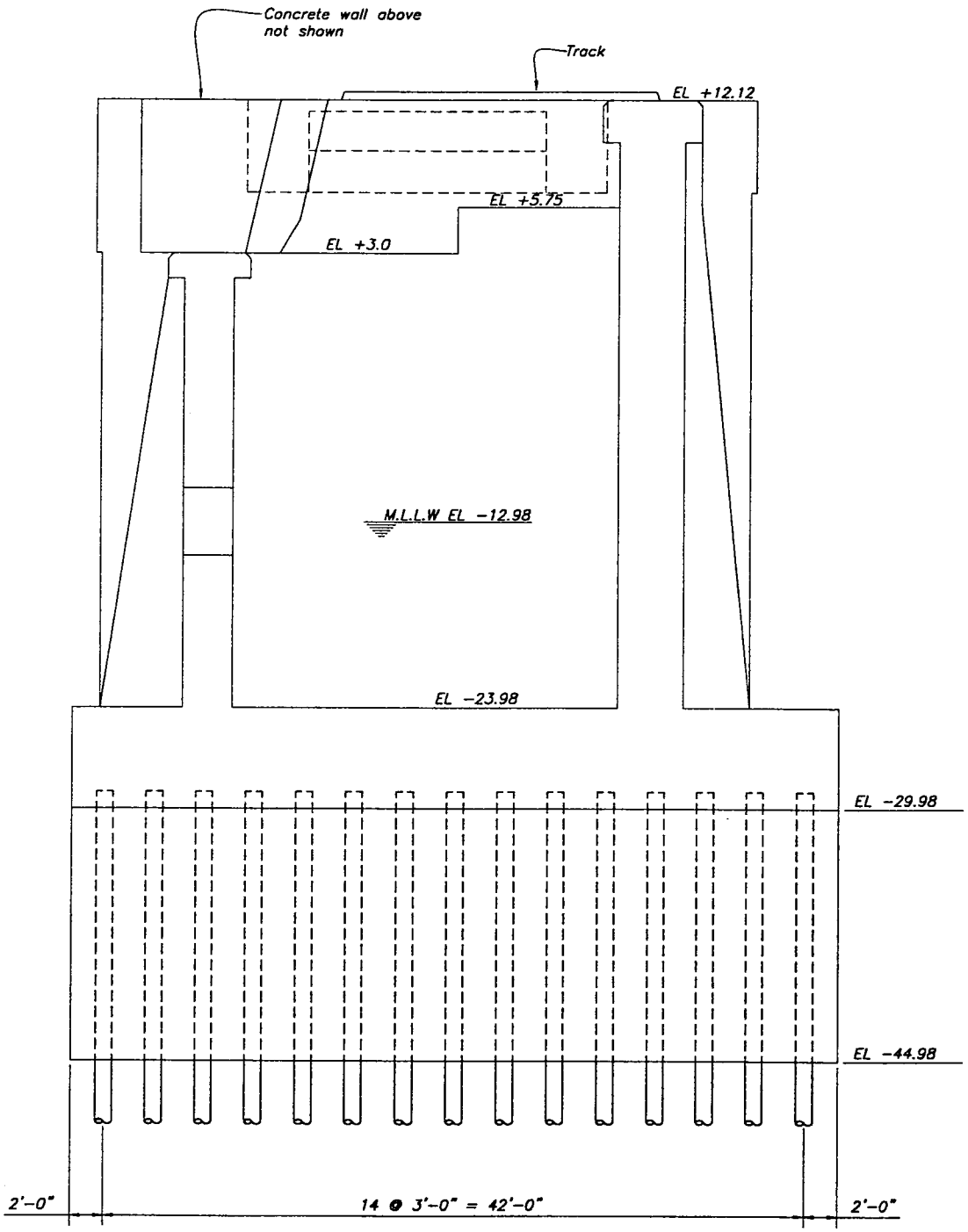


Figure 1.7 Section A-A of Bascule Pier

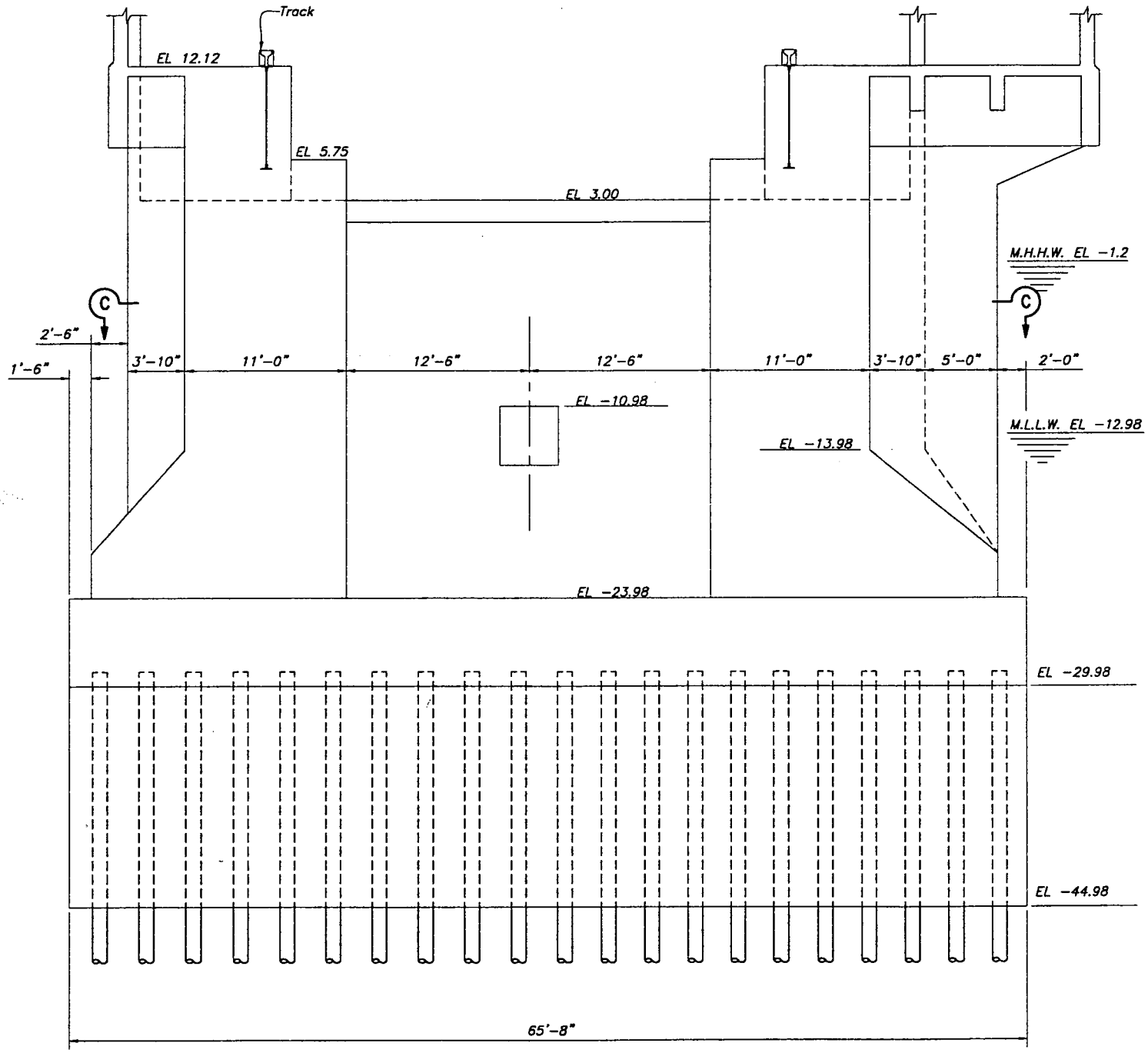


Figure 1.8 Section B-B of Bascule Pier

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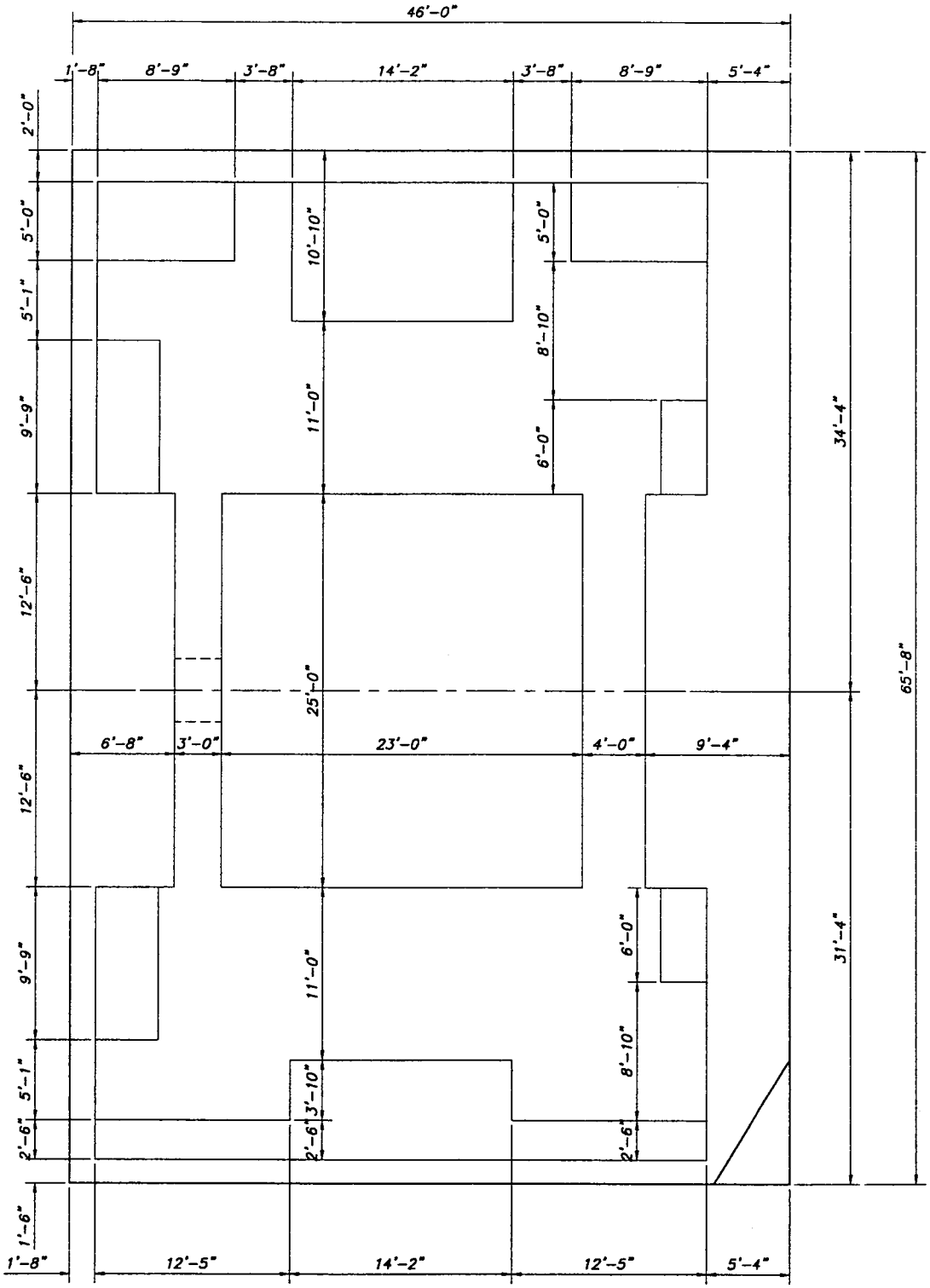

Figure 1.9 Section C-C of Bascule Pier

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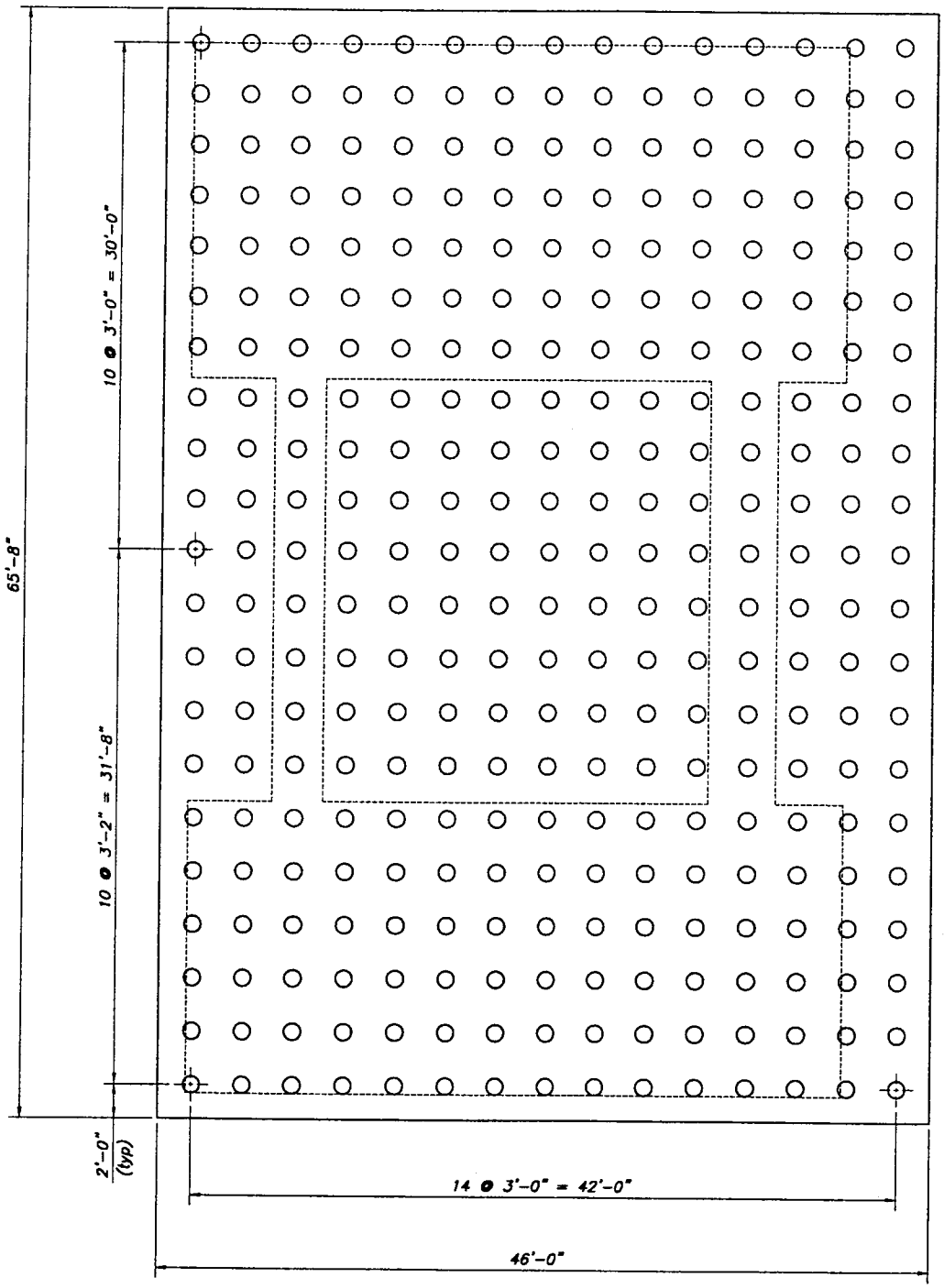

Figure 1.10 Bascule Pile Foundation Layout

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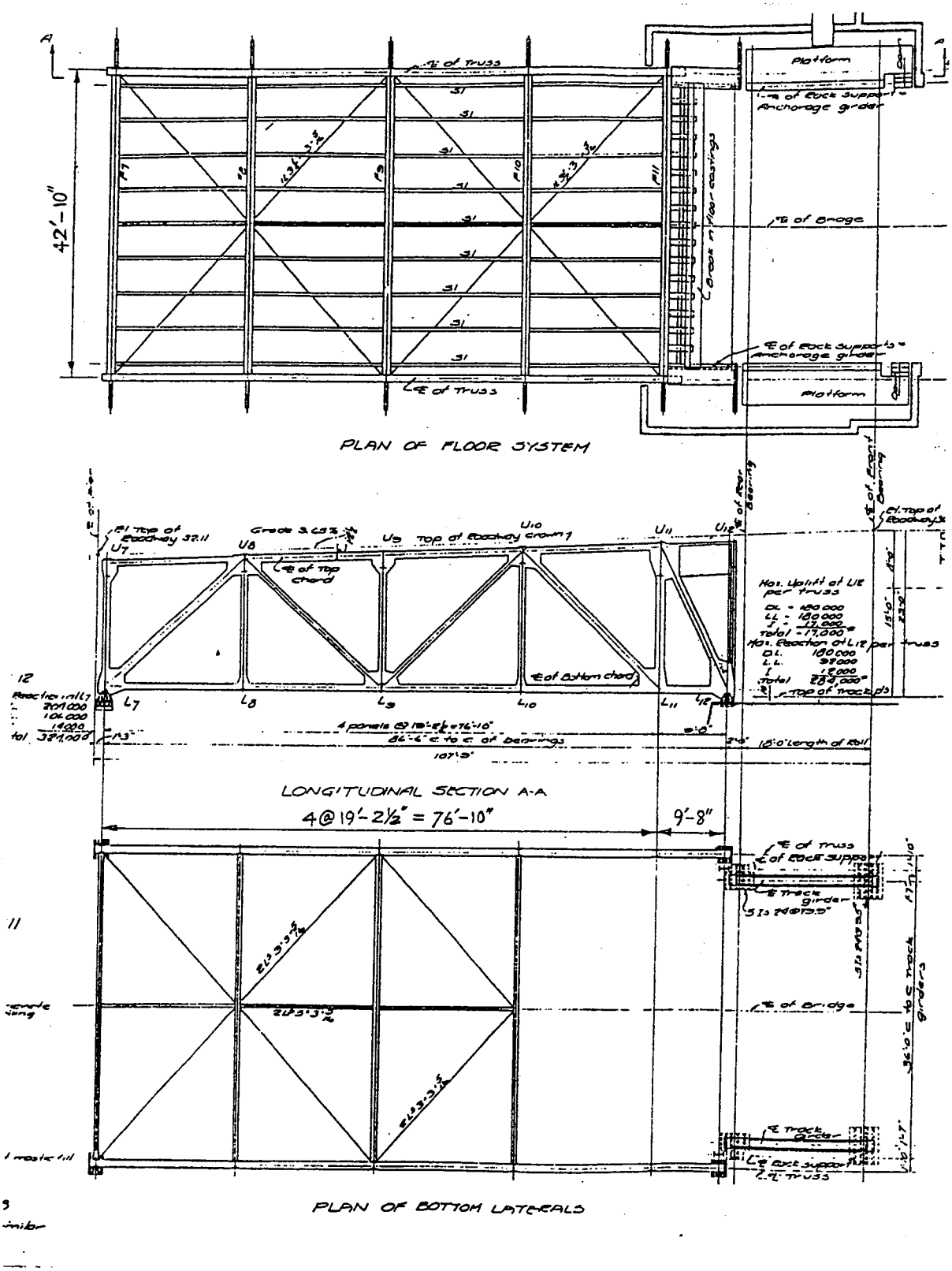


Figure 1.11 Framing of Steel Truss Span #1

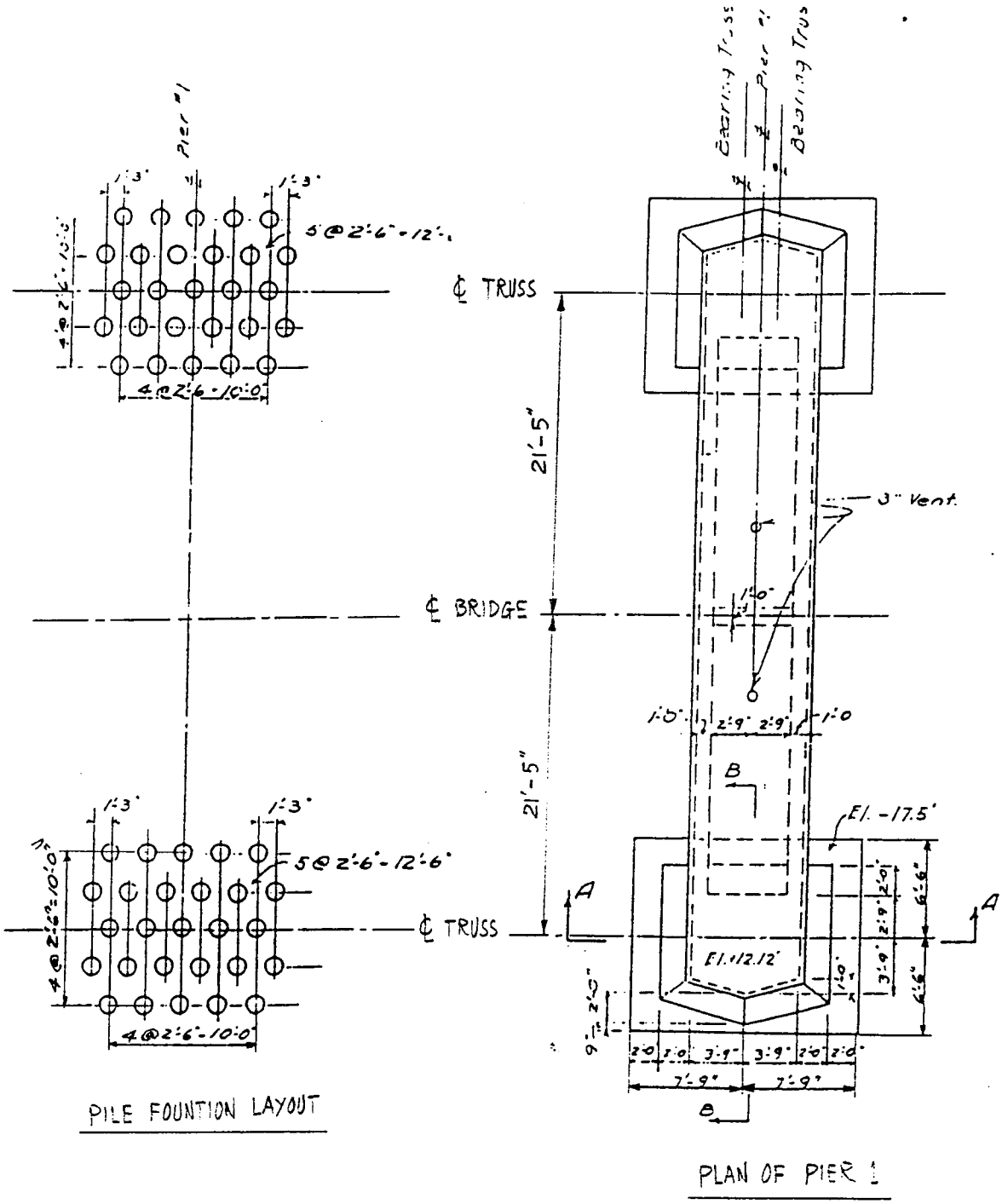


Figure 1.12 Pile Foundation Layout and Plan of Pier 1

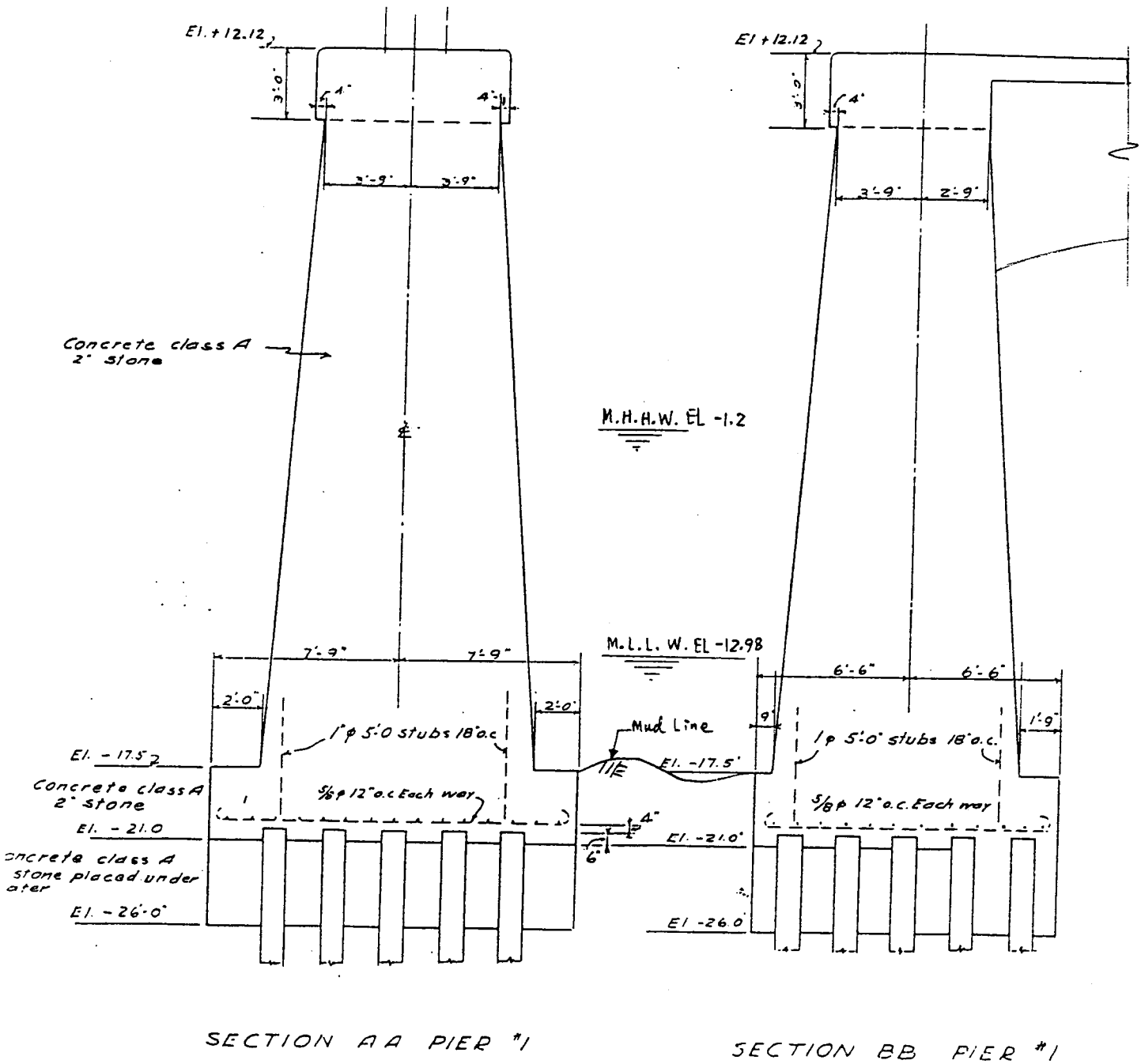


Figure 1.13 Sections of Pier 1

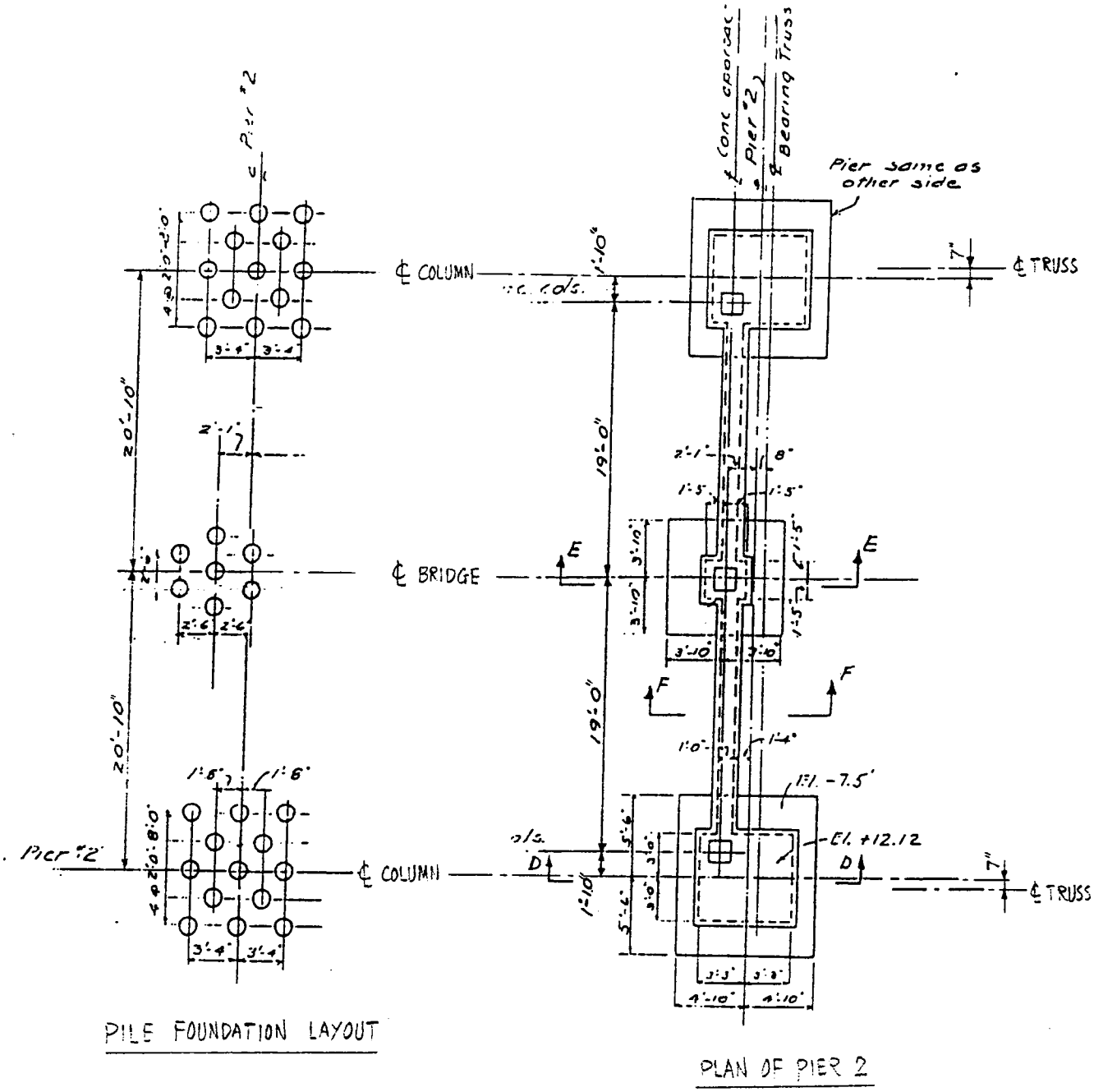


Figure 1.14 Pile Foundation Layout and Plan of Pier 2

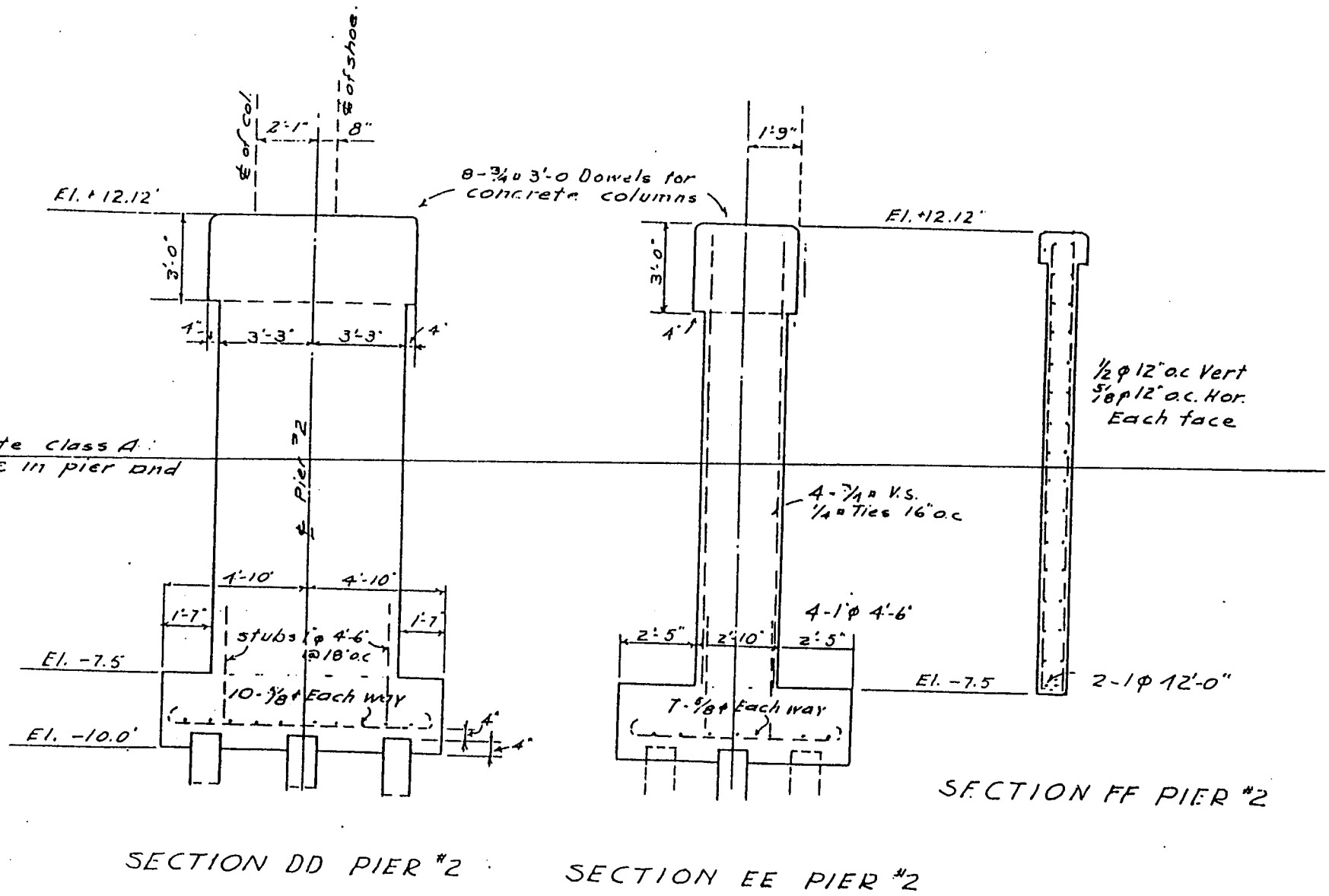


Figure 1.15 Sections of Pier 2

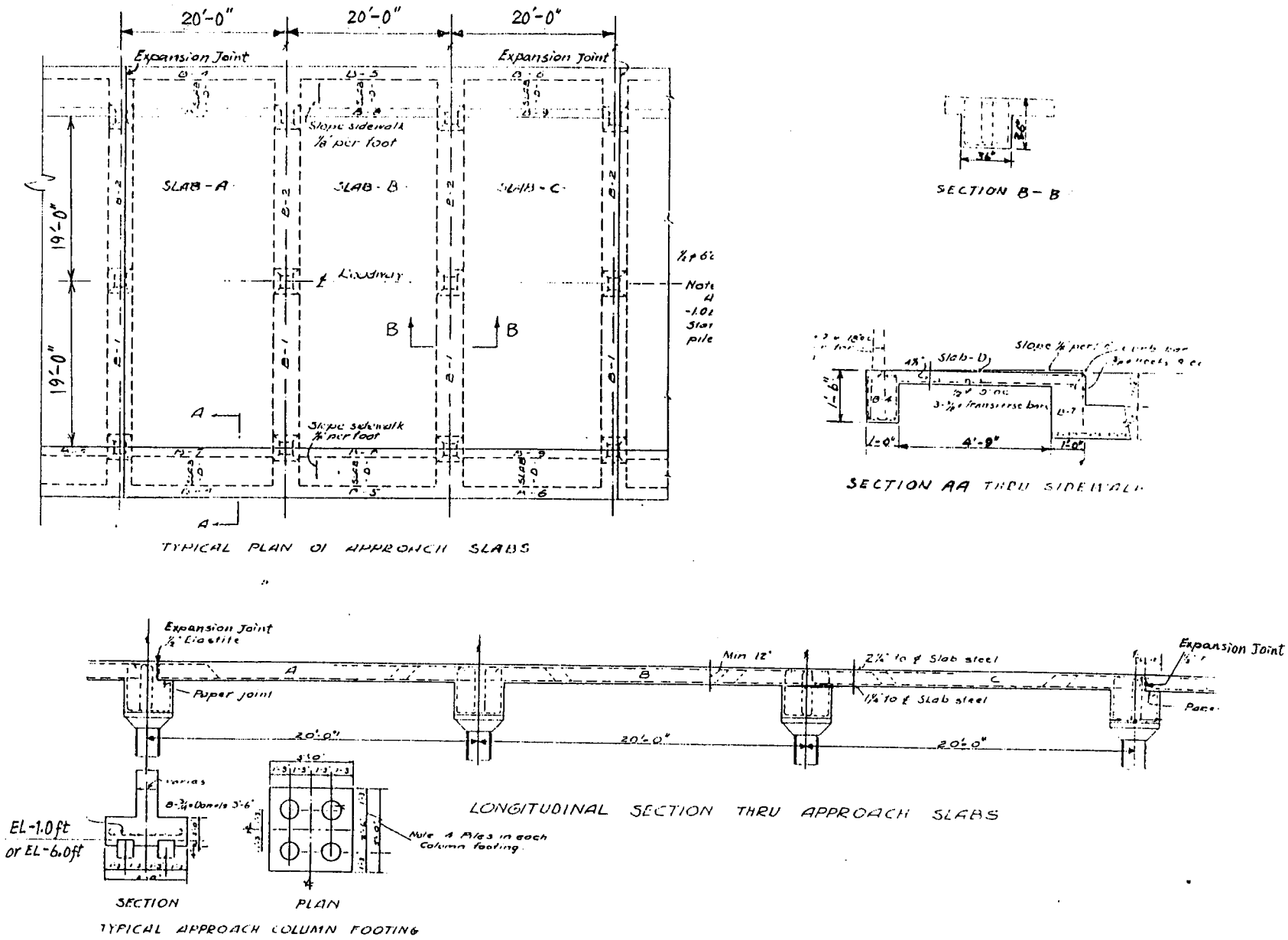


Figure 1.16 Typical Plan and Sections of Concrete Approach

2. SEISMIC EVALUATION CRITERIA

For new bridges, the goal of seismic design is to satisfy a predefined performance requirement for the given seismic hazard environment in accordance with the AASHTO Seismic Design Specifications, Division 1-A or owner's site specific seismic design criteria. The seismic hazard exposure in AASHTO Codes is defined as an earthquake event having a 10% probability to occur in 50 years (i.e., return period of 475 years). Recognizing that most bridges last much longer than 50 years, the service life has been defined as a 150-year in recent seismic retrofit design practices. This implicitly means the acceptable seismic risk of a new bridge design is about 30% on the average for an exposure duration of 150 year. On the other hand, the current AASHTO Codes require temporary bridges, having a service duration longer than 5 years, shall be treated as permanent bridges as far as seismic design is concerned. If the current bridge design standard is applied to a temporary bridge with 10 years of service life, the seismic risk of the bridge is much less than 10%. For a "non-lifeline" bridge, it is usually acceptable to retrofit the bridge to a seismic performance standard lower than what is required for a new bridge.

In this study, two levels of earthquakes are adopted for evaluation based on a 10% seismic risk assuming the bridge has 10 or 20 years of remaining service life. In addition, an earthquake level that has a 10% chance of exceedance in 50 years is considered when alternative retrofit schemes are investigated.

2.1 Response Spectrum Curves

Figure 2.1 shows the response spectrum curves corresponding to the following three levels of earthquakes:

1. Level 1: Earthquake of 95 year return period (95 year earthquake): 10% chance of exceedance in 10 years,
2. Level 2: Earthquake of 190 year return period (190 year earthquake): 10% chance of exceedance in 20 years,
3. Level 3: Earthquake of 475 year return period (475 year earthquake): 10% chance of exceedance in 50 years.

The site-specific response spectrum curves of the first two levels of earthquakes were developed by our subconsultant PanGEO, Inc. They represent earthquakes of magnitudes of approximately 6 to 6.5 and 6.5 to 7 occurring in the Seattle area. The curve of a 475 year earthquake is obtained from AASHTO assuming a site coefficient of 1.2. This curve represents an earthquake magnitude of approximately 7.0 to 7.5 occurring in the Seattle area with a corresponding Peak Ground Acceleration of 0.33g.

The Nisqually earthquake, which occurred on February 28, 2001, had a magnitude of 6.8 with an epicenter 30 miles southwest of Seattle. Initially our geotechnical engineer had estimated this earthquake was comparable to the 95-year earthquake based on the ground motions between 0.10 to 0.15g. This was reported during our presentation to King County on May 17, 2001. However, when the processed ground motion records became available later, it appears that ground accelerations of about 0.18 to 0.20g

would be appropriate for the general conditions in the area. Therefore, the Nisqually earthquake would be equivalent to the 190-year earthquake

2.2 Seismic Performance Criteria

A "no collapse" criteria has been established for the structure. For the retrofit design, the seismic performance criteria of the retrofitted bridge is summarized in the following table:

Ground Motion at Site	"No Collapse" Level	Damage Level	Retrofit Strategy
Level 1 Earthquake "95 year earthquake"	Limited (Highway)/ Limited (Shipping)	Repairable Damage	1
Level 2 Earthquake "190 year earthquake"	Closed (Highway)/ Closed (Shipping)	Major Damage (Replace Bridge)	1
Level 3 Earthquake "475 year earthquake"	Closed (Highway)/ Closed (Shipping)	Major Damage (Replace Bridge)	2

Where the "No Collapse" level of limited (highway)/limited (shipping) shall mean:

1. Closed to emergency vehicles immediately after the earthquake, open to emergency vehicles after inspection;
2. Closed to normal vehicles after the earthquake, open to emergency vehicles after 2-3 months of repair;

3. No ability to raise and lower the bridge to permit passage of shipping immediately after the earthquake, able to raise and lower the bridge to permit passage of shipping after 2-3 months of repair.

The "No Collpase" level of closed (highway)/closed (shipping) shall mean:

1. Closed to emergency vehicles permanently,
2. Closed to normal vehicles permanently,
3. No ability to raise and lower the bridge to permit passage of shipping permanently.

Repairable damage level shall mean:

1. Damage to structural components can be repaired within the period of 2-3 months,
2. Damage to mechanical and electrical components can be repaired within the period of 2-3 months.

The level of major damage (replace bridge) shall mean:

1. Damage to some structural and mechanical components that can not be repaired or are not economical to be repaired,
2. A replacement bridge is needed to restore the functionality of the bridge.

2.3 Liquefaction and Lateral Spreading

The liquefaction potential of the soil underlying the bridge alignment was analyzed by PanGEO, Inc. Their study concluded that liquefaction may occur in the top 15 feet of soil. For the evaluation of the bascule span, analyses are conducted for soil conditions with liquefaction and without liquefaction. See Appendix C, which contains the geotechnical report. Comparisons are made to this study and previous studies done by Shannon and Wilson.

Lateral spreading may occur after liquefaction. PanGEO, Inc. recommended to use an equivalent fluid weight of 30 pounds per cubit feet (pcf) to determine the incremental force from lateral spreading on the upland side of the bascule pier.

2.4 Passive Resistance of Soil Against Footings and Seals

PanGEO recommended to determine passive resistance of the soil based on an equivalent fluid weight of 350 pcf with a threshold displacement of 4 inches.

2.5 Load Combination

The following three earthquake load cases are considered in the response spectrum analysis:

1. Case 1: 1.0 Longitudinal + 0.3 Transverse + 0.3 Vertical,
2. Case 2: 0.3 Longitudinal + 1.0 Transverse + 0.3 Vertical,

3. Case 3: 0.3 Longitudinal + 0.3 Transverse + 1.0 Vertical.

The maximum earthquake load from the three cases is determined and combined with dead load to form the load demand for the evaluation,

Load Demand = Dead Load $\pm$ Maximum Earthquake Load

2.6 Hydrodynamic Effects

Response of structures submerged in water is influenced by the dynamic interaction between the structure and the surrounding water. For water within a bascule pier, the mass of the water is added to the system in the analysis. For water surrounding the piers, the dynamic effect of the water is also represented as a mass, which is defined as a fraction of the mass of the displaced water.

The total added mass is:

Added Mass = Cm * Mass of Displaced Water + Mass of Trapped Water

Where Cm is determined from Figure 2.2. In the figure, r_0 is an equivalent radius of the submerged structure, H is height of the structure above the base and z is the vertical distance from the base to the point of interest. Hydrodynamic effects are included in the analyses of the bascule span and Pier 1 of the steel truss approach. It is ignored for the analysis of Pier 2 because it's footing is relatively shallow and the amount of water mobilized during an earthquake is negligible.

2.7 Water Elevations

Two levels of water elevations are considered in this study. The Mean Lower Low Water Level (MLLW) has an elevation of −12.98 feet. The Mean Higher High Water Level (MHHW) has an elevation of −1.2 feet. Lower water level means a greater effective gravity load and a smaller added mass due to hydrodynamic effects.

2.8 Concrete and Steel Properties

The following properties of existing steel and concrete are assumed in the evaluation:

Steel: Fy = 40 ksi, E = 29000 ksi

Concrete: f_c' = 2.5 ksi, E = 2850 ksi, f_t = 163 psi

Where the value of f_c' is based on results from references 6 and 15 with consideration of deterioration in the concrete. ft is obtained from ACI318, and only 30% of this value will be used as flexure tensile capacity because the uncertainty of the locations of construction joints.

2.9 Timber Pile Capacities

Timber pile capacities, derived from soil support by PanGEO Inc., are as follows:

Tensile Capacity = 50 kips

Compression Capacity = 130 kips

Shear Capacity = based on a load defection curve.

Shear capacity of timber piles for Pier 2 is less than 17.2 kips when there is soil liquefaction. Timber pile capacities for different types of timbers are determined using AASHTO 1998 LRFD Table 8.4.1.1.4-1 and considering the strength reduction factor equal to 1.0 for tension and compression because of redundancy.

Type	Tensile Capacity (kips)	Compression Capacity (kips)	Shear Capacity (kips)
Spruce-Pine Fir Selected Structural No. 2 NLGA Grading	108	136	17.2
Douglas Fir-Larch Dense Selected Structural No.2 WWPA Grading	229	129	20.8

Therefore, the capacities of the timber pile used in the evaluation are 50 kips for tension, 130 kips for compression and 17.2 kips for shear, respectively.

Period (sec)	Sa/g		
	AASHTO	190 Years	95 Years
0.0000	0.825	0.200	0.150
0.0525	0.825	0.250	0.188
0.0750	0.825	0.300	0.225
0.0975	0.825	0.350	0.263
0.1200	0.825	0.400	0.300
0.2000	0.825	0.400	0.300
0.2974	0.825	0.400	0.300
0.3326	0.825	0.400	0.300
0.4000	0.825	0.400	0.300
0.4300	0.825	0.400	0.300
0.5200	0.735	0.400	0.300
0.6000	0.668	0.347	0.260
0.8000	0.551	0.260	0.195
1.0000	0.475	0.210	0.158
1.2500	0.409	0.166	0.125
1.5000	0.363	0.139	0.104
2.0000	0.299	0.100	0.075
3.0000	0.228	0.046	0.035
5.0000	0.162	0.017	0.013

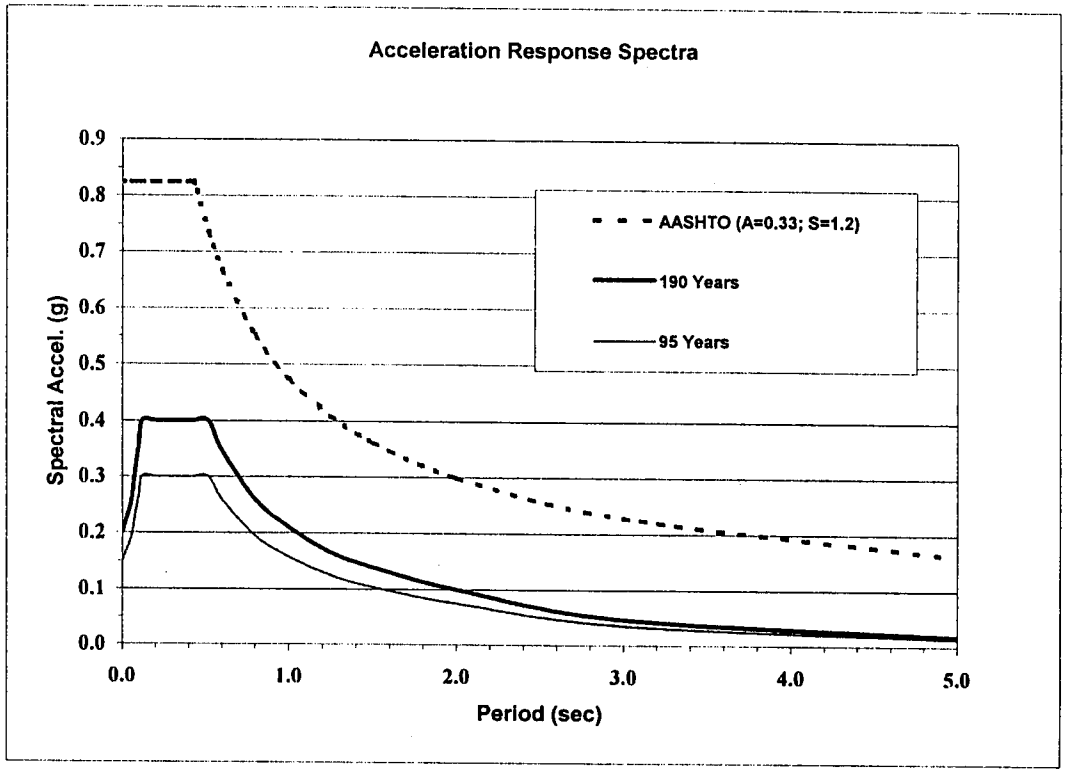


Figure 2.1 Acceleration Response Spectra

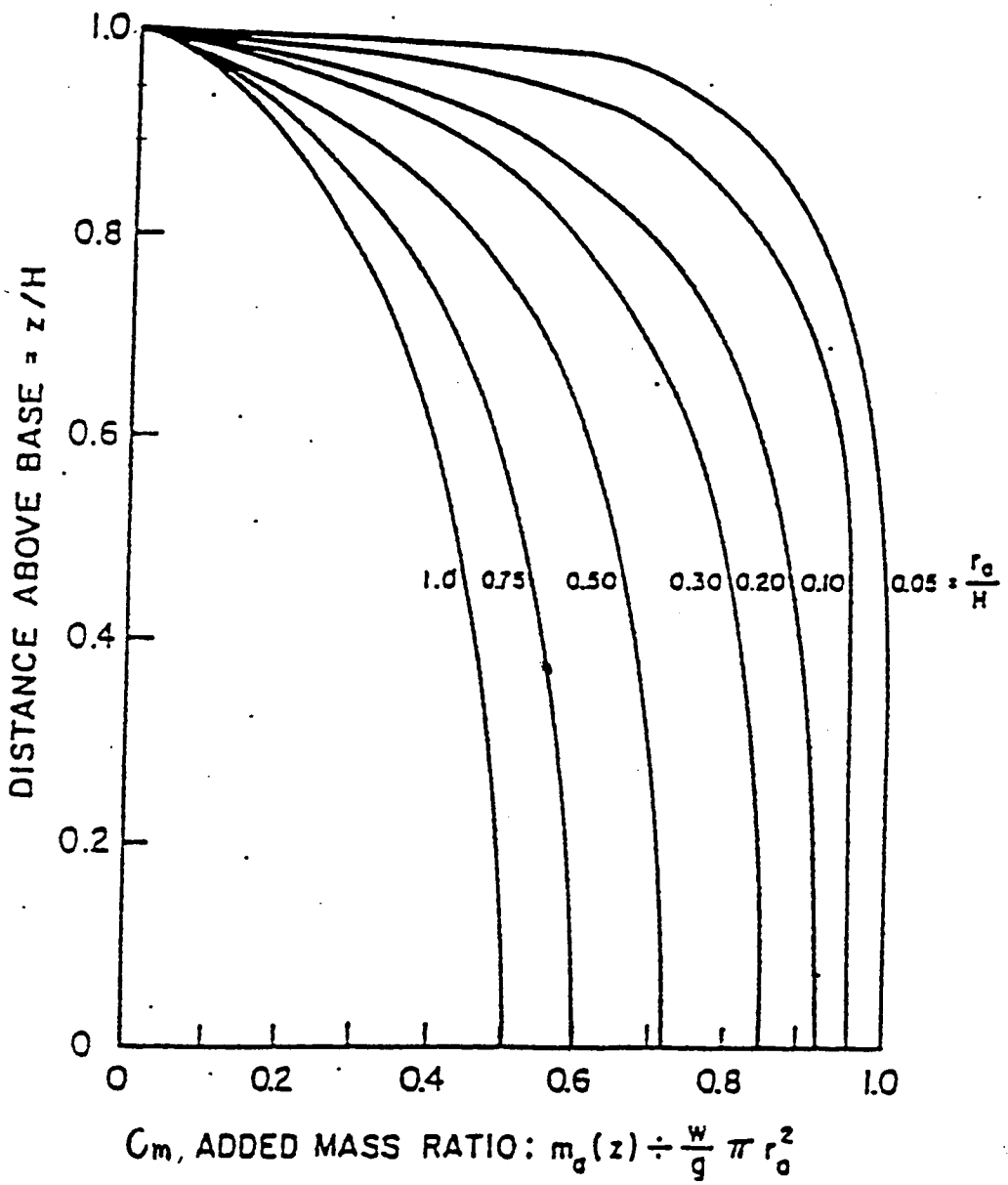


Figure 2.2 Added Mass Ratio of Submerged Structures

3. SEISMIC EVALUATION OF THE BASCULE SPAN

3.1 Analytical Model

Figure 3.1 shows the three-dimensional model used for the analysis of the bascule span in the closed position. See Figure 1.2 for location of the bascule span. The bascule span in the opened position has a very low probability of occurring during an earthquake because there are only 3 to 5 openings per day on average. Therefore, analysis in the opened position is not considered in this study. The superstructure is modeled with trusses consisting mainly of rigid members. Flexible beam members are used to connect the left half portion and right half portion of the structure at the center lock to account for the vertical displacement of the superstructure. The masses of the rolling span and the counterweight are modeled as concentrated masses at appropriate locations as illustrated in Figure 3.2. The mass contribution of a steel truss span is treated as concentrated masses on a rigid frame supported by the bascule as shown in Figure 3.2. For longitudinal ground motion input, the mass of the full truss span is considered to represent each truss span. For transverse and vertical ground motion input, the mass of half steel truss span is considered for each truss span. The operator house is represented with beam members. Beam members are connected to the counterweight from the bascule to simulate the anchorage of the truss approach span. The east and west walls of the bascule are modeled with beam members. The north and south walls are discretized with plate elements. The footing and the seal are modeled with beam members. Rigid members are used to connect the bascule walls and the footing. The pile foundation is represented with six linear springs. The spring constants are listed in Table 3.1. Passive resistance of soil against the footing and the seal is also modeled as linear springs. The spring constants

are also tabulated in Table 3.1. Both liquefaction case and no liquefaction case are included in the analyses. Soil liquefaction is simulated by releasing the soil springs of the passive resistance of top 15-foot soil against the footing and the seal. Mean Lower Low Water (MLLW) elevation at -12.98 feet and Mean Higher High Water (MHHW) elevation at −1.2 feet are considered in the analyses.

3.2 Structural Dynamic Characteristics

Table 3.2 summarizes modal periods and mass participation factors under MHHW condition when soil liquefaction does not occur. Figures 3.3 to 3.6 depict the mode shapes of modes 1, 2, 3 and 8. It is observed that mode 3 is a combination of longitudinal translation and rocking about transverse axis, which has a period of 0.78 second and a participation factor of 99.5% under longitudinal earthquake loading. Mode 2 is a longitudinal translation mode, which has little mass participation under longitudinal ground motion. Mode 1 is a transverse translation mode, which has a period of 0.80 second and a participation factor of 99.8% under transverse earthquake loading. Mode 8 is a dominant vertical mode, which has 98.4% of participation under vertical ground motion.

Table 3.3 lists modal periods and mass participation factors under MHHW condition when soil liquefaction occurs. Figures 3.7 to 3.10 illustrate the mode shapes of modes 1, 2, 3 and 8. It is noted that the periods of modes 1 to 3 increase because of soil liquefaction. The dominant modes are mode 2 under longitudinal ground motion and mode 3 under transverse ground motion, respectively.

3.3 Performance Evaluation for 190 Year Earthquake

3.3.1 Displacement Responses

Table 3.4 summarizes the maximum displacements at the top of the bascule pier under MHHW condition. The displacements increase when soil liquefaction occurs. The maximum displacements under a 190 year earthquake are 2 inches along longitudinal direction, 1.9 inches along transverse direction and 0.1 inches along vertical direction, respectively. The longitudinal displacement will be used to evaluate the seat width at Pier 1 of the truss approach.

3.3.2 Responses of the Foundation Pile Groups

The bascule pile foundation consists of 315 timber piles supporting a 6-foot thick footing above a 15 foot thick seal. The embedment of the timber piles is 6 inches. Tables 3.5 and 3.6 list dead load, earth pressure due to lateral spreading and 190 year earthquake load on the bascule pile foundation with and without soil liquefaction. Dead load in the tables is the result of a lengthy hand calculation in Appendix A. The maximum tension, compression and shear in the pile due to different load combinations are less than the capacities of the pile whether soil liquefaction occurs or not. The largest D/C ratio for compression is 0.91 under MLLW condition when soil liquefaction does not occur. The largest D/C ratio for tension is 0.77 under MHHW condition without soil liquefaction. The maximum D/C ratio for shear is 0.73 under MHHW condition after liquefaction occurs. Soil liquefaction increases the shear demand on the pile foundation group due to the loss

of passive earth resistance. Soil liquefaction also reduces the rigidity of the system, which causes a reduction of the horizontal seismic forces and the moment demand on the system. Figure 3.11 plots the nonlinear moment and rotation curve of the foundation pile group obtained from pushover analysis. The moment capacity about the transverse axis, My, is 320,000 kft, about twice of the maximum moment due to earthquake. It is noted that the contribution from earth pressure due to lateral spreading is quite small. Therefore, its effect will be ignored in the evaluation of other members discussed subsequently.

3.3.3 Responses of the Footing and Seal

Tables 3.7 to 3.9 summarize the loads, the maximum tensile and compressive stresses in the footing and the seal for different water levels and soil conditions. It is observed that maximum tensile stress of 33.9 psi occurs under MHHW condition when soil liquefaction does not occur. This tensile stress is less than the capacity of 49 psi. At the interface of the footing and the seal, the maximum tensile stress is 22.8 psi. When zero tensile capacity is assumed for the interface, the corresponding maximum tensile force of 31 kips will act at the top portion of the pile. With only 6 inches of embedment into the footing, the pile may not be able to develop such a tensile capacity from the embedment. Compressive stresses are generally small. Shear demand is also much smaller than shear capacity.

Figures 3.12 and 3.13 show the footing moment capacity calculated under different axial loads using a computer program called X Section. This program assumes zero tensile capacity of the cracked concrete. Under axial load of 11,720 kips caused by dead load, the moment capacity is 263,587 kft. Under reduced axial load of 7757 kips, the moment

capacity is 174,530 kft, about twice of the maximum moment of 95,884 occurring at the interface between the seal and the footing. Therefore, the tensile stress evaluation approach seems conservative.

If there is a separation between the footing and the seal, the footing and the bascule pier walls should be considered as a horizontal beam subjected to bending. If the effect of the vertical cracks in the north and the south walls is ignored, the footing and the north and south walls form an invert "T" section to resist bending transferring from the east and west walls. Obviously, this large "T" section has enough capacity to resist the bending. However, if full-depth vertical cracks exist in the north and south walls, the footing itself may not have sufficient capacity to resist the bending moment.

3.3.4 Responses of the East and West Bascule Walls

The east and west bascule walls, each 11 foot thick, are basically plain concrete blocks. Tables 3.10 to 3.12 show the loads, the maximum tensile and compressive stresses in the east and west bascule walls for different water levels and soil conditions. Compressive stresses are generally small. Maximum shear demand is 1498 kips, less than shear capacity of 3444 kips. Maximum tensile stress of 74 psi occurs at elevation −12.98 feet under MHHW condition before soil liquefaction occurs. This tensile stress is greater than the capacity of 49 psi. Therefore, cracks may occur.

Figures 3.14 and 3.15 show the moment capacities of the east and west walls obtained by X Section. The moment capacities are 18,280 kft about the longitudinal axis and 43,513 kft about the transverse axis, respectively. The capacities are more than twice of

the moments caused by earthquake at elevation −12.98 feet. Again, the stress evaluation approach seems very conservative as far as capacity is concerned.

It is observed that the liquefaction case is less critical for members above the footing. Therefore, only the no liquefaction case will be discussed subsequently for other members above the footing.

3.3.5 Responses of the North and South Bascule Walls

The north and south walls, each 3 or 4 foot thick are lightly reinforced concrete walls. They are modeled as plate elements in the analysis. The maximum shear forces at horizontal sections are tabulated in Table 3.13. Also listed in the table are capacities and D/C ratios of the walls. The D/C ratios are 0.3 for the 3-foot wall and 0.4 for the 4-foot wall, respectively. Around the opening of the 3-foot wall, there is severe concrete spalling and deterioration. Therefore, the real D/C ratio of the 3 foot wall is larger than 0.3.

3.3.6 Responses of the Bascule Walls Considering Existing Vertical Cracks

Severe vertical cracks exist in the north and south bascule walls as discussed in Chapter 1. These cracks can significantly reduce the connectivity between the east and west walls. Therefore, the four bascule walls can no longer form a box structure to effectively resist the seismic load. This will result in a significant increase of moments in the east and west walls under transverse seismic ground motion. To evaluate the impact of these cracks, four elements in the 3 foot wall and six elements in the 4 foot wall are eliminated to open a

vertical crack in each of the two walls. Appropriate concentrated masses are added to the model.

Table 3.14 shows the loads, the maximum tensile and compressive stresses in the east and west bascule walls. Compressive stresses are still small. Shear demand changed little. However, the maximum tensile stress has increase from 74 psi to 107 at elevation –12.98 foot under MHHW condition before liquefaction occurs. So cracks are more likely to occur when there are vertical cracks in the north and south walls.

3.3.7 Forces in the Counterweight and Track Girder Restrainers

The bascule span is anchored down by the truss approach span. During an earthquake, the counterweight has a tendency to move up, down and transversely. Additional restraint is necessary to prevent the counterweight from moving down and transversely. Analyses are conducted with restrainers, modeled as beam members, placed in appropriate locations. Table 3.15 shows a vertical force of 249 kips and a horizontal force of 39 kips in the restrainer for a 190 year earthquake.

Table 3.15 also lists the forces transferring to the track from the track girder for the 190 year earthquake. The maximum longitudinal and transverse forces are 254 kips and 253 kips, respectively. The minimum vertical force is 580 kips. It can be calculated that the required friction coefficient for no relative movement between the segmental girder and the track is 0.44, which exceeds the friction coefficient for steel on steel. Therefore, additional restraint is necessary.

3.3.8 Evaluation of the Superstructure

The bascule span superstructure is in good condition. However, there are components where retrofit is obviously necessary as shown in Figure 1.5. The top laterals between the counterweight and the Frame 0 consists of single angels of $L3^1/2x3^1/2$. Another angle can be added to each lateral to make them double angles. Frame 0 transfers the vertical load of the bascule span to the tracks. This frame does not have a lateral load path to transfer lateral earthquake loads to the tracks. New diagonals can be added to the top portion of the frame. The truss at the lower portion of the frame will need to be strengthened.

3.4 Performance Evaluation for 95 Year Earthquake

From the above discussion, it is known that for the bascule pier, only the east and west walls do not have enough capacity to resist the 190 year earthquake. Tables 3.16 and 3.17 summarize the loads, the maximum tensile and compressive stresses in the east and west bascule walls subjected to 95 year earthquake. Maximum tensile stress of 54 psi occurs at elevation –12.98 feet under MHHW condition. This tensile stress will increase to 79 psi when full height vertical cracks in the north and south walls separate the upper part of the bascule into two portions. Therefore, horizontal cracks in the east and west walls may develop.

Table 3.18 lists the forces in the counterweight restrainers subjected to 95 year earthquake. The maximum vertical and horizontal forces per restrainer are 187 kips and 26 kips, respectively. Also listed in the table are forces transferred from the track girders

to the tracks. The maximum longitudinal and transverse forces per girder are 191 kips and 190 kips, respectively. For minimum vertical force of 647 kips, the required friction coefficient without additional restraint on the girder is 0.3.

3.5 Damage State of the Bascule Span

The damage state of the existing bascule span under different levels of earthquakes is summarized in Chapter 8.

Table 3.1 Bascule Span Soil Spring Constants

Pile Foundation					
Longitudinal Translation (kips/in)	Transverse Translation (kips/in)	Vertical Translation (kips/in)	Rocking about Longitudinal Axis (k-in/rad)	Rocking about Transverse Axis (k-in/rad)	Rotation about Vertical Axis (k-in/rad)
2813	2813	72106	8.81E+09	5.30E+09	4.63E+08
North-South Soil Passive Resistance on Footing and Seal without Liquefaction					
Elevation (ft)	-26.98	-29.98	-37.48	-44.98	
Spring (kips/in)	69	35	948	216	
East-West Soil Passive Resistance on Footing and Seal without Liquefaction					
Elevation	-26.98	-29.98	-37.48	-44.98	
Spring (kips/in)	48	24	664	151	
North-South Soil Passive Resistance on Footing and Seal with Liquefaction					
Elevation (ft)	-26.98	-29.98	-37.48	-44.98	
Spring (kips/in)	0	0	52	156	
East-West Soil Passive Resistance on Footing and Seal with Liquefaction					
Elevation	-26.98	-29.98	-37.48	-44.98	
Spring (kips/in)	0	0	37	109	

Table 3.2 Bascule Span Structural Dynamic Characteristics (No Soil Liquefaction)

(under MHHW condition)

MODE	FREQUENCY (CYC/SEC)	PERIOD (SEC)
1	1.24	0.8
2	1.28	0.78
3	1.28	0.78
4	1.3	0.77
5	2.64	0.38
6	5.6	0.18
7	5.65	0.18
8	6.1	0.16
9	6.1	0.16
10	6.31	0.16
11	6.35	0.16
12	11.8	0.084
13	11.9	0.084
14	20.1	0.05
15	20.4	0.049
16	25.4	0.039
17	25.4	0.039
18	34.2	0.029
19	34.3	0.029
20	37.7	0.027
21	37.7	0.027
22	43.7	0.023
23	43.8	0.23
24	49.5	0.02
25	49.8	0.02
26	49.8	0.02
27	50.8	0.02
28	51.9	0.019
29	61.4	0.016
30	61.6	0.016

X DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	1.40E-04	2	6.08E-04	3	99.58	4	9.60E-06
5	3.35E-04	6	5.24E-06	7	0.42	8	1.83E-07
9	3.15E-03	10	1.84E-09	11	2.64E-05	12	6.49E-10
Y DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	99.82	2	2.14E-11	3	1.46E-04	4	2.59E-07
5	1.02E-02	6	3.68E-09	7	3.58E-06	8	6.44E-08
9	5.95E-04	10	1.81E-05	11	0.17	12	2.16E-06
Z DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	2.03E-15	2	9.30E-05	3	5.83E-10	4	1.84E-06
5	2.66E-10	6	1.126651	7	1.61E-05	8	98.41
9	6.19E-03	10	0.44	11	3.02E-05	12	3.47E-04

Table 3.3 Bascule Span Structural Dynamic Characteristics (with Soil Liquefaction)

(under MHHW Condiiton)

MODE	FREQUENCY (CYC/SEC)	PERIOD (SEC)
1	1.03	0.97
2	1.03	0.97
3	1.06	0.95
4	1.12	0.89
5	2.64	0.38
6	5.55	0.18
7	5.59	0.18
8	6.10	0.16
9	6.10	0.16
10	6.28	0.16
11	6.32	0.16
12	11.80	0.09
13	11.90	0.08
14	20.10	0.05
15	20.40	0.049
16	25.40	0.039
17	25.40	0.039
18	34.20	0.029
19	34.30	0.029
20	37.70	0.027
21	37.70	0.027
22	43.70	0.023
23	43.80	0.023
24	49.50	0.020
25	49.80	0.020
26	49.80	0.020
27	50.80	0.020
28	51.90	0.019
29	61.40	0.016
30	61.60	0.016

X DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	3.56E-04	2	99.79	3	1.33E-04	4	1.41E-07
5	8.30E-05	6	2.51E-06	7	0.2	8	5.66E-08
9	1.17E-03	10	8.93E-10	11	1.23E-05	12	3.09E-10
Y DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	1.28E-09	2	1.31E-04	3	99.89	4	1.94E-07
5	4.92E-03	6	1.91E-09	7	2.07E-06	8	4.20E-08
9	4.25E-04	10	1.01E-05	11	9.94E-02	12	1.32E-06
Z DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	1.28E-09	2	1.31E-04	3	99.89	4	1.94E-07
5	4.92E-03	6	1.91E-09	7	2.07E-06	8	4.20E-08
9	4.25E-04	10	1.01E-05	11	9.94E-02	12	1.32E-06

Table 3.4 Displacements of Bascule Span at EL 12.12 ft

(under 190 Years EQ & MHHW condition)

Soil Condition	Longitudinal (in)	Transverse (in)	Vertical (in)
190 Years Earthquake			
No Liquefaction	1.7	1.7	0.10
Liquefaction Occurs	2	1.9	0.10
95 Years Earthquake			
No Liquefaction	1.3	1.3	0.08
Liquefaction Occurs	1.5	1.4	0.08

Table 3.5 Evaluation of Bascule Pile Foundation
(under 190 Years EQ)

		Under MHHW					Under MLLW				
		DL	Earth-P	Case-1	Case-2	Case-3	DL	Earth-P	Case-1	Case-2	Case-3
Pile Group Loads	Vx (kips)	0.0	0.0	2900.9	876.8	893.2	1	0	2842.0	859.6	877.9
	Vy (kips)	0.0	0.0	939.0	3119.9	943.4	0	0	918.3	3053.0	920.8
	Fz (kips)	11720.0	0.0	1411.2	1390.2	4580.5	12624	0	1352.7	1327.3	4380.3
	Mx (kip-ft)	8	0	44171	143948	47046	-1026.6	0.0	42445.6	139300.1	44231.1
	My (kip-ft)	3516	0	151385	47685	54816	-8540.8	0.0	147470.4	46675.9	54313.0
	Mz (kip-ft)	0	0	2281	2683	1235	2.4	0.0	2039.8	3029.0	1225.1
Single Pile Loads	Fz (kips)	38.6	0.0	77.1	64.3	49.7	43.8	0.0	74.9	62.3	48.0
	Vx (kips)	0.0	0.0	9.2	2.8	2.8	0.0	0.0	9.0	2.7	2.8
	Vy (kips)	0.0	0.0	3.0	9.9	3.0	0.0	0.0	2.9	9.7	2.9
		DL+EP+Max(Case1, Case2, Case3)					DL+EP+Max(Case1, Case2, Case3)				
	Fz (kips)	116					**119**				
	Fz (kips)	**-39**					-31				
	V (kips)	**10.3**					10.1				

Maximum Pile Forces			Pile Capacities			**D/C Ratios**		
Compression	Tension	Shear	Compression	Tension	Shear	Compression	Tension	Shear
119	39	10.3	130	50	17.9	**0.91**	**0.77**	**0.57**

Section Properties

Pile Group Properties		
# of Piles	Sx (ft)	Sy (ft)
315	3517	2520

Table 3.6 Evaluation of Bascule Pile Foundation
(under 190 Years EQ Return Period & with Liquefation)

		Under MHHW					Under MLLW				
		DL	Earth-P	Case-1	Case-2	Case-3	DL	Earth-P	Case-1	Case-2	Case-3
Pile Group Loads	Vx (kips)	0.0	-221.5	3738.1	1128.3	1146.8	0	-140	3668.6	1108.1	1128.6
	Vy (kips)	0.0	0.0	1125.0	3740.3	1130.3	0	0	1101.9	3665.7	1105.0
	Fz (kips)	11720.0	0.0	1399.7	1385.3	4578.1	12624	0	1340.5	1323.0	4378.2
	Mx (kip-ft)	8	0	39059	126954	42018	-1026.8	0.0	37502.0	122899.6	39329.7
	My (kip-ft)	3516	-2436	127913	40550	47523	-8555.1	-1874.3	124751.3	39763.6	47235.0
	Mz (kip-ft)	0	0	1686	1961	933	1.5	0.0	1524.3	2229.2	922.7
Single Pile Loads	Fz (kips)	38.6	1.0	66.3	56.6	45.3	43.8	0.7	64.4	54.9	43.8
	Vx (kips)	0.0	0.7	11.9	3.6	3.6	0.0	0.4	11.6	3.5	3.6
	Vy (kips)	0.0	0.0	3.6	11.9	3.6	0.0	0.0	3.5	11.6	3.5
		DL+EP+Max(Case1, Case2, Case3)					DL+EP+Max(Case1, Case2, Case3)				
	Fz (kips)	106					**109**				
	Fz (kips)	-18					**-21**				
	V (kips)	**13.1**					12.6				

Maximum Pile Forces			Pile Capacities			**D/C Ratios**		
Compression	Tension	Shear	Compression	Tension	Shear	Compression	Tension	Shear
109	21	13.1	130	50	17.9	**0.84**	**0.41**	**0.73**

Section Properties

Pile Group Properties		
# of Piles	Sx (ft)	Sy (ft)
315	3517	2520

Table 3.7 Responses of Footing and Seal

(under 190 Years EQ & MHHW Condition)

Footing and Seal Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2001	11720.0	1.7	0.4	282	9899	1411.2	3201.8	1007.5	44171	151385	1390.2	967.8	3347.5	143948	47685	-33.9	87.8	-21.9	75.8
2002	9244.7	0.4	0.4	285	9912	1163.4	3864.9	1097.8	36634	127436	1144.9	1181.7	3633.7	118882	40453	-30.3	72.8	-19.8	62.3
2003	7756.7	0.3	0.5	288	9915	939.8	3234.3	906.8	28423	98526	923.6	1002.6	2987.9	91677	31621	-22.8	58.5	-14.1	49.8
2004	7756.7	0.5	0.0	9916	289	939.8	929.9	3338.9	88844	25709	923.6	3064.5	1034.0	28624	82728	-12.8	48.4	-15.5	51.2
2036	7756.7	0.5	0.0	9916	291	939.8	929.9	3338.9	78855	22929	923.6	3064.5	1034.0	25535	73553	-9.8	45.5	-12.1	47.8

Max. Shear Force = 4018 kips < Shear Capacity = 22621 kips

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Section Properties

Footing & Seal Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
3021	33200	23200

Joint Elevations

Joint	2001	2002	2003	2004	2005
Elevation(ft)	-44.98	-37.48	-29.98	-26.98	-23.98

Table 3.8 Responses of Footing and Seal

(under 190 Years EQ & MLLW Condition)

Footing and Seal Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2001	12614.0	1.5	0.1	1027	8541	1352.7	3136.9	985.3	42446	147470	1327.3	948.8	3275.7	139300	46676	-29.9	87.9	-18.4	76.4
2002	10139.1	0.4	0.0	1027	8554	1104.2	3760.2	1062.7	35074	124009	1082.4	1152.1	3524.1	114772	39587	-26.5	73.1	-16.4	63.0
2003	8651.1	0.3	0.0	1027	8555	880.1	3112.7	865.0	27124	95884	861.4	968.6	2859.7	88386	30980	-19.3	59.1	-11.0	50.7
2004	8651.1	0.0	0.0	8555	1027	880.1	887.6	3215.6	86567	24535	861.4	2935.1	999.5	28085	79821	-9.7	49.5	-12.3	52.1
2036	8651.1	0.0	0.0	8555	1027	880.1	887.6	3215.6	76948	21880	861.4	2935.1	999.5	25100	71034	-6.9	46.7	-9.1	48.8

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Section Properties

Footing & Seal Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
3021	33200	23200

Joint Elevations

Joint	2001	2002	2003	2004	2005
Elevation(ft)	-44.98	-37.48	-29.98	-26.98	-23.98

Table 3.9 Reponses of Bascule Footing and Seal

(under 190 Years EQ, MHHW Condition & with Soil Liquefaction)

Footing and Seal Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2001	10191.7	0.0	0.4	282	9916	1399.7	3738.1	1125.0	39059	127913	1385.3	1128.3	3740.3	126954	40550	-29.3	76.1	-19.9	66.7
2002	9244.7	0.0	0.4	285	9916	1153.8	3149.7	940.7	30637	99940	1140.9	966.9	3111.2	98939	32112	-20.8	63.3	-12.9	55.4
2003	7756.7	0.0	0.4	289	9916	932.1	2607.6	772.0	23596	76369	920.3	813.8	2539.2	75640	24882	-15.1	50.8	-8.5	44.2
2004	7756.7	0.4	0.0	9916	290	932.1	772.0	2607.6	68562	21285	920.3	2539.2	813.8	22448	68033	-7.2	42.8	-9.8	45.4
2036	7756.7	0.4	0.0	9916	291	932.1	772.0	2607.6	60758	18975	920.3	2539.2	813.8	20016	60430	-4.9	40.5	-7.0	42.6

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Section Properties

Footing & Seal Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
3021	33200	23200

Joint Elevations

Joint	2001	2002	2003	2004	2005
Elevation(ft)	-44.98	-37.48	-29.98	-26.98	-23.98

Table 3.10 Responses of Bascule East and West Walls

(under 190 Years EQ & MHHW Condition)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2580.6	23.3	25.0	7201	343	772.3	298.1	1314.3	32179	3414	1630.0	960.8	448.2	12719	11020	-69	132	-58	133
2131	2312.2	0.5	23.9	7413	418	670.0	215.5	1154.5	25455	2113	1406.9	684.5	394.1	9867	6726	-61	122	-44	112
2231	2013.5	11.6	24.4	7785	601	545.8	133.7	951.1	20004	1372	1112.7	396.0	325.9	7170	4327	-67	128	-38	99
2331	1696.4	13.1	26.2	8341	866	401.6	67.0	665.3	14130	960	777.3	154.1	232.9	5387	2713	-54	106	-21	72
2431	1367.6	35.4	30.1	8870	1223	323.4	78.0	523.9	10660	1118	570.9	185.2	195.0	5374	2592	-54	95	-22	63
2531	1231.8	45.1	29.9	9712	1348	254.5	92.7	455.8	9217	735	468.7	221.2	167.8	6006	2143	-51	88	-21	58
2032	3140.1	4.2	19.8	8959	463	807.5	326.4	1462.1	35460	4455	1667.0	1067.7	493.7	13782	14443	-67	133	-56	131
2132	2864.3	30.7	22.7	8740	577	707.9	243.9	1303.7	28058	2971	1455.2	790.9	440.3	10737	9488	-61	127	-45	117
2232	2549.3	67.0	25.0	8566	1020	583.1	160.0	1101.6	21957	2064	1164.8	492.9	372.9	7846	6472	-74	145	-51	122
2332	2202.7	95.3	26.8	8442	1585	431.9	84.4	818.4	15052	1312	821.2	235.5	281.3	5671	4105	-57	118	-27	88
2432	1840.3	107.8	27.7	8417	1800	341.4	124.0	678.6	10458	1123	603.3	272.4	243.2	5321	3363	-49	100	-23	74
2532	1682.9	102.0	27.1	8699	2097	265.6	132.9	611.8	8387	1271	496.8	307.2	216.9	5763	2784	-49	96	-20	67

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft)	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Table 3.11 Responses of Bascule East and West Walls

(under 190 Years EQ & MLLW Condition)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2957.6	30.2	24.0	7204	334	737.6	275.2	1246.1	31201	3201	1578.1	902.1	435.4	12345	10451	-65	136	-54	136
2131	2689.3	3.0	22.8	7410	423	640.5	195.3	1081.8	24857	1999	1367.0	630.5	379.2	9637	6413	-57	126	-39	115
2231	2264.7	12.6	23.3	7779	615	523.2	118.1	885.2	19800	1334	1089.2	350.1	311.7	7093	4222	-62	131	-33	101
2331	1697.2	16.4	25.1	8339	899	399.7	70.3	670.2	14435	1014	780.6	165.0	238.5	5461	2926	-56	107	-23	74
2431	1367.5	39.6	29.0	8863	1224	323.5	80.1	535.4	10914	1166	574.5	198.5	198.3	5376	2738	-55	96	-23	64
2531	1231.8	49.3	28.8	9701	1331	254.8	93.4	467.6	9423	773	471.7	232.9	172.4	5986	2234	-52	89	-21	59
2032	3520.2	11.2	18.6	8938	492	779.3	305.6	1401.5	34583	4194	1621.8	1003.3	483.4	13440	13729	-64	136	-52	134
2132	3244.5	28.1	21.5	8723	553	683.8	223.8	1238.2	27527	2817	1420.6	731.0	427.9	10526	9073	-57	130	-41	121
2232	2802.9	68.3	23.9	8555	1000	564.6	142.5	1042.5	21792	2004	1144.9	440.3	361.2	7778	6318	-69	147	-45	124
2332	2204.7	99.0	25.8	8446	1575	432.9	85.9	829.2	15354	1383	826.9	241.1	289.0	5750	4332	-58	120	-29	90
2432	1840.9	112.0	26.7	8427	1817	343.5	125.9	695.7	10665	1158	609.1	282.2	248.7	5307	3540	-50	101	-24	75
2532	1683.4	106.1	26.0	8711	2134	267.4	133.1	629.2	8517	1317	501.7	315.2	223.8	5713	2938	-50	97	-21	68

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft)	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Table 3.12 Responses of Bascule East and West Walls

(under 190 Years EQ, MHHW Condition & with Soil Liquefaction)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2580.6	23.3	24.9	7201	343	681.5	248.3	1021.7	24820	2831	1384.2	796.0	349.5	10080	9099	-55	117	-46	117
2131	2312.2	0.5	23.9	7413	418	590.1	178.8	895.9	19578	1749	1193.5	566.3	307.2	7802	5542	-48	108	-33	99
2231	2013.5	11.6	24.4	7785	601	480.1	110.1	736.5	15330	1136	944.0	326.9	254.0	5639	3556	-52	113	-25	86
2331	1696.4	13.1	26.2	8341	866	350.7	54.0	513.5	10790	792	658.4	126.3	182.6	4253	2221	-43	95	-12	64
2431	1367.6	35.4	30.0	8870	1223	281.3	63.1	404.1	8150	913	484.6	151.5	154.8	4316	2121	-45	86	-14	55
2531	1231.8	45.1	29.9	9712	1348	224.9	75.1	351.1	7080	617	399.6	180.9	132.7	4879	1761	-44	81	-14	51
2032	3140.1	4.2	19.7	8958	463	717.8	271.0	1139.2	27389	3692	1421.9	884.0	385.9	10922	11921	-54	118	-44	117
2132	2864.3	30.7	22.6	8740	577	628.7	202.3	1013.9	21608	2458	1240.4	654.2	344.0	8492	7816	-48	113	-34	104
2232	2549.3	67.0	24.9	8565	1020	518.2	132.9	854.8	16846	1704	993.9	407.7	291.2	6176	5318	-57	128	-35	106
2332	2202.7	95.3	26.8	8442	1585	383.0	70.5	633.1	11488	1086	701.1	194.9	220.3	4467	3361	-45	107	-16	78
2432	1840.3	107.8	27.7	8417	1800	303.5	100.9	524.5	7964	934	517.5	224.2	192.2	4261	2756	-41	92	-14	65
2532	1682.9	102.0	27.1	8699	2097	241.1	108.0	472.3	6396	1042	428.5	252.3	170.7	4670	2280	-42	89	-12	59

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft)	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Table 3.13 Responses of Bascule North and South Walls
(under MHHW Condition)

(EQ of 190 Years Return Period)

Loads Under MHHW		
Load Case	3 foot wall	4 foot wall
	V (in plane) (kips)	V (in plane) (kips)
EQX	93	103
EQY	300	622
EQZ	32	38
Case-1	193	301
Case-2	338	664
Wall Shear Capacity		
Vc	907	1440
Vs	160	160
ϕVn	907	1360
D/C Ratio	0.4	0.5

(EQ of 95 Years Return Period)

Loads Under MHHW		
Load Case	3 foot wall	4 foot wall
	V (in plane) (kips)	V (in plane) (kips)
EQX	70	77
EQY	225	467
EQZ	24	29
Case-1	144	226
Case-2	253	498
Wall Shear Capacity		
Vc	907	1440
Vs	160	160
ϕVn	907	1360
D/C Ratio	0.3	0.4

Notes:

Case-1: 1.0 $EQ_{long.}$+0.3$EQ_{trans.}$+0.3 $EQ_{vert.}$

Case-2: 0.3 $EQ_{long.}$+ 1.0 $EQ_{trans.}$+0.3 $EQ_{vert.}$

Table 3.14 Responses of Bascule East and West Walls with Vertical Cracks in North and South Walls

(under 190 Years EQ & MHHW Condition)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2821.6	-4.5	20.5	8459	1830	608.5	273.1	1257.0	32103	5448	1168.5	895.0	438.2	10686	17178	-82	148	-74	148
2131	2547.3	-30.8	20.8	8728	1846	521.4	228.2	1096.5	26014	4191	999.1	743.2	384.4	9023	12942	-77	142	-64	133
2231	2205.7	27.5	22.2	8927	1623	424.3	177.9	919.2	20970	3213	800.5	567.0	324.1	7790	9675	-88	154	-68	135
2331	1818.9	10.0	21.9	9008	1377	312.9	114.6	677.5	14776	2065	582.7	340.9	246.6	5974	5983	-66	121	-40	95
2431	1432.8	1.9	21.7	8952	1444	258.6	97.9	553.4	10409	1342	455.4	269.7	208.5	4627	3842	-54	97	-25	69
2531	1263.0	13.6	24.1	9315	1597	212.5	83.2	492.1	8456	1247	401.1	218.3	187.7	4391	2905	-52	90	-20	58
2032	3218.9	8.9	19.8	9539	2387	828.8	411.0	1472.7	36621	7711	1743.8	1323.4	510.2	14249	24752	-84	151	-84	162
2132	2943.3	3.6	21.6	9430	2325	726.5	350.8	1329.0	29015	5862	1512.4	1100.8	459.9	11226	18520	-80	147	-78	152
2232	2616.7	5.6	22.4	9370	2341	595.5	277.7	1116.9	22556	4595	1185.5	827.7	388.0	8387	14066	-104	178	-107	180
2332	2242.8	-1.7	21.3	9341	2296	440.7	174.7	805.0	15091	3389	812.1	532.9	288.3	5343	9353	-79	142	-64	127
2432	1851.1	-14.8	19.1	9329	2166	352.9	230.6	652.8	10296	2631	574.6	553.6	236.9	3825	6385	-65	116	-41	93
2532	1681.2	-17.4	18.5	9463	2000	277.7	211.7	586.5	8275	1517	472.7	535.9	218.9	4044	4227	-52	99	-26	73

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft)	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Plain concrete tensile strength per ACI-318: $f_t = 5\phi\,(f'_c)^{0.5}$ $\phi = 0.65$, f'c=2500 psi, f_t =163 psi

Using 30% of f_t due to construction joint, f_{te} = 49 psi < f_{max} = 107 psi N.G.

Using 36mm HS bar (A722 type II) tiedown for tetrofit, assuming effective bar stree f_{be} = 100 ksi,

Total number of bar required, $N_b = A^*(f_{max} - f_{te}) / a_b{}^*f_{be}$ = 27

Max. Shear Demand = 1529 kips < Shear Capacity = 3444 kips

Table 3.15 Loads at Tracks and Anchorage Girders
(under 190 Years EQ)

Load Case	Under MHHW Condition						Under MLLW Condition					
	Anchorage Girder			Track			Anchorage Girder			Track		
	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)
DL	0	0	0	833	0	0	0	0	0	833	0	0
Case1	243	0	39	247	250	94	249	0	34	253	254	96
Caes2	234	0	32	171	108	248	236	0	27	175	106	253
Case3	117	0	17	131	90	81	119	0	15	132	91	83
DL+max.EQ	243	0	39	1080	250	248	249	0	34	1086	254	253
DL-max.EQ	243	0	39	586	250	248	249	0	34	580	254	253

Under 190 years EQ, required friction coefficient = 254/580 = 0.44

Table 3.16 Responses of Bascule East and West Walls

(under 95 Years EQ & MHHW Condition)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2580.6	23.3	25.0	7201	343	579.2	223.6	985.7	24135	2561	1222.5	720.6	336.2	9539	8265	-53	113	-41	111
2131	2312.2	0.5	23.9	7413	418	502.5	161.6	865.9	19092	1585	1055.2	513.4	295.6	7400	5045	-46	105	-29	93
2231	2013.5	11.6	24.4	7785	601	409.4	100.3	713.3	15003	1029	834.5	297.0	244.4	5377	3246	-49	110	-21	82
2331	1696.4	13.1	26.2	8341	866	301.2	50.3	498.9	10597	720	583.0	115.6	174.7	4040	2035	-42	93	-9	60
2431	1367.6	35.4	30.1	8870	1223	242.5	58.5	392.9	7995	839	428.2	138.9	146.3	4031	1944	-43	84	-11	52
2531	1231.8	45.1	29.9	9712	1348	190.9	69.5	341.8	6913	551	351.5	165.9	125.9	4504	1607	-43	80	-11	48
2032	3140.1	4.2	19.8	8959	463	605.6	244.8	1096.6	26595	3341	1250.3	800.8	370.3	10337	10832	-51	115	-39	110
2132	2864.3	30.7	22.7	8740	577	530.9	182.9	977.7	21043	2228	1091.4	593.2	330.2	8053	7116	-46	110	-30	99
2232	2549.3	67.0	25.0	8566	1020	437.3	120.0	826.2	16468	1548	873.6	369.7	279.7	5884	4854	-54	125	-29	100
2332	2202.7	95.3	26.8	8442	1585	323.9	63.3	613.8	11289	984	615.9	176.6	211.0	4253	3078	-43	105	-12	74
2432	1840.3	107.8	27.7	8417	1800	256.0	93.0	508.9	7844	842	452.5	204.3	182.4	3991	2523	-39	90	-11	62
2532	1682.9	102.0	27.1	8699	2097	199.2	99.7	458.8	6290	953	372.6	230.4	162.7	4322	2088	-40	87	-9	56

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft)	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Plain concrete tensile strength per ACI-318: $f_t = 5\phi\,(f'_c)^{0.5}$ $\phi = 0.65$, f'c=2500 psi, f_t =163 psi

Using 30% of f_t due to construction joint, f_{te} = 49 psi < f_{max} = 54 psi N.G.

Using 36mm HS bar (A722 type II) tiedown for tetrofit, assuming effective bar stree f_{be} = 100 ksi,

Number of bar required per wall, $N_b = A*(f_{max} - f_{te}) / a_b*f_{be}$ = 3

Max. Shear Demand = 1124 kips < Shear Capacity = 3444 kips

Table 3.17 Responses of Bascule East and West Walls with Vertical Cracks in North and South Walls

(under 95 Years EQ & MHHW Condition)

East Wall & West Wall Forces, Tensile and Compressive Stresses

Joint	DL					Case1: 1.0Ex+0.3Ey+0.3Ez					Case2: 0.3Ex+1.0Ey+0.3Ez					DL & Case1		DL & Case2	
	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	fmax (psi)	f'max (psi)	fmax (psi)	f'max (psi)
2031	2821.6	-4.5	20.5	8459	1830	456.4	204.8	942.8	24078	4086	876.4	671.2	328.7	8014	12884	-64	128	-53	123
2131	2547.3	-30.8	20.8	8728	1846	391.1	171.1	822.4	19511	3144	749.3	557.4	288.3	6767	9707	-60	123	-44	111
2231	2205.7	27.5	22.2	8927	1623	318.2	133.4	689.4	15728	2410	600.4	425.2	243.1	5842	7256	-66	133	-43	109
2331	1818.9	10.0	21.9	9008	1377	234.6	86.0	508.1	11082	1549	437.0	255.7	185.0	4481	4487	-51	106	-23	78
2431	1432.8	1.9	21.7	8952	1444	194.0	73.4	415.0	7807	1007	341.5	202.3	156.4	3470	2881	-44	87	-14	57
2531	1263.0	13.6	24.1	9315	1597	159.4	62.4	369.1	6342	935	300.8	163.7	140.8	3293	2178	-43	82	-10	48
2032	3218.9	8.9	19.8	9539	2387	621.6	308.2	1104.5	27465	5783	1307.8	992.6	382.6	10686	18564	-65	130	-61	134
2132	2943.3	3.6	21.6	9430	2325	544.8	263.1	996.7	21761	4397	1134.3	825.6	344.9	8419	13890	-62	127	-54	125
2232	2616.7	5.6	22.4	9370	2341	446.6	208.3	837.7	16917	3446	889.1	620.8	291.0	6290	10550	-79	153	-71	144
2332	2242.8	-1.7	21.3	9341	2296	330.5	131.0	603.8	11318	2541	609.1	399.6	216.2	4008	7015	-62	124	-40	103
2432	1851.1	-14.8	19.1	9329	2166	264.6	173.0	489.6	7722	1973	430.9	415.2	177.7	2869	4789	-52	104	-24	76
2532	1681.2	-17.4	18.5	9463	2000	208.2	158.8	439.9	6206	1137	354.5	401.9	164.2	3033	3170	-43	90	-14	61

Notes:

fmax: maximum tensile stress obtained from DL-EQ.

f'max: maximum compressive stress obtained from DL+EQ.

Joint Elevations

	West Wall						East Wall					
Joint	2031	2131	2231	2331	2431	2531	2032	2132	2232	2332	2432	2532
Elevation(ft	-23.98	-18.48	-12.98	-4.99	3	7.56	-23.98	-18.48	-12.98	-4.99	3	7.56

Wall Section Properties

West Wall, North Bascule					East Wall, North Bascule					Elevation
A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	A (ft^2)	Sy (ft^3)	Sz (ft^3)	e_y (ft)	e_z (ft)	(ft)
460	2537	1024	0	0	497	2692	957	0	0	-12.98
599	3730	1621	0.96	0.78	674	4193	1985	1.29	0.71	-18.48
676	4393	1952	1.49	1.21	773	5027	2556	2	1.11	-23.98

Plain concrete tensile strength per ACI-318: $f_t = 5\phi\,(f'_c)^{0.5}$ $\phi = 0.65$, f'c=2500 psi, f_t =163 psi

Using 30% of f_t due to construction joint, f_{te} = 49 psi < f_{max} = 79 psi N.G.

Using 36mm HS bar (A722 type II) tiedown for tetrofit, assuming effective bar stree f_{be} = 100 ksi,

Total number of bar required, $N_b = A^*(f_{max} - f_{te}) / a_b{}^*f_{be}$ = 15

Max. Shear Demand = 1147 kips < Shear Capacity = 3444 kips

Table 3.18 Loads at Tracks and Anchorage Girders
(under 95 Years EQ)

Load Case	Under MHHW Condition						Under MLLW Condition					
	Anchorage Girder			Track			Anchorage Girder			Track		
	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)	Vert. Fz (kips)	Long. Fx (kips)	Trans. Fy (kips)
DL	0	0	0	833	0	0	0	0	0	833	0	0
Case1	182	0	29	186	188	70	187	0	26	190	191	72
Caes2	175	0	24	128	81	186	177	0	20	132	80	190
Case3	88	0	13	98	67	61	90	0	11	99	68	63
DL+max.EQ	182	0	29	1019	188	186	187	0	26	1023	191	190
DL-max.EQ	182	0	29	647	188	186	187	0	26	643	191	190

Under 95 years EQ, required friction coefficient = 191/643 = 0.30

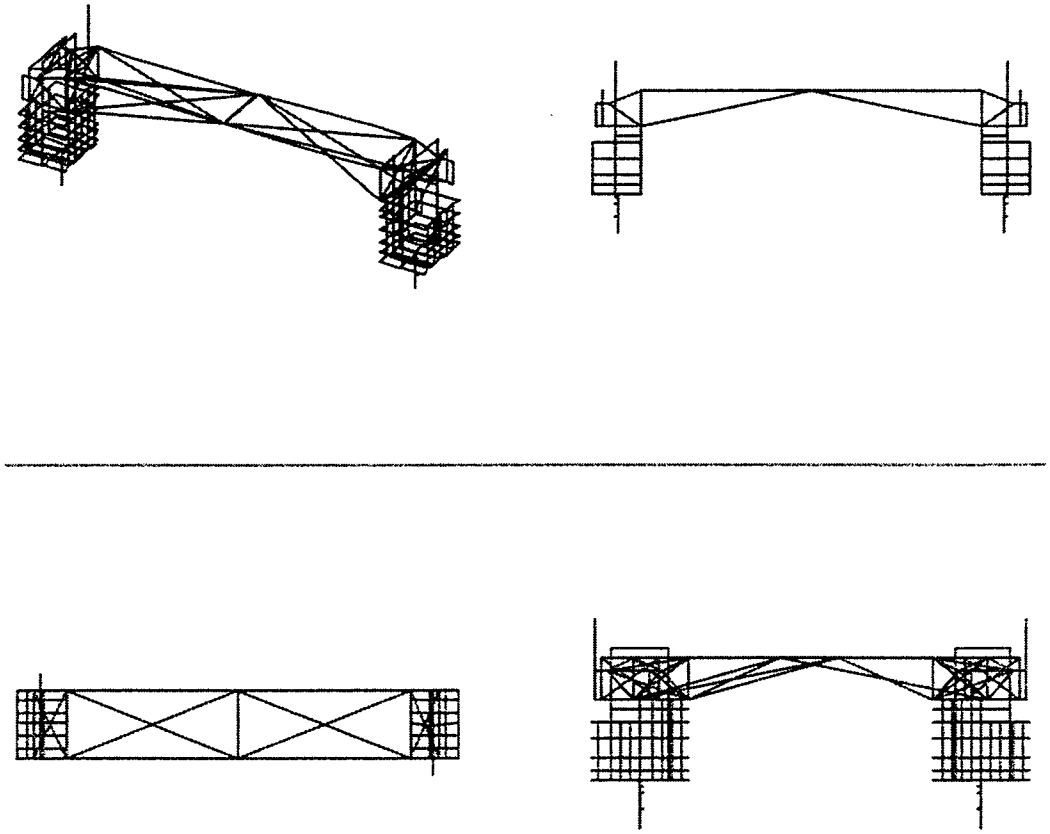
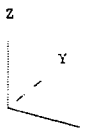


Figure 3.1 Bascule Span Gtstrudl Model



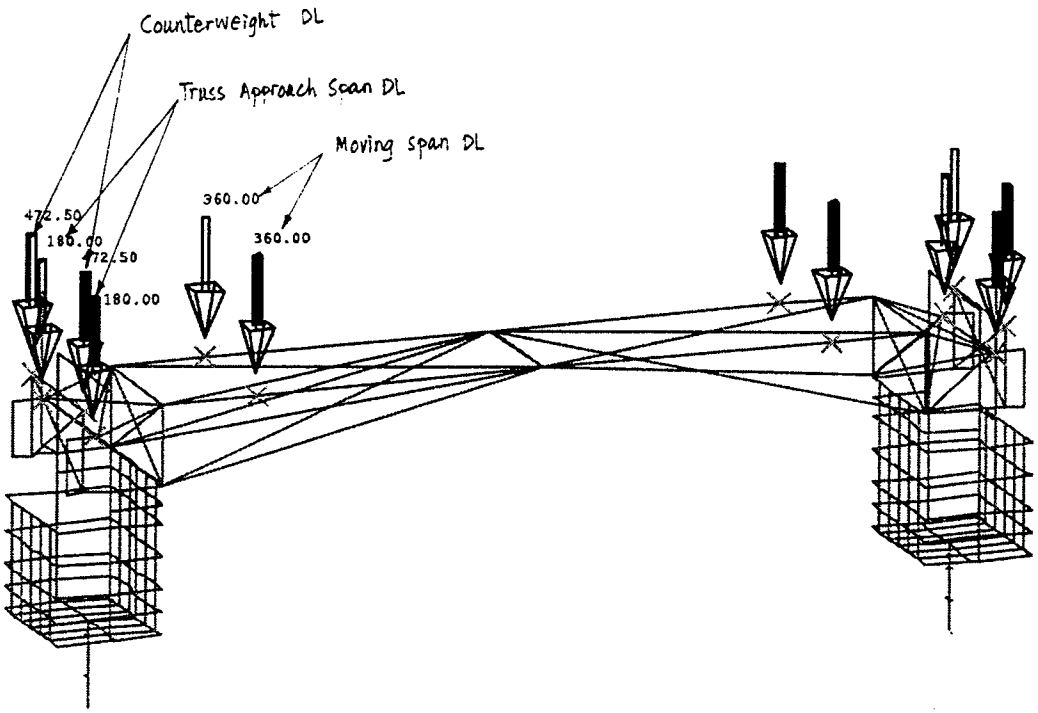
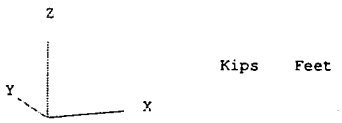


Figure 3.2 Modeling of Dead Load of Bascule Span Superstructure



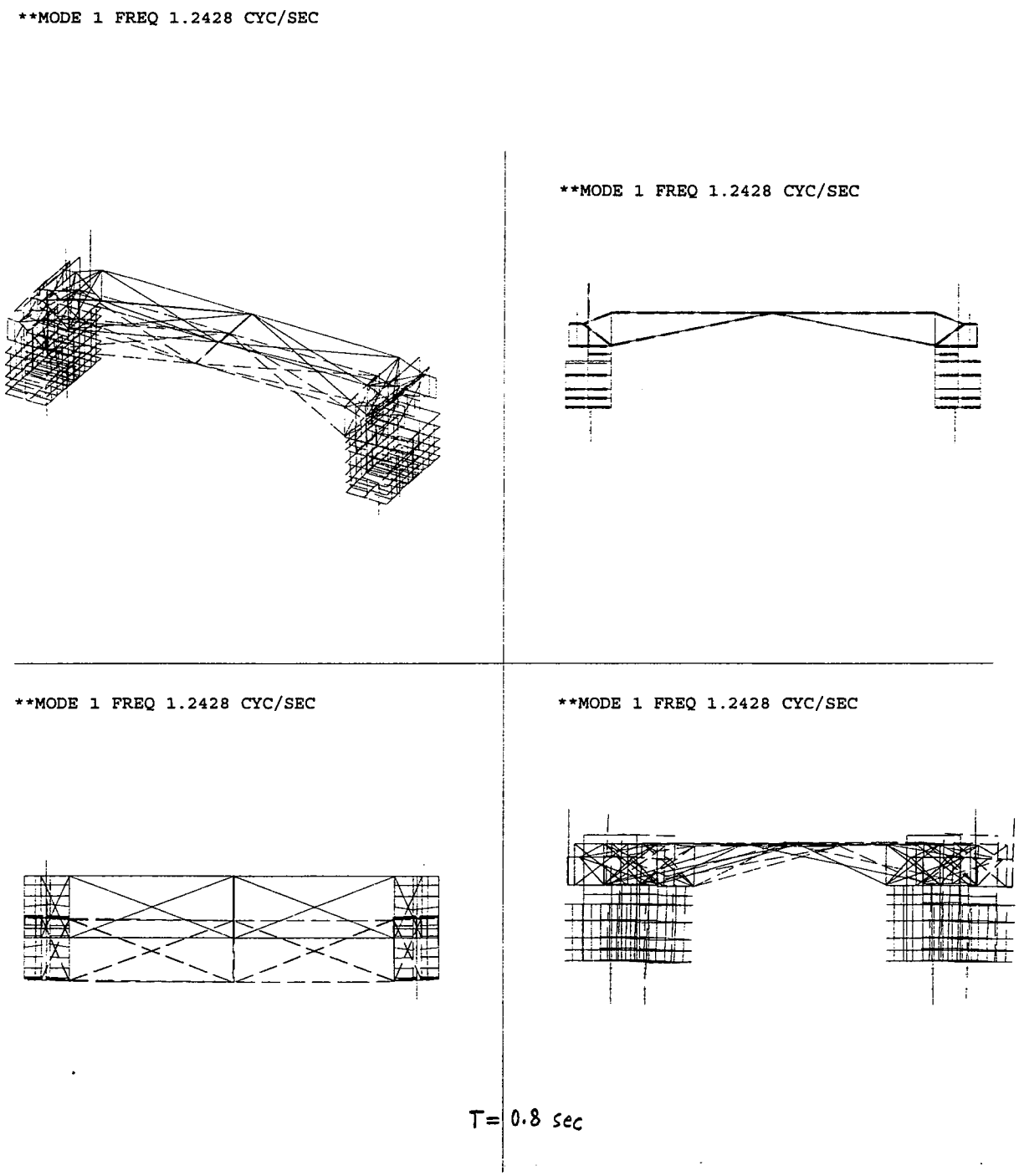


Figure 3.3 Bascule Span Mode Shape # 1 (under MHHW Condition)

Z
Y
X

Feet Kips

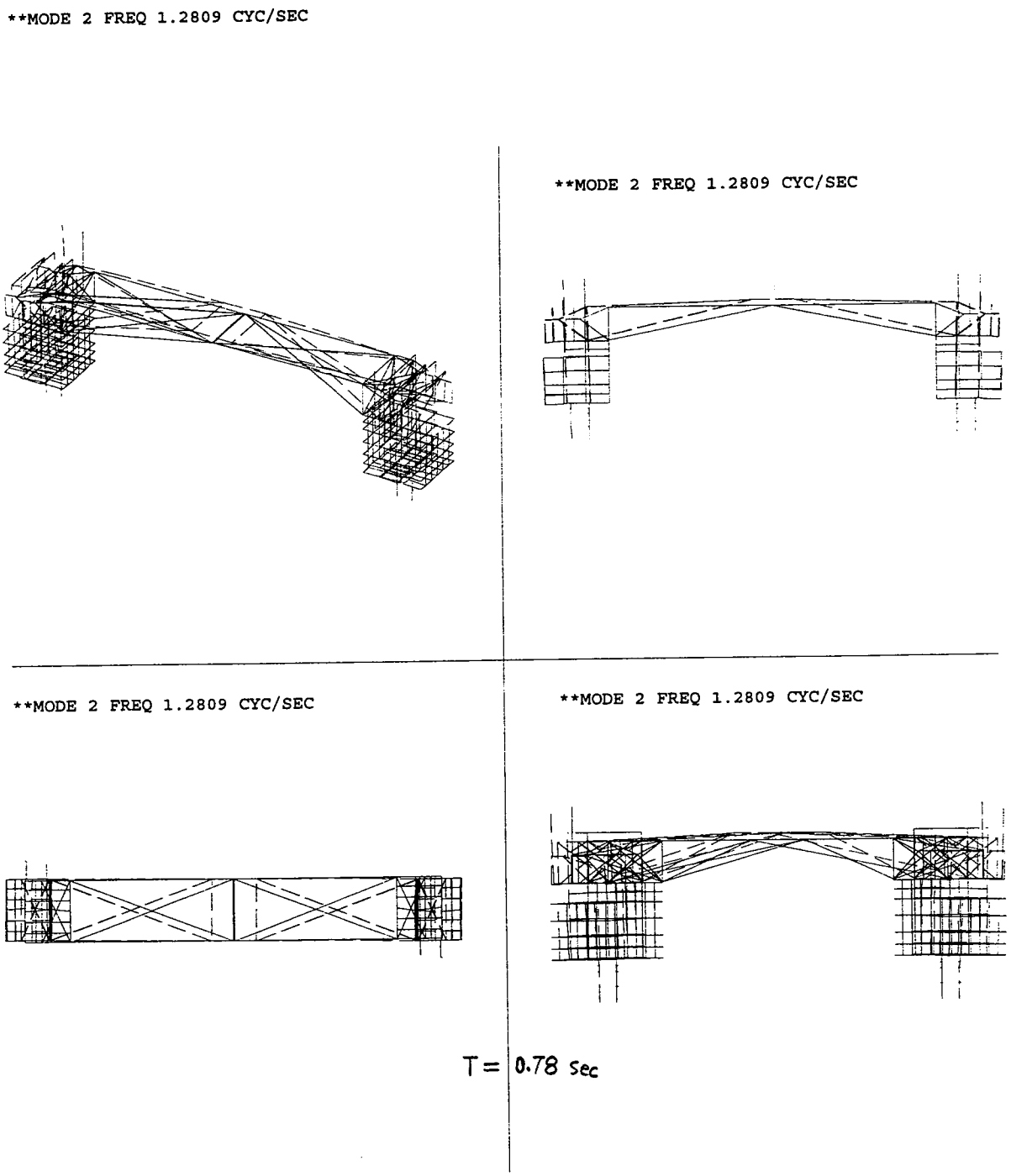


Figure 3.4 Bascule Span Mode Shape # 2 (under MHHW Condition)

Z
Y
X

Feet Kips

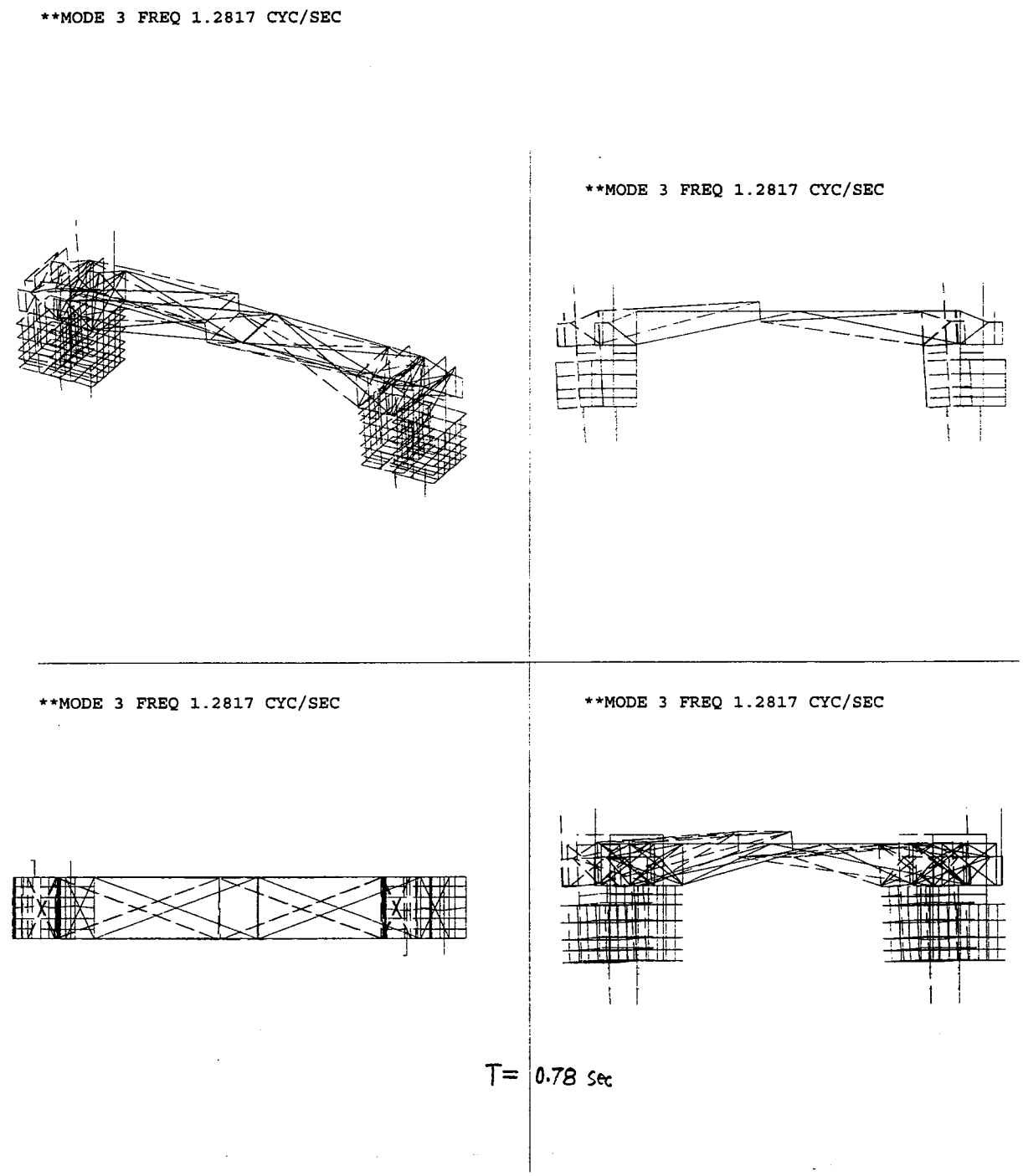


Figure 3.5 Bascule Span Mode Shape # 3 (under MHHW Condition)

Z
Y
X

Feet Kips

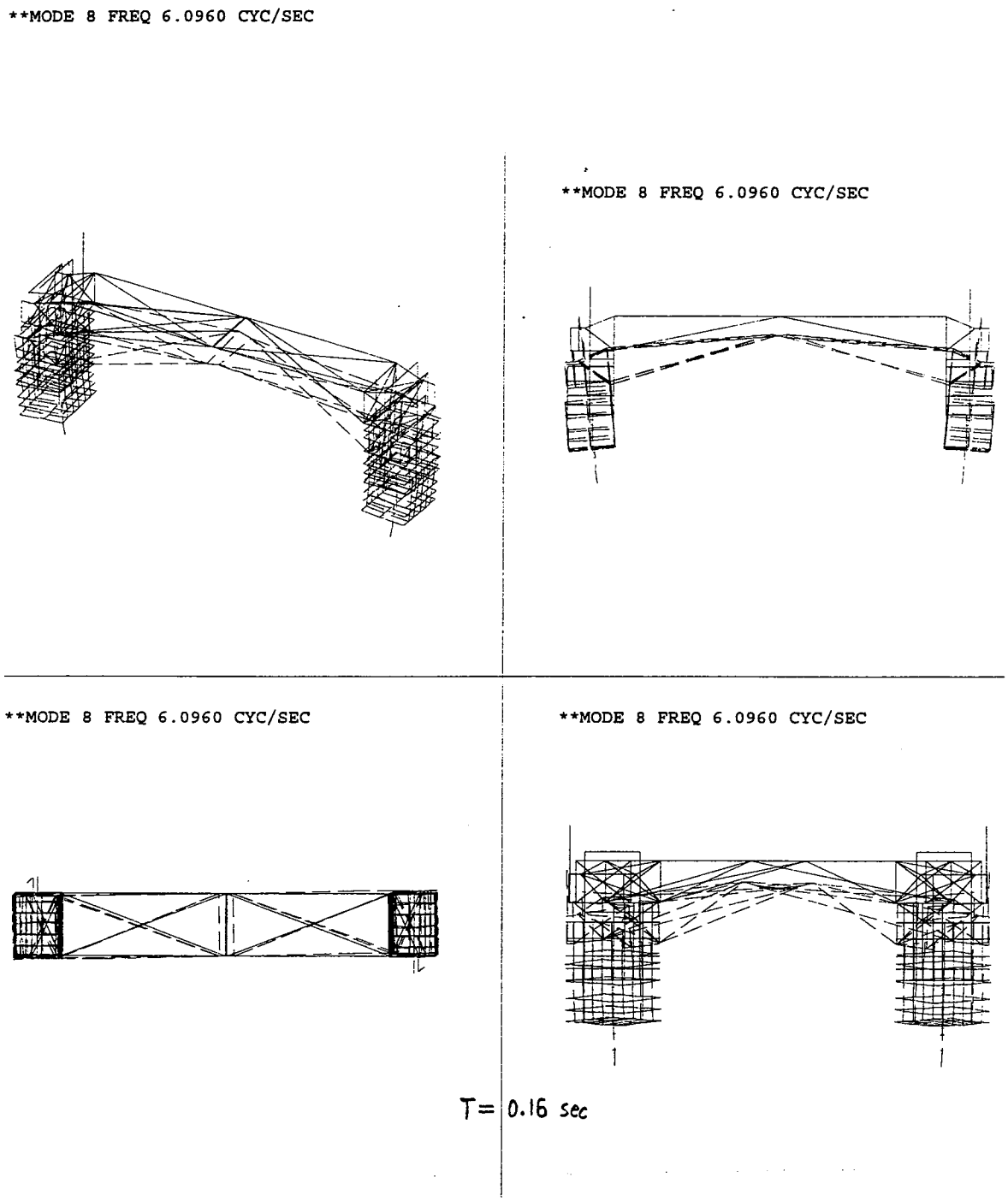


Figure 3.6 Bascule Span Mode Shape # 8 (under MHHW Condition)

Z
Y
X

Feet Kips

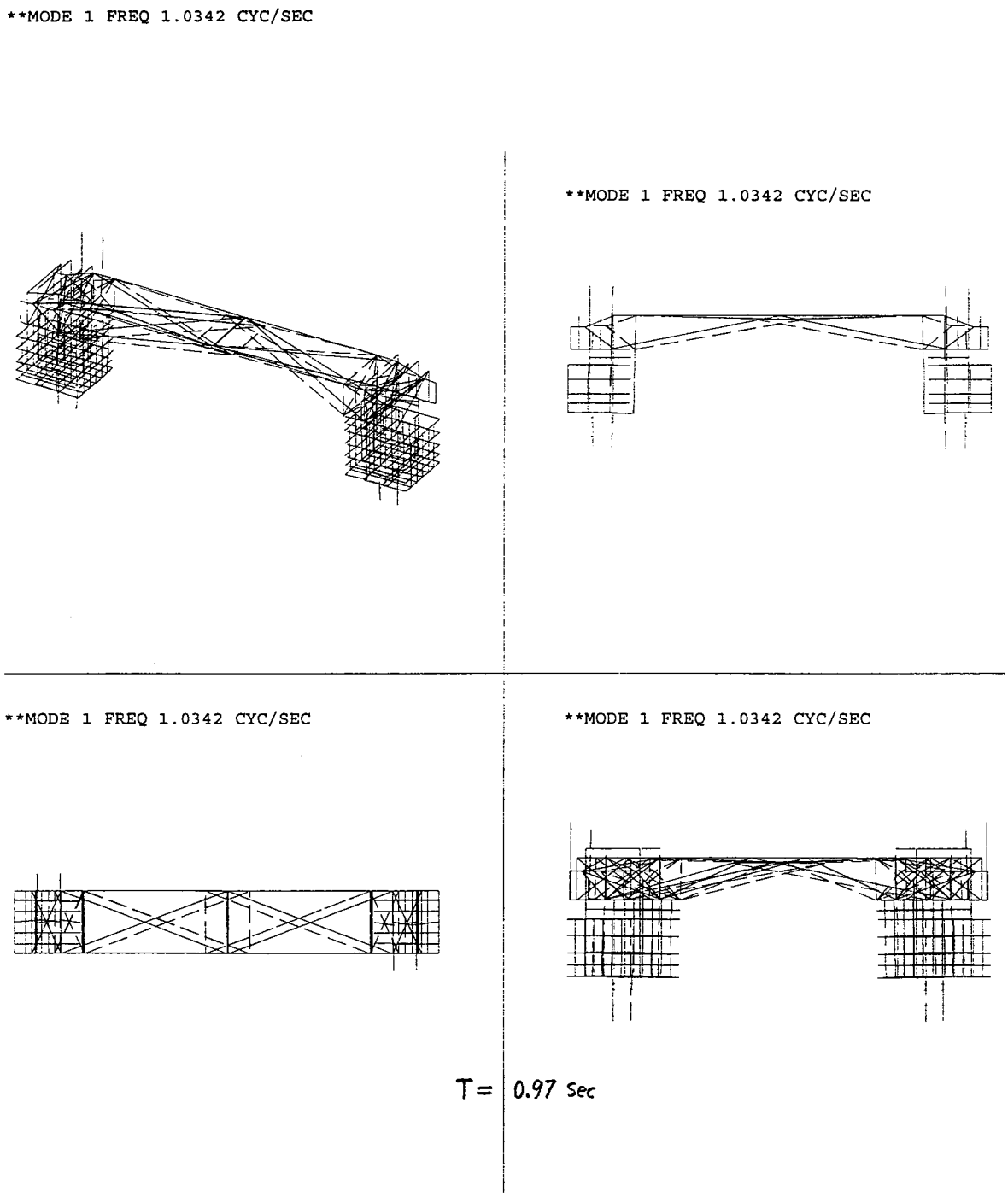


Figure 3.7 Bascule Span Mode Shape # 1 (under MHHW & with Soil Liquefaction Condition)

Z
Y
X

Feet Kips

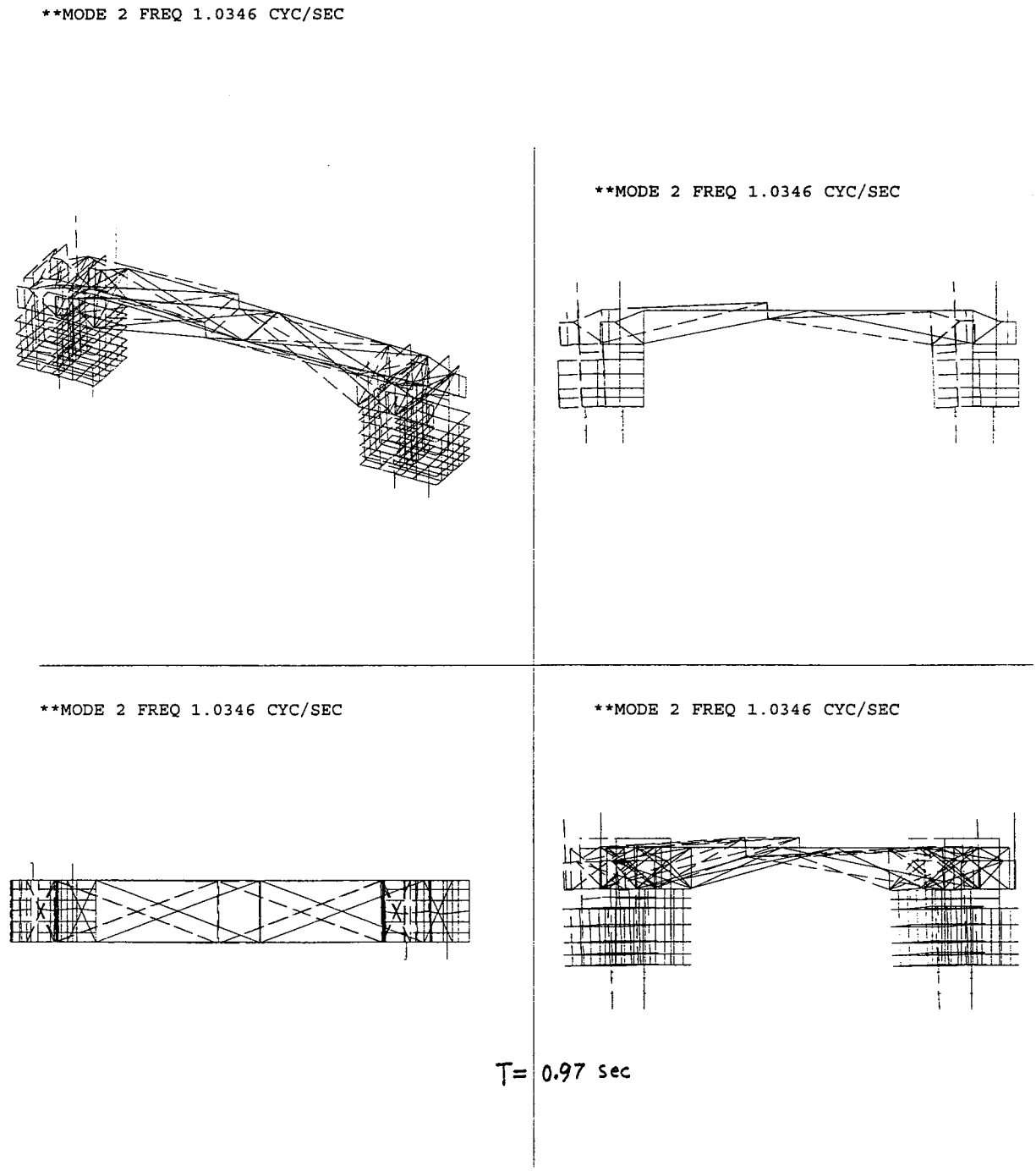


Figure 3.8 Bascule Span Mode Shape # 2 (under MHHW & with Soil Liquefaction Condition)

Z
Y
X

Feet Kips

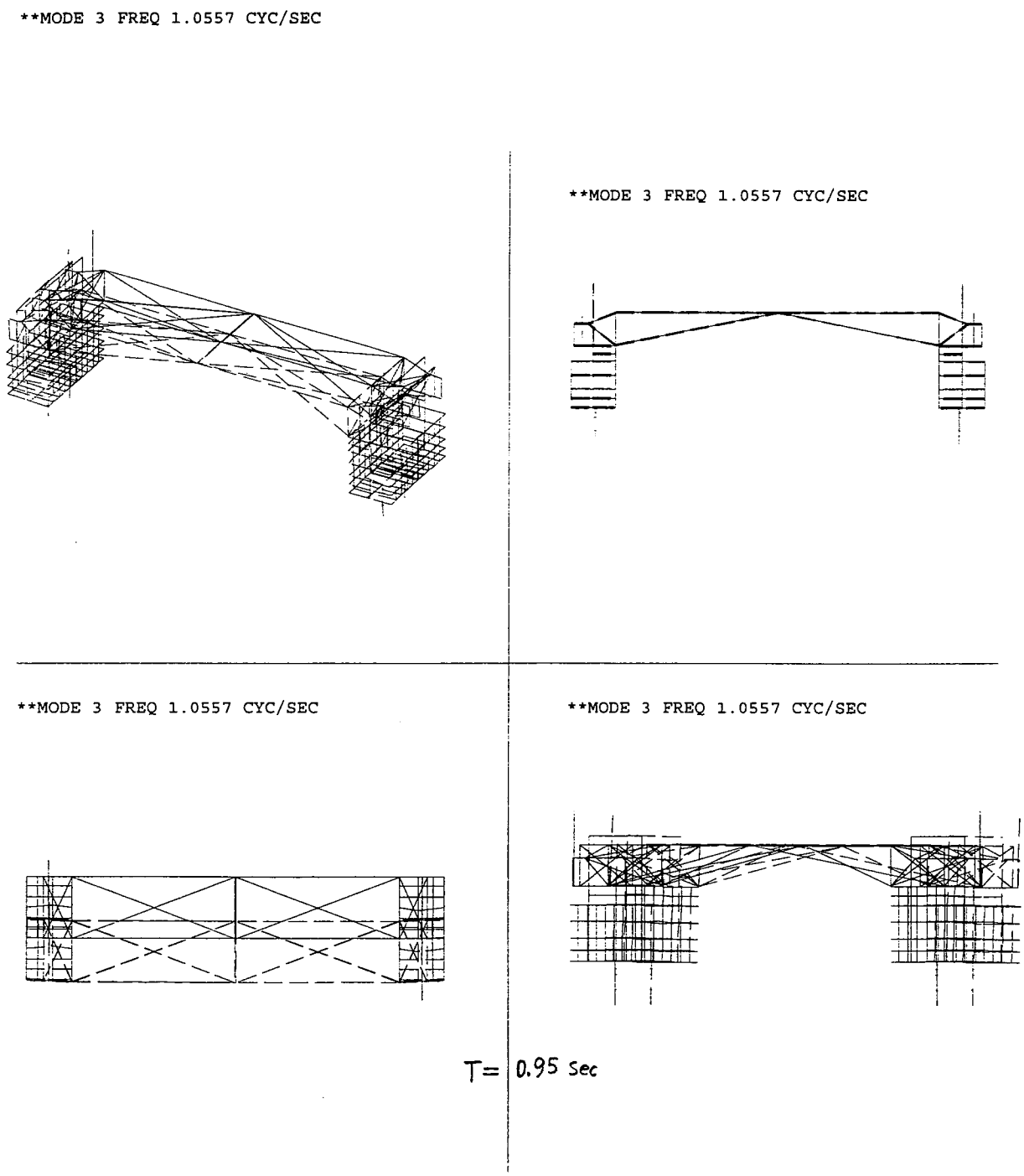


Figure 3.9 Bascule Span Mode Shape # 3 (under MHHW & with Soil Liquefaction Condition)

Z
Y
X

Feet Kips

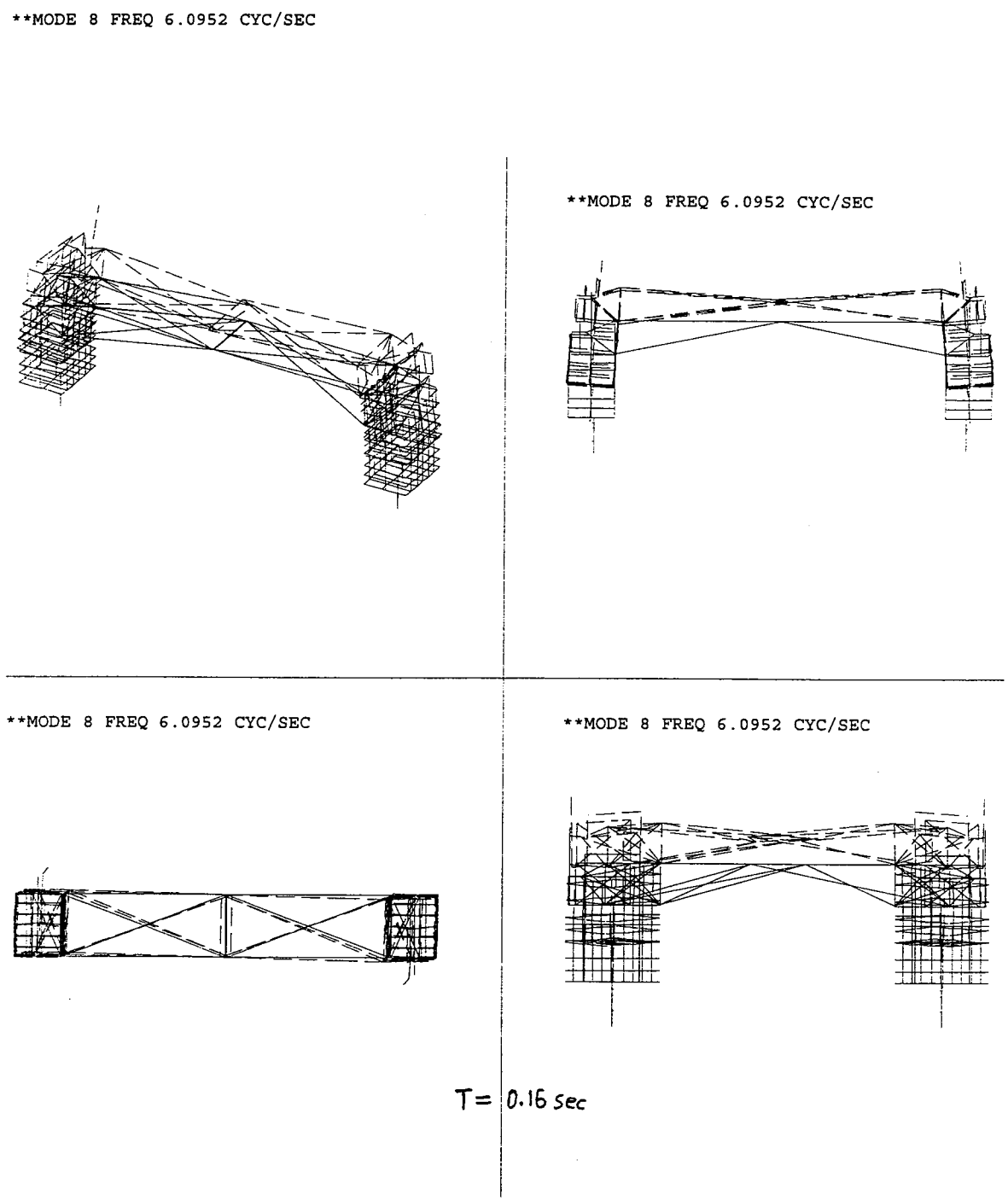


Figure 3.10 Bascule Span Mode Shape # 8 (under MHHW & with Soil Liquefaction Condition)

Z
Y
X

Feet Kips

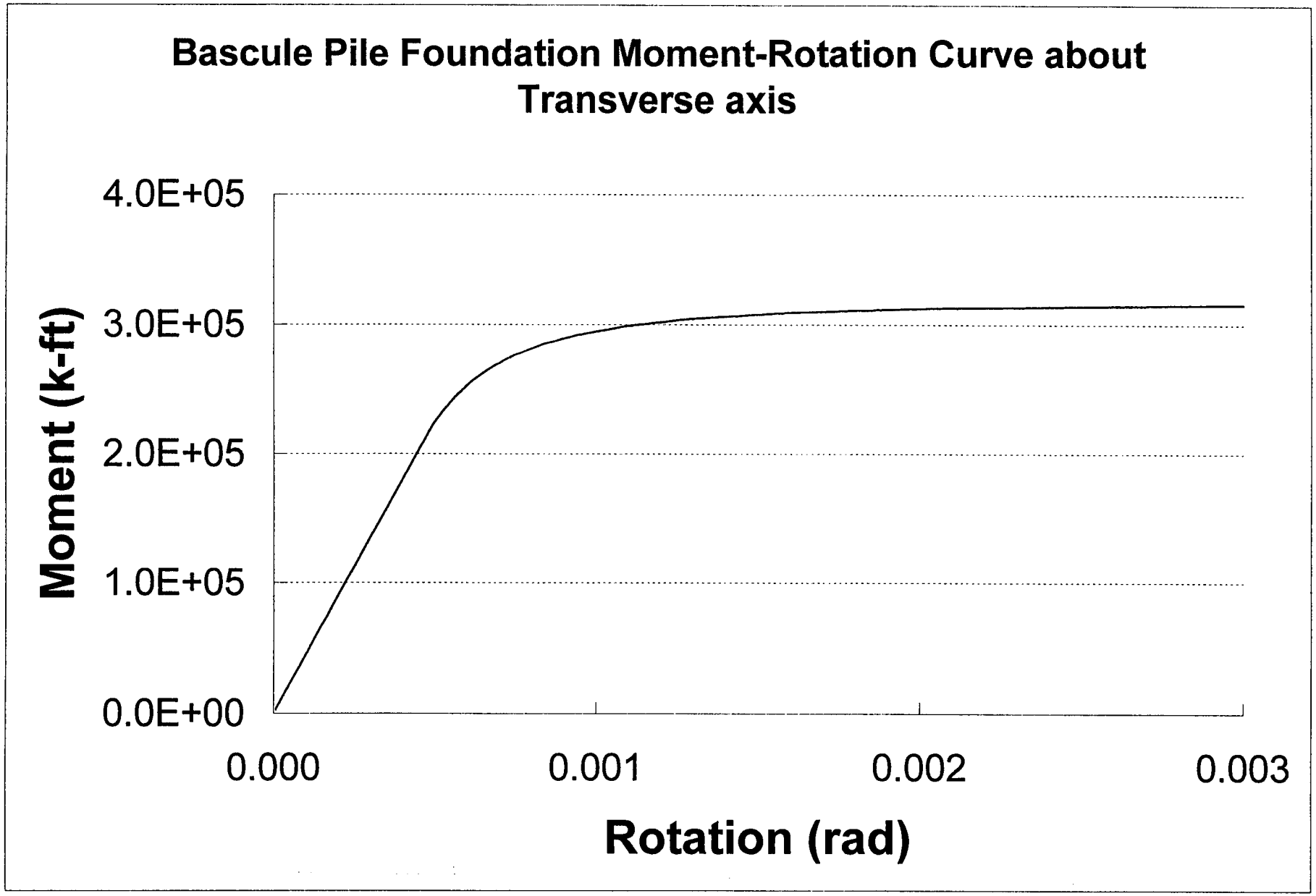


Figure 3.11 Bascule Pile Foundation Moment-Rotation Curve about Transverse Axis

```
xSECTION
VER._2.30,_MAR-08-98
(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
12/29/2000, 11:00
File: test.xs
Bascule Footing
Area_G(ft^2)=3017.67
I_G(ft^4)= 531771.60
Axial (kips)=11720.0
Percent steel=  0.00
--------------------
Moment-Curv. Curve :
Mmax ft-k=  263587.0
phi_max    = 0.000235
--------------------
Rebar Yield info:
My (k-ft)=  254957.1
phi_y      = 0.000006
Ec   (ksi)=     2851
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=  265591.2
phi_p      = 0.000229
Icr(ft^4)=   8957.79
```

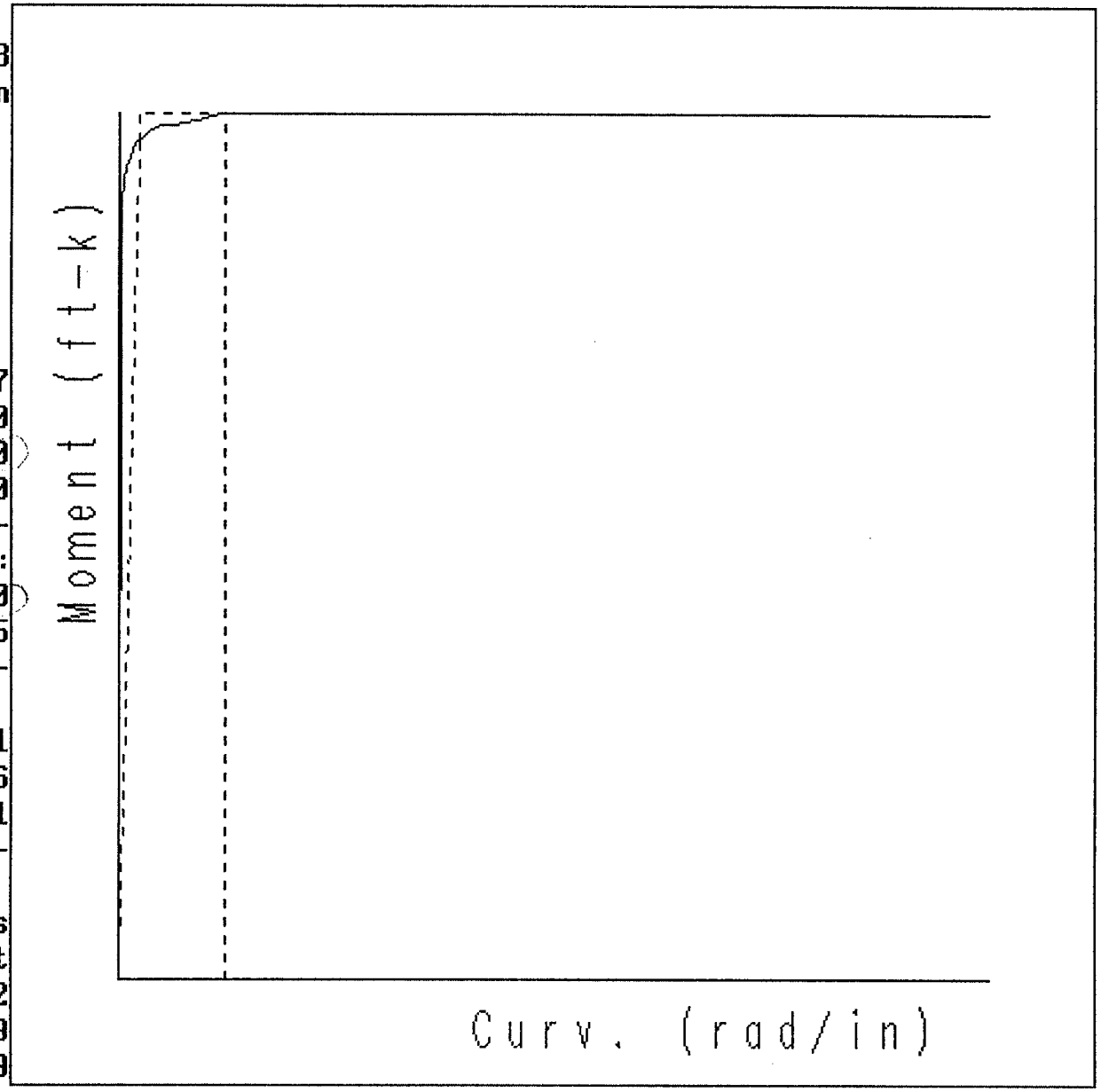


Figure 3.12 Bascule Footing Moment Capacity under DL Axial Load

```
xSECTION
VER._2.30,_MAR-08-98
(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
12/29/2000, 11:22
File: test.xs
Bascule Footing
Area_G(ft^2)=3017.67
I_G(ft^4)= 531771.60
Axial (kips)= 7757.0
Percent steel=  0.00
--------------------
Moment-Curv. Curve :
Mmax ft-k=  174529.8
phi_max    = 0.000228
--------------------
Rebar Yield info:
My (k-ft)=  170874.4
phi_y      = 0.000006
Ec   (ksi)=     2851
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=  175834.0
phi_p      = 0.000222
Icr(ft^4)=   6166.58
```

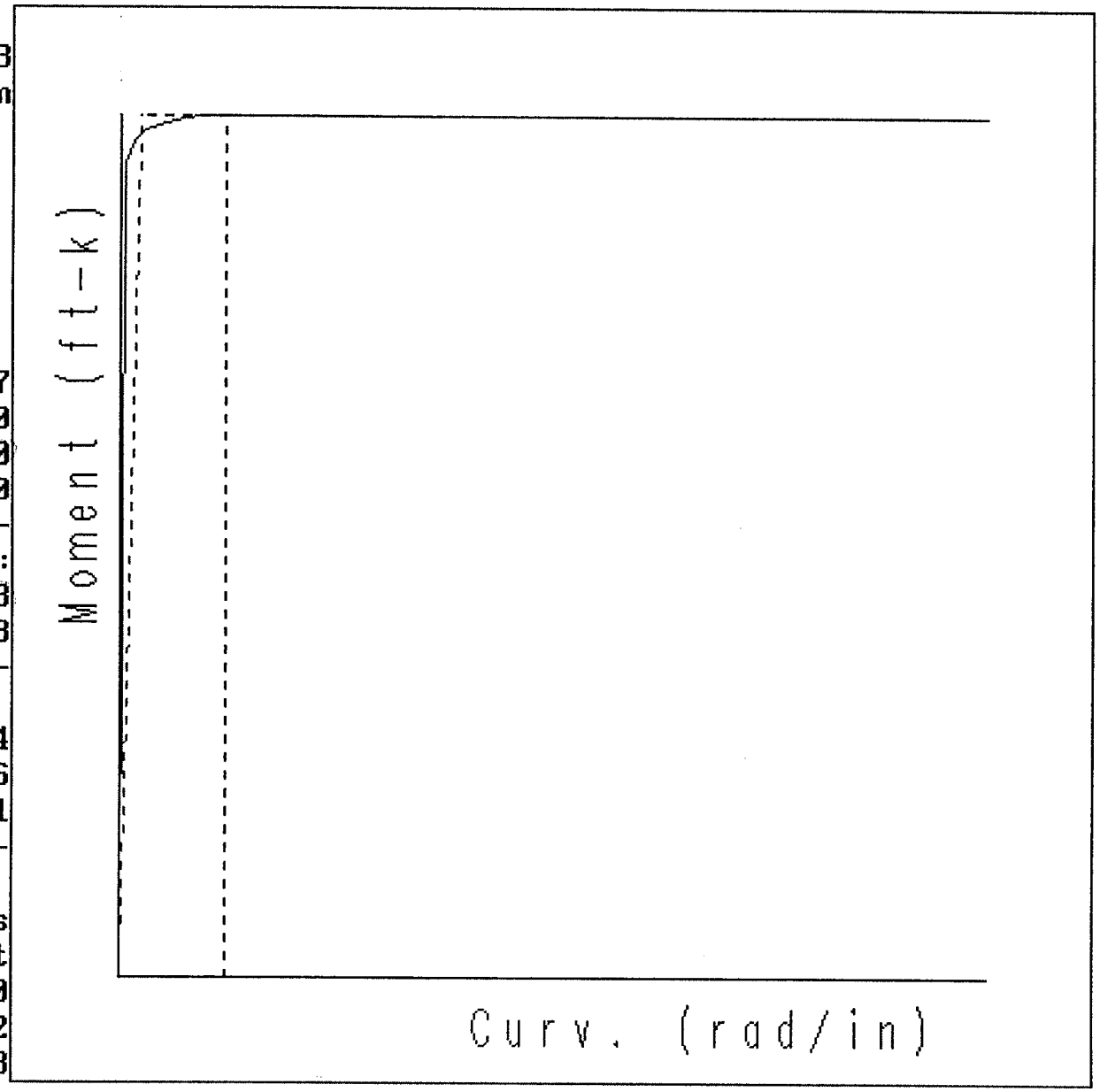


Figure 3.13 Bascule Footing Moment Capacity under Minimum Axial Load

```
xSECTION
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(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
01/02/2001, 13:55
File: bascule.xs
Bascu. W. Wall
Area_G(ft^2)= 453.65
I_G(ft^4)=   7572.35
Axial (kips)= 2580.0
Percent steel=  0.00
--------------------
Moment-Curv. Curve :
Mmax ft-k=   18681.8
phi_max    = 0.000735
--------------------
Rebar Yield info:
My (k-ft)=   18280.5
phi_y      = 0.000019
Ec   (ksi)=     2851
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=   18856.8
phi_p      = 0.000715
Icr(ft^4)=    194.75
```

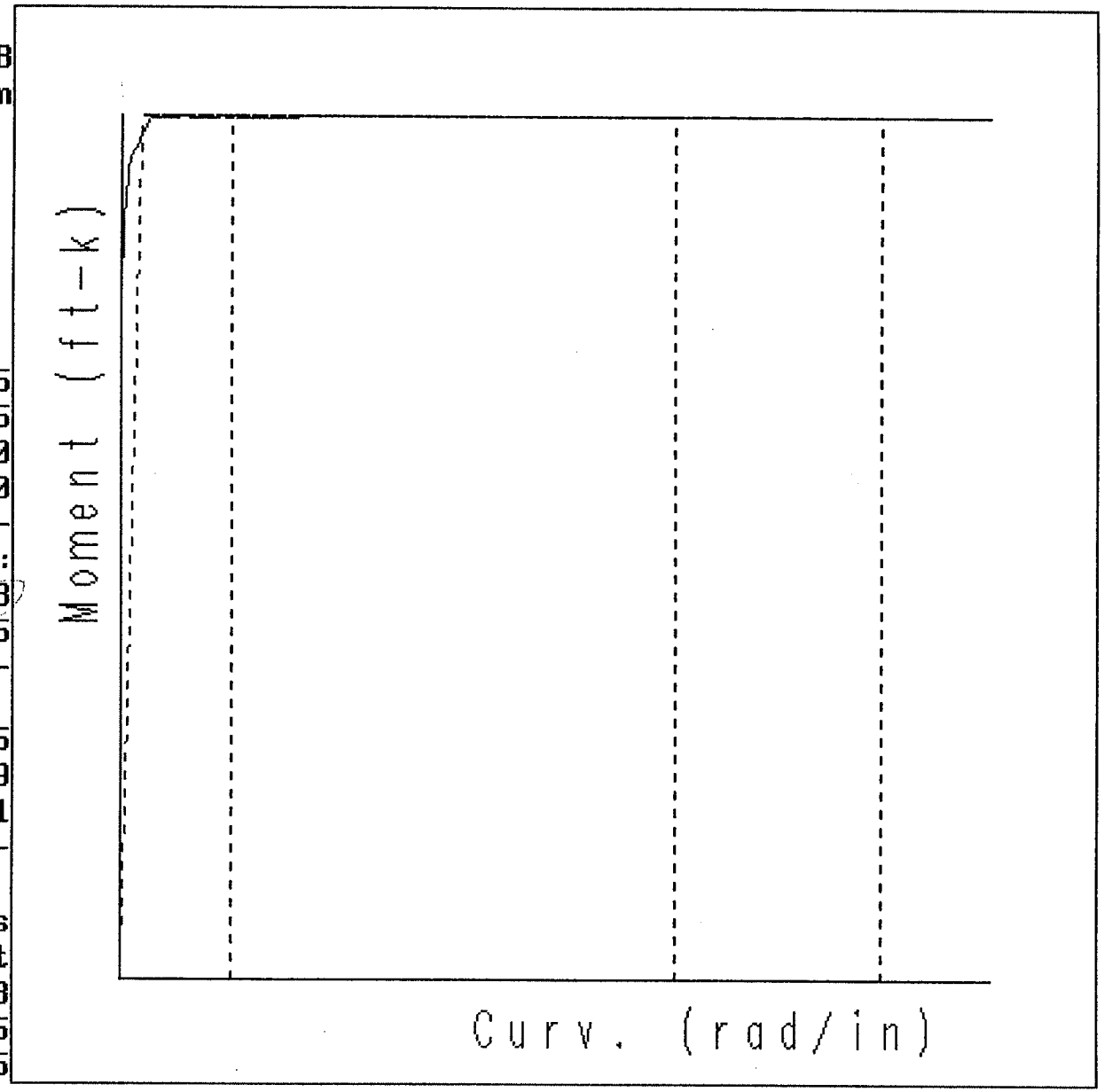


Figure 3.14 Bascule West Wall Moment Capacity about Longitudinal Axis

```
xSECTION
VER._2.30,_MAR-08-98
(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
01/02/2001, 13:56
File: bascule.xs
Bascu. W. Wall
Area_G(ft^2)= 453.65
I_G(ft^4)=  52131.36
Axial (kips)= 2580.0
Percent steel=  0.00
--------------------
Moment-Curv. Curve :
Mmax ft-k=   45427.1
phi_max    = 0.000278
--------------------
Rebar Yield info:
My (k-ft)=   43513.4
phi_y      = 0.000008
Ec   (ksi)=     2851
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=   45829.9
phi_p      = 0.000270
Icr(ft^4)=   1148.83
```

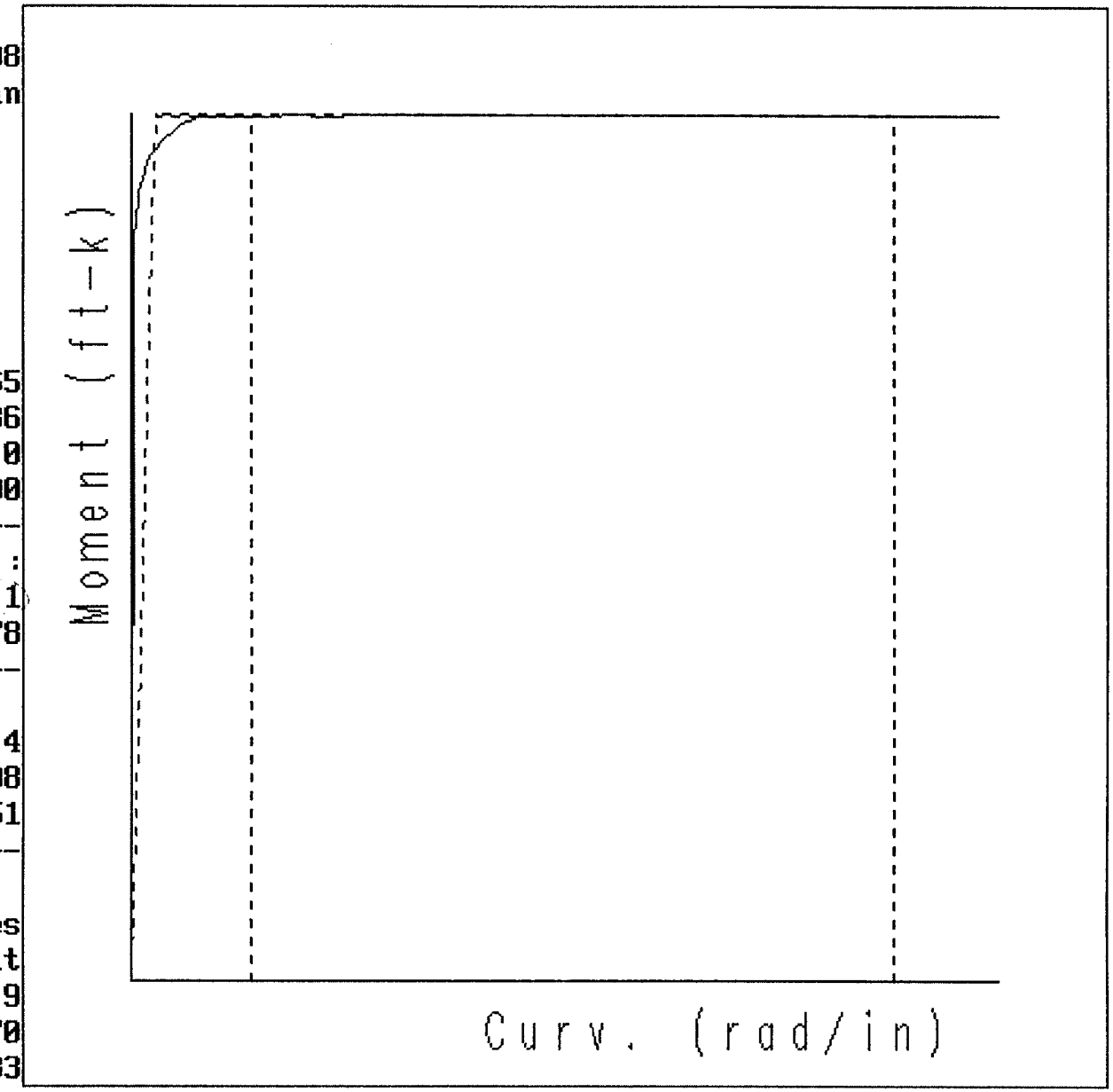


Figure 3.15 Bascule West Wall Moment Capacity about Transverse Axis

4. SEISMIC EVALUATION OF THE STEEL TRUSS APPROACH SPANS

The superstructures of steel truss spans are in good condition. See Figure 1.2 for location of the steel truss approach spans. Therefore, analyses and evaluation are mainly focused on the substructures. Pier 1 and Pier 2 are analyzed separately. See Figures 1.13 and 1.15, for sections of Piers 1 and 2, respectively.

4.1 Seismic Evaluation of Pier 1

4.1.1 Analytical Model

Figure 4.1 shows the Gtstrudl model used for the analysis of Pier 1 of the truss approach. The superstructure is treated as concentrated masses. When transverse ground motion is under consideration, these masses are placed approximately at the center of gravity of the superstructure resting on a rigid frame supported by the pier. When longitudinal ground motion is under consideration, these concentrated masses are placed directly on the top of the pier. The columns of the pier, the footing and the seal are modeled with beam members. The as-built drawings mistakenly show two concrete walls connecting the columns. Field inspection reveals that the connection is a cap beam. The dimensions of the cap beam are estimated from photos taken in the field and no reinforcement is assumed. The pile foundation is represented by six linear springs. The spring constants are listed in Table 4.1. Passive resistance of soil against the footing and the seal is ignored. The effect of soil liquefaction is also neglected. Only MHHW condition is considered in analyses.

4.1.2 Structural Dynamic Characteristics

Table 4.2 summarizes modal periods and mass participation factors. Figures 4.2 to 4.4 show the mode shapes of modes 1, 2 and 8. These three modes are the dominant modes under longitudinal, transverse and vertical ground motion. Mode 1 has a period of 0.86 second and a participation factor of 97% under longitudinal earthquake loading. Mode 2 has a period of 0.80 second and a participation factor of 99% under transverse earthquake loading. Mode 8 has a period of 0.12 second and a participation factor of 99.9% under vertical ground motion.

4.1.3 Performance Evaluation for 190 Year Earthquake

4.1.3.1 Displacement Responses

Table 4.3 lists the maximum displacements at the top of the pier. The maximum displacements for the 190 year earthquake are 1.9 inches along longitudinal direction, 1.7 inches along transverse direction and 0.1 inches along vertical direction, respectively. By using the SRSS rule to combine the longitudinal displacements at the top of the bascule and Pier 1, the longitudinal relative displacement between the roller support and the top of the pier is 2.6 inches. The seat width is 28 inches. Therefore, the D/C ratio is only 0.1.

4.1.3.2 Response of the Cap Beam

Table 4.4 summarizes the loads, capacities and D/C ratios of the cap beam. Under 190 year earthquake, the D/C ratios are 1.5 for shear and 2.7 for moment, respectively. The beam has very little ductility and will fail if the capacity is exceeded. Therefore, retrofit is necessary.

4.1.3.3 Responses of the Pile Foundation

Table 4.5 shows the loads of the pile group and the critical piles on the edge of the group subjected to 190 year earthquake. The D/C ratios of the critical piles are 0.61 for shear, 1.30 for compression and 1.96 for tension, respectively. Therefore, piles subjected to maximum tension and compression will yield.

Figure 4.5 is the pile group moment and rotation curve obtained from a nonlinear pushover analysis assuming the footing and the seal can act as a pile cap to hold all the piles together to resist bending. It is observed that the critical piles start to yield at a moment of 5500 kip-ft and the moment capacity of the group is 8000 kip-ft. This capacity is significantly greater than the yield moment and also greater than the moment demand My of 7725 kip-ft listed in Table 4.5.

If the footing and the seal are damaged and can no longer hold the piles together, these piles can still resist moment on the footing by shifting the reaction off the center of gravity. Figure 4.6 shows the moment and rotation curve obtained from a nonlinear pushover analysis. The moment capacity of the pile group reduces to 4350 kip-ft. The rocking stiffness of the pile group also reduces significantly. If the stiffness reduction is

appropriately simulated in the analysis, the seismic forces in the system will decrease significantly. Therefore, the foundation may not fail after damage occurring in the seal.

4.1.3.4 Responses of the Footing

The 3.5 foot thick footing above a 5 foot thick seal is supported by 27 timber piles. The footing is reinforced at the bottom with #5 bars spaced 12 inches on center each way. Dowels of 5 foot long #8 bars are placed 18 inches on center along the periphery of the columns. Table 4.6 shows the loads at the seal and footing interface. Shear and compressive stresses are small, 9.2 psi for shear and 131 psi for compression. A hypothetical tensile stress of 71 psi is obtained when calculated based on beam theory. Usually, zero concrete tensile capacity is assumed at the interface. So the corresponding tensile forces have to be resisted by the timber piles, which have 6 inches of embedment in the footing. The maximum tensile force in a pile calculated in this fashion is 80 kips which is very likely to exceed the tensile capacity of the 6 inch embedment. Therefore, timber piles may be pulled out of the footing, resulting a separation between the footing and the seal.

4.1.3.5 Responses of Columns

The columns of Pier 1 are plain concrete with varied rectangular section. Tables 4.7 and 4.8 summarize the results at the bottom of the column and at elevation -12.98 feet. The maximum tensile stresses are 141 psi and 118 psi, respectively. At the bottom of the column, there are #8 dowels spaced at 18 inches on center. Under the axial load of 632 kips, the moment capacity at the bottom of the column about the transverse axis, My,

calculated from X Section is 8565 kip-ft as shown in Figure 4.7. The moment D/C ratio is 0.65.

At elevation –12.98 feet, there is no longitudinal reinforcement. So significant cracks may occur at this location.

4.1.5 Performance Evaluation for 95 Year Earthquake

Table 4.9 summarizes the result of the pile foundation response. Pile D/C ratios are 1.3 for tension and 1.04 for compression, respectively. But as mentioned previously, the pile group will not fail after the critical piles yield.

Table 4.10 shows the result at the seal and footing interface. The maximum hypothetical tensile stress is 46 psi. Assuming zero concrete tensile capacity at the interface, a maximum tensile force of 50 kips will develop at the critical pile, which may pull the pile out of the footing.

Table 4.11 lists the loads and stresses in the column at elevation –12.98 feet. The maximum tensile stress is 76 psi, which may result in significant cracks in the column.

4.2 Seismic Evaluation of Pier 2

4.2.1 Analytical Model and Structural Dynamic Characteristics

As discussed in Chapter 1, Pier 2 consists of two outside columns and a middle column connected by a concrete wall. The pier is capable of resisting larger seismic load along

the transverse direction than along the longitudinal direction. Therefore, only the outside column and its foundation under the longitudinal and vertical ground motions are investigated. Figure 4.8 depicts the Gtstrudl model used for the analysis. The supports of the truss span superstructure at Pier 2 are rollers along longitudinal direction. So its effect on the longitudinal responses of the pier can be ignored. The tributary mass of half span of the concrete approach is modeled as a concentrated mass at the top of the pier. The column of the pier and the footing are modeled with beam members. The pile foundation is represented by six linear springs. The spring constants are tabulated in Table 4.12. The effect of soil liquefaction is neglected. Only MLLW condition is considered in analyses.

Table 4.12 also lists modal periods and mass participation factors. The dominant mode under longitudinal ground motion is mode 1 with a period of 0.7 second and a participation factor of 99.5%. The dominant mode under vertical ground motion is mode 3 with a period of 0.08 second and a participation factor of 99.9%.

4.2.2 Responses and Evaluation

Table 4.13 shows the longitudinal displacements of Pier 1 and 2. If the SRSS rule is adopted to combine these displacements, a maximum relative displacement of 2.5 inches between the roller supports and the top of the pier is obtained. By comparison with the seat width of 14 inches at Pier 2, the D/C ratio of the longitudinal displacement is 0.2 for the 190 year earthquake.

Each outside column of Pier 2 is supported by 13 timber piles. These piles are embedded 4 inches in the footing. The footing is 2.5 foot thick and is reinforced at the bottom with

10 #5 bars each way. Table 4.14 summarizes the loads, capacities and D/C ratios on a foundation pile. The capacities of the pile are difficult to determine because of the possible deterioration due to the elevation change of ground water and uncertainty of the tensile capacity developed from 4 inches of embedment in the footing. Assuming the pile has no deterioration and can develop 15 kips of tension from the embedment. For the 190 year earthquake, the D/C ratios are 0.7 for compression, 1.6 for tension and 0.5 for shear, respectively. For the 95 year earthquake, the D/C ratios are 0.6 for compression, 0.8 for tension and 0.4 for shear, respectively.

The rectangular columns of Pier 2 are constructed with plain concrete. Table 4.15 lists the maximum tensile and compressive stresses in the column. Compressive stresses are small. The tensile stress at the bottom of the column is 108 psi for the 190 year earthquake and 68 psi for the 95 year earthquake, respectively. Even though the tensile stress is greater than 49 psi, no significant crack will occur at this location because there are #8 dowels spacing at 18 inches on center, which can help, resist tension. At elevation –5.0 feet, tensile stress is 75 psi for the 190 year earthquake and 39 psi for the 95 year earthquake, respectively. Therefore, cracks may occur above elevation –5.0 feet when the structure is subjected to the 190 year earthquake.

4.3 Seismic Evaluation of Superstructures

The superstructures of the steel truss spans are in good condition so detail analyses are not conducted. However, by careful examination of the as-built drawings and hand calculations, the components which need retrofit, can be identified. Figure 1.11 illustrates the superstructure framing of the truss span #1. No bottom laterals exist between Frame

10 and the fixed supports at 12. In addition, there is no lateral load path in Frame 10 because space is needed when the counterweight goes down during bridge opening. Therefore, new braced frames need to be installed at location 11 to transfer the lateral earthquake load from the deck level to the supports.

The roller supports are restrained transversely. These supports can easily be damaged during transverse ground motion. Transverse shear keys can be installed at the top the pier to prevent this damage. The anchor bolts of the fixed supports are rusted. Additional anchor bolts may be necessary.

4.4 Damage State of Steel Truss Approaches

The damage state of the steel truss approaches under different levels of earthquakes is summarized in Chapter 8.

Table 4.1 Pier 1 Pile Foundation Spring Constants

Longitudinal Translation (kips/in)	Transverse Translation (kips/in)	Vertical Translation (kips/in)	Rocking about Longitudinal Axis (k-in/rad)	Rocking about Transverse Axis (k-in/rad)	Rotation about Vertical Axis (k-in/rad)
484	484	9430	5.13E+07	6.76E+07	4.93E+06

Table 4.2 Pier 1 Pile Foundation Spring Constants

(under MHHW Condiiton)

MODE	FREQUENCY (CYC/SEC)	PERIOD (SEC)
1	1.16	0.86
2	1.25	0.8
3	1.44	0.69
4	4.68	0.21
5	5.36	0.19
6	7.41	0.13
7	7.66	0.13
8	8.63	0.12
9	9.25	0.11
10	10.4	0.1

X DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	97.5	2	9.00E-29	3	1.34E-17	4	2.5
5	7.85E-29	6	1.27E-28	7	2.07E-29	8	1.57E-29
9	2.62E-26	10	7.53E-30	11	1.71E-22	12	1.25E-29
Y DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	2.55E-30	2	99.0013	3	1.99E-29	4	5.58E-30
5	0.976833	6	2.15E-29	7	7.46E-23	8	5.97E-27
9	1.59E-30	10	1.94E-02	11	7.50E-31	12	2.40E-03
Z DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	8.79E-33	2	1.05E-27	3	1.09E-32	4	1.18E-31
5	6.07E-25	6	5.66E-28	7	1.08E-28	8	99.93539
9	1.46E-27	10	6.75E-24	11	3.73E-29	12	1.24E-27

Table 4.3 Displacements at Top of Pier 1

(Under 190 Years Earthquake)

Longitudinal Displacement (in)	Transverse Displacement (in)	Vertical Displacement (in)	Bascule Span Long. Disp. (in)	Long. Relative Disp. Roller Support & Pier (in)	Pier Seat Width (in)	D/C Ratio
1.92	1.66	0.05	1.7	2.56	28	0.1

Table 4.4 Evaluation of Pier 1 Cap Beam

DL		Max. EQ		Governing Force			Capacities			D/C Ratio		
V (kips)	M (kip-ft)	V (kips)	M (kip-ft)	V (kips)	Mt (kip-ft)	Mb (kip-ft)	V (kip-ft)	Mt (kip-ft)	Mb (kip-ft)	V	Mt	Mb
190 Years Earthquake												
138	966	338	7242	476	8208	6276	318	4573	2326	1.5	1.8	2.7
95 Years Earthquake												
138.0	966	254.0	5432	392.0	6398.0	5432.0	318	4573	2326	1.2	1.4	2.3

Table 4.5 Evaluation of Pier-1 Pile Foundation
(under 190 Years EQ)

		Under MHHW				
		DL	Earth-P	Case-1	Case-2	Case-3
Pile Group Loads	Vx (kips)	0	0	261	78	78
	Vy (kips)	0	0	86	286	86
	Fz (kips)	953	0	192	642	400
	Mx (kip-ft)	0	0	974	3248	974
	My (kip-ft)	0	0	7725	2318	2318
	Mz (kip-ft)	0	0	0	0	0
Single Pile Loads	Fz (kips)	35.3	0.0	133.4	102.4	58.0
	Vx (kips)	0.0	0.0	9.7	2.9	2.9
	Vy (kips)	0.0	0.0	3.2	10.6	3.2
		DL+EP+Max(Case1, Case2, Case3)				
	Fz (kips)	**169**				
	Fz (kips)	**-98**				
	V (kips)	**11.0**				

Notes: DL = Dead Load - Buoyancy

Pile Group Properties		
# of Piles	Sx, Sx' (ft)	Sy, Sy' (ft)
27	65	65
	130	81

Maximum Pile Forces		
Compression	Tension	Shear
169	98	11.0
Pile Capacities		
Compression	Tension	Shear
130	50	17.9
D/C Ratios		
Compression	Tension	Shear
1.30	**1.96**	**0.61**

Table 4.6 Evaluation of Pier 1 Footing
(under 190 Years EQ)

		Under MHHW				
		DL	Earth-P	Case-1	Case-2	Case-3
Footing Loads	Vy (kips)	0	0	78	259	78
	Vz (kips)	0	0	239	72	72
	Fx (kips)	870	0	183	610	368
	My (kip-ft)	0	0	6431	1929	1929
	Mz (kip-ft)	0	0	547	1823	547
	Mx (kip-ft)	0	0	0	0	0
Footing Stresses	fx (psi)	29.9	0.0	100.7	75.7	47.1
	v_y (psi)	0.0	0.0	2.7	8.9	2.7
	v_z (psi)	0.0	0.0	8.2	2.5	2.5
		DL+EP+Max(Case1, Case2, Case3)				
	fx (psi)			**131**		
	fx (psi)			**-71**		
	v (psi)			**9.2**		
Single Pile Loads	Fx (kips)	32.2	0.0	112.5	74.8	36.9
	Vy (kips)	0.0	0.0	2.9	9.6	2.9
	Vz (kips)	0.0	0.0	8.9	2.7	2.7
		DL+EP+Max(Case1, Case2, Case3)				
	Fx (kips)			**145**		
	Fx (kips)			**-80**		
	V (kips)			**10.0**		

Notes: DL = Dead Load - Buoyancy

Footing Section Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
202	521	437

Pile Group Properties		
# of Piles	Sx, Sx' (ft)	Sy, Sy' (ft)
27	65	65
	130	81

Compressive Stress =	131 psi	OK	Pile Compression =	145 kips	N.G.
Tensile Stress =	71 psi	N.G.	Pile Tension =	80 kips	N.G.
Shear Stress =	9.2 psi	OK	Pile Shear =	10 kips	OK

Table 4.7 Evaluation of Pier 1 at Bottom of Column
(under 190 Years EQ)

			Under MHHW				
			DL	Earth-P	Case-1	Case-2	Case-3
Column	Loads	Vy (kips)	0	0	72	240	71
		Vz (kips)	0	0	224	67	50
		Fx (kips)	808	0	176	588	346
		My (kip-ft)	0	0	5599	1680	1021
		Mz (kip-ft)	0	0	277	925	277
		Mx (kip-ft)	0	0	0	0	0
Column	Stresses	fx (psi)	46.4	0.0	187.5	114.7	59.7
		v_y (psi)	0.0	0.0	4.1	13.8	4.1
		v_z (psi)	0.0	0.0	12.9	3.8	2.9
			DL+EP+Max(Case1, Case2, Case3)				
		fx (psi)	**234**				
		fx (psi)	**-141**				
		v (psi)	**14.3**				

Notes: DL = Dead Load - Buoyancy

Column Section Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
121	231	211

Compressive Stress =	234 psi		OK
Tensile Stress =	141 psi	> 49 psi	N.G.
Shear Stress =	14.3 psi		OK

Moment capacity developed from #8 @ 18" dowels and P = 632 kips:
Myn = 8565 k-ft
D/C Ratio = My/Myn = 0.65 OK

Table 4.8 Evaluation of Pier 1 at EL -12.98 ft
(under 190 Years EQ)

		Under MHHW				
		DL	Earth-P	Case-1	Case-2	Case-3
Column Loads	Vy (kips)	0	0	64	213	64
	Vz (kips)	0	0	202	61	40
	Fx (kips)	784	0	167	557	316
	My (kip-ft)	0	0	4589	1377	795
	Mz (kip-ft)	0	0	66	221	66
	Mx (kip-ft)	0	0	0	0	0
Column Stresses	fx (psi)	48.6	0.0	166.7	88.9	48.7
	v_y (psi)	0.0	0.0	4.0	13.2	4.0
	v_z (psi)	0.0	0.0	12.5	3.8	2.5
		DL+EP+Max(Case1, Case2, Case3)				
	fx (psi)	**215**				
	fx (psi)	**-118**				
	v (psi)	**13.7**				

Notes: DL = Dead Load - Buoyancy

Column Section Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
112	207	189

Compressive Stress =	215 psi		OK
Tensile Stress =	118 psi	> 49 psi	N.G.
Shear Stress =	13.7 psi		OK

Table 4.9 Evaluation of Pier 1 Pile Foundation
(under 95 Years EQ)

			Under MHHW				
			DL	Earth-P	Case-1	Case-2	Case-3
Pile Group	Loads	Vx (kips)	0	0	196	59	59
		Vy (kips)	0	0	65	215	65
		Fz (kips)	953	0	144	482	300
		Mx (kip-ft)	0	0	731	2436	731
		My (kip-ft)	0	0	5794	1739	1739
		Mz (kip-ft)	0	0	0	0	0
Single Pile	Loads	Fz (kips)	35.3	0.0	100.1	76.8	43.5
		Vx (kips)	0.0	0.0	7.3	2.2	2.2
		Vy (kips)	0.0	0.0	2.4	7.9	2.4
			DL+EP+Max(Case1, Case2, Case3)				
		Fz (kips)	**135**				
		Fz (kips)	**-65**				
		V (kips)	**8.2**				

Notes: DL = Dead Load - Buoyancy

Pile Group Properties		
# of Piles	Sx, Sx' (ft)	Sy, Sy' (ft)
27	65	65
	130	81

Maximum Pile Forces		
Compression	Tension	Shear
135	65	8.2
Pile Capacities		
Compression	Tension	Shear
130	50	17.9
D/C Ratios		
Compression	Tension	Shear
1.04	**1.30**	**0.46**

Table 4.10 Evaluation of Pier 1 Footing
(under 95 Years EQ)

		Under MHHW				
		DL	Earth-P	Case-1	Case-2	Case-3
Footing Loads	Vy (kips)	0	0	58.5	194.2	58.5
	Vz (kips)	0	0	179.2	54.0	54.0
	Fx (kips)	870	0	137.2	457.5	276.0
	My (kip-ft)	0	0	4823.2	1446.7	1446.7
	Mz (kip-ft)	0	0	410.2	1367.2	410.2
	Mx (kip-ft)	0	0	0	0	0
Footing Stresses	fx (psi)	29.9	0.0	75.5	56.7	35.3
	v_y (psi)	0.0	0.0	2.0	6.7	2.0
	v_z (psi)	0.0	0.0	6.2	1.9	1.9
		DL+EP+Max(Case1, Case2, Case3)				
	fx (psi)			**105**		
	fx (psi)			**-46**		
	v (psi)			**6.9**		
Single Pile Loads	Fx (kips)	32.2	0.0	82.4	55.8	35.6
	Vy (kips)	0.0	0.0	2.2	7.2	2.2
	Vz (kips)	0.0	0.0	6.6	2.0	2.0
		DL+EP+Max(Case1, Case2, Case3)				
	Fx (kips)			**115**		
	Fx (kips)			**-50**		
	V (kips)			**7.5**		

Notes: DL = Dead Load - Buoyancy

Footing Section Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
202	521	437

Pile Group Properties		
# of Piles	Sx, Sx' (ft)	Sy, Sy' (ft)
27	65	65
	130	81

Compressive Stress =	105 psi	OK	Pile Compression =	115 kips	OK
Tensile Stress =	46 psi	N.G.	Pile Tension =	50 kips	N.G.
Shear Stress =	6.9 psi	OK	Pile Shear =	7.5 kips	OK

Table 4.11 Evaluation of Pier 1 at EL -12.98 ft
(under 95 Years EQ)

		Under MHHW				
		DL	Earth-P	Case-1	Case-2	Case-3
Column Loads	Vy (kips)	0	0	48.0	159.8	48.0
	Vz (kips)	0	0	151.5	45.8	30.0
	Fx (kips)	784	0	125.3	417.8	237.0
	My (kip-ft)	0	0	3441.8	1032.8	596.3
	Mz (kip-ft)	0	0	49.5	165.8	49.5
	Mx (kip-ft)	0	0	0.0	0.0	0.0
Column Stresses	fx (psi)	48.6	0.0	125.0	66.6	36.5
	v_y (psi)	0.0	0.0	3.0	9.9	3.0
	v_z (psi)	0.0	0.0	9.4	2.8	1.9
		DL+EP+Max(Case1, Case2, Case3)				
	fx (psi)	**174**				
	fx (psi)	**-76**				
	v (psi)	**10.3**				

Notes: DL = Dead Load - Buoyancy

Column Section Properties		
A (ft^2)	Sy (ft^3)	Sz (ft^3)
112	207	189

Compressive Stress =	174 psi		OK
Tensile Stress =	76 psi	> 49 psi	N.G.
Shear Stress =	10.3 psi		OK

Table 4.12 Pier 2 Foundation Spring Constants & Structural Dynamic Characteristics

Foundation Spring Constants		
Longitudinal Translation (kips/in)	Vertical Translation (kips/in)	Rocking about Longitudinal Axis (k-in/rad)
62	4540	1.50E+07
Structural Dynamic Characteristics		
Mode #	Period (sec.)	Mass Participation (%)
1	0.7	99.5
3	0.08	99.9

Table 4.13 Longitudinal Displacement at Top of Pier 2

(under 190 Years EQ)

Pier 2 Displacement (in)	Pier 1 Displacement (in)	Relative Displacement Roller Support & Pier (in)	Pier Seat Width (in)	D/C Ratio
1.6	1.9	2.5	14	0.2

Table 4.14 Evaluation of Pier 2 Pile Foundation

Loads: DL+EP+EQ			Capacities			D/C Ratios		
Compression (kips)	Tension (kips)	Shear (kips)	Compression (kips)	Tension (kips)	Shear (kips)	Compression (kips)	Tension (kips)	Shear (kips)
(under 190 Years Earthquake)								
92	24	8.3	130	15	17.2	0.7	1.6	0.5
(under 95 Years Earthquake)								
80	12	6.9	130	15	17.2	0.6	0.8	0.4

Table 4.15 Evaluation of Pier 2

Location	Compressional Stress (psi)	Tensile Stress (psi)
190 Years Earthquake		
Bottom of Column	246	108
EL. -5.0 ft	210	75
95 Years Earthquake		
Bottom of Column	206	68
EL. -5.0 ft	175	39

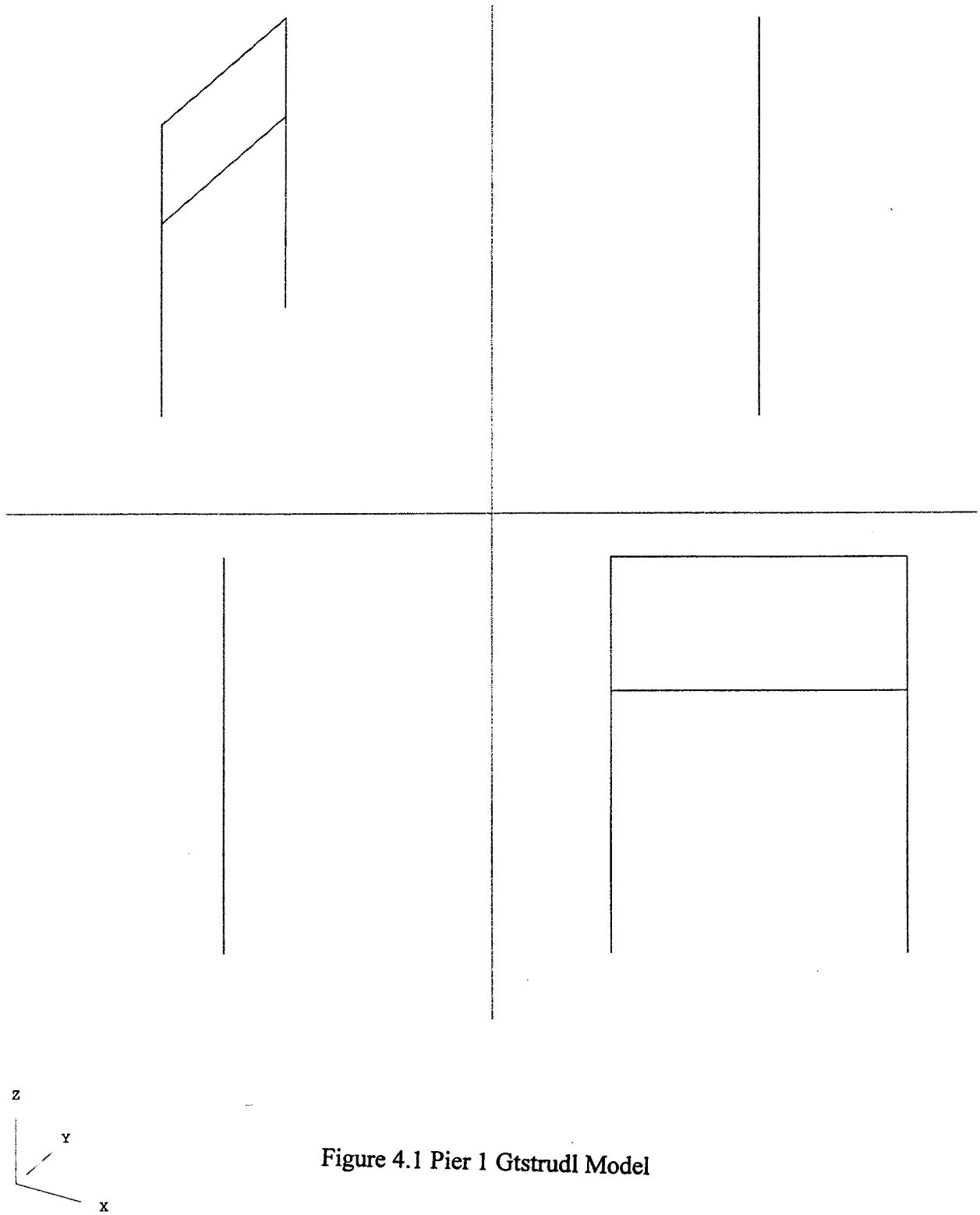


Figure 4.1 Pier 1 Gtstrudl Model

Feet Kips

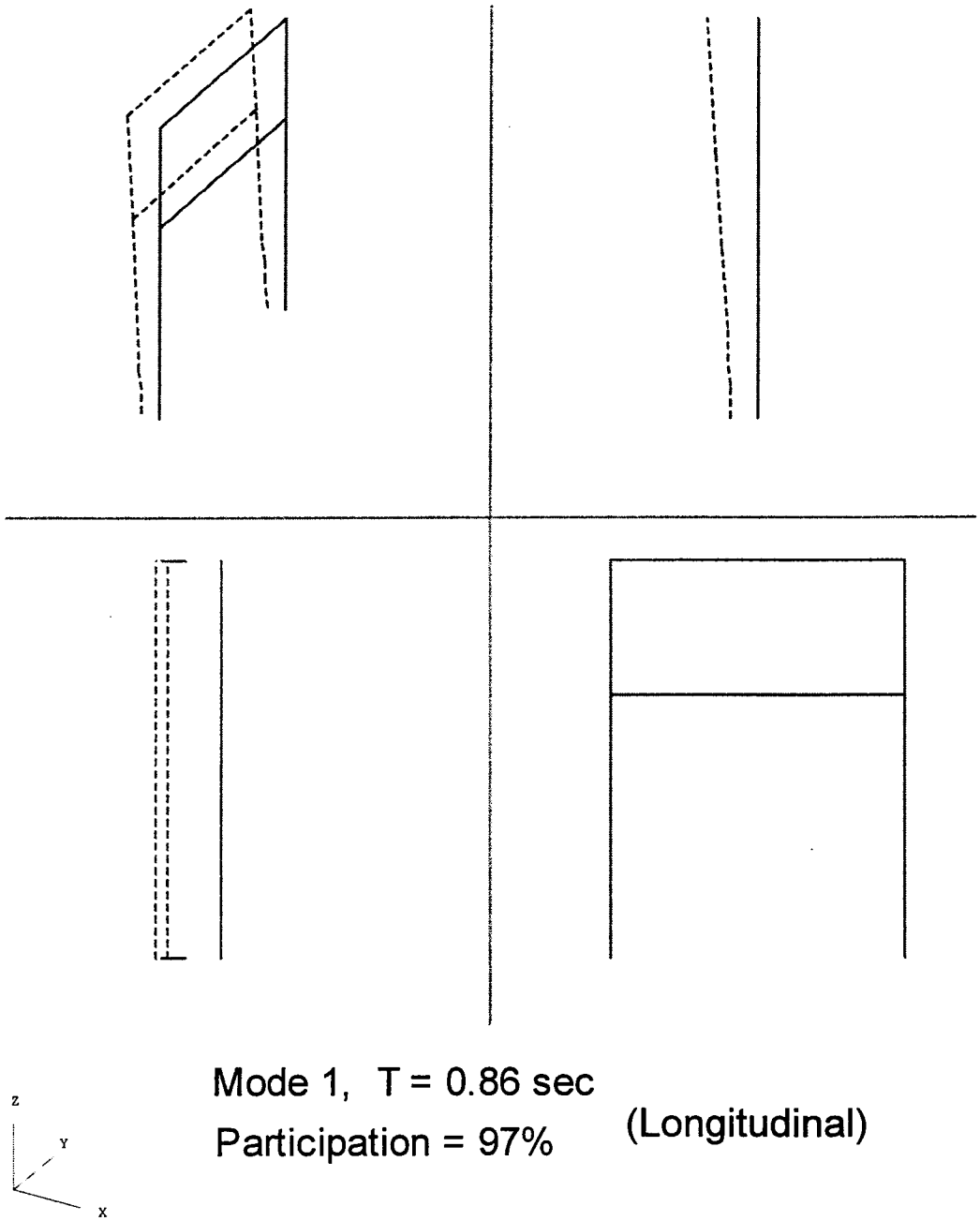


Figure 4.2 Pier 1 Mode Shape # 1

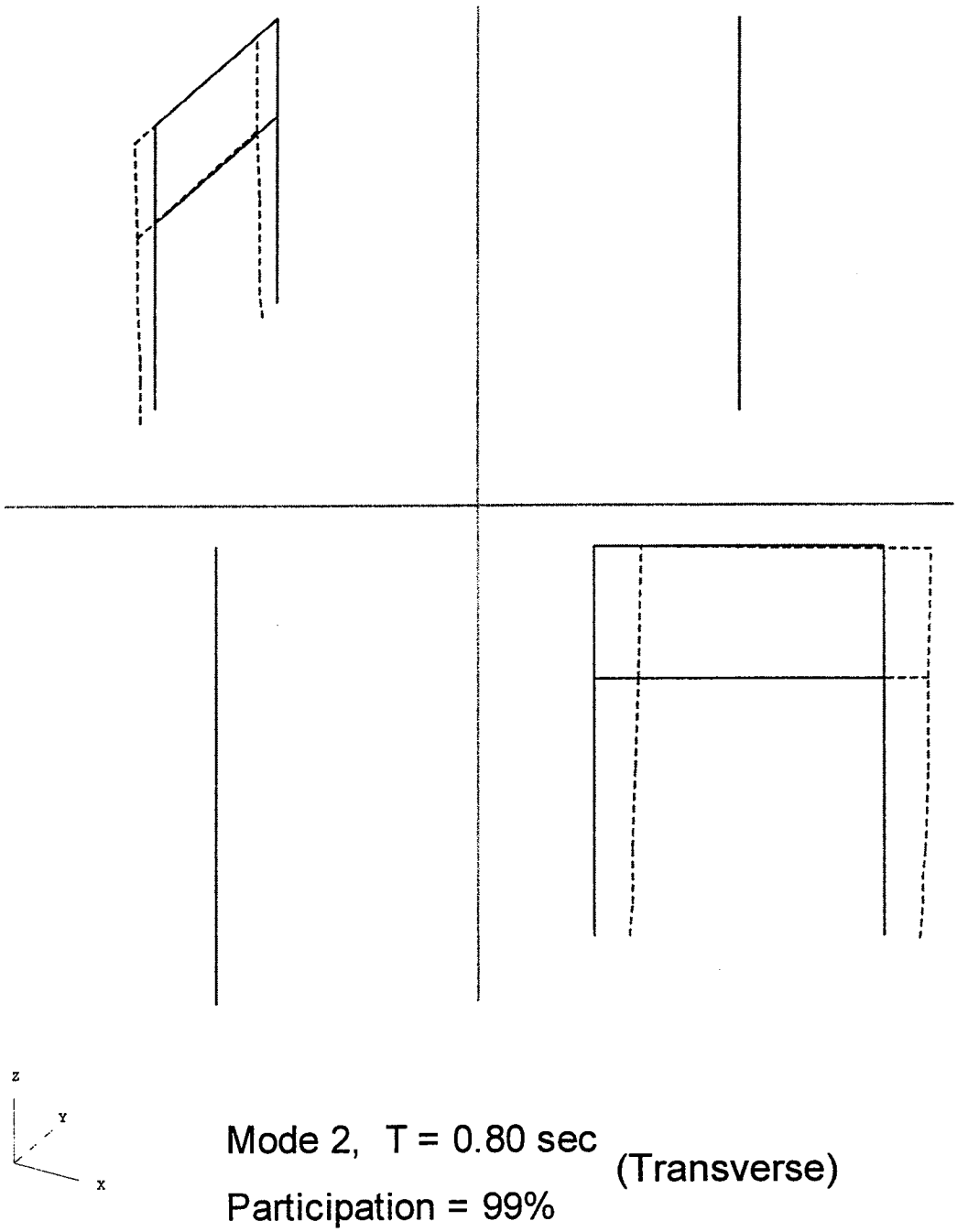
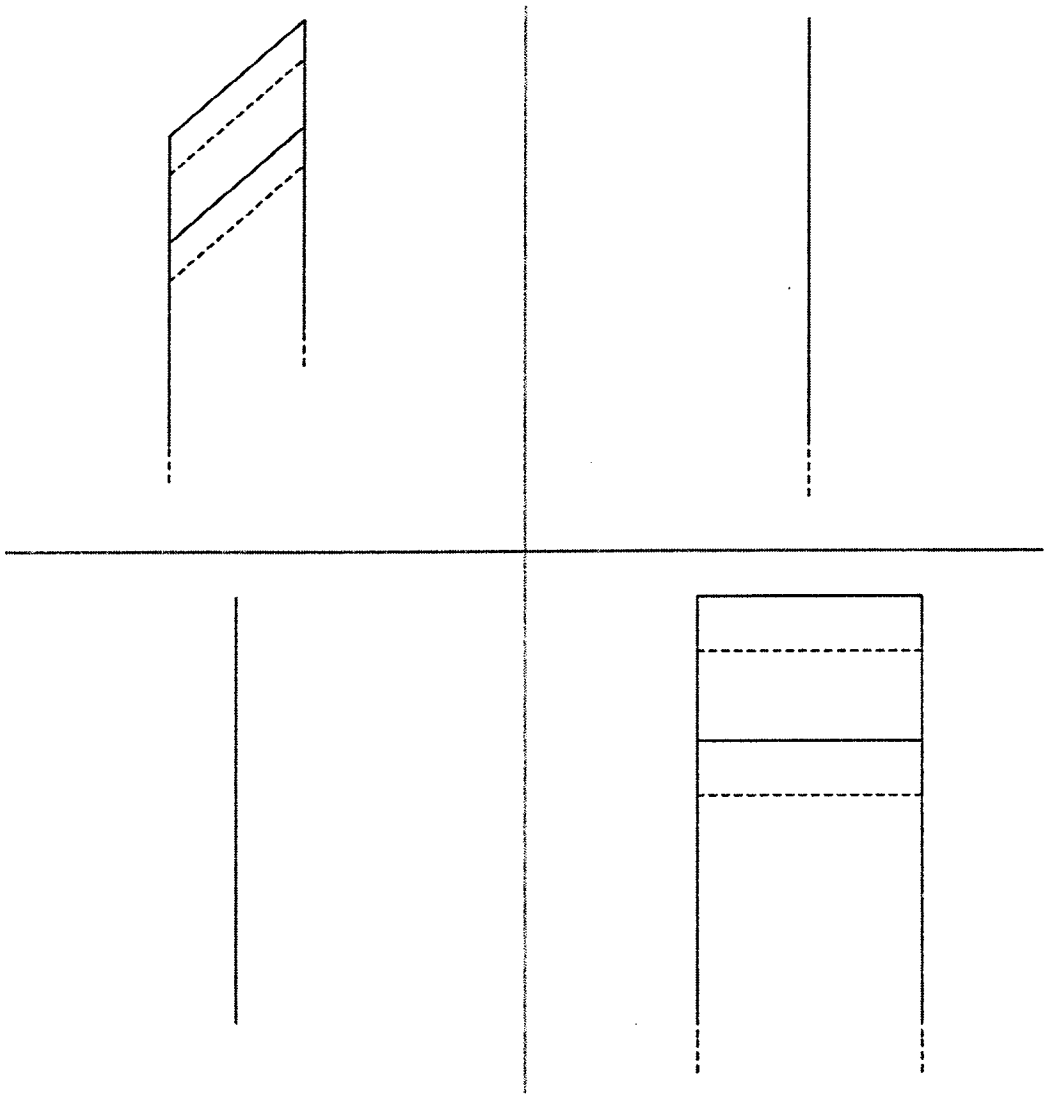
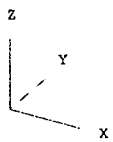


Figure 4.3 Pier 1 Mode Shape # 2





Mode Shape 8, T = 0.12 sec

Participation = 99.9% (Vertical)

Figure 4.4 Pier 1 Mode Shape # 8

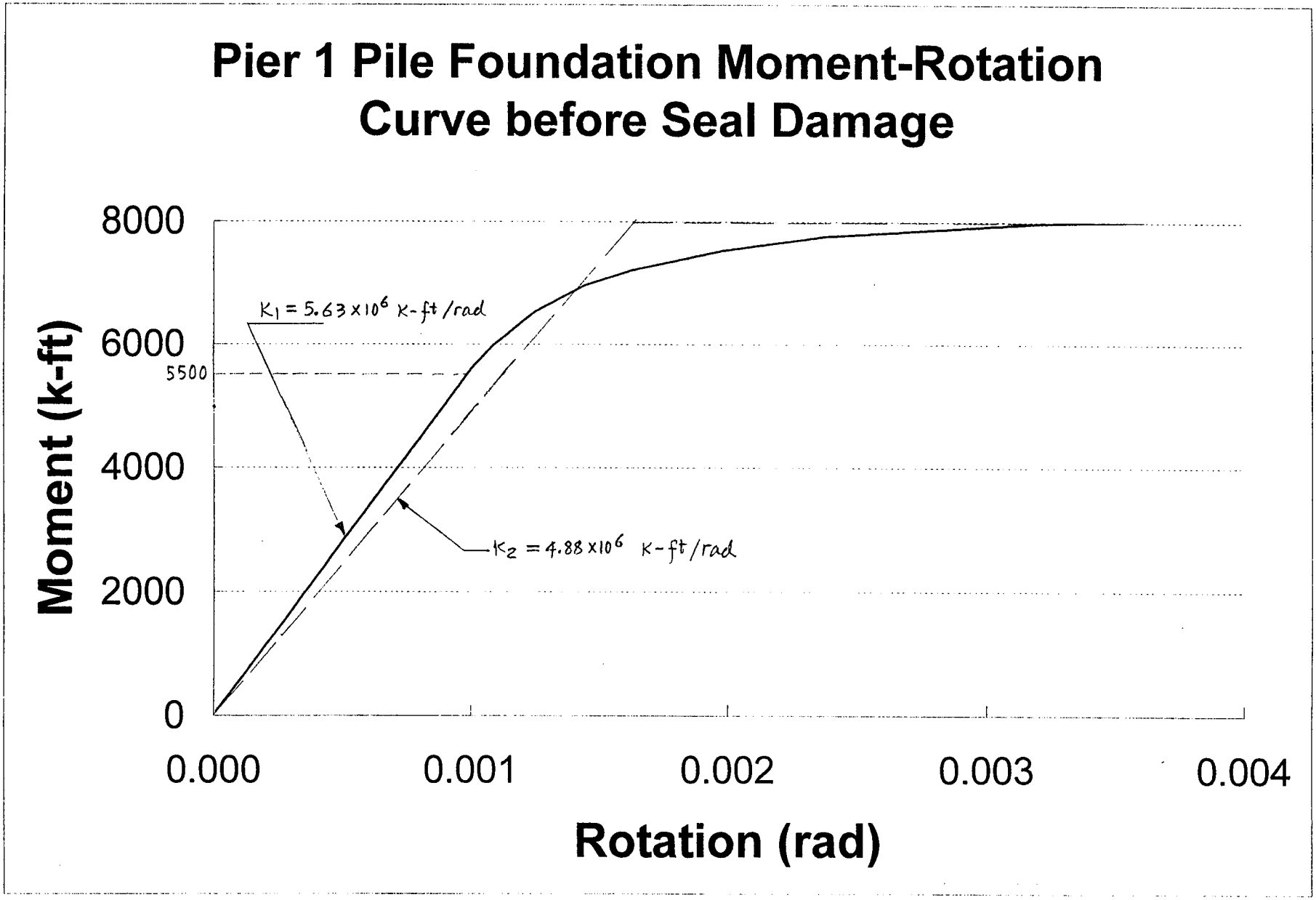


Figure 4.5 Pier 1 Pile Foundation Moment-Rotation Curve before Damage of Seal

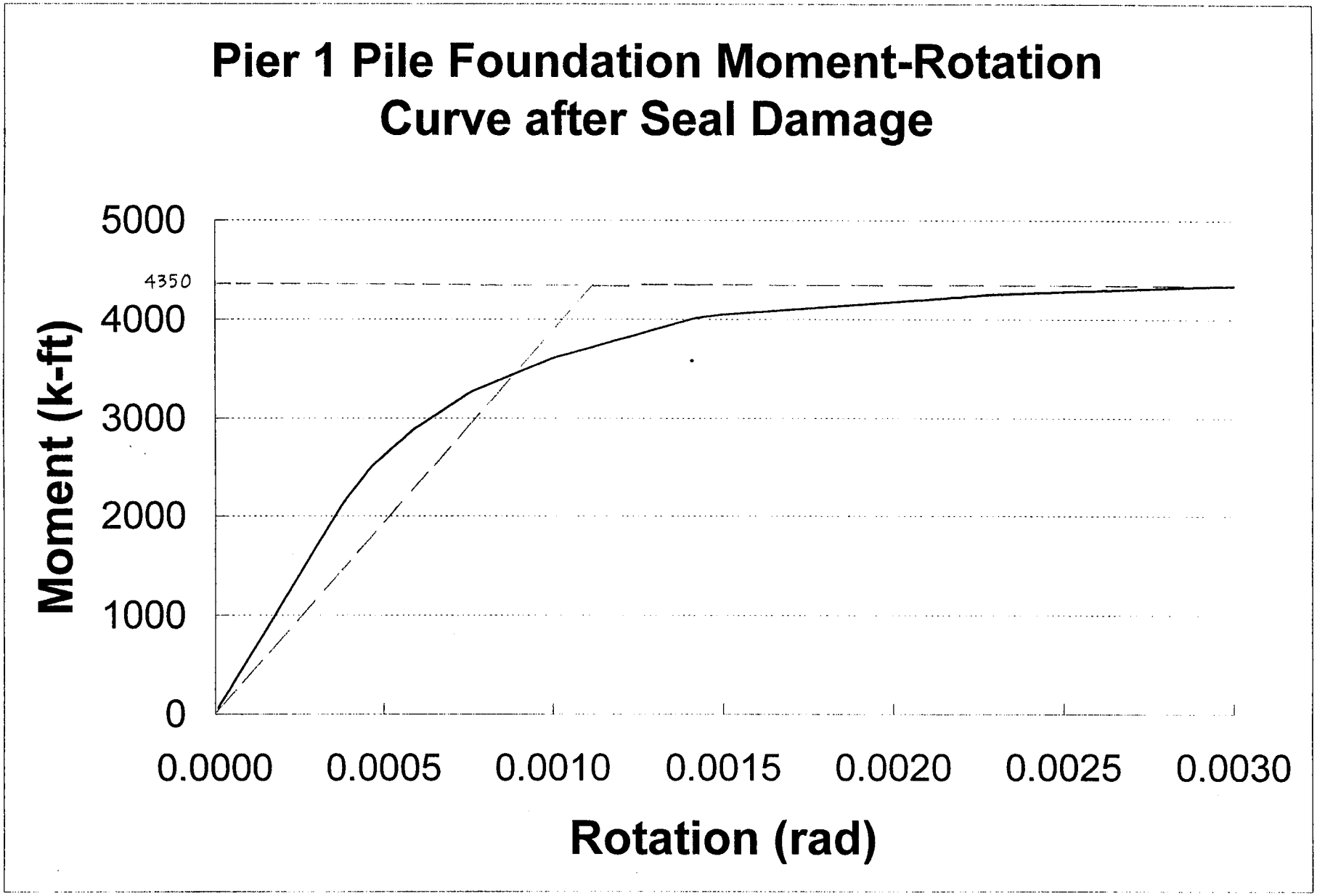


Figure 4.6 Pier 1 Pile Foundation Moment-Rotation Curve after Damage of Seal

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xSECTION
VER._2.30,_MAR-08-98
(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
01/02/2001, 15:49
File: kingpier.xs
P1 Bot.of.Col
Area_G(ft^2)= 120.75
I_G(ft^4)=   1330.01
Axial (kips)=  632.0
Percent steel=  0.13
--------------------
Moment-Curv. Curve :
Mmax ft-k=    8565.4
phi_max   = 0.000368
--------------------
Rebar Yield info:
My (k-ft)=    7090.0
phi_y     = 0.000012
Ec   (ksi)=     3123
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=    8432.6
phi_p     = 0.000354
Icr(ft^4)=    108.81
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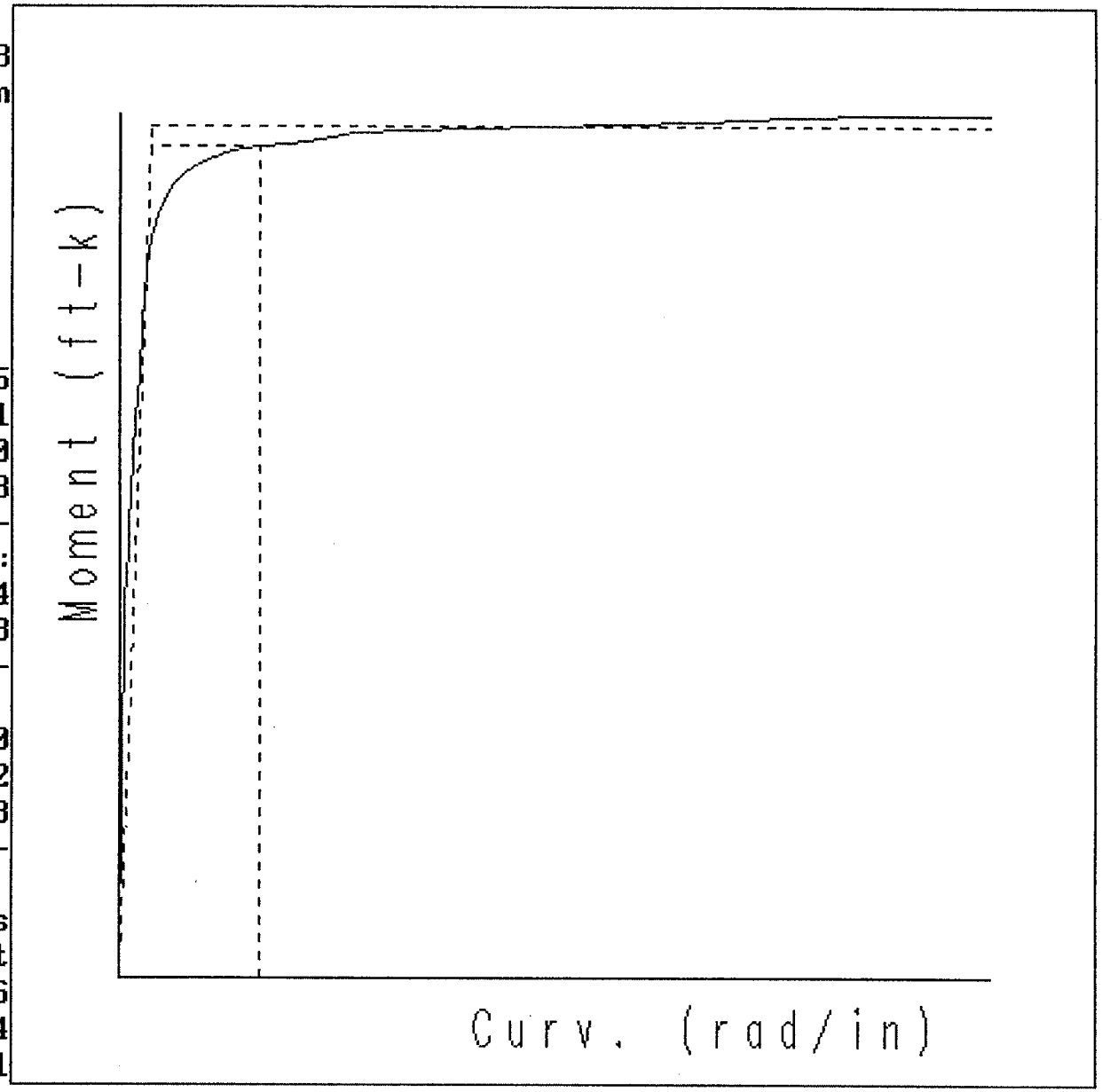


Figure 4.7 Pier 1 Footing Moment Capacity about Transverse Axis

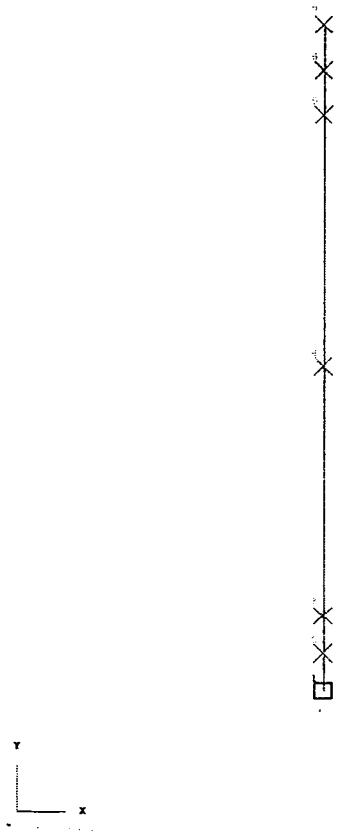


Figure 4.8 Pier 2 Gtstrudl Model

5. SEISMIC EVALUATION OF CONCRETE APPROACHES

5.1 Analytical Model

The concrete approach spans are separated with expansion joints every three spans. See Figure 1.2 for location of concrete approaches. Figure 5.1 illustrates the three-dimensional model used for the analysis of a typical three continuous span model. The continuous concrete slab is modeled with rigid truss members. Cap beams, longitudinal beams and columns are modeled with beam members. Footings and pile foundations are represented by translation and rocking springs. Table 5.1 presents the spring constants. The as-built drawings do not show any column reinforcement extending into the bent cap, which implies that the connection between the column and the bent cap is a pinned connection. However, both pinned and rigid connections are considered in the analyses. It was reported that foundation underpinning retrofit was carried out in 1970's. But information about the locations of the underpinning is not available at present. Therefore, no underpinning is assumed in analysis.

5.2 Structural Dynamic Characteristics

For the model with pinned connections between columns and cap beams, Table 5.2 lists the significant modal periods and mass participation factors. Figures 5.2 to 5.10 show the corresponding mode shapes. It is observed that the dominant mode under longitudinal ground motion is mode 2 with a period of 2.3 second. The significant modes under transverse ground motion are modes 1 and 3 with periods of 2.8 second and 2.1 second, respectively. For the model with rigid connections between columns and cap beams, Table

5.3 lists the major modal periods and mass participation factors. Again, mode 2 is the dominant mode for longitudinal vibration. Modes 1 and 3 are the important modes for transverse vibration. The periods of modes 1, 2 and 3 are 1.20 second, 1.16 second and 0.85 second, respectively.

5.3 Performance Evaluation

5.3.1 Displacement Responses

Table 5.4 shows the maximum longitudinal and transverse displacements at the cap beams. For rigid connection conditions, maximum longitudinal and transverse displacements for the 190 year earthquake are 2.8 inches and 2.0 inches, respectively. For the pinned connection condition, the maximum longitudinal and transverse displacements increase to 5.0 inches and 4.6 inches, respectively. Relative longitudinal displacement between the cap beam and the slab at the expansion joint is 4.0 inches for the rigid connection condition and 7.1 inches for pinned connection condition according to the SRSS combination. The seat width for concrete slab scaled from the as-built drawings is 9 to 12 inches. The longitudinal displacement D/C ratios are 0.44 for rigid connection condition and 0.79 for pinned connection condition, respectively.

5.3.2 Responses of the Pile Foundations

Each column of the concrete approach rests on a 2.33-foot thick footing, which is supported by 4 timber piles. The footing is reinforced at the bottom with #4 bars spaced at 6 inches on center each way. Table 5.5 lists the loads and maximum pile forces. For the

190 year earthquake, with a rigid connection condition, the maximum shear, tension and compression of a pile are 5.2 kips, 7 kips and 53 kips, respectively. With a pinned connection condition, the forces are 2.3 kips, 35 kips and 75 kips, respectively. The real capacities of the pile are not easily determined because of the lack of information about the deterioration and the underpinning. In addition, the timber pile has only 4 inches of embedment into the footing. Tensile capacity developed from this embedment is difficult to calculate. Assuming a tension capacity of 15 kips and no deterioration, the D/C ratios of the timber pile are calculated and listed in the table. For rigid connection condition, the D/C ratios are 0.6 or less. With a pinned connection condition, the D/C ratios for shear and compression are 0.13 and 0.87, respectively. The D/C ratio for tension is 2.33 for the 190 year earthquake and 1.40 for the 95 year earthquake, respectively.

5.3.3 Responses of the Columns

A typical 22 inch by 22 inch rectangular column with eight ¾ inch square bars is evaluated. Table 5.6 summarizes the maximum loads, capacities and D/C ratios for critical columns. The bending capacity of 197 kip-ft is calculated using X Section as shown in Figure 5.11. At the bottom of the column, the splice length of the ¾ inch square bars is about 18 inches, which is not sufficient to develop the strength of the bars. Therefore, a 50% reduction of the moment capacity is assumed. For both connection conditions, D/C ratio for shear is 0.53 or less. With a rigid connection condition, the moment D/C ratio at the top of the column is 2.2 for the 190 year earthquake and 1.69 for the 95 year earthquake, respectively. With a pinned connection condition, the moment D/C ratio at the bottom is 1.81 and 1.36 for these two levels of earthquake, respectively. Since the column has only ¼ inch square ties spaced at 8 inches on center, the column has very little

ductility and is likely to fail, if the capacity is exceeded. Therefore, a moment D/C ratio of 1.36 at the bottom of the column can indicate a collapse of the approach.

5.3.4 Responses of the Cap Beams

The as-built drawings show the cap beams are 36 inch by 36 inch with four 1¼ inch square bars top and bottom. With a rigid connection condition between the cap beams and the columns, under dead load and the 190 year earthquake, the shear and moment demands are 42 kips and 400 kip-ft, respectively. The shear and moment capacities are 121 kips and 663 kip-ft, respectively. The shear and moment D/C ratios are 0.35 and 0.60, respectively.

5.4 Damage State of the Concrete Approaches

The damage state of the concrete approaches under different levels of earthquakes is summarized in Chapter 8.

Table 5.1 Concrete Approach Pile Foundation Spring Constants

Longitudinal Translation (kips/in)	Transverse Translation (kips/in)	Vertical Translation (kips/in)	Rocking about Longitudinal Axis (k-in/rad)	Rocking about Transverse Axis (k-in/rad)	Rotation about Vertical Axis (k-in/rad)
26	26	1400	1.35E+05	1.35E+05	0.00E+00

Table 5.2Concrete Approach Structural Dynamic Characteristics

(Pinned Connection Condition)

MODE	FREQUENCY (CYC/SEC)	PERIOD (SEC)
1	0.36	2.8
2	0.44	2.29
3	0.49	2.06
4	2.88	0.35
23	4.8	0.21
24	4.8	0.21
27	10.3	0.097
28	10.5	0.095
29	10.7	0.094
30	10.9	0.091
31	11.7	0.086
32	11.7	0.085
33	12.8	0.078
34	1	0.077
35	14	0.071

X DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	1.11E-16	2	88.41	3	3.12E-17	4	4.22E-30
20	4.96E-11	21	2.73E-11	22	1.70E-18	23	8.79
Y DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	35.99	2	1.21E-16	3	52.44	4	7.00E-32
22	3.47E-02	23	3.12E-19	24	8.79		
Z DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
26	8.30E-15	27	15.77	28	3.14E-24	29	0.15
30	2.88E-26	31	56.77	32	2.63E-02	33	5.92
34	2.29E-25	35	20.3	36	2.90E-22		

Table 5.3 Concrete Approach Structural Dynamic Characteristics

(Rigid Connection Condition)

MODE	FREQUENCY (CYC/SEC)	PERIOD (SEC)
1	0.83	1.2
2	0.86	1.16
3	1.18	0.85
4	2.88	0.35
5	2.95	0.34
28	10.9	0.092
29	11.3	0.088
30	11.3	0.088
31	11.8	0.085
32	11.8	0.085
33	13.4	0.074
34	13.6	0.074
35	14.2	0.07

X DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	4.58E-16	2	92.86	3	4.08E-18	4	3.82E-32
5	1.26E-30	6	4.95E-30	7	1.3	8	2.29E-03
9	3.93	10	6.82E-18				
Y DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
1	35.88	2	2.22E-16	3	59.58	4	2.39E-32
5	7.85E-32	6	1.93E-30	7	5.29E-23	8	9.37E-22
Z DIRECTION NORMALIZED MASS PARTICIPATION FACTORS							
MODE	PERCENT	MODE	PERCENT	MODE	PERCENT	MODE	PERCENT
28	29.42	29	7.62E-22	30	0.91	31	42.58
32	1.52	33	5.10E-21	34	14.23	35	10.18

Table 5.4 Concrete Approach Displacements at Bent Caps

Under 190 years Earthquake

	Longitudinal Displacement (in)	Transverse Displacement (in)	Long. Relative Disp. Bent Cap & Slab (in)	Bent Cap Seat Width (in)	D/C Ratio
Rigid Connection Bent Cap & Column	2.8	2	4	9	0.44
Pinned Connection Bent Cap & Column	5	4.6	7.1	9	0.79

Table 5.5 Evaluation of Concrete Approach Pile Foundation

Bent Cap & Column Connection	DL	Maximum EQ Load					Pile Force			Capacities			D/C Ratio		
	P (kips)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	Shear (kips)	Tension (kips)	Compresion (kips)	Shear	Tension	Compression	Shear	Tension	Compression
(190 Years Earthquake)															
Rigid	90.4	25.3	20.5	4	20	98	5.2	7	53	17.2	15	88	0.3	0.47	0.6
Pinned	82	3.8	5.4	7.4	156	119	2.3	35	76	17.2	15	88	0.13	2.33	0.86
(95 Years Earthquake)															
Rigid	90.4	19.0	15.4	3.0	15	74	3.9	0	45.2	17.2	15	88	0.23	0	0.51
Pinned	82	2.9	4.1	5.6	117	89	1.7	21	62	17.2	15	88	0.1	1.4	0.7

Table 5.6 Evaluation of Concrete Approach Columns

Bent Cap & Column Connection	Member	DL					Maximum EQ Load					Governing Force		Capacities		D/C Ratio	
		P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	P (kips)	Vy (kips)	Vz (kips)	My (kip-ft)	Mz (kip-ft)	Shear	Moment	Shear	Moment	Shear	Moment
(EQ of 190 Years Return Period)																	
Rigid	11	89	0.3	0.3	1.3	0.2	17.6	4	20.5	99.4	17.1	21.2		40		0.53	
Rigid	204	60	0.1	0.5	13.3	3.4	34.9	16.5	10.7	213	363		430		193		2.2
Pinned	1	82	0	0	0	0.2	3.8	2.2	8	170	47	8	176	40	97	0.2	1.81
(EQ of 95 Years Return Period)																	
Rigid	11	89	0.3	0.3	1.3	0.2	13.2	3.0	15.4	74.6	12.8	16.2		40		0.41	
Rigid	204	60	0.1	0.5	13.3	3.4	26.2	12.4	8.0	159.8	272.3		326		193		1.69
Pinned	1	82.0	0.0	0.0	0	0	2.9	1.7	6.0	128	35	6.0	132	40	97	0.15	1.36

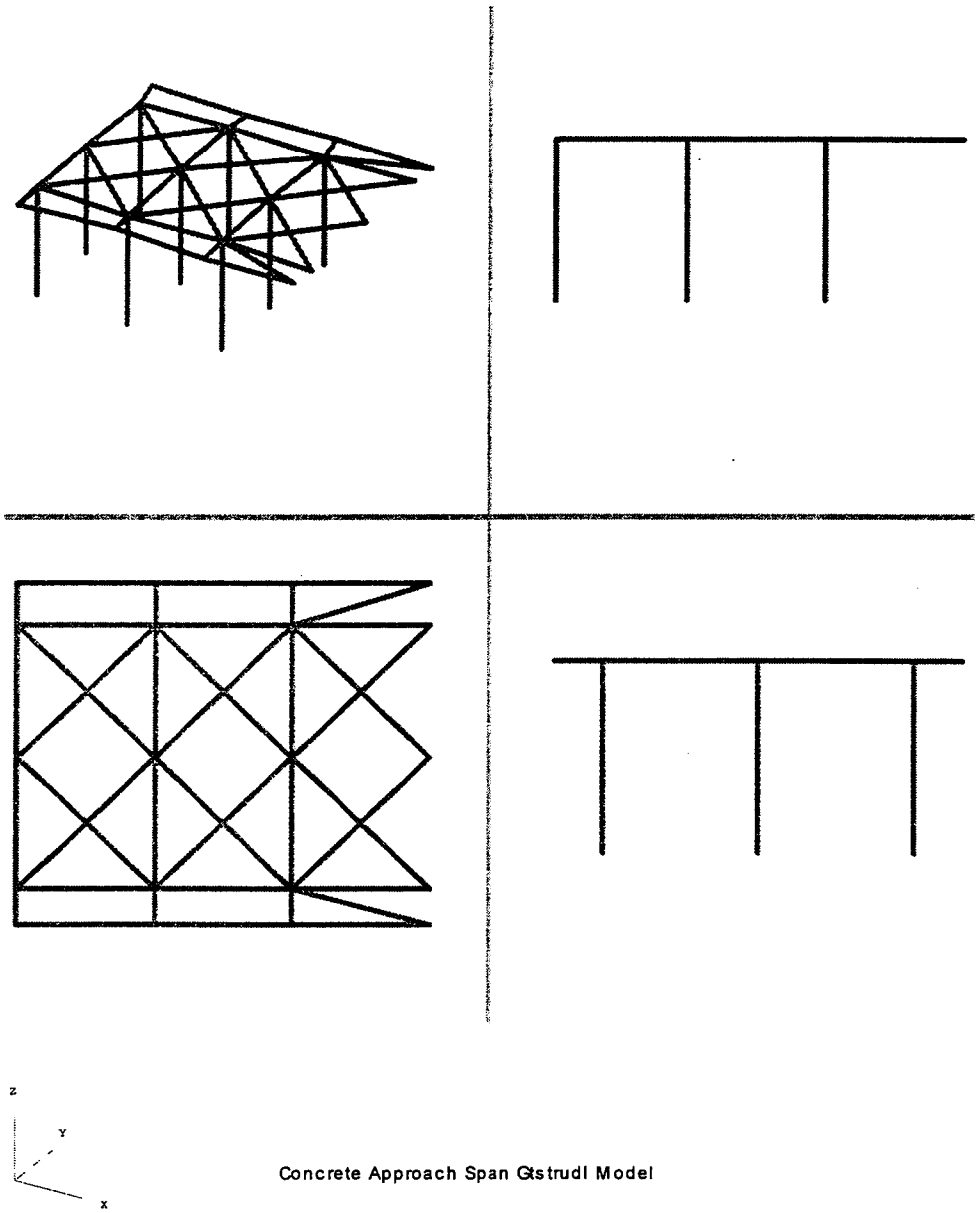
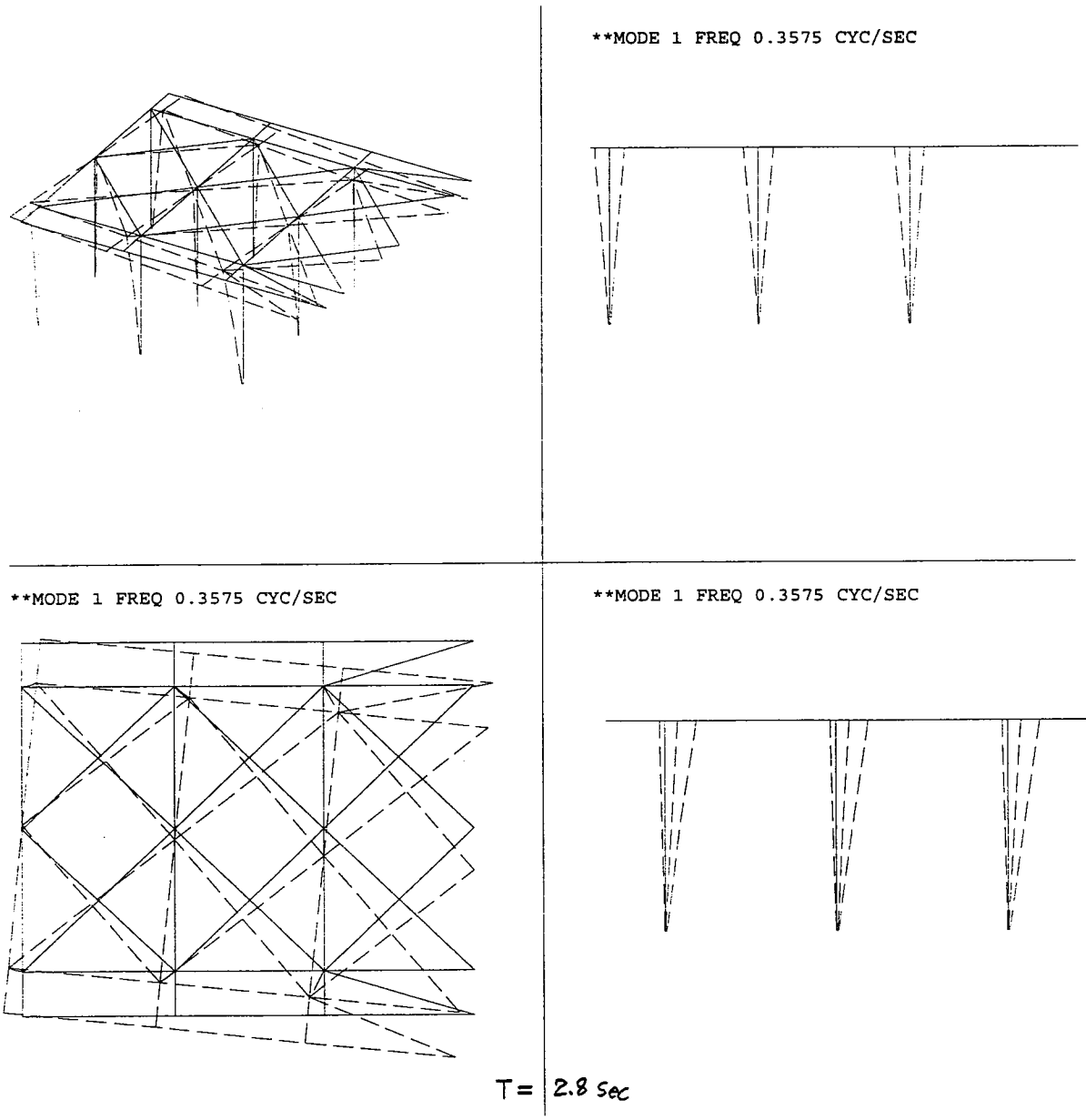


Figure 5.1 Concrete Approach Gtstrudl Model

**MODE 1 FREQ 0.3575 CYC/SEC



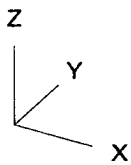
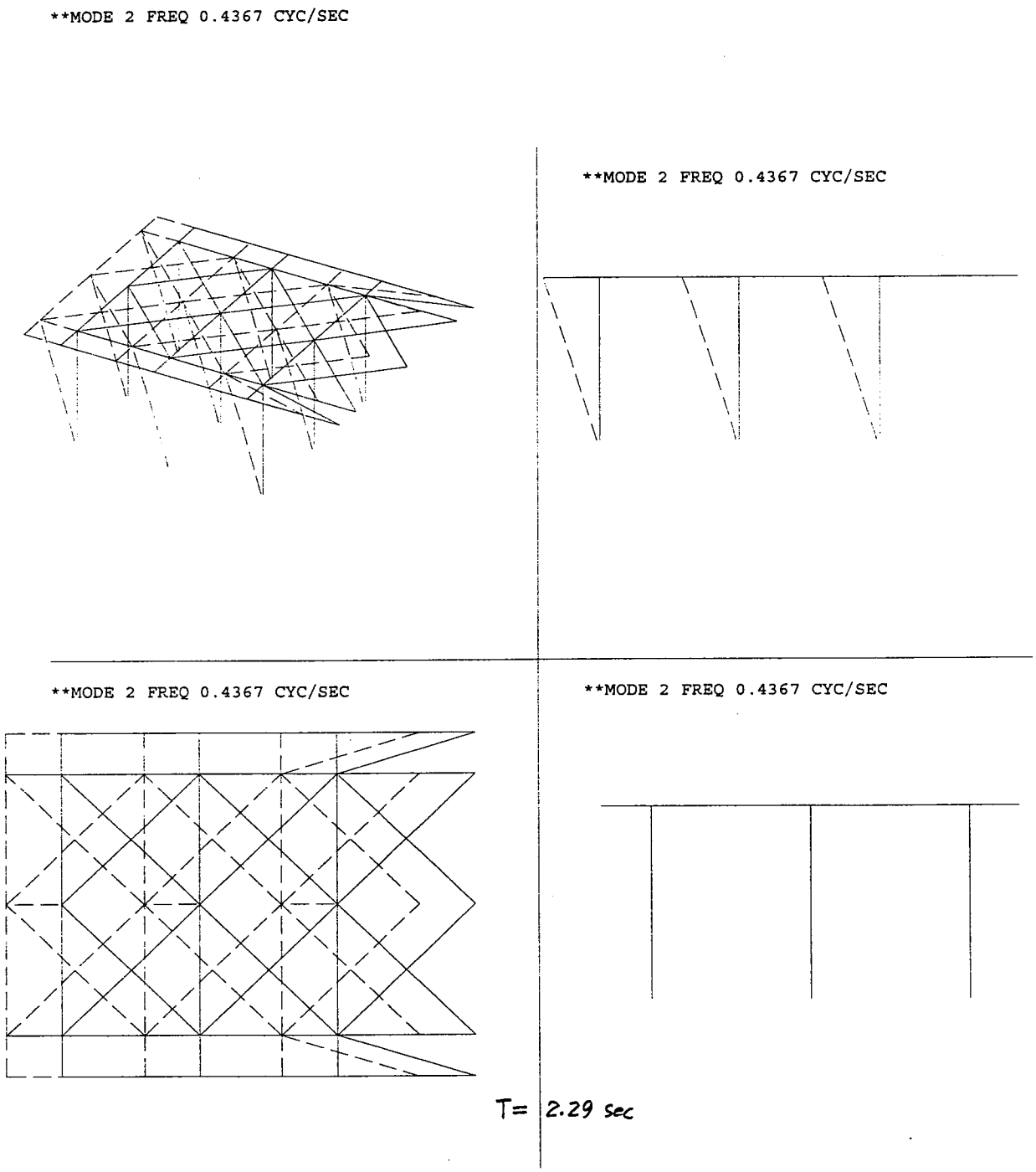


Figure 5.2 Concrete Approach Mode Shape # 1

Feet Kips



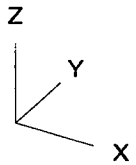
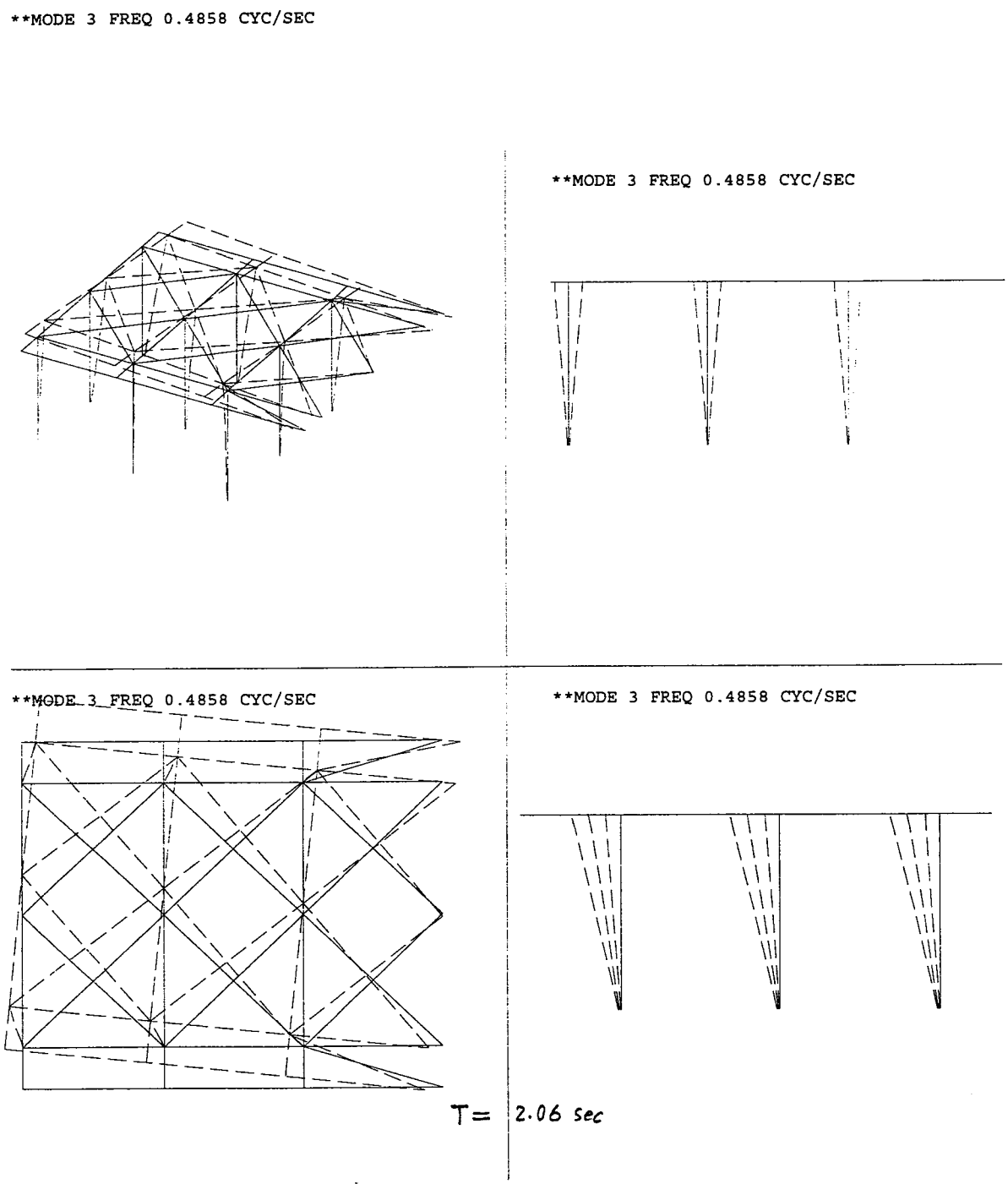


Figure 5.3 Concrete Approach Mode Shape # 2

Feet Kips



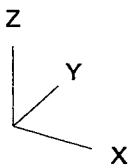
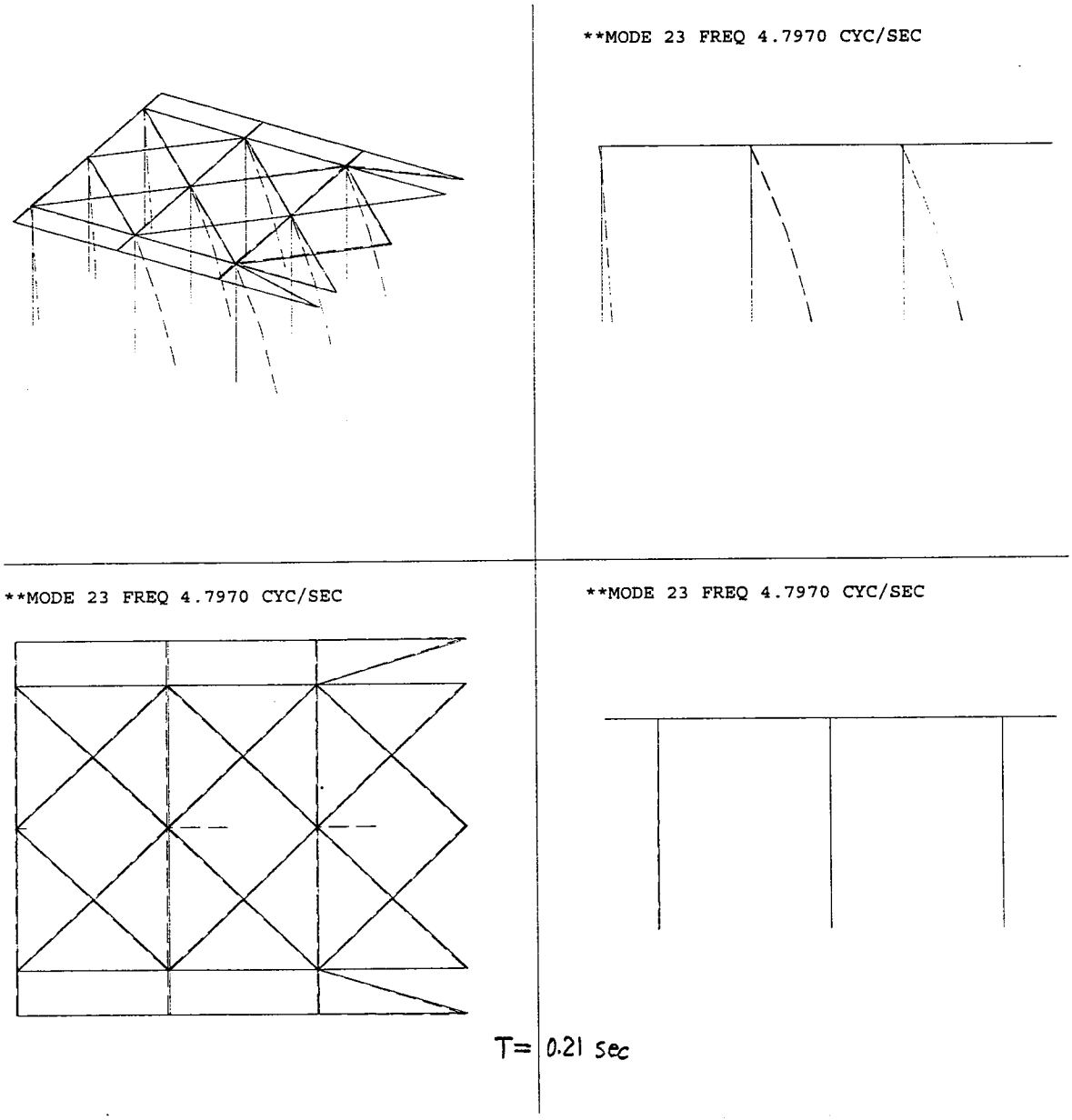


Figure 5.4 Concrete Approach Mode Shape # 3

Feet Kips

**MODE 23 FREQ 4.7970 CYC/SEC



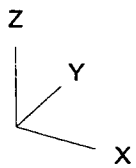


Figure 5.5 Concrete Approach Mode Shape # 23

Feet Kips

**MODE 24 FREQ 4.7971 CYC/SEC

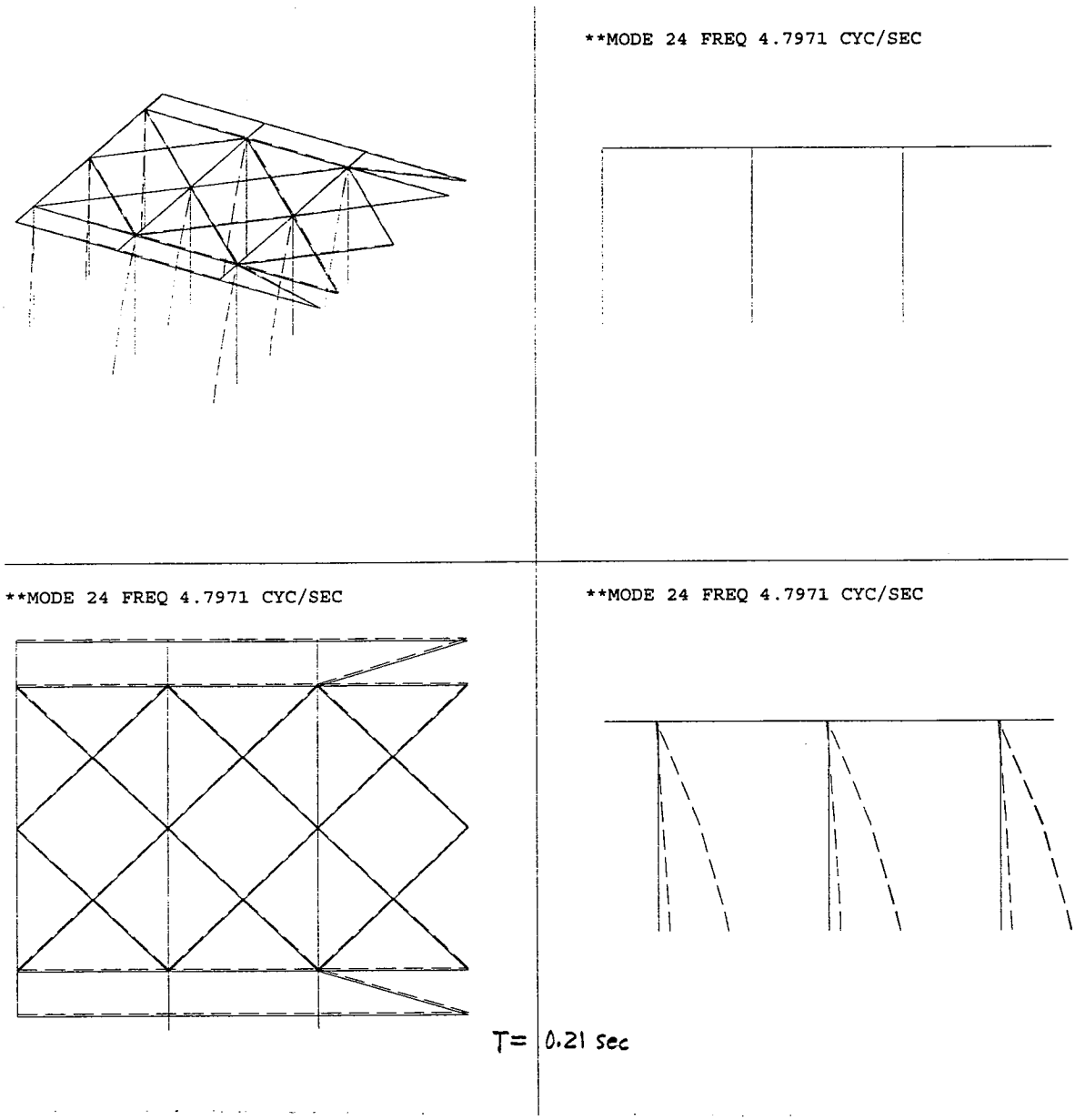
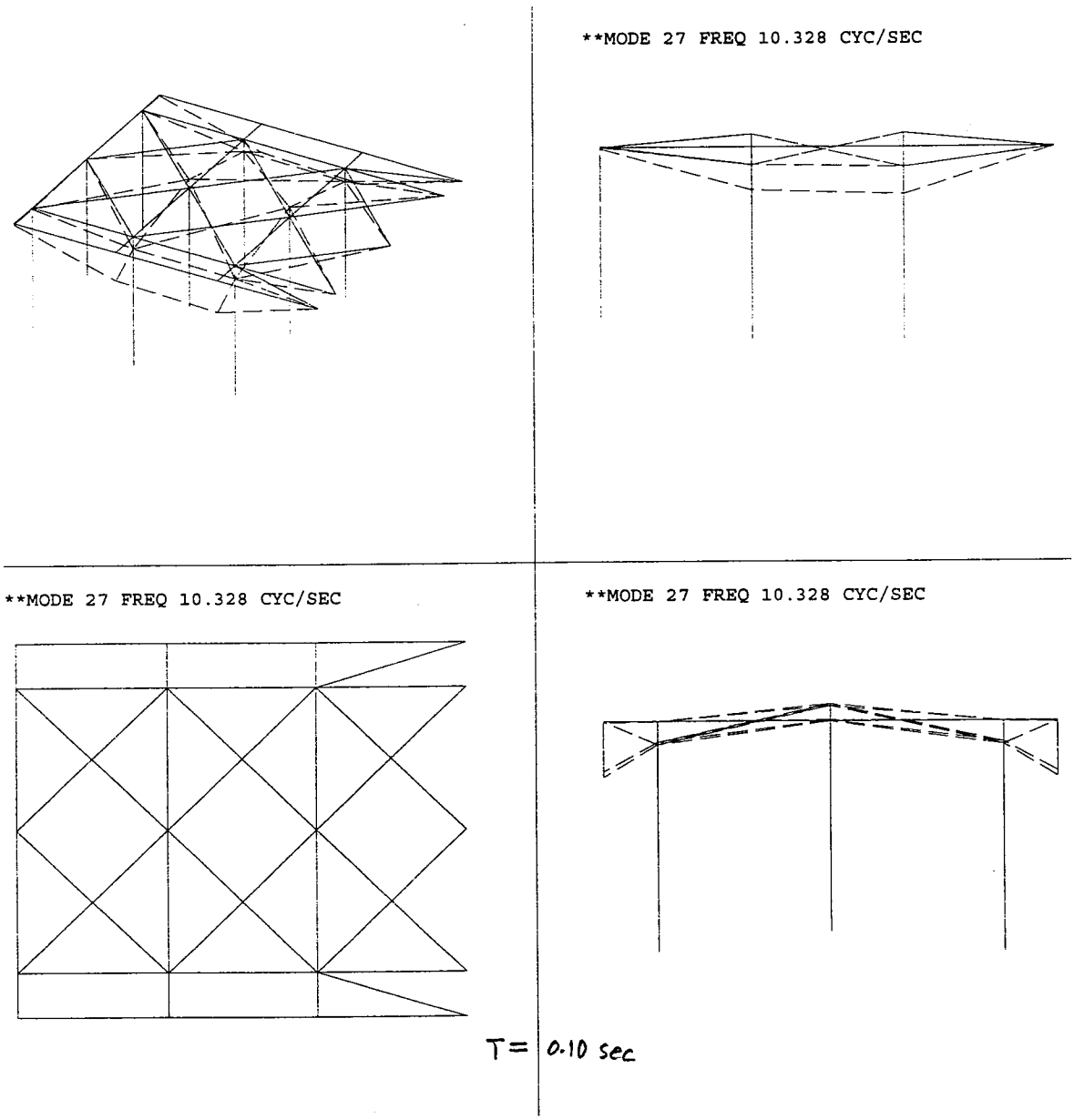


Figure 5.6 Concrete Approach Mode Shape # 24

Z
Y
X

Feet Kips

**MODE 27 FREQ 10.328 CYC/SEC



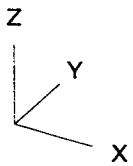


Figure 5.7 Concrete Approach Mode Shape # 27

Feet Kips

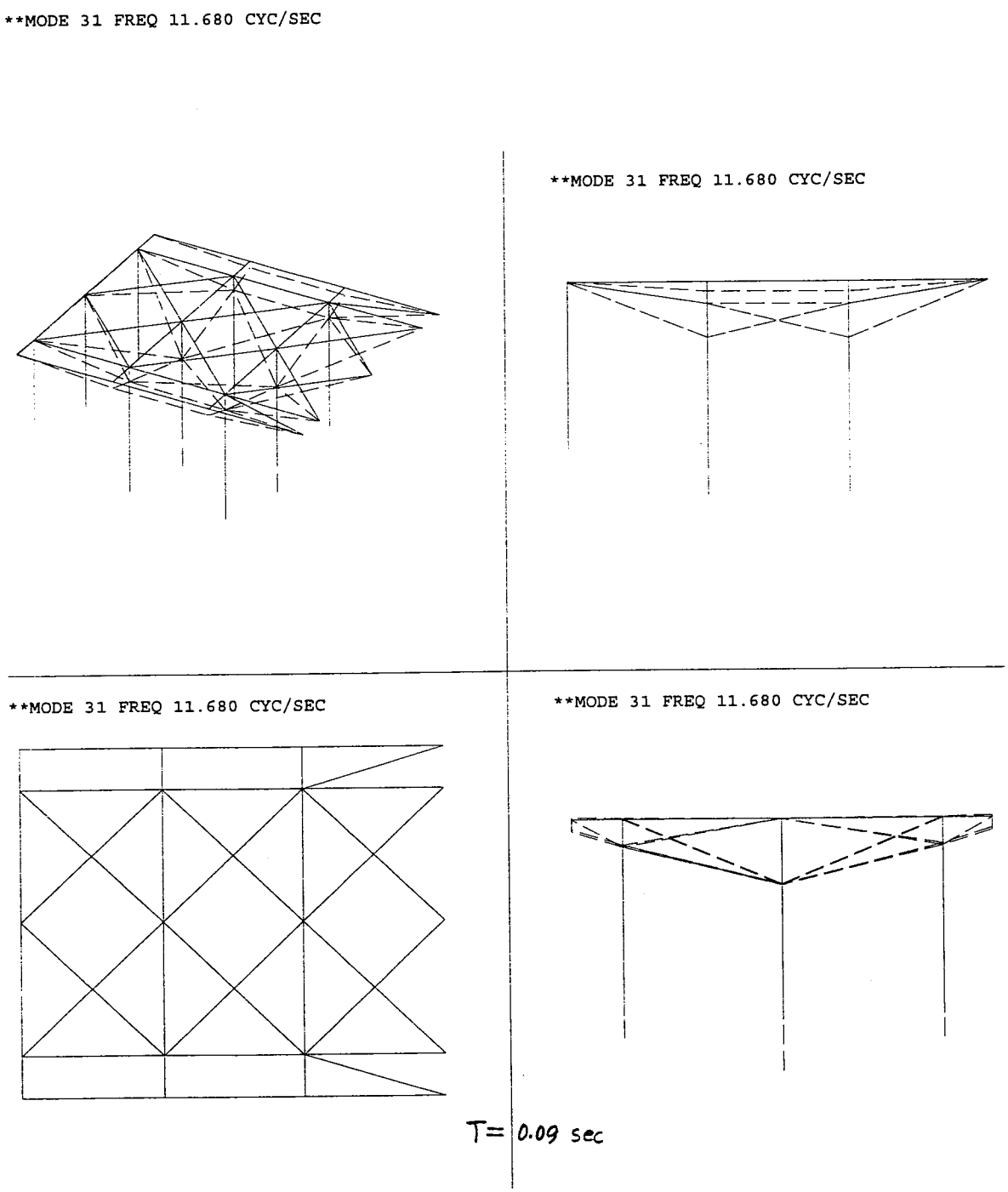
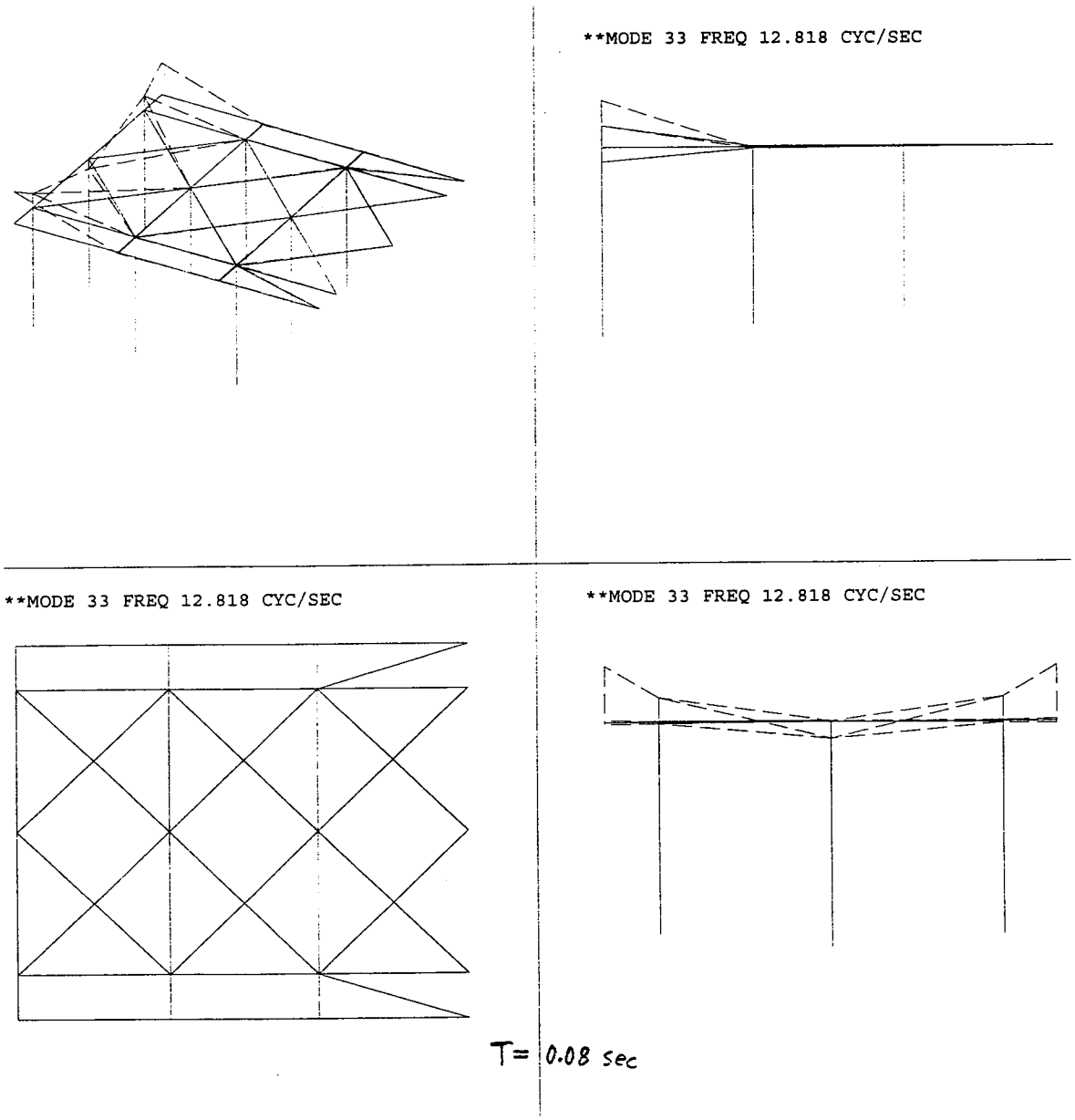


Figure 5.8 Concrete Approach Mode Shape # 31

Feet Kips

**MODE 33 FREQ 12.818 CYC/SEC



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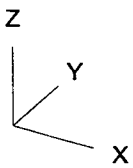

Figure 5.9 Concrete Approach Mode Shape # 33

Feet Kips

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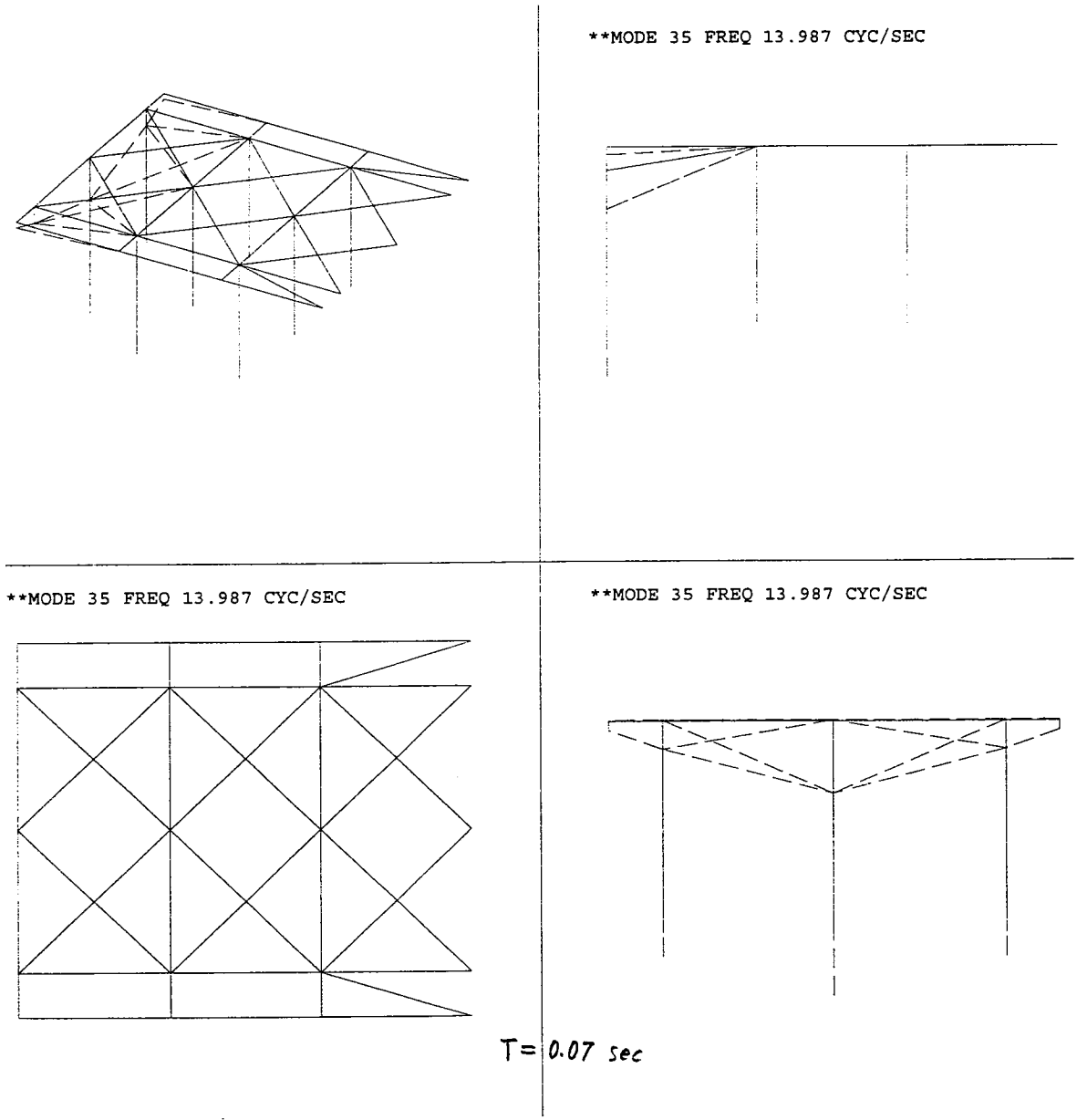


T= 0.07 sec

Figure 5.10 Concrete Approach Mode Shape # 35

Z
Y
X

Feet Kips

```
xSECTION
VER._2.30,_MAR-08-98
(C)'98 Mark S. Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
01/02/2001, 16:10
File: kingcol2.xs
KING COUNTY SQARE CO
Area_G(ft^2)=    3.36
I_G(ft^4)=       0.94
Axial (kips)=    25.0
Percent steel=   0.93
--------------------
Moment-Curv. Curve :
Mmax ft-k=      193.1
phi_max    = 0.000833
--------------------
Rebar Yield info:
My (k-ft)=      118.4
phi_y      = 0.000081
Ec   (ksi)=      3123
--------------------
Idealized values:
(CT) dark blue lines
others see print-out
Mp (k-ft)=      187.0
phi_p      = 0.000704
Icr(ft^4)=       0.27
```

Angle of Rotation = 30°

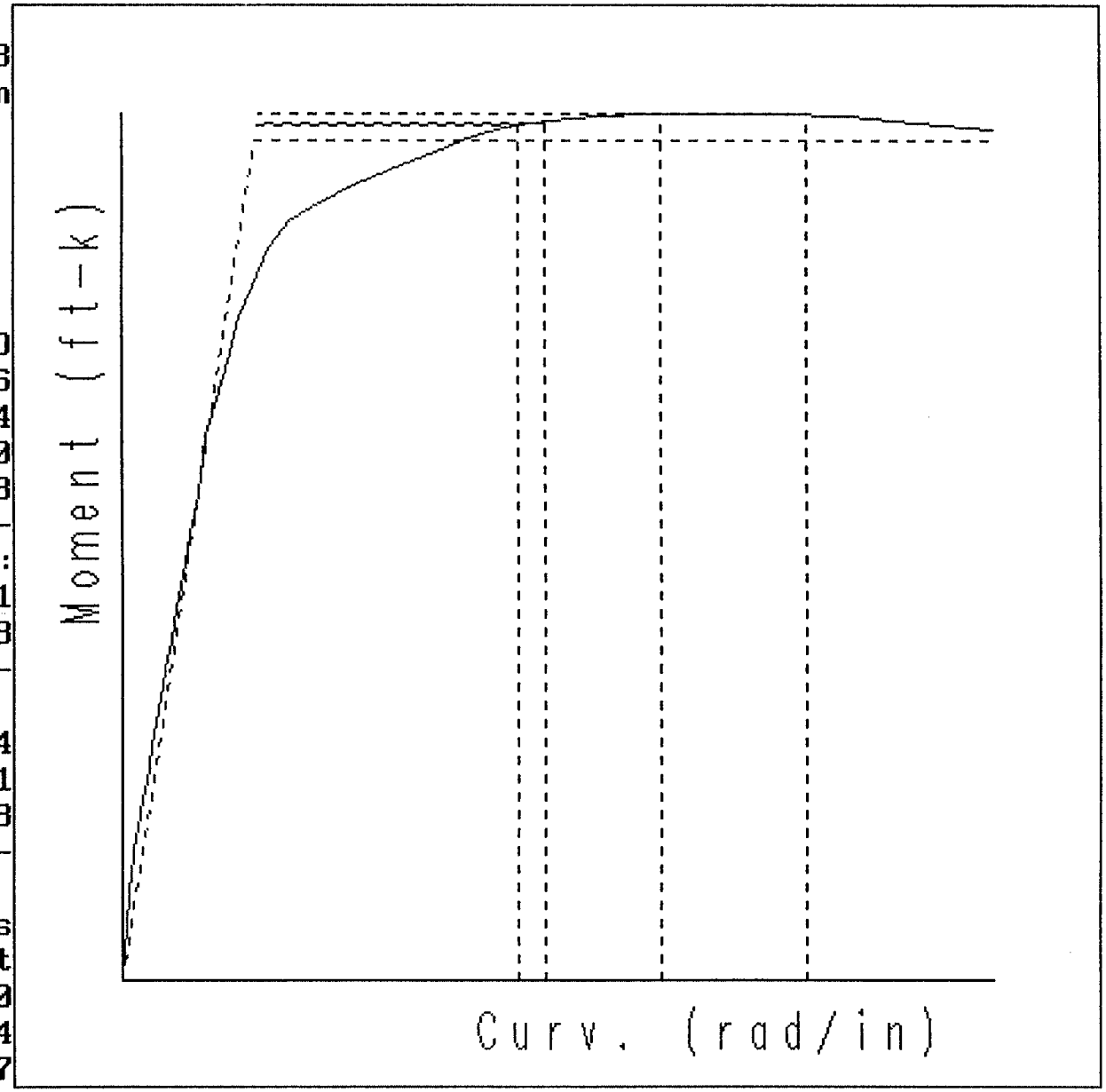


Figure 5.11 Moment Capacity of Concrete Approach Column

6. SEISMIC RETROFIT STRATEGIES

The following two alternative strategies of seismic retrofit are considered in this study:

Strategy I: The bridge will not collapse and will be repairable after the 95 year earthquake,

The bridge will not collapse under the 190 year earthquake.

Strategy II: The bridge will not collapse under the 475 year earthquake.

6.1 Level I Seismic Retrofit

6.1.1 Bascule Span Superstructure Retrofit (Item A)

Figure 6.1 shows the locations of different items of retrofit. Item A is the bascule span superstructure retrofit which including items A1 and A2 as illustrated in Figure 6.2. The existing top laterals between the counterweight and the Frame 0 consist of light single angles. New angles are added to the laterals to change them to double angles (Item A1). Frame 0 is located at the bearing of the bascule span in closed position. This frame does not have a complete lateral load path to bring the lateral earthquake load to the bearing from the deck level. New double angles are added to the frame. Some vertical members of the truss are replaced with large size members. A W18 wide flange beam is added to the bottom chord of the truss (Item A2).

6.1.2 Bascule Span Substructure (Item B)

Figures 6.3a to 6.3c illustrate the retrofit scheme for the bascule piers (Items B1 and B2). High strength tiedowns are placed in the bascule walls to tie the concrete seal, the footing and the wall together. These tiedowns are prestressed to create vertical compressive stresses in the footing and the walls. These compressive stresses increase the bending capacities of the bascule pier and the walls. In addition, they help to close or reduce the width of the existing horizontal cracks in the bascule walls. There are severe vertical cracks in the 3 foot and 4 foot thick walls. These cracks may make the 11 foot thick east and west walls to resist the loads independently. Two 1 foot thick walls are casted adjacently to the 3 foot and 4 foot thick walls. High strength prestressed bars are installed in these two new walls. These two walls and the high strength bars help to tie the four bascule walls together. The severely deteriorated concrete near the inlet is replaced with new concrete.

The fascia wall on the opposite side of the operator house depicted in Figure 6.4 is not braced at the top of the wall under transverse seismic loading. Two braced frames are installed at the bascule pier to brace the fascia wall (Item B3).

6.1.3 Truss Approach Span Superstructure (Item C)

Space is needed for the counterweight when the bridge is opened. The steel truss approaches do not have a complete lateral load path to transfer lateral seismic loads from the deck level to the supports at the bascule pier. One braced frame is installed on each side of the pier to brace the truss as shown in Figure 6.5 (Item C1). Figure 6.6

illustrates high strength rod restrainers installed between truss approach #1 and truss approach #2 (Item C3). Additional anchor bolts are also installed at the fixed support (Item C2). Figures 6.7a and 6.7b show a transverse shear key at the top of Pier 1 (Item C4). A longitudinal slot in the new concrete block at the top of the pier allows free longitudinal relative movement between the bottom strut and the concrete block. A similar shear key is placed at the top of Pier 2.

6.1.4 Truss Approach Span Substructure (Item D)

Figure 6.8 illustrates the retrofit scheme for Pier 1. Two concrete piles with steel casings are installed adjacent to the existing pier columns (Item D1). These two piles are connected to the pier by extending the existing cap and the existing spandrel beam. Two concrete beams are casted adjacent to the existing spandrel beam (Item D2). High strength bars are placed in the spandrel beams and prestressed to tie the new piles to the existing substructure.

Figure 6.9 shows a retrofit scheme for Pier 2 similar to Pier 1. Two new piles are installed (Item D3). These piles are connected to the existing substructure by extending the pier cap. The pier cap is also widened and prestressed (Item D4).

6.1.5 Concrete Approaches

Figures 6.10a and 6.10b depict the retrofit scheme for the concrete approaches. Each concrete approach consists of four 3-continuous spans. X braces and grade beams are placed along longitudinal and transverse directions (items E1 and E3). These braces are

connected to the cap beams and grade beams to avoid the transfer of large shear to the columns. In addition, rod restrainers are installed at the expansion joints (Item E2).

6.1.6 Track Girder and Counterweight Restrainers

Two track girder restrainers and two counterweight restrainers are installed at each bascule (Items F1 and F2). The locations of the restrainers are illustrated in Figures 6.11a to 6.11d. When the bridge is in the closed position, the track girder restrainer helps the existing shear lugs to prevent the slippage of the track girder on the track due to an earthquake. The counterweight restrainer restrains transverse and downward movement of the counterweight. The upward movement of the counterweight is restrained by the anchorage girder of the truss approach span. Prior to opening the bridge, the restrainers are retracted to the fully open position. Two hydraulic power units are installed at each bascule, one on each side (item F3). These two power units are interconnected so as to permit one power unit to operate the four restrainers in case of failure of one of the power units.

6.2 Level II Seismic Retrofit

Figure 6.12 shows spectral acceleration ratio of the 475 year earthquake and the 190 year earthquake. The ratio increases from 1.8 to 5.0 when the period changes from 0.5 second to 3.0 seconds. This ratio can be used to scale the seismic forces due to the 190 year earthquake and to obtain the seismic forces due to the 475 year earthquake.

The seismic retrofit scheme for the Level II retrofit includes all the retrofit items mentioned above for Level I retrofit and other items described subsequently.

Figure 6.8 shows the longitudinal shear key for the approach structure at Pier 1. This type of shear key is effective to prevent collapse of the superstructure after Pier 1 is subjected to severe damage. Figure 6.10a illustrates that additional cross braces and grade beams are needed to prevent collapse during the 475 year earthquake. Figure 6.10b shows the compaction or jet grouting at the end of concrete approaches to prevent lateral spreading of liquefied soil caused by the 475 year earthquake.

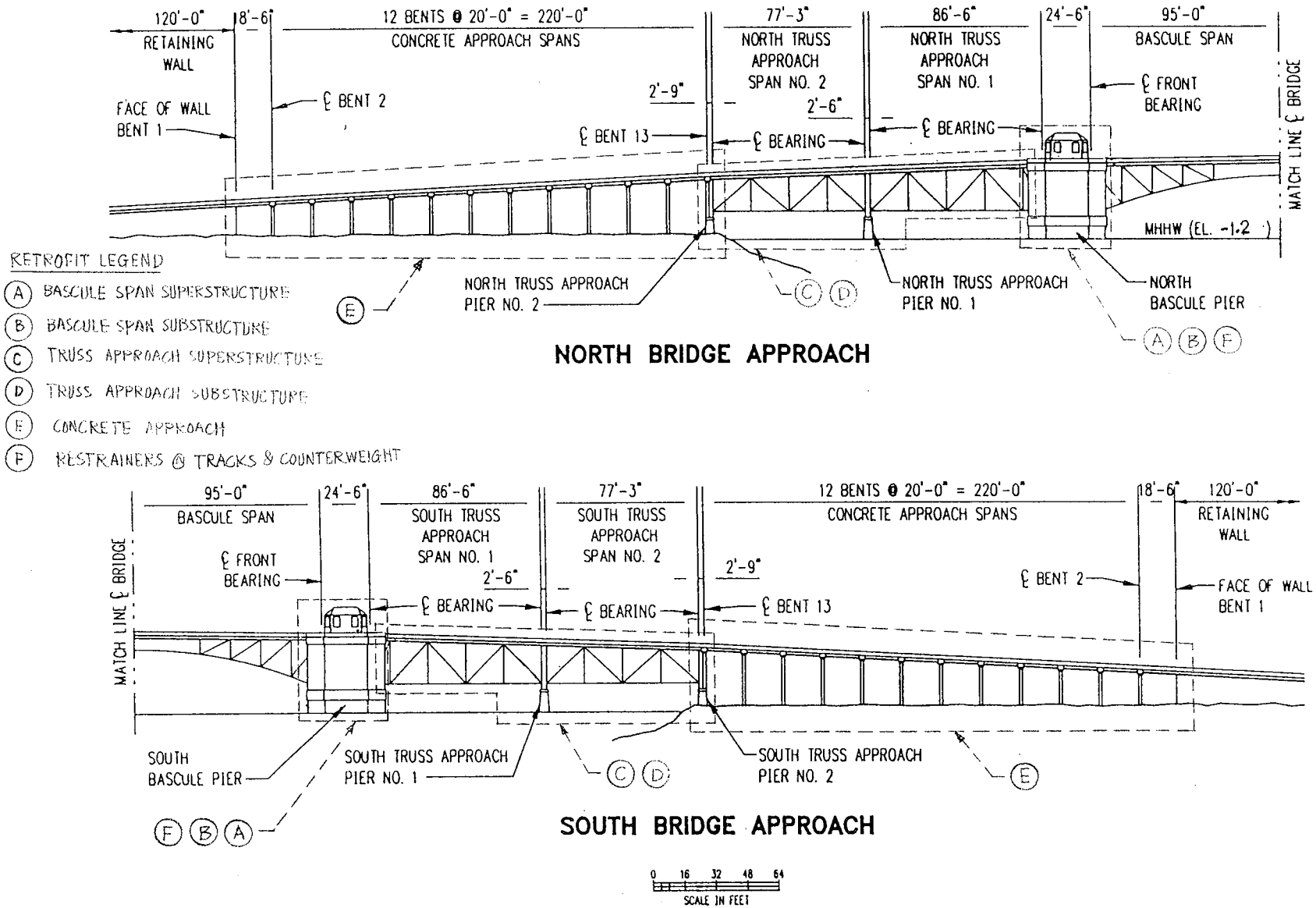
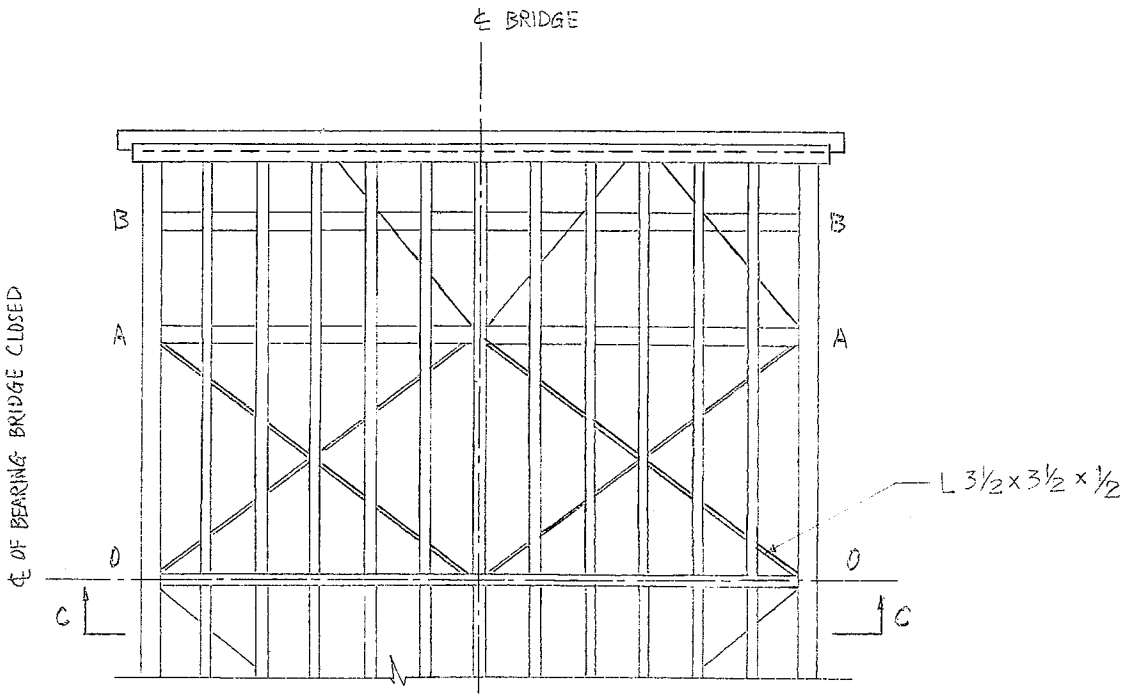
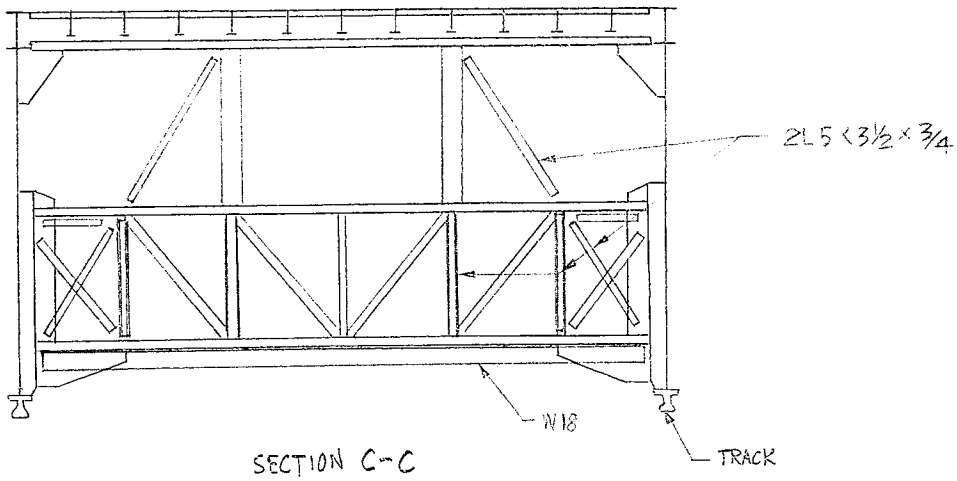


Figure 6.1 Locations of Retrofit Items



PARTIAL PLAN

(A') ADD ANGLES TO TOP LATERALS BETWEEN COUNTERWEIGHT & FRAME 0

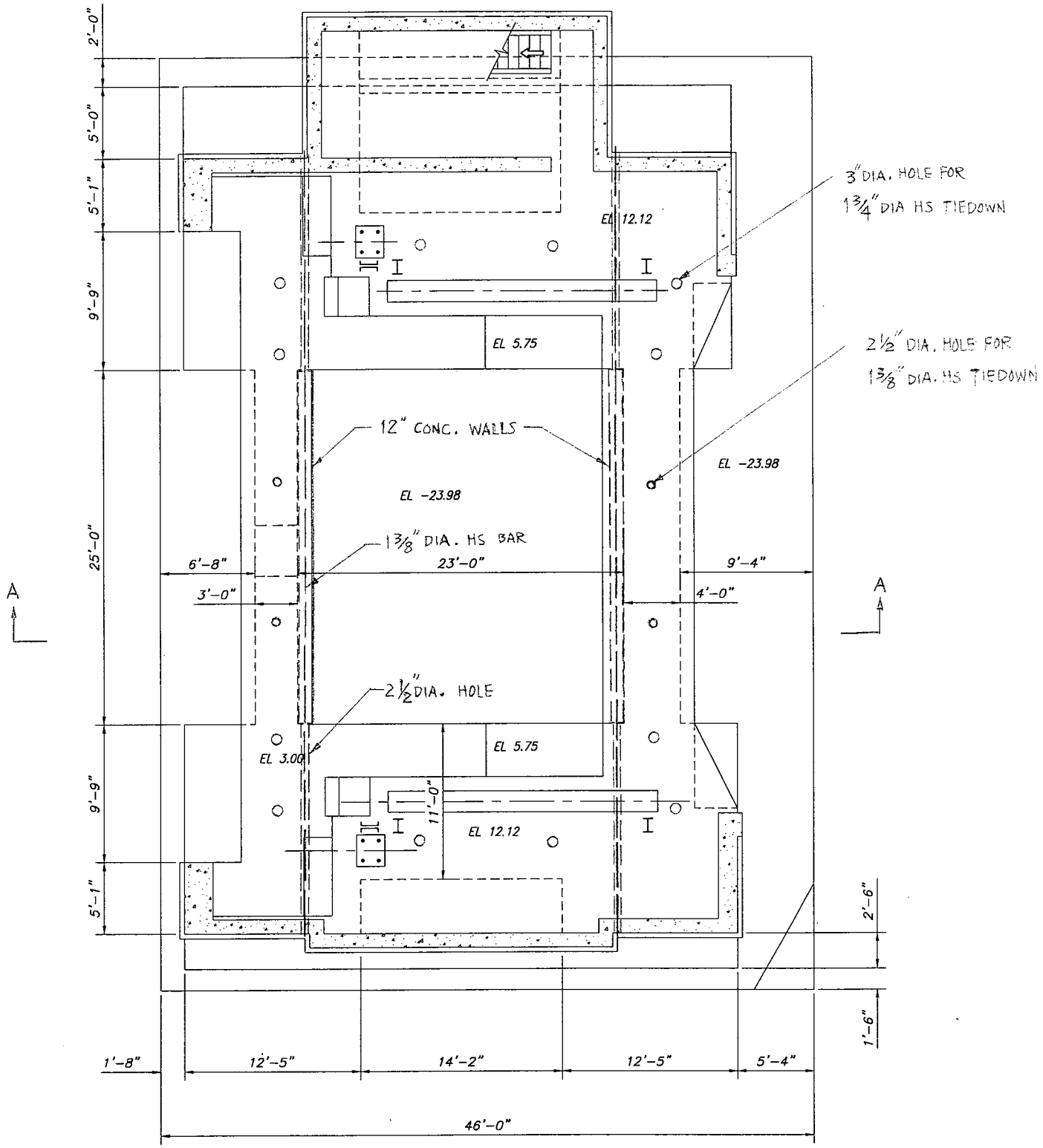


SECTION C-C

(FRAME 0)

(A2) ADD NEW MEMBERS TO FRAME 0

Figure 6.2 Retrofit of Bascule Span Superstructure



BACULE PLAN

(B1) TIEDOWNS FOR BASCULE RETROFIT

(B2) CONCRETE WALLS & HS BARS FOR BASCULE RETROFIT

Figure 6.3a Retrofit of Bascule Span Substructure

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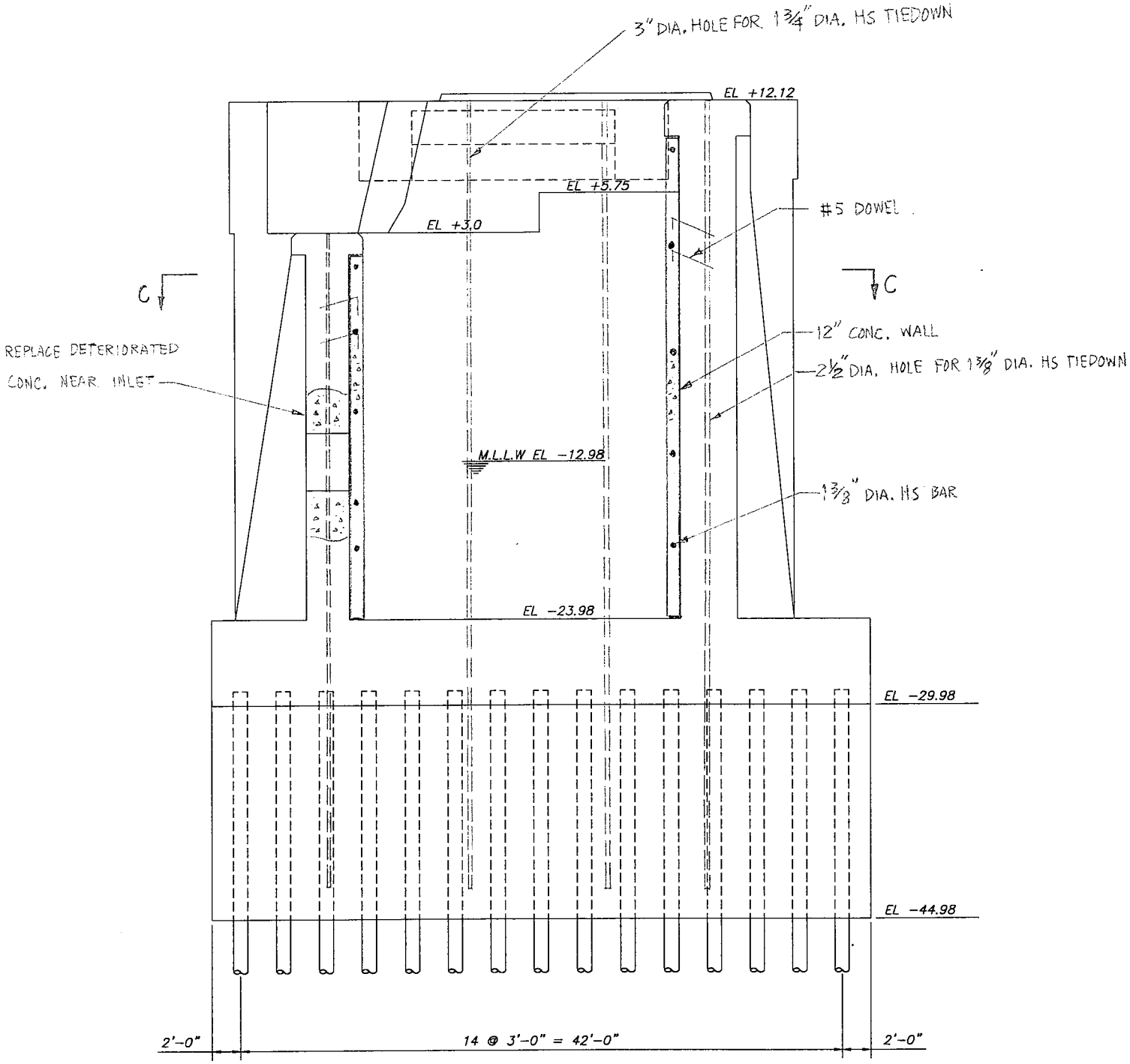

6-9

SECTION A - A

(B1) TIEDOWNS FOR BASCULE RETROFIT

(B2) CONCRETE WALLS & HS BARS FOR BASCULE RETROFIT

Figure 6.3b Retrofit of Bascule Span Substructure

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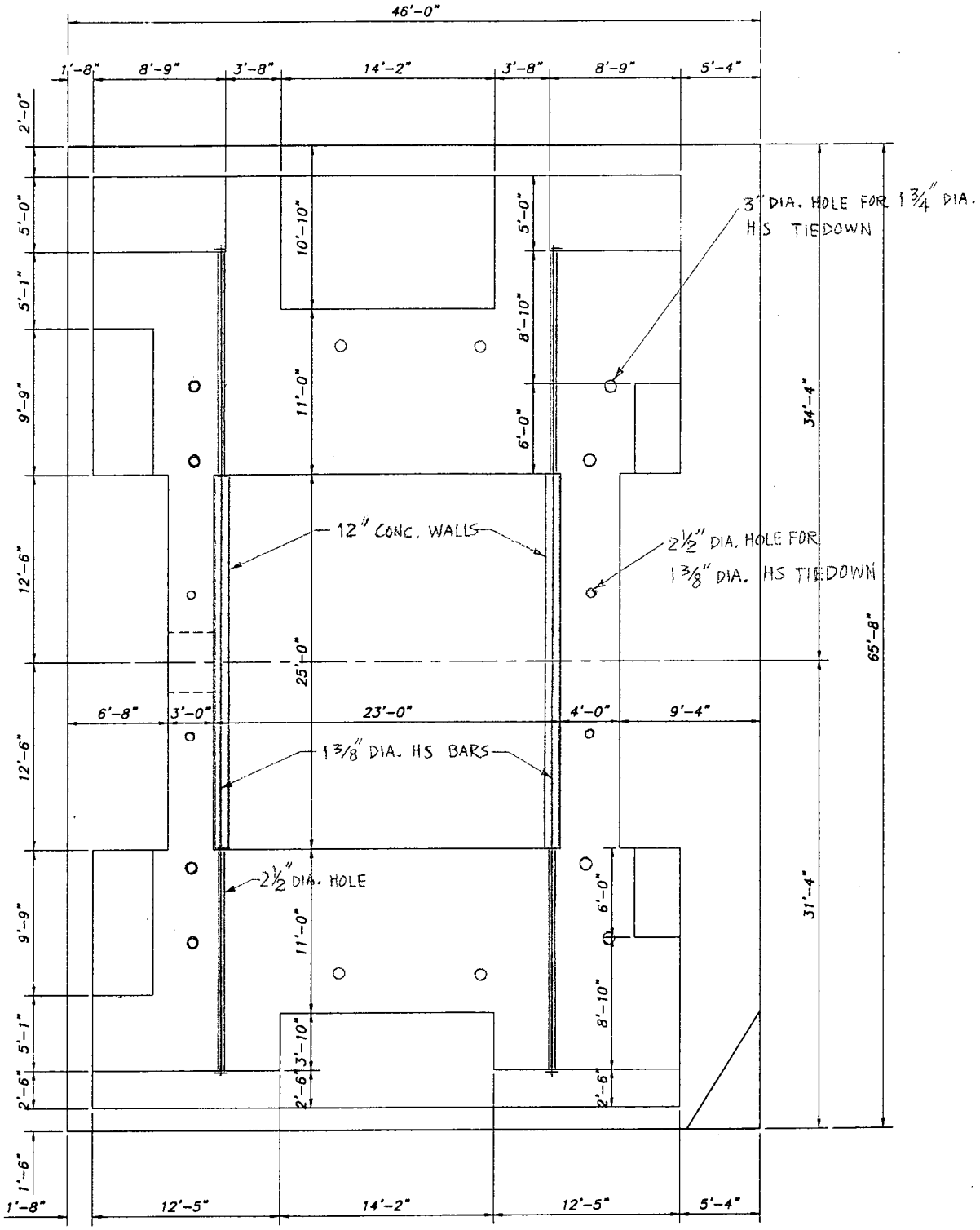

SECTION C–C

(B1) TIEDOWNS FOR BASCULE RETROFIT

(B2) CONCRETE WALLS & HS BARS FOR BASCULE RETROFIT

Figure 6.3c Retrofit of Bascule Span Substructure

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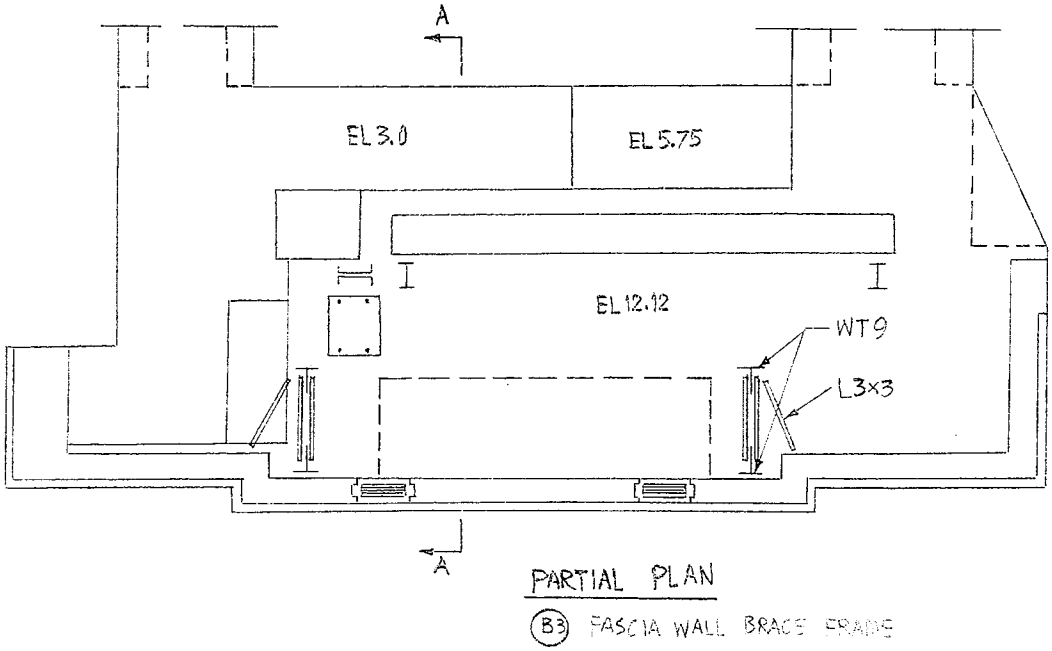


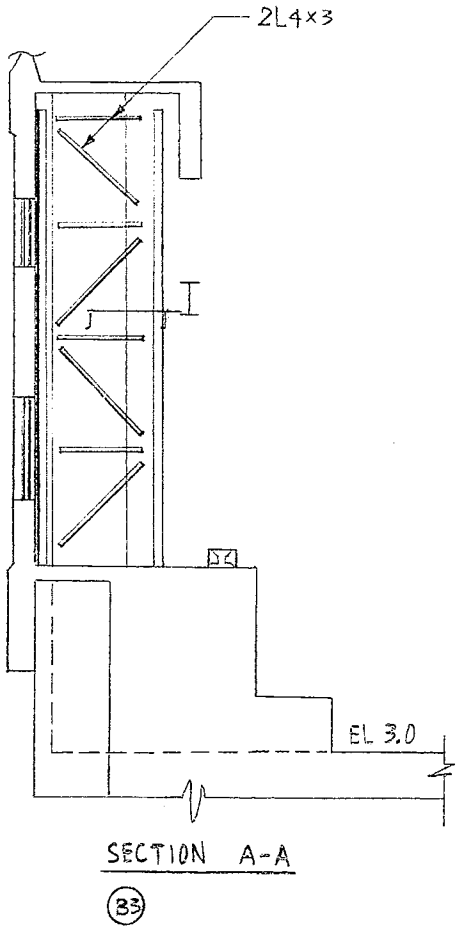
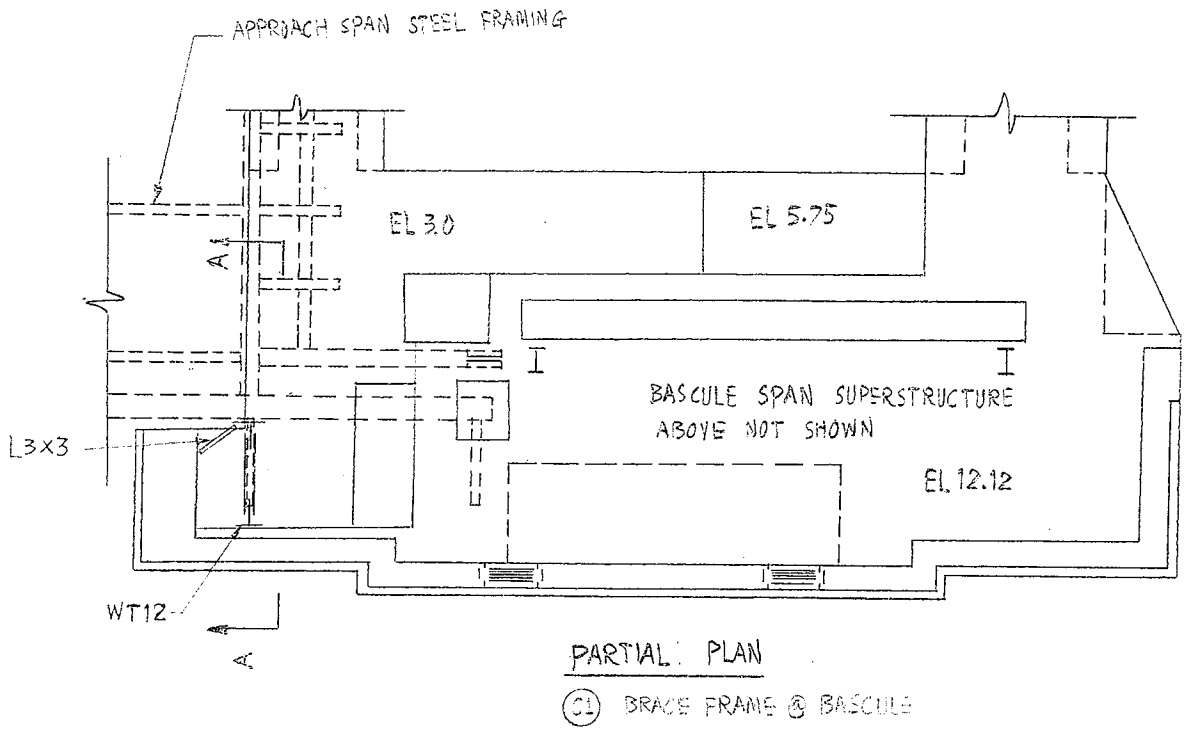


Figure 6.4 Retrofit of Bascule Fascia Wall



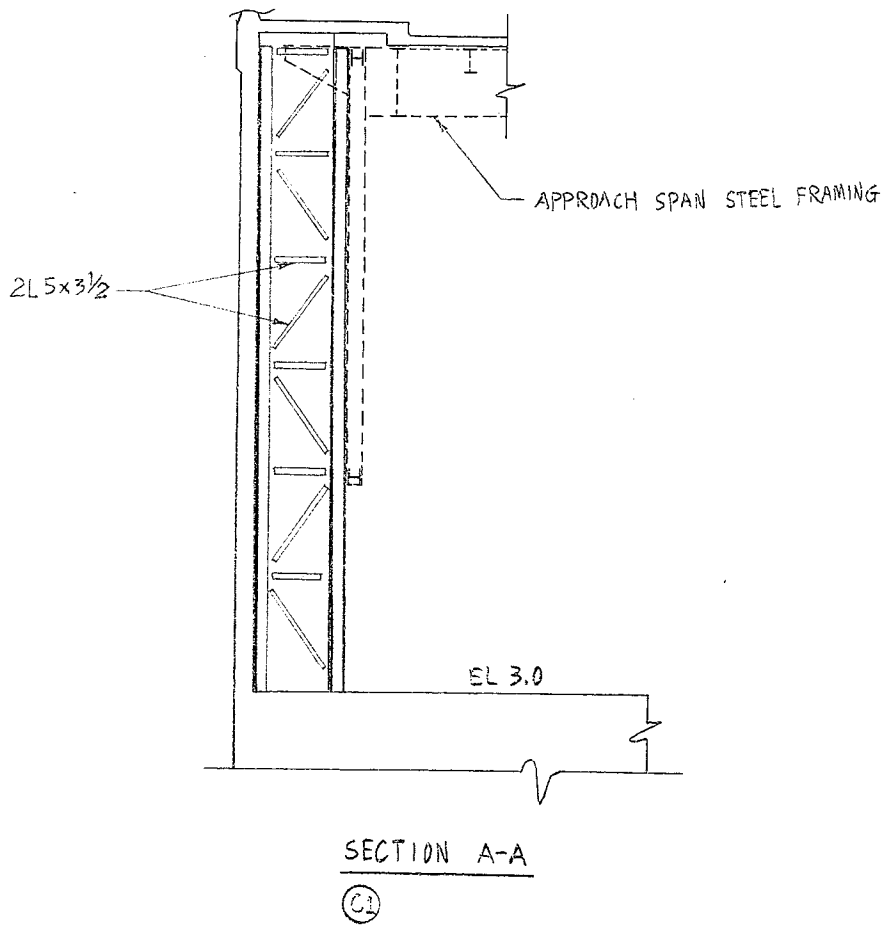


Figure 6.5 Braced Frame for Truss Approach Span Retrofit

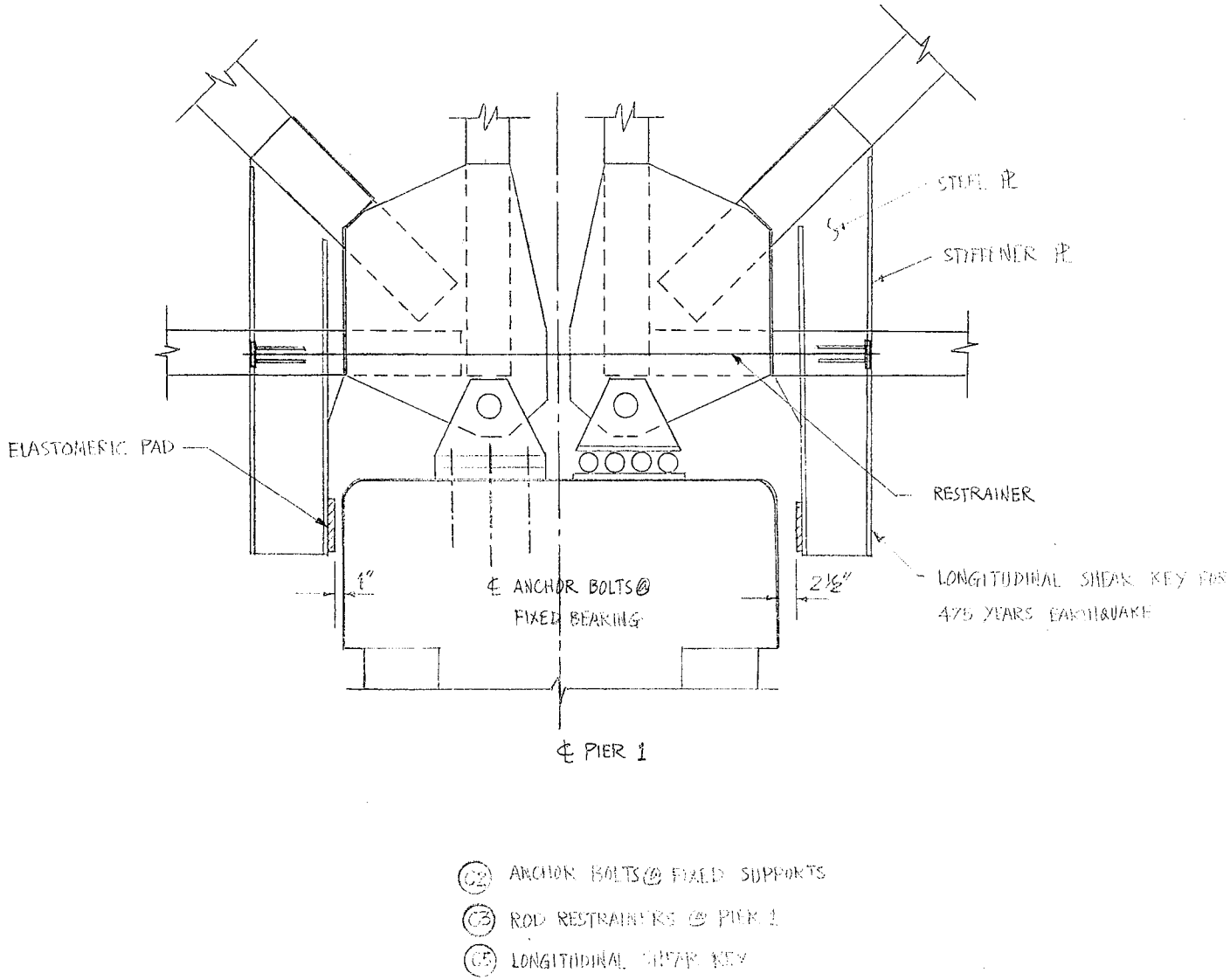
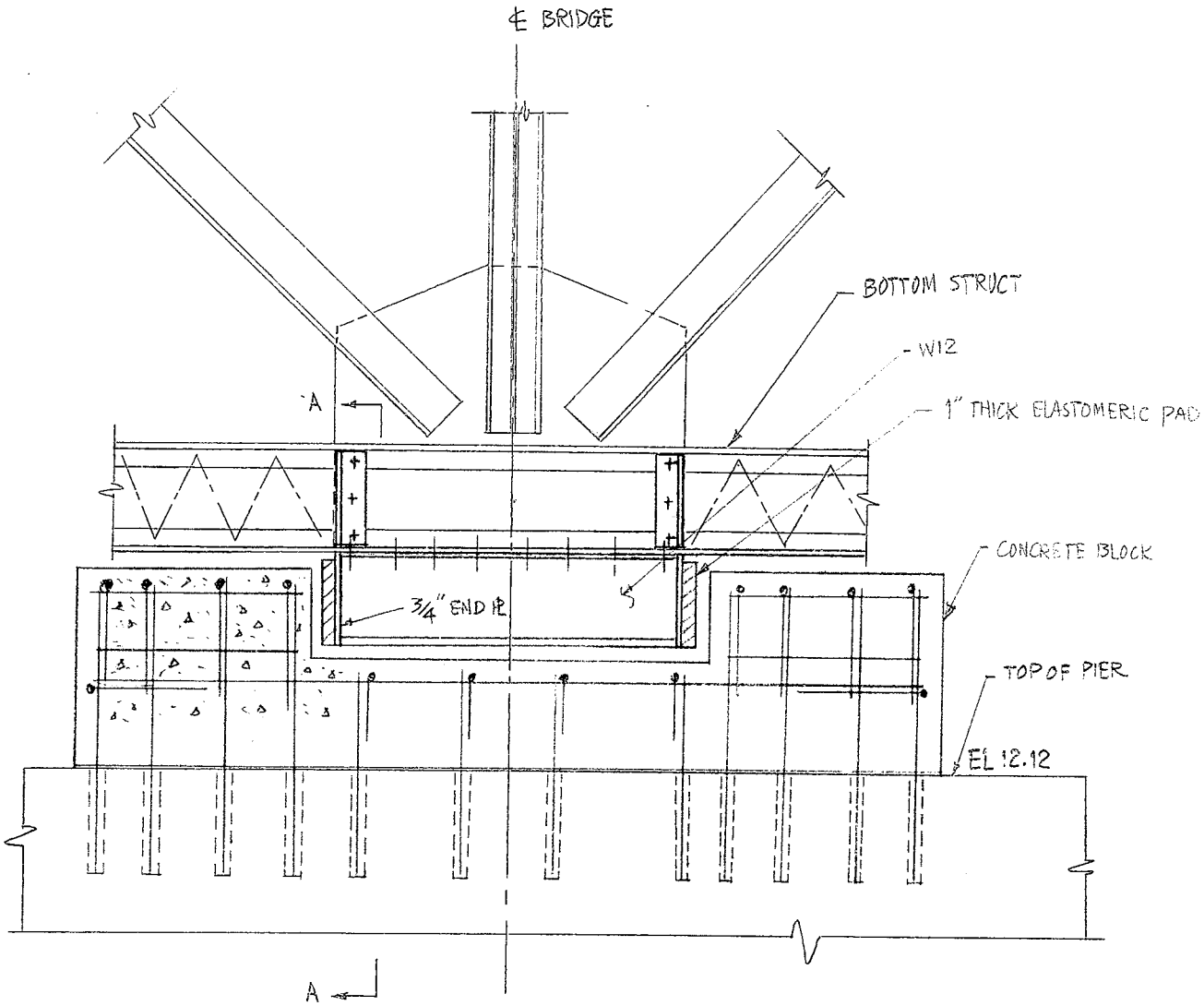


Figure 6.6 Truss Span Superstructure Retrofit at Pier 1



ELEVATION

(C4) TRANSVERSE SHEAR KEY

Figure 6.7a Truss Span Transverse Shear Key

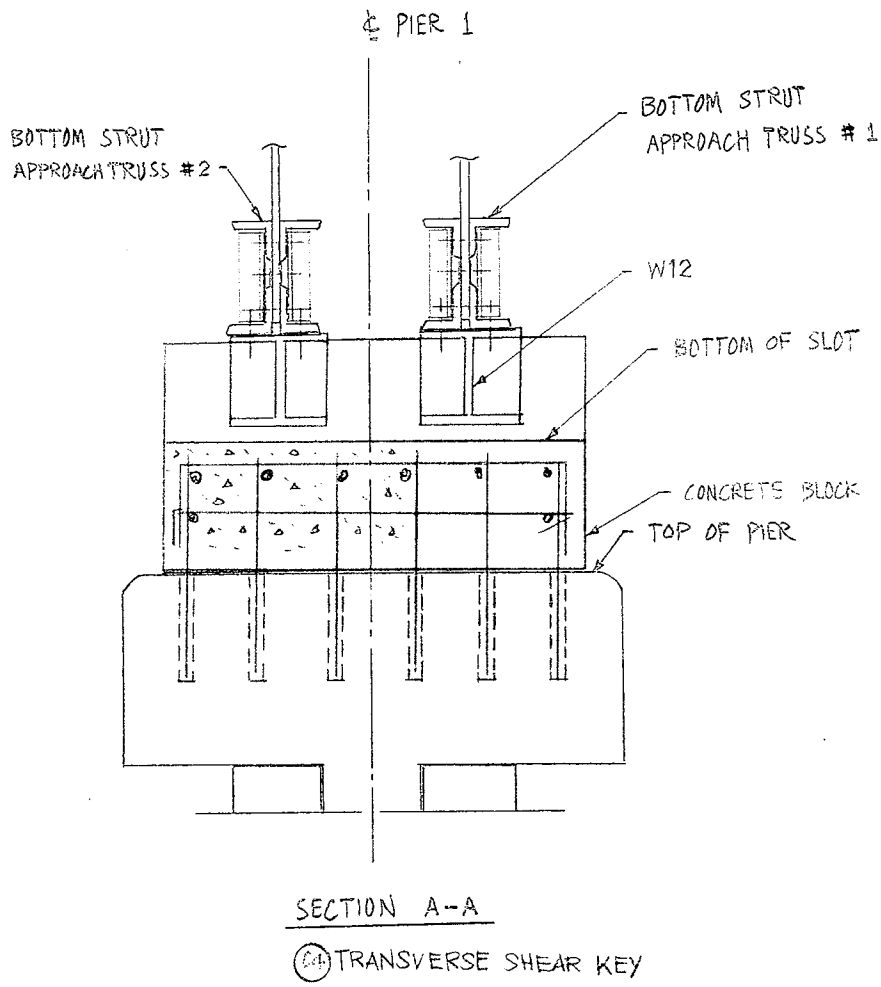


Figure 6.7b Truss Span Transverse Shear Key

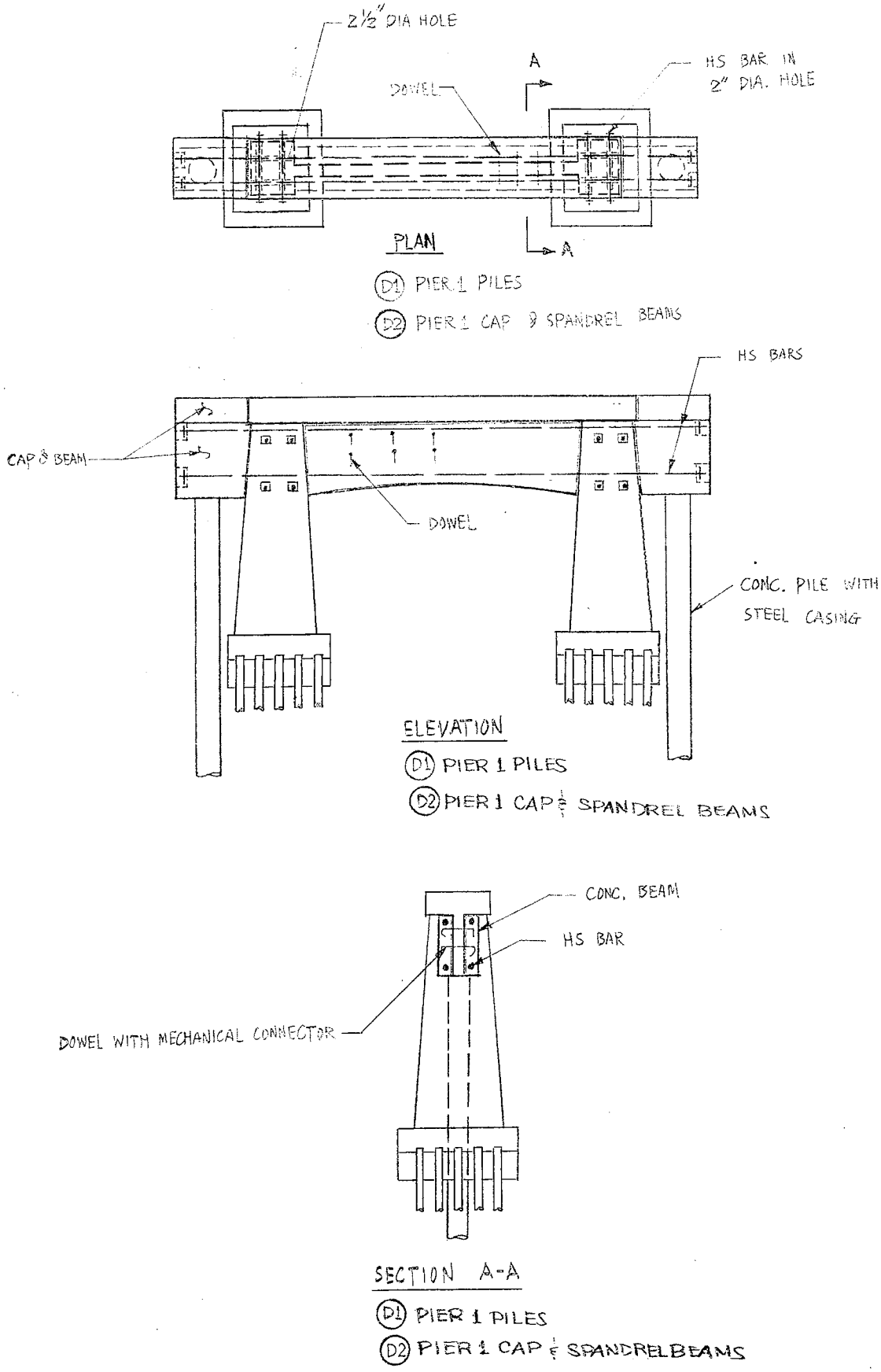


Figure 6.8 Retrofit of Pier 1

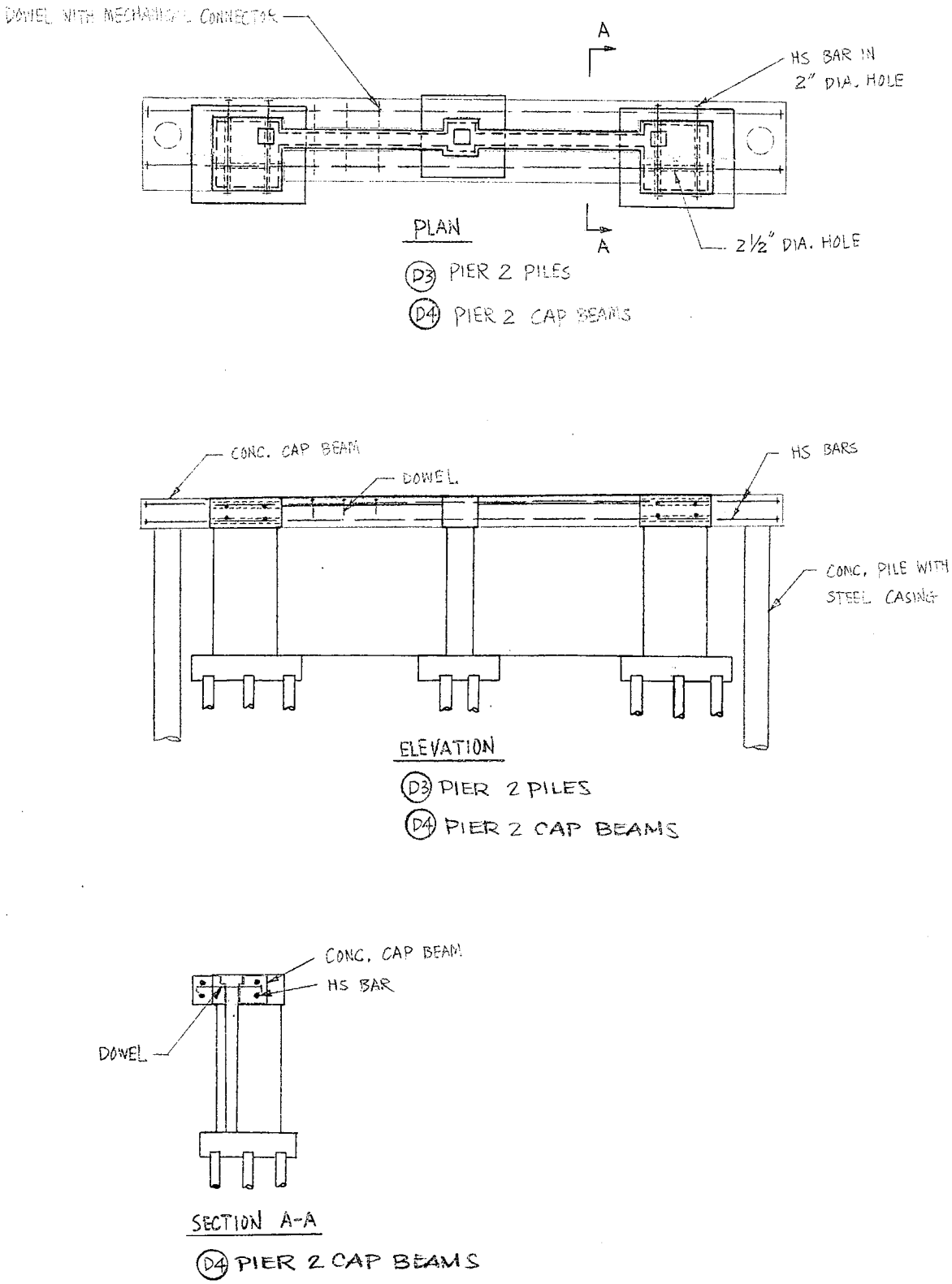


Figure 6.9 Retrofit of Pier 2

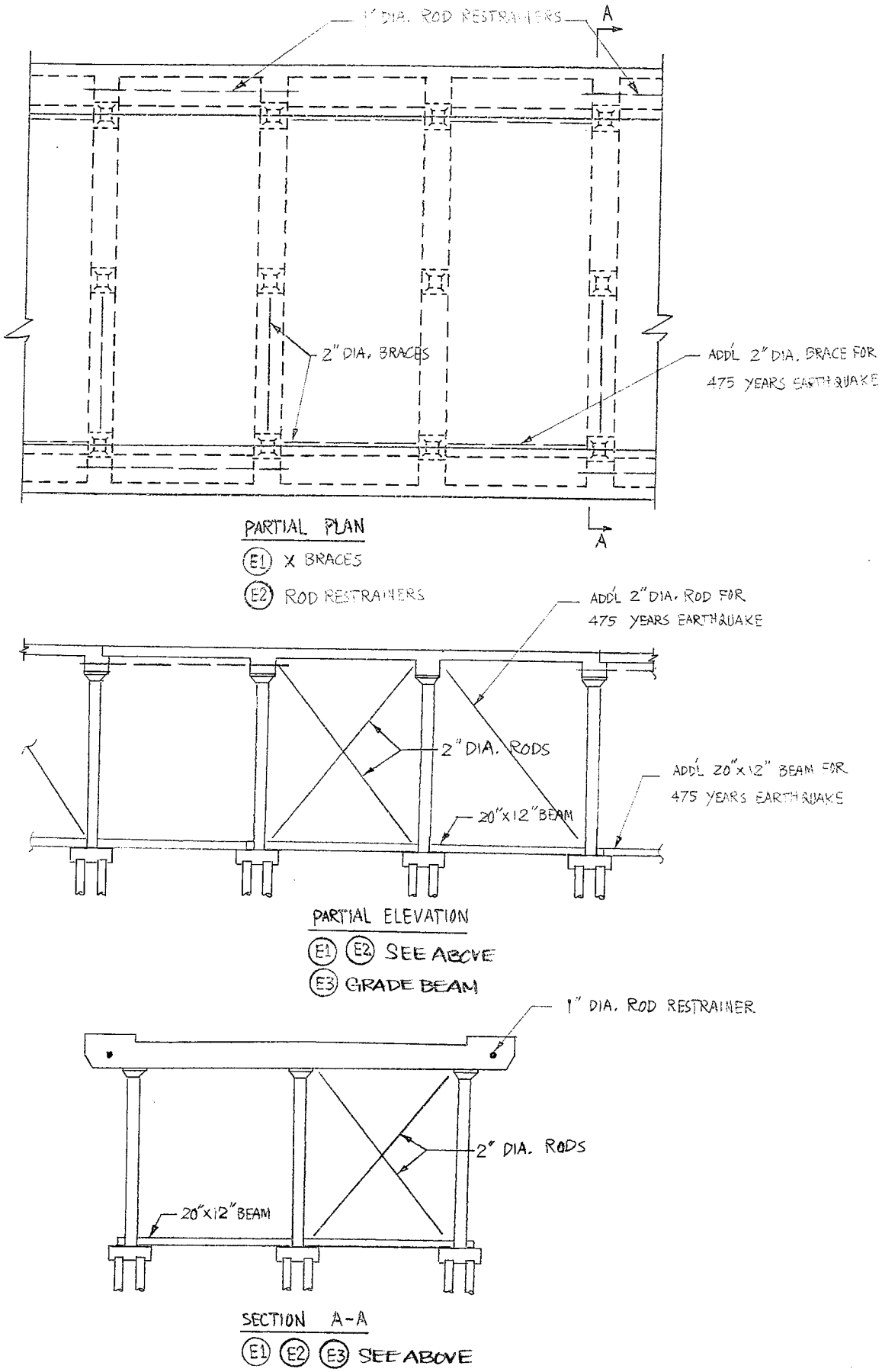


Figure 6.10a Retrofit of Concrete Approach

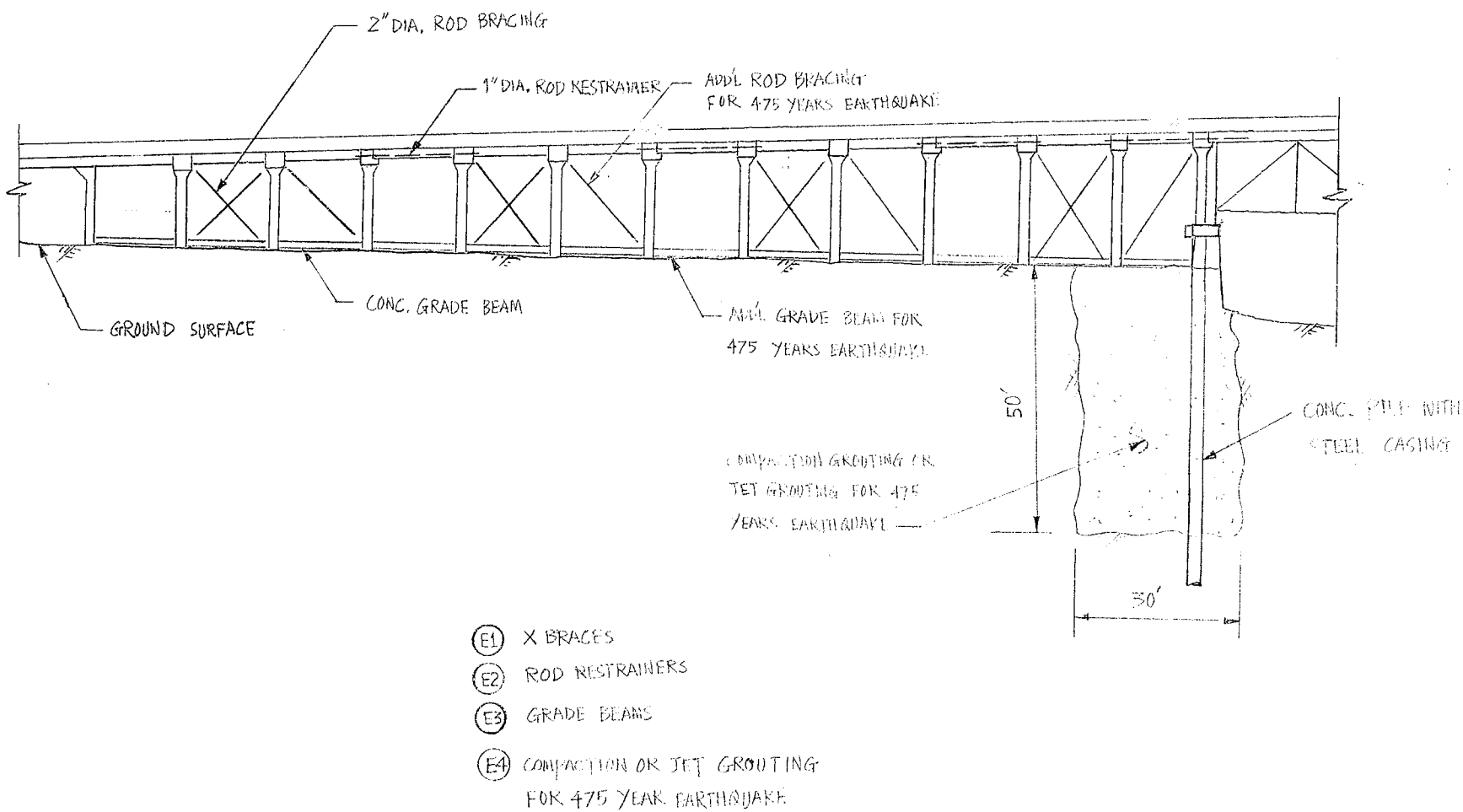
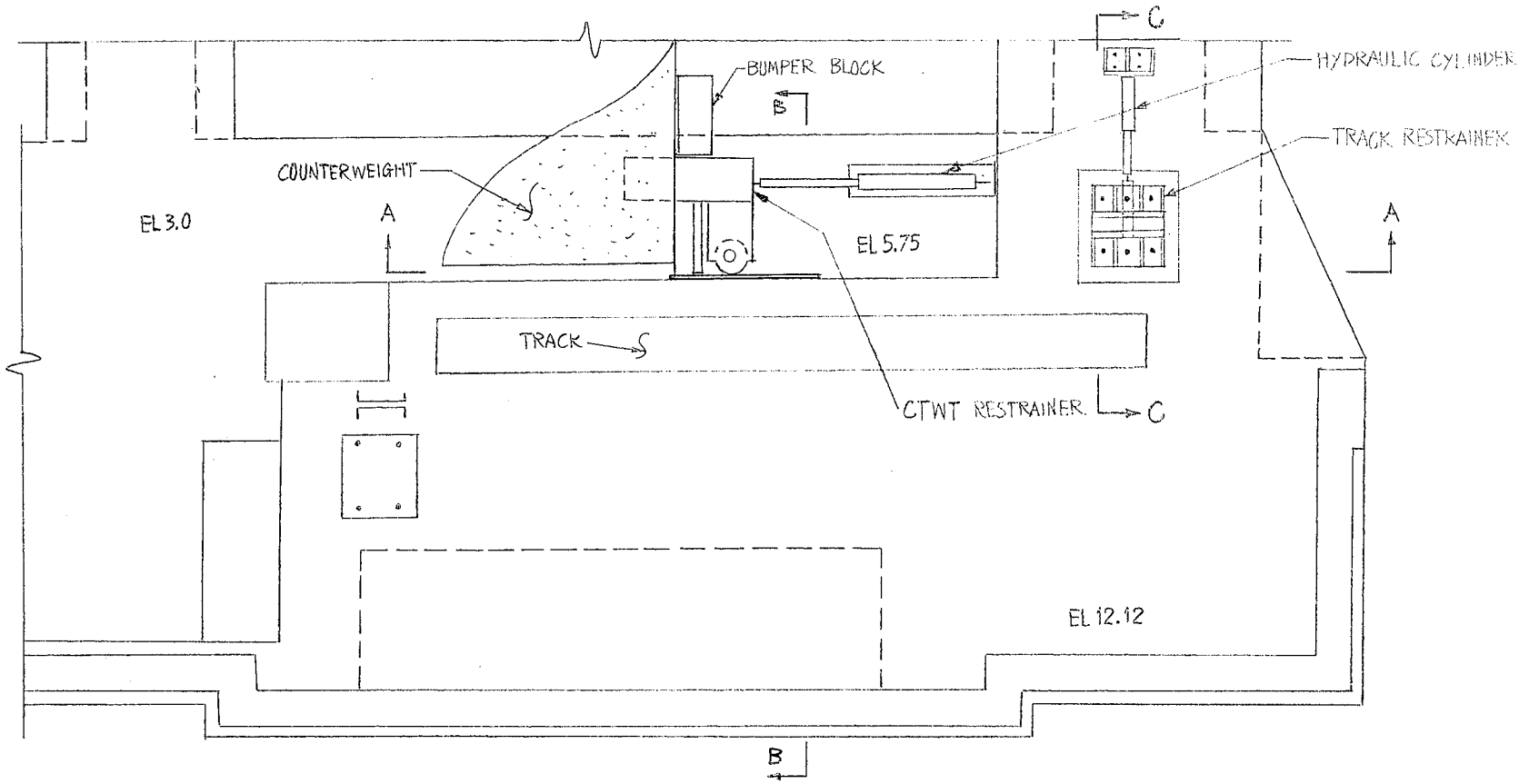


Figure 6.10b Retrofit of Concrete Approach



PLAN OF TRACK & COUNTERWEIGHT RESTRAINERS

Figure 6.11a Track and Counterweight Restrainers

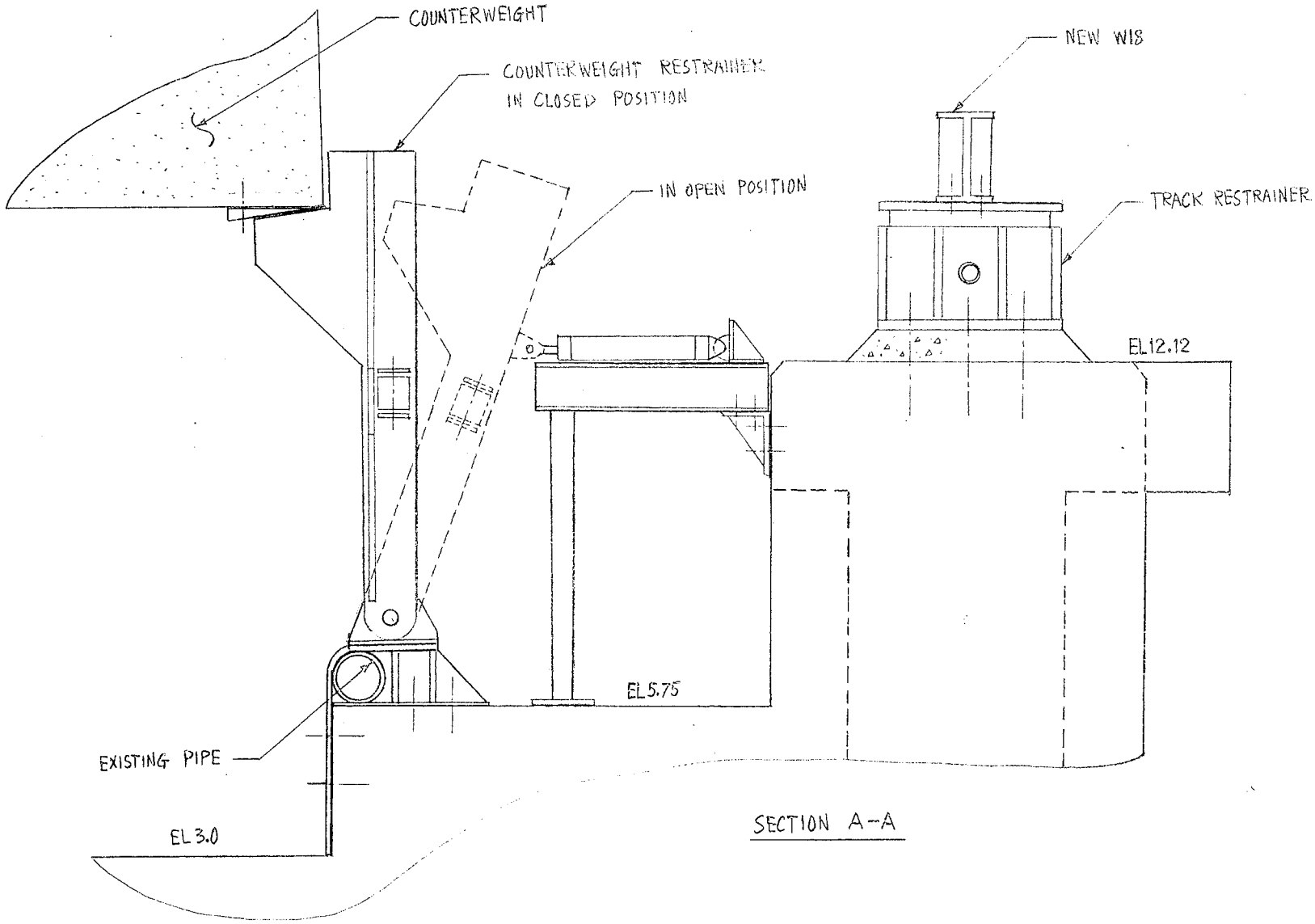


Figure 6.11b Track and Counterweight Restrainers

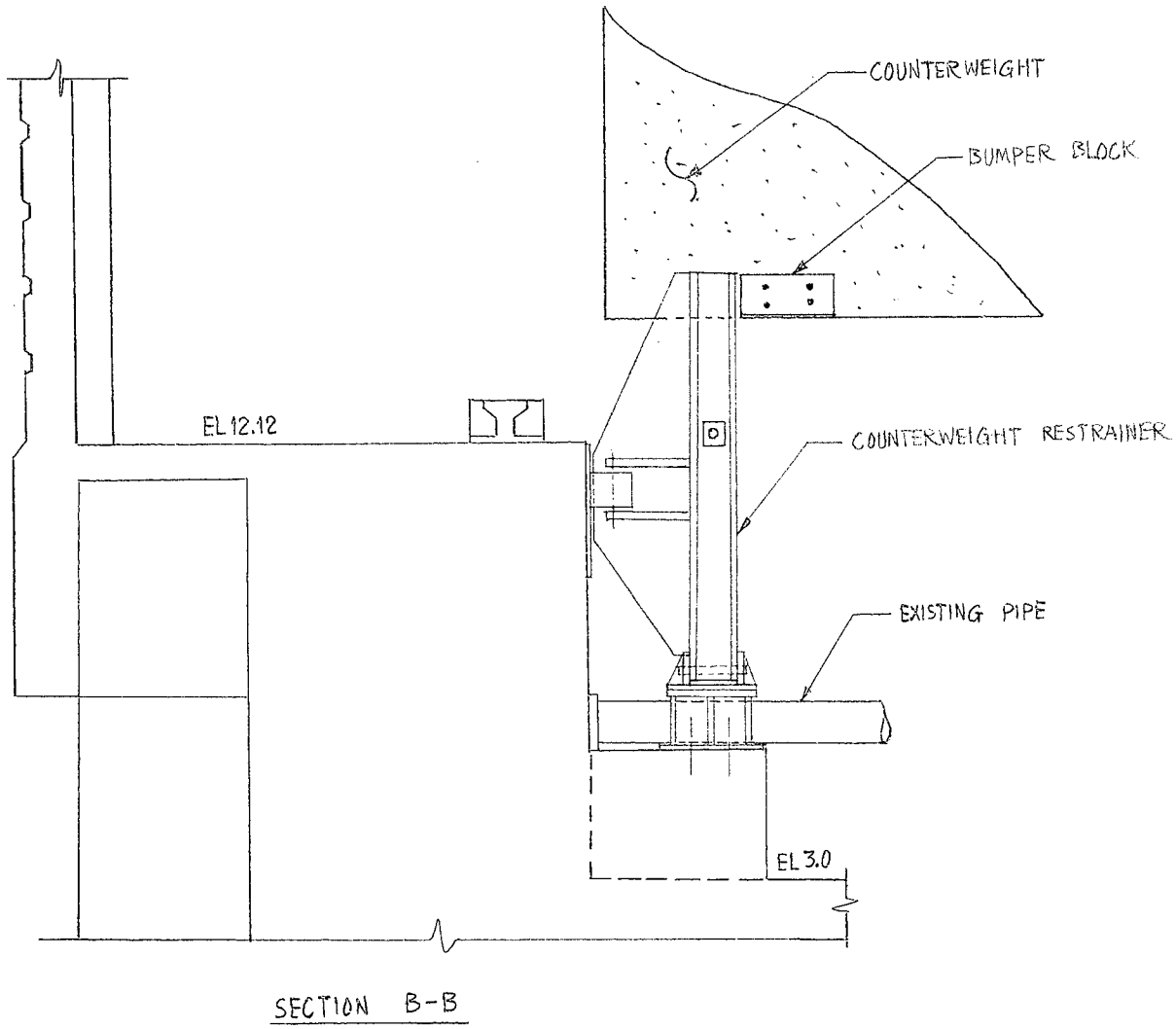


Figure 6.11c Track and Counterweight Restrainers

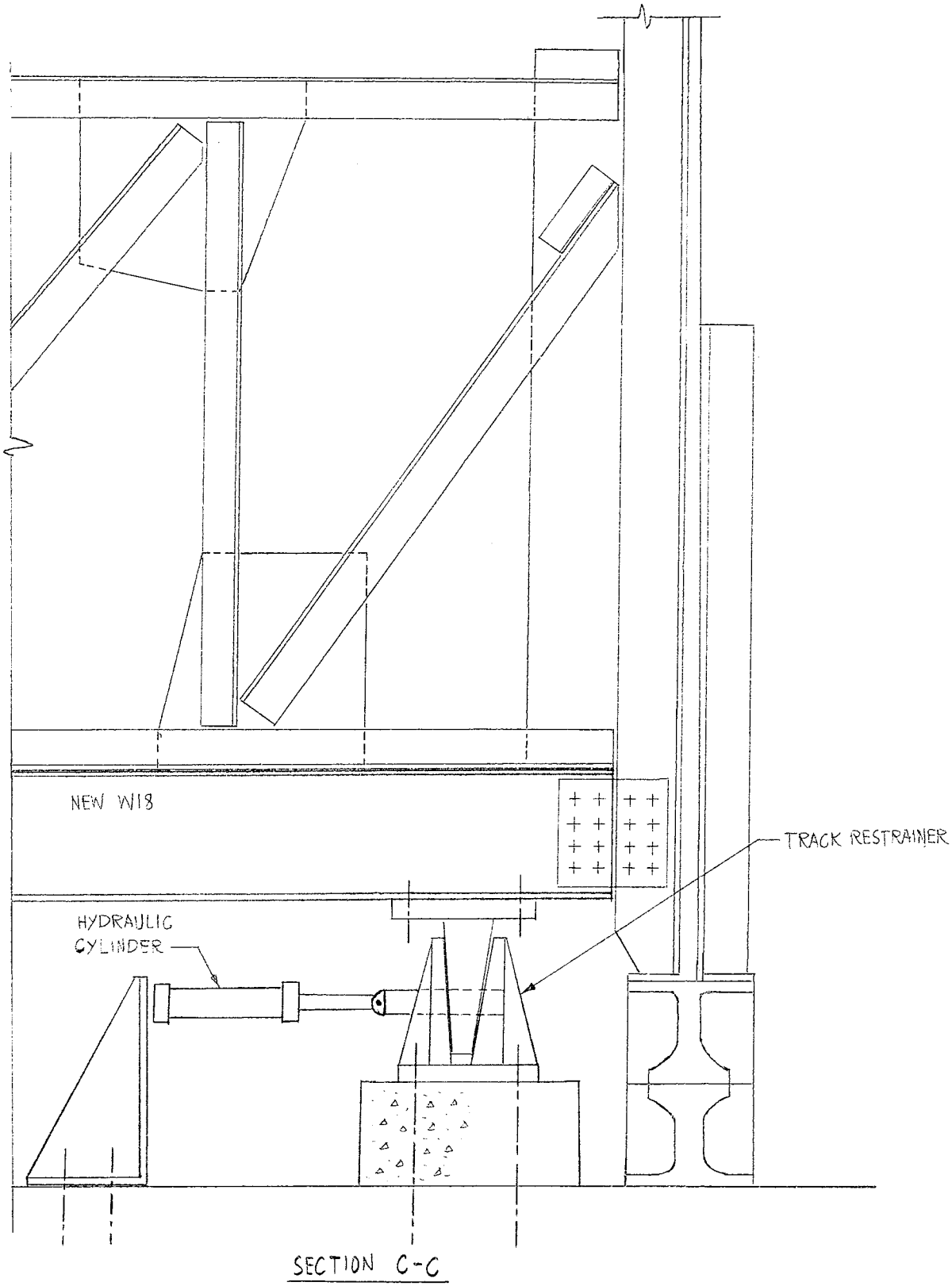


Figure 6.11d Track and Counterweight Restrainers

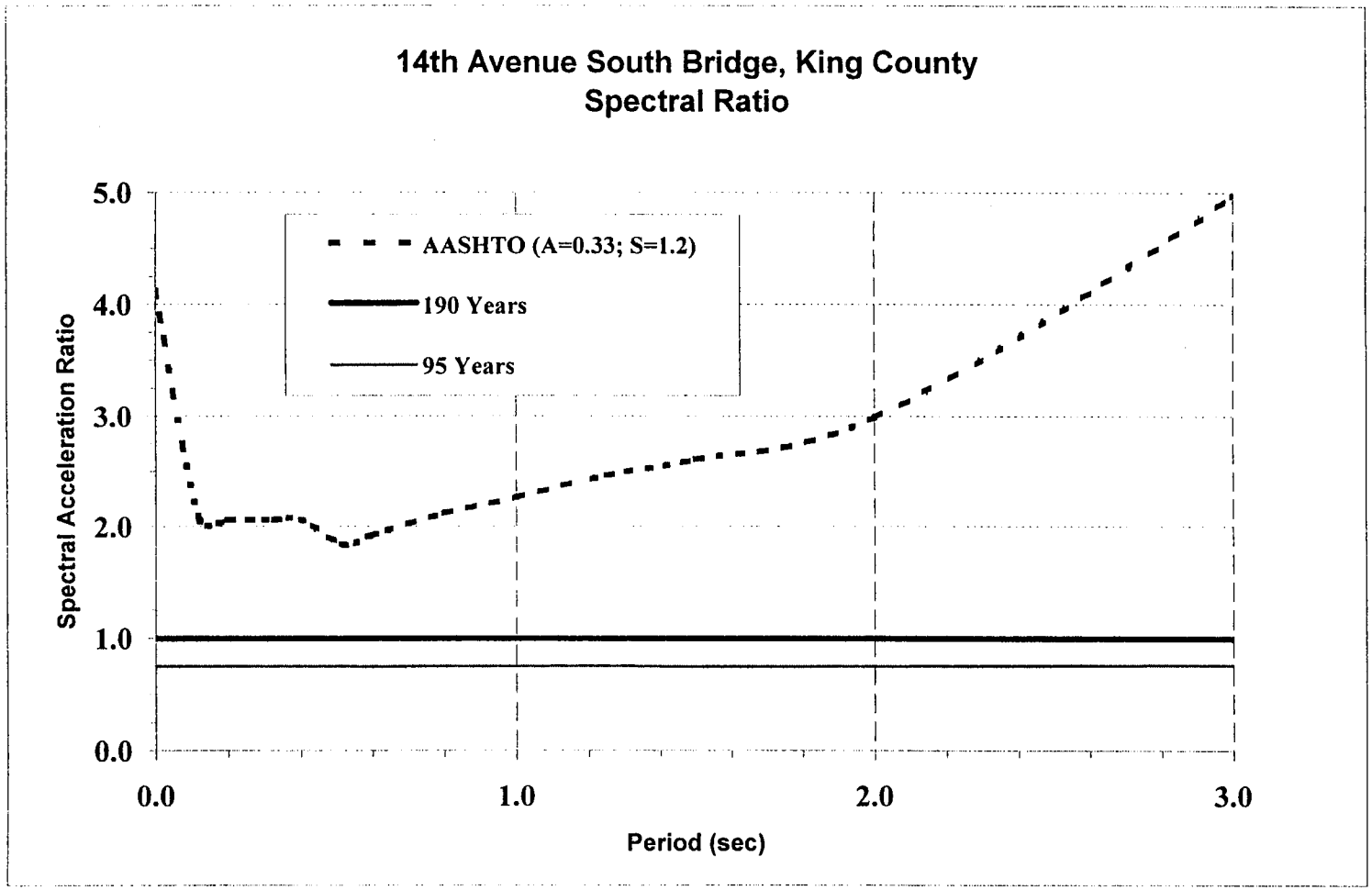


Figure 6.12 Spectral Acceleration Ratio of Differenct Levels of Earthquakes

7. RETROFIT CONSTRUCTION COST

Retrofit costs for this report only cover costs for the structural portion of the project. Estimates have not been included for engineering designs, permitting, potential environmental mitigation, related electrical and mechanical work, and inspection expenses.

Two levels of seismic retrofit are considered in this study. Table 7.1a to 7.1c list the item and total cost. The item description in the table is mainly for Level I seismic retrofit. As discussed in Chapter 6, the acceleration spectral ratio for the 475 year earthquake and the 190 year earthquake is approximately 2 for the period range of interest, based on spectral curves with 5% damping ratio. The real damping ratio is probably higher for the 475 year earthquake than for the 190 year earthquake. So the ratio of seismic forces for these two levels of earthquakes is smaller than that shown in Figure 6.12. In addition, the increase of labor cost is likely to be less than the increase of material quantity, for the same item of retrofit. Therefore, for items used for both levels of seismic retrofit, the item cost for Level II seismic retrofit is estimated to be 50% higher than the cost for Level I seismic retrofit.

As shown in Table7.1a to Table 7.1c, the cost of seismic retrofit for bascule span superstructure (Item A) is approximately 5% of the total cost if the track and counterweight restrainers are excluded. The restrainers (Item F) account for about 15% of the total cost. The cost of seismic retrofit for bascule span substructure is approximately 24% of the total cost. The cost of seismic retrofit for truss approaches is about 10% of total cost for the superstructure and 27% of the total cost for the substructure, respectively. The cost of seismic retrofit for the concrete approaches is 19% of total cost

for Level I retrofit and 29% of total cost for Level II retrofit, respectively. The higher cost percentage of concrete approach for Level II retrofit is due to the additional cost needed for soil improvement to prevent soil lateral spreading next to the river channel.

The total cost of seismic retrofit, of the structural portion of work, is approximately $1,688,000 for Level I retrofit and $2,876,000 for Level II retrofit, respectively. This cost includes 10% mobilization and 30% contingency. The cost of Level II retrofit is 70% higher than the cost of Level I retrofit.

The most recent cost for a replacement structure is $70.9 million. This cost includes engineering design, permitting, potential environmental mitigation, related electrical and mechanical work, and inspection expenses.

Table 7.1a Seismic Retrofit Cost Estimate

Item #	Item Description	Unit	Quantity	Unit Cost	Cost	Item Total		Item Percentage	
						Level I EQ	Level II EQ	Level I EQ	Level II EQ
A	Bascule Span Superstructure				$63,916	$63,916	$95,873	5.5%	4.7%
A1	Add Angles to Top Laterals between Counterweight & Frame 0				$4,200				
	8- L3^{1}/2x3^{1}/2x1/2, 11' long	lb	977	3.5	$3,420				
	Connections	lb	195	4	$780				
A2	Frame 0 New Members				$59,716				
	16- 2L5x3^{1}/2x3/4, 10' long	lb	6336	3.5	$22,176				
	2-w18x143, 36' long	lb	10296	3	$30,888				
	Connections	lb	1663	4	$6,652				
B	Bascule Span Substructure				$281,176	$281,176	$421,763	24.1%	20.5%
B1	Tiedowns for Bascule Retrofit				$117,893				
	8- 1^{3}/8" dia. HS bars	lb	1344	3.5	$4,704				
	Drill 2^{1}/2" dia. Holes	ft	400	35	$14,000				
	24- 1^{3}/4" dia. HS bars	lb	10535	3.5	$36,873				
	Drill 3" dia. Holes	ft	1200	45	$54,000				
	HS bar anchorages	lb	2376	3.5	$8,316				
B2	Concrete Walls & HS Bars				$91,115				
	4 ksi concrete for 12" concrete wall	cy	59	500	$29,500				
	Replace concrete at inlets	cy	9	800	$7,200				
	12- 1^{3}/8" dia. HS bars, 55' long	lb	3670	3.5	$12,845				
	Drill 2^{1}/2" dia. Holes	ft	582	35	$20,370				
	A706 bars & dowels	lb	17600	1	$17,600				
	HS bar anchorages	lb	720	5	$3,600				
B3	Fascia Wall Braced Frame				$72,168				
	16- WT9x25, 24' long	lb	9600	3.5	$33,600				
	32- 2L4x3x3/8, 7' long	lb	4070	3.5	$14,245				
	32- 2L4x3x3/8, 4' long	lb	2330	3.5	$8,155				
	32- L3x3x1/4, 5' long	lb	784	3.5	$2,744				
	Connections	lb	3356	4	$13,424				

Table 7.1b Seismic Retrofit Cost Estimate

Item #	Item Description	Unit	Quantity	Unit Cost	Cost	Item Total		Item Percentage	
						Level I EQ	Level II EQ	Level I EQ	Level II EQ
C	Truss Approach Superstructure				$109,910	$109,910	$212,492	9.4%	10.3%
C1	Brace Frames @ Bascules				$85,524				
	8- WT12x38, 33' long	lb	10032	$3.5	$35,112				
	24- 2L5x3^{1}/2x3/4, 6.5 long	lb	6178	$3.5	$21,623				
	24- 2L5x3^{1}/2x3/4, 3.5 long	lb	3326	$3.5	$11,641				
	24- L3x3x1/4, 3' long	lb	353	$3.5	$1,236				
	Connections	lb	3978	$4.0	$15,912				
C2	Anchor Bolts @ Fixed Supports				$800				
	16- 1^{1}/2" dia. HS anchor bolts	each	16	$50.0	$800				
C3	Rod Restrainers				$336				
	8- 1" dia. 4' long HS rod restrainers	each	8	$42.0	$336				
C4	Transverse Shear Key				$23,250				
	4 ksi Concrete	cy	40	$500.0	$20,000				
	A706 reinforcement	lb	1760	$0.9	$1,584				
	6- W12x22, 3' long & connections	lb	476	$3.5	$1,666				
C5*	Longitudinal Shear Key				$47,628				
	32- 6'x1.5'x1/2 plates	lb	5880	$3.5	$20,580				
	64- 6'x6"x3/4 plates	lb	5880	$3.0	$17,640				
	Connections	lb	2352	$4.0	$9,408				
D	Truss Approach Substructure				$319,714	$319,714	$479,570	27.5%	23.3%
D1	Pier 1 Piles				$103,200				
	4- 3' dia. 120' long concrete piles with steel casings	ft	480	$215.0	$103,200				
D2	Pier 1 Caps & Spandrel Beams				$67,816				
	4 ksi concrete	cy	63	$500.0	$31,500				
	8- 1^{3}/8" dia. HS bars, 64' long	lb	2847	$3.5	$9,965				
	Drill 2^{1}/2" dia. Holes	ft	144	$35.0	$5,040				
	16- 1" dia. HS bars, 7' long	lb	623	$3.7	$2,305				
	Drill 2" dia. Holes	ft	128	$35.0	$4,480				
	A706 reinforcement	lb	16140	$0.9	$14,526				
D3	Pier 2 Piles				$103,200				
	4- 3' dia. 120' long concrete piles with steel casings	ft	480	$215.0	$103,200				
D4	Pier 2 Cap Beams				$45,498				
	4 ksi concrete	cy	38	$500.0	$19,000				
	8- 1^{3}/8" dia. HS bars, 64' long	lb	2847	$3.5	$9,965				
	Drill 2^{1}/2" dia. Holes	ft	56	$35.0	$1,960				
	16- 1" dia. HS bars, 7' long	lb	623	$3.7	$2,305				
	Drill 2" dia. Holes	ft	112	$35.0	$3,920				
	A706 reinforcement	lb	9276	$0.9	$8,348				

Table 7.1c Seismic Retrofit Cost Estimate

Item #	Item Description	Unit	Quantity	Unit Cost	Cost	Item Total		Item Percentage	
						Level I EQ	Level II EQ	Level I EQ	Level II EQ
E	Concrete Approaches				$217,700	$217,700	$586,554	18.7%	28.6%
E1	X Braces				$148,957				
	68- 2 1/2' dia. 28' long rod braces	lb	31797	$2.5	$79,493				
	136 clevises	lb	3536	$8.0	$28,288				
	68 turnbuckles	lb	1574	$10.0	$15,740				
	Connections	lb	6359	$4.0	$25,436				
E2	Rod Restrainers				$3,200				
	16- 1" dia. 20' long rod restrainers	each	16	$200.0	$3,200				
E3	Grade Beams				$65,543				
	34- 20"x12", 20' long grade beams								
	4 ksi concrete	cy	126	$500.0	$63,000				
	Reinforcements	lb	2826	$0.9	$2,543				
E4*	Compaction or Jet Grouting				$260,004				
	2- 30'x65'x50' near pier 2 of truss approaches	cy	21667	$12	$260,004				
F	Restrainers at Tracks & Counter Weights				$172,000	$172,000	$258,000.0	14.8%	12.6%
F1	Track Restrainers	each	4	$18,000.0	$72,000				
F2	Counterweight Restrainers	each	4	$15,000.0	$60,000				
F3	Hydraulic Power Units	each	4	$5,000.0	$20,000				
F4	Electrical Power & Control	each	2	$10,000.0	$20,000				
All Items	Subtotal without Contingency					$1,164,414	$2,054,253	100.0%	100.0%
	Mobilization & Contingency					$523,986	$821,701		
	Total					**$1,688,400**	**$2,875,954**		

NOTES: Items C5 and E4 are for Level II retrofit only. All other item descriptions are for Level I retrofit.
Item cost for Level II retrofit are assumed to twice of item cost for Level I retrofit.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions Based on the Analytical Model

The existing bridge was constructed from 1929 to 1931 with no consideration of seismic design. Table 8.1 lists the damaged state of the existing bridge components subjected to three different levels of earthquakes based on the response spectrum method of analysis.

Under Level 1 (95 year) earthquake, the following components will be severely damaged:

- Frame 0 at the support of the bascule span in closed position, due to lack of lateral load path,
- The fascia wall opposite to the operator house, due to lack of bracing at the top,
- Vertical members of the truss approach near the bascule, due to lack of lateral load path,
- Columns of the concrete approach, due to insufficient capacity.
- In addition, the vertical movement of the counterweight will result in pounding of the counterweight to the truss approach anchorage girder. Also, the rolling span may slip off the track. Other damage will occur in other components of the bridge as shown in the table.

Under Level 2 (190 year) earthquake, the following components may collapse or cause collapse:

- Frame 0 at the support of the bascule span in closed position,
- The fascia wall opposite to the operator house,
- Vertical members of the truss approach near the bascule,
- Columns of the concrete approach,
- The footing of Pier 1, due to overturning,
- The footing of Pier 2, due to overturning.

Severe damage will occur in other components. In addition, restrainers will be needed to restraint the counterweight and the track girder.

Under Level 3 (475 year) earthquake, the following components will collapse or cause collapse:

- Frame 0 at the support of the bascule span in closed position,
- The fascia wall opposite to the operator house,
- Vertical members of the truss approach near the bascule,
- Columns of the concrete approach,
- The footing of Pier 1,
- The footing of Pier 2,
- Columns of Pier 1, due to insufficient capacity,
- Columns of Pier 2, due to insufficient capacity.

Severe damage will occur in other components without seismic retrofit.

Based on the analysis procedure, it is concluded that without seismic retrofit, the existing bridge will be severely damaged during the 95-year earthquake, and it may collapse when subjected to the 190 year earthquake or higher intensity earthquakes.

After implementation of seismic retrofit strategy I, the damage of the bridge under three different levels of earthquakes is:

- For Level 1 earthquake, the bridge will be subjected to repairable damage,
- For Level 2 earthquake, the bridge will be subjected to major damage, but there is no collapse,
- For Level 3 earthquake, the bridge will be subjected to major damage and may collapse.

After implementation of seismic retrofit strategy II, the damage of the bridge under three different levels of earthquakes is:

- Under Level 1 earthquake, the bridge will be subjected to repairable damage,
- Under Level 2 earthquake, the bridge will be subjected to major damage, but the damage may be repairable,
- Under Level 3 earthquake, the bridge will be subjected to major damage, but there is no collapse.

8.2 Correlation with the Nisqually Earthquake Data

The Nisqually earthquake, which occurred on February 28, 2001, had a magnitude of 6.8 with an epicenter 30 miles southwest of Seattle. Initially our geotechnical engineer had estimated this earthquake was comparable to the 95-year earthquake based on the ground motions between 0.10 to 0.15g. This was reported during our presentation to King County on May 17, 2001. However, when the processed ground motion records became available later, it appears that ground accelerations of about 0.18 to 0.20g would be appropriate for the general conditions in the area. Therefore, the Nisqually earthquake would be equivalent to the 190-year earthquake.

The damage recorded after the Nisqually earthquake was less than predicted by the analytical procedures and model used in this study. Some reasons for these differences are noted below:

1. 5% damping was used in the development of the response spectrum curves. The actual structure has been designated "obsolete" with cracking and deterioration of the concrete elements. The inherent damping in the structure may be as high as 10%, which would lower the force level that the structure would actually see during an earthquake.
2. For higher periods of vibration the Nisqually earthquake acceleration levels are much less than the 95 and 190-year earthquake. See Figure 15 in the geotechnical report (Appendix C). The concrete approaches have fundamental modes in the 2 second-period range. Therefore, the damage would be expected to be lower from the Nisqually earthquake than predicted by the analysis.

3. Another consideration is if the ground motion shaking had been longer there would have been much more damage to the structure than occurred during the Nisqually earthquake.

8.3 Recommendations

It is recommended that a seismic retrofit program of the South Park Bridge be considered. Either strategy I or II of seismic retrofit can be adopted depending on the funding available. A review of the Nisqually earthquake data should be considered in this evaluation process. After the retrofit, the bridge will be able to provide an additional 10 or 20 years of service life with seismic risk similar to the risk of newly designed bridges. A replacement program is also a reasonable option if funding is available.

Table 8.1 Damage State of Bridge Subjected to Different Levels of Earthquakes

Item	Subitem	95 Years Earthquakes	190 Years Earthquakes	475 Years Earthquakes
Bascule Span Superstructure	Truss Members	o.k.	o.k.	o.k.
	Vertical Frames	Frame 0 severely damaged	Frame 0 may collapse	Frame 0 collapses
	Top Laterals	Some top laterals damaged	Some top laterals severely damaged	Some top laterals severely damaged
	Bottom Laterals	o.k.	o.k.	o.k.
	Counterweight	Needs restrainers	Needs restrainers	Needs restrainers
	Tracks	Needs restrainers	Needs restrainers	Needs restrainers
	Fascia Walls	Severely damaged	May collapse	Collapses
	Operator's House	o.k.	Damaged	Severely damaged
Bascule Span Substructure	Pile Foundations	o.k.	o.k.	Damaged
	Footings	o.k.	Damaged	Severely damaged
	East & West Pier Walls	Damaged	Severely damaged	May collapse
	North & South Pier Walls	o.k.	Damaged	Severely damaged
Truss Approach Superstructure	Truss Members	o.k.	o.k.	o.k.
	Vertical Frames	Severely damaged near bascule	May collapse near bascule	Collapses near bascule
	Top Laterals	o.k.	o.k.	o.k.
	Bottom Laterals	Not continuous	Not continuous	Not continuous
	Supports	Damaged	Severely damaged	May cause collapse
Truss Approach Substructure	Pier 1 Pile Foundation	Damaged	Severely damaged	Severely damaged
	Pier 1 Footing	Damaged	May be unstable and cause collapse	Unstable and causes collapse
	Pier 1 Columns	Damaged	Severely damaged	Collapses
	Pier 2 Pile Foundation	Damaged	Severely damaged	Severely damaged
	Pier 2 Footing	Damaged	May be unstable and cause collapsed	Unstable and causes collapse
	Pier 2 Columns	Damaged	Severely damaged	Collapses
Concrete Approach Superstructure	Cap Beams	o.k.	o.k.	o.k.
	Columns	Severely damaged	May collapse	Collapses
Concrete Approach Substructure	Pile Foundations	Damaged	Severely damaged	Severely damaged
	Footings	Damaged	Severely damaged	Severely damaged

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22. ASSHTO 1998 LRFD Specifications

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24. Seismic Retrofitting Manual for Highway Bridges, FHWA, May 1995 (FHWA-RD-94-052)

Appendix A
Dead Load Calculation of Bascule Pier

Dead load calculation of the bascule pier is carried out in the following steps:

1. The bascule is divided into different elements.
2. The plan areas of the elements are calculated.
3. The weights of the elements are calculated based on the areas and the elevations.
4. Buoyancies of the elements are calculated.
5. The total dead load, total buoyancy and total effective dead load are obtained.

The calculation is carried out by hand in the following sheets.

Tables A.1 to A.6 show the calculation for the center of gravity of half bascule span under different loading condition. Dead load and live load of the superstructure are obtained from the as-built drawings. These results will be used for the calculation of the maximum and minimum pile axial loads in Appendix 2.

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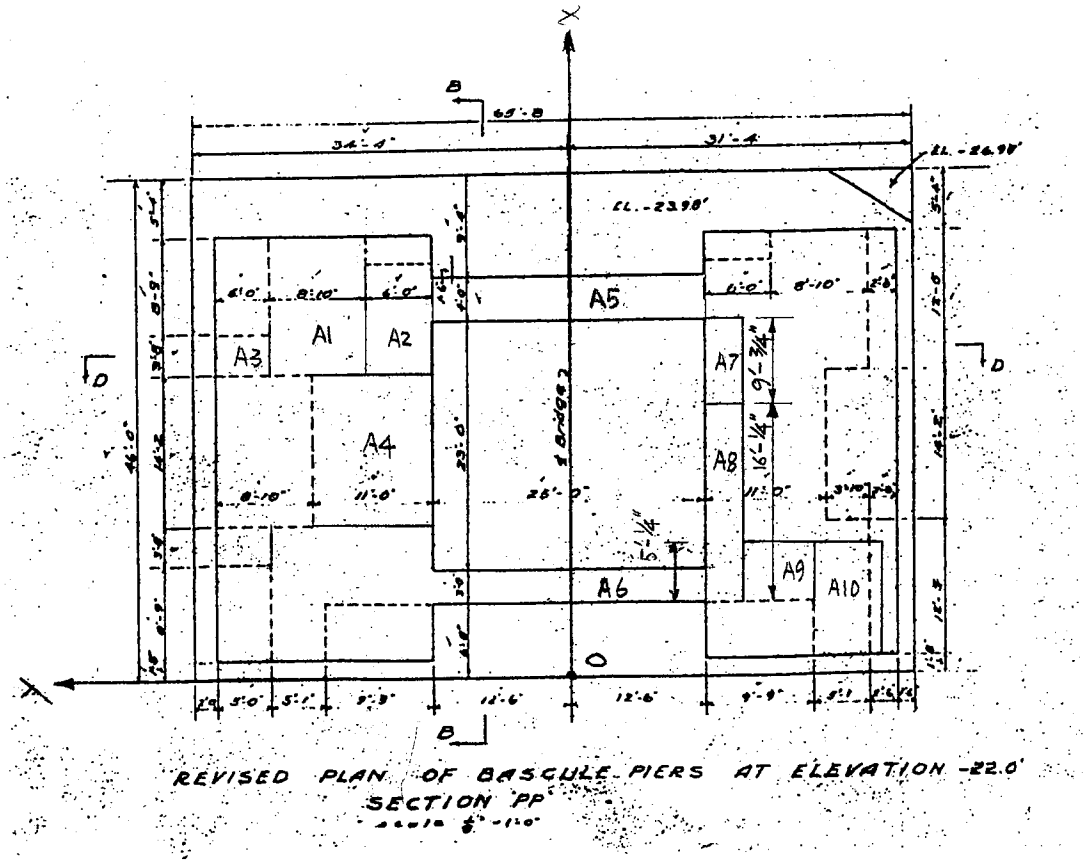
Area Calculation for DL Elements:



$A1 = 12.42 \times 8.83 = 109.7\ ft^2$

$A2 = 9.92 \times 6 = 59.52\ ft^2$

$A3 = 3.67 \times 5 = 18.35\ ft^2$

$A_4 = 14.17 \times 11 = 155.87\ ft^2$

$A5 = 4 \times 25 = 100\ ft^2$

$A6 = 3 \times 25 = 75\ ft^2$

$A7 = 9.75 \times 3.75 = 36.56\ ft^2$

$A8 = 16.25 \times 3.75 = 60.94\ ft^2$

$A9 = 5.25 \times 6 = 31.5\ ft^2$

$A10 = 10.25 \times 6.25 = 64.06\ ft^2$

Job Title: 14th Avenue South Bridge		Job No. W118Z	Sheet
Subject: Weight of Bascule Pier	Designed By: PY	Date	of
	Checked By	Date	Total Sheet
			of

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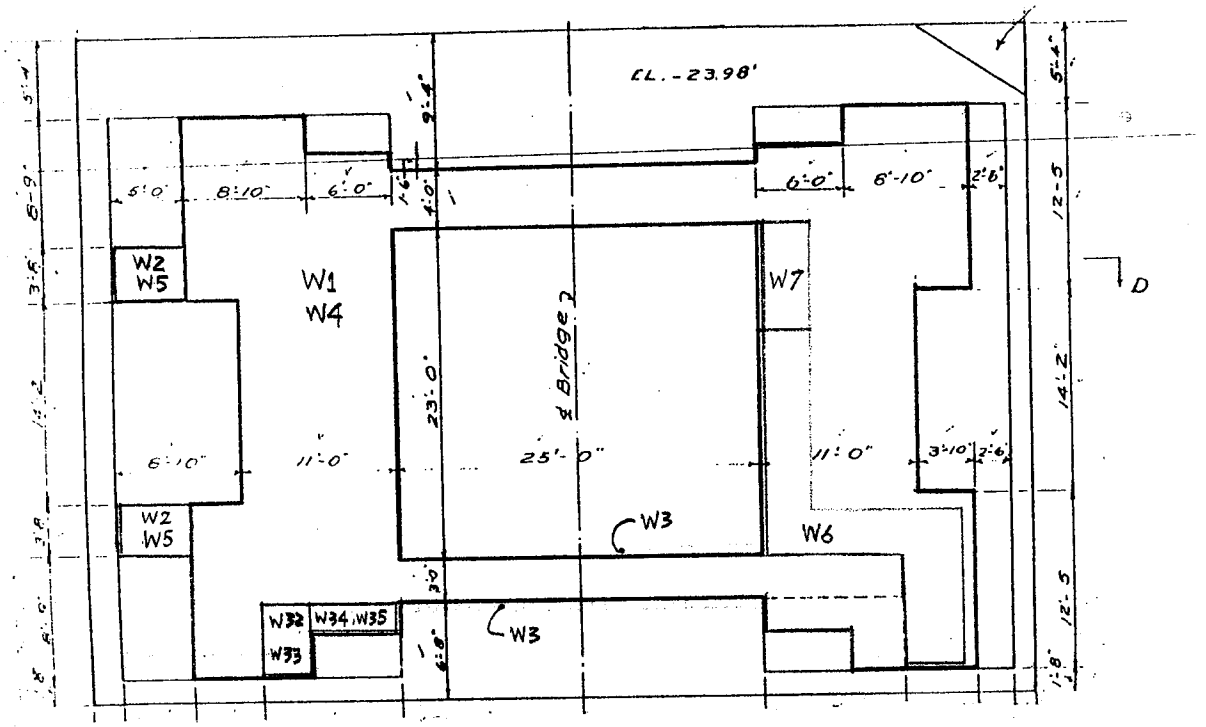
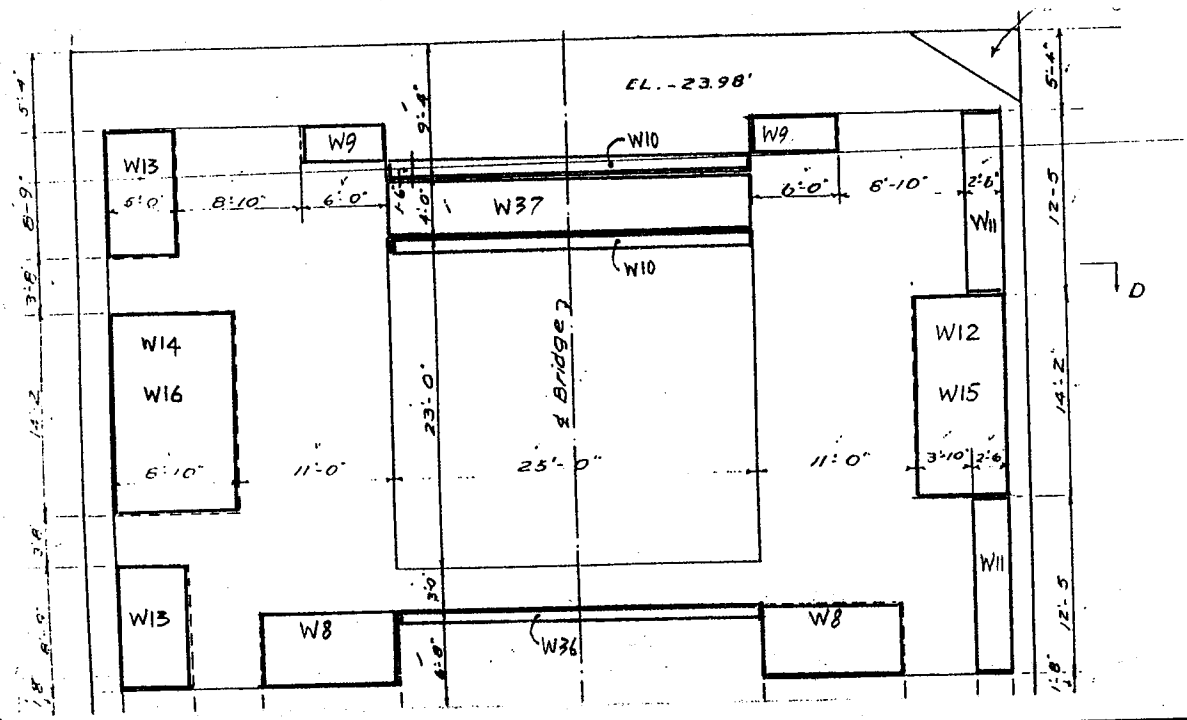
Locations of DL Elements:





Job Title 14th Avenue South Bridge		Job No. W1182	Sheet
Subject Weight of Bascule Pier	Designed By PY	Date	of
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From EL. -23.98' ~ EL. 3':

$$W_1 = [4\times(A_1 + A_2) + 2A_4 + A_5 + A_6](3 + 23.98)\times 0.145$$

$$= [4\times(109.7 + 59.52) + 2\times 155.87 + 100 + 75](3 + 23.98)\times 0.145$$

$$= 4552.^K$$

$$W_2 = 2A_3(3 + 23.98)\times 0.145$$

$$= 2\times 18.35\times(3 + 23.98)\times 0.145$$

$$= 144^K$$

$$W_3 = (1+1)\times 25\times 1.5\times 0.145$$ at top of 3' wall

$$= 11^K$$

From EL. 3' ~ EL. 12.12':

$$W_4 = [4\times(109.7 + 59.52) + 2\times 155.87 + 100](12.12 - 3)\times 0.145$$

$$= 1440^K$$

$$W_5 = 2\times 18.35\times(12.12 - 3)\times 0.145$$ A_3

$$= 49^K$$

$$W_6 = -2(A_8 + A_9 + A_{10})(12.12 - 3)\times 0.145$$

$$= -2(60.94 + 31.5 + 64.06)(12.12 - 3)\times 0.145$$

$$= -414^K$$

$$W_7 = -2A_7(12.12 - 5.75)\times 0.145$$

$$= -2\times 36.56(12.12 - 5.75)\times 0.145$$

$$= -68^K$$

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Subject Weight of Bascule Pier	Designed By PY	Date	of
	Checked By	Date	Total Sheet
			of

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From EL. -23.98' ~ 12.12', additional weight:

$W_8 = 2 \times 0.5 \times (9.75 \times 5 + 9.75 \times 1.0)(3 + 23.98) \times 0.145$

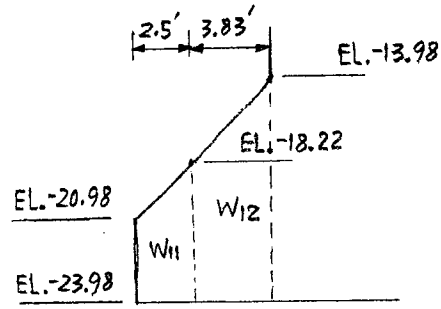
$= 229^K$

$W_9 = 2 \times 0.5 \times (6 \times 2.5 + 6 \times 0)(12.12 + 23.98) \times 0.145$

$= 79^K$

$W_{10} = (1.5 + 1.5) \times 25 \times 2.5 \times 0.145$ at top of 4' wall

$= 27^K$



$W_{11} = 39 \times 2.5 \times 0.5 \times (3 + 5.76) \times 0.145$

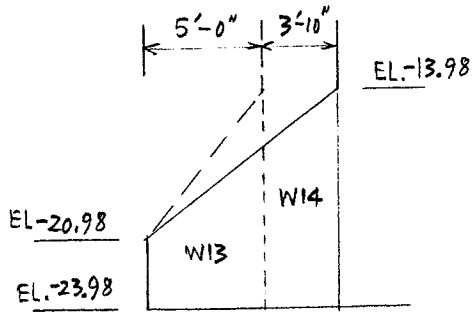
$= 62^K$

$W_{12} = 14.17 \times 3.83 \times 0.5 \times (5.76 + 10) \times 0.145$

$= 72^K$

$W_{13} = 2 \times 8.75 \times 0.5 (3 + 10) \times 5 \times 0.145$

$= 82^K$



$W_{14} = 14.17 \times 0.5 (3 + 10) \times 8.83 \times 0.145$

$= 118^K$

$W_{15} = 14.17 \times 1.33 \times (12.12 - 6.62) \times 0.15$

$+ 14.17 \times 3.83 \times 0.67 \times 0.15$

$= 20^K$ at the top of pier

$W_{16} = 2 \times 6.17 \times 3.67 \times (12.12 - 6.62) \times 0.15$ operator' house

$+ 14.17 \times 1.33 \times (12.12 - 6.62) \times 0.15$ basement level 2 Floor

$+ 2 \times 2.33 \times 1.0 \times 14.17 \times 0.15$

$+ 15 \times 14.17 \times 0.67 \times 0.15$

$= 83^K$

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$W17 = (20.5 + 11.17) \times 2 \times 1.0 \times (25.05 - 12.12) \times 0.15 \times 0.9$ operator's house basement level 2, concrete wall

$= 110^K$

$W18 = 19.5 \times 1.5 \times 1.0 \times 0.15 + 19.5 \times 6.5 \times 0.67 \times 0.15$ operator's house basement level 1 floor

$= 17^K$

$W19 = (20.5 + 11.17) \times 2 \times 1.0 \times (40.7 - 25.05) \times 0.15 \times 0.9$ Basement level 1, conc. wall

$= 134^K$

$W20 \doteq W18 = 17^K$ operator's house, operator's floor

$W21 = (20.5 + 11.17) \times 2 \times 1.0 \times 8.25 \times 0.14 \times 0.9$ operator's floor, brick wall

$= 65^K$

$W22 = 21.5 \times 12.17 \times 0.06 = 16^K$ operator's house, roof

operator's house total weight

$DL_{op} = \sum W16:22$

$= 83 + 110 + 17 + 134 + 17 + 65 + 16$

$= 442^K$

Job Title 14th Avenue South Bridge		Job No. W1182	Sheet
Subject Weight of Bascule Pier	Designed By PY	Date	of
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Bascule Fascia Wall & Concrete sidewalks

$W23 = 2 \times 5.75 \times 2.33 \times 22 \times 0.15$
$= 88^K$

$W24 = 2 \times 6.75 \times 1.33 \times 22 \times 0.15$
$= 59^K$

$W25 = 2 \times 7.5 \times 1.33 \times 22 \times 0.15$
$= 66^K$

$W26 = 2 \times 8.5 \times 1.33 \times 22 \times 0.15$
$= 75^K$

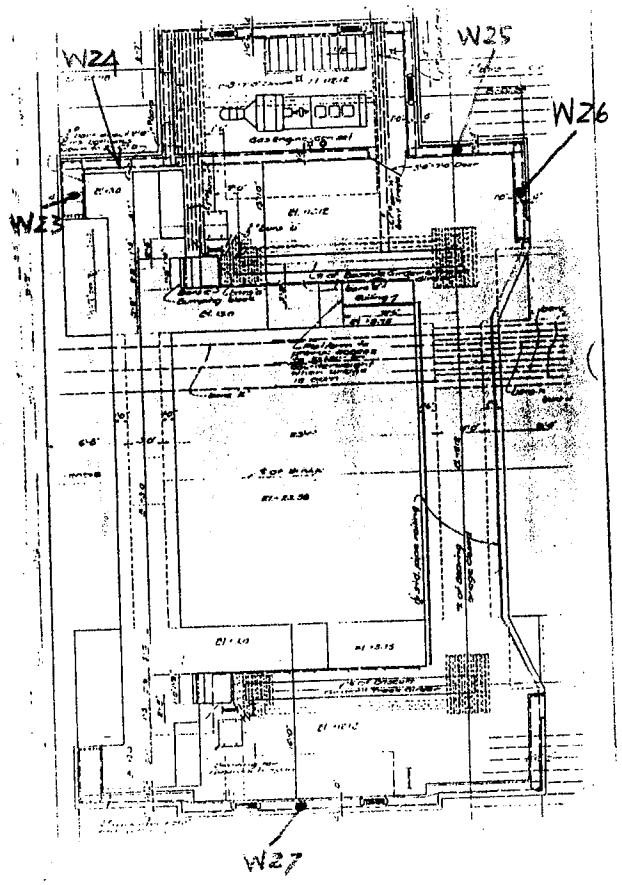
$W27 = 22.25 \times 1.33 \times 22 \times 0.15$
$= 98^K$

$W28 = 2 \times 25 \times 8 \times 0.375 \times 0.15$ sidewalk
$+ 2 \times 25 \times 1.17 \times 1.17 \times 0.15$
$= 33^K$

$W29 = 39 \times 0.67 \times 2.75 \times 0.145 \times 0.8$ hand rail
$= 8^K$

$W30 = 29 \times 0.67 \times 2.75 \times 0.145 \times 0.8$ handrail
$= 6^K$

$W31 = 20.75 \times 3.5 \times 1.0 \times 0.15$ entrance
$+ 9 \times 12 \times 0.5 \times 0.15$
$= 19^K$



Bascule Fascia wall & Concrete sidewalk:

$DL_f = \Sigma W23:31 = 452^K$

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	Checked By	Date	Total Sheet
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Unsymmetrical Corrections:

$W32 = -5 \times 3.75 \times (3 + 23.98) \times 0.145 \times 2 = -146.8^{K}$

$W33 = -5 \times 3.75 \times (12.12 - 3) \times 0.145 \times 2 = -49.6^{K}$

$W34 = -2.5 \times 6 \times (3 + 23.98) \times 0.145 \times 2 = -117.4^{K}$

$W35 = -2.5 \times 6 \times (12.12 - 3) \times 0.145 \times 2 = -39.6^{K}$

Total bascule pier DL:

$$DL_{pier} = \Sigma(W1 \sim W15) + \Sigma(W32 \sim W35)$$

$$= 4552 + 144 + 11 + 1440 + 49 - 414 - 68 + 229 + 79 + 27 + 62 + 72 + 82 + 118 + 20 - 146.8 - 49.6 - 117.4 - 39.6$$

$$= 6049.6^{K}$$

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Footing & Seal Weight:

$$DL_f = 46 \times 65.67 \times 6 \times 0.145 - 7.5 \times 5 \times 3 \times 0.5 \times 0.145 - 315 \times \frac{\pi}{4} \times 1^2 \times 6 \times (0.145 - 0.032)$$
$$= 2628 - 8 - 168 = 2452^K$$

$$DL_s = 46 \times 65.67 \times 15 \times 0.145 - 315 \times \frac{\pi}{4} \times 1^2 \times 15 \times (0.145 - 0.032)$$
$$= 6570 - 419 = 6151$$

Buoyant Force for MLLW = EL. -12.98':

M.L.L.W. EL = -12.98', Ext. H.W. EL. = 5.02' 0 tide = King County Datum 0.0

Seal: $F_1 = 46 \times 65.67 \times 15 \times 0.0625 = 2832^K$

footing: $F_2 = (46 \times 65.67 \times 6 - 7.5 \times 5 \times 3 \times 0.5) \times 0.0625 = 1129^K$

Pier:

$F_3 = [4 \times (109.7 + 59.52) + 2 \times 155.87 + 100 + 75](23.98 - 12.98) \times 0.0625 = 800^K$ W1

$F_4 = 2 \times 18.35 \times (23.98 - 12.98) \times 0.0625 = 25^K$ W2

$F_5 = 2 \times 0.5 \times (9.75 \times 5 + 9.75 \times 3.4)(23.98 - 12.98) \times 0.0625 = 56^K$ W8

$F_6 = 2 \times 0.5 \times (6 \times 2.5 + 6 \times 1.8)(23.98 - 12.98) \times 0.0625 = 17.7^K$ W9

$F_7 = 62 \times \frac{0.0625}{0.145} = 27^K$ W11

$F_8 = 72 \times \frac{0.0625}{0.145} = 31^K$ W12

$F_9 = 82 \times \frac{0.0625}{0.145} = 35^K$ W13

$F_{10} = 118 \times \frac{0.0625}{0.145} = 51^K$ W14

$F_{11} = -5 \times 3.75 \times (23.98 - 12.98) \times 0.0625 \times 2 = -25.8^K$ W32

Job Title 14th Avenue South Bridge		Job No. W1182	Sheet
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Buoyant Force for MLLW (cont'd):

$F_{12} = -2.5 \times 6 \times (23.98 - 12.98) \times 0.0625 \times 2 = -20.6^K$ W34

Total Buoyancy of the pier $= \Sigma(F3 \sim F12) = 996.3^K$ for MLLW

Total Buoyancy including footing & Seal $= \Sigma(F1 \sim F12) = 4957.3^K$ for MLLW

Additional Buoyant Forces for MHHW = EL. -1.20':

$F_{13} = [4 \times (109.7 + 59.52) + 2 \times 155.87 + 100 + 75](12.98 - 1.2) \times 0.0625 = 857^K$ W1

$F_{14} = 2 \times 18.35 \times (12.98 - 1.2) \times 0.0625 = 27^K$ W2

$F_{15} = 2 \times 0.5 \times (3.4 + 1.7) \times 9.75 \times (12.98 - 1.2) \times 0.0625 = 36.6^K$ W8

$F_{16} = 2 \times 0.5 \times (1.8 + 1.0) \times 6 \times (12.98 - 1.2) \times 0.0625 = 12.4^K$ W9

$F_{17} = -5 \times 3.75 \times (12.98 - 1.2) \times 0.0625 \times 2 = -27.6^K$ W32

$F_{18} = -2.5 \times 6 \times (12.98 - 1.2) \times 0.0625 \times 2 = -11.0^K$ W34

Additional buoyant force $= 894.4^K$

Total Buoyant Force $= 996.3 + 894.4 = 1890.7^K$ for MHHW

Total Buoyancy including footing & Seal $= \Sigma(F1 \sim F18) = 5851.7^K$ for MHHW

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Total Effective DL for MLLW:

Rolling Load	832.6 X 2 =	1665
Approach Truss	180 X 2 =	360
operator's house		442
Fascia & Sidewalk		452
Bascule pier		6050
Footing		2452
Seal		6151
Total DL		17572
Total Buoyancy		- 4957.3
Net DL		12614^K

Total Effective DL for MHHW:

Total DL	17572^K
Total Buoyancy	-5851.7^K
Net DL	11720^K

Job Title 14th Avenue South Bridge		Job No. W1182	Sheet
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Added Load Elements to Base Gravity Center Calculation:

1' wall next to the 3' wall from EL. -23.98' ~ 3'

$W_{36} = 25 \times 1.0 \times (3 + 23.98) \times 0.145 = 98^K$

4' wall above the 3' wall from EL. 3' ~ 12.12'

$W_{37} = 25 \times 4 \times (12.12 - 3) \times 0.145 = 132^K$

Corresponding Buoyancy:

$F_{20} = 25 \times 1.0 \times (23.98 - 12.98) \times 0.0625 = 17^K$ for MLLW

Additional

$F_{21} = 25 \times 1.0 \times (12.98 - 1.2) \times 0.0625 = 18^K$ for MHHW

Job Title 14th Avenue South Bridge		Job No. W1182	Sheet
Subject weight of Bascule Pier	Designed By PY	Date	of
	Checked By	Date	Total Sheet
			of

Table A.1

DL & Center of Gravity of Half Bascule Span

(effective DL for MLLW under closed position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	1665	34.67	0	12.12	57726	0	
DL at Anchor Col.	360	14.67	0	12.12	5281	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	3833	21.17	0	-8.8	81145	0	-33730
w2	119	21.17	29.83	-8.8	2519	3550	-1047
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	173	4.67	0	-7.7	808	0	-1332
w9	61	39	0	-7.7	2379	0	-470
w10	27	34.67	0	12.12	936	0	327
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-121	4.17	0	-3.9	-505	0	472
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-96.8	5.42	0	-3.9	-525	0	378
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-81	6.17	0	-10.5	-500	0	851
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	12614				306113	24966	-167577
center of Gravity		x0	y0	z0			
		24.3	2.0	-15.1			

Table A.2

DL & Center of Gravity of Half Bascule Span

(effective DL for MHHW under closed position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	1665	34.67	0	12.12	57726	0	
DL at Anchor Col.	360	14.67	0	12.12	5281	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	2958	21.17	0	-8.8	62621	0	-26030
w2	92	21.17	29.83	-8.8	1948	2744	-810
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	136.4	4.67	0	-7.7	637	0	-1050
w9	48.6	39	0	-7.7	1895	0	-374
w10	27	34.67	0	12.12	936	0	327
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-93.4	4.17	0	-3.9	-389	0	364
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-85.8	5.42	0	-3.9	-465	0	335
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-63	6.17	0	-10.5	-389	0	662
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	11719.6				286649	24160	-159601
center of Gravity		x0	y0	z0			
		22.7	1.9	-15.7			

Table A.3

DL+LL & Center of Gravity of Half Bascule Span

(effective DL for MLLW under closed position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	3089.2	34.67	0	12.12	107103	0	
DL at Anchor Col.	-326	14.67	0	12.12	-4782	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	3833	21.17	0	-8.8	81145	0	-33730
w2	119	21.17	29.83	-8.8	2519	3550	-1047
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	173	4.67	0	-7.7	808	0	-1332
w9	61	39	0	-7.7	2379	0	-470
w10	27	34.67	0	8	936	0	216
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-121	4.17	0	-3.9	-505	0	472
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-96.8	5.42	0	-3.9	-525	0	378
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-81	6.17	0	-10.5	-500	0	851
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	13352.2				345426	24966	-167688
center of Gravity		x0	y0	z0			
		25.9	1.9	-15.1			

Table A.4

DL+LL & Center of Gravity of Half Bascule Span

(effective DL for MHHW under closed position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	3089.2	34.67	0	12.12	107103	0	
DL at Anchor Col.	-326	14.67	0	12.12	-4782	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	2958	21.17	0	-8.8	62621	0	-26030
w2	92	21.17	29.83	-8.8	1948	2744	-810
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	136.4	4.67	0	-7.7	637	0	-1050
w9	48.6	39	0	-7.7	1895	0	-374
w10	27	34.67	0	8	936	0	216
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-93.4	4.17	0	-3.9	-389	0	364
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-85.8	5.42	0	-3.9	-465	0	335
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-63	6.17	0	-10.5	-389	0	662
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	12457.8				325962	24160	-159713
center of Gravity		x0	y0	z0			
		24.4	1.8	-15.7			

Table A.5

DL & Center of Gravity of Half Bascule Span

(effective DL for MLLW under opened position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	1665	16.67	0	12.12	27756	0	
DL at Anchor Col.	360	14.67	0	12.12	5281	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	3833	21.17	0	-8.8	81145	0	-33730
w2	119	21.17	29.83	-8.8	2519	3550	-1047
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	173	4.67	0	-7.7	808	0	-1332
w9	61	39	0	-7.7	2379	0	-470
w10	27	34.67	0	8	936	0	216
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-121	4.17	0	-3.9	-505	0	472
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-96.8	5.42	0	-3.9	-525	0	378
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-81	6.17	0	-10.5	-500	0	851
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	12614				276143	24966	-167688
center of Gravity		x0	y0	z0			
		21.9	2.0	-15.1			

Table A.6

DL & Center of Gravity of Half Bascule Span

(effective DL for MHHW under opened position)

Load Description	Value (kips)	x (ft)	y (ft)	z (ft)	w*x	w*y	w*z
Rolling Load	1665	16.67	0	12.12	27756	0	
DL at Anchor Col.	360	14.67	0	12.12	5281	0	
Footing	1323	23	1.5	-26.98	30429	1985	-35695
Seal	3319	23	1.5	-37.48	76337	4979	-124396
w1+w36	2958	21.17	0	-8.8	62621	0	-26030
w2	92	21.17	29.83	-8.8	1948	2744	-810
w3	11	8.17	0	3	90	0	33
w4+w37	1572	21.17	0	7.56	33279	0	11884
w5	49	21.17	29.88	7.56	1037	1464	370
w6	-414	10.31	0	7.56	-4268	0	-3130
w7	-68	27.8	0	8.94	-1890	0	-608
w8	136.4	4.67	0	-7.7	637	0	-1050
w9	48.6	39	0	-7.7	1895	0	-374
w10	27	34.67	0	8	936	0	216
w11	35	21.17	-28.33	-21.7	741	-992	-760
w12	41	21.17	-24.288	-20	868	-996	-820
w13	47	21.17	29.33	-20	995	1379	-940
w14	67	21.17	27.03	-20	1418	1811	-1340
w15	20	21.67	-25.3	10.5	433	-506	210
w16	83	21.17	34.5	10.5	1757	2864	872
w17	110	21.17	32.38	18.6	2329	3562	2046
w18	17	21.17	30.25	25.05	360	514	426
w19	134	21.17	32.38	32.9	2837	4339	4409
w20	17	21.17	30.25	40.7	360	514	692
w21	65	21.17	32.38	44.8	1376	2105	2912
w22	16	21.17	32.38	49	339	518	784
w23	88	3	0	24.1	264	0	2121
w24	59	6.8	0	24.1	401	0	1422
w25	66	36.6	0	24.1	2416	0	1591
w26	75	40.5	0	24.1	3038	0	1808
w27	98	21.17	-28	24.1	2075	-2744	2362
w28	33	27.7	0	36	914	0	1188
w29	8	21.17	-27.5	37.4	169	-220	299
w30	6	15	32.5	37.4	90	195	224
w31	19	4.7	34	36	89	646	684
w32	-93.4	4.17	0	-3.9	-389	0	364
w33	-49.6	4.17	0	7.56	-207	0	-375
w34	-85.8	5.42	0	-3.9	-465	0	335
w35	-39.6	5.42	0	7.56	-215	0	-299
-w36	-63	6.17	0	-10.5	-389	0	662
-w37	-132	7.67	0	7.56	-1012	0	-998
summation	11719.6				256679	24160	-159713
center of Gravity		x0	y0	z0			
		20.3	1.9	-15.7			

Appendix B

Maximum and Minimum Axial Loads on a Single Pile of the Bascule Pier under DL and LL

Tables A.7 to A.10 show the calculation of the maximum and minimum pile axial loads due to dead load and live load for MLLW and MHHW elevations. Both opened and closed positions are considered. The maximum and minimum pile axial loads are 57.8 kips and 27.0 kips which are caused by dead load and live load under MLLW elevation when the bridge is under closed position. The maximum axial load is smaller than the capacity of 130 kips for compression. These results are used to combine with seismic analysis results for bascule foundation evaluation in Chapter 3.

Table A.7

Max. Pile Force under DL & Neutral Axes of Pile Group

(effective DL for MLLW)

Pile Numbering	x (ft)	y (ft)	x-x0 (ft)	y-y0 (ft)	Ix0 (per row)	Iy0 (per row)	Px kips	Px' kips	Py kips	Py' kips
1	2	-29.33	-21.00	-31.23	975	441	-6.9	6.1	-6.9	-6.2
2	5	-26.17	-18.00	-28.07	788	324	-6.8	6.2	-5.9	-5.2
3	8	-23.00	-15.00	-24.90	620	225	-6.8	6.2	-5.0	-4.3
4	11	-19.83	-12.00	-21.73	472	144	-6.8	6.3	-4.1	-3.4
5	14	-16.67	-9.00	-18.56	345	81	-6.7	6.3	-3.2	-2.4
6	17	-13.50	-6.00	-15.40	237	36	-6.7	6.3	-2.2	-1.5
7	20	-10.33	-3.00	-12.23	150	9	-6.6	6.4	-1.3	-0.6
8	23	-7.16	0.00	-9.06	82	0	-6.6	6.4	-0.4	0.4
9	26	-4.00	3.00	-5.90	35	9	-6.6	6.4	0.6	1.3
10	29	-0.83	6.00	-2.73	7	36	-6.5	6.5	1.5	2.2
11	32	2.34	9.00	0.44	0	81	-6.5	6.5	2.4	3.1
12	35	5.34	12.00	3.44	12	144	-6.5	6.5	3.4	4.1
13	38	8.34	15.00	6.44	41	225	-6.4	6.6	4.3	5.0
14	41	11.34	18.00	9.44	89	324	-6.4	6.6	5.2	5.9
15	44	14.34	21.00	12.44	155	441	-6.4	6.7	6.1	6.9
16		17.34		15.44	238		-6.3	6.7		
17		20.34		18.44	340		-6.3	6.7		
18		23.34		21.44	460		-6.3	6.8		
19		26.34		24.44	597		-6.2	6.8		
20		29.34		27.44	753		-6.2	6.8		
21		32.34		30.44	926		-6.2	6.9		
summation/max.	345.00	39.89			7322	2520	-6.2	6.9	6.1	6.9
min.							-6.9	6.1	-6.9	-6.2

Neutral Axes & Center of Gravity	Neutral Axis x0	Neutral Axis y0	Center of Gravity xg	Center of Gravity yg
	23	1.90	24.3	2.0

Vertical DL =	12614	kips
Moment Mx =	1270	kips-ft
Moment My =	16398	kips-ft
# of Piles =	315	
# of Piles per Row	15	along x
# of Piles per Row	21	along y
Total Ix0 =	109837	
Total Iy0 =	52920	
DL per Pile =	40.0	kips
Max.Pile Load=	**46.9**	**kips**
Min.Pile Load=	**33.2**	**kips**

Notes:

Px, Px', Py and Py' are additional axial forces due to bending along lines CC, DD, AA AND BB.

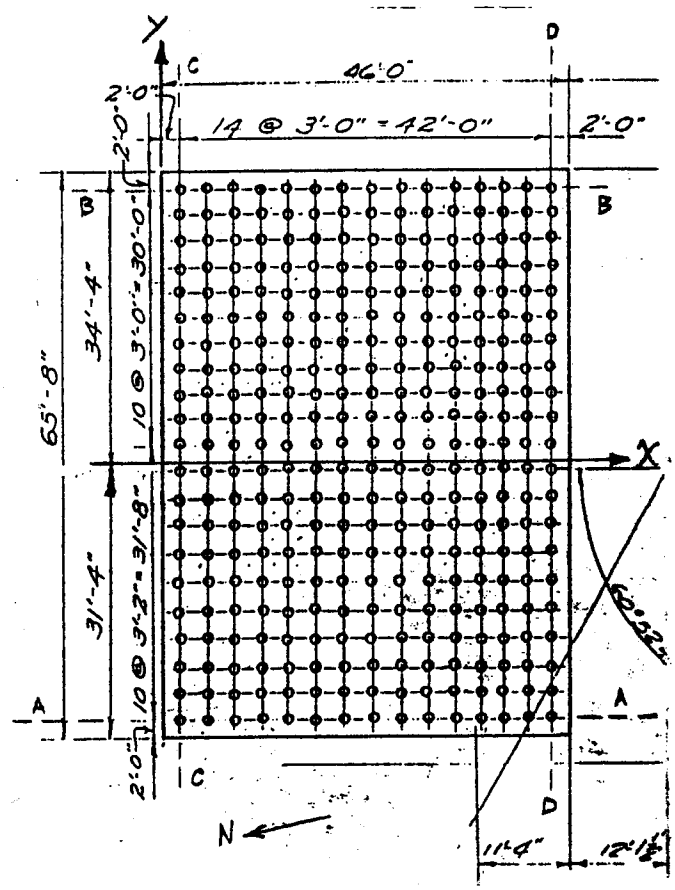

Table A.8

Max. Pile Force under DL & Neutral Axes of Pile Group

(effective DL for MHHW)

Pile Numbering	x (ft)	y (ft)	x-x0 (ft)	y-y0 (ft)	Ix0	Iy0	Px kips	Px' kips	Py kips	Py' kips
1	2	-29.33	-21.00	-31.23	975	441	1.4	-1.4	1.4	1.4
2	5	-26.17	-18.00	-28.07	788	324	1.4	-1.4	1.2	1.2
3	8	-23.00	-15.00	-24.90	620	225	1.4	-1.4	1.0	1.0
4	11	-19.83	-12.00	-21.73	472	144	1.4	-1.4	0.8	0.8
5	14	-16.67	-9.00	-18.56	345	81	1.4	-1.4	0.6	0.6
6	17	-13.50	-6.00	-15.40	237	36	1.4	-1.4	0.4	0.4
7	20	-10.33	-3.00	-12.23	150	9	1.4	-1.4	0.2	0.2
8	23	-7.16	0.00	-9.06	82	0	1.4	-1.4	0.0	0.0
9	26	-4.00	3.00	-5.90	35	9	1.4	-1.4	-0.2	-0.2
10	29	-0.83	6.00	-2.73	7	36	1.4	-1.4	-0.4	-0.4
11	32	2.34	9.00	0.44	0	81	1.4	-1.4	-0.6	-0.6
12	35	5.34	12.00	3.44	12	144	1.4	-1.4	-0.8	-0.8
13	38	8.34	15.00	6.44	41	225	1.4	-1.4	-1.0	-1.0
14	41	11.34	18.00	9.44	89	324	1.4	-1.4	-1.2	-1.2
15	44	14.34	21.00	12.44	155	441	1.4	-1.4	-1.4	-1.4
16		17.34		15.44	238		1.4	-1.4		
17		20.34		18.44	340		1.4	-1.4		
18		23.34		21.44	460		1.4	-1.4		
19		26.34		24.44	597		1.4	-1.4		
20		29.34		27.44	753		1.4	-1.4		
21		32.34		30.44	926		1.4	-1.4		
summation/max.	345.00	39.89			7322	2520	1.4	-1.4	1.4	1.4
min.							1.4	-1.4	-1.4	-1.4
Neutral Axes & Center of Gravity	Neutral Axis				Center of Gravity					
	x0	y0			xg	yg				
	23	1.90			22.70	1.90				

Vertical DL =	11720	kips
Moment Mx =	8	kips-ft
Moment My =	-3516	kips-ft
# of Piles =	315	
# of Piles per Row	15	along x
# of Piles per Row	21	along y
Total Ix0 =	109837	
Total Iy0 =	52920	
DL per Pile =	37.2	kips
Max.Pile Load=	**38.6**	**kips**
Min.Pile Load=	**35.8**	**kips**

Notes:

Px, Px', Py and Py' are additional axial forces due to bending along lines CC, DD, AA AND BB.

Table A.9

Max. Pile Force under DL+LL & Neutral Axes of Pile Group

(effective DL for MLLW under closed position)

Pile Numbering	x (ft)	y (ft)	x-x0 (ft)	y-y0 (ft)	Ix0	Iy0	Px kips	Px' kips	Py kips	Py' kips
1	2	-29.33	-21.00	-31.23	975	441	-15.4	15.4	-15.4	-15.4
2	5	-26.17	-18.00	-28.07	788	324	-15.4	15.4	-13.2	-13.2
3	8	-23.00	-15.00	-24.90	620	225	-15.4	15.4	-11.0	-11.0
4	11	-19.83	-12.00	-21.73	472	144	-15.4	15.4	-8.8	-8.8
5	14	-16.67	-9.00	-18.56	345	81	-15.4	15.4	-6.6	-6.6
6	17	-13.50	-6.00	-15.40	237	36	-15.4	15.4	-4.4	-4.4
7	20	-10.33	-3.00	-12.23	150	9	-15.4	15.4	-2.2	-2.2
8	23	-7.16	0.00	-9.06	82	0	-15.4	15.4	0.0	0.0
9	26	-4.00	3.00	-5.90	35	9	-15.4	15.4	2.2	2.2
10	29	-0.83	6.00	-2.73	7	36	-15.4	15.4	4.4	4.4
11	32	2.34	9.00	0.44	0	81	-15.4	15.4	6.6	6.6
12	35	5.34	12.00	3.44	12	144	-15.4	15.4	8.8	8.8
13	38	8.34	15.00	6.44	41	225	-15.4	15.4	11.0	11.0
14	41	11.34	18.00	9.44	89	324	-15.4	15.4	13.2	13.2
15	44	14.34	21.00	12.44	155	441	-15.4	15.4	15.4	15.4
16		17.34		15.44	238		-15.4	15.4		
17		20.34		18.44	340		-15.4	15.4		
18		23.34		21.44	460		-15.4	15.4		
19		26.34		24.44	597		-15.4	15.4		
20		29.34		27.44	753		-15.4	15.4		
21		32.34		30.44	926		-15.4	15.4		
summation/max.	345.00	39.89			7322	2520	-15.4	15.4	15.4	15.4
min.							-15.4	15.4	-15.4	-15.4

Neutral Axes & Center of Gravity	Neutral Axis		Center of Gravity	
	x0	y0	xg	yg
	23.00	1.90	25.90	1.90

Vertical DL = 13352 kips
Moment Mx = 9 kips-ft
Moment My = 38721 kips-ft
# of Piles = 315
# of Piles per Row 15 along x
# of Piles per Row 21 along y
Total Ix0 = 109837
Total Iy0 = 52920
DL per Pile = 42.4 kips
Max.Pile Load= 57.8 kips
Min.Pile Load= 27.0 kips

Notes:
Px, Px', Py and Py' are additional axial forces due to bending along lines CC, DD, AA AND BB.

Table A.10

Max. Pile Force under DL & Neutral Axes of Pile Group

(effective DL for MLLW under opened condition)

Pile Numbering	x (ft)	y (ft)	x-x0 (ft)	y-y0 (ft)	Ix0	Iy0	Px kips	Px' kips	Py kips	Py' kips
1	2	-29.33	-21.00	-31.23	975	441	5.1	-5.9	5.1	5.9
2	5	-26.17	-18.00	-28.07	788	324	5.2	-5.8	4.4	5.1
3	8	-23.00	-15.00	-24.90	620	225	5.2	-5.8	3.6	4.3
4	11	-19.83	-12.00	-21.73	472	144	5.3	-5.8	2.8	3.5
5	14	-16.67	-9.00	-18.56	345	81	5.3	-5.7	2.0	2.7
6	17	-13.50	-6.00	-15.40	237	36	5.3	-5.7	1.2	1.9
7	20	-10.33	-3.00	-12.23	150	9	5.4	-5.6	0.4	1.1
8	23	-7.16	0.00	-9.06	82	0	5.4	-5.6	-0.4	0.4
9	26	-4.00	3.00	-5.90	35	9	5.4	-5.6	-1.1	-0.4
10	29	-0.83	6.00	-2.73	7	36	5.5	-5.5	-1.9	-1.2
11	32	2.34	9.00	0.44	0	81	5.5	-5.5	-2.7	-2.0
12	35	5.34	12.00	3.44	12	144	5.5	-5.5	-3.5	-2.8
13	38	8.34	15.00	6.44	41	225	5.6	-5.4	-4.3	-3.6
14	41	11.34	18.00	9.44	89	324	5.6	-5.4	-5.1	-4.4
15	44	14.34	21.00	12.44	155	441	5.6	-5.4	-5.9	-5.2
16		17.34		15.44	238		5.7	-5.3		
17		20.34		18.44	340		5.7	-5.3		
18		23.34		21.44	460		5.8	-5.3		
19		26.34		24.44	597		5.8	-5.2		
20		29.34		27.44	753		5.8	-5.2		
21		32.34		30.44	926		5.9	-5.2		
summation/max.	345.00	39.89			7322	2520	5.9	-5.2	5.1	5.9
min.							5.1	-5.9	-5.9	-5.2

Neutral Axes & Center of Gravity	Neutral Axis		Center of Gravity	
	x0	y0	xg	yg
	23.00	1.90	21.90	2.00

Vertical DL =	12614	kips
Moment Mx =	1270	kips-ft
Moment My =	-13875	kips-ft
# of Piles =	315	
# of Piles per Row	15	along x
# of Piles per Row	21	along y
Total Ix0 =	109837	
Total Iy0 =	52920	
DL per Pile =	40.0	kips
Max.Pile Load=	**45.9**	**kips**
Min.Pile Load=	**34.2**	**kips**

Notes:

Px, Px', Py and Py' are additional axial forces due to bending along lines CC, DD, AA AND BB.

Appendix C
Geotechnical Report

GEOTECHNICAL REPORT

SOUTH PARK BRIDGE
SEISMIC EVALUATION
KING COUNTY, WASHINGTON

00-038 August, 2001

PanGEO
INCORPORATED

PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

August 15, 2001
File No: 00-038

Mr. Robert Schamber
Imbsen & Associates, Inc.
9912 Business Park Drive, Suite 130
Sacramento, CA 95827

RE: GEOTECHNICAL EARTHQUAKE ENGINEERING REPORT
South Park Bridge Seismic Evaluation
King County, Washington

Dear Bob,

The attached report provides earthquake engineering parameters and foundation capacities for your use in evaluating the seismic performance of the existing South Park Bridge. This report summarizes our discussions and technical memoranda that were provided to you during the course of our study. This report supersedes our February 26 submittal to include review comments from the County and a discussion of the effects of the February 28, 2001, Nisqually Earthquake upon the South Park Bridge.

Using existing site data, we developed site-specific response spectra at the foundation level of the bascules corresponding to events having a 10% probability of occurrence in the next 10 to 20 years (return intervals of 95 and 190-years) or the anticipated service life of the structure. We concluded that while liquefaction may occur during either of these events, it would not significantly affect the load capacity of the piles supporting the bascules. Also, we concluded that the distressed (cracked) condition of the bascules is not attributed to the performance of the bridge foundations.

We appreciate the opportunity to work with you and King County on this project. Please call if there are any questions on this report.

Sincerely,

W. Paul Grant

W. Paul Grant, P.E.
Principal

Encl.: Geotechnical Report

3414 NE 55th Street
Seattle, WA 98105
Tel: (206) 262-0370
Fax: (206) 262-0373

SOUTH PARK BRIDGE
SEISMIC EVALUATION
KING COUNTY, WASHINGTON

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APPENDIX A - EXISTING EXPLORATIONS

SOUTH PARK BRIDGE
SEISMIC EVALUATION
KING COUNTY, WASHINGTON

1.0 PURPOSE AND SCOPE

This report summarizes our study to provide earthquake engineering parameters and foundation capacities to evaluate the seismic performance of the existing South Park Bridge across the Duwamish River in King County (See Figure 1). Our study included developing site-specific response spectra for the bridge site corresponding to earthquakes that would have a low (10%) probability of occurrence during the 10 to 20 year remaining service life of the bridge. We also evaluated the capacity of the pile foundations of the bridge to resist both static and dynamic loads, include potential effects of liquefaction. Our efforts also included securing the services of a surveyor (Minister and Glaeser Surveying Inc.) to perform a bathymetric survey of the channel and to obtain elevations at selected locations on the bridge.

The results of our studies were provided to you in various technical memoranda dated between September 8 and 22, 2000. The survey data were provided to you and King County in a separate transmittal of electronic files and AutoCAD drawings. This report summarizes the results of our studies and our various conversations.

2.0 BRIDGE DESCRIPTION AND PERFORMANCE

The South Park Bridge, formerly known as the 14th/16th Avenue South Bridge, is a 1,285-foot long structure that carries 4 traffic lanes and two sidewalks over the Duwamish River. The bridge, which was constructed in 1930 –1931, includes a Scherzer Rolling Lift double leaf moveable bascule (span length of 190-feet) that is flanked on each side with two deck truss approach spans and 12 concrete slab approach spans. The movable bascule leaves are mounted on a track that permits each leaf to move horizontally a distance of 18 feet during the bridge opening. The bascule spans are supported on reinforced concrete piers that are founded on timber piling. The steel truss and concrete slab approach spans are similarly supported on timber piling.

Over the past 70 years, the bridge has experienced some distress and operational difficulties. A number of engineering studies have been conducted to evaluate the cause of the distress to the bridge and various repairs have been made to the bridge to address the operational deficiencies. The bridge distress includes excessive cracking of the

bascules and difficulties in closing the leaves of the bascules. Significant cracking of the bascules was noted following the 1965 magnitude 6.5 Puget Sound Earthquake (Sverdrup, 1991). To address the closure difficulties, remedial post tensioning was conducted on the bascules in the early 1980's to improve the performance of the bascule leaves.

In addition to the distress experienced in the bascules, several spans of the approaches have experienced settlement related distress. The settlement developed because the timber piles supporting the approaches extended above the water table and the piles deteriorated over time causing the approaches to settle. Selected piers in the north and south approaches were rehabilitated in the early 1970's by underpinning the foundations with augercast piles or by removing the deteriorated sections of wood piles and replacing these segments with steel beams. Currently, Bents 10 and 11 at the north approach show signs of excessive settlement (perhaps 3 inches). It is unclear if this settlement resulted from the foundation underpinning in the 1970's or if the settlement occurred later. Regardless, King County should monitor the elevations of these bents to confirm whether the bents are stable.

3.0 PREVIOUS STUDIES

Numerous bridge inspection reports and engineering studies have been conducted to review the condition of the bridge, to speculate on the cause of the distress to the structure, and to develop schemes to rehabilitate the bridge. Conditional surveys of the bridge include elevational surveys of the bridge superstructure (King County 1991, 1992, and 1998), and underwater inspection surveys of the bridge (Echelon Engineering, 1997). Additionally, Sverdrup performed engineering studies to evaluate the condition of the bridge (Sverdrup, 1987) and provide specific recommendations for the rehabilitation of the North Bascule (Sverdrup, 1991). Finally, in conjunction with the various Sverdrup studies, Shannon and Wilson provided several letters and reports evaluating the foundation performance of the bridge (Shannon and Wilson, 1987 and 1991) and evaluating the liquefaction potential along the bridge corridor (Shannon and Wilson, 1991 and 1994).

Shannon and Wilson's 1991 report suggests that the observed cracking of the North Bascule is related to the performance of the underlying pile foundations. Specific conclusions derived from Shannon and Wilson's study and related supporting evidence include the following:

- **Current Foundations Have Inadequate Capacity**
  - Piles had estimated allowable capacities of 6 to 12 tons (Design Load of 22 tons)
- **Foundation Settlement Resulted in Superstructure Cracking**
  - Estimated long term settlement of bascule of 2 to 3-inches
  - Estimated long term differential settlement of bascule of 1 to 2-inches
  - The outer piles in the bascule may be overstressed during the normal operation of the bascule (i.e. leading to pile overloading and resulting settlement)
  - Wind on the bascule in the open position may have overstressed the outer rows of piles, leading to foundation settlement
  - Dredging of the channel may have exposed the piles, leading to attack from marine borers (resulting in pile deterioration and consequent settlement)
- **Extensive Foundation Rehabilitation Needed (including compaction grouting and new piles) To Address the Above Foundation Concerns**

4.0 NEW SURVEY DATA

As a component of our current study, Minister and Glaeser Surveying, Inc. was retained to provide bathymetric and topographic survey information to be used in our evaluation of the performance of the bridge. A bathymetric survey was conducted to provide information on the elevation of the channel underlying the bridge alignment. This data was used in our liquefaction and lateral spreading analysis of the alignment. The bathymetric survey also provided data to confirm whether the channel was being scoured, which could potentially expose the foundation piles. A limited topographic survey was conducted to provide elevations on discrete points of the bridge to allow us to confirm whether any differential settlement has occurred within individual bascules or between the bascules. Survey elevations were also obtained on the cross beams within the bridge approaches to allow us to evaluate potential settlements of the bents.

Minister and Glaeser performed the site Survey on July 25 and 26, 2000 (Minister-Glaeser, 2000). The bathymetric survey was obtained along four section lines spaced at 25 foot intervals both upstream and downstream of the bridge. The bascule survey consisted of determining the elevations of the tracks at both bascules. The results of the survey were provided to King County in electronic data files and in AutoCAD drawings.

The Minister and Glaeser survey data was not tied into the King County benchmarks. However, the Minister and Glaeser elevation data may be approximately converted to the NAVD88 datum by subtracting 81 feet from the Minister and Glaeser data. Similarly, subtracting 90.6 feet from the Minister and Glaeser elevations will provide elevations approximating the City of Seattle datum.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions beneath the main span of the bridge were inferred from existing borings and construction records of the bridge foundations. Subsurface conditions were inferred from borings drilled by Shannon and Wilson (SW) in previous studies of the bridge (Shannon and Wilson, 1991 and 1994) as well as borings drilled by Dames and Moore (DM) for the foundation studies for Boeing facilities at the north end of the bridge (Lamont, 2000). The locations of the borings used in our evaluation are shown on Figure 2 and individual logs from these explorations are presented in Appendix A. A generalized cross section depicting subsurface conditions beneath the bascules is presented in Figure 3. Standard Penetration Test (SPT) resistance values (N-Values) are shown on the profile for the corresponding boring logs.

The mudline shown on Figure 3 was derived from the recent bathymetric survey of the channel (Minister-Glaeser, 2000). This recent survey data agrees fairly well with the mudline elevations obtained by Echelon Engineering in 1994 and 1997 (Echelon Engineering, 1997). Therefore, the channel survey data suggest that no significant aggradation or scour has occurred in the vicinity of the bridge between 1994 and 2000.

The subsurface stratigraphy depicted on Figure 3 was largely derived from Shannon and Wilson's 1991 report. Essentially, the underlying subsurface conditions consist of recent alluvium and estuarine deposits that are underlain at depth by glacially consolidated soils (Glacial Till). The upper 15 feet of alluvium is generally very loose to loose, clean to silty fine sand. The underlying alluvium and estuarine deposits are generally medium dense to dense, clean to silty, fine sand. A very soft silt layer was encountered between about elevations –80 to –90 feet (City of Seattle datum). Finally, glacially consolidated sediments (glacial till) were encountered in borings at the North Bascule between elevations –95 and –100 feet. Inferences from pile driving records suggest that these same glacial soils are present at about elevation –70 feet at the South Bascule.

As shown in Figure 3, the timber piling supporting the south bascule are inferred to be driven to provide end bearing in the underlying glacial till stratum. The timber piles at the North Bascule may be driven to tip elevations that are just above the hard, glacial till stratum.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 EARTHQUAKE EVALUATION CRITERIA

Assuming that the remaining service life of the existing South Park Bridge may be 10 to 20 years, our seismic evaluations (ground motion and liquefaction) were conducted for earthquakes having a 10% chance of occurrence in 10 and 20-years (return periods of 95 and 190 years, respectively). This 10% probability of occurrence during the remaining service life is a conservative standard for evaluating existing structures and it is consistent with seismic design standards for new bridges that are based on ground motions that have a 10% chance of occurring during the bridge design life of 50 years. The recommended approach for the South Park Bridge is conservative because most new bridges have a service life of well beyond 50 years. Therefore, a 10% probability of occurrence in 10 years would appear to be more consistent with the relative seismic risk for new bridges.

6.2 FAULTING

There are no known faults directly underlying the bridge. The closest active fault is the Seattle Fault that is located about 4 miles north of the bridge. The Seattle Fault is an east-west trending structure that is inferred to extend from Bainbridge on the west to Issaquah on the east. The structure was last presumed to have moved about 1,100 years ago. The fault movement is inferred from dating of geologic evidence including 1) uplifted terrace deposits on the south side of the fault, 2) submerged shorelines on the north side of the fault, 3) tsunami deposits on Whidbey Island, and 4) major landslide features preserved within Lake Washington.

Ground rupture associated with the Seattle Fault has only been observed at a few localized areas, including a location on Bainbridge and another location in the Eastgate area of Bellevue. Other than these locations, there are no known locations of ground rupture associated with this fault. In general, movement of this fault has commonly resulted in a broad uplifting of the area (as much as 20 feet).

Although the last major rupture of the Seattle Fault has been estimated to have had occurred about 1,100 years ago, several researchers from the United States Geological Survey (USGS) believed they have found evidence on Bainbridge Island of multiple

ruptures of this fault that occurred within the past 12,000 years. In their seismic hazard evaluation of the contiguous United States, the USGS has characterized the Seattle Fault as having a characteristic earthquake magnitude of about 7 with a recurrence interval of about 5,000 years.

Based on the past performance of this fault and the fact that the bedrock at the site is mantled by over 3,000 feet of Quaternary sediments, ground rupture from faulting is not considered to be a design concern for the seismic renovation of the bridge.

6.3 Site Specific Response Spectra

One-dimensional ground response analyses were conducted for the site to provide site-specific response spectra for use in evaluating the seismic performance of the existing bridge. USGS derived equal hazard response spectra were used to model the source motions for the 95 and 190-year events. Time histories were then selected that approximately matched the equal hazard spectra (EHS). These time histories were subsequently used in our one-dimensional (SHAKE 91) ground response studies of the bridge. The subsurface conditions at the site were generalized into a singular soil profile with dynamic soil properties selected based upon our experience from other sites underlain by similar geologic conditions where we have conducted extensive field testing to determine the dynamic soil properties. Ground motions and response spectra were calculated at the foundation level for the bascules for each level of ground shaking. The following summarizes the different elements of the study.

6.3.1 Equal Hazard Spectra and Source Motions

The ground motions for the site were developed from time histories that were selected to approximately match the United States Geological Survey (USGS) equal hazard response spectra corresponding to a site underlain by "rock" or "rock-like" conditions. Equal hazard response spectra for the site were obtained from the results of regional probabilistic hazard analyses conducted by the USGS for the Pacific Northwest. The ground motions derived from these regional probabilistic hazard studies include considerations for the potential occurrence of subduction zone earthquakes as well as events on the nearby Seattle Fault. In our opinion, these regional studies are sufficient to establish the equal hazard response spectra for use in evaluating the seismic performance of the bridge. The resulting equal hazard response spectra are presented in Figure 4 for the 95-year event and in Figure 5 for the 190-year event. For general purposes, the 95-year spectrum may be considered to result from a magnitude 6.0 to 6.5 event and the 190-year spectrum may correspond to a 6.5 to 7.0 event. A typical "code" level earthquake

(i.e. 10% probability of occurrence in 50-years or a 475-year return) may correspond to a magnitude 7.0 to 7.5 event.

6.3.2 Source Motions

Having defined the equal hazard response spectra for the evaluation earthquakes, we then selected time histories that approximated the equal hazard spectra. We selected two time histories to provide this match. The time histories recorded at the Olympia Highway Test Lab during the magnitude 7.1, Puget Sound Earthquake of April 13, 1949, were modified for use in our analysis to approximate both the 95-year and the 190-year bedrock motions. Specifically, we used the "natural" West (S86W) component of this record, only modified for acceleration, to match the equal hazard spectrum for both events. However, we performed a spectral match of the South (S04E) component of this record to provide a better fit to the equal hazard spectra in the period range of interest for the bridge (0.2 to 2.0 seconds). As indicated in Figures 4 and 5, both records provide a very good match to the target response spectra in the anticipated period range of the bridge and, therefore, are suitable for use in the ground response analyses.

6.3.3 Soil Model

The ground response analyses were conducted for a singular soil profile that was developed based upon our understanding of the site geology and our expectations of the dynamic properties of the subsurface geologic units. The soil profile was mainly categorized as to whether the underlying soil unit was alluvium/estuarine deposits or glacially consolidated soil. The profile was further simplified by assuming that very stiff, "till-like" material underlies both bascules at a depth of 70 feet and that glacial till underlies both bascules at a depth of 85 feet. This simplification results in being able to use one set of ground motions for both the North and South Bascules.

Another simplification of the soil model was the selection of shear wave velocities for the subsurface units and determining the appropriate dynamic soil properties to represent the underlying soils. The shear wave velocities were selected based upon our experience gained from geophysical downhole velocity measurements at other sites in the region that are underlain by similar geologic units. Lacking site-specific data, the selected distribution should provide a reasonable approximation of the conditions along the bridge alignment. The dynamic properties (i.e. modulus degradation and damping) of all materials underlying the bridge were modeled using standard relationships for sands (Idriss and Sun, 1992). The following soil model was used in our evaluations:

Generalized Soil Model				
Layer	**Depth (ft)**	**Material**	**V_S (fps)**	**Density (pcf)**
1	0 – 10	Sand (Alluvium)	350	105
2	10 - 25	Sand (Alluvium)	500	105
3	25 - 40	Sand (Alluvium)	600	105
4	40 - 55	Sand (Alluvium)	700	105
5	55 - 60	Silt (Alluvium)	450	90
6	60 - 70	Sand (Estuarine)	650	105
7	70 - 85	Silt (Till-Like)	850	120
8	85 - 130	Hard Silt (Till)	1400	125
9	130 - 200	Hard Silt (Till)	1800	125
10	>200	"Rock"	2500	130

6.3.4 Computed Response Spectra

The dynamic response of the soils at the site was evaluated using the one-dimensional computer program SHAKE91 (Idriss and Sun, 1992) along with the time histories of the records presented in Figures 4 and 5 to analyze the soil profile discussed above. The ground response analyses were performed to provide values of ground motion and response spectra at the base of the concrete seals for the bascules. Motions at this level would then be used as input to the dynamic structural model of the bridge.

The response spectra computed at a depth of 25 feet (base of the concrete seal) are presented in Figures 6 and 7 for the 95-year and 190-year events, respectively. Both figures also contain our recommended smoothed response spectrum for each event that

may be used for the seismic evaluation of the bridge. The smoothed response spectrum was largely derived as an upper bound to the computed results in the constant acceleration portion of the response spectrum and as more of an average value in the constant velocity portion of the response spectrum. Specific values of the recommended response spectra are as follows:

Recommended Response Spectra PSA (g)		
Period (Seconds)	95 Yr. Event	190 Yr. Event
0.01	0.15	0.20
0.03	0.15	0.20
0.12	0.30	0.40
0.52	0.30	0.40
1	0.16	0.21
2	0.08	0.10
3	0.035	0.046
5	0.012	0.017

The recommended response spectra for the 95-year and 190-year events are also presented on Figure 8, along with response spectra from the Uniform Building Code (UBC, 1997), the American Association of State Highway and Transportation Officials (AASHTO, 1996), and the Olympia recording of the April 13, 1949, Puget Sound Earthquake. The UBC and the AASHTO spectra correspond to ground surface motions of an event having a 10% probability of occurrence in 50 years (475-year return). Consequently, the UBC and AASHTO response spectra are expected to be larger than the recommended site spectra. However, the recommended spectra have general shapes that appear to be consistent with UBC and AASHTO when considering appropriate reductions for foundation embedment and the size of the design event. Also, the recommended response spectra have shapes that appear to reasonably represent the actual recorded motion in Olympia during the April 13, 1949, Puget Sound Earthquake.

Again, the recommended response spectra represent an approximation of the site response. A detailed site-specific response analysis, with multiple time histories matched to the target spectra, is beyond the current scope of our work.

6.4 Liquefaction Evaluation

6.4.1 Current Study

The liquefaction potential of the soils underlying the bridge alignment was analyzed for ground shaking having a 10% chance of occurrence in 10 and 20-years (return periods of 95 and 190 years, respectively). Based upon the results of our ground response analyses, peak ground surface accelerations of about 0.16g and 0.20g were used to be consistent with these probabilities. The liquefaction analyses were conducted using existing boring data along the bridge corridor and the simplified liquefaction procedures of Seed and others (Seed and Idriss, 1982 and Seed and Others, 1984).

The borings most pertinent to the analysis were the three explorations drilled along the bridge Right-of-Way by Shannon and Wilson in 1991 and 1994 (Borings B-1 through B-3). These borings were drilled using mud rotary drilling techniques and sampling was conducted using Standard Penetration Test (SPT) procedures. Both mud rotary drilling and SPT sampling are needed to provide the best quality data for liquefaction evaluations.

The SPT data obtained from these borings and the computed factors of safety are presented in Figure 9 and 10 for the 95-year and 190-year events, respectively. The results indicate that liquefaction would occur for both levels of ground shaking to a depth of about 15 feet below the ground surface.

In addition to the Shannon and Wilson borings, we also reviewed the results of borings advanced by Dames and Moore for the Boeing facilities located near the north abutment. Although the Dames and Moore borings were not performed using SPT procedures, the blow counts obtained from these explorations are plotted in Figure 11 along with the SPT N-values from the Shannon and Wilson borings. This plot shows that the blow counts from the Dames and Moore explorations are generally in good agreement with the SPT N-values of the Shannon and Wilson explorations. Therefore, while liquefaction analyses were not specifically performed for the Dames and Moore borings, it is inferred from the data presented in Figure 11 that the liquefaction potential of the soils along the north approach is similar to the potential at other locations along the bridge alignment. Specifically, liquefaction may occur to a depth of about 15 feet along the bridge corridor for both levels of ground shaking.

6.4.2 Comparison With Previous Studies

Prior studies by Shannon and Wilson have addressed the liquefaction potential along the project corridor (Shannon and Wilson, 1991 and 1994). Shannon and Wilson's 1991 study addressed the liquefaction potential at the bascules with respect to the historic 1949 and 1965 Puget Sound Earthquakes. This study was conducted using the subsurface data from borings B-1 and B-2 that were drilled adjacent to the north bascule (see Figure 2). Because the intent of the evaluation was to confirm the potential occurrence of liquefaction during either the 1949 or the 1965 earthquakes, the liquefaction evaluation was based upon a peak ground acceleration of 0.15g. As a result of this study, Shannon and Wilson concluded, "...liquefaction may have occurred within 10 to 15 feet of the mudline (average elevation –25 feet) during either the 1949 or the 1965 earthquakes."

Shannon and Wilson's conclusion about liquefaction potentially occurring to a depth of 10 to 15 feet at the bascules agrees with the data presented in Figure 9 for the 95-year event (PGA = 0.16g) which similarly shows that liquefaction is likely to occur (i.e. factors of safety of less than 1) to a depth of about 15 feet. Therefore, there is excellent agreement between the estimated depths of liquefaction between the 1991 Shannon and Wilson study and our current evaluation. This agreement is to be expected because the same data were used in both evaluations.

In a later study, Shannon and Wilson (1994) evaluated the liquefaction potential beneath the approaches to the bridge. This subsequent study, however, was performed to evaluate the liquefaction potential corresponding to the occurrence of an event having a 10% chance of occurrence in 50-years (i.e. 475-year return interval). For this evaluation, Shannon and Wilson drilled a boring at the south approach span to the bridge (boring B-3 shown on Figure 2) and used existing borings from adjacent Boeing facilities at the north approach to the bridge. Shannon and Wilson used a peak ground acceleration of 0.30g to characterize the ground shaking in the liquefaction analysis. As a result of this later study, Shannon and Wilson concluded that, "...most of the soils above elevation –30 have factors of safety significantly less than 1.0, indicating that these soils will liquefy under the design ground motions (0.30g)."

Because Shannon and Wilson's 1994 analysis was conducted for a significantly larger event than our current study (i.e. 95-year and 190-year events) the results of the two studies are not directly comparable. However, the trends shown in our liquefaction evaluation for the 190-year event may be extrapolated and compared to the results of the 1994 Shannon and Wilson study. Specifically, the liquefaction results presented in Figure 10 (190-year event and PGA = 0.20g) show that liquefaction is quite probable to a depth of about 15 feet (i.e. factors of safety significantly less than 1). However, the data for boring B-3 show factors of safety close to 1 down to a depth of about 32 (approximate elevation –25 feet). Clearly, then, a larger earthquake acceleration would reduce these

factors of safety below unity and suggest the occurrence of liquefaction to about elevation –25 feet. Therefore, based on this extrapolation, we conclude that our liquefaction assessments are in general agreement with the earlier studies conducted by Shannon and Wilson.

6.5 Lateral Spreading Evaluation

The occurrence of liquefaction may trigger lateral spreading of the slopes along the shoreline. Empirical relationships developed by Youd and others (1999) were used to determine ground displacements that may result from lateral spreading. These empirical procedures indicate that ground movements from lateral spreading may be on the order of 6 inches along the adjacent riverbanks. The past performance of slopes in the region would suggest that this displacement might be a realistic estimate for earthquake induced ground movements. The occurrence of lateral spreading may result in additional lateral earth pressures on the bridge piers and bascules. The magnitude of these additional pressures is discussed subsequently.

6.6 Foundation Capacity

Timber pile foundations are used to support the bascules as well as the approach structures. Both of the bascules are supported on approximately 315 piles. Design loads for these piles are 22 tons. Structural analyses conducted by Imbsen and Associates indicated that the maximum loading on the outboard piling may be on the order of 25 tons during the 190-year earthquake.

Piles at the South bascule were driven to refusal and terminate within a glacial till stratum at an average tip elevation of –73 feet (City of Seattle Datum). Piles at the north bascule were driven to an average tip elevation of –97 feet (City of Seattle Datum) and terminate within either an Estuarine sand deposit or weathered glacial till. Driving records for the piles in the north bascule indicate that the piles did not reach refusal. However, over 97% of the piles were driven to final penetrations of 2 inches or less for the last blow of the pile driving.

To determine the capacity of the piles supporting the North Bascule we used an accepted empirical driving relationship developed by the Washington State Department of Transportation (WSDOT, 2000). This relationship provides an estimate of the ultimate capacity of the piles based on the average penetration that occurred for the last foot of pile driving. The bridge construction records indicate that the piles supporting the bascules were driven with a Vulcan 01 pile-driving hammer. Using information available for this hammer and the WSDOT pile driving relationship, we developed the curve presented in Figure 12 that presents ultimate pile capacity at the end of pile driving.

Because this curve represents field conditions during driving, the indicated capacities are conservative for the long-term performance of the piles because they do not reflect conditions that would develop after a pile is allowed to set and gain additional strength from "freeze back" (i.e. pore water pressure dissipation after pile driving). Accordingly, to account for potential freeze back, a resistance of 25 tons may be added to the ultimate capacities shown on Figure 12. This additional resistance is unaffected by the potential occurrence of liquefaction because liquefaction would not likely extend below the base of the seals at the bascules and, hence, would not affect pile capacity.

Therefore, based solely on the ultimate pile capacity information provided in Figure 12, 97% of the piles beneath the North Bascule would have ultimate capacities in excess of 40 tons (final driving penetrations of 2-inches or less) and corresponding factors of safety of about 2 or greater for the design load of 22 tons. Of the remaining piles, 4 piles had penetrations of 3 inches or greater on the final blow of driving. Three of these piles were located in the interior of the pile cap (located behind the perimeter row of piles). Two of these piles had 3 inches of penetration while the third had 3.5 inches of penetration. Accordingly, all three piles would have ultimate capacities of 26 to 30 tons and corresponding factors of safety of 1.2 to 1.4 for the design load of 22 tons (see Figure 12) excluding any potential beneficial effects of "freeze-back".

Only one pile had a penetration that would lead to the conclusion that it may have a factor of safety of 1 or less. This pile, which was driven in one of the rows at the perimeter of the cap, had a penetration of 4 inches during the final blow of the pile driving. Based on the information presented in Figure 12, this pile would have an ultimate capacity of about 22 tons corresponding to conditions immediately following pile driving. Assuming that the outer row of the piles may be temporarily loaded to 25 tons, then this singular pile may have a factor of safety of less than 1. Because the piles are closely spaced within the cap (spacing at 3-foot centers), conditions of a low factor of safety would result in load being shed to adjacent piles that have reserve capacity. Therefore, a single pile with a low factor of safety does not necessarily lead to concern for the performance or safety of the bascule. Similarly, when considering the additional capacity associated with "freeze-back", this pile would have a factor of safety greater than unity.

For simplifying purposes, the piles supporting the bascules may be assumed to have an ultimate compressive capacity of 65 tons and an ultimate uplift capacity of 25 tons. The compressive capacity corresponds to the 40-ton ultimate load derived from Figure 12 that would apply to over 97% of the piles and the additive freeze back capacity of 25 tons. The recommended tension capacity of the piles is conservative when considering that a pullout test conducted during construction on one of the piles driven at the South Bascule resulted in an ultimate tensile load of 45 tons.

From the above discussions and the information presented in Figure 12, we conclude that the capacity of the foundations of the bascules provides an adequate factor of safety for the design load of 22 tons and potential temporary loads at the perimeter piles of 25 tons. By considering the additional 25 tons of "freeze back" capacity that would be available in addition to the capacities shown on Figure 12, all piles supporting the North Bascule would have a factor of safety of about 2 or greater.

The conclusion on the adequacy of the existing pile foundations is further substantiated with the following:

- **Tiltmeter Monitoring** – Tiltmeters were installed on the North and South Bascules in the early 1990's to determine if the bridge openings resulted in settlement or movement (tilting) of the bascules that could be related to aberrant behavior of the foundations (Sverdrup, 1991). The recorded tiltmeter data show no significant movement or tilting at either the North or South Bascules. Hence, the tiltmeter data supports satisfactory performance of the foundations.

- **Bridge Settlement Data** - Survey measurements of the elevations of the tracks upon which the rolling leaves of the bascules travel show that the recent measurements of track elevation (King County, 1991, 1992, and 1998; and Minister-Glaeser, 2000) have not changed appreciably from the design elevations. Therefore, the survey measurements confirm that there is no differential settlement between the North and South Bascules and that there is no significant differential settlement within either the North or South Bascules. Thus, the bridge survey data supports satisfactory performance of the foundations.

- **Bridge Repairs** – There is no evidence on the track or beneath the track to suggest that the tracks may have been shimmed or otherwise repaired to address differential settlement. Hence, the lack of bridge repair data to the tracks supports satisfactory performance of the foundations.

- **Bridge Distress** – The observed cracking pattern within the bascules is not consistent with any mode of deformation that would be suggestive of poor foundation performance. Hence, the lack of any cracking pattern reflective of poor foundation performance leads to the conclusion that supports the satisfactory performance of the foundations.

Therefore, based on the above, we conclude that the foundations for the bascules have performed satisfactorily to date and can adequately support the existing static load and the anticipated dynamic loads and that extensive foundation rehabilitation is not required.

The observed distress to the bascules, including cracking above elevation 0 and spalling below this level is more suggestive of thermal effects or other structural considerations as opposed to foundation performance. The observed vertical through going cracks on both the North and South Bascules combined with the improved operational performance of the bascules following the post tensioning rehabilitation of the bascules suggests that the deficiencies of the bridge are related to the superstructure and not the substructure.

6.7 FOUNDATION STIFFNESS

6.7.1 Base Spring Constants

Spring constants for the dynamic model of the bridge may be formulated using two different approaches. The first method involves evaluating the stiffness of the individual piles and then extrapolating this constant for the entire foundation based upon the number of piles contained in the foundation. For this evaluation, the vertical spring constants of the piles may be determined by approximating the ultimate pile load in either compression or tension and dividing this ultimate load by a threshold displacement corresponding to the yield condition of the pile/soil system.

The vertical spring constants of the piles may be simplified by assuming that the piles may resist an ultimate load of 130 kips at a deflection of 2.5 inches (plastic state) and an ultimate tension load of 50 kips at a deflection of 0.5 inches.

Appropriate translational spring constants may be determined from the information provided in Figure 13, which shows load-deflection curves for piles within the North and South Bascules and the two adjacent piers (Pier 1 North and South). Capacity curves for both the North and South Bascule as well as the adjacent piers (Piers 1N and 1S) are quite similar reflecting the fixed pile head conditions that was assumed to exist at these locations because of the relatively thick pile cap and base seal used in the foundation construction. However, conditions at Pier 2 (north and south) are considerably different, reflecting the free head conditions that exist because of the relatively thin pile cap and no base seal. The pile capacities presented in Figure 13 include the influence of pile group effects and potential liquefaction.

Alternatively, the base spring constants may be determined using the information presented in Figure 14 and the shear moduli values presented below. The information presented in Figure 14 was derived from solutions for footings at the surface of an elastic half space (Whitman and Richart, 1967). This particular method approximates the stiffness of the pile foundation by replacing the pile stiffness with an equivalent soil having the stiffness values indicated below. A potential advantage of this procedure is that it may simplify computation of the rotational and rocking spring constants.

	SHEAR MODULUS (G) - ksi	
	Vertical and Rocking	Translation and Rotation
Bascules	30	1.5
Approaches	15	1.0

The spring constant charts presented in Figure 14 were developed based upon a unit shear modulus of 1 ksi. The recommended shear moduli values in the above tabulation should be used in conjunction with Figure 14 to develop the appropriate spring constants for the foundations. The shear moduli values in the above tabulation have been adjusted to reflect the presence of the pile foundations and two different moduli values are given depending upon the orientation of the foundation spring. Because the stiffness of the piles in the axial direction dominates the ultimate response of the foundation, the corresponding shear moduli for vertical loading and rocking are relatively large. However, because the piles are relatively weak in resisting torsional and translational loading, lower shear moduli are provided to reflect these conditions.

Lower shear moduli values were recommended for the approaches as compared to the bascules to reflect the lower level of confinement afforded to the more shallow approach foundations. The shear moduli provided for the bascules apply equally to both the north and south bascule, without differentiation between the bascules. This recommendation is consistent with our conclusion on the adequacy of the piles supporting the north bascule. Consequently, the existing site data does not warrant a separate characterization of the bascules.

Spring constants derived using either of the above procedures do not reflect embedment effects and additional spring constants will need to be added to represent these conditions, as discussed below.

6.7.2 Horizontal Spring Constants Above Base

Additional spring constants acting on the vertical faces of the foundations may be computed using passive pressures and displacement estimates necessary to achieve to achieve the passive pressure limit. Displacements beyond the passive pressure threshold would not result in any increased resistance (i.e. bilinear stiffness relationship).

The potential occurrence of liquefaction adds another complicating factor into the analysis. First, because liquefaction does not develop upon the onset of ground shaking, full soil support would be provided for perhaps at least half the duration of the earthquake to as much as the full duration of the event. Accordingly, it may be appropriate to bracket the analyses using both fully supported conditions as well as conditions where the soil adjacent to the pile caps liquefies and exerts a lateral pressure on a portion of the pile cap because of lateral spreading.

The following presents equivalent fluid weights that may be used to represent the passive and active condition of the soil for the fully supported and liquefied conditions. The threshold displacement values represent the foundation movements that may be required to develop full active and passive pressures. The full force at the corresponding threshold displacement would define the spring constant, with displacements beyond the threshold value yielding no further increase in force (i.e. plastic state). The following summarizes our recommendations for lateral earth pressures:

Bascules

- Assume mudline exists at the top of the pile caps
- For full support conditions, determine passive resistance based on an equivalent fluid weight of 350 pcf with a threshold displacement of 4-inches
- For liquefied conditions, determine passive resistance neglecting any resistance from the top of the pile cap downward for a distance of 15 feet and then determining passive resistance based on an equivalent fluid weight of 350 pcf (threshold displacement of 2-inches)
- Use an equivalent fluid weight of 30 pcf to determine the incremental force from lateral spreading on the upland side of the pile cap.

Approach Foundations

- For full support conditions, determine passive resistance based on an equivalent fluid weight of 350 pcf with a threshold displacement of 2-inches
- For liquefied conditions, determine passive resistance neglecting any resistance from the top of the pile cap downward for a distance of 15 feet and then determining passive resistance based on an equivalent fluid weight of 350 pcf (threshold displacement of 1-inches)
- Use an equivalent fluid weight of 30 pcf to determine the incremental force from lateral spreading on the upland side of the pile cap.

6.8 SCOUR

The current channel survey data as well as previous measurements made by King County (1991, 1992, and 1998) all suggest that scour or aggradation are not significant factors for the performance of the bascules. However, scour or undermining may potentially affect Pier 2 North. King County should maintain a watch on this pier to confirm that the pier is not undermined.

6.9 NISQUALLY EARTHQUAKE

The South Park Bridge suffered some damage resulting from the February 28, 2001 Nisqually Earthquake. This event occurred following the submission of our engineering report for the bridge. However, the following discussion amends our original report to briefly discuss the effects of the Nisqually Earthquake.

The Nisqually Earthquake was a magnitude 6.8 event that occurred near the mouth of the Nisqually delta in the vicinity of Olympia, Washington. The earthquake originated within the subducting Juan de Fuca plate at a depth of about 33 miles. The earthquake was instrumentally recorded at a number of sites in the Puget Sound area, including the King County airport (i.e. Boeing Field) at a site that is less than 1 mile east of the South Park Bridge. This recording site (TKCO) is likely underlain by soil conditions similar to those underlying the South Park Bridge. Therefore, the ground motions recorded at the TKCO site may be a reasonable approximation of the ground motions experienced at the South Park Bridge. Horizontal ground accelerations of 0.17 and 0.25g were recorded at the TKCO site and the response spectra from the recorded motions are plotted on Figure 15. Essentially, the response spectra presented in Figure 15 show that the ground motions from the Nisqually Earthquake are intermediate to those estimated for the 190-year event and the current code level earthquakes (475-year event).

Liquefaction occurred at the site resulting from the ground shaking of the Nisqually Earthquake. Liquefaction occurred adjacent to pier 3E at the south approach and at adjacent property at the south approach. While the potential occurrence of liquefaction was suggested in our liquefaction evaluations of the site, it appears that the development of liquefaction did not significantly affect the performance of the bridge.

In general, lateral spreading at the bridge site would appear to be limited to relatively small movements or localized occurrences. Specifically, minor movements (i.e. less than 2 inches) occurred at a boat access ramp adjacent to the south approach. Similarly, it would appear that the soils underlying the south approach may have shifted to the north (towards the river) by about one inch as reflected in cracks observed in nearby paved areas and closure difficulties of the bascules (i.e. convergence of the leaves of the

bascules). This observation is in reasonable agreement with the lateral spreading analyses that predicted ground displacements of about 6-inches.

The Nisqually Earthquake also resulted in some ground settlement in the approach fill at the south abutment and possibly in the abutment of the south approach fill. Localized settlement of the fill in the roadway may be on the order of 1 to 2 inches.

In general, the pile supported structures (i.e. bascules and approach structures) did not appear to experience any adverse performance from the foundations (i.e. no apparent settlement or loss of support). This observation further confirms our conclusions on the adequacy of the piles supporting the bridge. Most of the damage experience by the bridge appears to be related to pounding between elements with different supports, such as pounding at railing where the south approach fill connects with the concrete spans of the approach structure. Also, some distress or settlement was observed at the south abutment. Finally, incipient plastic hinging was observed in the columns of the south approach. The bridge essentially remained operational following the earthquake.

7.0 LIMITATIONS

Recommendations contained in this report are based on a site reconnaissance, review of existing bridge drawings and construction records, review of nearby borings, and our understanding of the proposed project. The investigation was performed using a mutually agreed-upon scope of work. It is our opinion that the study was a cost effective method to evaluate the subject site, and evaluate some of the potential geotechnical concerns. The scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

This report has been prepared for planning and preliminary design purposes for specific application to the seismic evaluation of the South Park Bridge in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Advances in our understanding of applied science, particularly the earthquake hazards in the region, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 much from its issuance. PanGEO should be notified if the project is delay by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

Any party other than the client who wishes to use this report shall notify PanGEO such intended use and for permission to copy this report. Based on the intended use of the

report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

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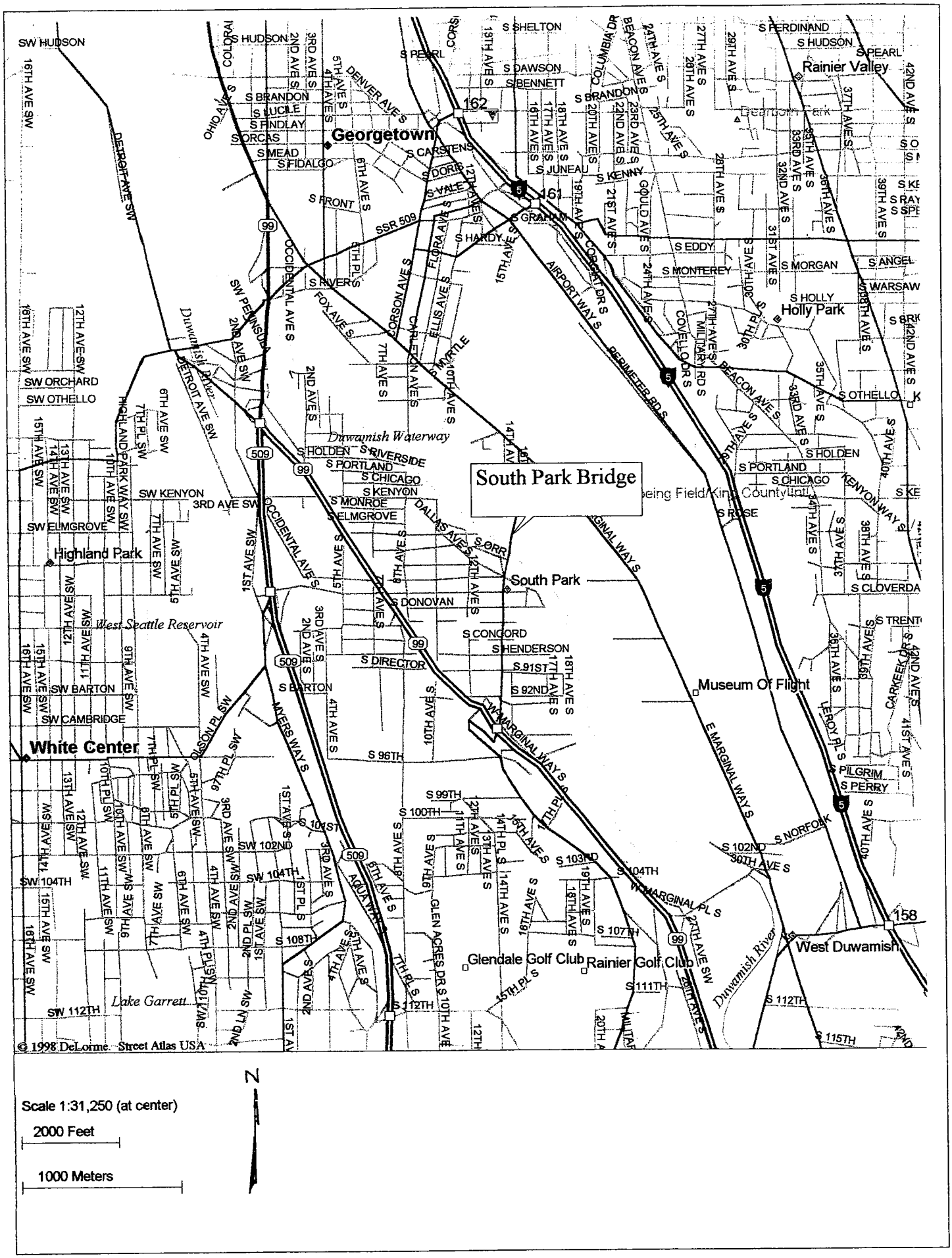


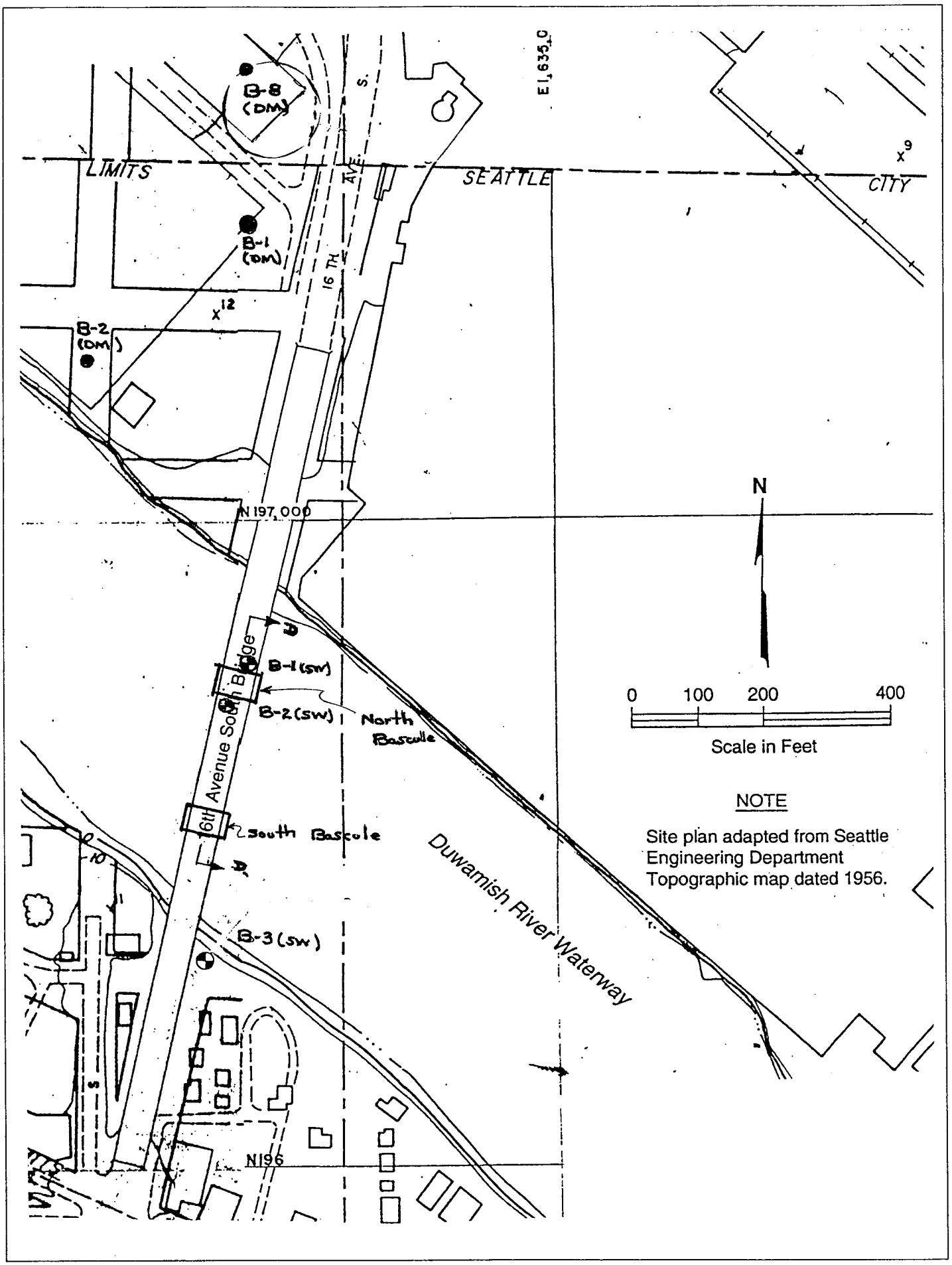
PanGEO

South Park Bridge
Seismic Evaluation

Vicinity Map

Project No. 00-038

Figure 1

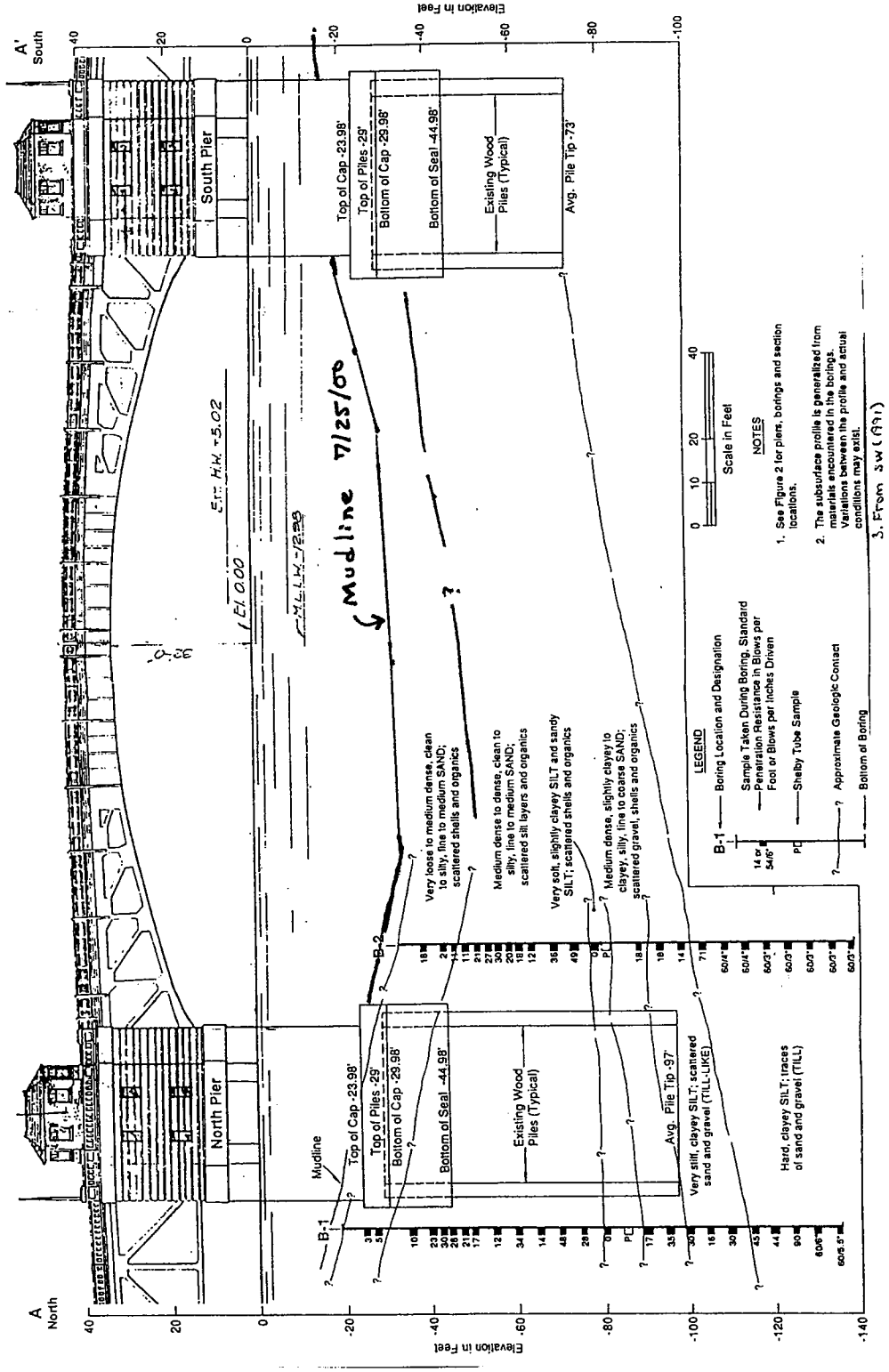


PanGEO INCORPORATED

South Park Bridge
Seismic Evaluation

Site and Exploration Plan

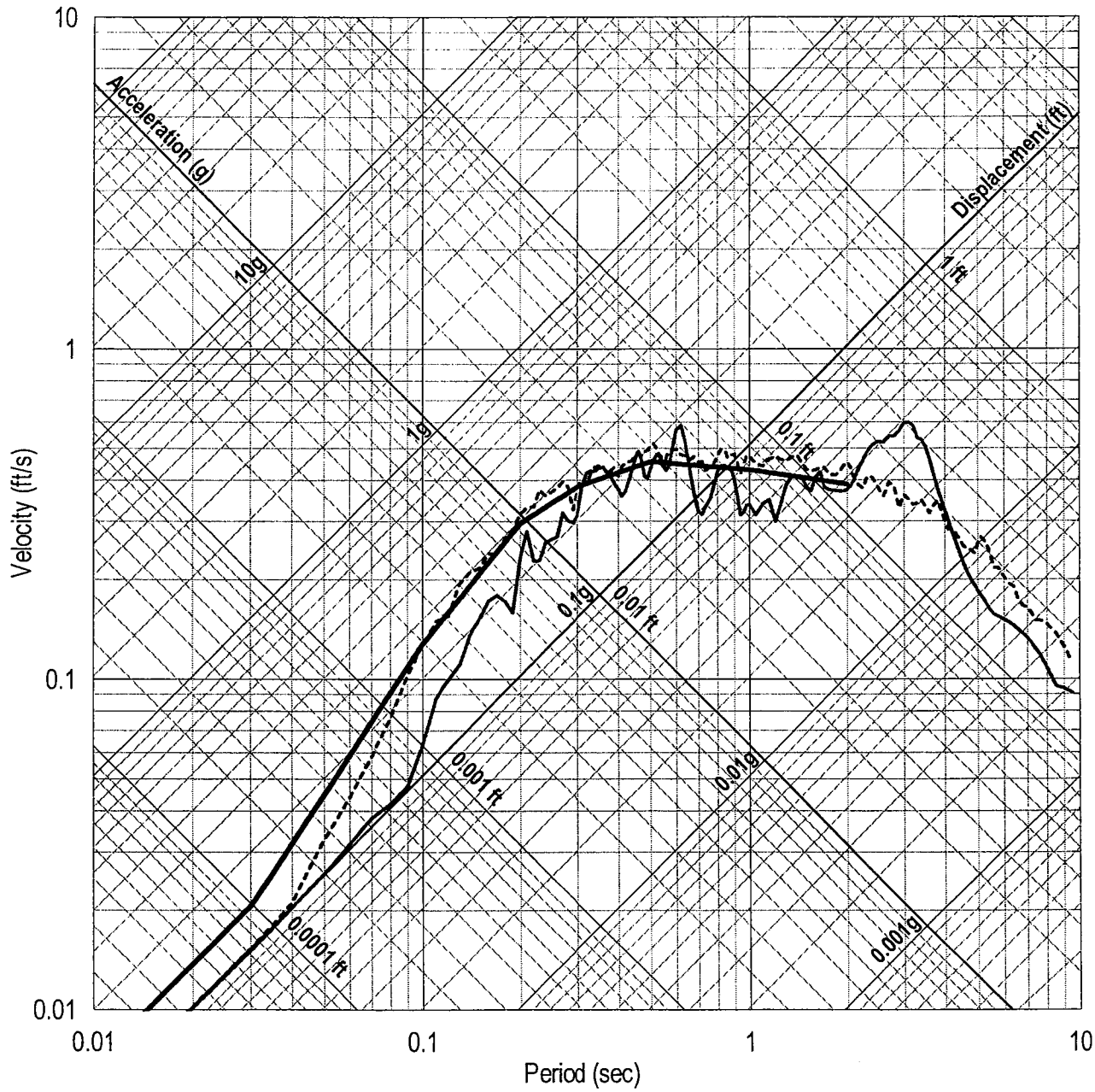
Project No. 00-038 Figure 2



PanGEO

South Park Bridge
Seismic Evaluation

Subsurface Profile A-A'

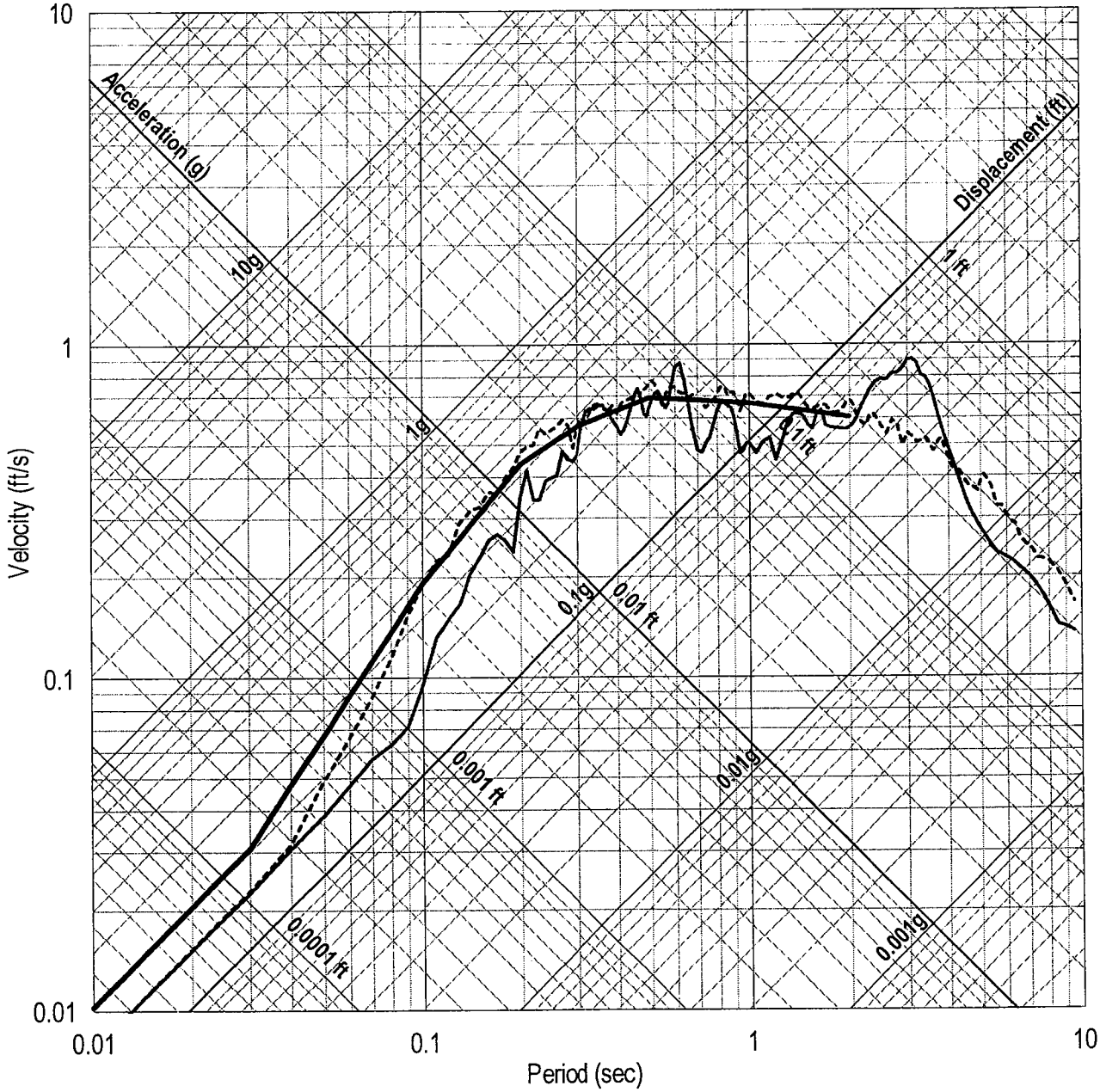


Legend
- Olympia South
- Olympia West
- Equal Hazard Spectrum

Notes:
1. Spectra correspond to horizontal motions at 5% damping at bedrock outcrop.
2. Olympia West is the 1949 W. natural record scaled only for acceleration.
3. Olympia South is the 1949 S. record adjusted to match the USGS Equal Hazard Spectrum.

PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Bedrock Spectra 95-year Event	
		Project No. 00-038	Figure No. 4

Bedrock RSP @95yr.grf w/ TSaccel.dat, TSdispl.dat,Spectral Bedrock RSP @95yr.xls 8/15/01 (17:04) TEA2

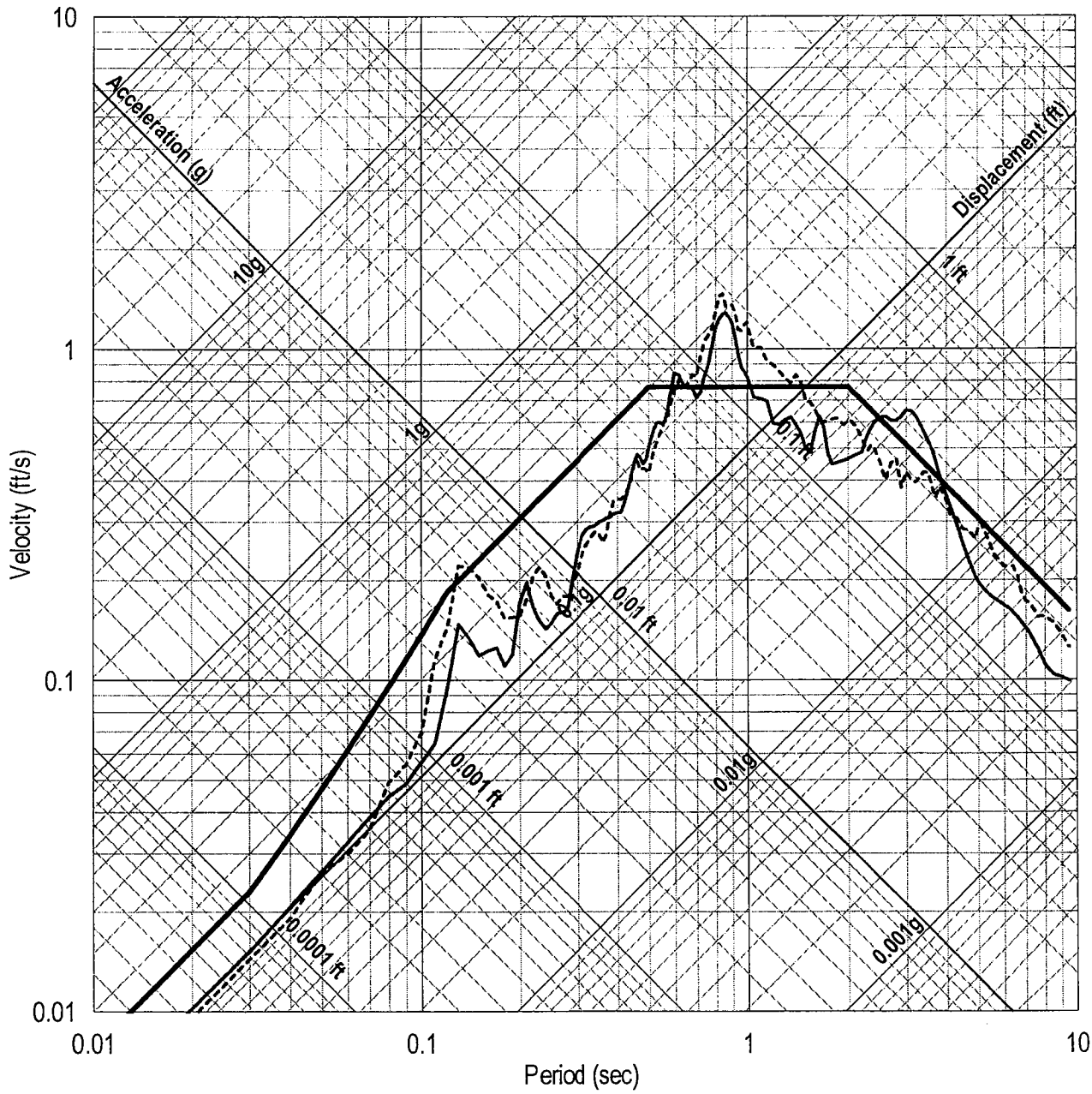


Legend
- Olympia South
- Olympia West
- Equal Hazard Spectrum

Notes:
1. Spectra correspond to horizontal motions at 5% damping at bedrock outcrop.
2. Olympia West is the 1949 W. natural record scaled only for acceleration.
3. Olympia South is the 1949 S. record adjusted to match the USGS Equal Hazard Spectrum.

PanGEO INCORPORATED	**South Park Bridge Seismic Evaluation**	**Bedrock Spectra 190-year Event**
		Project No. 00-038 / Figure No. 5

Bedrock RSP @190yr.grf w/ TSaccel.dat, TSdispl.dat,Spectral Bedrock RSP @190yr.xls 8/15/01 (17:15) TEA2

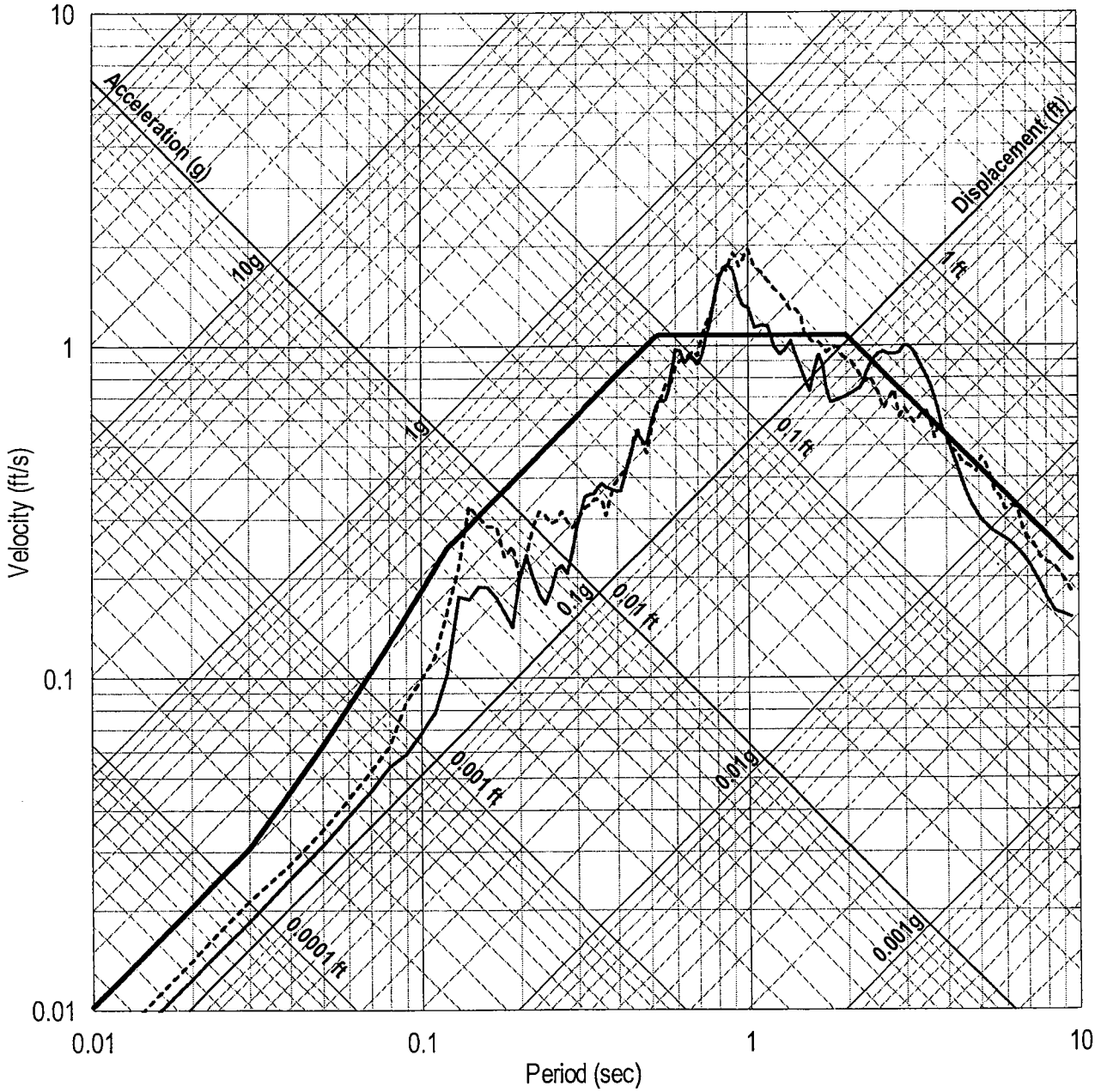


Legend
- Olympia South
- Olympia West
- Recommended Spectrum

Notes:
1. Spectra correspond to free field, horizontal motions at the base of concrete seal at 5% damping.
2. Vertical motions correspond to 2/3 of the horizontal values.

PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Base of Seal Spectra 95-year Event	
		Project No. 00-038	Figure No. 6

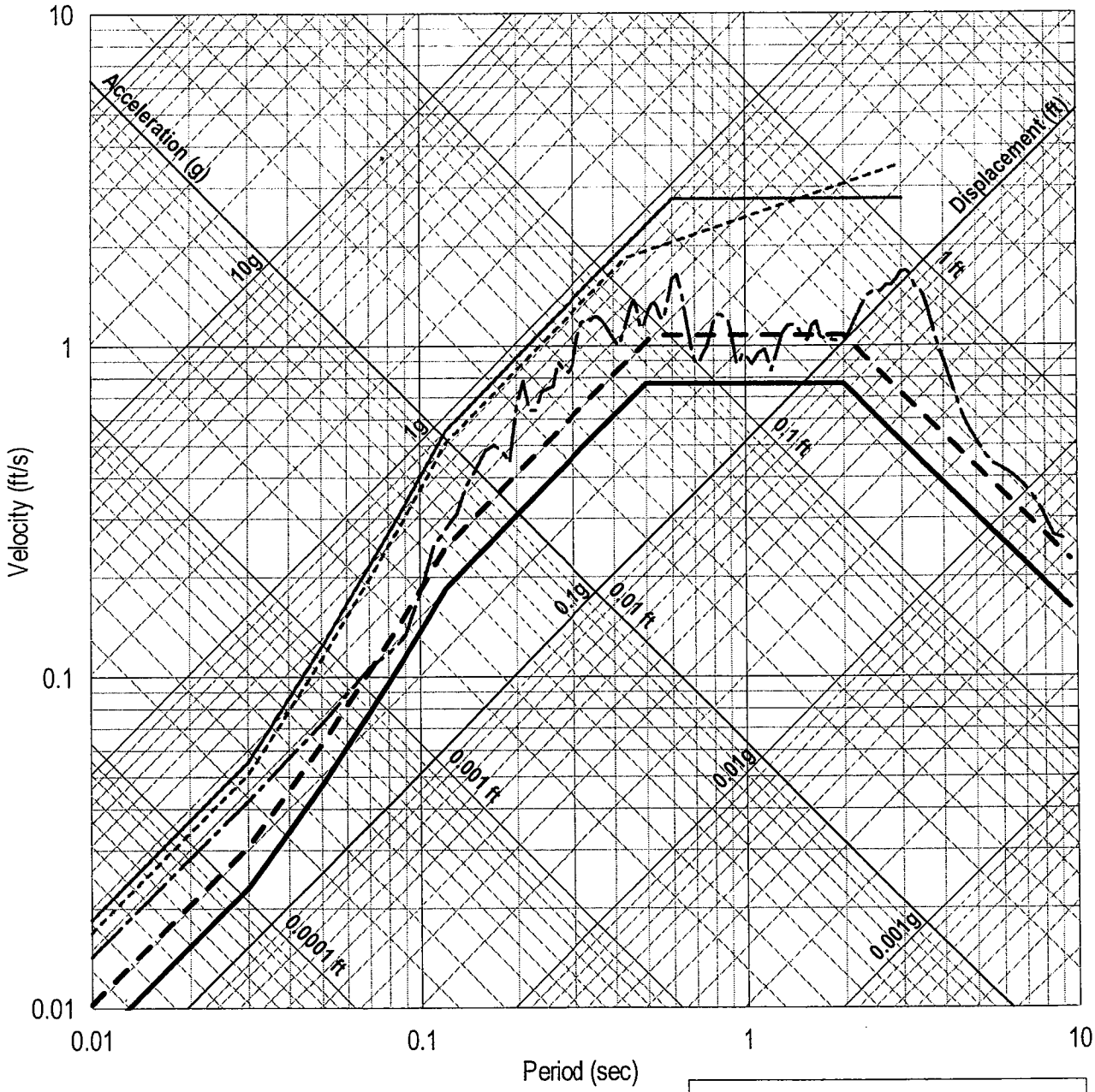
Base Seal RSP @95 YR.grf w/ TSaccel.dat, TSdispl.dat,Base Seal RSP @95 YR.xls 8/15/01 (17:19) TEA2



Notes:
1. Spectra correspond to free field, horizontal motions at the base of concrete seal at 5% damping.
2. Vertical motions correspond to 2/3 of the horizontal values.

PanGEO INCORPORATED	**South Park Bridge Seismic Evaluation**	**Base of Seal Spectra 190-year Event**
		Project No. 00-038 / Figure No. 7

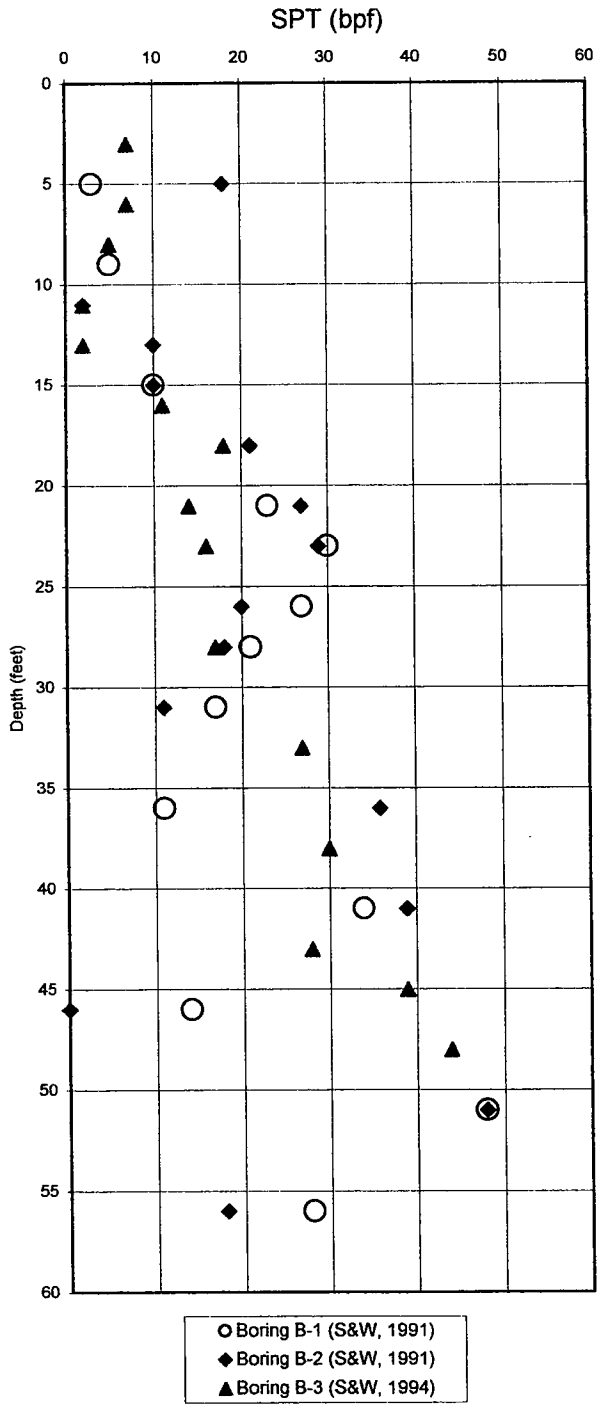
Base Seal RSP @190 YR.grf w/ TSaccel.dat, TSdispl.dat,Base Seal RSP @190 YR.xls 8/15/01 (17:21) TEA2

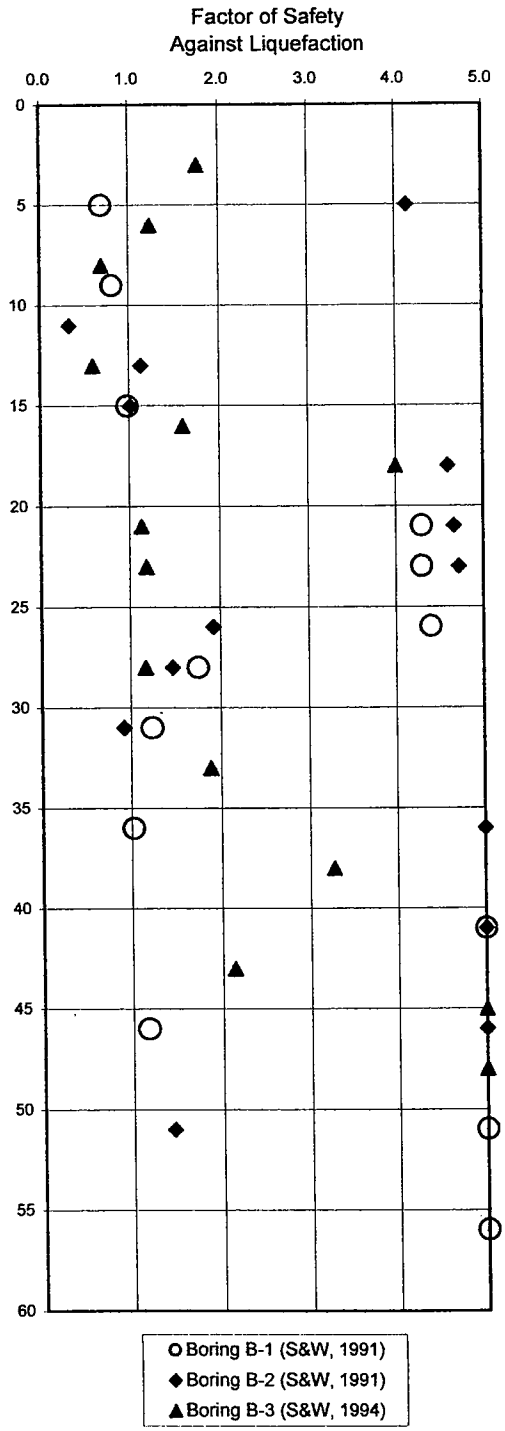


Notes:
1. Spectra correspond to free field motions at the foundation level for 5% damping.
2. Vertical motions correspond to 2/3 of the horizontal values.

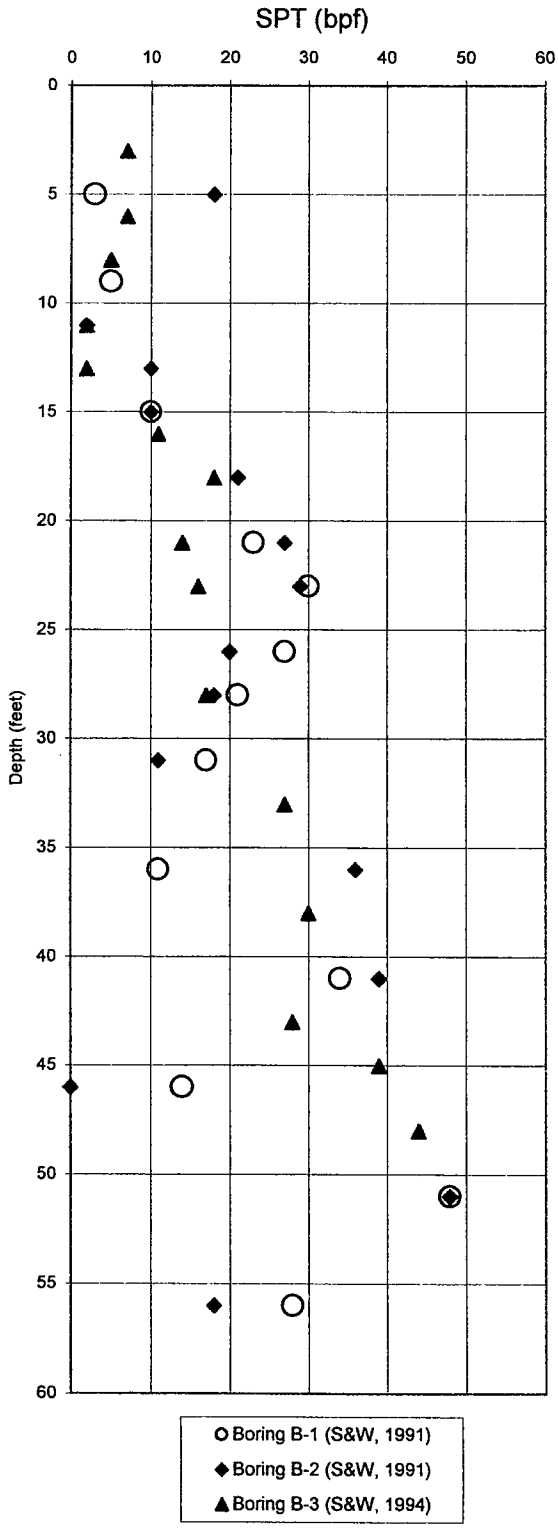
PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Response Spectra Comparison	
		Project No. 00-038	Figure No. 8

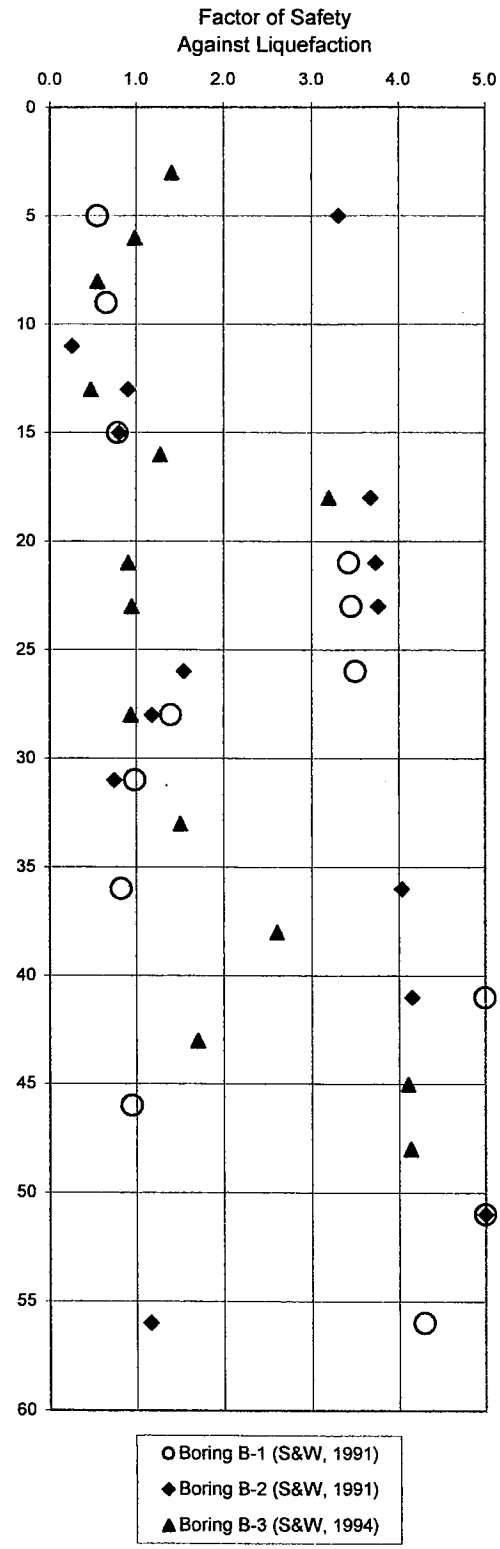
Spectral Comparison.grf w/ TSaccel.dat, TSdispl.dat,Spectral Comparison.xls 8/15/01 (17:39) TEA2



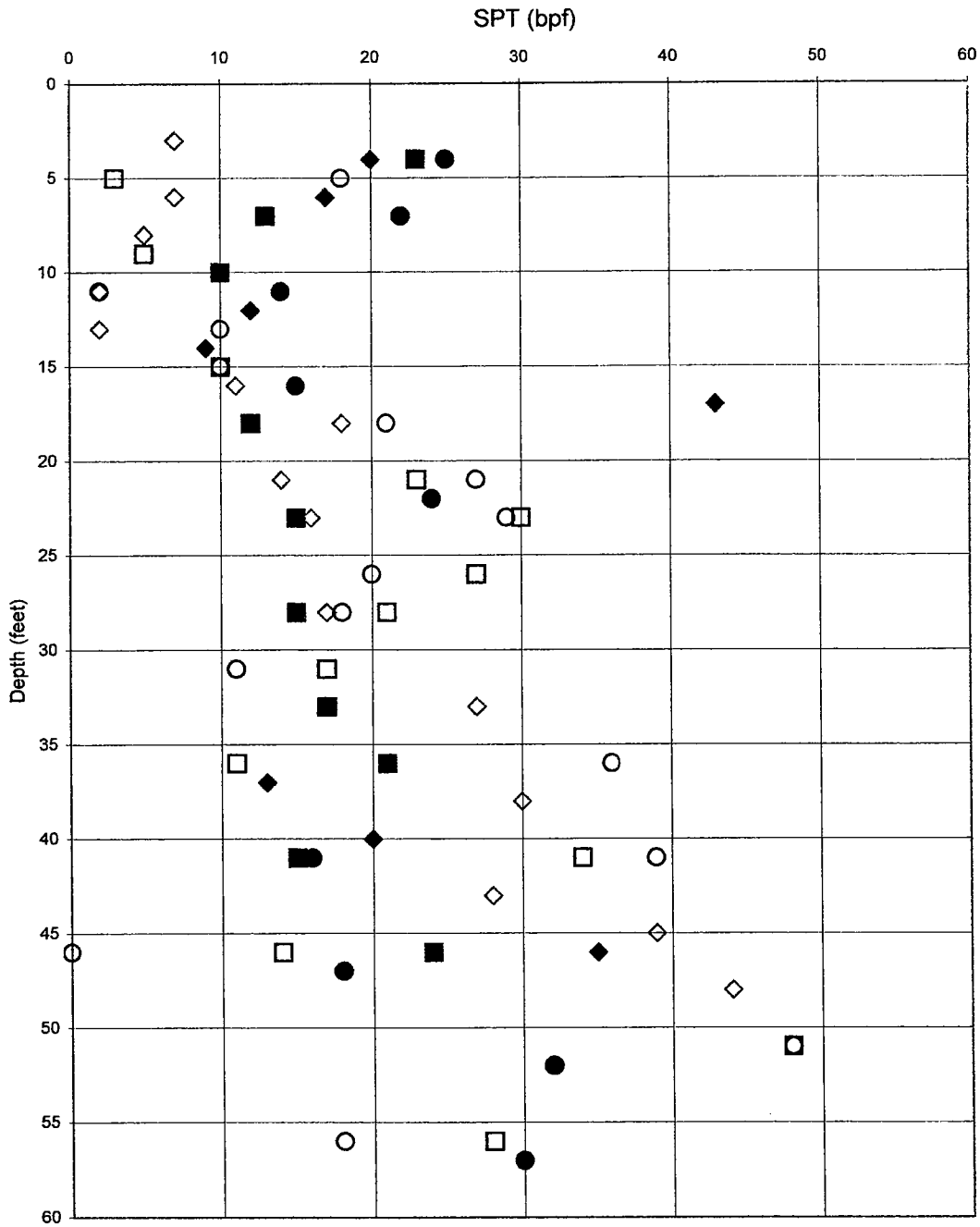


PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Liquefaction Analysis for 95-Year Event M=7.5 & PGA=0.16g	
		Project No. 00-038	Figure No. 9





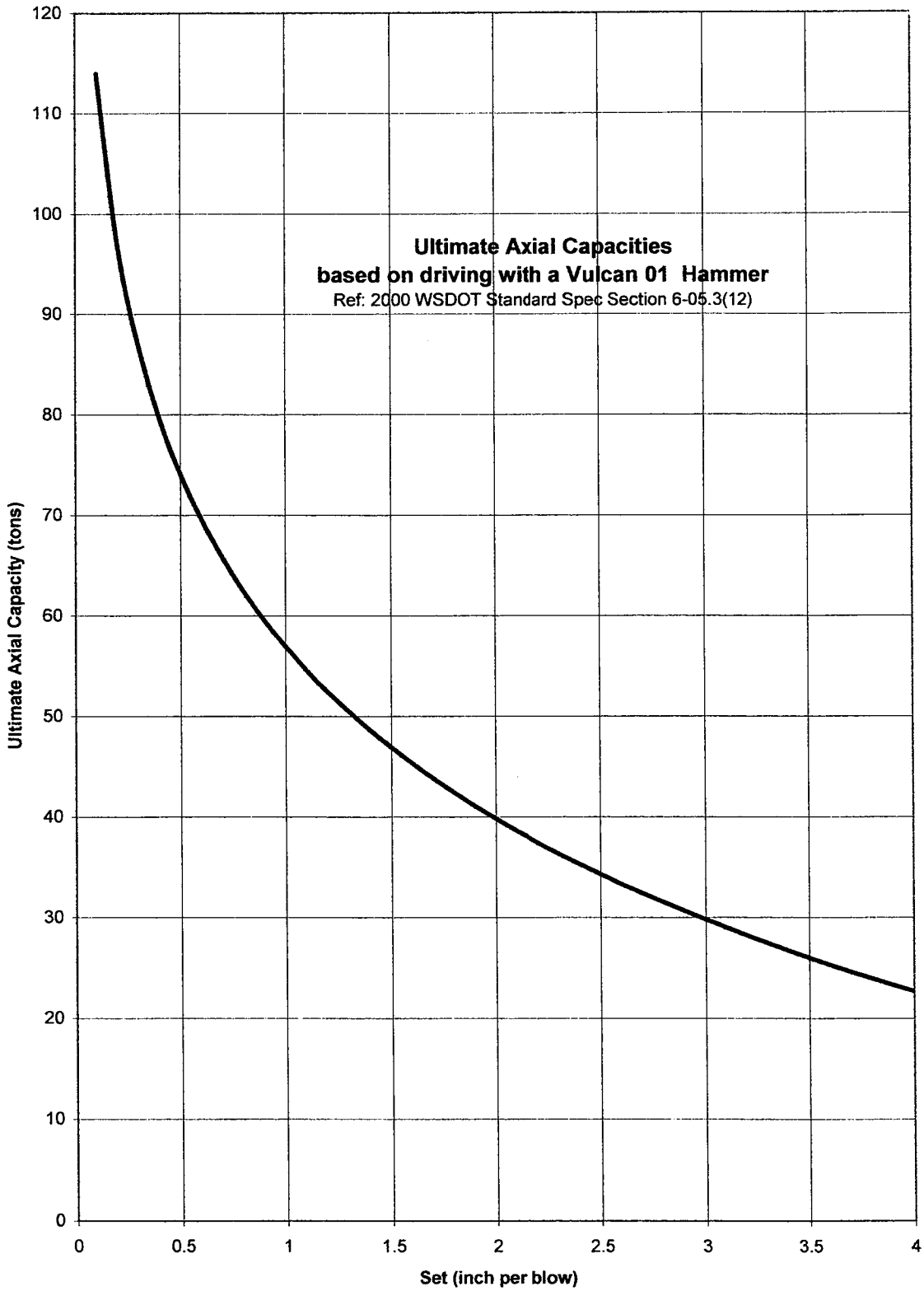
PanGEO INCORPORATED	**South Park Bridge Seismic Evaluation**	**Liquefaction Analysis for 190-Year Event M=7.5 & PGA=0.2g**	
		Project No. **00-038**	Figure No. **10**



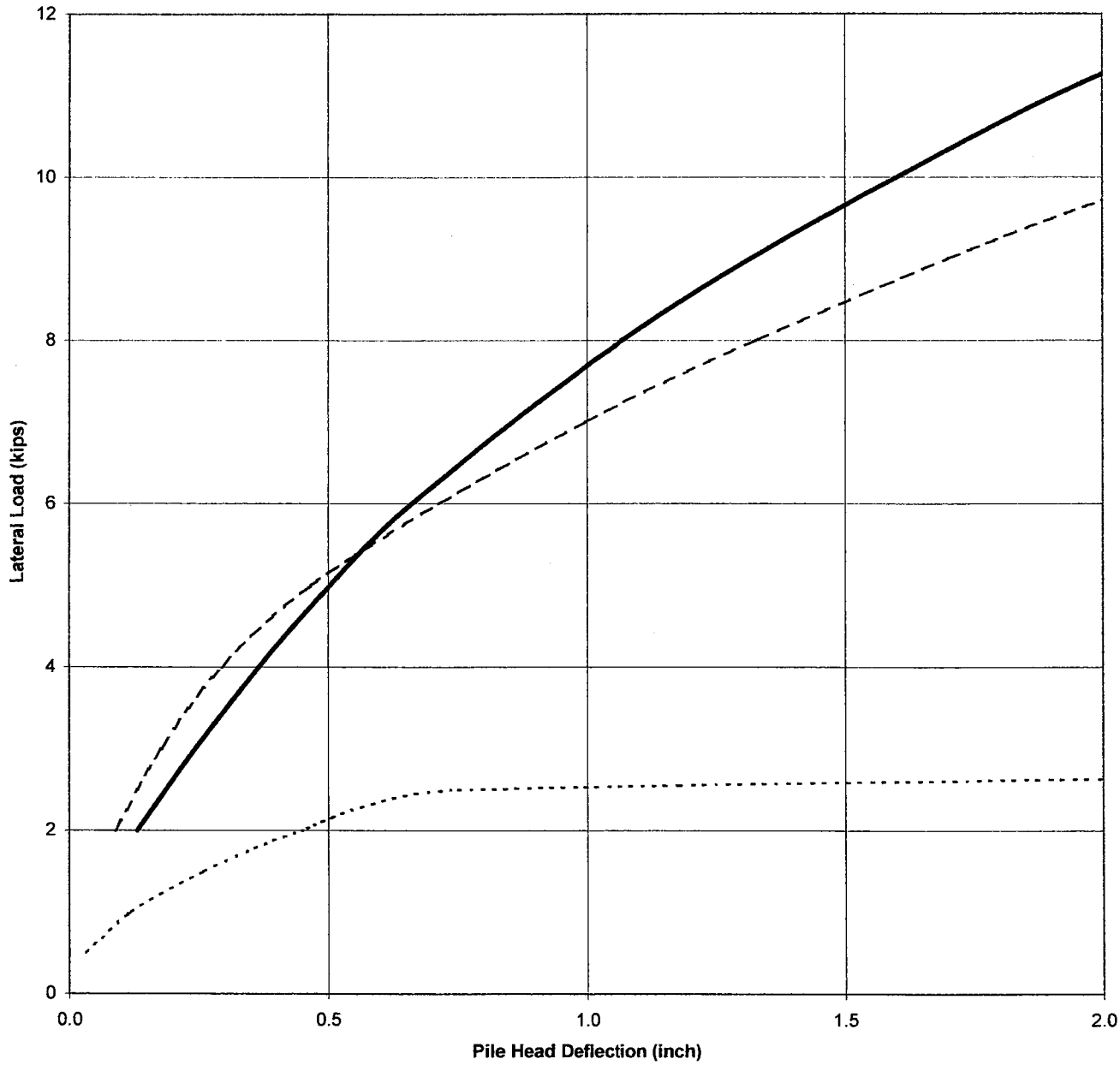
Note:
DM N-Values are raw data and not SPT procedures.

- ■ Boring 1 (DM)
- ● Boring 2 (DM)
- ◆ Boring 8 (DM)
- □ Boring B-1 (S&W, 1991)
- ○ Boring B-2 (S&W, 1991)
- ◇ Boring B-3 (S&W, 1994)

PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Summary of SPT N-Values	
		Project No. 00-038	Figure No 11



PanGEO INCORPORATED	**South Park Bridge Seismic Evaluation**	**Ultimate Pile Capacities**
		Project No. **00-038** / Figure No. **12**



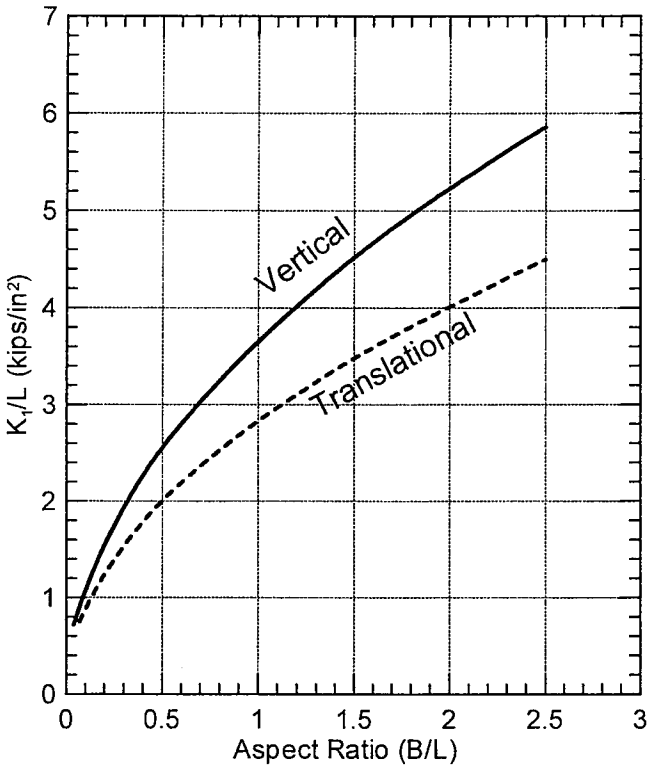
North & South Bascules and Pier 1 South - Fixed Head Condition

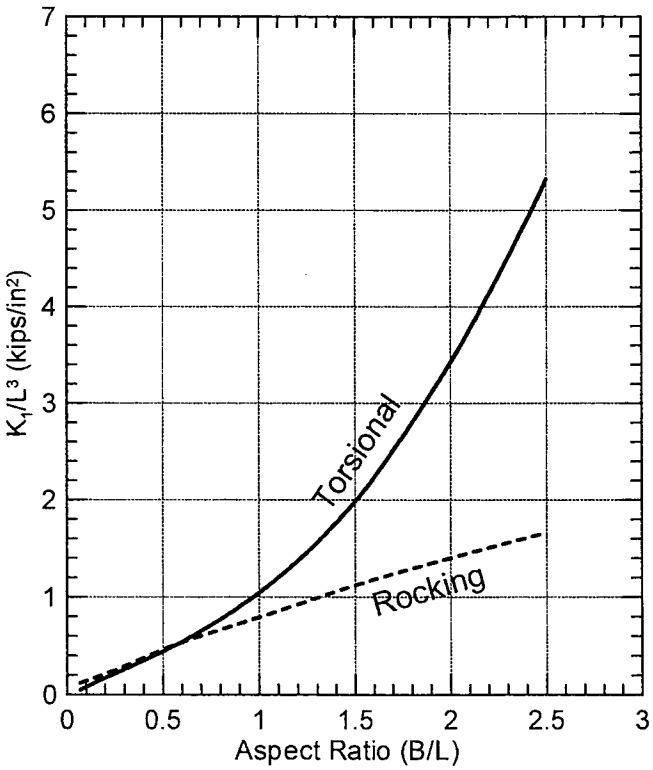
Pier 1 North - Fixed Head Condition

Pier 2 North and South - Free Head Condition

Note: Results shown above include pile group effects.

PanGEO INCORPORATED	**South Park Bridge Seismic Evaluation**	**Lateral Pile Capacity Single Timber Pile**
		Project No. **00-038** / Figure No. **13**





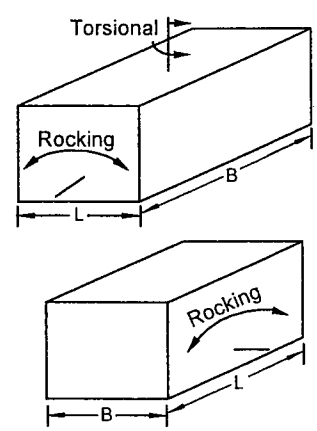
EXPLANATION

The charts on this figure are used as follows to calcuate soil spring constants for use in the dynamic structural analyses:

1. Calculate Aspect Ratio (B/L) for the footing.
2. Use appropriate chart to estimate K_1/L (or K_1/L^3). Note that the charts on this figure were developed based on a shear modulus of 1 ksi and a Poisson's ratio of 0.4.
3. Calcualte K_1, the soil spring constant for a shear modulus of 1 ksi as follows:
   $K_1 = (K_1/L)xL$ or $K_1 = (K_1/L^3)xL^3$
   where 'L' is measured ininches
4. Calcualte K_1, the recommended soil spring constant as follows:
   $K = K_1x(G/1ksi)$
   where 'G' is the recommended shear modulus. Recommended shear moduli to be used in calcualtions are provided in text.

NOTE

For rocking, B is measured parallel to axis of rocking and L is measured in plane of rocking.



Spring Consaint.grf w/ [00-006-100-2.jpg] 8/15/01(17:50) TEA2

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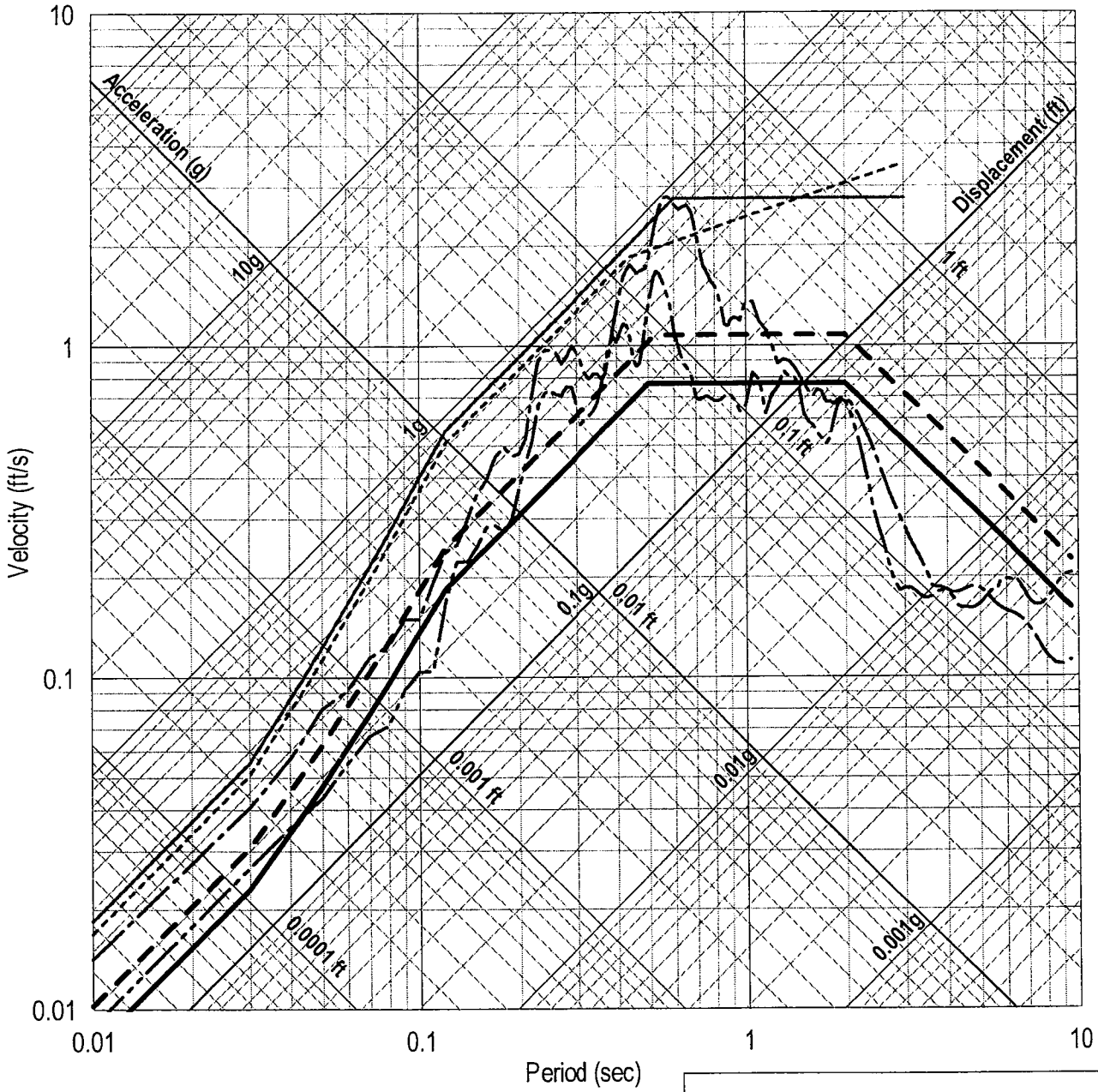
South Park Bridge Seismic Evaluation

SOIL SPRING CONSTANTS

Project No.	Figure No.
00-038	14



Legend
- 95 Years
- 190 Years
- Nisqually - TKCO NS
- Nisqually - TKCO EW
- 97 UBC Zone 3 S_D
- AASHTO Type II

Notes:
1. Spectra for the 95 and 190 year events correspond to free field motions at the foundation level for 5% damping.
2. AASHTO, UBC, and Nisqually spectra correspond to ground surface motions at 5% damping.

PanGEO INCORPORATED	South Park Bridge Seismic Evaluation	Response Spectra Comparison Nisqually Earthquake	
		Project No. 00-038	Figure No. 15

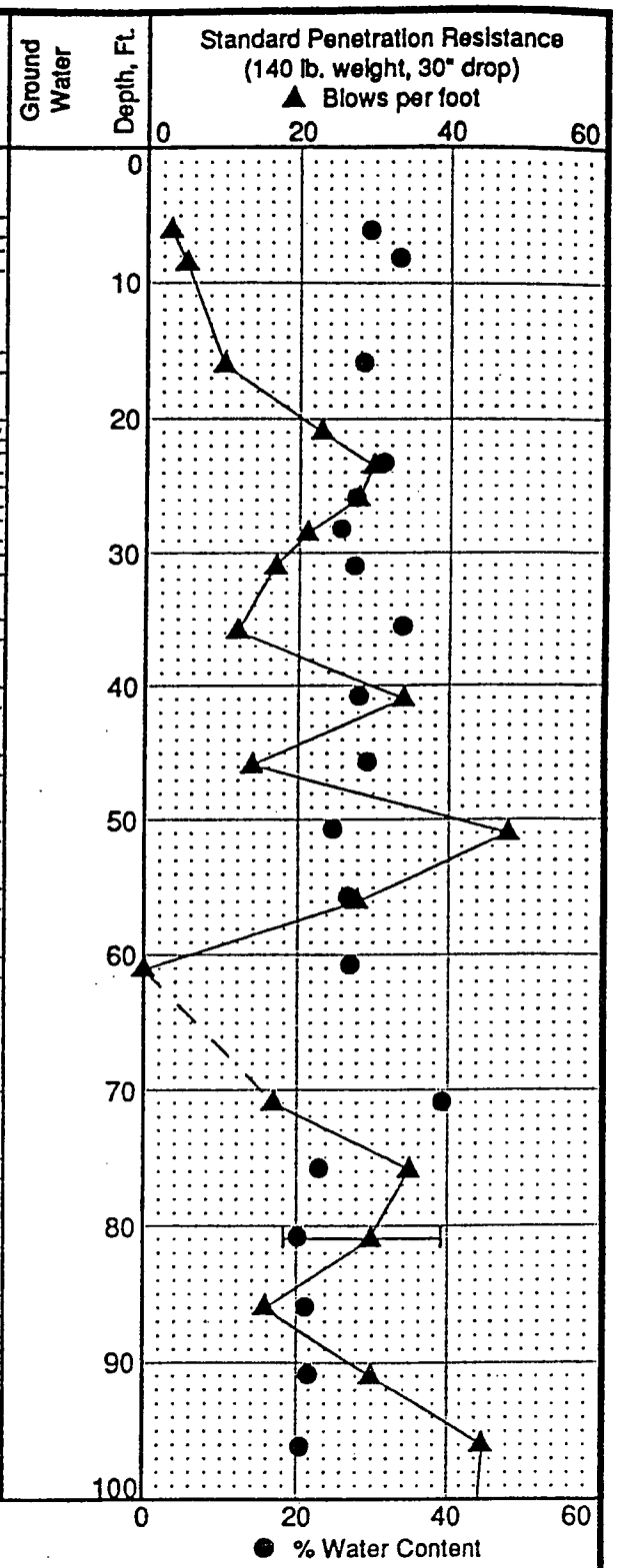
Nisqually Comparison.grf w/ TSaccel.dat, TSdispl.dat,Spectral Comparison.xls, Nisqually.xls 8/15/01 (17:40) TEA2

APPENDIX A

EXISTING EXPLORATIONS

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples
Very loose to loose, dark gray, slightly silty to silty, fine to medium SAND; wet with some gravel (Organic smell at 7.5')		1, 2
Medium dense, dark gray, clean to silty, fine to medium SAND; wet with zones of silt	10	3, 4*, 5, 6, 7, 8
Medium dense to dense, dark gray, slightly silty to silty, fine SAND; moist to wet with layers and zones of silt	35	9, 10, 11, 12, 13
Very soft, gray, slightly clayey, fine to medium sandy SILT; moist with shells and scattered gravel	60	14, 15
Medium dense, gray, slightly clayey to clayey, silty fine to coarse SAND; moist, with shells and scattered gravel	70	16, 17
Very Stiff, gray-brown, clayey SILT; moist with scattered coarse sand and a trace of gravel (TILL-LIKE)	80	18, 19
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	90	20, 21

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

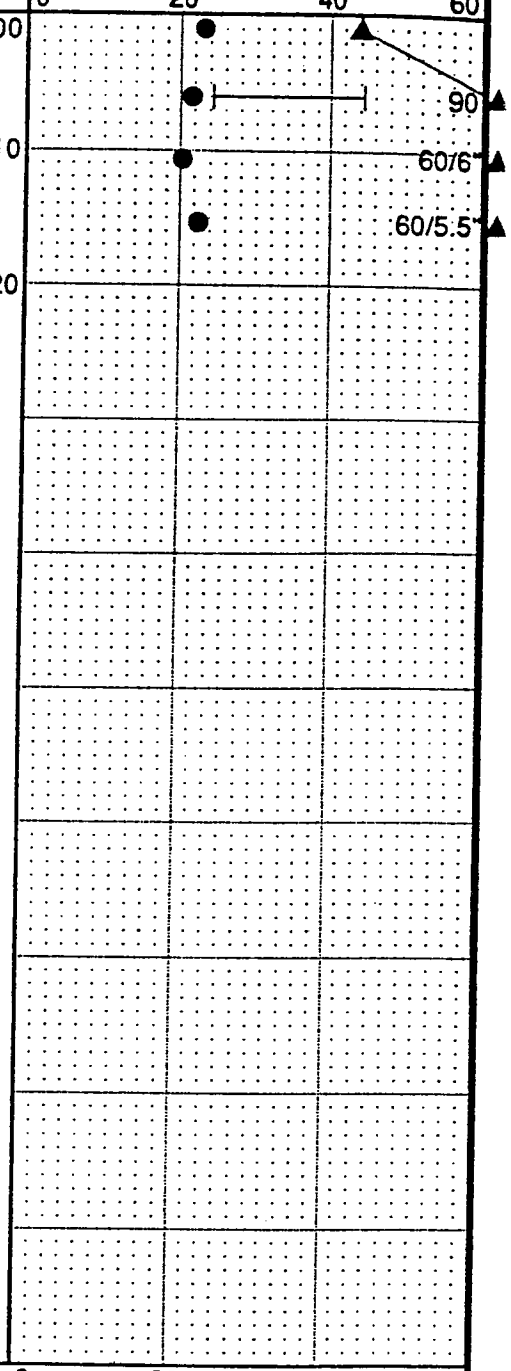
LOG OF BORING B-1

February 1991 W-5749-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-1a

SOIL DESCRIPTION Mud Line Elevation: Approx. -19 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water	Depth, Ft.	Standard Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot
Hard, gray, clayey SILT; with a trace of coarse sand and gravel (TILL)	100	22		100	
		23			90
		24		110	60/6"
	116	25			60/5.5"
BOTTOM OF BORING COMPLETED 1-27-91				120	



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-1

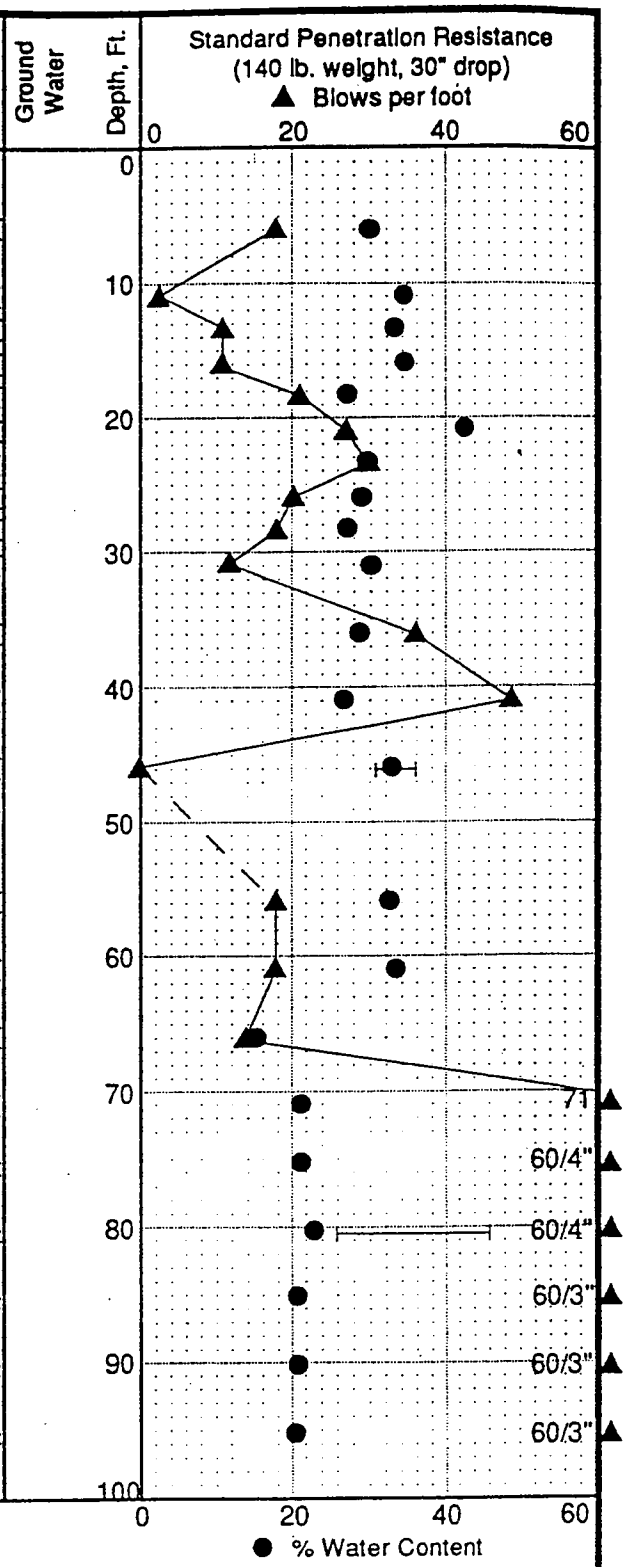
February 1991 W-5749-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-1b

SOIL DESCRIPTION Mudline Elevation: Approx. -33 Feet (See Fig.3)	Depth, Ft.	Samples
Very loose to medium dense, dark gray, clean to silty, fine to medium SAND; wet with some shells and organics (Organic smell)		S-1, S-2, S-3, S-4
Medium dense to dense, dark gray, slightly silty to silty, fine to medium SAND; wet with scattered organics and layers and zones of silt	15	S-5, S-6, S-7, S-8, S-9, S-10, S-11, S-12
Very soft to soft, gray-brown, silty CLAY; moist to wet with a trace of sand and organics	45	S-13, S-14
Medium dense, gray, slightly clayey, silty, fine to coarse SAND; moist to wet with gravel, organics and shells	50	S-15
Very stiff, light brown and gray, clayey SILT; moist with laminated layers of: silt, sand and gravel	58	S-16
Stiff, gray-brown, gravelly, fine to coarse sandy, clayey SILT; moist (Till-Like)	63	S-17
Hard, gray, clayey SILT; dry with a trace of sand (Till)	68	S-18, S-19, S-20, S-21, S-22, S-23

(Boring log continued)



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue Bridge
Seattle, Washington

LOG OF BORING B-2

February 1991 W-5749-01

SHANNON & WILSON, INC.
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FIG. 2a

SOIL DESCRIPTION Mud Line Elevation: Approx. -33 Feet (See Fig. 3)	Depth, Ft.	Samples	Ground Water Depth, Ft.	Standard Penetration Resistance (140 lb. weight, 30" drop) ▲ Blows per foot (0 20 40 60)
Hard, gray, clayey SILT; dry with a trace of sand (TILL)	100	24	100	60/3" ▲
	105.7	25		60/3" ▲
BOTTOM OF BORING COMPLETED 2-10-91			110	

0 20 40 60
● % Water Content

LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-2

February 1991 W-5749-01

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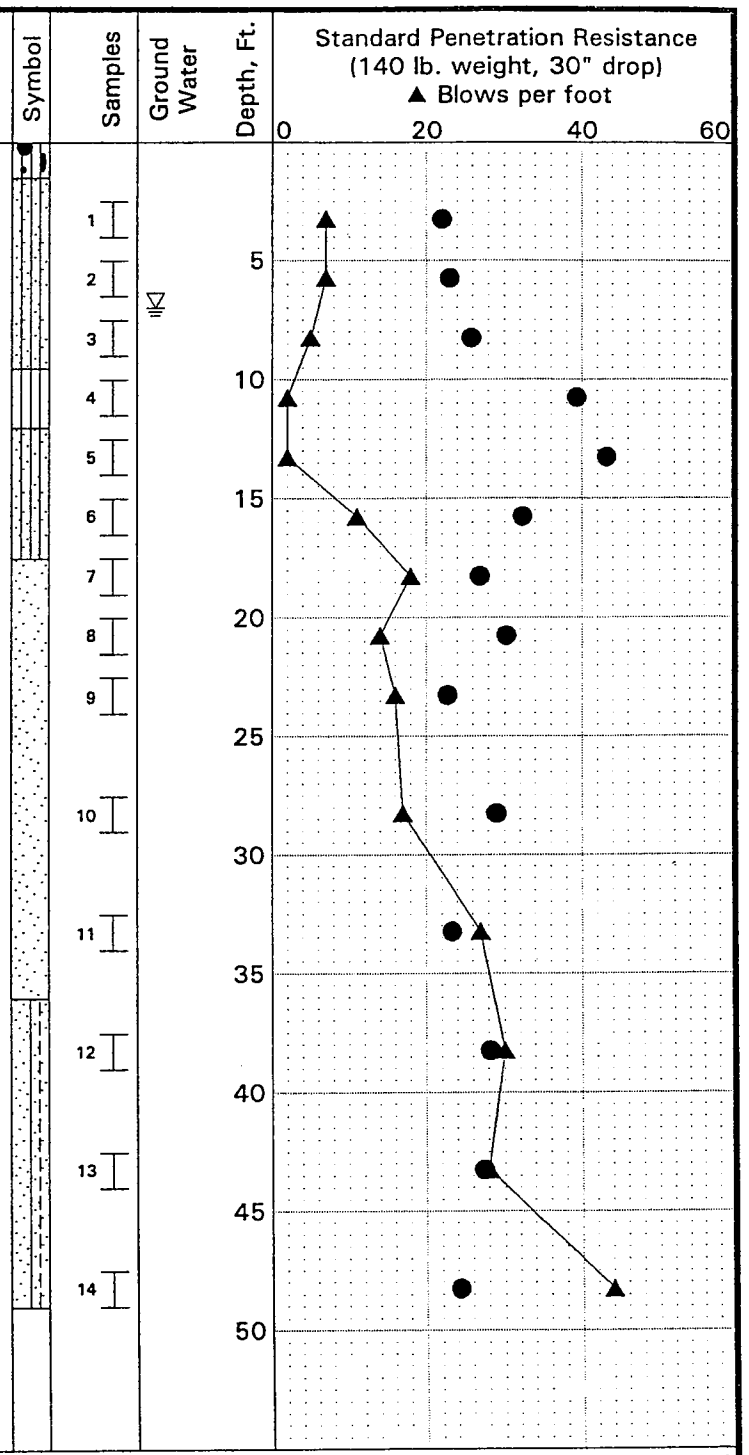
FIG. A-2b

MASTERLG 8/25/94

MATERIAL DESCRIPTION Surface Elevation: Approx. 7 Feet	Depth, Ft.
Loose, brown, silty, sandy GRAVEL; moist; (Fill) GM.	1.5
Loose, brown, slightly silty to silty, fine SAND; moist to wet below 7 feet; (Alluvium) SM.	9.5
Very soft, gray-brown SILT, trace of clay and trace of fine sand; wet; (Alluvium) ML.	12.0
Very loose to medium dense, gray-brown, silty, fine SAND; wet; scattered iron-stained zones; (Alluvium) SM.	17.5
Medium dense, dark gray, clean to slightly silty, fine to medium SAND; wet; (Alluvium) SP.	36.0
Medium dense to dense, dark gray, slightly silty, fine SAND; wet; (Alluvium) SP-SM.	49.0
BOTTOM OF BORING COMPLETED 7/8/94	



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

16th Avenue South Bridge
Seattle, Washington

LOG OF BORING B-3

July 1994 W-5749-02

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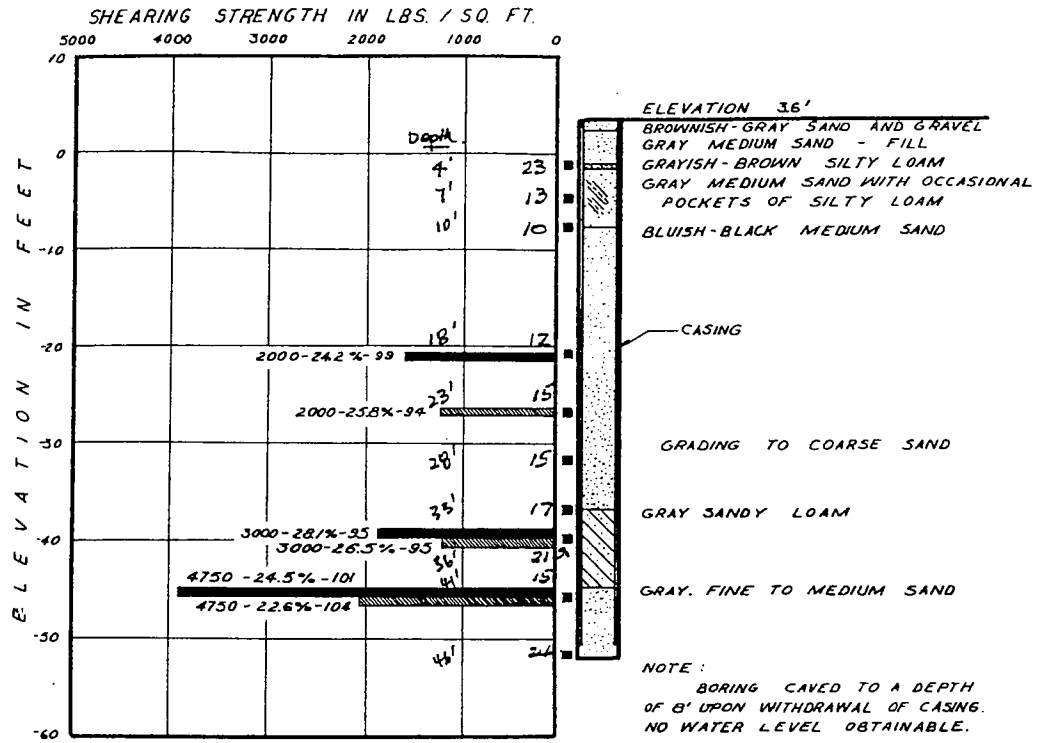
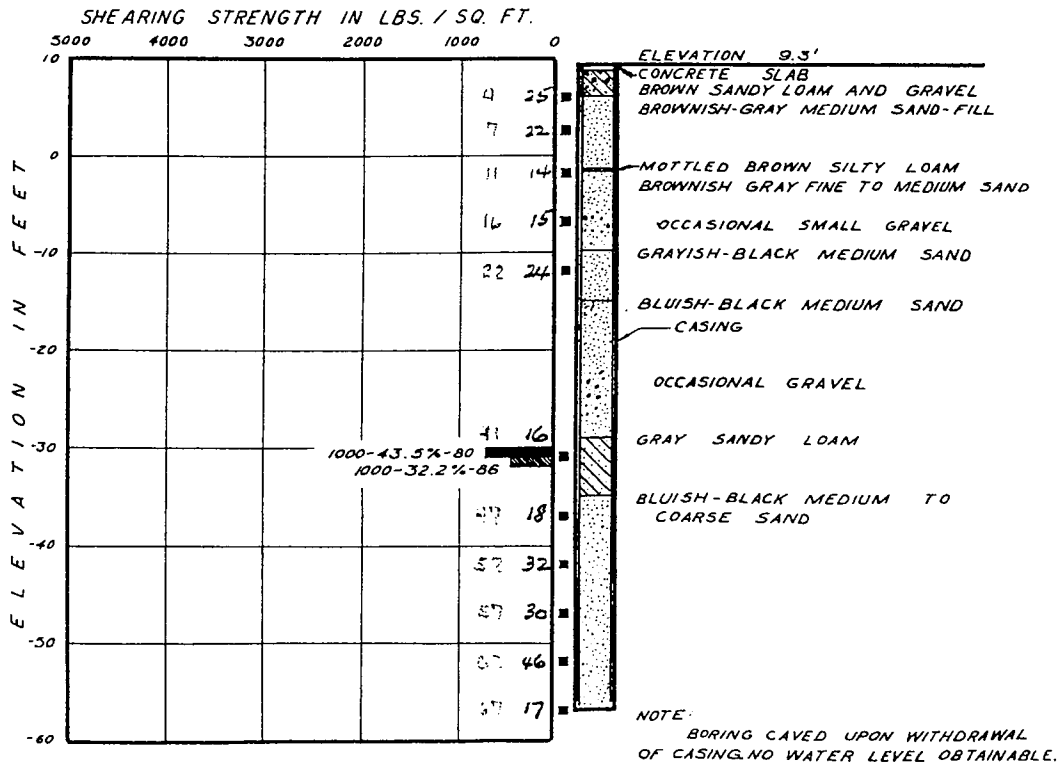
FIG. A-2

BORING 1



BORING 2

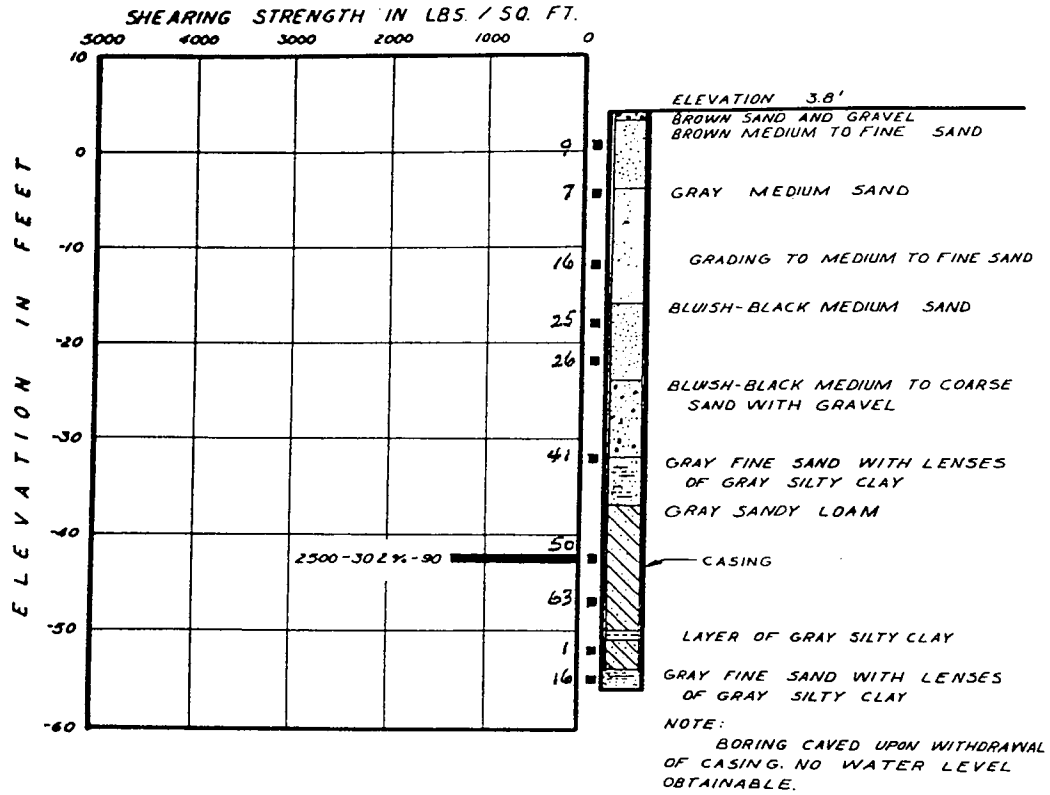


LOG OF BORINGS

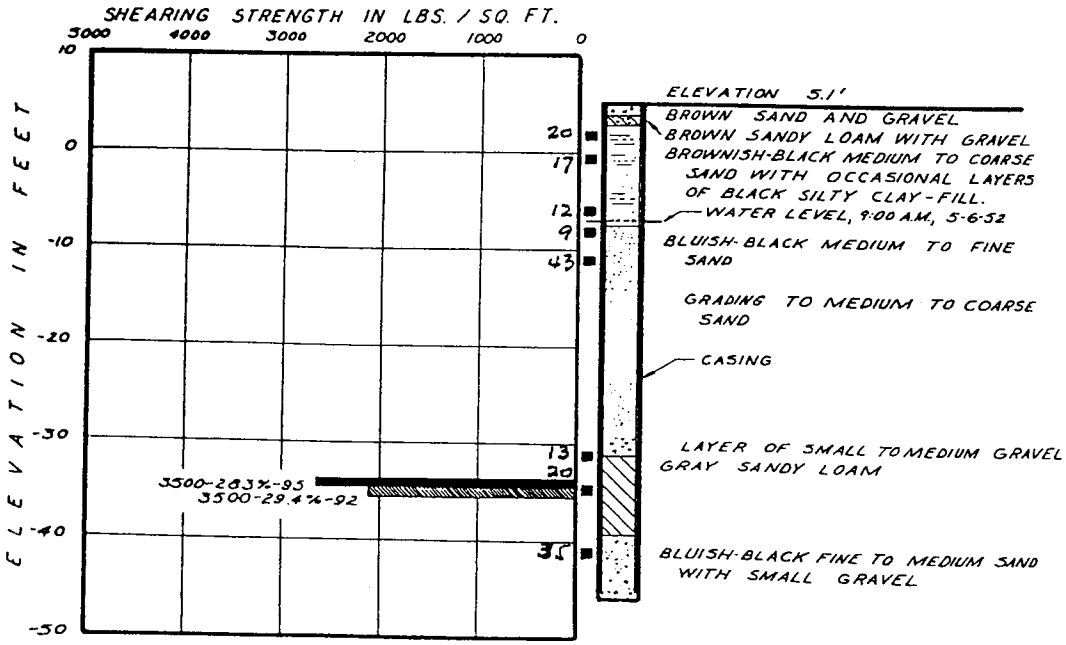
NOTE: Figures in red are blows per foot

NOTE:

BORING 7



BORING 8



LOG OF BORINGS

NOTE:

Appendix D
Field Investigation by PanGEO

Transportation Structures

Event Name/Date: Nisqually / 2-28-01 **Name of investigator:** Paul Grant 206-406-4036

Short description of observation: Bending @ approaches / slight Plastic Hinging **Date of observation:** 2-28-01 & 3-2-01 & 3-5-01

I. Location: *(please be as detailed as possible):*

a. Descriptive: South Park Bridge (14th/16th Ave. So) over Duwamish

b. Street: 14th/16th Ave So.

Address Number	Direction (N, S, E, W)	Street Name	Suffix (Rd, St, Ave)
Cross Street (if available):		16th Ave. So. & S. Orr street	

c. City: Seattle / Tukwila **d. County:** King **e. Zip:**

f. Map Reference (Quad, etc.): **g. Latitude:** N47° 31.770' **h. Longitude:** W122° 18.860'

i. Thomas Bros. Page: **Grid:** **j. Station ID:**

II. Bridge Information:

Bridge ID: Route/Interchange:

Length: 1285' Width: 60' When Built: 1931

Superstructure: 190' (clear span) Rolling leave Bascule flanked on both sides by a steel deck truss approach and 12 concrete slab approach spans

Substructure: Bascules & approaches - Timber Piling (~60T ultimate)

Abutments: Earth Fill with RC cantilever retaining walls - (assume spread footing)

Hinges/Bearings: Fixed bearings on steel truss approach

Skew/Curve: N/A

Retrofit: N/A (Earthquake) bascules improved with horizontal post tensioning in 1980's

Pre-Earthquake Condition: Fair

Strong motion recording instruments present? No

III. Earthquake Damage:

Bending of RC Bailing @ Truss Approach (spalling) / Concrete approach @ so. end. Minor conc. spalling at approaches. Hairline cracks on concrete columns (bents 5-12) in South approach - Horiz. cracks 6"-12" below tops of columns (incipient plastic hinging). Minor conc. spalling on S. Approach. Liquefaction at column 3E on S. Approach (No damage). So. Abutment settlement ~ 1"

Transportation Structures (continued)

Total estimated loss:

Less than 10% X 10 - 50% over 50%

Is lifeline functional? yes - slight binding in bascule operation

If no, effect on route:

Estimated time to repair:

IV. Site Information:

Estimated Modified Mercalli Intensity/PGA: 0.10 - 0.15g

Types of soils: Alluvium - Loose, clean to silty fine SAND (0-60') over Glacial Till

Sand boils present? yes - South Conc. Approach est. 3 ft

Ground faulting present? NO

Foundation movement? NO (Columns in bent 10 moved ± 1" at ground surface)

V. Miscellaneous:

Film or digital images (include filename and/or roll information: Photos Available

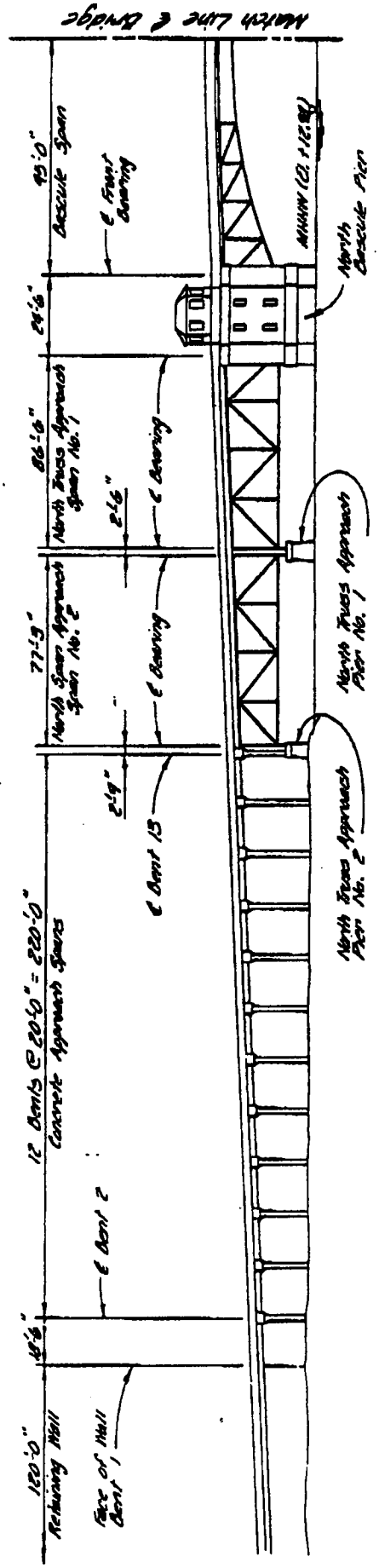
Sketches/Comments:

Profile Attached

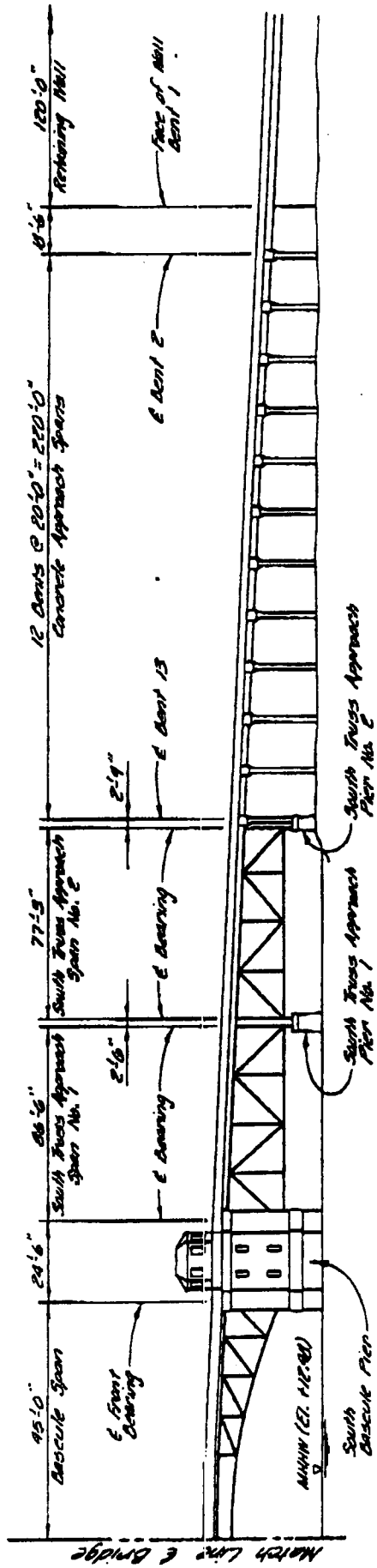
Imbsen Associates recently completed seismic evaluation of existing structure for an earthquake with ground shaking comparably to Nisqually effects.

Potential Repairs

- shoring SW beam @ south abutment - spalling reduced beam seat width
- Epoxy injection of cracks in columns 5-12 of S. approach
- Patch spalled concrete on railing
- Survey elevations of bents and bascule tracks to correlate w/ minister-Glaeser 2000 spot elevations.



NORTH BRIDGE APPROACH



SOUTH BRIDGE APPROACH

BRIDGE ELEVATION
FIGURE 2

Seattle Earthquake Reconnaissance
Site Summary

Site: Southpark Bridge (over Duwamish) (14th/16th Ave. S.)

Location: (W) N E S of Boeing Field

Team:
Paul Grant (206-406-4036)
PanGeo 2/28/01
Tim Lane - King Co. & 3/2/01
Scott Smith - King Co. & 3/5/01

Latitude: N 47° 31.770'
Longitude: W 122° 18.860'
Highway:
Address:
Milepost:

Features (circle all that apply):

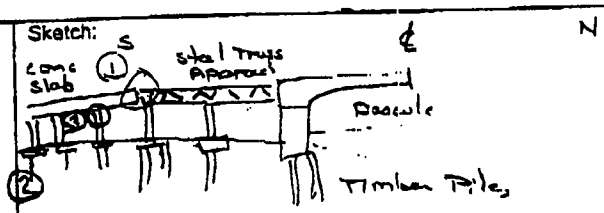
Fault rupture	Foundation failure	Building	
Landslide	Structural failure	(Bridge)	
(Sand boils)	Soil improvement	Embankment/levee	
Lateral spreading	(Lifeline facility)	Retaining structure	

Observations:

Minor structural Damage:

- Pounding (spalling) of Conc. railing at Conc. Approach/Steel Truss approach.
- incipient Plastic hinging at tops of columns supporting conc. Approach (bents 5-12)
- Minor Conc spalling @ S. approach

Sketch:



1. Pounding @ Rail.
2. Liquefaction boil @ bent col. 3E on S. approach.
3. Plastic hinging @ tops of columns in bents 5-12 of S. Approach.

- South abutment settled ~ 1"
- Liquefaction occurred (sand boil) adjacent to column 3E on south conc. Approach - No damage - Column pile supported (Timber friction piles)

Bridge supported on timber piles - both end bents & friction

Recommended Follow-up Work:

N/A

Follow-up Mechanism (circle one)
By second recon team
By long-term project

Follow-up Priority (circle one)
High
Medium
Low