CBJ EVALUATION OF REPURPOSED BREAKWATER

DECEMBER 14, 2023

232084.01

PREPARED FOR:

CITY & BOROUGH OF JUNEAU
155 South Seward Street
Juneau, AK 99801

PREPARED BY:

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INTRODUCTION

This report is a summary of the condition overview performed by PND Engineers, Inc. (PND) for the CBJ Evaluation of Repurposed Breakwater project. The purpose of this report is to provide the City and Borough of Juneau (CBJ) with a general overview of the current condition of the float and to identify specific areas and components that may need repair and/or replacement and to determine an approximate remaining service life of the float. The findings will further be used by CBJ to support their considerations to purchase the float from Western Marine Construction and repurpose it for their use at the Auke Bay Loading Facility.

The condition overview consisted of a top-side visual examination of major float system components. The float was examined for structural and mechanical damage, including rot, corrosion and other evidence of deterioration. No non-destructive or any other field-testing instruments were used to assist in evaluating the condition of float elements.

CBJ contracted with Global Diving and Salvage (GDS) to perform an underwater dive inspection as part of their condition overview program for the float. The condition overview and dive inspection were performed on September 27, 2023, during which PND was onboard the dive vessel and observed the live video feed recorded during the inspection. GDS's dive inspection report is included as an appendix to this report.

BACKGROUND

The float is a 24-foot wide x 180-foot long x 7-feet deep post-tensioned concrete breakwater pontoon float that was installed sometime in the mid 2000's and until 1-2 years ago provided wave protection for the U.S. Coast Guard at their base within Tongass Narrows in Ketchikan, Alaska.

The exterior pontoon walls are 6-inch thick reinforced concrete with 5-inch thick interior walls separating EPS foam filled compartments. There are a total of twelve, 20-inch diameter hot-dip galvanized hawse pipes embedded through the float. Each hawse pipe would be used to secure the float to an anchor chain and block. The anchor chains and blocks were not observed under this effort.

OBSERVATIONS

The following conditions were observed:

- **Concrete Deck –** The concrete deck appears to be in overall good condition. No significant spalling or cracking was observed.
- **Hawse Pipes –**The hot-dip galvanized hawse pipes are in good condition. There are some locations of rust spotting on the interior of the steel pipes where the galvanized coatings have been damaged, but overall, the majority of coatings are in-tact.
- **Timber Rubboard –** The timber rubboards are in fair condition. There are several locations of plant growth on the timber indicating that preservative treatments are no longer effective. Some wear and softening of the outer timber surface was observed.

ESTIMATED REMAINING SERVICE LIFE

Precast concrete structures such as these pontoons are known to be relatively low maintenance, with long service lives. Although it is estimated that the float has already been in service for approximately 20 years, with continued monitoring and maintenance of the pontoon and an adequate anchor system, it is estimated that the pontoon has a remaining useful service life of 20-25 years or more considering regular maintenance and monitoring of the pontoon condition and its installation within an environment suitable for its design capabilities.

INSTALLED CONSTRUCTION COST NEW

PND reviewed recent (2022-23) bid costs for similar floating concrete mooring structures currently being installed at USCG Bases in Ketchikan and Kodiak. Float manufacture costs range from $600-$1,000/SF FOB Pacific Northwest, with the higher unit costs for monolithic post stressed designs similar to this breakwater. These unit costs include a few ancillary operational improvements items such as conduits and utilidors necessary for permanent small vessel moorage which would not be needed for a wave attenuator. An estimated 2024 unit price for the manufacturing of a new monolithic floating wave attenuator is approximately $800/SF. The following estimate summarizes the costs for a new facility installed in Juneau. The CBJ can use these figures to determine a reasonable offer for the used pontoon delivered from Ketchikan and installed per original USCG design with anchors and chain in Juneau.

Item	Item Description	Units	Quantity	Unit Cost	Item Cost
1	Manufacture 24x180 Concrete Pontoon	SF	4,320	$800	$3,456,000
2	Deliver & Install Pontoon	LS	All Req'd	$700,000	$700,000
3	Provide & Install Anchor Chain	Shot	45	$4,000	$180,000
4	Provide & Install Concrete Anchors	EA	34	$7,500	$255,000
5	Provide & Install Clump Weight	EA	16	$1,500	$24,000
	Total Installed Construction Cost New				**$4,615,000***

*The above costs do not include project contingency or indirect costs for planning, permitting, design, contract administration and construction inspection.

Photo 1. Overview of concrete breakwater float

Photo 2. Overview of concrete breakwater float

Photo 3. Overview of concrete breakwater float

Photo 4. Overview of concrete breakwater float

Photo 5. Inside of hawse pipe with chain keeper

Photo 6. Inside of hawse pipe with chain keeper

Photo 7. Hawse pipe and water tight manhole cover

Photo 8. Hawse pipe and water tight manhole cover

Photo 9. Embedded anchor bolts from previously installed navigation light

Photo 10. Worn timber rubboards with vegetation.

PND ENGINEERS, INC.

MEMORANDUM

PROJECT NO. 232023.10 **DATE: December 14, 2023**

PROJECT: CBJ Evaluation of Repurposed Breakwater

To: Carl Uchytil, P.E. Port Director
CC: Matthew Sill, P.E., Port Engineer

FROM: Alexander Khokhlov, Coastal Engineer, PND Engineers. Inc.

SUBJECT: CBJ Evaluation of Repurposed Floating Breakwater – Efficacy Evaluation

Study Objectives

This memo analyzes Metocean (meteorological and oceanographic) criteria in the vicinity of the Auke Bay Loading Facility drive-down float and evaluates the efficacy of the proposed 180-foot by 24-foot floating breakwater for this facility. The analysis includes wind and wave data from measured and hindcast sources affecting the site. Locally generated wind waves were calculated using wave growth formulas found in the U.S. Army Corps of Engineers (USACE) Shore Protection Manual (USACE, 1984) and Coastal Engineering Manual (USACE, 2005).

As part of this study, wave transmission was analyzed from four directions associated with the largest fetch distances to the site. The memo presents predicted wave heights behind the proposed breakwater and describes the comparisons of the predicted performance of the proposed floating breakwater. Performance is measured in terms of the potential sheltering achieved behind the structure. Four potential locations are assessed, each associated with straight-line fetch distances.

The Auke Bay Loading Facility drive-down float structures are exposed to waves generated along straight-line fetch distances, as illustrated in Figure 1. Additionally, the structure is subject to long-period swell waves and boat wakes. Long-period swell waves from the southwest can reach the proposed site only by diffracting and refracting around headlands and islands.

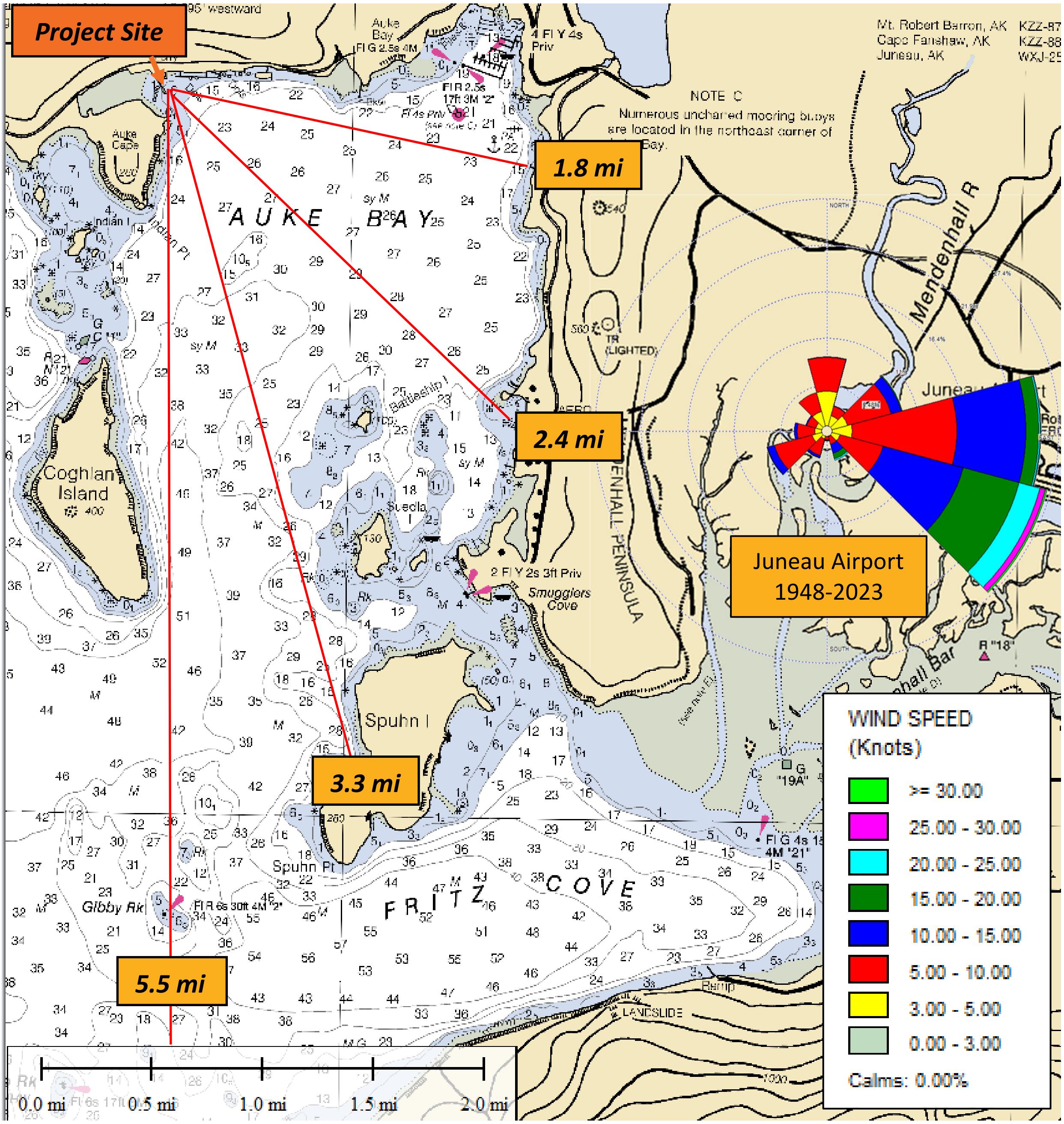

Figure 1. Auke Bay Area Map and Wave Directions (NOAA Chart # 17315 – Depths in Fathoms)

1. WIND

The analysis of wind speeds for the wave hindcast calculations focuses on the extremes. Wind data from the Juneau Airport station were selected for extremal wind analysis due to location and data availability. Measured wind data is available from the Juneau Airport (1948 – 2023). The Juneau Airport is located in approximately 4.3 miles southeast from the project site. The wind data are 2-minute average wind speeds for land stations. Wind direction is defined as the direction from which winds are traveling. The station location and the corresponding wind rose is shown in Figure 1. The highest recorded wind speed was a southeasterly 62 knots at the Juneau Airport station. Winds from the southeast are prevailing at the Airport station. Winds at the site could be affected by the potential funneling effects of the bay. Local topography can funnel winds and concentrate them at various locations within the bay. Due to the local wind data limitations, an upper 90th percentile return period is recommended for wind analysis.

The highest recorded wind speeds, filtered by direction, were analyzed to determine the wind speed associated with a given return period. The extremal analysis was carried out according to the Automated Coastal Engineering System (ACES) technical reference.

Table 1 shows the ten most significant wind speeds from all directions measured at the Juneau Airport station.

Table 1. Most Considerable Recorded Annual Wind Speeds – All Directions

Rank	Juneau Airport (1948-2023)		
	Date	Speed (knots)	Dir (deg)
1	5/20/1976	62	130
2	10/17/1985	58	220
3	7/6/1985	57	200
4	8/7/1978	55	10
5	9/24/1979	55	290
6	6/3/1985	53	100
7	12/18/1979	53	90
8	4/29/1981	53	10
9	6/13/1981	53	120
10	7/17/1979	52	150

Table 2 shows the ranked wind speeds for Juneau Airport filtered by fetch directions. The return period wind speeds are summarized in Table 3.

Table 2. Most Considerable Recorded Wind Speeds – Filtered by Fetch Direction (Juneau Airport Station)

Rank	East (75°-105°)			East-Southeast (105°-135°)		
	Date	Speed knots	Dir deg	Date	Speed knots	Dir deg
1	6/3/1985	53	100	5/20/1976	62	130
2	12/18/1979	53	90	6/13/1981	53	120
3	2/19/1973	42	100	5/3/1981	49	130
4	10/8/1973	42	100	7/27/1981	49	120
5	12/10/1998	41	100	11/22/1984	48	120
6	10/2/2021	41	100	10/30/1949	44	113
7	12/9/1984	36	100	12/30/1963	44	113
8	9/26/1986	36	100	12/4/1963	43	113
9	1/14/2008	36	100	4/9/2023	43	110
10	11/28/1986	35	90	11/29/1968	42	120
Rank	South-Southeast (135°-165°)			South (165°-195°)		
	Date	Speed knots	Dir deg	Date	Speed knots	Dir deg
1	7/17/1979	52	150	9/16/1982	43	170
2	4/15/1997	45	150	11/22/1984	28	170
3	2/15/1973	42	150	4/20/1985	25	180
4	5/9/1973	42	150	10/4/1961	24	180
5	5/15/1973	42	150	6/22/1984	24	180
6	10/30/1973	42	150	9/20/1948	23	180
7	4/28/1974	42	150	10/2/1952	23	180
8	7/17/1975	42	150	11/19/1952	23	180
9	11/2/1987	34	140	10/8/1951	22	180
10	10/5/1987	33	140	10/29/1958	22	180

Table 3. Juneau Airport - Return Period Wind Speed Analysis Summary

Direction	Juneau Airport				
	2-yr Wind Speed (knots)	5-yr Wind Speed (knots)	10-yr Wind Speed (knots)	50-yr Wind Speed (knots)	100-yr Wind Speed (knots)
All Directions	41	47	51	61	65
All Directions – upper 90th percentile	43	49	54	65	70
East (75°-105°)	29	33	37	44	47
East (75°-105°) – upper 90th percentile	31	35	39	47	51
East-Southeast (105°-135°)	37	41	44	51	54
East-Southeast (105°-135°) - upper 90th percentile	38	43	46	54	57
South-Southeast (135° - 165°)	27	31	35	43	46
South-Southeast (135° - 165°) - upper 90th percentile	28	33	37	47	51
South (165°-195°)	18	21	24	29	32
South (165°-195°) – upper 90th percentile	19	23	26	32	35

2. WAVE HINDCAST CALCULATIONS

Measured wave data is unavailable for the site. Waves at the project site were estimated using wind data and hindcast formulae in the U.S. Army Corps of Engineers Coastal Engineering Manual (USACE, 1984). The project site is exposed to waves generated along straight-line fetch distances, as shown in Figure 1. Long-period swell waves can reach the proposed site only by diffracting and refracting around headlands and islands.

Fetch-limited wave calculation methods were applied to determine the wave height and period associated with the wind speeds and fetch lengths. The hindcast significant wave height (Hs), peak period (Tp), and maximum wave height (Hmax) are calculated and listed in Table 4. The wave heights estimated are for deep water, meaning they originate in a depth offshore before they can feel the bottom and shoal or refract.

These results would be a worst-case wave height as it does not account for wave height shoaling or refraction as the waves transform around landforms near the project site.

The significant wave height is the average of the highest one-third of all waves measured. The maximum wave height is the most significant single wave during a storm event and is assumed to be equal to 1.7 times the significant wave height. The wind speed analysis for hindcast calculations was directional, meaning the return period winds aligned with the associated fetch direction were used to calculate the return period wind speed. The simple wind wave desktop calculation methods are limited by their underlying assumptions. Numerical models of wind wave generation and transformation are recommended prior to a final design.

The highest 50-year return period significant wave height at the new breakwater location is estimated to be 4.0 ft with a peak wave period of 2.5 seconds from east-southeast direction.

Table 4. Wave Hindcast Analysis

No.	Return Period	Wind Speed (knots)	H_s (feet)	H_{max} (feet)	T_p (s)
East Fetch = 1.8 Miles					
1	2-Year Return Period	31	1.8	3.3	1.8
2	5-Year Return Period	35	2.0	3.8	1.9
3	10-Year Return Period	39	2.3	4.3	2.0
4	50-Year Return Period	47	2.9	5.5	2.2
5	100-Year Return Period	51	3.2	6.0	2.3
East-Southeast Fetch = 2.4 Miles					
1	2-Year Return Period	38	2.6	4.8	2.2
2	5-Year Return Period	43	3.0	5.6	2.3
3	10-Year Return Period	46	3.3	6.1	2.4
4	50-Year Return Period	54	4.0	7.5	2.5
5	100-Year Return Period	57	4.3	8.0	2.6
South-Southeast Fetch = 3.3 Miles					
1	2-Year Return Period	28	2.1	3.9	2.2
2	5-Year Return Period	33	2.5	4.7	2.3
3	10-Year Return Period	37	2.9	5.5	2.4
4	50-Year Return Period	47	3.9	7.3	2.7
5	100-Year Return Period	51	4.4	8.1	2.8
South Fetch = 5.5 Miles					
1	2-Year Return Period	19	1.7	3.1	2.2
2	5-Year Return Period	23	2.1	3.9	2.4
3	10-Year Return Period	26	2.4	4.5	2.5
4	50-Year Return Period	32	3.1	5.8	2.7
5	100-Year Return Period	35	3.5	6.4	2.8

3. BREAKWATER PERFORMANCE ASSESSMENT

In this section, the performance of the proposed floating breakwater is assessed in terms of the projected level of sheltering achieved for the drive-down float. The breakwater is calculated as a floating rectangular box, measuring 180 feet in length, 20 feet in width, and with a draft of 5 feet. The calculations assume a uniform water depth of 30 feet, and the incident wave is considered to be perpendicular to the breakwater.

These analyses utilize four wave approach angles, as shown in Figure 1. The incident wave climate considered corresponds to the 50-year return period summarized in Table 4. When waves approach a floating breakwater, the incident wave is partially transmitted, partially reflected, and partially dissipated. Waves also diffract or bend around the two ends of the breakwater into its lee.

Wave transmission calculations are primarily based on empirical methods, providing results that closely align with experiments in wave tanks under laboratory conditions, with waves approaching perpendicular to the floats. Tests by various researchers have indicated that wave transmission is highly sensitive to the input wave period within a narrow range, about 3 to 4 seconds for floating breakwaters. For longer period waves, the transmitted wave shows minimal attenuation due to the breakwater's presence. This is because the floating breakwater moves upwards and downwards with the waves, rejecting very little wave energy and resulting in a large transmission coefficient.

There are several formulae and methods for calculating wave transmissions behind floating breakwaters. PND applied the formulae based on work by Macagno's (1954), the wave transmission coefficient for a rectangular, fixed, and infinitely long breakwater with a draft, d, and width, w, which is subject to regular waves can be estimated by following equation:

$$K_{tm} = \frac{1}{\sqrt{1 + \left(kw \dfrac{\sinh(kh)}{2\cosh(kh - kd)}\right)^2}} \qquad (1)$$

Where: k is the wave number and, h, is the water depth.

The 50-year calculated transmission coefficients for the proposed breakwater are summarized in Table 5. According to the analysis, the highest 50-year transmission coefficient, Kt = 0.25, is observed for waves approaching from the south-southeast direction This results in a transmitted wave height immediately behind the structure of Hs = 1.0 feet.

Table 5. Proposed Floating Breakwater Transmitted Wave Results for 50-Year Incident Waves

Wave Direction	50-Year Incident Wave		Transmission Coefficient (Kt)	Transmitted Wave (Hs, ft)
	Hs (ft)	Tp (s)		
East	2.9	2.2	0.11	0.3
East-Southeast	4.0	2.5	0.19	0.7
South-Southeast	3.9	2.7	0.25	1.0
South	3.1	2.7	0.25	0.8

The significant wave height at a series of field points on a grid in the lee of the breakwater is computed using MIKE21 Spectral Wave model, applied to the simplified grid mirroring the project bathymetry. Color contours depicted the significant wave height are then plotted and superimposed on the site plan. It is recommended to conduct a site-specific model simulation to validate the breakwater's performance.

Figure 2 illustrates the effectiveness of the proposed breakwater positioned 200 feet east of the drive-down float. With an incident significant wave height of 2.9 feet, the breakwater successfully reduces the significant wave height to a range of 2.4 to 2.7 feet near the southeast corner of the float. This equates to a reduction of 0.5 feet to 0.2 feet, corresponding to a percentage reduction of 18% to 7%.

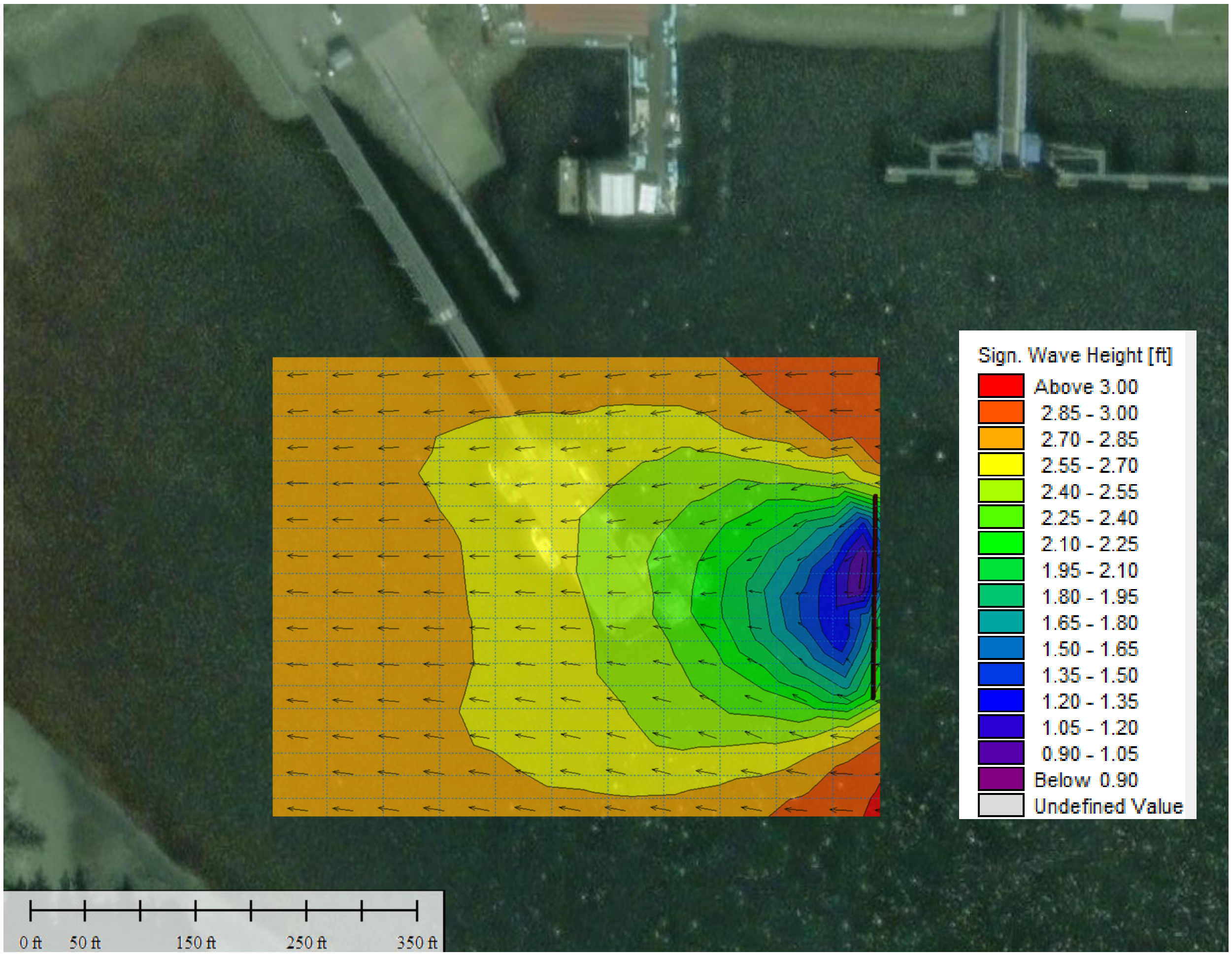

Figure 2. Floating Breakwater Performance for Easterly Waves. Contours of significant wave height in the lee of existing breakwater.

Figure 3 illustrates the performance of the proposed breakwater positioned 200 feet to the east-southeast of the drive-down float. With an incident significant wave height of 4.0 feet, the breakwater effectively mitigates the significant wave height, reducing it to a range of 3.2 and 3.6 feet near to the southeast corner of the float. This results in a reduction of 0.8 to 0.4 feet, equivalent to a percentage decrease of 20% to 10%.

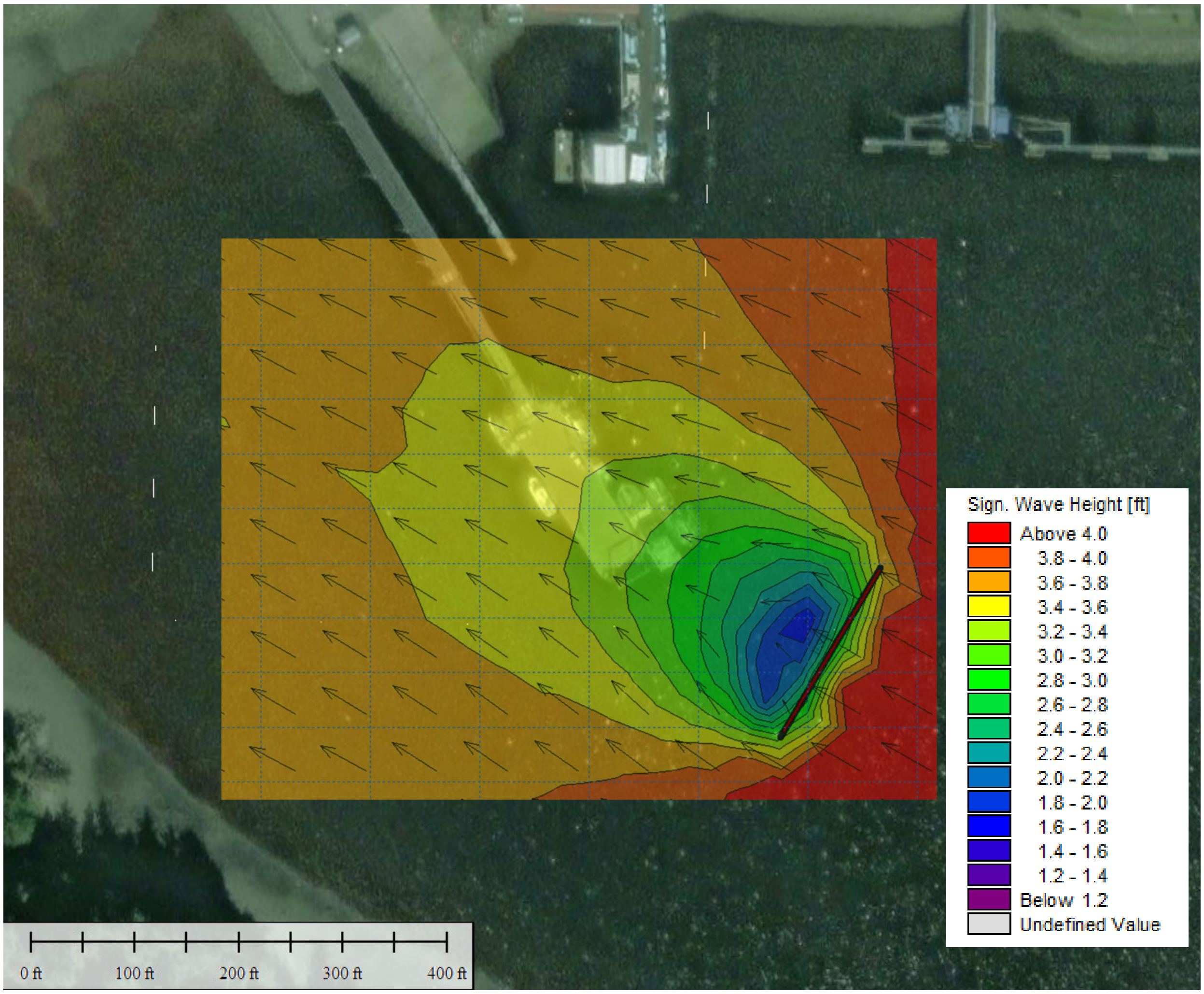

Figure 3. Floating Breakwater Performance for East-Southeasterly Waves. Contours of significant wave height in the lee of existing breakwater.

Figure 4 illustrates the performance of the proposed breakwater positioned 200 feet to the south-southeast of the drive-down float. With an incident significant wave height of 3.9 feet, the breakwater effectively diminishes the significant wave height, bringing it down to a range of 3.0 to 3.3 feet near the southeast corner of the float. This results in a reduction of 0.9 to 0.6 feet, corresponding to a percentage decrease of 23% to 15%.

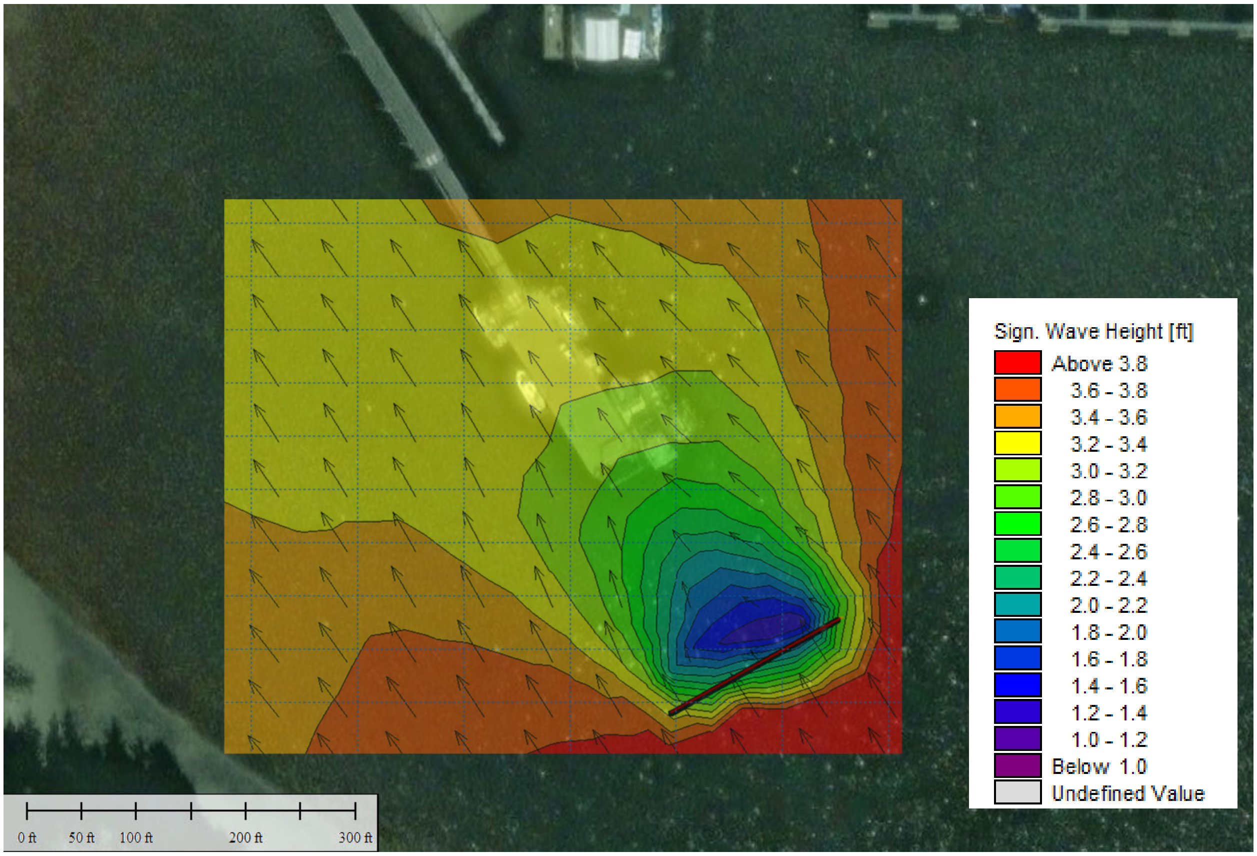

Figure 4. Floating Breakwater Performance for South-Southeasterly Waves. Contours of significant wave height in the lee of existing breakwater.

Figure 5 illustrates the performance of the proposed breakwater positioned 200 feet to the south of the drive-down float. The intentional shift of the breakwater to the east accounts for the natural protection of the western corner of the drive-down float by Auke Cape, where wave storms from the south are naturally diffused and diminished. With an incident significant wave height of 3.1 feet, the breakwater effectively lowers the significant wave height to a range of 2.2 to 2.5 feet near the southeast corner of the float. This results in a reduction of 0.9 to 0.6 feet, corresponding to a percentage decrease of 29% to 19%.

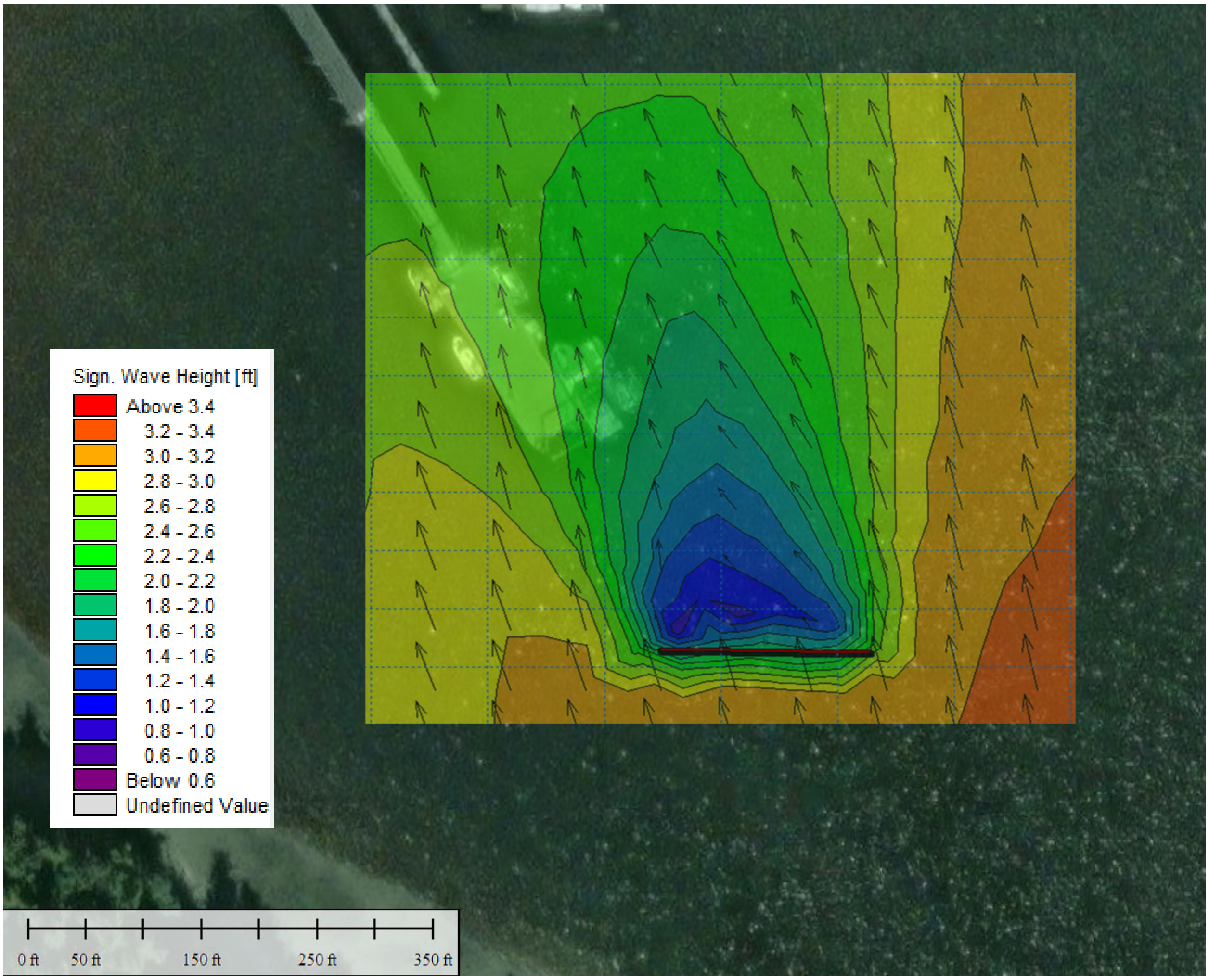

Figure 5. Floating Breakwater Performance for Southerly Waves. Contours of significant wave height in the lee of existing breakwater.

4. CONCLUSIONS AND RECOMMENDATIONS

Considering the local wave environment, the proposed breakwater promises substantial wave reduction immediately behind the structure. However, due to its relatively short length (only 180 feet), waves are expected to diffract around the structure, limiting the optimal wave reduction to approximately 20%-25% when placed around 200 feet from the drive-down float. If the breakwater was shifted closer to the structure, its shadow would shift progressively from the weather side of the drive-down float. However, this adjustment comes with drawbacks, including limited space for turning and potential hindrance to boat traffic.

It's important to note that all wave transmission calculations are inherently approximate, primarily relying on empirical methods that closely align with experiments conducted in wave tanks under laboratory conditions, particularly when waves approach perpendicular to the floats.

The recommended location for proposed floating breakwater is approximately 150-200 feet away from the drive down float. In this position, considering the most likely wave approach angle, the majority of the breakwater's shadow falls on the eastern side of the pier, providing protection to moorages on this side. The most exposed southeast corner of the drive-down float can expect a reduction of about 0.8 feet (or 20%) from an incident significant wave height of 4.0 feet.

It's worth noting that the wave height contours were approximated using a simplified version of the MIKE21 model without considering the site-specific environment. For a more accurate analysis of wave transmission and the breakwater's performance, advanced hydraulic numerical models such as the MIKE 21 Mooring Analysis (MA) Module, applied to the site-specific environment, should be considered. The MIKE21 MA software calculates the dynamics of a floating body exposed to incident environmental forces (wave, wind, current) and provides amplification factors around the breakwater.

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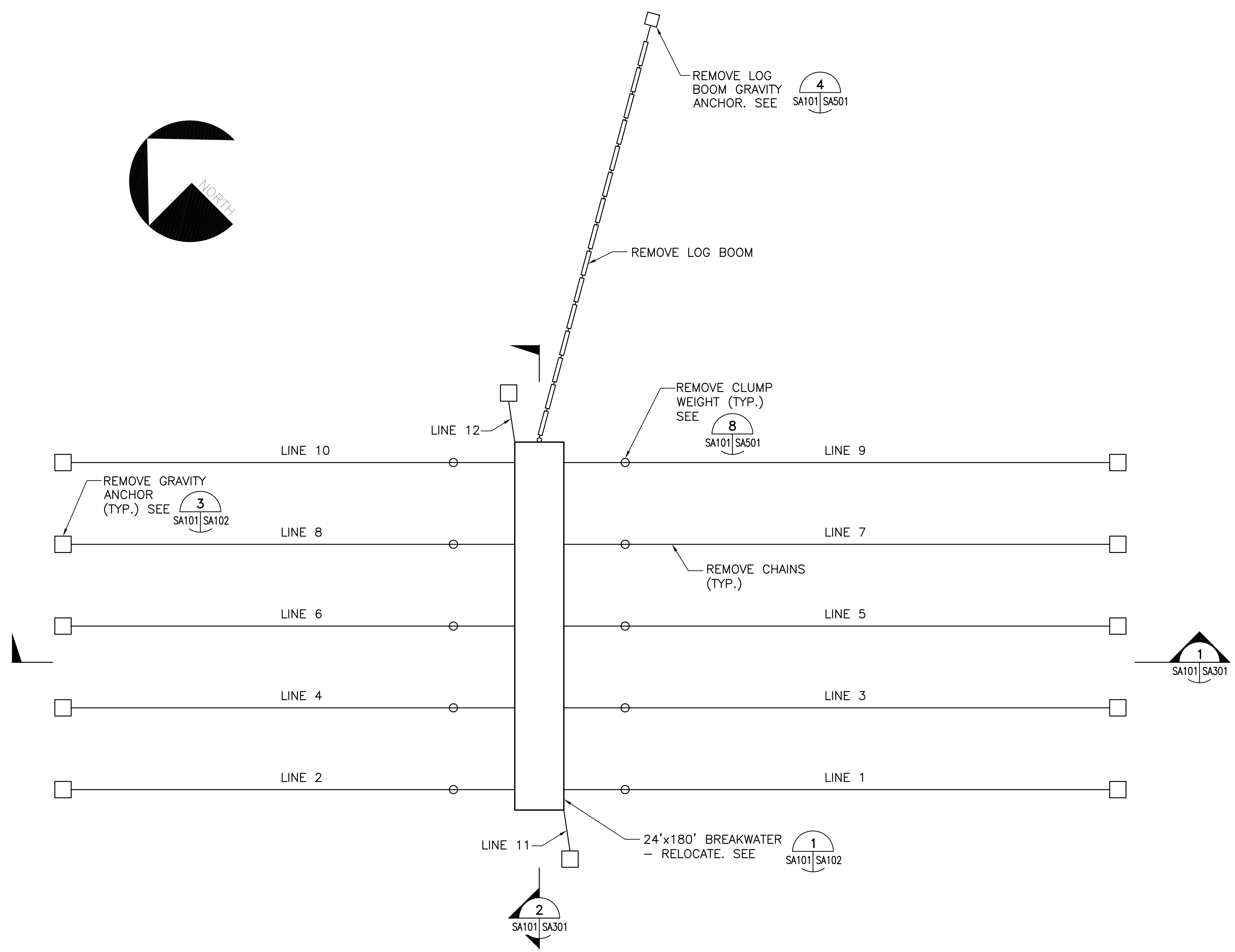

EXISTING WAVE ATTENUATOR PLAN

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RECORD DRAWING
DATED – 11/11/2015

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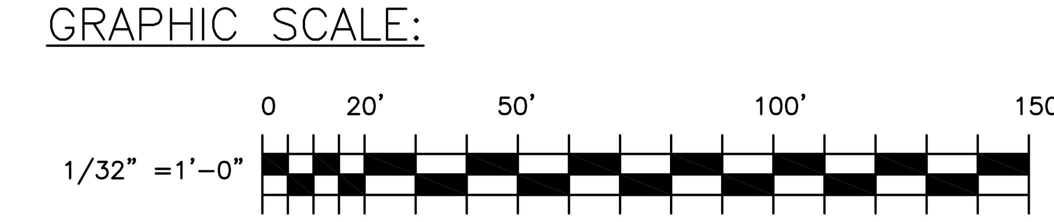

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Mortenson construction

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ISSUE

MARK	DATE	DESCRIPTION
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DRAWN BY:	ESN
EDITED BY:	ESN
CHECKED BY:	RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(E) WAVE ATTENUATOR LAYOUT

REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
PROJECT ENG.	BRANCH CHIEF	TECH. DIRECTOR

APPROVING OFFICER | DATE

PROJECT NUMBER	DRAWING NUMBER
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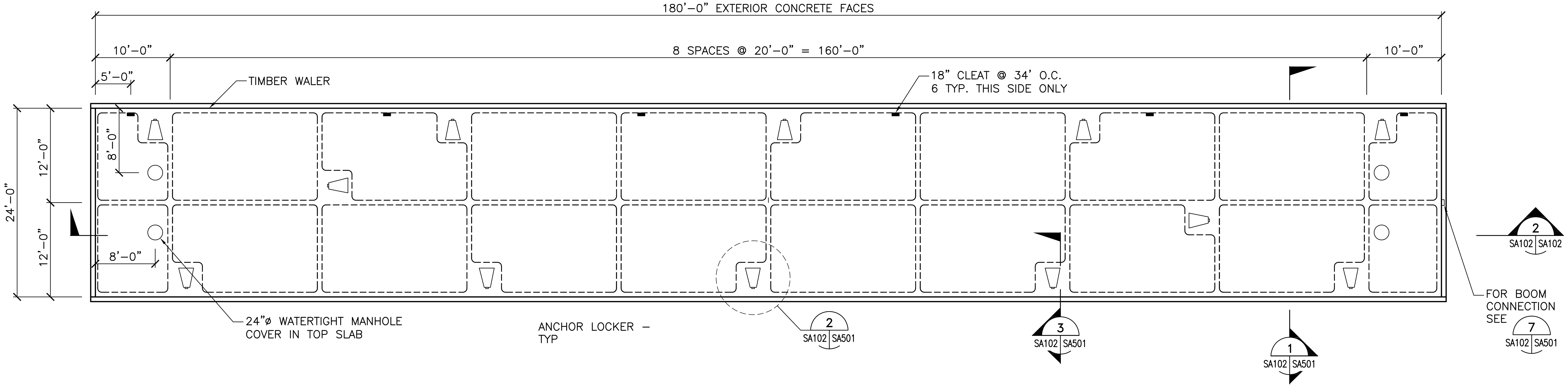
DISCIPLINE/SHT NO
SA101

SHEET 78 OF 230



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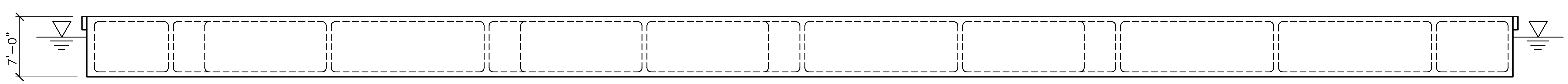

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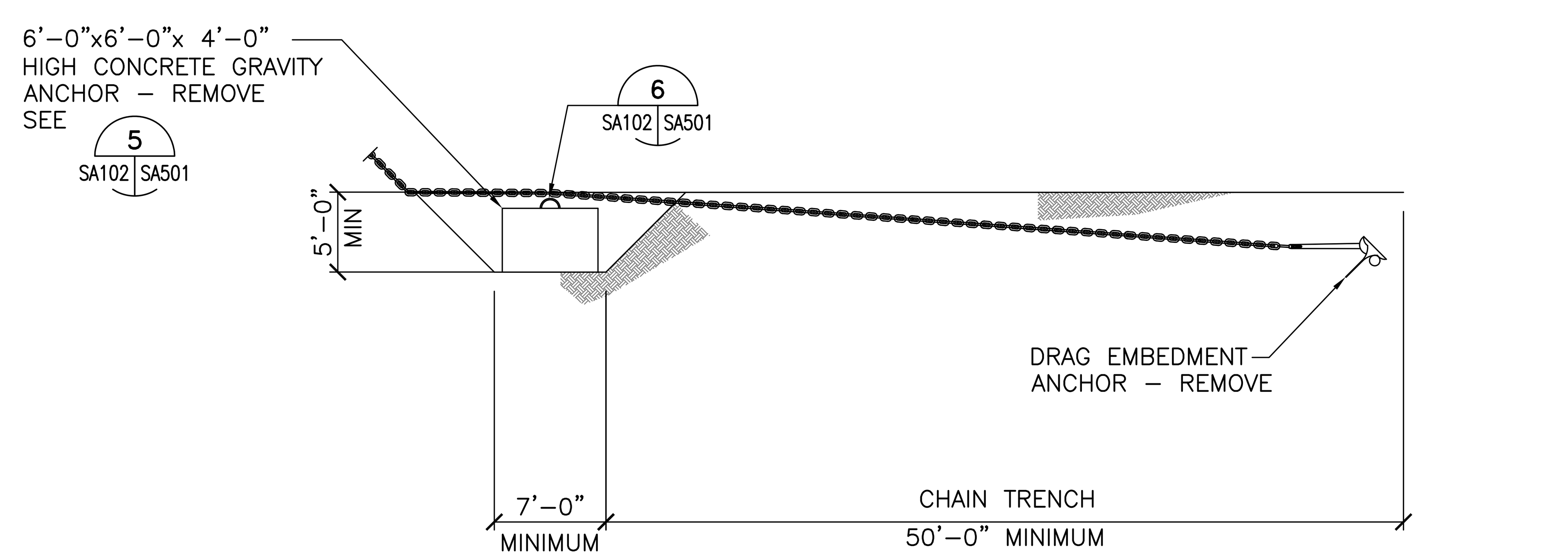
NOTE:
ALL FLOAT COMPARTMENTS ARE FILLED
W/ EXPANDED POLYSTYRENE (EPS)



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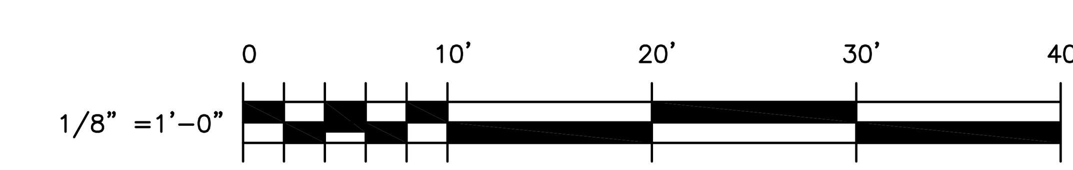

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ISSUE

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DESIGNED BY: CDD
DRAWN BY: ESN
EDITED BY: ESN
CHECKED BY: RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(E) WAVE ATTENUATOR PLAN

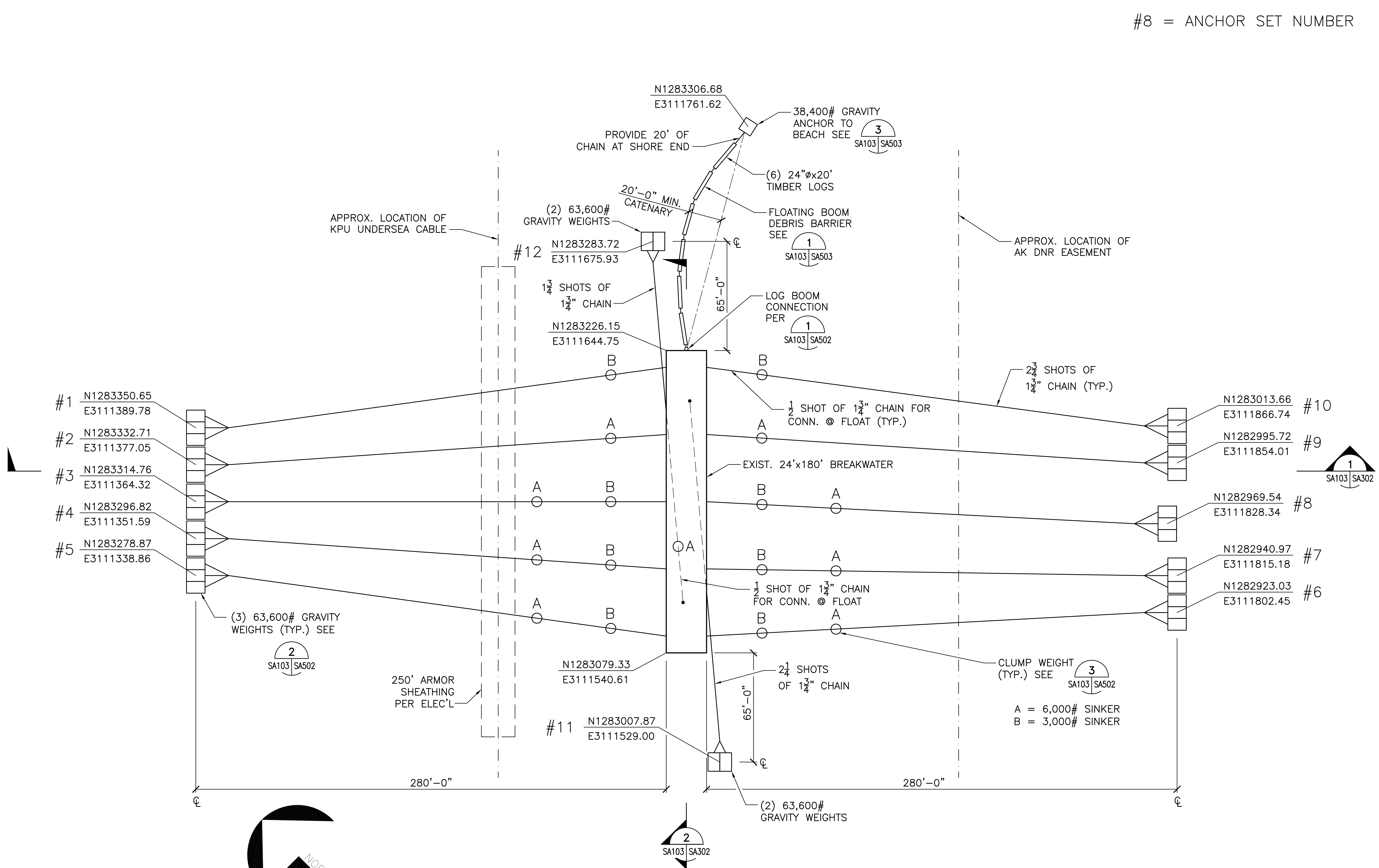
REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
PROJECT ENG.	BRANCH CHIEF	TECH. DIRECTOR

APPROVING OFFICER | DATE

PROJECT NUMBER	DRAWING NUMBER
4138942	4138942SA102

DISCIPLINE/SHT NO
SA102

SHEET 79 OF 230



NEW WAVE ATTENUATOR MOORING ARRANGEMENT 1 SW101 SA103

SCALE: 1/32" = 1'-0"

RECORD DRAWING
DATED – 11/11/2015

GRAPHIC SCALE:

0 20' 50' 100' 150'

1/32" =1'-0"

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STATE OF CALIFORNIA
REGISTERED PROFESSIONAL ENGINEER
STRUCTURAL No. S4150

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A/E PROJECT NO: 4138942
CAD FILE NAME: 4138942SA103.DWG
DESIGNED BY: CDD
DRAWN BY: ESN
EDITED BY: ESN
CHECKED BY: RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
NEW WAVE ATTENUATOR PLAN

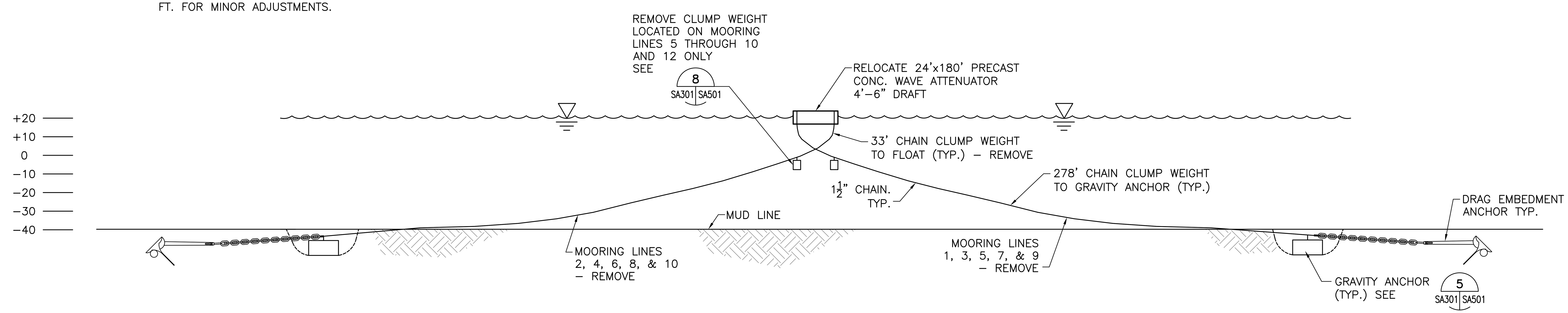
REVIEWED BY: PROJECT ENG. | REVIEWED BY: BRANCH CHIEF | REVIEWED BY: TECH. DIRECTOR

APPROVING OFFICER | DATE

PROJECT NUMBER: 4138942
DRAWING NUMBER: 4138942SA103
DISCIPLINE/SHT NO: SA103

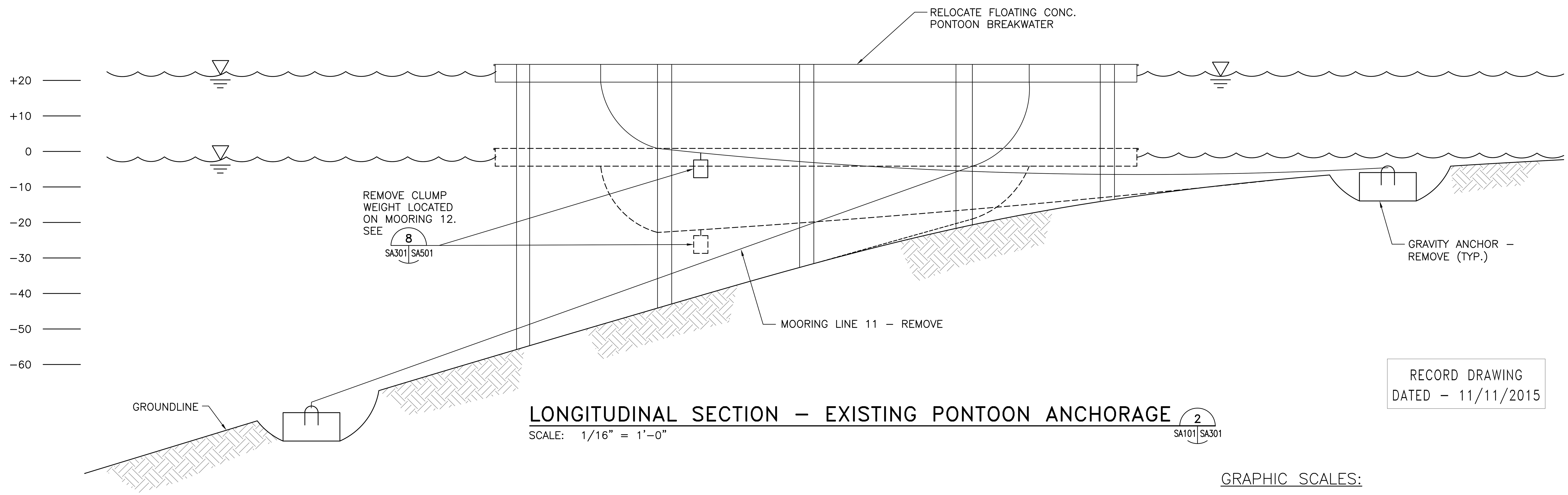
MOORING LINE NOTES:

1. THE NOMINAL MOORING LINE LENGTHS SHOWN INCLUDE LENGTH OF ASSOCIATED HARDWARE SUCH AS SHACKLES AND LINKS.
2. NOMINAL MOORING LINE LENGTHS HAVE BEEN DETERMINED USING APPROXIMATE BOTTOM ELEVATION BASED UPON BATHYMETRIC DATA AND HORIZONTAL ANCHOR LOCATIONS. THE NOMINAL MOORING LINE LENGTH INCLUDES AN ADDITIONAL 10 FT. FOR MINOR ADJUSTMENTS.



TRANSVERSE SECTION – EXISTING PONTOON ANCHORAGE (1 SA101 SA301)

SCALE: 1/32" = 1'–0"

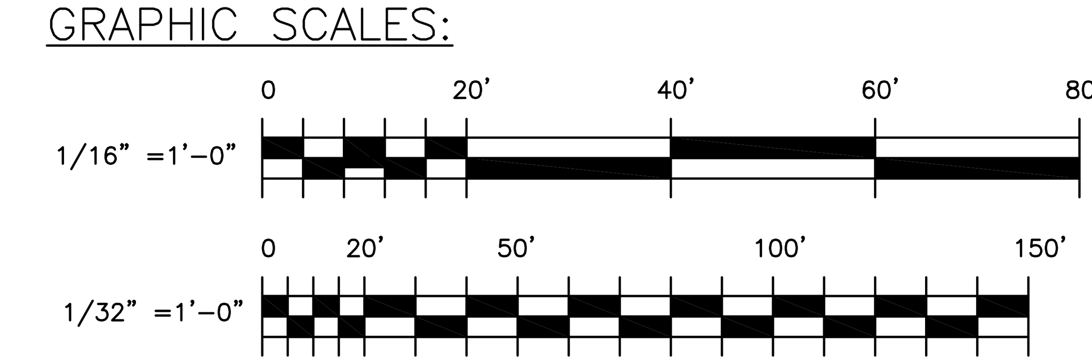


RECORD DRAWING DATED – 11/11/2015

LONGITUDINAL SECTION – EXISTING PONTOON ANCHORAGE (2 SA101 SA301)

SCALE: 1/16" = 1'–0"

GRAPHIC SCALES:



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CAD FILE NAME:	4138942SA301.DWG
DESIGNED BY:	CDD
DRAWN BY:	ESN
EDITED BY:	ESN
CHECKED BY:	RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

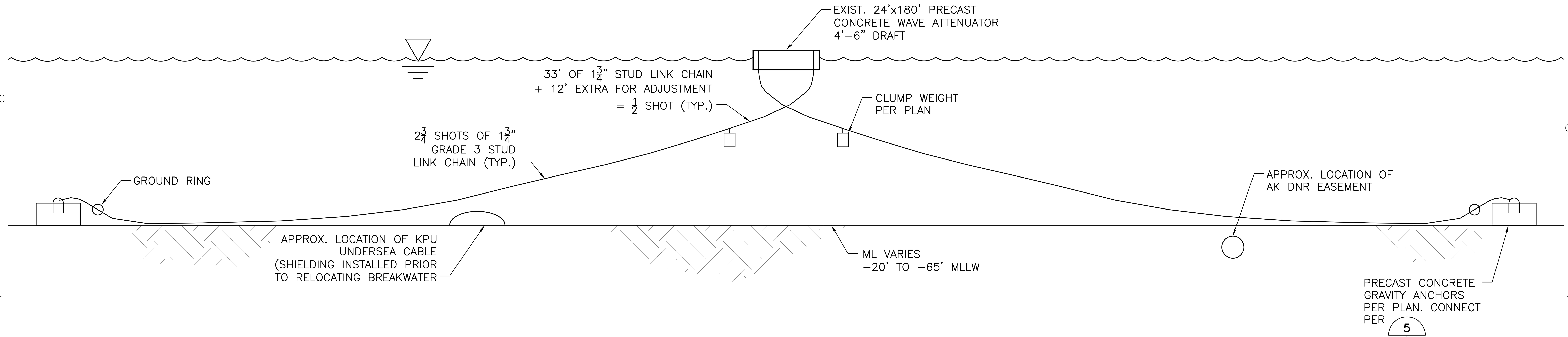
SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(E) WAVE ATTENUATOR SECTIONS

REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
PROJECT ENG.	BRANCH CHIEF	TECH. DIRECTOR

APPROVING OFFICER | DATE

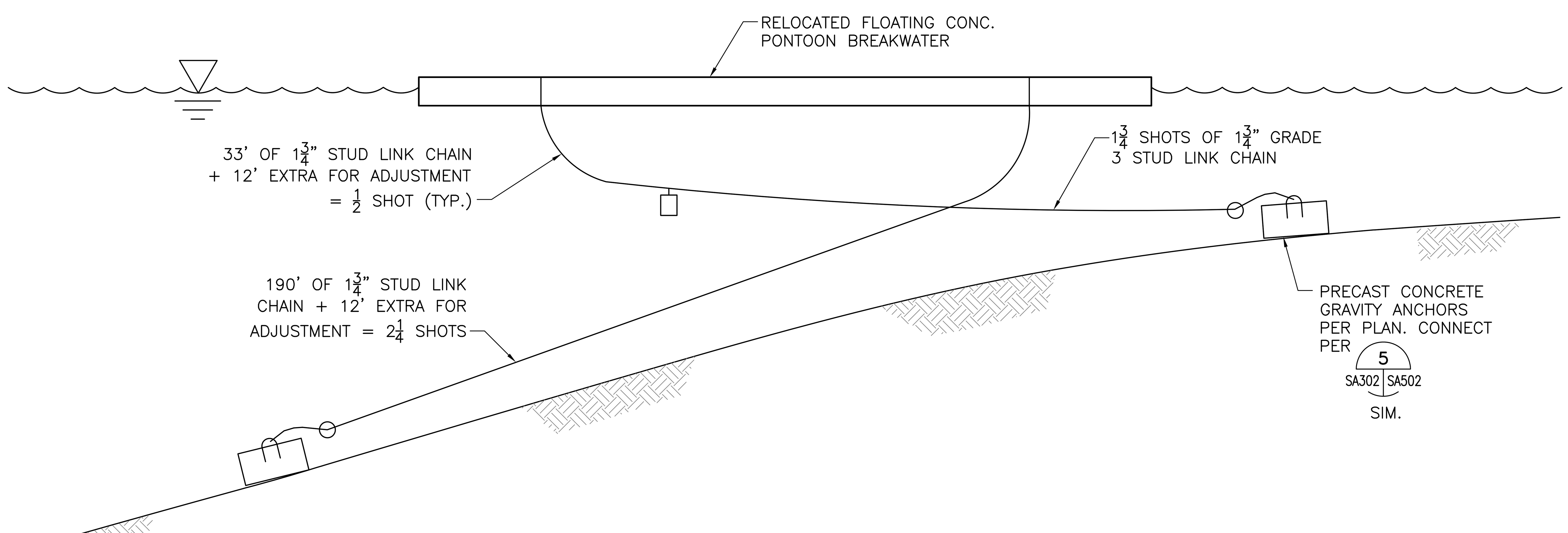
PROJECT NUMBER: 4138942
DRAWING NUMBER: 4138942SA301
DISCIPLINE/SHT NO: SA301



NEW WAVE ATTENUATOR ANCHORAGE – TRANSVERSE SECTION

SCALE: 1" = 20'

1
SA103 | SA302



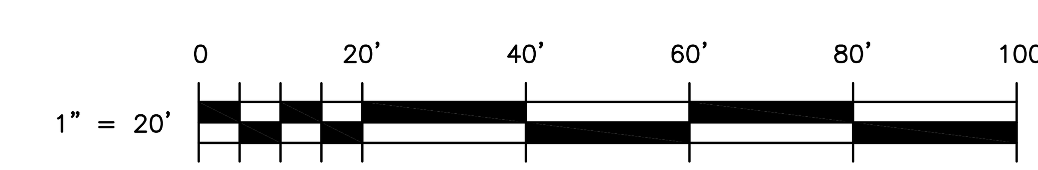
NEW WAVE ATTENUATOR ANCHORAGE – LONGITUDINAL SECTION

SCALE: 1" = 20'

2
SA103 | SA302

RECORD DRAWING
DATED – 11/11/2015

GRAPHIC SCALE:



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	11/11/15	RECORD DRAWINGS

A/E PROJECT NO: 4138942
CAD FILE NAME: 4138942SA302.DWG
DESIGNED BY: CDD
DRAWN BY: ESN
EDITED BY: ESN
CHECKED BY: RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(N) WAVE ATTENUATOR SECTIONS

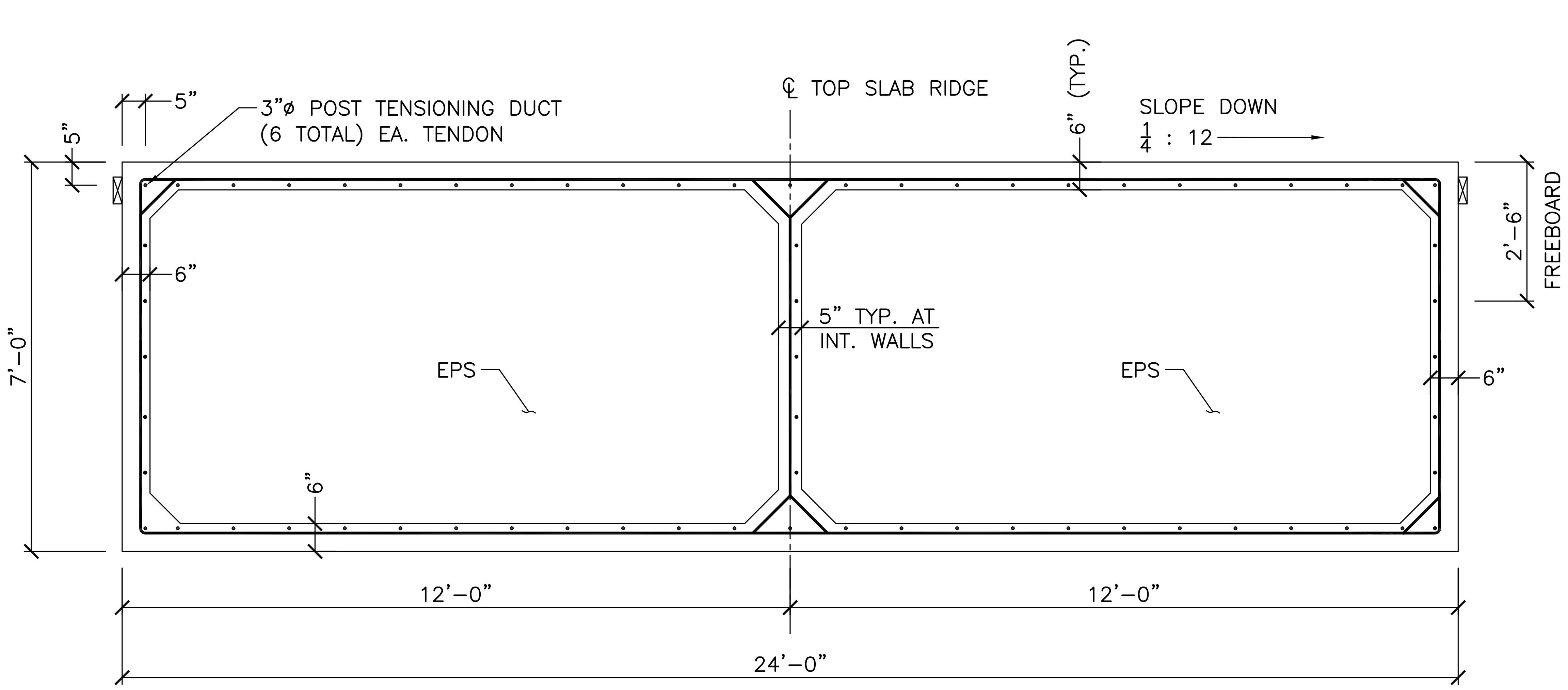
REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
PROJECT ENG.	BRANCH CHIEF	TECH. DIRECTOR

APPROVING OFFICER
DATE

PROJECT NUMBER	DRAWING NUMBER
4138942	4138942SA302

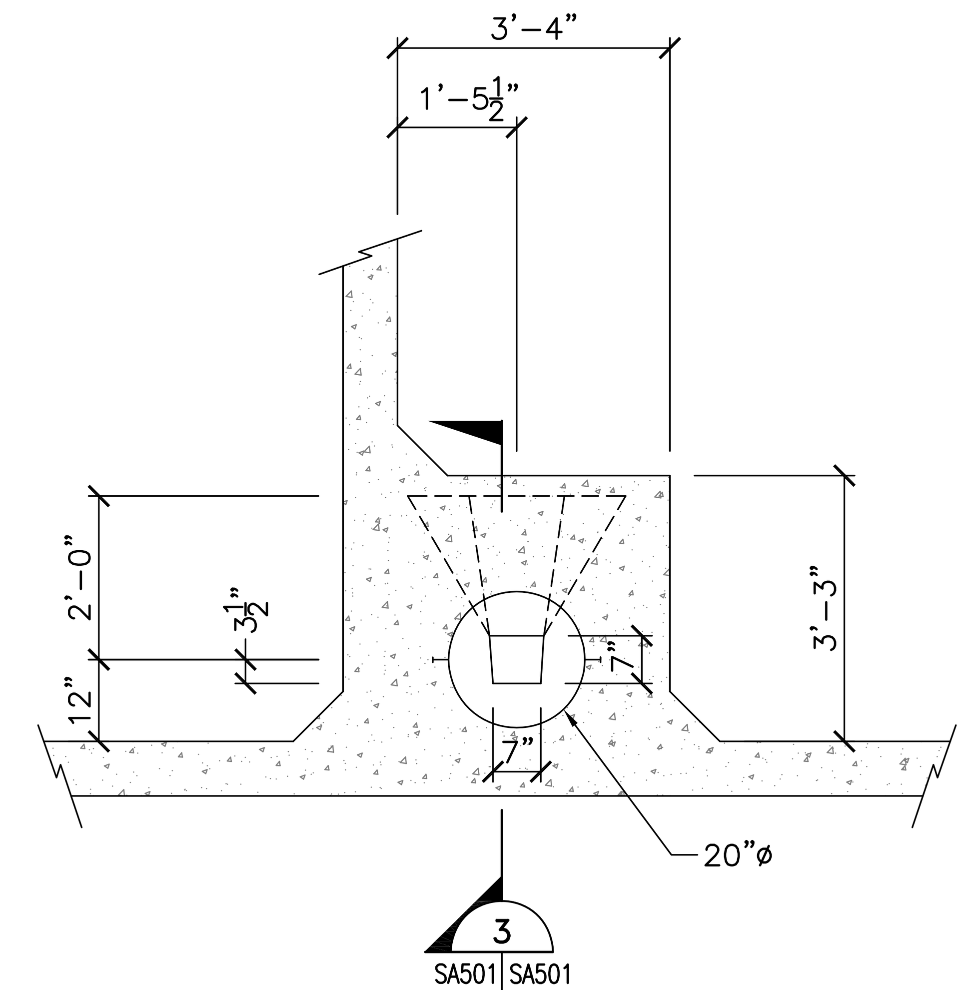
DISCIPLINE/SHT NO
SA302

SHEET 82 OF 230



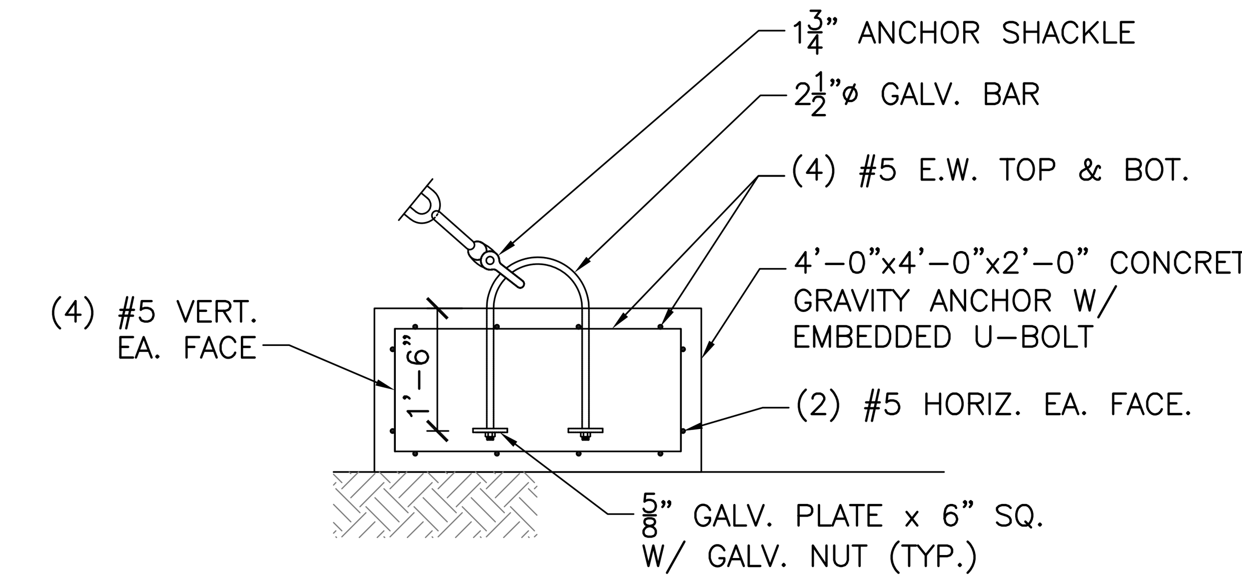
SECTION – EXISTING PONTOON
1 SA102 | SA501

SCALE: 1/2" = 1'-0"



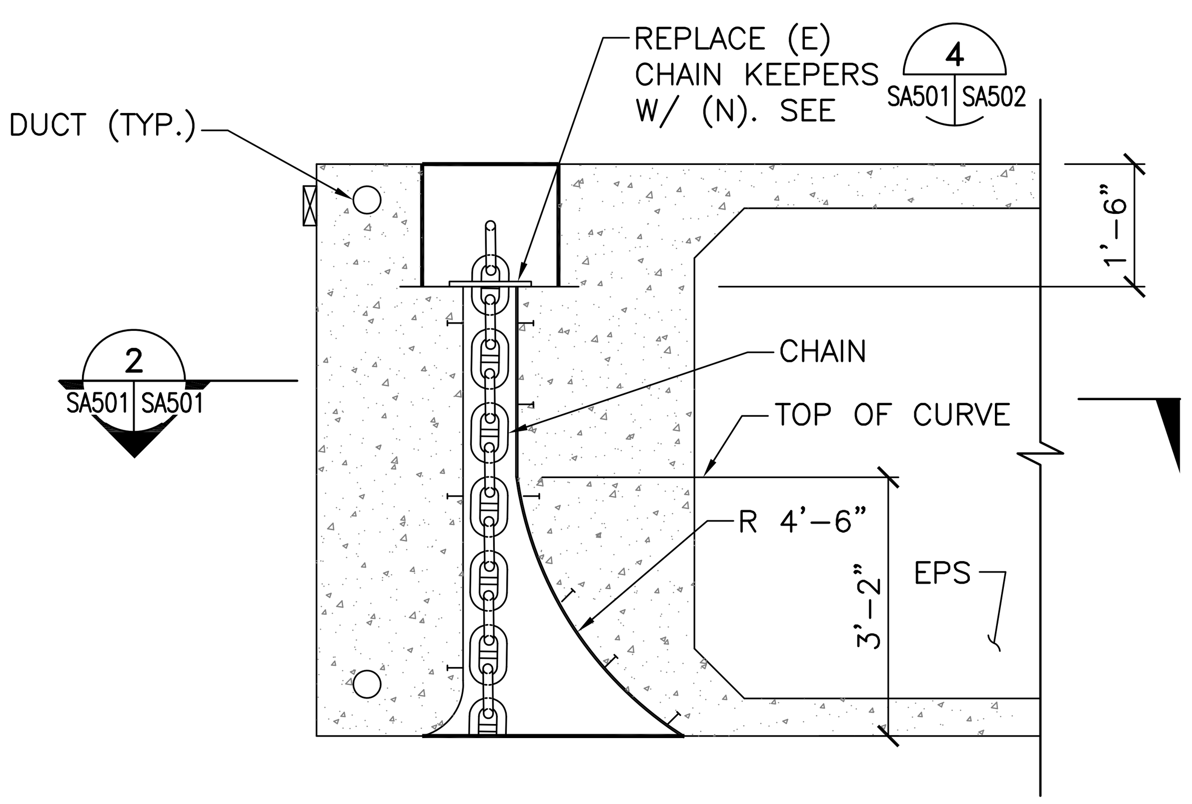
PLAN – EXISTING ANCHOR HAWSE
2 SA102 | SA501

SCALE: 1/2" = 1'-0"



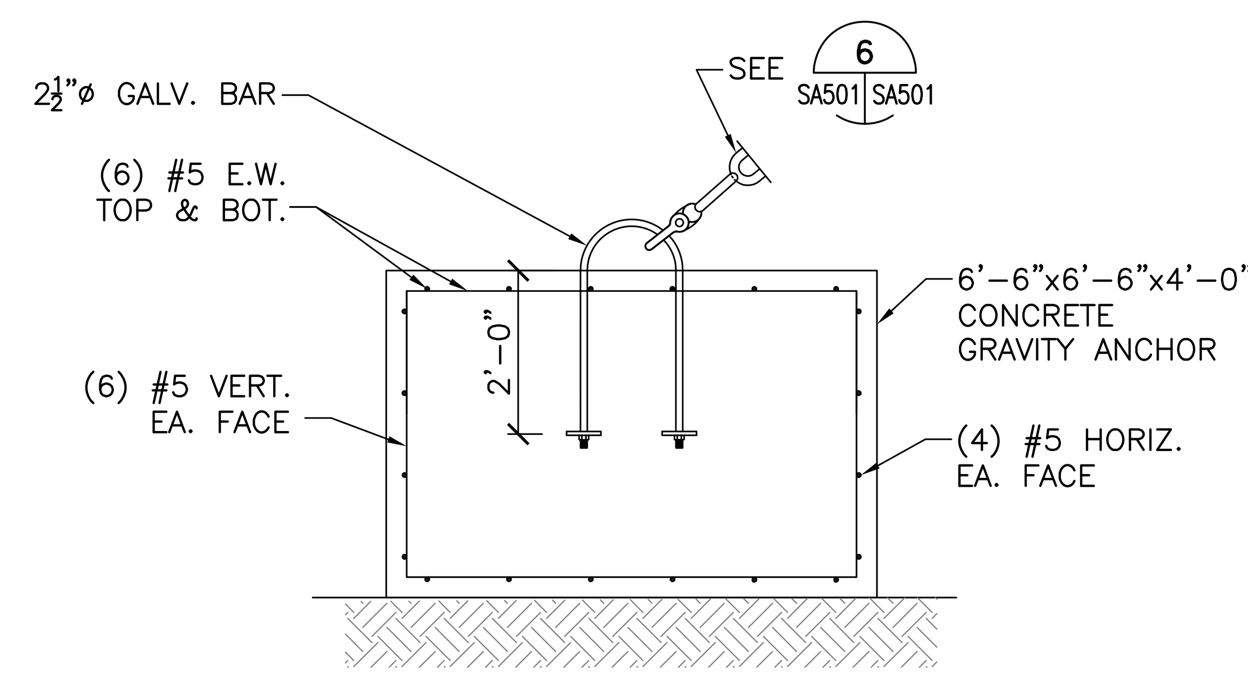
DETAIL – EXISTING LOG BOOM GRAVITY ANCHOR
4 SA101 | SA501

SCALE: 1/2" = 1'-0"



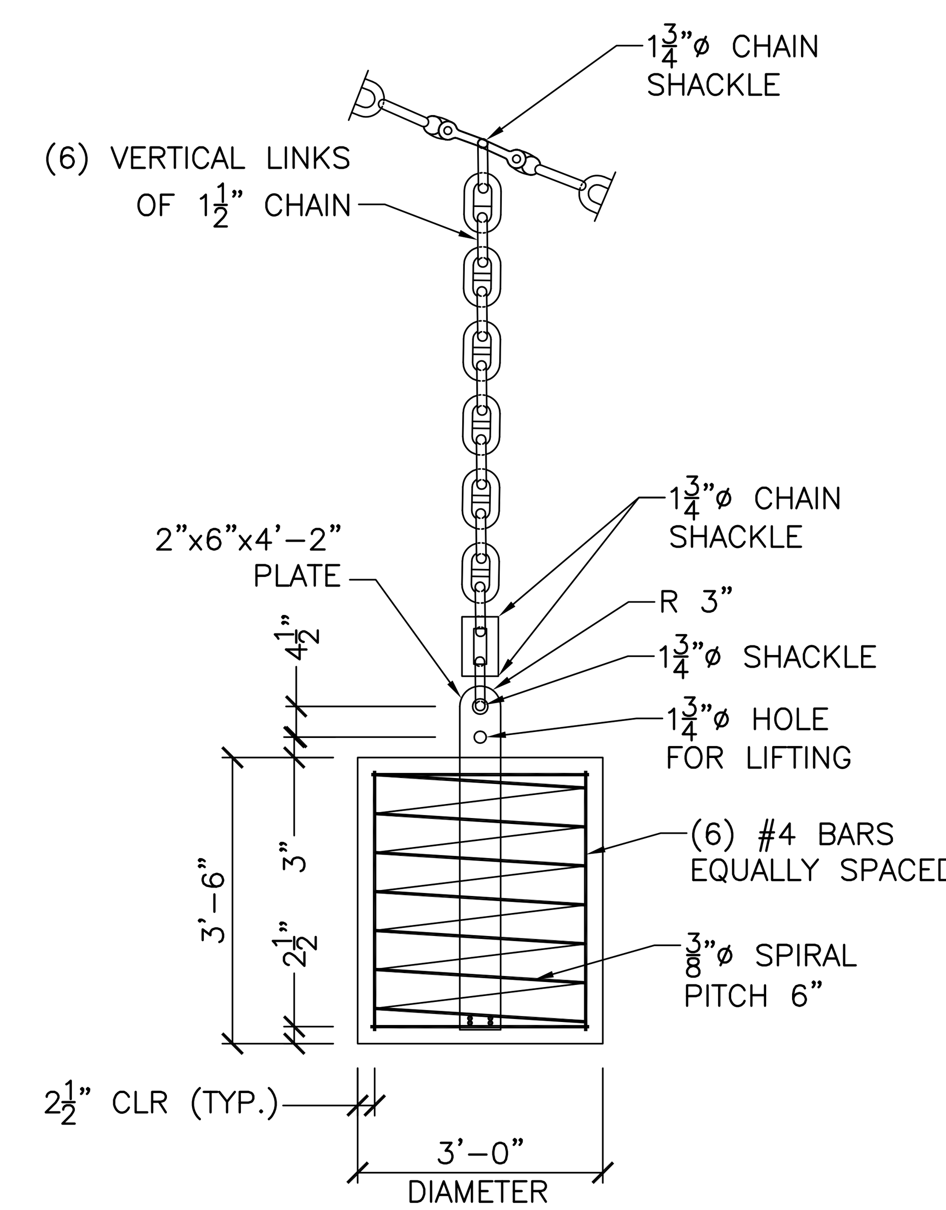
SECTION – EXISTING ANCHOR HAWSE
3 SA501 | SA501

SCALE: 1/2" = 1'-0"



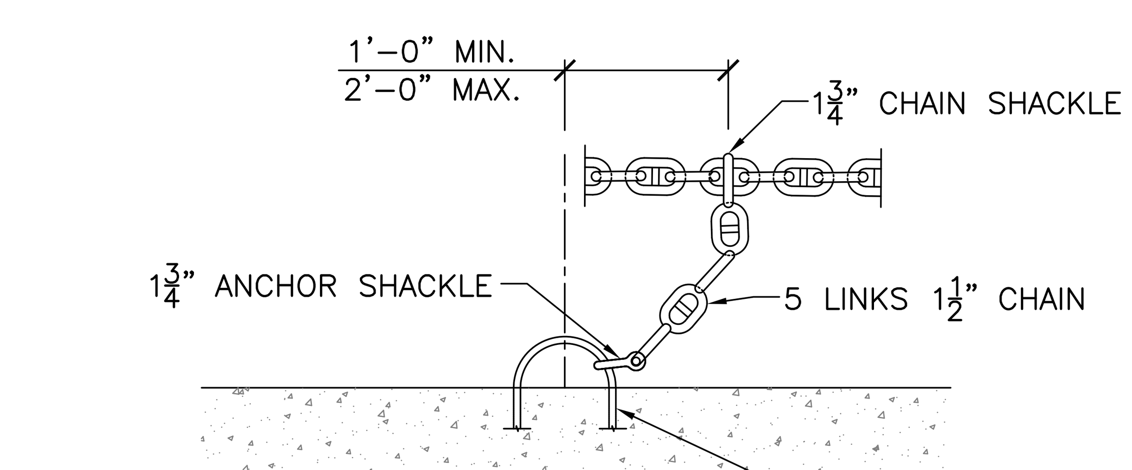
DETAIL – EXISTING DRAG EMBEDMENT GRAVITY ANCHOR
5 SA101, SA102, SA301 | SA501

SCALE: 1/2" = 1'-0"



DETAIL – EXISTING CLUMP WEIGHT
8 SA101, SA301 | SA501

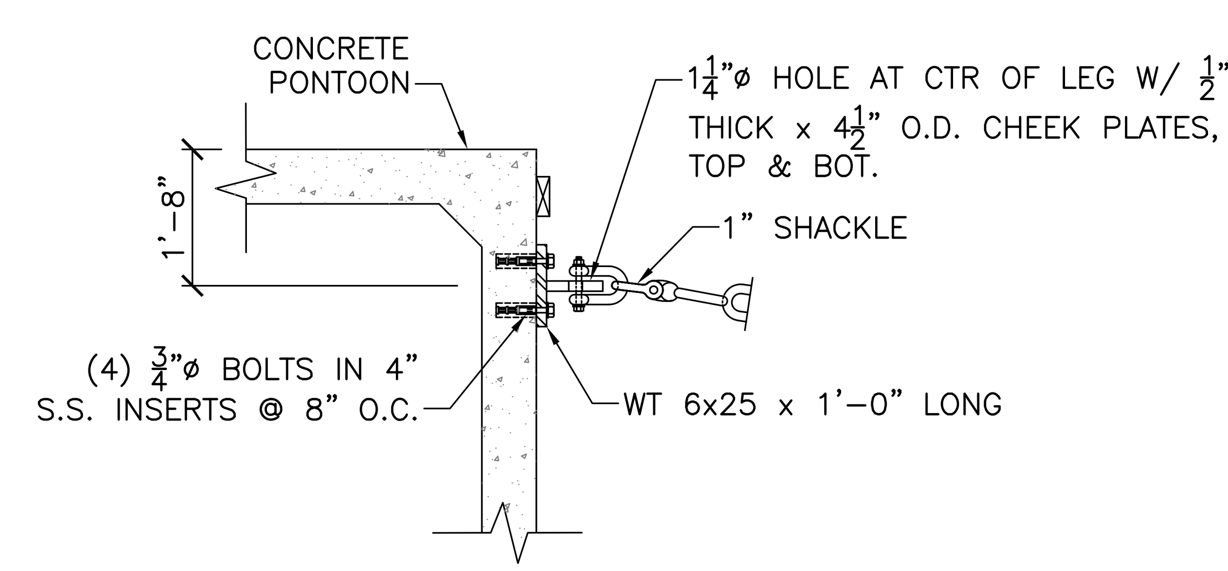
SCALE: 1/2" = 1'-0"



NOTE:
AT LINE 11 & 12 ANCHORS, MOORING CHAINS ARE CONNECTED DIRECTLY TO GRAVITY ANCHOR WITH SHACKLE THRU CHAIN LINK.

DETAIL – EXISTING CHAIN CONNECTION
6 SA102, SA501 | SA501

SCALE: 1/2" = 1'-0"



DETAIL – EXISTING LOG BOOM CONN. @ PONTOON
7 SA102 | SA501

SCALE: 1/2" = 1'-0"

RECORD DRAWING
DATED – 11/11/2015

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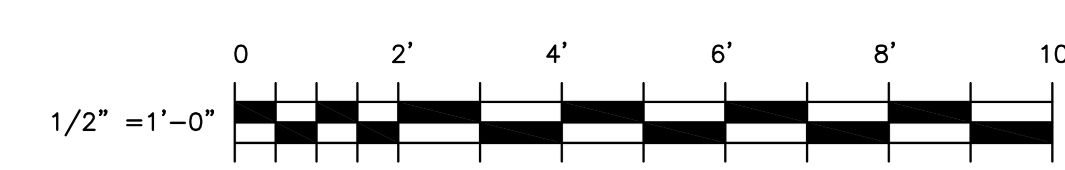

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	11/11/15	RECORD DRAWINGS

A/E PROJECT NO:	4138942
CAD FILE NAME:	4138942SA501.DWG
DESIGNED BY:	CDD
DRAWN BY:	ESN
EDITED BY:	ESN
CHECKED BY:	RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(E) WAVE ATTENUATOR DETAILS

REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
PROJECT ENG.	BRANCH CHIEF	TECH. DIRECTOR

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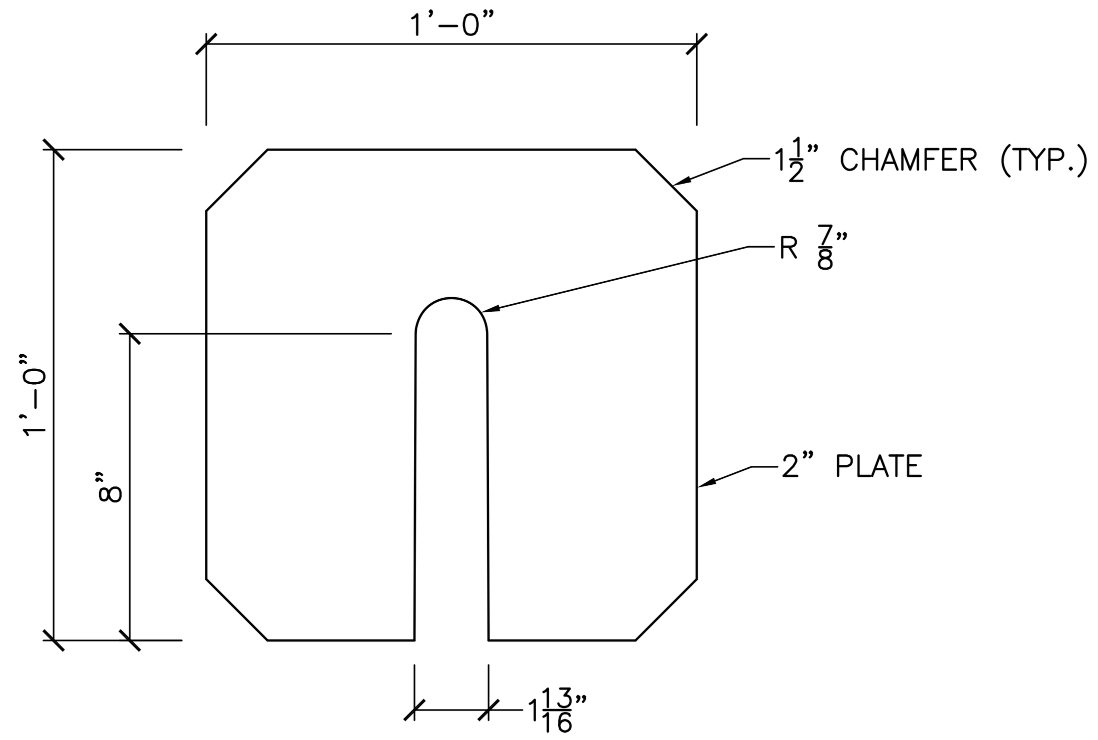
PROJECT NUMBER	DRAWING NUMBER
4138942	4138942SA501

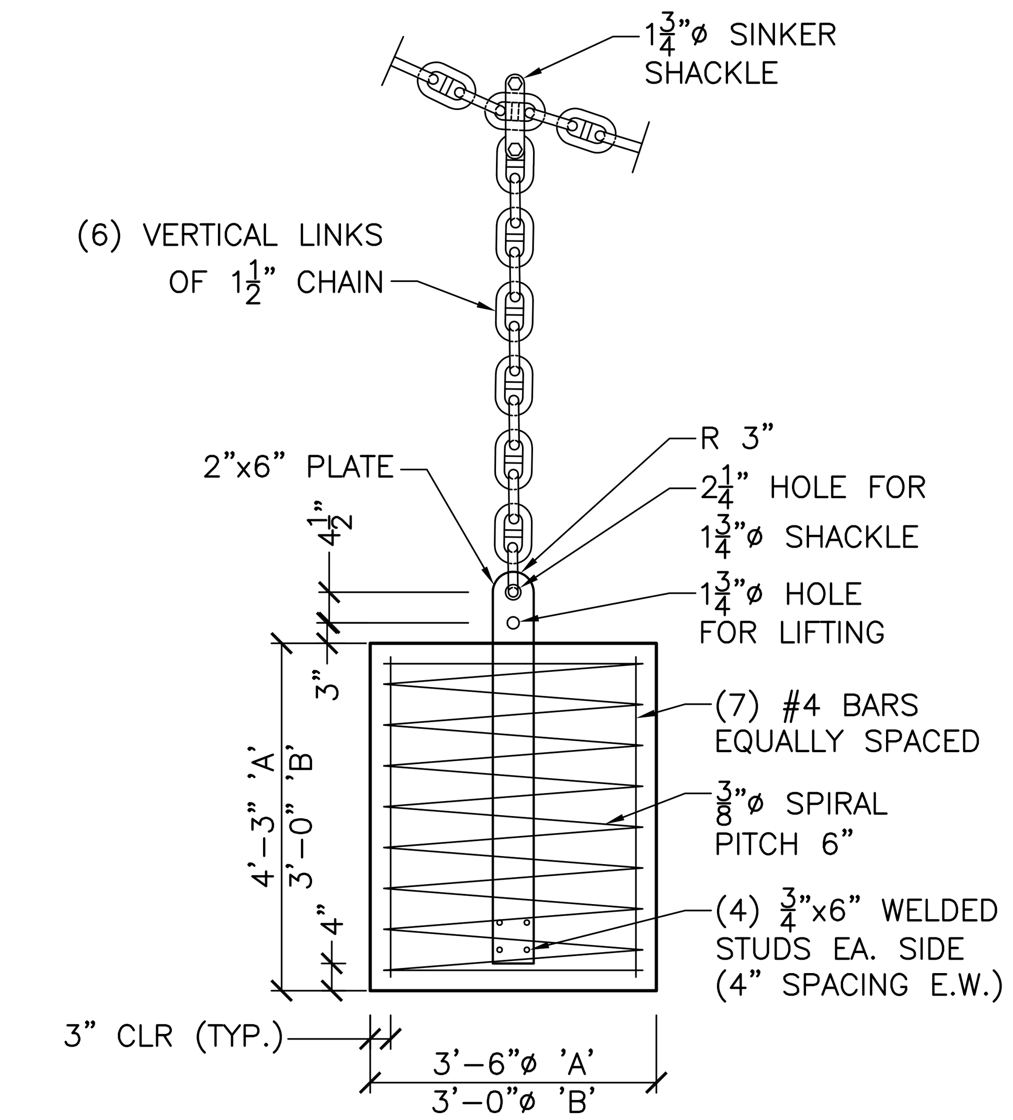
DISCIPLINE/SHT NO
SA501



[PROVIDE (12) NEW KEEPERS]

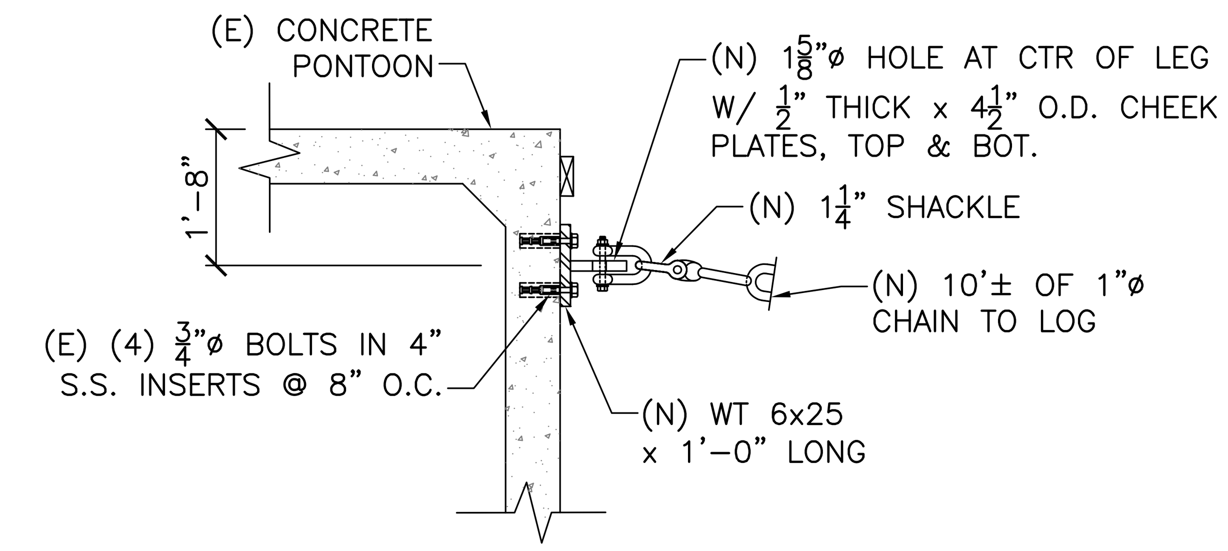
CHAIN KEEPER DETAIL 4 SA501 | SA502

SCALE: 3" = 1'-0"



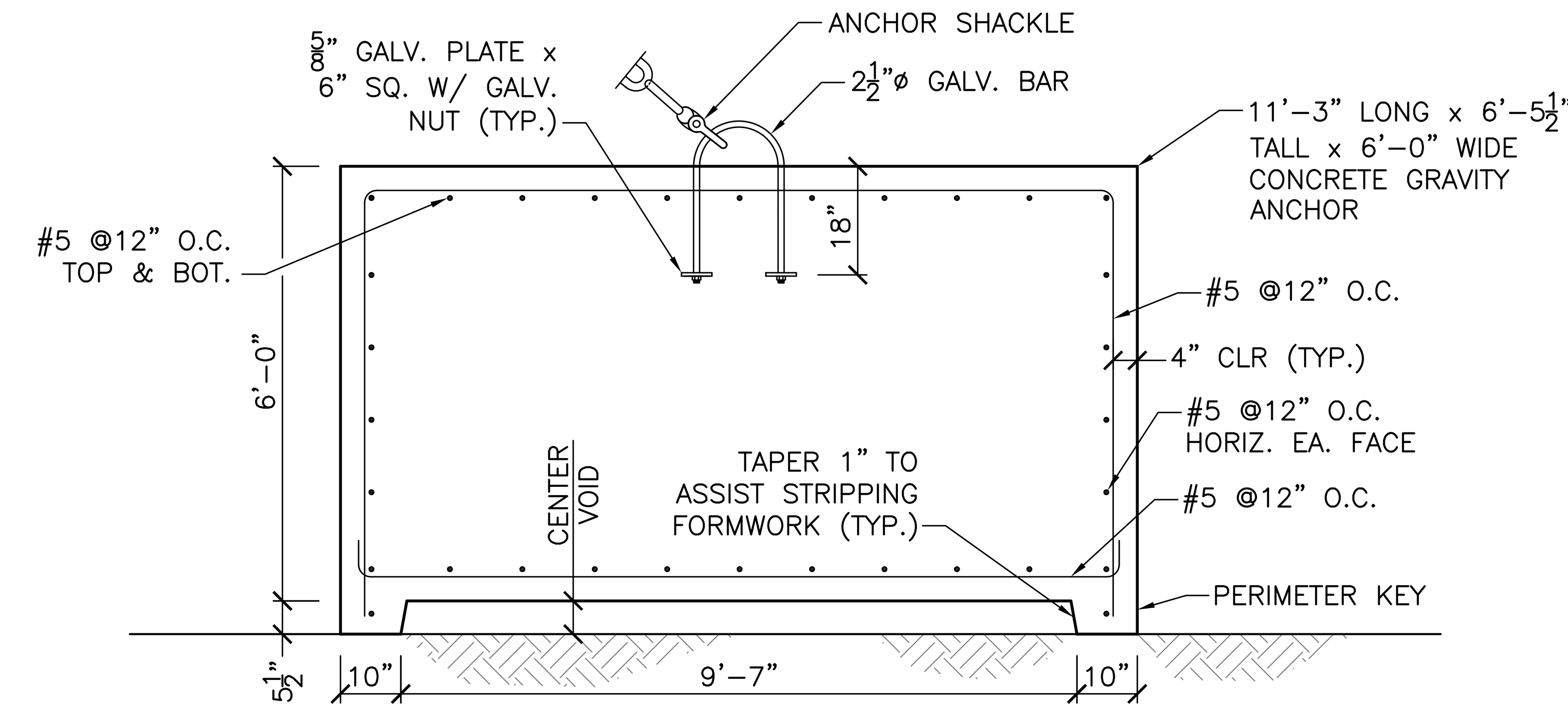
CONCRETE CLUMP WEIGHTS 3 SA103 | SA502

SCALE: 1/2" = 1'-0"



LOG BOOM CONN TO FLOAT 1 SA103 | SA502

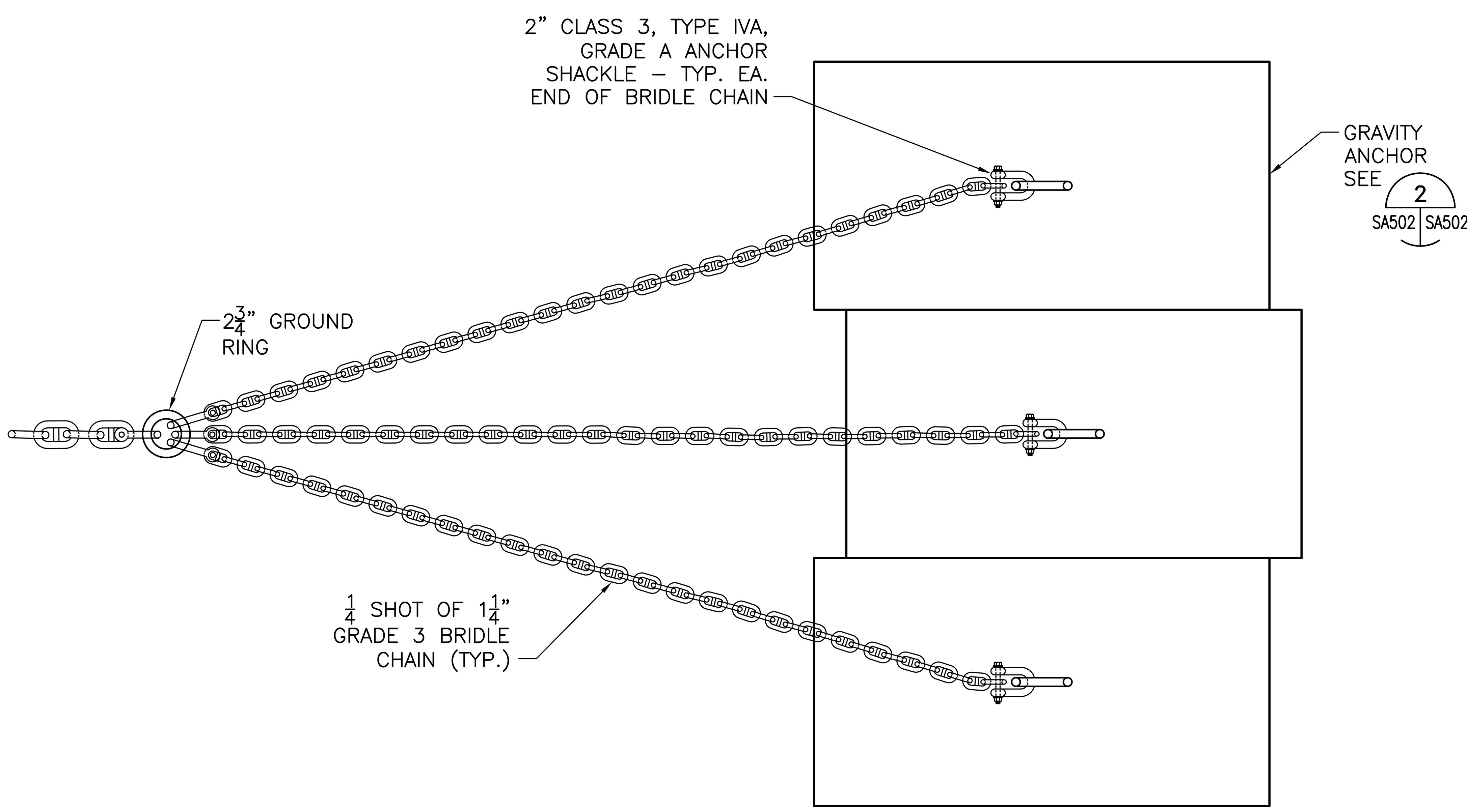
SCALE: 1/2" = 1'-0"



[PROVIDE (34) TOTAL]

CONCRETE GRAVITY ANCHOR 2 SA103, SA502 | SA502

SCALE: 1/2" = 1'-0"



GRAVITY ANCHOR BRIDLE PLAN 5 SA302 | SA502

SCALE: 3/8" = 1'-0"

RECORD DRAWING
DATED – 11/11/2015

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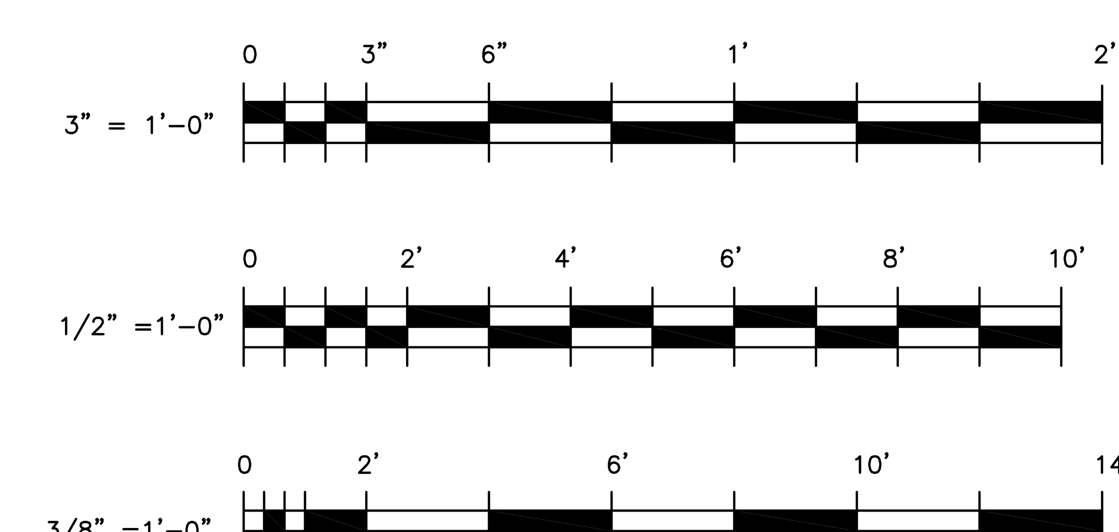

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SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
NEW WAVE ATTENUATOR DETAILS

REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
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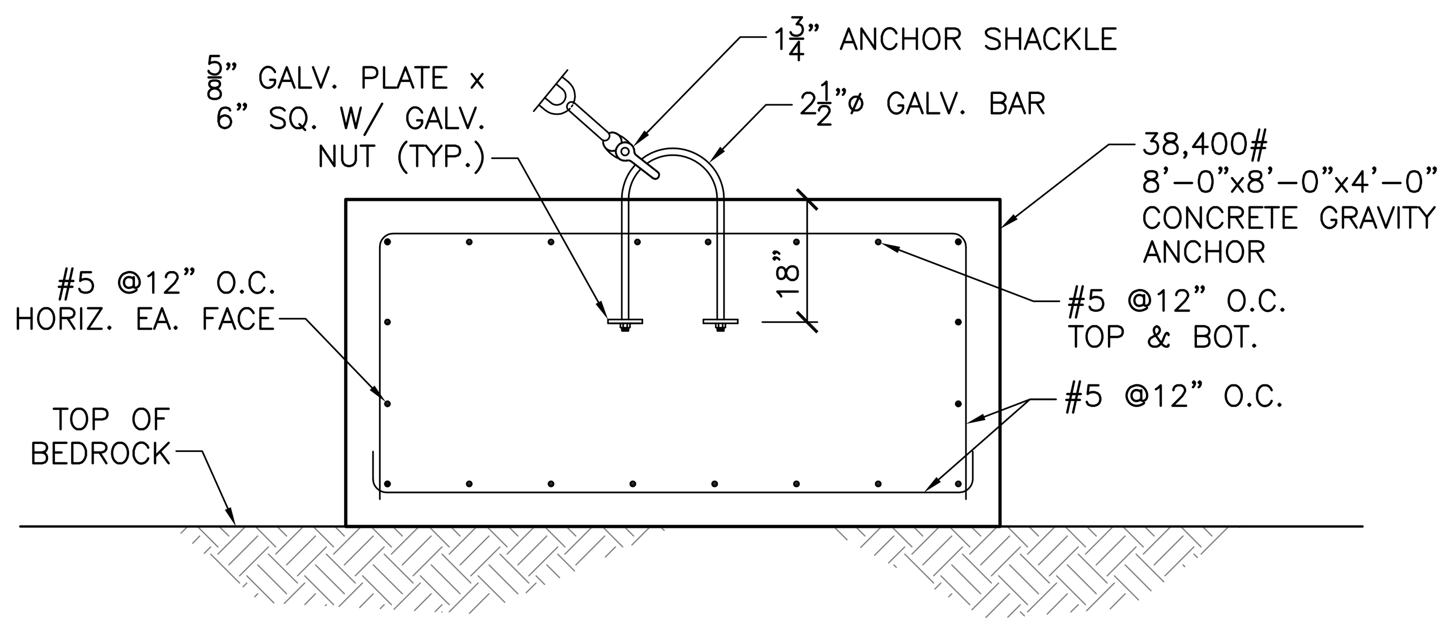
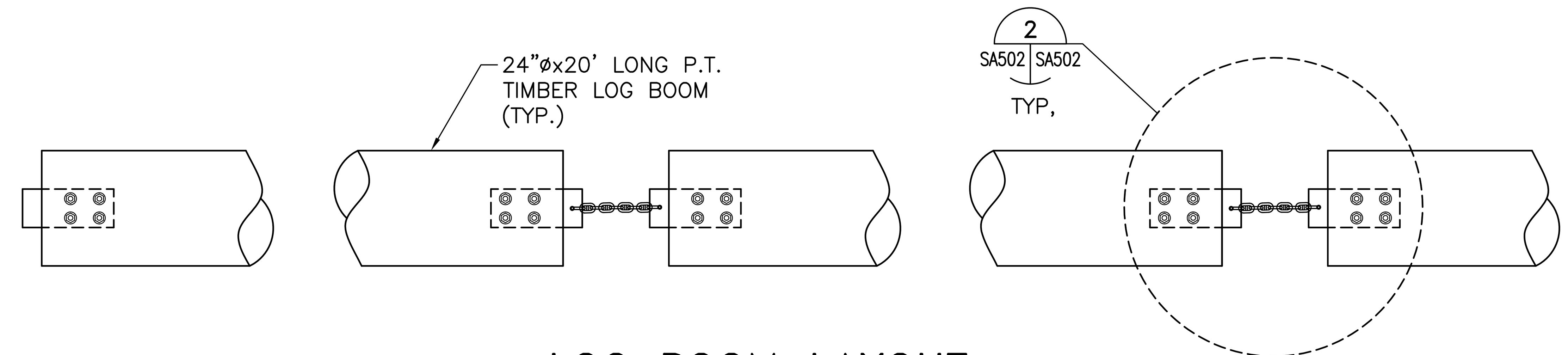
APPROVING OFFICER | DATE

PROJECT NUMBER: 4138942
DRAWING NUMBER: 4138942SA502
DISCIPLINE/SHT NO: SA502
SHEET 84 OF 230



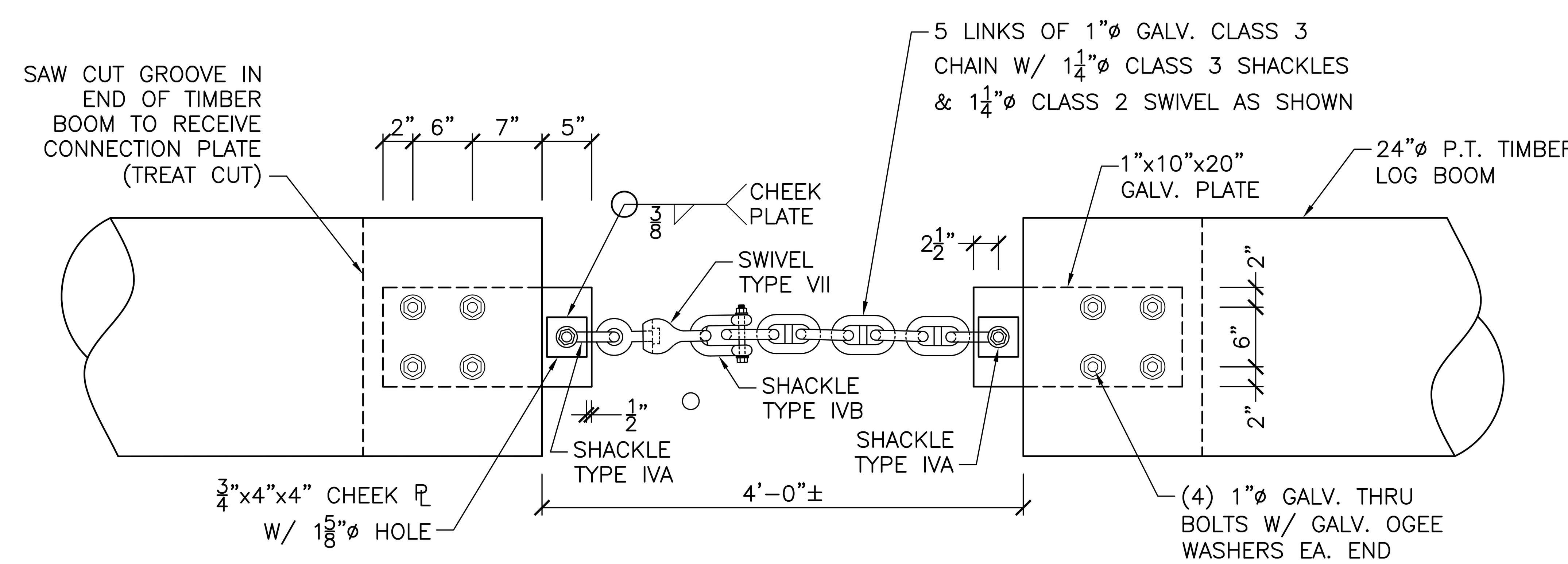
LOG BOOM GRAVITY ANCHOR
3 SA103 | SA503

SCALE: 1/2" = 1'-0"



LOG BOOM LAYOUT
1 SW101,SA103 | SA503

SCALE: 1/2" = 1'-0"

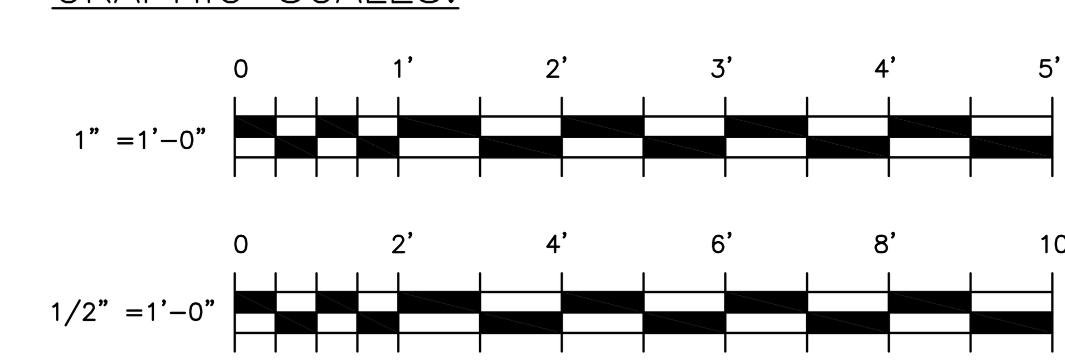


LOG BOOM CONNECTION DETAIL
2 SA503 | SA503

SCALE: 1" = 1'-0"

RECORD DRAWING
DATED – 11/11/2015

GRAPHIC SCALES:



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EDITED BY: ESN
CHECKED BY: RWS

SCALE: AS INDICATED PLOT SCALE: 1 : 1

SHEET TITLE

FRC HOMEPORT UPGRADES
BASE KETCHIKAN
KETCHIKAN ALASKA
WATERFRONT
STRUCTURAL
(N) LOG BOOM DEBRIS BARRIER

REVIEWED BY:	REVIEWED BY:	REVIEWED BY:
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PROJECT NUMBER: 4138942
DRAWING NUMBER: 4138942SA503
DISCIPLINE/SHT NO: SA503
SHEET 85 OF 230

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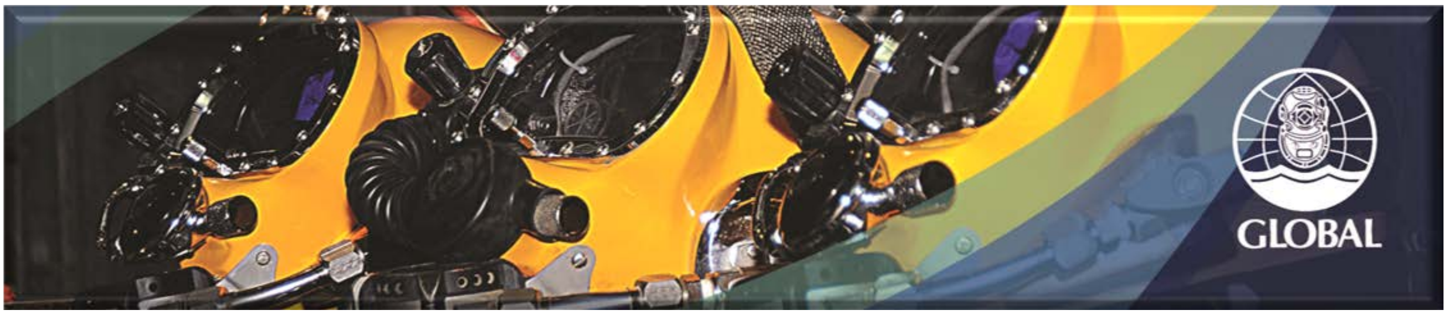
CONCRETE FLOAT BREAKWATER INSPECTION
CITY AND BOROUGH OF JUNEAU, HARBOR MAINTENANCE
Juneau, Alaska



Submitted To:

City & Borough of Juneau
Docks & Harbors
76 Egan Drive
Juneau, Alaska 99801
907-586-0398

Submitted By:
Global Diving & Salvage Inc.
5304 Eielson Street
Anchorage, AK 99518
907-563-9060

CONTENTS

1. INTRODUCTION

On September 27, 2023, Global Diving & Salvage mobilized a four-man dive team onboard the DSV "Ashley T", from our dock facility in Auke Bay, Alaska. A shallow air diving system with a digital underwater video recording system and specialized tooling were setup on the Ashley T to complete the scope of work as listed below. All work was completed by request per the current, Harbor Maintenance Term Contract.

SCOPE OF WORK

- Inspection of a concrete breakwater previously owned by the USCG, to support consideration of purchase by City of Juneau Docks and Harbors group.
    - Inspection was made in coordination with PND Engineers, Inc. who observed the video feed from the diver and directed the diver to examine certain areas of the float more closely.
- Provide a list of deficiencies noted, as well as the dive videos with audio in electronic format.

The inspection was considered a general assessment swim through. No non-destructive testing was performed during this inspection. The diver swam the perimeter of the float, paying particular attention to the chamfered edges, and where the mooring chain goes through the structure.

All diving activities were performed in accordance with the following regulations and industry guidance publications. Global personnel and their subcontractors follow the strictest requirement on the work site.

- Occupational Safety and Health Administration (OSHA) Construction Industry Standards, 29 CFR 1926
- Occupational Safety and Health Administration (OSHA) General Industry Standards, 29 CFR 1910
- Occupational Safety and Health Administration (OSHA) Commercial Diving Standards 29 CFR Part 1910, and Subpart T
- Occupational Safety and Health Administration (OSHA) Hazardous Waste Operations and Emergency Response, 29 CFR 1926.65 or 29 CFR 1910.120
- United States Coast Guard (USCG), 46 CFR 197, Subpart B
- ADCI (Association of Diving Contractors International), Industry Standards, 6th Edition

Prior to beginning diving operations, an onsite safety meeting was held to familiarize the crew with the scope of work and any hazards that may exist. The crew boat schedule for the day was noted along with potential weather hazards.

2. GENERAL FLOAT CONDITIONS

2.1 *Work Location and Operating Conditions*

The concrete breakwater is a marine structure that was moored at the time of inspection off Norway Point, located 1.66 miles from the Wayside Park Float. The 24 foot by 180-foot concrete breakwater was previously owned by the USCG and was installed in Ketchikan. The City of Juneau Docks and Harbors group is considering the purchase of the breakwater from a contractor.

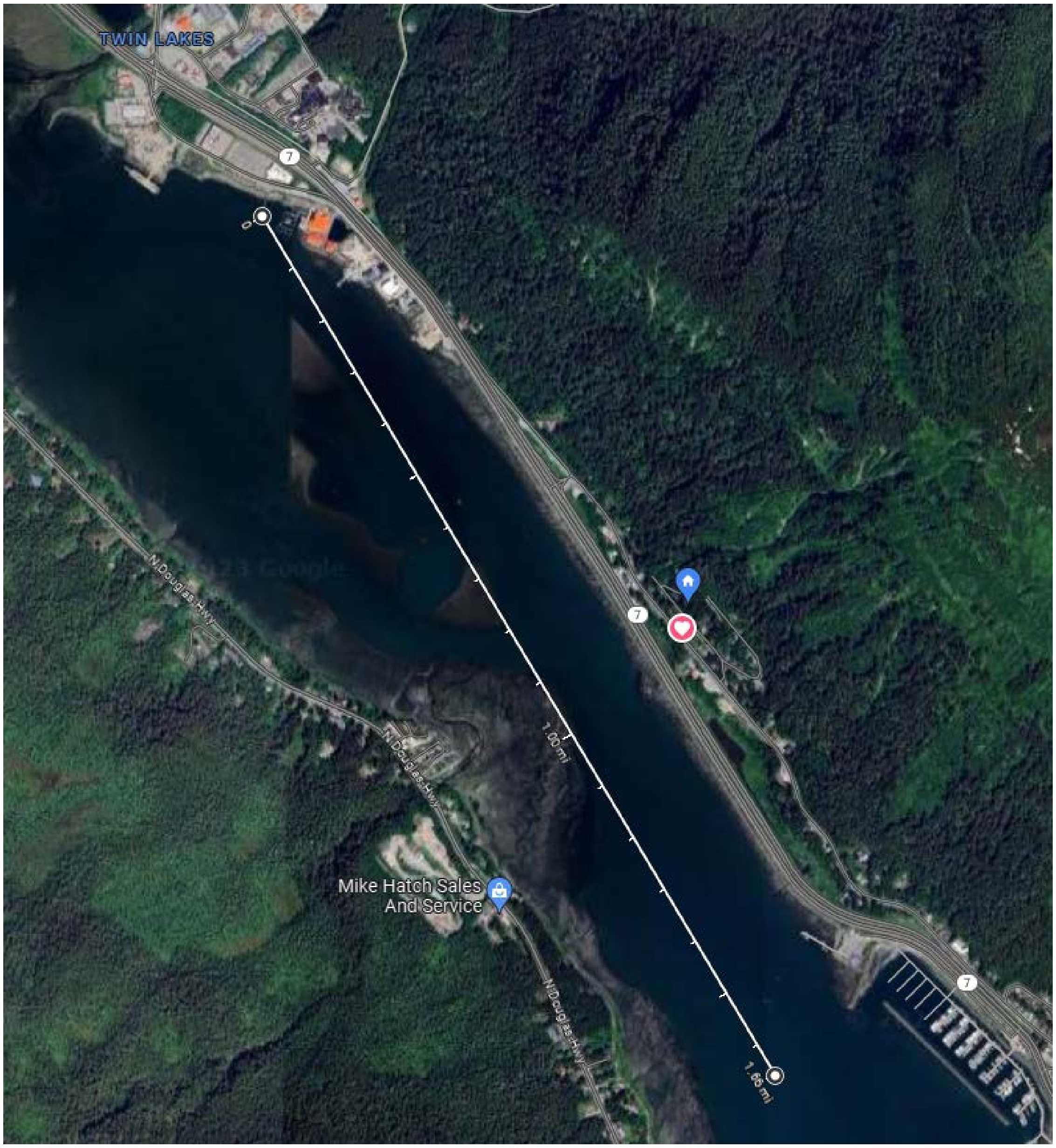


Image 1 – Current location of concrete breakwater.

Weather conditions during the inspection were overcast with light veritable winds, and calm water in the area of the inspection. Due to the recent continuous rain, visibility was affected by a surface layer of fresh and saltwater ‘brine’ mix which produces a layer of water that is milky and fuzzy to see through, additionally run off from shore added to the suspended particulars, limiting visibility during the inspection to 1 to 3 feet of water.

2.2 *General Float Conditions*

The float appears to be in excellent condition, with no signs of damage. The perimeter has a 1” chamfered edge around the bottom of the structure. With the exception of light marine growth and typical rust staining, the breakwater is in “as-built” condition.

No discrepancies were found between the plans provided and the dive inspection.

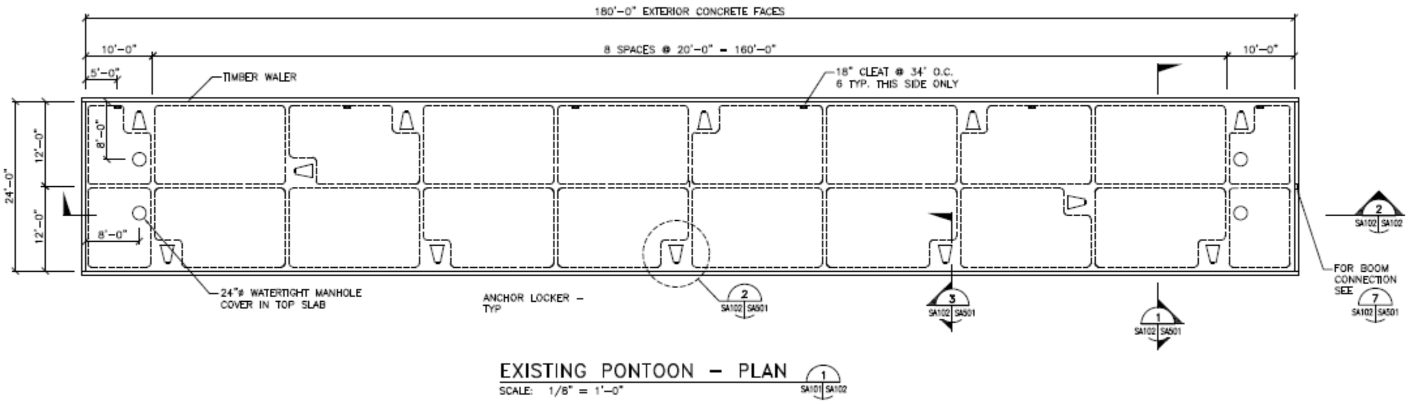


Figure 1 – Breakwater design.

Image 2 – Light marine growth with tubeworms throughout the surface.

Image 3 – Typical chain houser, very good condition with no damage noted.