CARMEL AREA WASTEWATER DISTRICT
BOARD OF DIRECTORS
SPECIAL BOARD MEETING
3945 Rio Road, Carmel, CA 93923 (831) 624-1248

NOTICE & AGENDA
10:45 a.m. Thursday, December 27, 2018

CALL TO ORDER - ROLL CALL

D'Ambrosio_____Rachel______Siegfried _____Townsend______White _____

1. **Appearances/Public Comments:** *Anyone wishing to address the Board on a matter not appearing on the agenda may do so now. Public comment shall be limited to 3 minutes per person per topic. No action shall be taken on any item not appearing on the agenda. During consideration of any agenda item, public comment shall be limited to 3 minutes per person per topic and will be allowed prior to Board action on the item under discussion. Note: if you believe you possess any disability that would require special accommodations in order to attend this meeting, please call the Carmel Area Wastewater District at 831-624-1248.*

2. **Agenda Changes**: Any requests to move an item on the agenda will be considered at this time.

RESOLUTIONS/ORDINANCES:

3. **Resolution No. 2018-55:** A Resolution Authorizing the General Manager to Execute a Professional Services Agreement with Kennedy/Jenks Consultants in an amount not to Exceed $116,387 for Engineering Services Pertaining to Lagoon Crossing Structural Repairs. *Staff report by Patrick Treanor*

4. **ADJOURNMENT:** *The next regular meeting will be held at 9:00 a.m., Thursday, January 31, 2019 or an alternate acceptable date, in the Board Room of the District Office, 3945 Rio Road, Carmel, CA 93923.*

STAFF REPORT

To: Board of Directors

From: Patrick Treanor, Plant Engineer

Subject: Lagoon Crossing Structural Repair Design

Date: December 27, 2018

RECOMMENDATION

It is recommended that the Board of Directors adopt a resolution authorizing the General Manger to execute a contract for $116,387 for design of repairs to the lagoon crossing structure.

DISCUSSION

CAWD Staff recently engaged Kennedy/Jenks Consultants to develop structural calculations for the existing lagoon pipe crossing structural elements including the piping and individual structure elements. The results of that analysis are in the attached report titled "Outfall Pipeline Bridge Evaluation" dated 14 December 2018. The structural analysis concluded that the existing cross beams should be repaired as soon as is feasible. One set of two existing piles was also found to be vulnerable to failure in a 100-year flood event according to the structural loading calculations. The two vulnerable piles are located in the deepest part of the lagoon and therefore have greater exposure to bending forces than the other piles which are in shallower water.

In order to maintain ongoing operation of the lagoon crossing structure and to mitigate risk it is recommended that the District proceed with design so that we proactively maintain this asset. The design process will help to reveal the best alternative for repair work at this time. Kennedy/Jenks has provided a proposal for design that includes repairs to address both the cross beams and the tallest piles, however the actual project description will not be fully defined until design is well underway and the options to improve these elements are further evaluated.

Staff anticipates that the construction of this project will fall under an emergency project due to the circumstances and the risk of a potential spill in the lagoon. Therefore, the Kennedy/Jenks proposal includes services during construction under emergency project implementation. A recommendation for emergency implementation of construction will be brought to the board for approval once design is near completion.

FUNDING

The funding for this contract will come from existing capital reserves.

ATTACHMENTS

1. Kennedy/Jenks proposal dated 21 December 2018
2. Outfall Pipeline Bridge Evaluation Report dated 14 December 2018

RESOLUTION NO. 2018-55

A RESOLUTION AUTHORIZING THE GENERAL MANAGER TO EXECUTE A PROFESSIONAL SERVICES AGREEMENT WITH KENNEDY/JENKS CONSULTANTS IN AN AMOUNT NOT TO EXCEED $116,387 FOR ENGINEERING SERVICES PERTAINING TO LAGOON CROSSING STRUCTURAL REPAIRS

-oOo-

WHEREAS, the District is committed to proactively maintaining existing lagoon crossing structural assets to maintain service; and

WHEREAS, Kennedy/Jenks has submitted a scope of work and fee proposal agreeable to the Carmel Area Wastewater District General Manager to design and provide engineering services during construction of a project to conduct repair work on the existing lagoon crossing structure;

NOW, THEREFORE, BE IT RESOLVED by the Board of Directors of the Carmel Area Wastewater District that it does hereby authorize the General Manager to enter into a professional service agreement with Kennedy/Jenks Consultants for engineering services pertaining to lagoon crossing structural repairs in an amount not to exceed $116,387.

PASSED AND ADOPTED at a regular meeting of the Board of Directors of the Carmel Area Wastewater District duly held on December 27, 2018, by the following vote:

AYES: BOARD MEMBERS:

NOES: BOARD MEMBERS:

ABSENT: BOARD MEMBERS:

____

President of the Board

ATTEST:

____

Secretary of the Board

Kennedy/Jenks Consultants

Engineers & Scientists

2350 Mission College Boulevard, Suite 525
Santa Clara, CA, 95054
Office PH: (650) 852-2800

21 December 2018

Mr. Patrick Treanor, P.E.
Plant Engineer
Carmel Area Wastewater District
3945 Rio Road
Carmel, CA 93922

Subject: Proposal for Professional Engineering Services
Design Services for Emergency Outfall Pipeline Bridge Bent Repair Project
K/J B106810

Mr. Patrick Treanor,

We are pleased to submit this proposal for providing professional engineering services for design, contracting support, and construction phase services for the Emergency Outfall Pipeline Bridge Bent Repair Project.

This proposal is to develop contract documents for bid and construction of the emergency repairs for the 24-inch-diameter outfall pipeline bridge bents. We have enclosed our project understanding, scope of services, and budget and we will mobilize the engineering resources necessary to complete this project. We can begin work as soon as we receive a signed contract and a notice to proceed.

Terms and Conditions

This proposal is based on current projections of staff availability and costs and, therefore, is valid for 90 days following the date of this letter. To assure a clear understanding of all matters related to our mutual responsibilities, the attached Standard Conditions dated 1 Jan 2018 are made a part of our agreement (part of Exhibit B). If this proposal meets with your approval, please prepare a contract using this proposal, fee and schedule of charges for our signature.

We look forward to continuing to work with you and your staff. Should you have any questions, please feel free to call me at (406) 578-4501.

Very truly yours,

KENNEDY/JENKS CONSULTANTS

James Bowland

James Bowland P.E.
Project Manager

Peter C. Talbot

Peter C.Talbot P.E.
Vice President

Project Understanding

In preparing this scope of work, we have the following project understanding based upon our discussions with District staff and our previous work with the District in preparing the structural evaluation of the outfall pipeline bridge. This project is needed to repair the pile bent supports of an existing pile supported lagoon outfall pipe crossing that has been found to be in very poor condition. The key elements of our understanding are listed below:

1. The District intends to promptly initiate a project to repair/strengthen the outfall structure elements that were identified as having potential risk of failure (i.e. the pile bent cross beams and hardware and the middle-bent piles).
2. There will not be any repairs to the outfall pipeline.
3. The District understands that feasible repair options for the damaged elements of the pile bents and insufficient piles will be identified and designed.
4. The District intends to bid this work under an emergency bid process given the outfall structure condition and its potential impact on wastewater treatment plant operations.
5. Kennedy/Jenks will provide engineering services for the design and technical support during construction .
6. The District will administer the bid (contacting contractors for informal bids and drafting the contract). The District will prepare any procurement and contracting documents.
7. Kennedy/Jenks will prepare Division 1 General Requirements and Technical Specifications in Divisions 2 through 17.
8. The District plans to complete the design within approximately 1 month and will plan to informally bid this project early next year for construction in early 2019.
9. The project construction will generally include –
    a. Demo, removal and disposal of deteriorated elements of the bent supports;
    b. Design of emergency repairs for the pile bents including pile bent cross beams and hardware;
    c. Design of pile repairs or supplemental piles for support of the pile bents;
    d. If necessary, removing one or more of the existing concrete filled steel piles.
10. It is understood that the design fee is based on T&M. If the scope of work changes from the description in Item 9, the fee will be negotiated to reflect the new scope of work.
11. The District will identify and provide a summary of environmental constraints to Kennedy/Jenks, which will include items such as any special environmental controls.
12. No topographic survey information collection is needed to support this project.
13. Geotechnical input is needed for design criteria for replacement concrete filled steel pipe piles in the lagoon, we have budged for GTC to support this effort.
14. There will be no work associated with identifying hazardous materials or preparing contracting requirements for removal and disposal of hazardous materials.
15. The District will handle public outreach and coordination of the site access.

Scope of Services

Based on the above understanding, the following is a description of our proposed scope of services:

Task 1 – Project Management, QA/QC

Task 1 will consist of the day to day management of the project including the following items:

Task 1.1 – Meetings

Kennedy/Jenks will facilitate the following meetings with the District:

a. One kick off meeting: 2-hr meeting via conference call
b. Design Review Meeting at 90% submittal phase. (one 2-hour meeting)

Task 1.2 – Project Management

Kennedy/Jenks will provide project management focused on managing project costs, meeting the District's schedule requirements, and communicating key issues to the District. A project file will be maintained, that will include copies of correspondence, reports, summaries of meetings, and memoranda.

Task 1.3 – Quality Assurance/Quality Control (QA/QC)

Quality assurance reviews will be performed in accordance with Kennedy/Jenks' standards. Senior technical engineers with a background in structural design will perform technical review of the project's concepts and criteria and will review each deliverable before submission to the District.

Task 1 Deliverables and District Responsibilities

Deliverables:

- Meeting agendas and meeting summaries (electronic copy),
- Monthly progress reports and invoices.

District Assistance

- Attend meetings
- Retain an environmental consultant for CEQA and permitting services

Task 2 – Design

Task 2.1 – 90% Design

The 90% Design will be developed containing all the drawings and details and technical specifications. During this task the geotechnical firm will consulted for input regarding replacement pile design. The 90% submittal will be provided to the District to review prior to issuing a request for informal bids from select contractors. These deliverables will be near bid ready and contain near final drawings, final specifications, and a cost estimate.

Deliverables:

- Division 1 and 2 through 17 technical specifications near final (electronic copy),
- 90% Drawings (near final), and (electronic PDF copy)
- 90% level Cost Estimate, AACE Class 2 (electronic PDF copy)

District assistance:

- Identify and provide a summary of environmental constraints to Kennedy/Jenks, which will include items such as any special environmental controls.
- Identify any constructability, public outreach and site access requirements to be placed on contractor.

The estimated list of design drawings and specifications are enclosed as Exhibit A - List of Drawings and Specifications and serve as the basis for estimating our design effort. Deliverables will be submitted in electronic PDF form

Task 2.2 - Final Design (informal bid documents)

Kennedy/Jenks will produce final documents that address the District's comments on the 90% submittal. Following a final quality control review, we will provide final signed design documents to CAWD for issuing for obtaining informal bids from select contractors. The final design deliverable will include complete signed Contract Documents design drawings, front end specifications, and technical specifications.

Deliverables:

- Signed and stamped drawings and specifications. One full size hard copy of the drawings and ½ and full size PDFs of the drawings and specifications.

District assistance:

- Provide and pay for reproduction and distribution of plans and specifications.

Task 3 – Bid Phase Assistance

Task 3 includes assisting the District in obtaining informal bids from a prequalified set of contractors. This task includes the following items:

Task 3.1 – Pre-Bid Meeting

The Kennedy/Jenks Design Project Manager will assist the District as needed in a pre-bid meeting and then a site walk for coordination with a select number of bidders.

Task 3.2 – Prepare Addendum

Kennedy/Jenks will respond to bidders' questions during the bid phase and provide additional contract document clarifications via one addendum. We assume that the District will serve as the point of contact during the bid phase and will compile Contractor questions and deliver them to Kennedy/Jenks for response in the addendum.

Task 3.3 – Conformed Drawings

Kennedy/Jenks will prepare conformed drawings and specifications. This package will include any changes from the addendum phase and integrate those changes into one set of drawings and specifications.

Deliverables:

- Up to one addendum: One PDF copy to plan room for distribution.
- One PDF and one hard copy set of conformed drawings and specifications.

District assistance:

- Administer the bid (contacting contractors for informal bids and drafting the contract).
- Prepare any procurement and contracting documents.
- Assist in facilitating the pre-bid meeting and site walk.
- Printing of bid documents
- Distribution of bid documents to select bidders, subcontractors, and material suppliers as required.
- Review of bids
- Award the contract.

Task 4 – Construction Phase Assistance

Task 4 includes providing engineering services during construction for the project to perform on-site construction management and office engineering for the project. This scope of services and budget allocation provides an estimate of construction phase assistance in support of Kennedy/Jenks design elements. The estimated level of construction phase assistance and the budget allocations presented herein are the minimum anticipated during the construction period (length of construction period is to be determined). Additional services and an increase in the budget allocations may be required to address the awarded General Contractor's schedule and performance, unforeseen conditions, and issues that surface during construction including time extensions.

Task 4.1 – Construction Management

When requested by the District, Kennedy/Jenks will attend construction coordination meetings with the District and Contractor, or participate in meetings by conference call, that pertain to work elements designed by Kennedy/Jenks. In addition, when requested, Kennedy/Jenks will make periodic visits to the construction site to make construction observations and to review progress of the work elements designed by Kennedy/Jenks for conformity with the Contract drawings and specifications.

The allocated budget provides for attending 2 meetings/site visits or conference calls, at an estimated average of 8 hours per person per site visit or conference call (one to two Kennedy/Jenks team members per meeting/site visit or conference call at 3 hours each, which includes travel, meeting prep time and site visit observation notes).

Task 4.2 – Request for Information/Clarifications

Kennedy/Jenks will assist the District, when requested, in responding to requests for information (RFIs) for project elements designed by Kennedy/Jenks and its subconsultants. Timely responses to RFIs and RFS' will be provided to the District in the form of a type-written response memorandum that can be emailed. The District will be responsible for preparing and issuing the response to the contractor.

The allocated budget provides for up to 4 RFIs/Clarifications/Questions at 3 hours per RFI/Clarification/Question on average for review, preparing a written response or clarification, and conducting a quality control review by a senior level engineer. The estimated number of RFI's is based on approximately 1 RFI/Clarification per design drawing prepared by Kennedy/Jenks. Kennedy/Jenks will respond to RFI's on average within 5 working days. It is assumed that some of the RFI's may need response within 2 working days, and an expected return date to the contractor will be provided by the District.

Task 4.3 – Submittal Review

Kennedy/Jenks will review construction submittals for elements of the project designed by Kennedy/Jenks and its subconsultants. It is assumed that submittals will be received by the District and forwarded electronically to Kennedy/Jenks for review. Submittals will be reviewed for conformity with the plans and specifications requirements. Submittal review comments will be presented in a shop drawing review letter (SDRL) that is returned with each reviewed submittal. Each submittal will be itemized and conformed including mark-ups on fabrication drawings. Kennedy/Jenks will review and return submittals electronically to the District within an average of 14 calendar days.

Task 4.4 – Potential Change Review

Upon request, Kennedy/Jenks will assist the District in preparing and reviewing potential change order scopes, costs and schedules. Kennedy/Jenks will inform the District of Kennedy/Jenks's initial review and assessment of potential changes before proceeding with engineering. The allocated budget provides for engineering assistance to the District in preparing and reviewing up to 2 potential changes at estimated level of effort of approximately 8 hours per change request for a total of 16 hours.

Task 4.5 – Record Drawings

Kennedy/Jenks will prepare Record Drawings based on marked-up drawings received from the Contractor at the end of construction. The mark-ups will be in the form of a red-line set of contract drawings maintained during construction by the Contractor. The mark-ups are expected to contain clarifications, change order work, and other significant construction revisions. Kennedy/Jenks will not be responsible for verifying the validity of the red-line set. The allocated budget provides for approximately 1.5 hours per design drawing for CAD drafting of red-lined mark-ups provided by District, plus back checking.

Deliverables:

- Construction related correspondence including submittals, RFI responses, clarifications, and potential changes, record drawings and other construction related correspondence.

District assistance:

- Schedule meetings.
- Overall construction management for the project including the inspectors, testing, third party suppliers, etc. that are required for completion of the construction.

- Coordinate jobsite safety requirements for visiting the project site during construction and accessing work areas under construction.

Estimated Schedule

The design and bid project schedule is anticipated to be approximately 6 weeks from notice to proceed to the conclusion of bid services.

The proposed project schedule, assuming notice to proceed is issued on January 2, 2019, is as follows:

Item	Finish Date	Duration
Notice to Proceed with Design	2 January 2019	1 Day
90% Design Submittal	1 February 2019	4 Weeks
District Reviews 90% Design	15 February 2019	1 Week
Final Design Submittal	25 February 2019	1 Week
Bid Meeting, Site Walk, Bids Received	8 March 2019	1 Week
Construction Notice to Proceed	18 March 2019	1 Day
Construction	10 May 2019	8 Weeks

Basis of Compensation

We propose that compensation for our services be on a time and expense reimbursement basis in accordance with our attached Schedule of Charges dated 1 January 2018. Payments shall be made monthly based on invoices which describe services and list actual costs and expenses. Based on our estimate of services required, we propose a not-to-exceed budget of $116,387. The budget summarized in the table below, will cover design, bid phase, and construction phase services. We will notify you prior to beginning any additional scope of work items to negotiate a revised scope and appropriate budget amendment. A detailed fee estimate and January 1, 2018 Schedule of Charges is contained in Exhibit B. The Schedule of Charges will be used through December 31, 2019.

TASKS	ESTIMATED BUDGET
Task 1 – Project Management	
Task 1.1 – Meetings	$5,972
Task 1.2 – Project Management	$3,935
Task 1.3 – QA/QC	$1,267
Task 1 - Subtotal	**$11,173**
Task 2 - Design	
Task 2.1 – 90% Design	$44,918
Task 2.2 – Final Design (Bid Documents)	$14,245
Task 2 - Subtotal	**$59,163**
Task 3 – Bid Phase Services	
Task 3.1 – Pre-Bid Meeting	$3,407
Task 3.2 – Prepare Addendum (Max 1)	$3,718
Task 3.3 – Conformed Drawings	$2,493
Task 3 - Subtotal	**$9,618**
Task 4 – Construction Phase Services	
Task 4.1 – Construction Management	$10,897
Task 4.2 – Request for Information/Clarifications	$12,422
Task 4.3 – Submittal Review	$6,922
Task 4.4 – Potential Change Order Review	$4,614
Task 4.5 – Record Drawings	$1,597
Task 4 - Subtotal	**$36,433**
All Phases Total	**$116,387**

Enclosure(s)

Exhibit A–List of Drawings and Specifications

Exhibit B– Schedule of Charges, and Standard Conditions

Exhibit A

List of Drawings[1]

G01	Title Sheet, Sheet Index, General Notes
G02	Site Access and Staging Areas
S01	Structural General Notes, Special Inspection and Testing Schedule
S02	Pile Bent Demolition and Repair Details
S03	Pile Plan, Section, and Details

[1] List of drawings may change based on actual methods of repairing the pipe bridge

List of Specifications

DIVISION 0 – BIDDING REQUIREMENTS, CONTRACT FORMS, CONTRACT CONDITIONS (not included)

DIVISION 1 – GENERAL REQUIREMENTS

Section 01010	Summary of Work and Contract Considerations
Section 01040	Coordination and Project Requirements
Section 01140	Environmental Protection
Section 01300	Submittals
Section 01500	Construction Facilities and Temporary Controls

DIVISIONS 2 through 17 – TECHNICAL SPECIFICATIONS

Section 03100	Concrete Formwork and Formwork Accessories
Section 03150	Concrete Joints and Accessories
Section 03200	Concrete Reinforcement and Reinforcement Supports
Section 03300	Concrete Mixtures, Handling, Placing and Constructing
Section 03306	Cold Weather Concreting
Section 03330	Concrete Curing and Protection
Section 03350	Concrete Finishes
Section 05090	Fasteners
Section 05100	Structural Metal Framing
Section 09960	Protective Coatings

Exhibit B

Schedule of Charges, and Standard Conditions

Dwg Sht Row | Jan 3 | WIBU |

Proposal Fee Estimate (Assoc. Proj. Costs per Schedule of Charges)

Kennedy/Jenks Consultants

CLIENT Name: Carmel Area Wastewater District

PROJECT Description: Outfall Pipeline Bridge Bent Repairs

Proposal/Job Number: 12/19/2018

January 1, 2018 Rates											KJ	KJ	Sub	KJ	KJ	KJ				
Classification:	Eng-Sci-9	Eng-Sci-8 D. Barraza	Eng-Sci-7	Eng-Sci-6 P. Symonds	Eng-Sci-5 J. Bowland	Eng-Sci-2 K. Castillo	Eng-Sci-1	Sr. CAD-Design N. Bravo	Project Administrator	Total	Labor	Escalation	GTC	Sub-Markup	ODCs	ODCs Markup	Total Labor	Total Subs	Total Expenses	Total Labor + Subs + Expenses
Hourly Rate:	$295	$280	$260	$230	$205	$160	$130	$165	$125	Hours	Fees	3%	Fees	10%	Fees	10%				Fees
Task 1 - Project Management, QA/QC																				
Task 1.1 Meetings		16							2	18	$4,730	$142		$0	$1,000	$100	$4,872	$0	$1,100	$5,972
Task 1.2 Project Management										0	$0	$0		$0		$0	$0	$0	$0	$0
PM Communications		2			2					4	$970	$29		$0		$0	$999	$0	$0	$999
Project Status Updates		2			2					4	$970	$29		$0		$0	$999	$0	$0	$999
Project Set-up and invoicing		2			2				4	8	$1,470	$44		$0		$0	$1,514	$0	$0	$1,514
Health & Safety					2					2	$410	$12		$0		$0	$422	$0	$0	$422
Task 1.3 Quality Assurance/Quality Control (QA/QC)					6					6	$1,230	$37		$0		$0	$1,267	$0	$0	$1,267
										0	$0	$0		$0		$0	$0	$0	$0	$0
Phase 1 - Subtotal	0	22	0	0	14	0	0	0	6	42	$9,780	$293	$0	$0	$1,000	$100	$10,073	$0	$1,100	$11,173
Phase 2 - Design																				
Task 2.1 90% Design		60		12	20	27		30		149	$32,930	$988	$10,000	$1,000		$0	$33,918	$11,000	$0	$44,918
Task 2.2 Final Design		20		12	10	9		12		63	$13,830	$415		$0		$0	$14,245	$0	$0	$14,245
										0	$0	$0		$0		$0	$0	$0	$0	$0
										0	$0	$0		$0		$0	$0	$0	$0	$0
Phase 2 - Subtotal	0	80	0	24	30	36	0	42	0	212	$46,760	$1,403	$10,000	$1,000	$0	$0	$48,163	$11,000	$0	$59,163
Task 3																				
Task 3.1 Pre-Bid Meeting		8								8	$2,240	$67		$0	$1,000	$100	$2,307	$0	$1,100	$3,407
Task 3.2 Prepare Addendum		12							2	14	$3,610	$108		$0		$0	$3,718	$0	$0	$3,718
Task 3.3 Conformed Drawings		4				4		4		12	$2,420	$73		$0		$0	$2,493	$0	$0	$2,493
Phase 3 - Subtotal	0	24	0	0	0	4	0	4	2	34	$8,270	$248	$0	$0	$1,000	$100	$8,518	$0	$1,100	$9,618
Task 4																				
Task 4.1 Construction Management		32								32	$8,960	$269		$0	$1,500	$150	$9,229	$0	$1,650	$10,879
Task 4.2 RFI/RFC		24								24	$6,720	$202	$5,000	$500		$0	$6,922	$5,500	$0	$12,422
Task 4.3 Submittal Review		24								24	$6,720	$202		$0		$0	$6,922	$0	$0	$6,922
Task 4.4 Potential Change Review		16								16	$4,480	$134		$0		$0	$4,614	$0	$0	$4,614
Task 4.5 Record Drawings		2						6		8	$1,550	$47		$0		$0	$1,597	$0	$0	$1,597
Task 4 - Subtotal	0	98	0	0	0	0	0	6	0	104	$28,430	$853	$5,000	$500	$1,500	$150	$29,283	$5,500	$1,650	$36,433
All Phases Total	0	224	0	24	44	40	0	52	8	392	$93,240	$2,797	$15,000	$1,500	$3,500	$350	$96,037	$16,500	$3,850	$116,387

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Kennedy/Jenks Consultants

303 Second Street, Suite 300 South
San Francisco, California 94107
415-243-2150
FAX: 415-896-0999

Outfall Pipeline Bridge Evaluation

14 December 2018

Prepared for

Carmel Area Wastewater District
3945 Rio Road
Carmel, CA 93922

K/J Project No. 1768027*00

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Table of Contents

List of Tables

List of Figures

List of Appendices

Section 1: Introduction

This report documents the purpose, scope of work, goals and objectives, background information, field observations and condition survey, potentially hazardous conditions, structural evaluation, repair and replacement alternatives, and findings and recommendations associated with the 24-inch-diameter outfall pipeline bridge from the Carmel Area Wastewater District (District) Wastewater Treatment Plant spanning the Carmel River Lagoon near Carmel, CA. The outfall pipeline bridge location is shown on Figure 1 and the pipeline bridge location with respect to the water pollution control plant, Highway 1, and the Carmel River is shown on Figure 2.

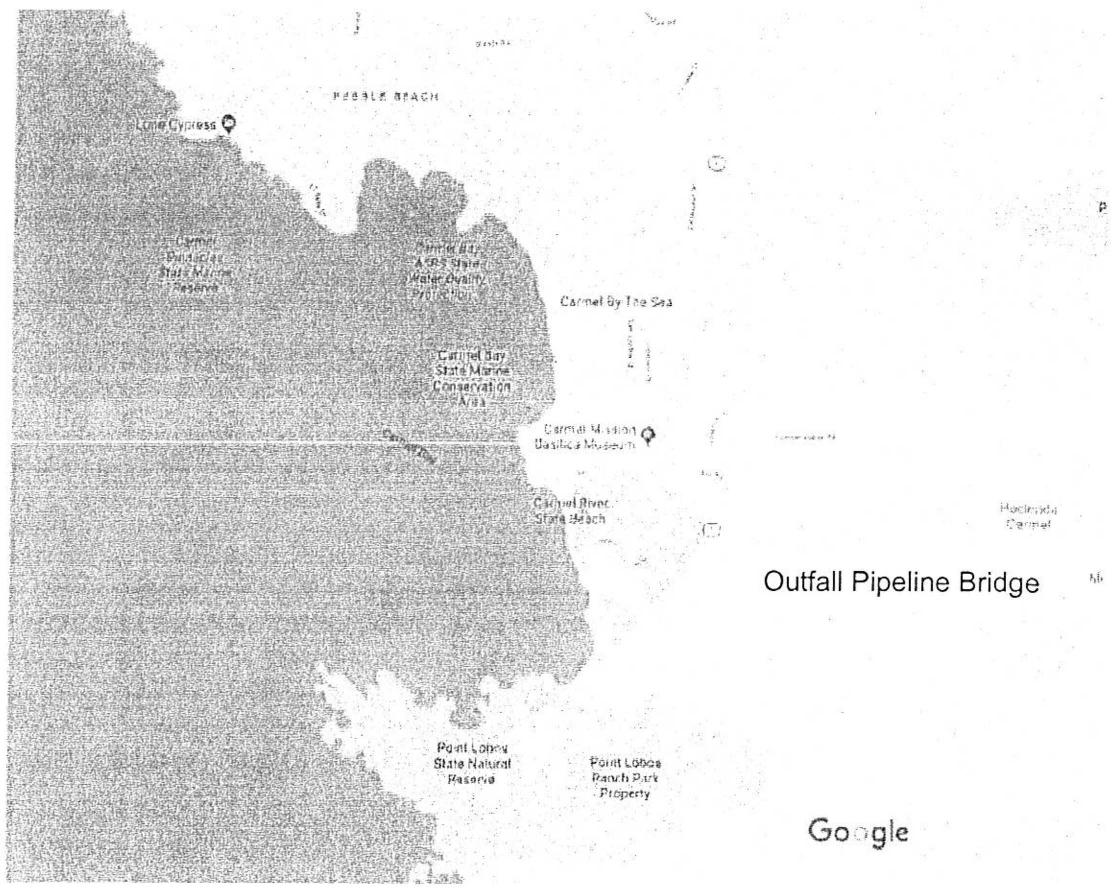

Figure 1: Carmel Area Wastewater District 24-Inch-Diameter Outfall Line

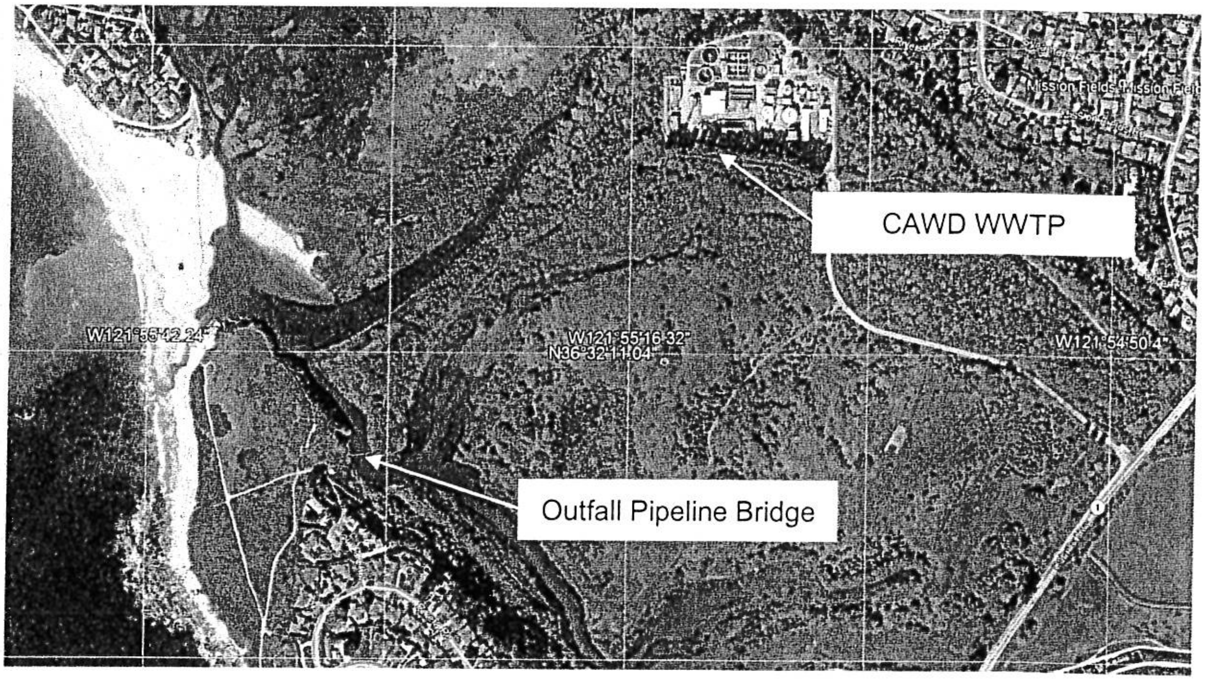

Figure 2: Carmel Area Wastewater District 24-Inch-Diameter Outfall Line

1.1 Purpose and Scope

The purpose of this report was to provide a preliminary source of information to the District on the condition and structural evaluation of the outfall pipeline bridge and bent supports. Kennedy/Jenks Consultants proposed scope of work included the following tasks:

- Perform a site visit with summary memo of observations and photos.
- Review as-built drawings and existing conditions.
- Prepare structural calculations of elements of the pipeline and bent supports: Compare installed conditions and evaluate horizontal flood loads based on velocity data provided by the District. Prepare calculations for various flood water velocities to determine critical velocity for pipe bridge failure.
- Provide constructability input.

The District was to assist with equipment and personnel flotation devices for access to lagoon and outfall pipeline.

The work was performed under the Constructability Review Task 2.4 of the agreement for professional services for the Lagoon Crossing Final Design.

1.2 Goals and Objectives

A structural evaluation of the outfall pipeline bridge should consider the following goals and objectives:

- To determine the structural adequacy and integrity of the pipe bridge and selected elements.
- To identify the type and seriousness of structural problems and evaluate distress which have resulted from unusual loading or exposure conditions. Distress may be caused by overloads, flood, foundation settlement and deterioration resulting from weathering and inadequate maintenance.
- To identify the deficiencies in the existing structure with respect to conforming with the current building code with respect to flood and earthquake loading.

It should be recognized that there is no absolute measurement of structural safety in an existing structure, particularly in structures that have deteriorated due to prolonged exposure to the environment, or that have been damaged by a physical event.

1.3 Documentation and Background Information

The following plans, specifications, and reports were reviewed as part of gathering information on the outfall pipeline bridge and the flood loads:

1. Carmel Sanitary District, Water Pollution Control Plant, As-Built Drawings prepared by Kennedy Engineers, Outfall Line Plan and Profile Sheet No. G-4 and Outfall Line Details Sheet No. G-5, 27 October 1970.
2. Carmel Sanitary District, Water Pollution Control Plant, As-Built Specifications prepared by Kennedy Engineers, Section 4 Pile Construction (Section 4-09, pg. 4.9) and Section 16 Piping (pgs. 16.9-16.11), 27 October 1970.
3. Memo of Supplementary 2D Model Results for CRFREE Project Existing Conditions, Proposed Conditions, and a Reduced Conveyance Alternative, prepared by Balance Hydrologics, Inc., 12 June 2018.
4. Calle La Cruz Force Main, Carmel Area Wastewater District, Carmel, California, Demolition Plan and Profile with bathymetric survey information of approximate existing mud line and piles, Sheet D01, prepared by Kennedy/Jenks Consultants, March 2018.
5. Geotechnical Report, CAWD Calle La Cruz Force Main, Carmel, California, prepared by Geotechnical Consultants, Inc., January 2014.

No construction records were reviewed associated with the outfall pipeline bridge. Copies of background documentation and information were provided by the District.

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Section 2: Field Observations and Conditions Survey

2.1 Planning

A site visit with Mr. Donald Barraza, Kennedy/Jenks Consultants, and Mr. Patrick Treanor, Carmel Area Wastewater District, was performed on the morning of 30 October 2018. The outfall pipeline and support bent beam were visible because the water level in the Carmel River Lagoon was low; however, the concrete filled steel pipe piles were submerged.

2.2 Verification of As-Built Construction

Review of the as-built drawings and specifications indicated that the outfall pipeline bridge and pile bents were in reasonable conformance with the construction drawings. While no measurements were collected in the field it appeared that the construction of the pile bents was in conformance with the construction drawings with the arrangement of the critical structural elements of the pile bents matching those on the drawings. The actual dimensions of the structure were not measured or compared with the as-built drawings.

2.3 Pipeline Design Criteria and Specifications

The nominal 24-inch-diameter outfall pipeline is a steel cement mortar lined and coated pipeline (non-prestressed). The pipeline was specified to conform to AWWA Standard C-201. The minimum steel cylinder wall thickness was ¼-inch. Cement mortar for the lining and coating was specified at ¾-inch minimum thickness with Type II cement. The nominal diameter was specified to be the net lined diameter; therefore, a steel cylinder with an inside diameter of 26 inches was utilized in the calculations. A 100-psi internal pressure was utilized for hoop stress calculations. While the pipeline was specified to contain rod reinforcement, stressed or partially stressed rod, spirally wrapped around and welded to the steel cylinder with a minimum area of 0.3 square inches per foot, wire size not less than 7/32-inch-diameter, with a spacing of not more than 1.5 inches center to center. The rod reinforcement was assumed to be provided for resisting hoop stresses was not included in evaluations of longitudinal, saddle, or effective stress calculations for the pipeline. Bolts, nuts, and hardware submerged in liquid were specified to be of 316 stainless steel materials and receive a 20-mil coal tar epoxy protective coating. There was no evidence observed of the coal tar epoxy protective coating on steel elements or hardware.

2.4 Pile Design Criteria and Specifications

The 140 feet-0-inch pipeline crossing of the slough is supported by five equally spaced (4 spaces at 35 feet-6 inches = 140 feet-0-inch) pile bents consisting of W 12x65 steel beam supported by two 10-inch pipe shell concrete filled piles at 3 feet-10 inches on center. Pile shells were specified of steel pipe with a minimum of 10-inch outside diameter, a minimum of 0.188-inch wall thickness, and a minimum length of 30 feet. Pipe for pile cylinders was specified as ASTM Designation A 252, Grade 1. Bottom of pipe cylinders were to be closed with a circular plate, ¾-inch thick, welded to the pipe. Maximum projection of plate beyond outside diameter of pile was to be ¼-inch. Pile shells were to be filled with concrete after driving, placing reinforcing, and accurately cutting extra pile shell and setting anchor bolts. Concrete was specified with minimum compressive strength of 4,000 psi at age 28 days. Piles were to be driven to develop a load bearing capacity of 15 tons as determined by the "Engineering News" formula. The two ¾-inch anchor bolts on each side of the pipeline anchoring the W12x65 pile bent cross beam to the pipe piles submerged in liquid were specified to be of 316 stainless steel. See Figure 3 for the Pile Bent detail shown on the as-built drawings.

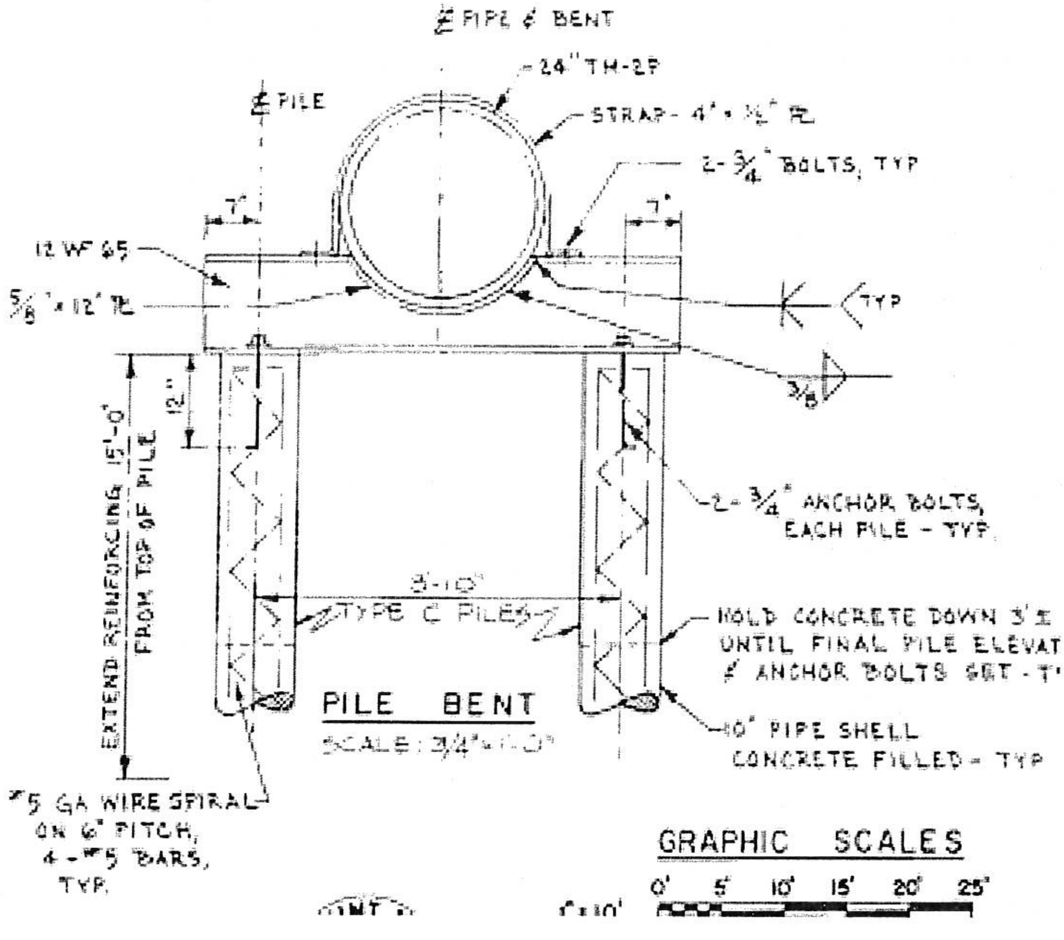

Figure 3: Carmel Area Wastewater District 24-Inch-Diameter Outfall Line Pile Bent Support Detail

Based on the as-built documents pipe piles were to have a minimum length of 30 feet and five pairs of piles were to be driven at approximately 35 feet-6 inches center to center spacing across the Camel River Lagoon. The as-built documents indicated that one pair of piles at approximately Sta. 20+77 would have the greatest exposed pile length with approximately 7 feet of pile length exposed above the mud line (see Figure 4). However, based on the bathymetric survey provided by the District and shown on the demolition profile for the Calle La Cruz Force Main the greatest exposed pile length for the pair of piles at approximately Sta. 20+77 would be approximately 12 feet above the mud line. For structural calculations in the evaluation of pile bending, a moment arm of 15 feet was utilized (12 feet + 3 feet) to fixity in the subgrade materials. This value should be verified with a geotechnical consultant for the design of replacement piles.

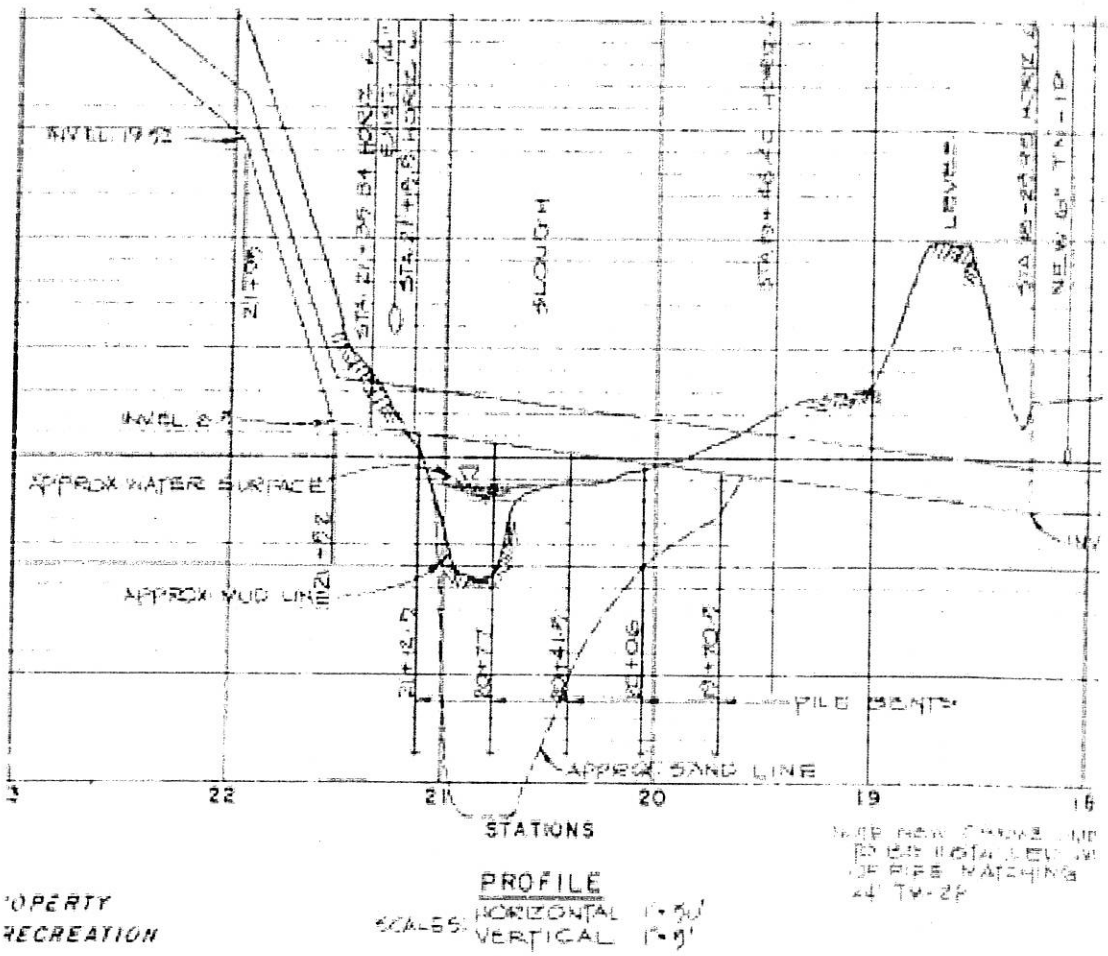

Figure 4: Carmel Area Wastewater District 24-inch-Diameter Outfall Line Profile

2.5 Condition Assessment

Recorded observations on the condition of the pipeline and pile bent are presented below. The principal focus of the condition survey was to focus on the nature and extent of observed problems and identifying affected structural members and the frequency and severity of problems. Photographs taken during the site visit showing deterioration of the outfall pipeline and pipe bents are included in Appendix A.

2.5.1 24-Inch-Diameter Outfall Pipeline Condition

Based solely on the exterior appearance the 24-inch-diameter outfall pipeline appeared to be in favorable condition with few observed defects in the mortar coating or staining. There were small longitudinal hairline cracks observed in the mortar coating with evidence of efflorescence.

There was also observed efflorescence in the mortar coating repair material where butt straps and field welding of the pipeline was performed. There was no evidence observed of the coal tar epoxy protective coating on steel elements or hardware.

2.5.2 Pile Bent Assembly Condition

The pile bent assembly exhibits significant corrosion of the unprotected non-stainless-steel elements consisting of the W 12x65 bent cross beam and the steel pipe piles. The W 12x65 bent cross beam displayed significant corrosion of the metal on the beam web including holes through the beam web and significant flaking and delamination of the brittle steel. The bottom flange of the W 12x65 beam was significantly deflected downward upwards of ¾-inch. Corrosion was also observed on the 4-inch x ½-inch plate strap over the pipe at the pile bents and along the post baseplates for the chain link gate assemblies mounted on each of the pipeline segment crossing the lagoon. Previous pictures collected during an earlier site visit and provided by the District when water levels in the lagoon were lower and the top of pipe piles were exposed would appear to convey significant corrosion of the steel pipe pile elements with delamination of steel and flaking of brittle steel materials in the top of the pipe piles.

2.6 Sampling and Material Testing

No testing of the materials of construction for the pipeline or pile bents was performed. If detailed evaluations are to be performed it is recommended that nondestructive testing be performed on the W 12x65 pile bent cross beam webs and the 10-inch-diameter steel pile pipe shells to determine the thickness of the steel. Nondestructive testing including ultrasonic thickness measurements of the steel in the webs of the cross beams would provide detailed information on the maximum web shear capacity of the beams. Ultrasonic thickness measurements of the steel in the pipe piles would provide detailed information on the maximum flexural (bending) strength of the pipe piles to resist lateral loading.

If rehabilitation of the outfall pipeline and pile bents were to be considered the District may elect to undertake further materials evaluation of the steel and concrete elements of construction to determine the strength and quality of materials compared with the original material specifications.

2.7 Unsafe or Potentially Hazardous Conditions

The W 12x65 web elements of the pile bent cross beams were observed to have significant corrosion of the metal resulting in a significant reduction in the thickness of web steel and holes in the web of the beams as well as web crippling or crushing of the web over the pipe pile supports. Web crippling occurs due to the stress concentrations at the junction of the flange and the web. Failure occurs when the metal begins to fail at the toe of the fillet in bearing and the flange and web have tendency to fold over each other. The web crippling would indicate that the pile bent cross beams are vulnerable to a shear failure which would be considered serious distress or deficiencies which would result in unsafe or potentially hazardous conditions. Significant deflection was observed in the beam cross section which will continue to sag as additional metal is corroded in the beam web. Recommendations for repair of the cross beam deteriorated condition are presented in Section 3.3.

Section 3: Structural Evaluation

The results of the preliminary structural evaluation of the outfall pipeline and the pipe bent are presented in this section. This section documents the loading requirements, structural capacity checks, structural problems, strengthening requirements, and needs for further investigation for elements of the pipeline and pipe bents. Structural evaluation is a process of determining the adequacy of the components of the structure for the intended use by analyzing systematically the available information to determine the load-carrying capacity of the critical elements of the structure. Structural evaluation was performed by analysis, the most common method, because sufficient as-built information was available and did not include development of a structural model of the pipeline bridge and pile bents.

3.1 Service and Strength Loading Requirements

The ability of the outfall pipeline bridge structure to support all present and anticipated loads according to the 2016 California Building Code (CBC) were considered. Dead loads including the weight of materials, live loads include the weights and pressures of contents in the structure, and environmental loads include earthquake and flood loads. Wind load, flood debris impact loads, and earthquake sloshing loads on the outfall pipeline in the lagoon were considered beyond the scope of work of a preliminary evaluation. Where the 2016 CBC requirements could not be met with the structure in its current condition, appropriate strengthening methods and techniques are required. Concentrated or moving loads from pedestrians walking on the pipeline and the fence gate posts of the vandal barrier system were not evaluated.

3.1.1 Dead Loads

Dead loads include the weight of all materials of construction incorporated into the pipeline bridge structure including the pipeline steel cylinder, cement mortar lining and coating and W 12x65 steel beam. A unit weight of 490 lb./cu. in. was utilized for the weight of the steel cylinder with a ¼-inch wall thickness. A unit weight of 150 lb./cu. ft. was utilized for the weight of the cement mortar lining and coating with a ¾-inch thickness for both.

3.1.2 Live Loads and Internal Pressure

Live loads include the weight of contents or wastewater in the pipeline. A unit weight of 62.43 lb./cu. ft. was utilized for the weight of fluid in the pipeline. The total weight per foot for the pipeline and liquid contents was 391 lb./ft. The total weight for the 35 feet-6-inch span length between pile bents was 13,893 lbs.

For ring or hoop stress calculations in the pipeline a maximum internal pressure of 100 psi as documented in the as-built specifications was utilized.

3.1.3 Earthquake Loads

Earthquake loads were determined in accordance with the 2016 CBC and ASCE 7-10 utilizing the equivalent lateral force procedure to determine the inertial effects of the pipeline and pile bent structures in an earthquake event. While the total design lateral seismic base shear force for non-building structures could be determined utilizing ASCE 7 Eq. 15.4-5 the use of ASCE 7 Eq. 12.8-2 is more conservative and more in line with ACI 350 for the determination of seismic base shear. Based on the pipeline bridge coordinates, an assumption of Site Class D for stiff soil, and Risk Category III structure the seismic lateral base shear was calculated at approximately 0. 68W or approximately 68% of the effective seismic weight. Vertical seismic load effects were calculated in accordance with ASCE 7 Eq. 12.4-4 at approximately 0.22D or approximately 22% of the effect of dead load. As previously noted, earthquake sloshing loads on the outfall pipeline in the lagoon were considered beyond the scope of work of a preliminary evaluation.

3.1.4 Flood Loads

The provisions of ASCE 7-10 Minimum Design Loads for Buildings and Other Structures, Chapter 5, Flood Loads which applies to other structures located in areas prone to flooding as defined on a flood hazard map was utilized to determine flood loads on the outfall pipeline.

The base flood is the flood having a 1% chance of being equaled or exceeded in any given year. The base flood elevation (BFE) is the elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year. The design flood is the greater of the following two flood events: 1) the base flood, affecting those areas identified as Special Flood Hazard Areas on the community's FIRM; or 2) the flood corresponding to the area designated as a Flood Hazard Area on a community's Flood Hazard Map or otherwise legally designated. The design flood elevation (DFE) is the elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map. The flood hazard area is the area subject to flooding during the design flood. The flood hazard map is the map delineating flood hazard areas adopted by the authority having jurisdiction. The flood insurance rate map (FIRM) is an official map of a community on which the Federal Insurance and Mitigation Administration has delineated both special flood hazard areas and the risk premium zones applicable to the community.

In accordance with ASCE 7-10, Section 5.4.3 Hydrodynamic Loads, dynamic effects of moving water shall be determined by a detailed analysis utilizing basic concepts of fluid mechanics. Where water velocities do not exceed 10 ft/s, dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing the DFE for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to:

$$d_h = \frac{aV^2}{2g}$$

Where:

V = average velocity of water in ft/s
g = acceleration due to gravity, 32.2 ft/s^2
a = coefficient of drag or shape factor (not less than 1.25)

The equivalent surcharge depth shall be added to the DFE design depth and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tail water shall be subject to the hydrostatic pressures for depths to the DFE only.

Based on the flowrate, water surface elevation, and velocity data contained in the memo of Supplementary 2D Model Results for CRFREE Project Existing Conditions, Proposed Conditions, and a Reduced Conveyance Alternative, prepared by Balance Hydrologics, Inc. and the above equation from ASCE 7, Section 5.4.3, the equivalent surcharge depth and hydrostatic pressures on the outfall pipeline were determined and are summarized in Table 1.

Table 1: Flood Loads Based on Modeled Flow Event (1)

Modeled Flow Event	Flowrate, Q (cfs)	Water Surface Elevation, WSE (ft, NAVD88)	Velocity, V (ft/s)	Equivalent Surcharge Depth, dh (ft)	Equivalent Hydrostatic Load (psf)
10-Year Event					
Existing Condition	2,800	10.4	3.6	0.25	15.61
Reduced Alternative	3,000	10.5	3.9	0.30	18.73
Proposed Condition	4,000	10.8	4.9	0.47	29.34
100-Year Event					
Existing Condition	10,200	13.3	8.8	1.50	93.65
Reduced Alternative	10,600	13.4	9.3	1.68	104.88
Proposed Condition	11,300	13.6	9.5	1.75	109.25

Notes:

(1) Flowrate, water surface elevation, and velocity for the modeled flow events were obtained from the memo of Supplementary 2D Model Results for CRFREE Project Existing Conditions, Proposed Conditions, and a Reduced Conveyance Alternative, prepared by Balance Hydrologics, Inc.,12 June 2018.

3.2 Results of Analysis

The critical structural elements identified for evaluation based on review of the as-built drawings and materials included: 1) the cement mortar lined and coated welded steel pipeline; 2) the 4-inch x ½-inch plate strap and two ¾-inch bolts holding down the pipeline to the W 12x65 saddle beam; 3) the W 12x65 saddle cross beam; 4) the four ¾-inch-diameter anchor bolts between the saddle beam and the concrete pipe piles; and 5) the 10-inch-diameter combined steel and reinforced concrete pipe piles. The capacities of structural elements were determined utilizing both serviceability and strength design depending on the element.

3.2.1 24-Inch-Diameter Outfall Pipeline

Pipeline stresses for the 24-inch-diameter cement mortar lined and coated welded steel cylinder pipeline were determined in accordance with AWWA M11 equations for pipelines with saddle supports. The theoretical pinned and fixed deflection for the pipeline with a 35 feet-6-inch span was calculated at approximately 0.28 and 0.06 inches, respectively. The maximum longitudinal stress caused by beam bending was determined to be 5,73 lbs./sq. in. and the maximum longitudinal stress caused by ring or hoop stress at the 100-psi internal pressure was determined to be 5,200 lbs./sq. in. The pinned maximum total longitudinal stress incorporating both the flexure stress and ring stress was determined to be 7,297 lbs./sq. in. The circumferential stress at the local saddle supports on the pipe cylinder was determined to be 7,202 lbs./sq. in. and the effective stress combining the effects of circumferential saddle stress with longitudinal stress in the shell because of longitudinal movement of the shell under temperature changes was determined to be 7,743 lbs./sq. in.

Where the effects of vertical seismic loads were added to the gravity loads the stresses were minimally increased. Where the effects of flood loads were evaluated there was little increase since loading was in a horizontal principal direction versus the gravity and seismic loads in the vertical principal direction. The load combinations with seismic and flood loads, and pipe stresses are summarized in Table 2.

The equivalent stresses are not permitted to exceed 33% of the tensile strength of the pipeline cylinder and based on an assumption of A36 or better pipe material with an ultimate tensile strength of 58,000 psi the capacity of the pipeline cylinder was determined to be 19,333 psi. Assuming minimal corrosion and deterioration in the pipeline cylinder since it is protected by cement mortar lining and coating and an impressed current cathodic protection system it is unlikely that the outfall pipeline would be structurally compromised for the evaluated loading conditions.

3.2.2 Pile Bent Assembly

The pile bent assembly consists of the following elements: 1) the 4-inch x ½-inch plate strap and two ¾-inch bolts holding down the pipeline to the W 12x65 saddle beam; 2) the W 12x65 saddle cross beam; 3) the four ¾-inch-diameter anchor bolts between the saddle beam and the concrete pipe piles; and 4) the 10-inch-diameter combined steel and reinforced concrete pipe piles.

3.2.2.1 4-Inch x ½-Inch Plate Strap and Four ¾-Inch-Diameter Machine Bolts

Based on the pile bent detail and observations and photographs taken a 4-inch wide by ½-inch thick plate strap was to be placed over the pipeline and anchored to the W 12x65 saddle cross beam utilizing two ¾-inch-diameter bolts on each side of the pipeline. Based on as-built specifications bolts and hardware submerged were to be of 316 stainless steel with a tensile strength of 84 ksi. The allowable shear for two bolts with threads included in the shear plane, conservative, would be approximately 12,618 lbs. and for seismic loading 16,824 lbs. For flood loading at 10 ft./sec. velocity,the maximum lateral shear on the bolt group is approximately 6,452 lbs. and for lateral earthquake loading the maximum lateral shear on the bolt group is approximately 11,388 lbs. The 316 stainless steel bolt material should be verified for bolts and hardware since some corrosion appeared evident on bolts in photographs. Since the pipeline is resting in a saddle support it is unlikely that the shear loading would be imparted on the bolts under flood or earthquake loading without overcoming the dead load of the pipeline. It is possible that a prying action could result in tensile loading on the bolts and hardware.

3.2.2.2 W 12x65 Saddle Cross Beam

The W 12x65 pile bent saddle support cross beam was evaluated in both shear and flexure for the combined effects of the concentrated load at midspan associated with the reaction from the pipeline segments and the uniform load associated with the weight of the beam. Service load maximum shear was calculated at 7,109 lbs. and strength load maximum shear under earthquake loading was calculated at 10,050 lbs. Service load maximum bending moment was calculated at 13.433 kip-ft. and strength load maximum bending moment under earthquake loading was calculated at 19.031 kip-ft.

Shearing stress is not constant across a beam cross section but is zero at the outermost fibers and has its largest value at the neutral axis. The values of shear stress in the web of wide flange sections are uniform throughout the web. A conservative approach is to only utilize the web depth in figuring the average shear and neglect the depth of the fillets. The allowable web shear capacity of a new beam would be 65.448 kips. However, if we considered the reduced web depth at the saddle, approximately 5.62 inches, the shear capacity would be reduced to 30.348 kips. However, both values are based on the original 3/8-inch thickness of the web which has been shown to be severely compromised because of corrosion with holes observed in several areas of the web. Web crippling or crushing occurs due to the stress concentrations at the junction of the flange and the web where the beam is trying to transfer compression in the relatively wide flange to the narrow web. Failure will occur when the metal begins to fail at the toe of the fillet in bearing and the flange and the web may tend to fold over each other. As previously documented web crippling was observed in the bearing area over the concrete filled pipe piles in the W 12x65 saddle beams.

Typically, if the webs are overstressed in shear they can be thickened or reinforced, or shear plates can be connected to the webs in the zones of excessive stress; however, due to the extensive corrosion of the beam webs and flanges, the deflection of the beam flanges, and the crippling of the beam webs it would be difficult to add shear plates to the existing beams. Recommendations for repair of the cross beam deteriorated condition are presented in Section 3.3.

The allowable maximum moment capacity of the theoretical beam cross section was calculated at 174.042 kip-ft. well above the service load maximum bending moment; however, as noted above for shear due to extensive corrosion it is unlikely that the theoretical section modulus, Sx, of the beam is intact. This is also supported by the theoretical maximum deflection of the beam under service loading which was calculated at approximately 0.002 inches; however, a significantly larger approximately ¾-inch deflection of the bottom flange of the beam was observed in the field relative to the water surface.

3.2.2.3 Four ¾-Inch-Diameter Anchor Bolts

Based on the pile bent detail and observations and photographs taken the W 12x65 saddle cross beam was to be anchored to the 10-inch-diameter concrete filled steel pipe piles utilizing two ¾-inch-diameter bolts on each side of the beam. Based on as-built specifications bolts and hardware submerged were to be of 316 stainless steel with a tensile strength of 84 ksi. The allowable shear for four bolts with threads included in the shear plane, conservative, would be approximately 25,236 lbs. and for seismic loading 33,648 lbs. For flood loading at 10 ft./sec. velocity the maximum lateral shear on the bolt group is approximately 6,452 lbs. and for lateral earthquake loading the maximum lateral shear on the bolt group is approximately 11,388 lbs. The 316 stainless steel bolt material should be verified for bolts and hardware since some corrosion appeared evident on bolts and hardware in photographs. Based on the 12-inch embedment shown on the as-built drawings, it appears that the anchor bolts are sufficiently embedded in the reinforced concrete piles with no significant deterioration.

3.2.2.4 10-Inch Pipe Shell Concrete Filled Piles

As previously noted, the 10-inch-diameter concrete filled pile shells were specified of steel pipe with a minimum of 10-inch outside diameter, a minimum of 0.188-inch wall thickness, and a minimum length of 30 feet. Structural calculations based on the 4,000 lb./sq. in. compressive strength in the concrete and the four-#5 vertical longitudinal bars with rebar orientation in the most conservative configuration, indicated that piles were designed with an axial strength at the balanced strain point, φPb, of magnitude 55.867 kips. However, based on the as-built specifications, the piles were to be driven to develop an allowable load bearing capacity of 15 tons (30 kips). The strength factored axial load on piles considering load combinations with earthquake induced vertical acceleration should not exceed 10.050 kips.

Structural calculations for the moment capacity of the reinforced concrete section in the pile indicated that at the balanced strain point the design moment strength, φMb, magnitude was 18.883 kip-ft. with rebar orientation in the most conservative configuration. Since the design moment capacity of the piles alone is insufficient to resist the moment demand on the piles during flood loading conditions the design (plastic) moment capacity of the steel pipe section was also evaluated and determined to have a magnitude of 40.729 kip-ft. at the yielding limit states condition. When the flexural strength of the reinforced concrete section of the pile was combined with the strength of the steel pipe pile section, the combined section would yield a maximum moment strength capacity of 44.892 kip-ft utilizing the square root of the sum of the squares method for combining strengths.

It should be noted that the flexural capacity of the piles to resist lateral loading, flood or earthquake, is highly dependent on the 10-inch-diameter by ¼-inch thick steel pipe pile section strength, which may have been corroded significantly over the 50-year life of the submerged and exposure of the pile in the tidal zone. The capacity of the pile to resist lateral loading is dependent on verification of the remaining steel pipe pile wall thickness.

To evaluate the resistance of the pile to flood loads based on modeled flow events, the modeled flow event, flowrate, velocity, and pile maximum combined flexural moment capacity is listed in Table 2. The pile maximum moment is based on a 15-foot moment arm from the mud line in the lagoon. Based on the original profile and the bathymetric survey information of approximate existing mud line and piles, it is believed that only one pile bent located near the middle of the lagoon has a 15-foot moment arm. The next longest exposed pile bent has an approximate 9-foot moment arm and the remaining piles have only a few feet of exposed pile length. Therefore, the results of calculations presented in Tables 1 and 2 are conservative for the given piles. The critical average velocity where the pile maximum combined moment is exceeded is 8 ft./sec. assuming the full ¼-inch thick steel pipe pile section is intact. The piles with the decreased moment arm, 9 feet or less, all exceeded the flexural moment strength of the piles at the maximum 10 ft./sec. velocity evaluated.

Table 2: Pile Maximum Combined Moment Capacity Based on Modeled Flow Event (1)

Modeled Flow Event	Flowrate, Q (cfs)	Velocity, V (ft/s)	Pile Maximum Moment (kip-ft)
10-Year Event			
Existing Condition	2,800	3.6	9.099
Reduced Alternative	3,000	3.9	10.678
Proposed Condition	4,000	4.9	16.857
100-Year Event			
Existing Condition	10,200	8.8	54.368
Reduced Alternative	10,600	9.3	60.722
Proposed Condition	11,300	9.5	63.361

Notes:

(1) Flowrate and velocity for the modeled flow events were obtained from the memo of Supplementary 2D Model Results for CRFREE Project Existing Conditions, Proposed Conditions, and a Reduced Conveyance Alternative, prepared by Balance Hydrologics, Inc., 12 June 2018.

The results of the structural element evaluation including the load demands, element capacity, and the demand to capacity ratio and factor of safety for all the elements evaluated are summarized in Table 2. Additional structural calculations are provided in Appendix B.

Table 3: Structural Element Summary of Demand to Capacity Results

Structural Element	Demand	Capacity	D/C Ratio	Factor of Safety
24-Inch-Diameter Outfall Pipeline (1)(2)(3)				
Dead Load + Contents				
Total Longitudinal Stress, St	7,297 psi	19,333 psi	0.38	2.64
Localized Saddle Stress, Scs	7,202 psi	19,333 psi	0.37	2.68
Effective Stress, Se	7,743 psi	19,333 psi	0.40	2.49
Dead Load + Vertical Seismic				
Total Longitudinal Stress, St	7,974 psi	19,333 psi	0.41	2.42
Localized Saddle Stress, Scs	8,052 psi	19,333 psi	0.41	2.40
Effective Stress, Se	8,122 psi	19,333 psi	0.42	2.38
Flood Load at 10 feet/second				
Localized Saddle Stress, Scs	4,460 psi	19,333 psi	0.23	4.33
Effective Stress, Se	7,102 psi	19,333 psi	0.36	2.72
4-inch x ½-Inch Plate Strap and Two ¾-Inch-Diameter Machine Bolts				
Allowable shear stress threads included in shear plane flood loading at 10 ft/sec	6.452 k	12.618 k	0.51	1.95
Allowable shear stress threads included in shear plane for earthquake loading	11.388 k	16.824 k	0.67	1.47
W12x65 Saddle Cross Beam (4)(5)				
Max web shear under service load, V	7.109 k	30.348 k	Failed	None
Max moment under service load, M	13.433 k-ft	174.04 k-ft	0.07	12.95
Beam deflection under service load, Δ	0.002 in			
Four ¾-Inch-Diameter Anchor Bolts				
Allowable shear stress threads included in shear plane flood loading at 10 ft/sec	6.452 k	25.236 k	0.25	3.91
Allowable shear stress threads included in shear plane for earthquake loading	11.388 k	33.648 k	0.33	2.95
10-Inch Pipe Shell Concrete Filled Pile (6)(7)(8)				
Axial load & strength at balanced strain, φPb	10.050 k	30.000 k	0.33	2.98
Moment demand and strength at 4 ft/sec flood load at balanced strain, φMb	11.233 k-ft	44.892 k-ft	0.25	3. 9
Moment demand and strength at 10 ft/sec flood load at balanced strain, φMb	70.206 k-ft	44.892 k-ft	1.56	0.63

Notes:

(1) Equivalent stress is not permitted to exceed 33% of tensile strength. Assumes ASTM A36 or better pipe material with minimum ultimate tensile strength of 58,000 psi. Based on 0.25-inch pipe wall thickness.
(2) Total longitudinal stress is pinned flexural stress plus 0.3 x hoop stress assuming 100-psi internal pressure.
(3) Effective stress is function of local saddle stress plus stresses in the shell because of temperature changes.
(4) Web shear is based on reduced cross section beneath saddle. Web crippling observed under bearing reactions.
(5) Beam deflection under service load is theoretical actual deflection observed in field was on the order of ±0.75 in.
(6) Axial demand is based on strength load combination with vertical earthquake load. $E_v = 0.2 \times S_{DS} \times D = 0.286 \times D$.
(7) Moment arm is approximately 15 feet for one pile bent (worst case). Moment strength of reinforced concrete in piles is dependent on reinforcing bar orientation.
(8) Assumes 0.188-inch steel pipe pile wall thickness is intact. Should be verified utilizing ultrasonic thickness gauge measurements.

3.3 Repair and Replacement Alternatives

Should the District elect to repair or replace the outfall pipeline bridge components it is recommended that alternative repair methods as well as the possibility of the "do nothing approach" should be evaluated based on comparative cost estimates and relative levels of interference with the wastewater treatment plant operations.

Currently there does not appear to be a need to replace the 24-inch-diameter outfall pipeline and the 4-inch x ½-inch plate strap and the 10-inch-diameter reinforced concrete filled steel pipe piles if resistance to the 100-year flood and lateral forces from earthquake events was not considered a requirement.

The W12x65 pile bent saddle cross beam along with the anchor bolts holding down the pipeline plate strap to the beam should be repaired or replaced as soon as feasible by the District. The W12x65 web elements of the pile bent cross beams were observed to have significant corrosion of the metal resulting in a significant reduction in the thickness of web steel and holes in the web of the beams as well as web crippling or crushing of the web over the pipe pile supports.

Repair of the steel cross beam could include the addition of web stiffener plates or plates beneath the flange edges of the beams; however, these will be difficult to construct and weld in place due to the deflection of the beam and the poor condition and thickness of the base metal for welding.

It may be possible to repair the steel cross beam by converting it to a concrete beam by constructing formwork and placing reinforcing steel to encapsulate the steel beam cross section.

Temporary wood shoring including posts and cross beams may also be possible to be attached to the piles utilizing steel straps beneath the water surface to support the pipeline temporarily until the existing saddle beams can be removed and replaced in their current location.

Replacement of the deteriorated pile cross beams and piles could also be performed by constructing new larger piles with new cross beams to replace the existing pile bents and either abandon in-place or remove the existing pile bents assuming the spans are not increased between saddle supports.

There are materials and methods for jacketing the existing steel piles with carbon or glass fibers or FRP materials which could increase the flexural strength of the pipe piles; however, these methods will have challenges associated with working beneath the water surface and may be limited to repair above the mud line.

The pile bent located in the middle of the lagoon is at the highest risk for failure due to bending of the piles because it has the longest moment arm (in the deepest part of the channel). Therefore, other shorter pile bents may not require significant rehabilitation.

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Appendix A: Condition Assessment Photos

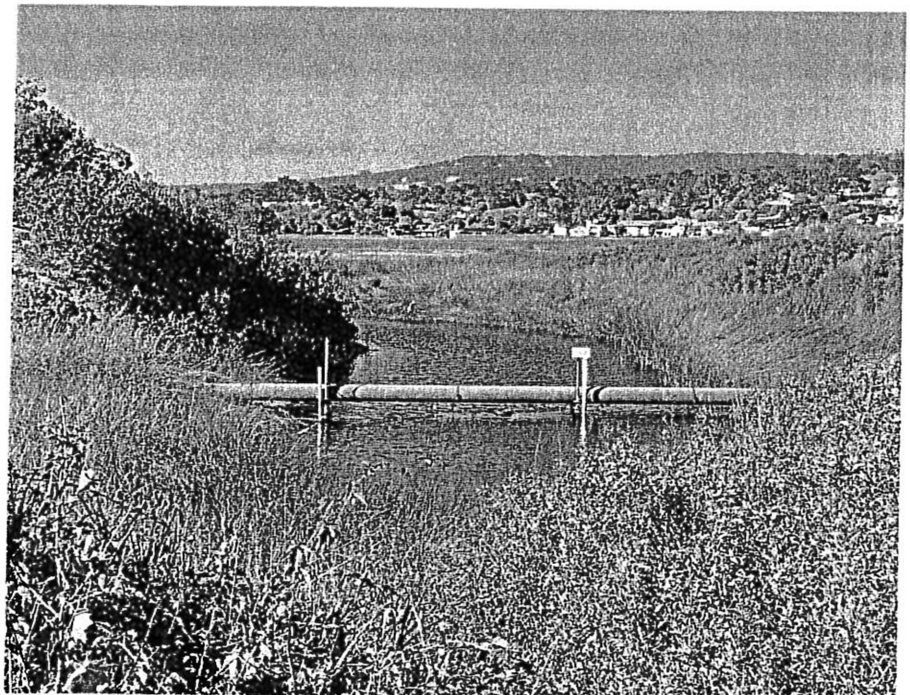

Photo 1: Outfall pipeline bridge spanning Carmel River Lagoon. View looking north.

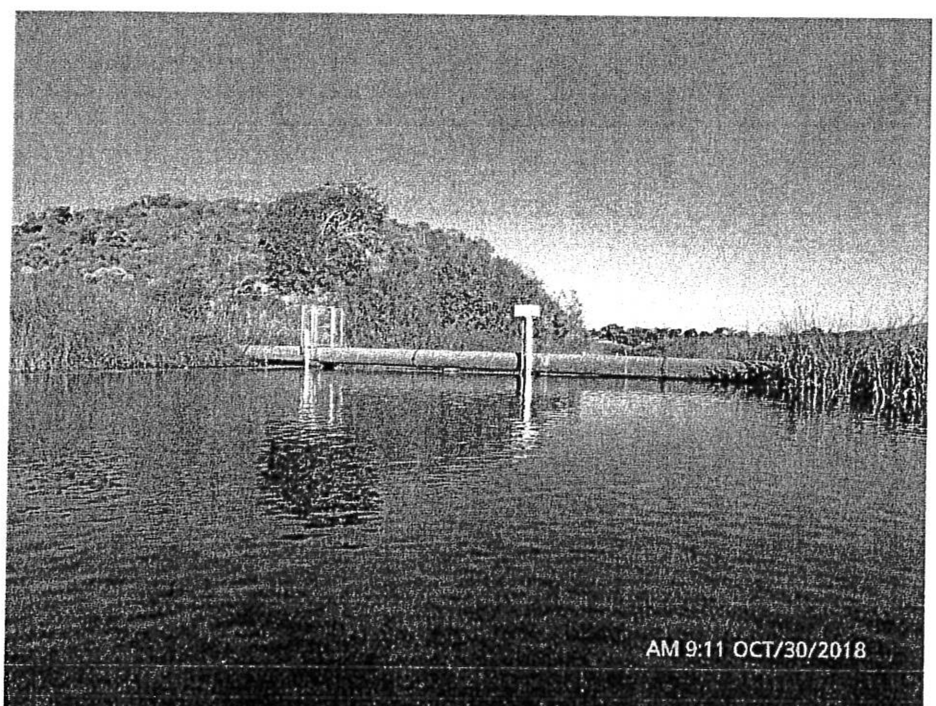

Photo 2: Outfall pipeline bridge spanning Carmel River Lagoon. View looking north from lagoon.

Photo 3: Pile bent at Sta. 21+12.5.

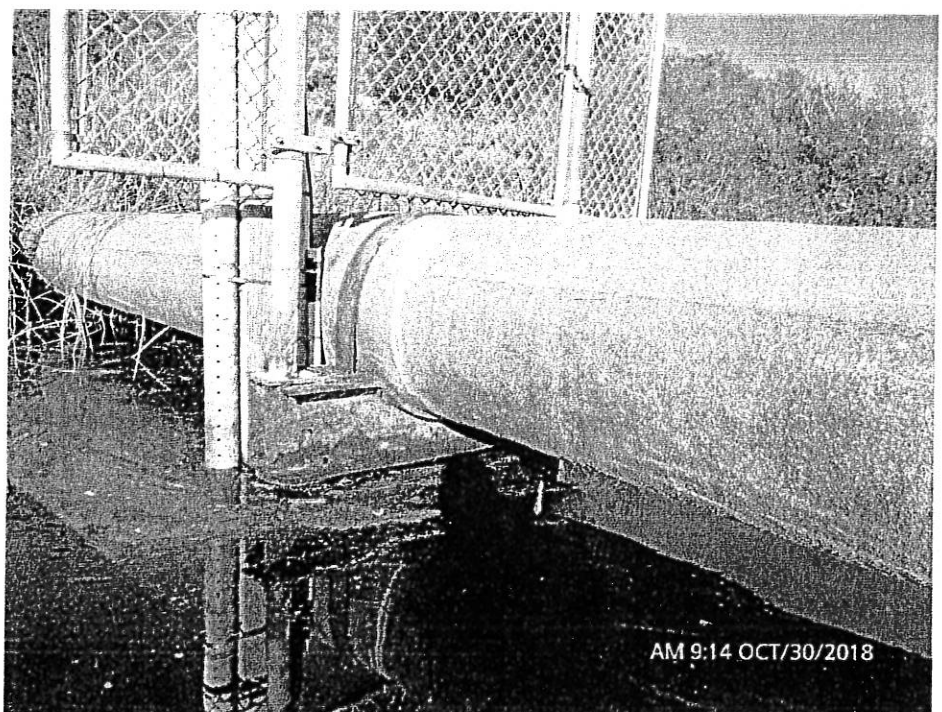

Photo 4: Pile bent at Sta. 21+12.5.

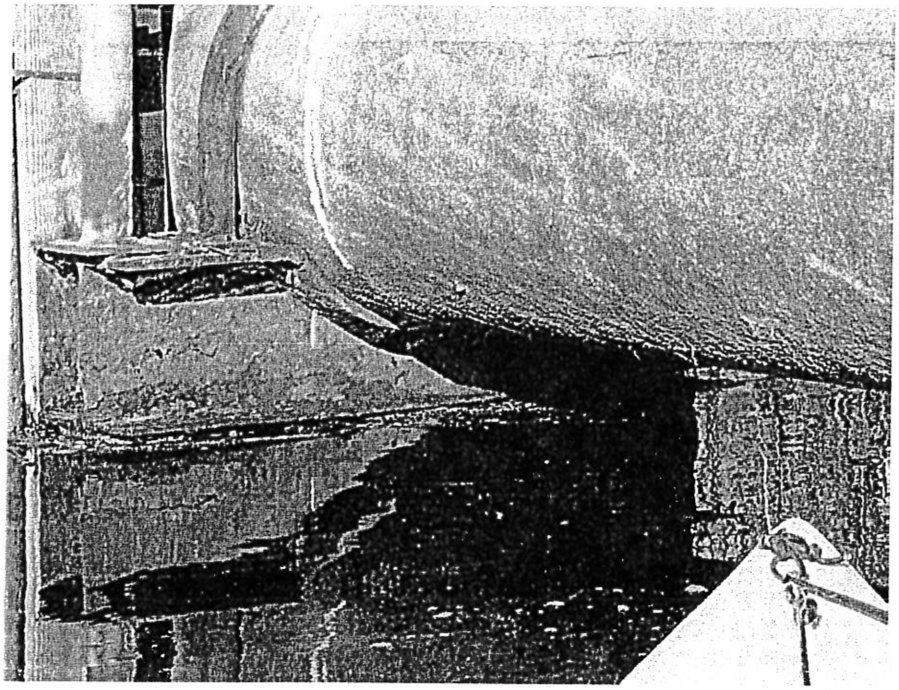

Photo 5: Pile bent at Sta. 21+12.5. Note deflection of bottom flange of W 12x65.

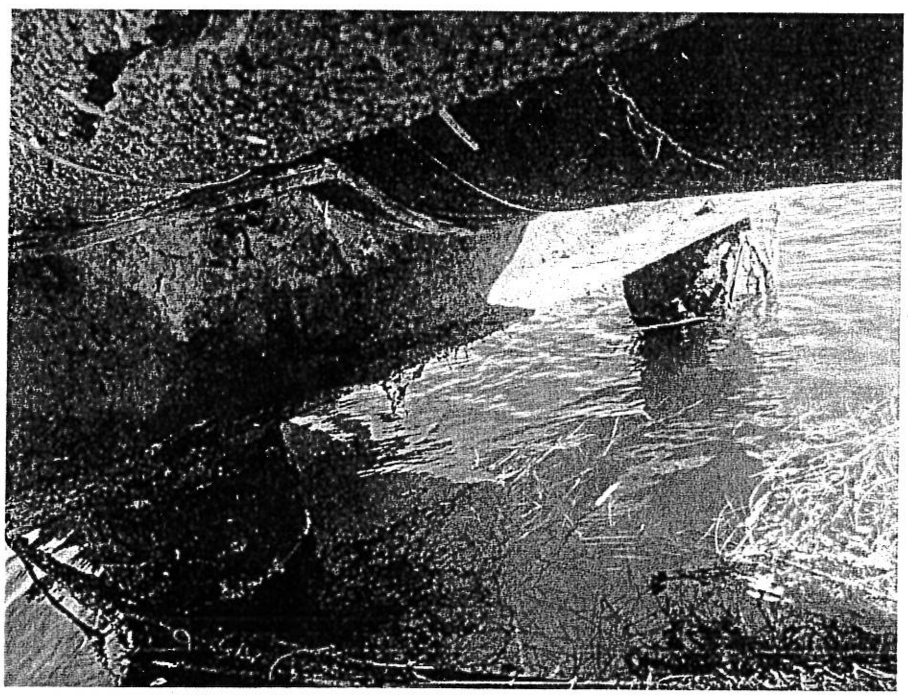

Photo 6: Pile bent.

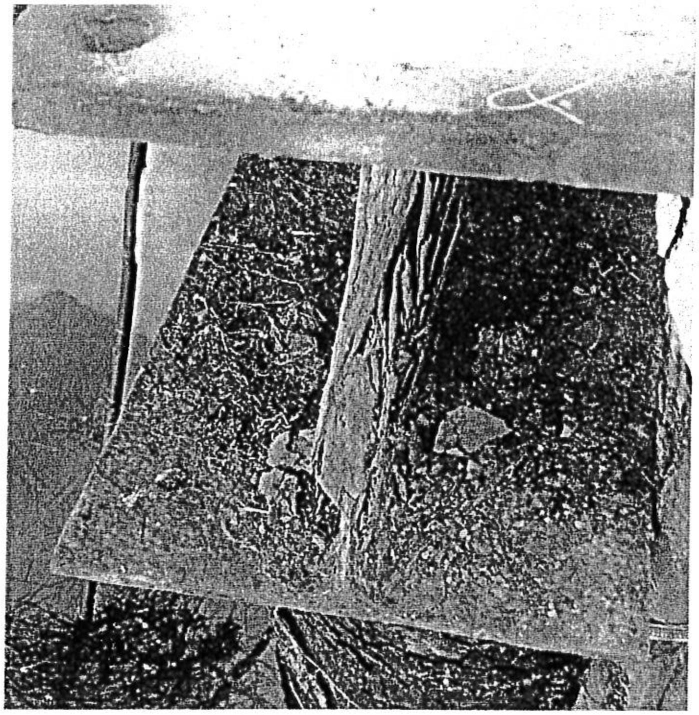

Photo 7: Web crippling of W 12x65.

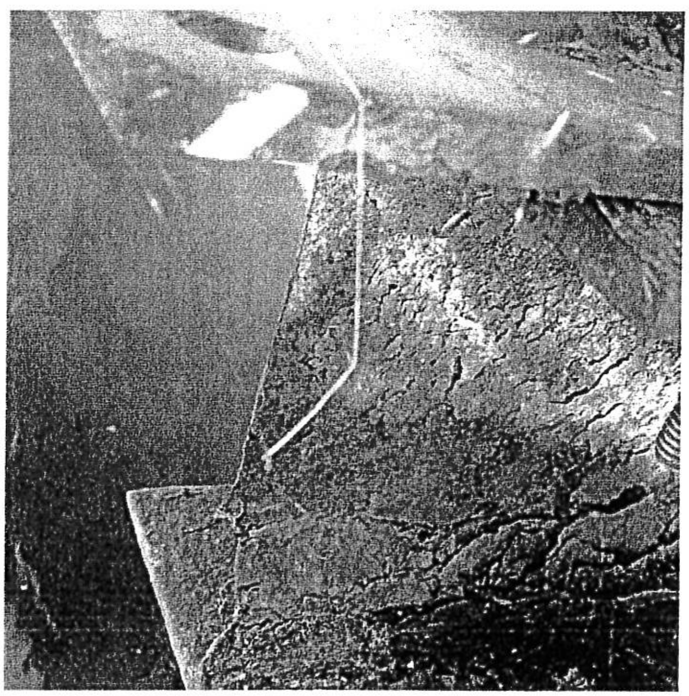

Photo 8: Web crippling of W 12x64 and loss of metal.

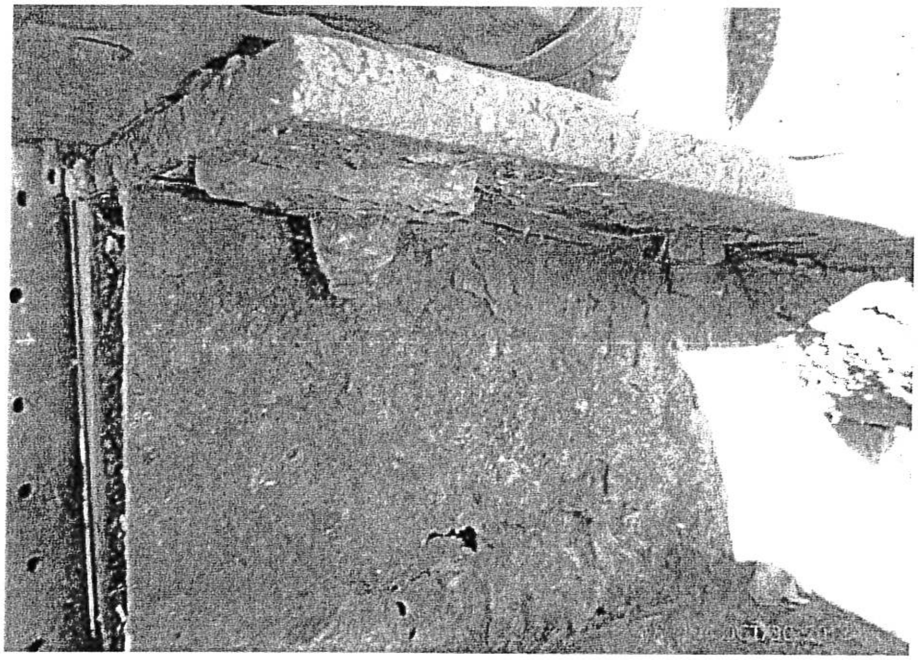

Photo 9: Pile bent at Sta. 21+12.5. Significant corrosion of W 12x65 web with holes.

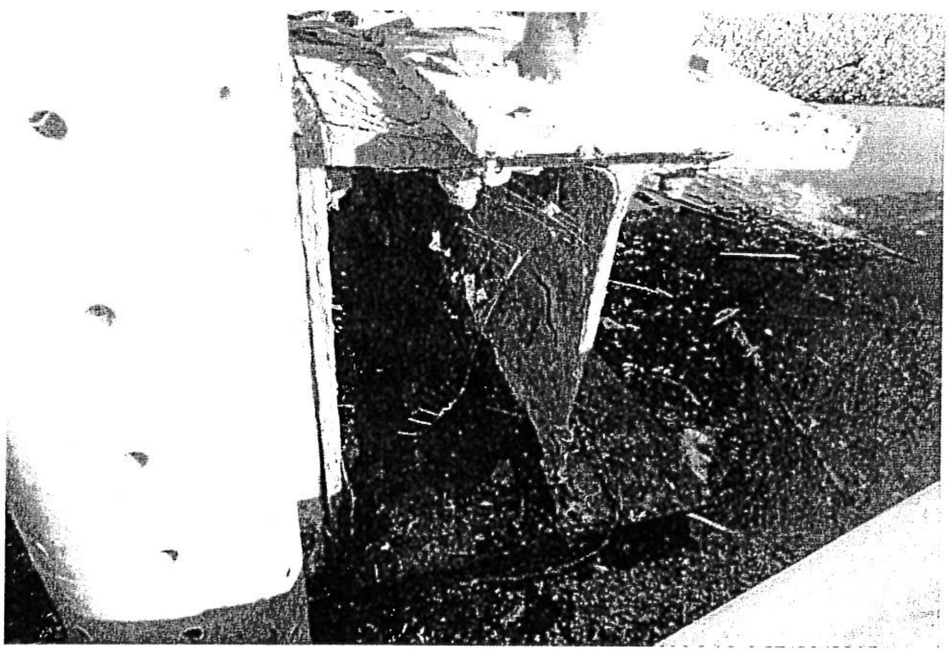

Photo 10: Pile bent at Sta. 21+12.5. Web crippling of W 12x65.

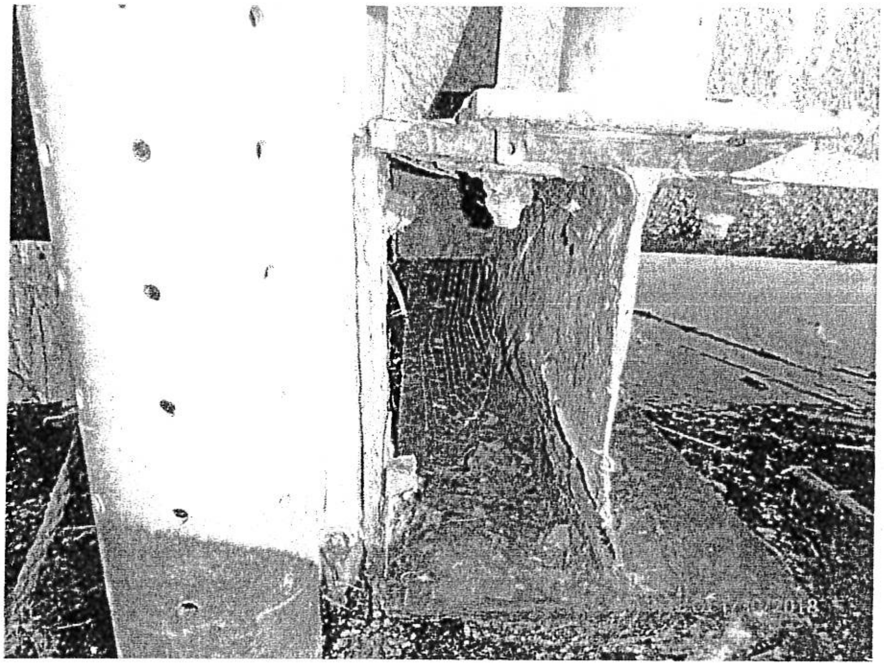

Photo 11: Pile bent at Sta. 21+12.5. Web crippling of W 12x65.

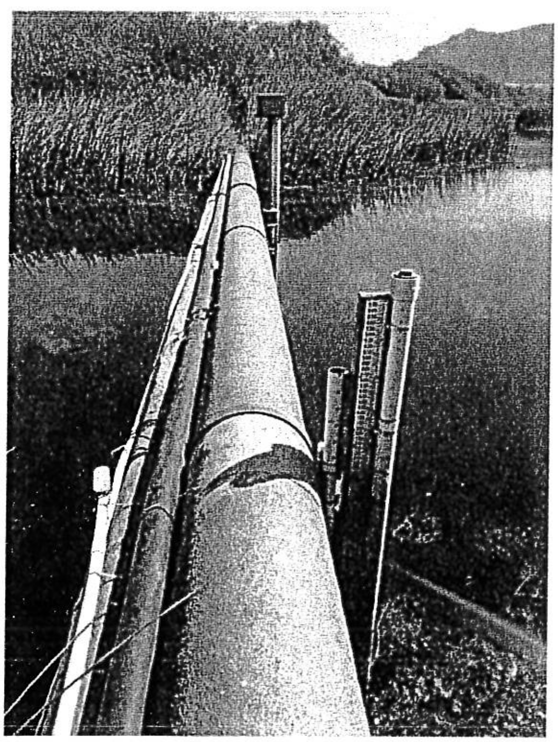

Photo 12: Top of pipeline. View looking east. Carmel River water level monitoring gauge near middle.

Appendix B: Structural Calculations

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By D. Barraza Date 11-2-18 Job # 1768027*00, Phase 2, Task 2.4
Checked by Date Project Carmel Area Wastewater District
Subject Outfall Pipeline Bridge Evaluation Sheet 1 of

- Supplementary 2D Model Results for CRFREE Project, Memo prepared by Balance Hydrologics, Inc. 12 June 2018

	Flowrate, Q (cfs)	Water Surface Elevation (NAVD88)	Velocity (ft/s)
- 10 Year Event			
Existing	2,800	10.4	3.6
Reduced	3,000	10.5	3.9
Proposed	4,000	10.8	4.9
- 100 Year Event			
Existing	10,200	13.3	8.8
Reduced	10,600	13.4	9.3
Proposed	11,300	13.6	9.5

- Background Information
- 24" Dia. Outfall Pipeline - AWWA C-201
  - Min. Wall Thickness = 0.250 in
  - Mortar Lining Thickness = 0.750 in
  - Mortar Coating Thickness = 0.750 in
  - Internal Pressure = 100 psi
  - Rod Reinforcing = ASTM A-15
  - Rod Reinforcing Min Area = 0.30 sq.in./ft.
  - Wire Size = 7/32 in.
  - Wire Spacing = 1.5 inches

Sta.	Inv. El.	T/Pipe El.
18 + 25.98	2.5	4.5
19 + 70.5	4.273006	6.27
20 + 06	4.708589	6.71
20 + 41.5	5.144172	7.14
20 + 77	5.579755	7.58
21 + 12.5	6.015337	8.02
21 + 52	6.5	8.5
	0.012270 slope	
	326 feet	

- Pile Construction - 10" Pipe Shell Concrete Filled
  - Outside Diameter = 10 in.
  - Wall Thickness = 0.188 inch
  - Length (minimum) = 30 feet
  - Pipe Specification = ASTM A252, Grade 1
  - Concrete f'_c = 4,000 psi
  - Allowable Load Bearing Capacity = 15 tons (30,000 #)

- ASCE 7-10 Minimum Design Loads for Buildings and Other Structures
  Chapter 5 - Flood Loads Section 5.4.3 Hydrodynamic Loads
  Where water velocities do not exceed 10 ft/s
  Equivalent hydrostatic loads
  Equivalent surcharge depth, $d_h = aV^2/2g$ (Equation 5.4-1)

By D. Barraza Date 11-2-18 Job # 1768027*00, Phase 2, Task 2.4
Checked by Date Project Carmel Area Wastewater District
Subject Outfall Pipeline Bridge Evaluation Sheet 2 of

- Seismic Design Requirements For Nonbuilding Structures

Nonbuilding Structures, $T \angle 0.06$ seconds (15.4-5)

$V = 0.30 \times S_{DS} \times W \times I_e = 0.30 \times 1.093 \times 1.25 \times W = 0.409875\ W$ if III

$0.30 \times 1.093 \times 1.50 \times W = 0.491850\ W$ if IV

Long = -121.925168

Lat = 36.585200

$S_{DS} = 1.093$

Site Class D - Stiff Soil

Risk Category III

- Equivalent Lateral Force Procedure - Seismic Base Shear

Approximate Fundamental Period

$T_a = C_t h_n^x = 0.02 \times 15^{0.75} = 0.15244$ seconds $T = 1.4 \times 0.15244$

$T_a \angle T_L = 12$ seconds $T = 0.213416$ sec

$V = C_s W$ (12.8-1)

$C_s = S_{DS} / (R/I_e) = 1.093 / (2/1.25) = 0.683125$ if III

0.819750 if IV

$C_s \angle \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{0.627}{0.213416(2/1.25)} = 1.836205$ if III

2.203443 if IV

$C_s > \dfrac{0.5 S_1}{(R/I_e)} = \dfrac{0.5(0.627)}{(2/1.25)} = 0.195938$ if III

0.235125 if IV

- Vertical Seismic Load Effect

$E_v = 0.2 S_{DS} D = 0.2 \times 1.093 \times D = 0.2186\ D$

Combining Nominal Loads Using Allowable Stress

$0.7E = 0.7 \times 0.2186\ D = 0.15302\ D$

$0.75(0.7E) = 0.75 \times 0.7 \times 0.2186\ D = 0.114765\ D$

- Pipeline Service and Factored Loads and Load Combinations

Wall Thickness, t = 0.25 in.

Steel Shell Inside Diameter, ID = 25.5 in

Area of Metal, A_p = 20.22 sq in.

Moment of Inertia, I = 1,676.38 in^4

Section Modulus, S = 128.95 in^3

Weight of Steel, W_p = 68.82 lb/ft

Weight of Liner & Coating, W_l = 126.40 lb/ft

Dead Load of Pipe = 195.22 lb/ft

Weight of Water, W = 196.13 lb/ft

Total Weight per Foot, W = 391.35 lb/ft

Total Pipe Dead Load on Span, W_1 = 6,930 lbs

Total Pipe + Water Dead Load on Span, W_2 = 13,893 lbs

Pinned Moment, $M_{u1} = Wl^2/8$ = 30,753 ft·lbs

Pinned Moment, $M_{u2} = Wl^2/8$ = 61,649 ft·lbs

Pinned Flexure Stress, $S_{f1} = M_1/S$ = 2,862 psi

Pinned Flexure Stress, $S_{f2} = M_2/S$ = 5,737 psi

Ring or Hoop Stress, S_h = 5,200 psi

Total Longitudinal Stress = 5,737 psi + 0.3 x 5,200 psi = 7,297 psi

Total Saddle Reaction = 13,893 lb/2 = 6,946 lbs

Localized Saddle Stress = $k \frac{P}{t^2} \log_e \left[\frac{R}{t}\right]$ = 7,202 psi

Highest Recorded Temperature = 104°

Lowest Recorded Temperature = 20°

Thermal Stress or Change in Unit Stress = $\frac{30 \times 10^6 \times 0.00065 \times 42}{100}$ = 8,190 psi

Effective Stress = $(S_{cs}^2 + S_{ls}^2 - S_{cs} S_{ls})^{1/2}$ = 7,743 psi

By D. Barraza Date 11-2-18 Job # 1768027*00, Phase 2, Task 2.4

Checked by Date Project Carmel Area Wastewater District

Subject Outfall Pipeline Bridge Evaluation Sheet 4 of

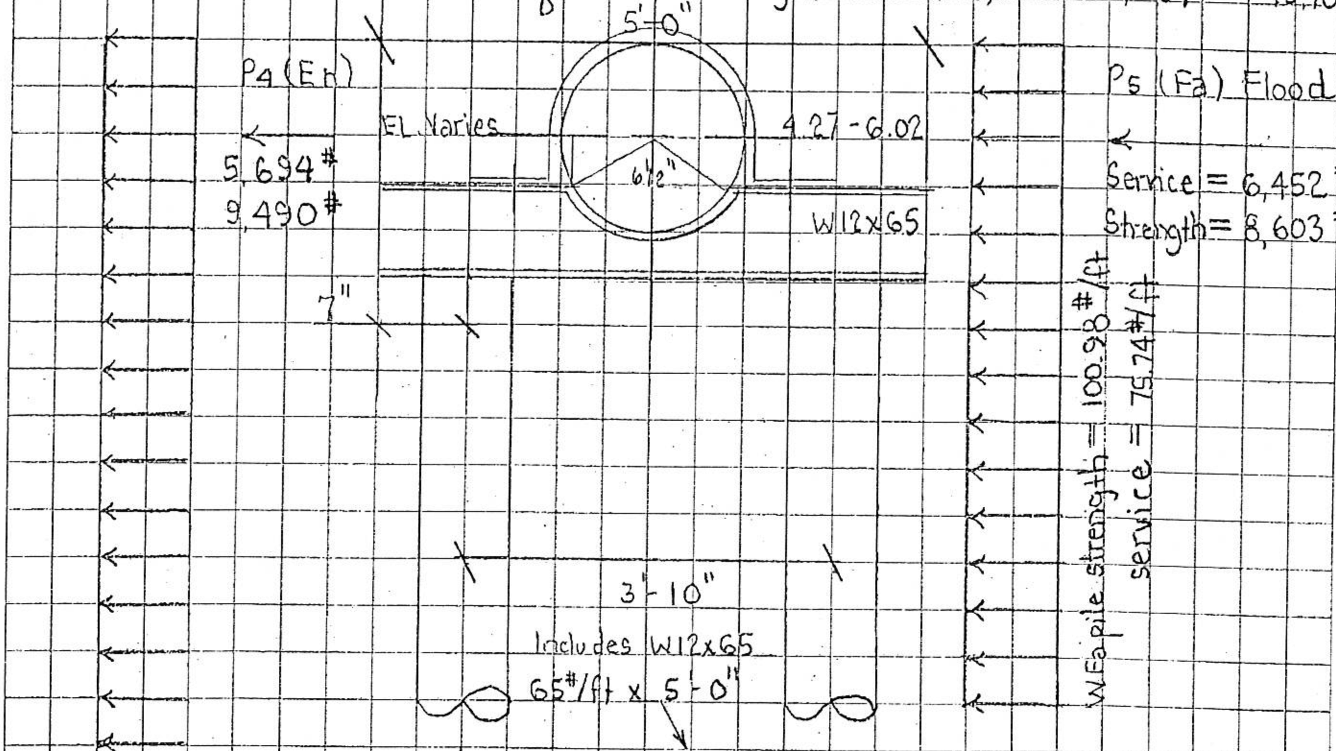
$P_1 + P_2 = 6{,}930^{\#} + 6{,}962^{\#} = 13{,}893^{\#}$

EL. 13.6 (NAVD88)

$P_3 = 3{,}037^{\#}$ (E_v)

EL. 10.4 ASCE 7-10 2.4.1 Eq. 5 Service $= 13{,}893^{\#} + 0.7 \times 3{,}037^{\#} = 16{,}019^{\#}$

2.3.2 Eq. 5 Strength $= 1.2 \times 13{,}893^{\#} + 3{,}037^{\#} = 19{,}709^{\#}$



Hydrodynamic Lateral Sloshing on Pipe & Pile P_6 = ?

$R_{serv} = 8{,}335^{\#}$

$R_{strength} = 10{,}245^{\#}$

R Service $= 16{,}019^{\#}/2 = 8{,}010^{\#}$

R Strength $= 19{,}709^{\#}/2 = 9{,}855^{\#}$

P_1 = Saddle Reaction From Pipe DL $= 195.22 \times 35'\text{-}6"/2 \times 2 = 6{,}930^{\#}$

P_2 = Saddle Reaction From Water DL $= 196.13 \times 35'\text{-}6"/2 \times 2 = 6{,}962^{\#}$

$13{,}893^{\#}$

P_3 = Vertical Seismic $E_v = 0.2186 \times (195.22 + 196.13) \times 35'\text{-}6"/2 \times 2 = 3{,}037^{\#}$

P_4 = Horiz. Seismic $E_h = 0.41 \times 391.35 \times 35'\text{-}6"/2 \times 2$ (Eq. 15.4-5) $= 5{,}694^{\#}$

$0.68 \times 391.35 \times 35'\text{-}6"/2 \times 2$ (Eq. 12.8-2) $= 9{,}490^{\#}$

P_5 = Flood Load $F_a = 242.35^{\#}/ft \times 35'\text{-}6"/2 \times 2$ (2.3.3-2) $= 8{,}603^{\#}$

Assuming V = 10 ft/sec $= 0.75 \times 8{,}603^{\#}$

(2.4.2-2) $= 6{,}452^{\#}$

By D.Barraza Date 11-2-18 Job # 1768027*00, Phase 2, Task 2.4
Checked by Date Project Carmel Area Wastewater District
Subject Outfall Pipeline Bridge Evaluation Sheet 5 of

- Type 316 Stainless Steel Bolts; 2 - 3/4" Bolts Each Side of Strap
  Allowable Shear Stress in a Threaded Fastener
  $F_v = 0.17 F_u$ (threads included in shear plane) $= 0.17 \times 84$ ksi $= 14.28$ ksi
  $F_v = 0.22 F_u$ (threads excluded from shear plane) $= 0.22 \times 84$ ksi $= 18.48$ ksi

Allowable Shear / Bolt $= 0.4418\ in^2 \times 14.28$ ksi $= 6.309$ kips single shear
$0.4418\ in^2 \times 18.48$ ksi $= 8.164$ kips single shear

$\frac{\text{Demand}}{\text{Capacity}} = \frac{6,452^{\#}}{6,309^{\#}} = 1.02 \rightarrow$ FOS $= 0.977$ for flood loading at 10 f/s

$\frac{\text{Demand}}{\text{Capacity}} = \frac{5,694^{\#}}{6,309^{\#}} = 0.90 \rightarrow$ FOS $= 1.108$ Non Building Structures for lateral seismic $I_e = 1.25$
$= 1.08$ $= 0.92$ for lateral seismic $I_e = 1.50$
Equ. 15.4-5

$\frac{\text{Demand}}{\text{Capacity}} = \frac{9,490^{\#}}{6,309^{\#}} = 1.50 \rightarrow$ FOS $= 0.66$ for lateral seismic $I_e = 1.25$
1.80 0.55 for lateral seismic $I_e = 1.50$
Equivalent Lateral Force
Equ. 12.8-2

If two fasteners are mobilize in shear capacity is increased to
$2 \times 6,309^{\#} = 12,618^{\#}$ (threads included from shear plane)
$2 \times 8,164^{\#} = 16,328^{\#}$ (threads excluded from shear plane

Allowable Tensile Stress in a Threaded Fastener
$F_t = 0.33 F_u = 0.33 \times 84$ ksi $= 27.72$ ksi (threaded area)
$F_t = 0.60 F_y = 0.60 \times 42$ ksi $= 25.20$ ksi (body area)

Gross Area $= 0.4418\ in^2 \times 25.20$ ksi $= 11.133$ kips
Effective Thread Area $= 0.4418\ in^2 \times 27.72$ ksi $= 12.246$ kips

Check moment arm on strap ?

- W12 x 65 Pile Bent Saddle Support or Cross Beam

$A = 19.1\ in^2$, $d = 12.12\ in$, $t_w = 0.390$ or $3/8\ in$, width $b_f = 12\ in$

thickness $t_f = 0.605$ or $5/8\ in$, $T = 9\frac{1}{2}\ in$, $k = 1\ 5/16$, $k_1 = 13/16\ in$

$b_f/2t_f = 9.9$, $F_y' = 43.0\ ksi$, $d/t_w = 31.1$, $r_T = 3.28\ in$, $d/A_f = 1.67$

$I_{x-x} = 533\ in^4$, $S_{x-x} = 87.9\ in^3$, $r_{x-x} = 5.28\ in$, $Z_x = 96.8\ in^3$

$I_{Y-Y} = 174\ in^4$, $S_{Y-Y} = 29.1\ in^3$, $r_{Y-Y} = 3.02\ in$, $Z_y = 44.1\ in^3$

Simple Beam Concentrated Load at Center

$R = V = \frac{P}{2} = \frac{14{,}218\#}{2} = 7{,}109\#$ Service DL + LL

$R = V = \frac{P}{2} = \frac{16{,}344\#}{2} = 8{,}172\#$ Service DL + LL + EL

$R = V = \frac{P}{2} = \frac{20{,}099\#}{2} = 10{,}050\#$ Strength 1.2DL + EL

$R = V = \frac{P}{2} = \frac{19{,}905\#}{2} = 9{,}952\#$ Strength 1.4DL

$M_{max} = \frac{PL}{4} + \frac{wl^2}{8} = \frac{13{,}893\# \times 3\text{-}10''}{4} + \frac{65 \times 3'\text{-}10''^2}{8} = 13{,}433\ ft{\cdot}lbs = 161{,}202\ in{\cdot}lbs$

$M_{max} = \frac{16{,}019\# \times 3'\text{-}10''}{4} + \frac{65 \times 3'\text{-}10''^2}{8} = 15{,}470\ ft{\cdot}lbs = 185{,}651\ in{\cdot}lbs$

$M_{max} = \frac{19{,}709\# \times 3'\text{-}10'}{4} + \frac{78 \times 3'\text{-}10''^2}{8} = 19{,}031\ ft{\cdot}lbs = 228{,}372\ in{\cdot}lbs$

$M_{max} = \frac{19{,}450\# \times 3'\text{-}10''}{4} + \frac{91 \times 3'\text{-}10''^2}{8} = 18{,}807\ ft{\cdot}lbs = 225{,}680\ in{\cdot}lbs$

$\Delta_{max} = \frac{PL^3}{48EI} + \frac{5wl^4}{384EI} = \frac{13{,}893\# \times 46''^3}{48 \times 29 \times 10^6 \times 533\ in^4} + \frac{5 \times 65 \times 46''^4}{384 \times 29 \times 10^6 \times 533\ in^4}$

$= 0.001823\ in + 0.000245\ in$

$= 0.002068\ in$

– Allowable Stress and Strength – Strong Axis Bending of W12 x 65 Beam

$F_b = 0.66\,F_y = 0.66 \times 36\,ksi = 23.76\,ksi$

$M_{max} = S_{xx} \cdot F_b = 23.76\,ksi \times 87.9\,in^3 = 2{,}088.504\,k{\cdot}in > 174.042\,k{\cdot}ft$

$L_c = \frac{76\,b_f}{\sqrt{F_y}} = \frac{76 \times 12}{\sqrt{36}} = 152\,in$ or 12'-8"

$L_c = \frac{20{,}000}{(d/A_f)\,F_y} = \frac{20{,}000}{1.67 \times 36} = 332\,in$ or 27'-8"

– Allowable Shear Stress and Strength

$h/t_w = 1\,5/16\,/\,3/8 = 3.50 < 380/\sqrt{36} = 63.33$

$F_v = 0.40\,F_y = 0.40 \times 36\,ksi = 14.40\,ksi$

$V_{max} = F_v\,d\,t_w = 14.40\,ksi \times 12.12\,in \times 3/8\,in. = 65.448\,kips > 7.109\,kips$

Critical Thickness Web $= \frac{7.109\,kips}{14.40\,ksi \times 12.12\,in} = 0.0407$ inches (0.0519 in)
5.62 in — 0.0878 inches about 1/12 in

$= \frac{10.050\,kips}{14.40\,ksi \times 12.12\,in} = 0.0576$ inches about 1/16 in.
5.62 in — 0.1242 inches (0.0734 in) about 1/8 in

If we only consider web depth between fillets (exclude flange & fillet)
If we consider web depth reduced at saddle to minimum depth
$V_{max} = 14.40\,ksi \times 9\,1/2\,in \times 3/8\,in = 51.300\,kips > 7.109\,kips$
$V_{max} = 14.40\,ksi \times 5.62\,in \times 3/8\,in = 30.348\,kips > 7.109\,kips$

– Web Crippling – AISC 1.10.10

Min. Bearing Length Reactions $= \frac{R}{t(N+k)} < 0.75\,F_y = \frac{8.172\,kips}{t(10\,in + 1\,5/16)} \le 0.75 \times 36 = 27.0\,ksi$

Web-Crippling Stress Under Concentrated Load $= \frac{R}{t(N+2k)} \le 0.75 F_y = \frac{16.344}{t(27.23 + 2 \times 1\,5/16)} \le 27.0\,ksi$

$t > 0.292$ inches

$t > 0.020$ inches $N = \pi \times 26/3 = 27.23\,in$

By D. Barraza | Date 11-2-18 | Job # 1768027*00, Phase 2, Task 2.4

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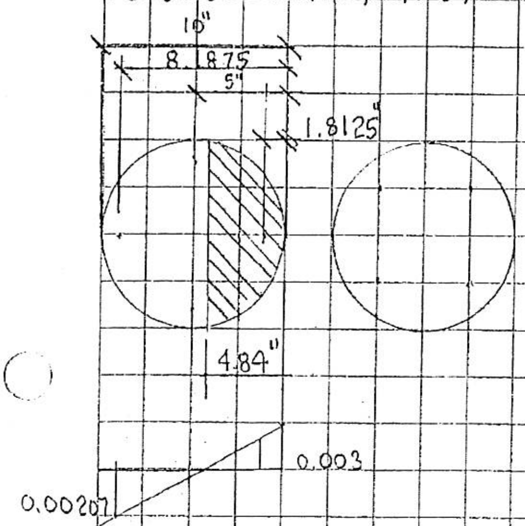
Subject Outfall Pipeline Bridge Evaluation | Sheet 8 of

– Pile Evaluation – Round Tied Column –

10 in dia. round tied column, $f'c = 4{,}000$ psi, $f_y = 60{,}000$ psi, 4-#5 bars

A_g =	78.53 in²	113.09 in²
A_{st} =	1.24 in²	1.24 in²
rho =	0.015788	0.010964
Net A_c =	77.29 in²	111.85 in²

Balance (ϕP_b, ϕM_b) Point 5



Compression Side

$$c = \left[\frac{0.003}{0.003 + 0.00207}\right] 8.1875 = 4.8456 \text{ in}$$

Compression Stress Block

$$a = \beta_1 c = [0.85 - 0.05\,(f'c - 4)]\; 4.8456 \text{ in}$$

$$a = 4.1188$$

Calculate Net Area of Concrete in Compression Stress Block, Acc (shaded)

$$\text{Angle} = \arcsin \frac{R - a}{R} = \arcsin \frac{5 - 4.1188}{5} = 10.15^\circ$$

$$\text{Total Angle of Sector} = 180^\circ - 2(10.15^\circ) = 159.69^\circ$$

$$A_{cc} = 78.53 \times \frac{159.69^\circ}{360^\circ} - 2 \times \tfrac{1}{2} \times 0.72 \times 4.92 - 0.31$$

$$A_{cc} = 30.984 \text{ in}^2$$

Forces in bars:

$$T_1 = 60(0.31) = 18.60 \text{ k}$$

$$T_2 = 60\left[\frac{0.000096}{0.00207}\right] .31 \times 2 = 1.72 \text{ k}$$

$$20.32 \text{ k}$$

$$C_{S1} = 60 \times 0.31 \times 1 \quad 18.60 \text{ k}$$

Compression in Concrete = C_c = 0.85 x 4 x 30.984 in² = 105.345 kips

ϕP_b = 0.70 (105.345 kips + 18.60 kips − 20.32 kips) = 72.537 kips

Pile Evaluation – Round Tied Column – Balance (ϕP_b, ϕM_b) Point 5

Moment

$$\bar{X} = \frac{(78.53\,in^2/2)(0.4244)5 - ((10+9.843477)/2\,(0.7268)(0.3634) - 0.31\,(3.1875)}{(78.53/2) - (0.7268)(9.921739) - 0.31}$$

$$\bar{X} = \frac{79.71\ in^3}{3.74\ in^2} = 2.51\ in$$

$$\phi M_b = 0.70\,[105.345 \times 2.51 + 0.31 \times 60 \times 3.1875]$$

$$\phi M_b = 226.59\ kip \cdot in = \underline{18.8827\ kip \cdot ft}$$

Alternate Bar Orientation

– Compression Side ; $c = \frac{0.003}{0.00207 + 0.003}(7.253903) = 4.293126\ in$

Compression "stress block" ; $a = \beta_1 c = 0.85 \times 4.293126 = 3.649157\ in$

Calculate net area of concrete in compression stress block, A_{cc} (shaded):

Angle ; $\arccos \frac{R - a}{R} = \frac{5 - 3.649157}{5} = 74.3257°$

$R(\sin 74.3257°) = 5\,(0.962813) = 4.814065\ in$

Sector Angle $= 2(74.3257°) = 148.6514°$

$$A_{cc} = 78.53 \times \left[\frac{148.6514}{360}\right] - 2\,(0.31) - (2)(1/2)(0.643969)(4.814065)$$

$$= 28.706543\ in^2$$

$$C_c = 0.85 \times 4 \times 28.706543 = 97.602\ kips$$

$$C_s = \frac{(2)(0.31)(60)(108)}{207} = \underline{19.408\ kips}$$

$$117.010\ kips = \text{Total } C$$

$$T = (2)(0.31)(60) = 37.20\ kips$$

$$\phi P_b = 0.70\,(117.010 - 37.200) = \underline{55.867\ kips}$$

– Design Moment Strength

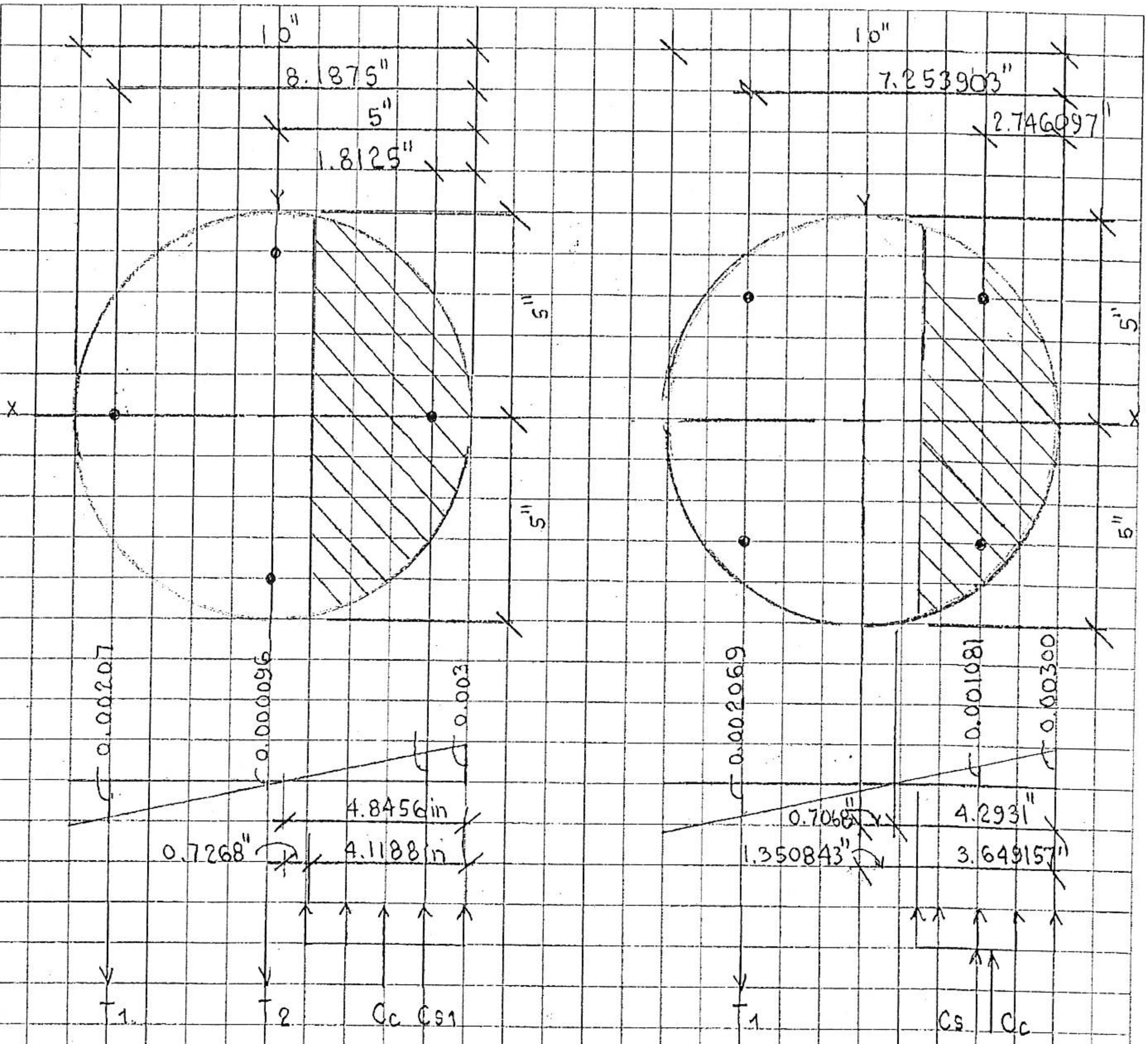
$$\bar{X} = \frac{(78.53\,in^2/2)(0.4244)(5) - ((10+9.62813)/2\,(1.35)(0.67) - 2\,(0.31)(2.253903)}{(78.53/2) - (1.35)(9.814065) - 2\,(0.31)}$$

$$\bar{X} = 73.046086/25.396012 = 2.876282\ in$$

$$\phi M_b = 0.70\,[(97.602)(2.876282) + (19.408 + 37.20)(2.253903)]$$

$$\phi M_b = 285.823\ kip \cdot in = \underline{23.818\ kip \cdot ft}$$

By D. Barraza Date 11-2-18 Job # 1768027*00, Phase 2, Task 2.4
Checked by Date Project Carmel Area Wastewater District
Subject Outfall Pipeline Bridge Evaluation Sheet 10 of



Bar Orientations at Balance Point

Assumptions: 1 1/2" concrete bar cover.
$f'_c = 4{,}000$ psi
$f_y = 60{,}000$ psi
$E_y = 29{,}000{,}000$ psi
4 - #5 bars, $A_s = 0.31\ in^2$/bar
No contribution of steel pipe in pile strength.

Pile Evaluation

Maximum Moment on Piles from Flood Loads:

$M_u = 8{,}603^{\#} \times 7\ ft/2 = 60.221\ kip\ ft\ /2 = 30.110\ k \cdot ft$

$8{,}603^{\#} \times 10\ ft/2 = 86.030\ kip\ ft\ /2 = 43.015\ k \cdot ft$

$S_x = \dfrac{\pi (d^4 - d_i^{\,4})}{32 d} = \dfrac{\pi (10^4 - 9.624^4)}{(32)(10)} = 13.9534\ in^3$

$M_{max} = S_x F_b = 23.76\ ksi \times 13.9534\ in^3 = 331.532\ kip \cdot in\ (27.627\ kip \cdot ft)$

Allowable Strength

$Z_x = \dfrac{d^3}{6} - \dfrac{d_i^3}{6} = \dfrac{10^3}{6} - \dfrac{9.624^3}{6} = 18.10198\ in^3$

Round HSS, F8 Yielding Limit States (Plastic Moment)

$M_n = M_p = F_y Z = 30{,}000\ psi \times 18.10198\ in^3 = 543.059\ kip \cdot in\ (45.255\ k \cdot ft)$

$\phi_b M_n = 0.90 \times 543.059\ kip \cdot in = 488.753\ kip \cdot in\ (\underline{40.729\ kip \cdot ft})$

$M_u = \dfrac{w l^2}{2} = \dfrac{100^{\#}/ft \times 7\ ft^2}{2} = 2{,}450\ ft \cdot lbs$

$= \dfrac{100^{\#}/ft \times 10 ft^2}{2} = 5{,}000\ ft \cdot lbs$

808

M_u @ 7'-0" to mudline $= 30.110\ k \cdot ft + 2.450\ k \cdot ft = 32.560\ k \cdot ft$

M_u @ 10'-0" to mudline $= 43.015\ k \cdot ft + 5.000\ k \cdot ft = 48.015\ k \cdot ft$

3.441 k·ft

$\phi M_b = 40.729\ k \cdot ft + 18.882 = 59.611\ k \cdot ft$

$\phi M_b = 40.729\ k \cdot ft + 23.818 = 64.547\ k \cdot ft$

$\phi M_b = \sqrt{40.729^2 + 18.882^2} = 44.892\ k \cdot ft$

$\phi M_b = \sqrt{40.729^2 + 23.818^2} = 47.182\ k \cdot ft$

$D/t = \dfrac{10}{0.188} = 53.191 < \dfrac{0.45 E}{30} = 435$ o.k.

4 ft/sec scenario

61° Carmel by the Sea, California

Universal Time: Friday, 16 Nov 2018, 17:59

Historic Average

Carmel by the Sea, California

Monthly Averages & Records - °F | °C

Date	Average Low	Average High	Record Low	Record High	Average Precipitation	Average Snow
January	43°	60°	22° (1949)	84° (1962)	4.19"	0"
February	45°	61°	26° (1989)	86° (1995)	3.75"	0"
March	46°	62°	32° (1976)	85° (1953)	3.53"	0"
April	47°	64°	35° (1976)	93° (1981)	1.48"	0"
May	48°	65°	38° (1950)	95° (1997)	0.5"	0"
June	50°	67°	42° (1962)	101° (1961)	0.2"	0"
July	52°	68°	43° (1953)	98° (1959)	0.09"	0"
August	53°	70°	45° (1956)	96° (1993)	0.11"	0"
September	53°	71°	43° (1950)	101° (1971)	0.28"	0"
October	51°	70°	35° (1949)	104° (1987)	1.06"	0"
November	47°	64°	35° (1993)	95° (1956)	2.43"	0"
December	43°	60°	20° (1990)	89° (1958)	2.73"	0"

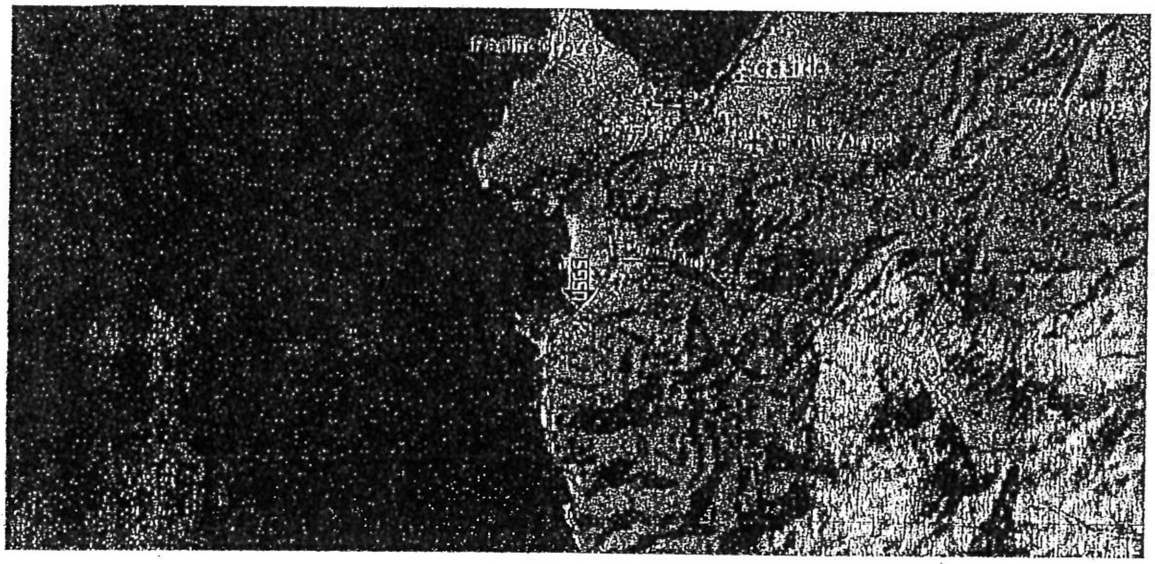
Daily Averages & Records - °F | °C

Date	Average Low	Average High	Record Low	Record High	Average Precipitation	Average Snow
Nov 1	49°	67°	40° (1949)	91° (1966)	0.06"	NA
Nov 2	49°	67°	42° (1949)	90° (1997)	0.06"	NA
Nov 3	49°	67°	41° (1949)	88° (1976)	0.07"	NA
Nov 4	48°	67°	39° (1973)	92° (1949)	0.07"	NA
Nov 5	48°	66°	41° (1986)	84° (1961)	0.07"	NA
Nov 6	48°	66°	42° (1996)	83° (1959)	0.07"	NA
Nov 7	48°	66°	41° (2001)	83° (1955)	0.07"	NA
Nov 8	48°	66°	40° (1971)	95° (1956)	0.08"	NA
Nov 9	48°	65°	39° (1961)	93° (1956)	0.08"	NA
Nov 10	47°	65°	40° (2000)	92° (1956)	0.08"	NA
Nov 11	47°	65°	38° (1994)	83° (1969)	0.08"	NA
Nov 12	47°	65°	35° (1985)	80° (1986)	0.08"	NA
Nov 13	47°	65°	35° (1985)	82° (1949)	0.08"	NA
Nov 14	47°	64°	36° (1978)	81° (1949)	0.08"	NA
Nov. 15	47°	64°	36° (1964)	83° (1949)	0.08"	NA
Nov 16	47°	64°	35° (1964)	81° (1949)	0.08"	NA
Nov 17	46°	64°	36° (1958)	78° (1949)	0.09"	NA
Nov 18	46°	64°	37° (1964)	84° (1989)	0.09"	NA
Nov 19	46°	63°	35° (1985)	82° (1989)	0.09"	NA
Nov 20	46°	63°	38° (1977)	82° (2002)	0.09"	NA
Nov 21	46°	63°	37° (1985)	75° (1954)	0.09"	NA
Nov 22	46°	63°	38° (1985)	80° (1954)	0.09"	NA
Nov 23	46°	63°	36° (1952)	80° (1959)	0.09"	NA
Nov 24	45°	63°	37° (1993)	85° (1956)	0.09"	NA
Nov 25	45°	62°	35° (1993)	79° (1956)	0.09"	NA
Nov 26	45°	62°	39° (1974)	82° (1959)	0.09"	NA
Nov 27	45°	62°	37° (1952)	78° (1969)	0.09"	NA

USGS Design Maps Summary Report

Jser-Specified Input

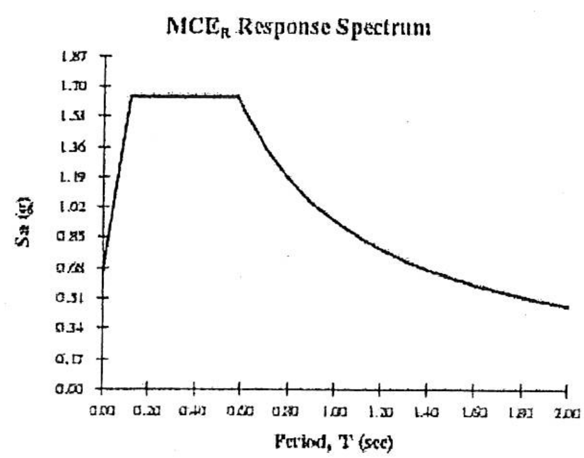
Report Title	Carmel Area Wastewater District Outfall Pipeline Thu November 15, 2018 22:56:54 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	36.5352°N, 121.92517°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

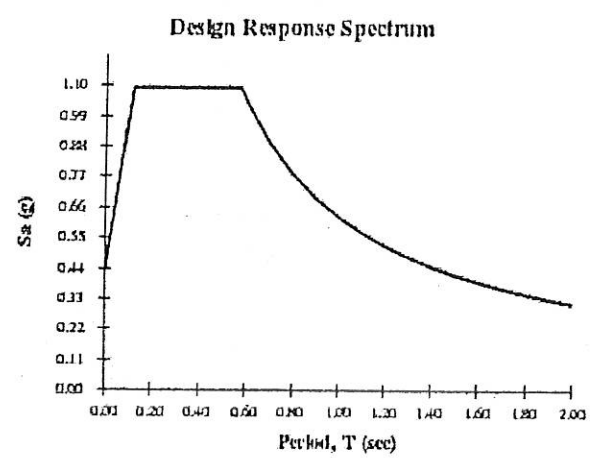


JSGS-Provided Output

$S_S =$	1.640 g	$S_{MS} =$	1.640 g	$S_{DS} =$	1.093 g
$S_1 =$	0.627 g	$S_{M1} =$	0.940 g	$S_{D1} =$	0.627 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and leterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.