4.0 Wall Analysis

To gain a sense of the original capacity of the wall against the loss of capacity due to decades of environmental attack and sustained deterioration, JMT analyzed the wall as originally designed and again with 50% section loss of the pile head.

4.1 Loading

a) Marine Loads

The primary purpose of a seawall is to form a coastal defense and protect areas of habitation from the sea and associated coastal forces. To determine the marine loading requirements of the seawall, the engineering analysis considered the water forces in three phases: hydrostatic, hydrodynamic and breaking wave.

Hydrostatic load is the pressure at a given depth in a non-moving liquid that is the result of the weight of the liquid acting on a unit area. Hydrodynamic loads incorporate the effects of the water in motion flowing against and around rigid structures. The breaking waves exert considerable loading onto the wall. The wave loading is a result of waves propagating over the water surface and striking the vertical face of a structural member. These are accompanied by an impact load that also represents miscellaneous debris that would be carried by high water conditions and striking the structure.

b) Dead, Live and Surcharge Loads

The term dead load refers to the material weight of the structure and any accompanying items that can be considered permanent attachments. In the Low Battery wall case, this is the weight of the building materials used for the wall along with accompanying backfill, sidewalk, railings and any permanent items. Live load describes temporary or transient pressures that have a short term effect on the structure such as pedestrian loads and vehicle loads on the sidewalk itself. A surcharge load is any load imposed on the ground surface in close proximity to cause lateral pressure acting on the structural system. This would be load induced by vehicular traffic on Murray Boulevard.

c) Seismic Loading

Seismic load is the application of an earthquake generated agitation of a building or structure acting at the contact surface of the structure and the ground. Due to the geotechnical aspects of the peninsula, liquefaction of the soils must also be considered. From the soil boring logs found in appendix B, the local strata was composed primarily of fine grain silts until the Cooper Marl is reached. During a seismic event, the pore pressures within the soil increases until the consistency of the soils achieve a liquid state. At that point the lateral capacity of the liquefied soil approaches zero. Fortunately the Marl, an over-consolidated clay layer, has an extremely low permeability that resists the increased pore pressures and maintains the skin friction support of embedded foundation piles. Due to the low resistance of the material above the Cooper Marl, and the fact that the timber piles are not all embedded into the marl, it is highly unlikely that the wall would survive the design earthquake of today’s codes in its current condition.

4.2 Analysis of Existing Structure

a) Structural Observations and Findings:

A structural analysis was performed on the wall for original as-built conditions, assuming that it was built according to details found in the archives and in 'Like New' condition. Loads were calculated and applied in accordance with ASCE 7-10 "Minimum Design Loads for Buildings and other Structures". The analysis determined that the system behaves in an adequate fashion and that piles were stressed to approximately 60 percent capacity.

The same loads were applied to the system assuming 50 percent degradation, based on the investigation and testing performed. It was found that with the reduced cross sectional area, the wooden piles become overstressed, resulting in a potential lack of stability to the overall system.

Considering the age and the extreme environmental conditions to which the seawall has been exposed, the wall was in better than expected condition. Most of the concrete gravity wall was composed of unreinforced concrete so chloride attack to reinforcing steel was not a problem except at the top which had reinforcing steel. The curb and barrier system at the sidewalk level was in very poor condition. These areas had cracks, spalls and leaching rust stains throughout the length of the Low Battery. Due to the extensive damage, the curb and barrier portions of the wall were recommended to be replaced instead of rehabilitated.

The seaward face of the wall has suffered from what appears to be a scouring effect that has eroded away the cement and paste components of the concrete, leaving the large aggregate exposed. This deterioration was particularly evident closer to the turn where, based on the wall orientation and the harbor and bathymetry, water velocities were expected to be greater.

The protective concrete skirt at the bottom of the wall was broken at various locations allowing water to infiltrate and to cause damage to the piles and the low level platform which supports the concrete gravity wall as well as allow an avenue for fill to migrate from behind the wall.

Timber connections at the low level platform were deteriorated and spongy and had corroded bolts. The notched and bolted connections between the piles and the low level platform was the critical link in the system. Because it was located in the tidal range it was susceptible to corrosion, rot and worm damage and displayed the expected deterioration.

Due to significant deterioration of the timber piles and low level platform, the seawall has experienced notable movement resulting in severe cracks in the unreinforced concrete. Foundation repairs were recommended in order to prevent continued deterioration of the seawall.

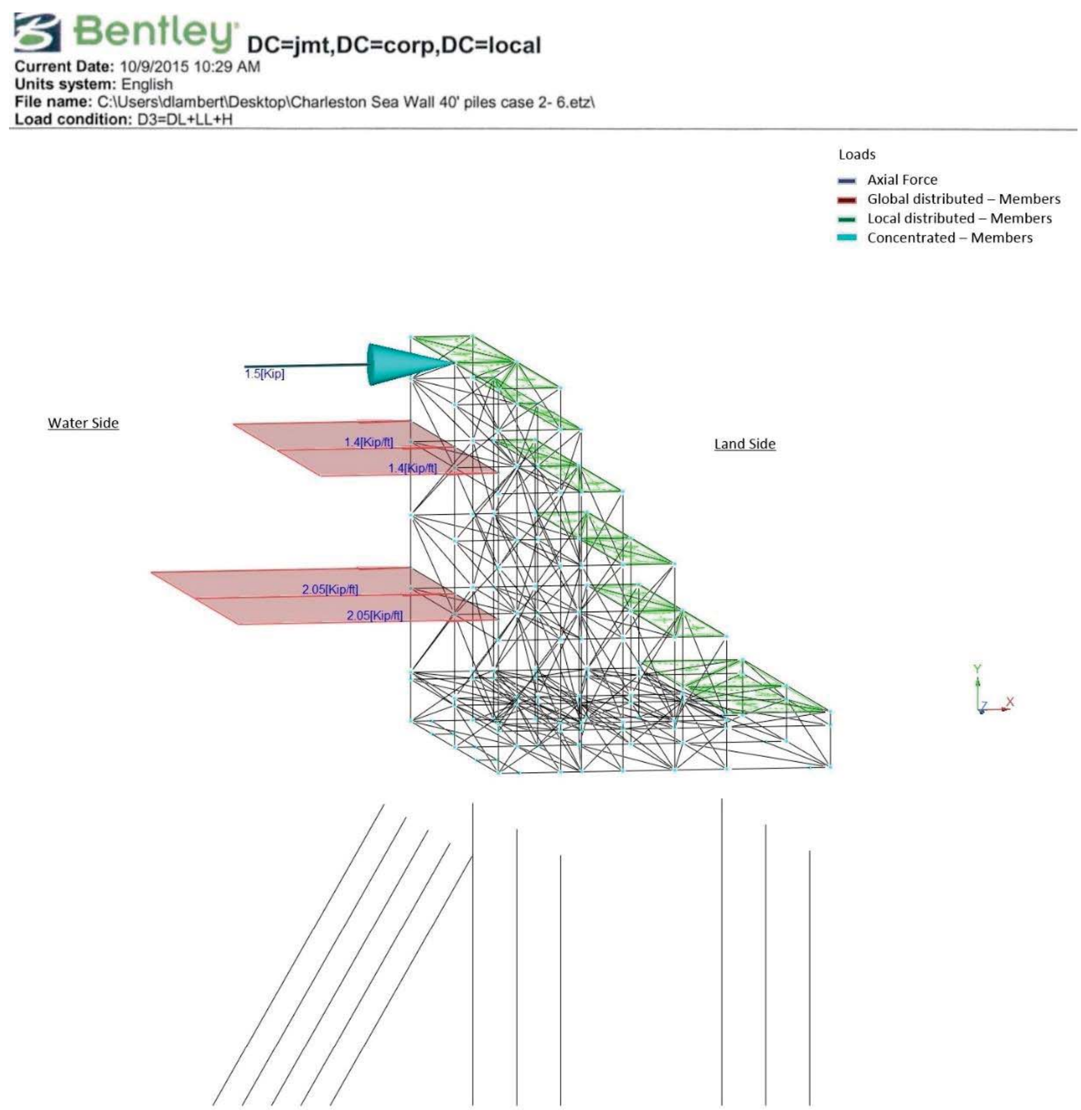
The horizontal and vertical joint deterioration may also impact the structural integrity of the wall. Through time, these joints have opened to the point that water is able to freely flow through the joints. There was evidence that the soils behind the wall had scoured out during falling tides causing settlement of the sidewalk and roadway. The open joints also permit the entry of marine borers and other organisms that cause devastating damage to untreated and newly exposed wood piles and timber foundation. The wood, which was already experiencing breakdown due to rot, will allow the organisms easy and quick access.

While phase one of the project was not directed to investigate the seismic analysis of the existing structure, it was readily apparent that a significant seismic event could be detrimental to the seawall. The large mass of wall concrete will create excessive force on the severely deteriorated pile heads and low strength soils providing resistance to lateral loads. This could cause severe rotational and sliding movement of the wall during a significant seismic event.

Future performance of the wall will depend mostly upon the frequency and severity of storms. A very severe storm combined with water washing under could cause a portion of the wall to collapse.

b) Reactions to the Existing Structure based on Load Cases

In looking at the wall from a historic loading/analysis perspective, the present day code-driven loads (described above) have been placed on the wall to ascertain their reactions to the support system. As such, the first phase was to examine the present environmental loads on the original structure to see the performance. JMT used RAM structural modeling software to analyze this scenario. Below is a partial graphic of the structural modeling depicting this scenario.



The preliminary results of this scenario indicated that the 1910 “good condition” system, as originally built with full capacity, was nearing the stress and design limits of the system under present day loading. This was estimated to be at 90-95% of the capacities of the piles and the wall system.

Bentley DC=jmt,DC=corp,DC=local

Current Date: 10/9/2015 12:39 PM
Units system: English
File name: C:\Users\dlambert\Desktop\Charleston Sea Wall 40' piles 6.etz\

50% Degraded Piles
Unity Check Comparison
Design code: ANSI/AF&PA NDS-2005 ASD

Wood Design

Report: Summary - For all selected load conditions

Load conditions to be included in design :

D1=DL
D2=DL+LL
D3=DL+LL+H
D4=DL+0.6H+1.5Fa
D5=0.6DL+0.6H+1.5Fa
D6=DL+H+0.7E
D7=DL+0.75LL+H+0.75E
D8=0.6DL+H+0.7E

Description	Section		Member	Ctrl Eq.	Ratio	Status	Reference
Timber Piles	*POST 6*	Land Pile	97	D1 at 0.00%	0.00	OK	
				D2 at 0.00%	0.00	OK	
				D3 at 81.62%	**0.75**	**OK**	(Eq. 3.9-3)
				D4 at 81.62%	0.28	OK	(Eq. 3.9-3)
				D5 at 81.62%	0.28	OK	(Eq. 3.9-3)
				D6 at 81.62%	0.51	OK	(Eq. 3.9-3)
				D7 at 81.62%	0.51	OK	(Eq. 3.9-3)
				D8 at 81.62%	0.51	OK	(Eq. 3.9-3)
		Middle Pile	98	D1 at 0.00%	0.00	OK	
				D2 at 0.00%	0.00	OK	
				D3 at 81.62%	**1.64**	**N.G.**	(Eq. 3.9-3)
				D4 at 81.62%	0.48	OK	(Eq. 3.9-3)
				D5 at 81.62%	0.48	OK	(Eq. 3.9-3)
				D6 at 81.62%	1.15	N.G.	(Eq. 3.9-3)
				D7 at 81.62%	1.15	N.G.	(Eq. 3.9-3)
				D8 at 81.62%	1.15	N.G.	(Eq. 3.9-3)
		Batter Pile	99	D1 at 0.00%	0.00	OK	
				D2 at 0.00%	0.00	OK	
				D3 at 81.91%	**1.31**	**N.G.**	(Eq. 3.9-1)
				D4 at 81.91%	0.63	OK	(Eq. 3.9-1)
				D5 at 81.91%	0.63	OK	(Eq. 3.9-1)
				D6 at 81.91%	0.82	OK	(Eq. 3.9-1)
				D7 at 81.91%	0.82	OK	(Eq. 3.9-1)
				D8 at 81.91%	0.82	OK	(Eq. 3.9-1)

The second phase of the analysis was to look at the same geometry and to compare it in its present state of deterioration – which was liberally estimated from the field work done, to be in the 50% range. Under this scenario, the piles were assumed to only have 50% of their original section properties and connectivity when in actuality, the degradation is most likely further advanced. Once again the model was run and the results showed that there was a great decrease in capacity such that the system was now overstressed. This indicates that the seawall did not provide adequate capacity and factors of safety for present environmental loadings.