Design and Construction of Haifa Port Expansion

By Bill Paparis, P.E.

The existing Haifa Port handles virtually all general cargo to and from northern Israel, in addition to accommodating most of the passenger ship traffic in the area. Currently, the Port operates beyond its design capacity. The Carmel Project in Haifa Port – Phase A, whose construction cost is approximately $100 MM, is the first stage of a long-ranged expansion of the Port of Haifa. The project is the central phase of the Israel Port Development and Assets Company Ltd's program developed to increase the capacity of the Port to 20 million tons of cargo annually, with a container capacity of 900,000 twenty foot equivalent units (TEUs).

The project includes construction of 2,000 meters (6562 feet) of quays, reclamation of approximately 270,000 square meters (2,900,000 square feet) of land, and construction of a container terminal, which includes rail mounted gantry runways. The project also includes dredging to Elevation -15.5 meters (-50.9 feet) Israel Land Survey Datum (6.5 centimeters (2.56 inches) above mean low water) to accommodate Post-Panamax container ships (vessels which are too wide to traverse the Panama Canal) and modern high capacity general and bulk cargo vessels. The maximum design vessel has a draft of 14.0 meters (45.9 feet), and the 15.5 meter (50.9 feet) elevation insures adequate under-keel clearance.

A plan of the terminal that is presently being constructed is shown in *Figure 1*. The major structures involved are as follows:

- Quay 2 (container quay), with a length of approximately 700 meters (2297 feet), will supplement the existing container berth at Quay 1.
- Quay 3, with a length of approximately 250 meters (820 feet), will service general and bulk cargo vessels.
- Retaining structure, with a length of approximately 1,000 meters (3281 feet), will serve as a boundary for the east side of the reclamation area.

The design needed to overcome a number of major issues, including difficulty in obtaining

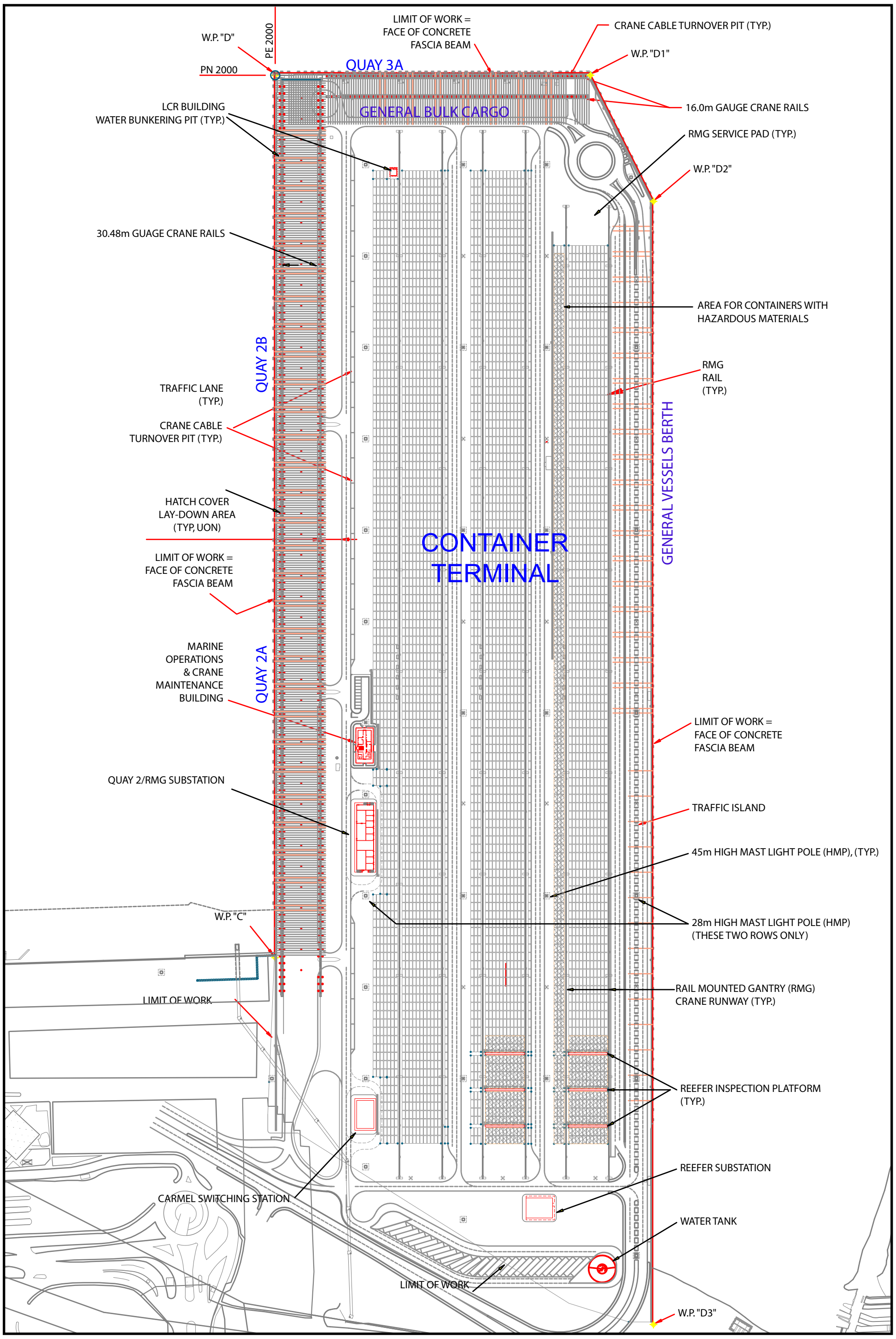

Figure 1: Plan of terminal

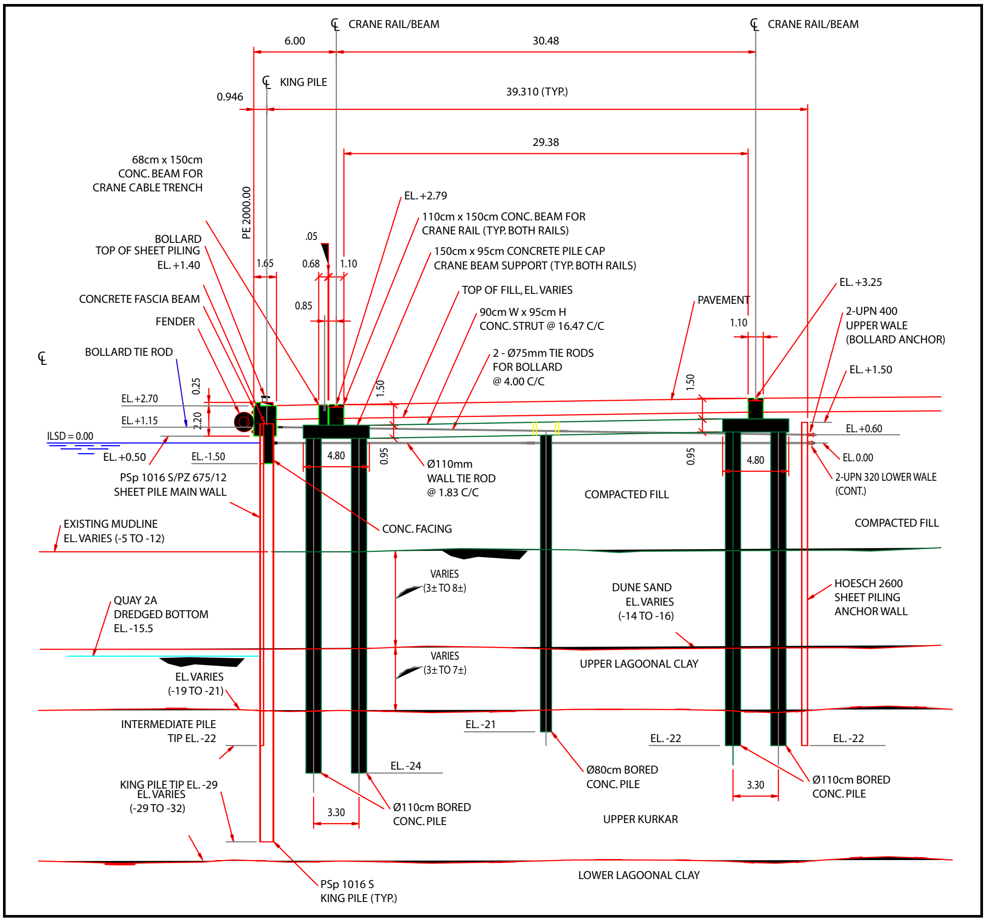

Figure 2: Typical section of container quay

adequate quantities of suitable sand for reclamation from the dredging operation. Design parameters had to account for a moderate level of earthquake accelerations and associated potential for liquefaction of hydraulic fill. In addition, construction related issues included potential difficulty in driving piles through cemented sandstones and large settlements of crane beams and pavement due to underlying clay layers.

In order to address these issues, numerous studies and investigations were carried out to obtain sufficient data to develop the most cost effective design. In addition, several alternatives for the quay design were evaluated, taking into account operational considerations, technical merit, constructability, and both capital and maintenance cost. The most cost effective structural system was determined to be a king pile system for the main wall, anchored to a sheet pile bulkhead via tie rods.

Three design earthquake levels were used in the analyses:

1. Level 1 (Operating Level Earthquake), 50% probability of exceedance in 50 years: Richter Scale magnitude 5.0, peak ground acceleration (PGA) 0.08 g.
2. Level 2 (Contingency Level Earthquake), 10% probability of exceedance in 50 years: Richter Scale magnitude 6.0, PGA 0.20 g.

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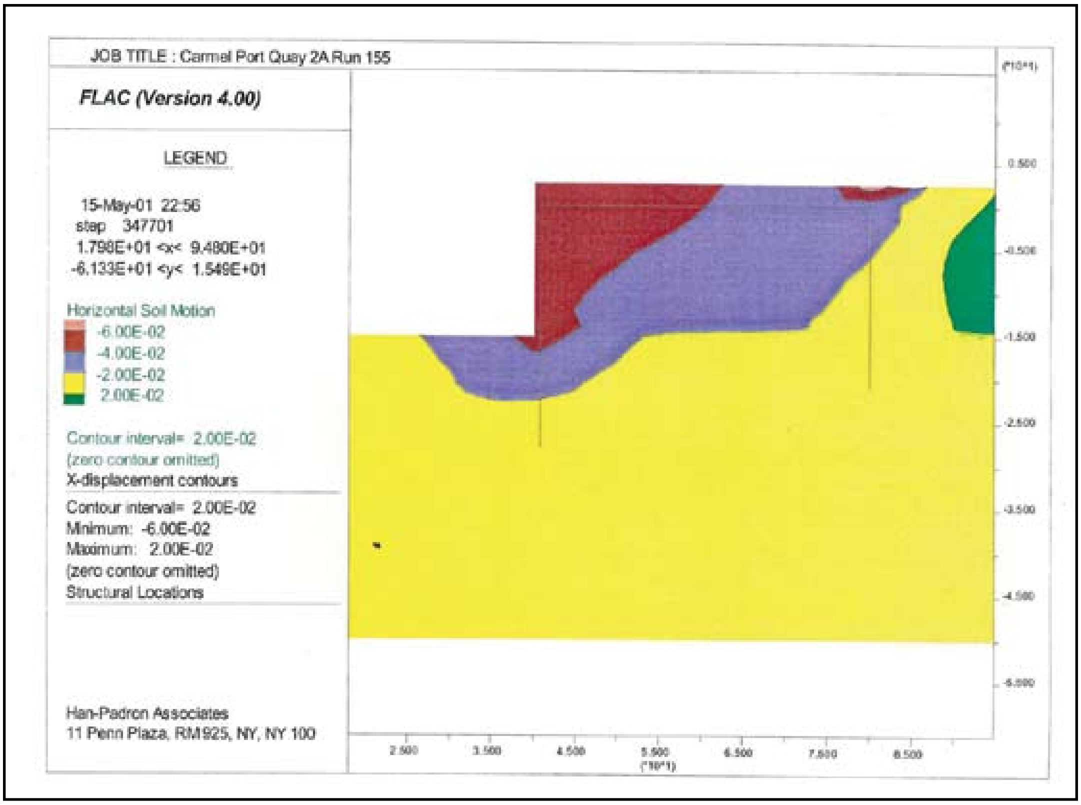


Figure 3: Displacement contours under contingency level earthquake

3. Retaining Structure Contingency Level Earthquake, 10% probability of exceedance in 20 years: Richter Scale magnitude 5.5, PGA 0.10 g. This reduced level of shaking accounts for the potentially temporary nature (20 year design life) of the Retaining Structure, as there is a planned expansion to the east, which will obviate its function in the future.

The geotechnical conditions of the site typically consist of a thin layer of silty clay/clayey silt (seabed mud), medium to dense dune sand and littoral sand, medium to stiff clay, cemented sandstone (known locally as kurkar), and alternating layers of clay and cemented sandstone.

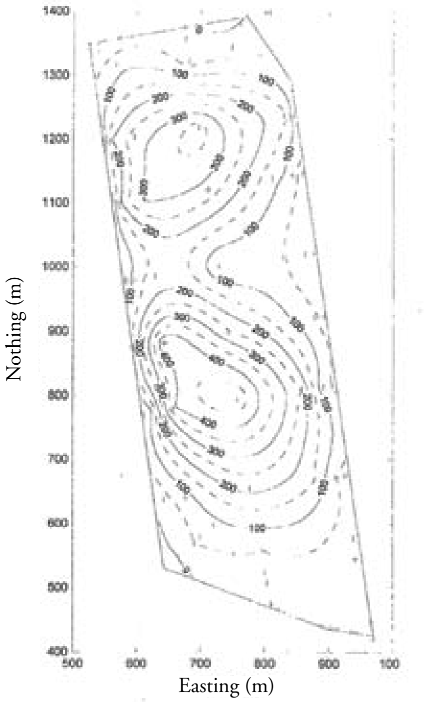
Figure 2 shows a typical section of Quay 2, the container quay, and also depicts the soil layers. As shown, the quays typically consist of a king pile system for the main wall and a steel sheet pile system for the anchor wall, with tie rods placed at approximately the low water level to connect the two walls through a steel whaler at the anchor wall. The main wall frames into a concrete fascia beam on which are mounted cylindrical rubber fenders and 120 metric tons (132 tons) mooring bollards. The crane support system consists of pairs of bored concrete piles on both the water side and the land side, with bents spaced at 5.69 meters (18.7 feet) centers. The piles support cast-in-place concrete pile caps, on which are founded cast-in-place concrete crane beams, to which A120 crane rails are anchored.

Sheet Pile System

The sheet pile walls were designed for static earth pressure and 5 metric tons per square meter (1,024 pounds per square foot) surcharge loading using conventional methods of analyses. However, preliminary pseudo-static analyses for seismic loading indicated that the size and depth of the sheet pile sections would have to increase significantly. Therefore, in an attempt to economize on the design, a dynamic analysis using the results of a site specific seismic survey was conducted. These analyses were carried out using the finite difference program Fast Lagrangian Analysis of Continuum (FLAC) developed by HCITasca, which accounts for potential loss of shear strength and liquefaction of the sand fill. As noted previously, two levels of shaking were considered: an operating level earthquake (OLE) and a contingency level earthquake (CLE). In order to minimize construction costs, a performance based design, which was recommended and agreed to by the Owner, was performed. Analyses were carried out whereby deflections were limited to 10 centimeters (3.94 inches) for the operating level earthquake and 30 centimeters (12 inches) for the contingency level earthquake. These deflection limits were based on past records of quay damage due to earthquakes, with 10 centimeters (4 inches) corresponding to minor easily repairable damage, and 30 centimeters (12 inches) corresponding to more significant damage which would result in a short term shutdown of operations, but no collapse. *Figure 3* shows displacement contours for the contingency level earthquake, which are well below the 30 centimeters (12 inches) limit. This is based on vibrocompacting the hydraulic fill to achieve a minimum relative density of 70%. Thus by carrying out a dynamic analysis using the finite difference method of analysis, cost savings were achieved in both the lengths and section properties of the sheet pile walls.

Once the design profiles and lengths were selected, there were still concerns as to whether the king piles and sheet piles could be driven to the design tip elevations, due to the pres-

Figure 4: Sheet pile installation along retaining structure



SETTLEMENTS BELOW UPPER LAGOONAL CLAY DURING DESIGN LIFE (BETWEEN YEARS 1 AND 60)

Figure 5: Settlement contours in reclamation area

ence of the cemented sandstone layers. Therefore, during the design stage, a driving test was conducted. The test consisted of driving three king piles and two pairs of intermediate sheet piles at two locations. These two locations were chosen based on boring logs which indicated the greatest degree of cementation of the sandstone. All piles were driven with a Delmag D62 diesel pile driving hammer, which has a maximum rated energy of 22.8 tonne-meters (164,620 pound-feet), and the required tip elevations were readily achieved.

Figure 4 shows the king pile system installation along the Retaining Structure.

Crane Supports

Bored piles were chosen to support the cranes because they are the most economical type of piling system in Israel. The main concern with the piles was how deep they would have to be drilled in order to limit settlements to tolerable values. Therefore, a three dimensional settlement analysis was conducted of the entire reclaimed site. Settlements were calculated below the upper lower lagoonal clay, i.e., at the top of the upper kurkar layer (approximately Elevation -22 meters (72 feet)) and below the lower lagoonal clay (approximately Elevation – 50 meters (164 feet)). The settlements were calculated at the end of construction (one year), and sixty years after construction, which corresponds to the project design life. The results of the analyses were compared to permissible differential settlements along each crane rail and between waterside and landside crane rails. *Figure 5* shows settlement contours from year one to year sixty. Based on the results of the evaluation, it was determined that the differential settlements could be maintained within acceptable limits if the piles were founded at the top of the upper kurkar layer. Nevertheless, it is anticipated that there will be some differential settlement between the crane beams and adjacent pavement during the life of the project, which will require re-leveling of the pavement.

Construction Sequence

Several issues required consideration in establishing the construction sequence:

- The maximum differential earth pressure that can be resisted by the sheet piling while acting as a cantilever, i.e., prior to installation and stressing of the tie rods: Calculations were performed and the maximum differential earth pressure was determined to be 8 meters (26 feet) for the main sheet pile wall at Quay 2, as governed by a maximum deflection limit of 5 centimeters (2 inches). Lower maximum

Figure 6: Overall view of sheet piling and reclamation during construction

differential pressures were calculated for other walls, and all limiting differential earth pressures were identified on the Drawings.

- The maximum wave loading that can be resisted by the sheet piling acting as a cantilever and prior to fill placement: Calculations were performed and the maximum wave height was calculated to be approximately 1.3 meters (4.3 feet) for the main walls. The Contractor was required to perform his own independent calculations and to temporarily brace the sheet piling if wave heights were anticipated to approach the limiting values.
- The potential for hydraulic fill to enter a nearby power plant intake if unconfined: In order to address this concern, it was stipulated that the minimum distance between the top of pile in the reclamation area and the northernmost extent of the sheet piling was to be 80 meters (260 feet). This was based on an assumed above water slope of 1:4, and an assumed below water slope of 1:20 in the wave affected zone and a 1:10 slope below the wave affected zone for the fill.
- The requirement to stabilize the existing shoreline south of the berth prior to dredging: This required the establishment of a detailed construction sequence in this area, which included pre-excavation, driving of sheet piling along the shoreline, dredging, placement of a rock dike out board of the sheet piling, and backfill behind the sheet piling.

A detailed construction sequence for the quay construction was also established, and included the following steps:

1) Dredge silt/mud from sea bed
2) Install main sheet pile wall, starting from shoreline and/or existing rubble mound wall
3) Dredge and place hydraulic fill and fill from other sources to no more than the maximum heights indicated on the drawings, so as to avoid overstressing the walls
4) Install anchor sheet pile wall
5) Install tie rods on temporary supports and initially tighten nuts so that they are snug tight
6) Hydraulically fill to Elevation 0.00, including removal of temporary supports and casting of concrete facing
7) Tension tie rods
8) Fill to Elevation $^{+}$0.5 meters (1.64 feet)
9) Install temporary working platform and perform vibrocompaction or vibroflotation with stone columns on the fill
10) Final tensioning of tie rods
11) Construct temporary platform to Elevation $^{+}$2.5 meters (8.2 feet) for bored concrete pile drilling
12) Construct bored concrete piles and cast pile caps and struts
13) Install bollard anchor rods and tension anchor rods
14) Cast fascia beam and concrete crane rail beams: backfill and grade to bottom of pavement sub-base elevation, and install pavement

The low bid for the project was significantly below the estimated cost. The project is currently in construction, with work proceeding according to schedule. *Figure 6* provides an overall view of the construction with Quay 2 and the Retaining Structure main sheet pile walls almost complete.

In summary, the Haifa Port Expansion project was carried out by identifying the key issues early on. Then, through a combination of field investigations and rigorous analyses, cost effective solutions were developed to address each major issue which ultimately resulted in significant cost savings.▪

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