

A Case History of Microtunneling through a Very Soft Soil Condition

the Contractor's Perspective

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Abstract- Waikiki Public Bath Force Main Replacement Project consists of 1,037 m of 400 mm force main that carries the wastewater from the Public Bath pump station to a gravity sewer on the Kuhio Avenue. This force main transports wastewater underneath Kalakaua Ave, very close to some of the most expensive real state in the world. This area is the heart of Waikiki beach, and it is full of beach resorts, five star hotels, shopping, etc. The soil strata can be described as 2.50 to 3.00 m deep of beach sand on top of half a meter of a very hard coral ledge on top of a lagoon deposit layer (very soft gray fat clay). The N_{SPT} values in this lagoon deposit layer range from zero to eight. The specified construction method for this project was microtunneling of 700 mm of Permalok steel casing with a 400 mm PVC carrier pipe inside the casing. The annular space between the two pipes was grouted with low density grout. Many problems were encountered during the construction phase of this project. This paper is a retrospective review of the project from the contractor's point of view. It covers the design and construction aspects of this project in addition to the encountered problems and the lessons learned from that project.

Keywords- Microtunneling; Soft soils, Grouting

I. INTRODUCTION

In December 1993, the City and County of Honolulu, department of Wastewater management (DWWM) decided to increase the capacity of the Public Baths Wastewater Pump Station (PBWPS) and forcemain to handle existing and future flow. PBWPS is located close to the War Memorial Natatorium at the south end of Waikiki Beach on Oahu, Hawaii. In 1996, the existing forcemain was 300 mm, and it needed to be replaced with a new 400 mm forcemain (Limtiaco1996). The 400 mm forcemain, which is the focus of this paper, was planned to transfer the flow from the pump station under Kalakaua Avenue for about 808 m and turn under Ohua Avenue for another 229 m to discharge the flow into the gravity sewer on Kuhu Avenue as shown in Figure 1. The scope of the project consisted of 1,037 m of 700 mm diameter Permalok steel casing, 400 mm diameter carrier pipe, tie-ins, and ancillary structures. Approximately 60% of the force main was laid under a highly congested commercial district where there are many businesses and five star hotels, and the other 40% was laid under very rare and historical trees in an environmentally sensitive recreation area. The project was also located along the very famous and congested beaches of Waikiki where disturbance to traffic, parks, beaches, and tourists must be avoided or kept to the minimum.

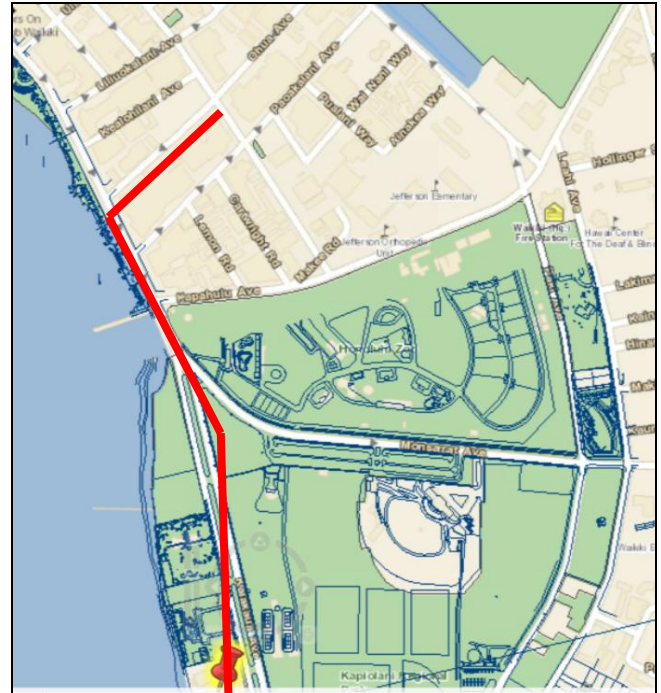


Figure 1 The project layout

Underground utility construction in Hawaii tended to get expensive and risky because of the many grades and degradations of rock and coral reefs as well as a high water table ^[1]. The geotechnical report can be summarized as 1.25 to 2.50m of sand beach layer overlaying a half meter-thick layer of coral ledge (cemented alluvial soil with coral formation), overlaying 6.00 to 9.00 m of a lagoon deposit layer (very soft gray fat clay mixed with loose coralline clayey gravel). The soft clay had low Standard Penetration Test - N_{SPT} values (zero to eight) and high water content. Dewatering on the site was allowed only for minimal and limited excavations because of potential settlement of the lagoon deposit layer and its potential effect on nearby structures. Additionally, the underground water was polluted in a few locations along the line necessitating biological treatment before discharging the water. This treatment would substantially increase the cost of dewatering. The force main could not be installed at a shallow depth because the area along the line was congested with existing utilities. All the above-mentioned reasons guided both the Owner and the Engineer to design it as a forcemain instead of a gravity sewer and to specify

microtunneling as the method of installation for this forcemain. Microtunneling is a trenchless technique that allows the installation of underground pipelines with adequate accuracy for gravity sewers at shallow depths, without the need of excavating trenches ^{[2] [3] [4] [5]}.

The Owner of the project was the Department of Wastewater Management for the City and County of Honolulu (DWWM) who hired Calvin Kim and Associates Inc. (CK) as the Engineer, and Obrien Kreitzberg (OK) as the Construction Manager. The contract was awarded to Delta Construction Corporation (DCC) who leased the microtunneling machine and its operator from Soltau Microtunneling Inc.

The original contract documents specified the invert of the force main to be 1.50 m deep. At this elevation the microtunneling boring machine (MTBM) would pass through the beach sand layer and the cemented hard coral ledge underneath it. During the submittal preparation phase of the project, DWWM decided to lower the invert of the forcemain about 1.10 m to the depth of 2.60 m between station 4+97 and station 8+86 because the existence of an unknown buried abandoned bridge and culvert in the pathway of the pipe alignment was discovered. Due to lowering the force main, the full face of the microtunneling machine was poisoned in the very soft lagoon deposit layer below the coral ledge layer.

II. ENCOUNTERED PROBLEMS

The jacking and receiving shafts were 4.27 m in diameter and were constructed prior to tunneling work. The selected method of construction for these shafts was sinking cast-in-place concrete caissons because of the soft soil conditions and the high cost of dewatering, filtering, and disposing of the underground water. The first three microtunneling drives were completed in the sand beach layer without major problem. After tunneling about 15 m in the fourth drive from Station 4+97 to Station 5+74 (crossing Kalakaua and Monsarrat Avenues towards the edge of the Kapiolani Park and the Honolulu Zoo) through the lagoon deposit layer, the tunneling operation was halted due to the following complications:

- The MTBM was sinking down about 25 mm in every 3.0 m pipe joint (slope = 0.83%) while the target slope was 0.00%.
- The 3 m pipe joint was pushed through the ground in 4 minutes without excavating any material and without any increase in jacking pressure.

The contractor (DCC) realized the impossibility of tunneling—with the required accuracy of the line and grade—through this very soft soil condition and promptly notified the owner-DWWM- and the Construction Manager (OK) of the situation in a meeting that took place on 3/12/98. DCC was instructed to conduct additional soil borings, to research the reasons of the deviation, and to find potential solutions with their approximate cost estimates.

The first step in the research was conducting more soil exploration boreholes close to the MTBM and at 15 m and 30 m ahead of the MTBM to verify and determine the soil conditions at the face of excavation. The additional soil borings (conducted by Pacific Geotechnical Engineers, Inc.) showed that the Standard Penetration Test— N_{SPT} at and under the pipe invert level ranged between zero and two. The geotechnical lab also conducted a battery of tests such as grain analysis, density, moisture content, etc. The second step was studying the records of the microtunneling machine during the previous runs.

In the beginning, it was thought that the problem was insufficient soil bearing capacity to support the head. However, the bearing capacity analysis proved the soil had enough bearing capacity to support the static MTBM. Analysis of the tunneling in this problematic 15 m section in the fourth drive showed that the head sunk faster when the cutting head was rotated and the slurry systems were operated (the normal operation of the microtunneling process). The very soft and submerged gray fat clay was semi-liquefied and, therefore, failed to support the front portion of the MTBM. The weight of the MTBM was more concentrated at the front where the articulated head which steers the MTBM was located.

Due to these challenging conditions, three brainstorming meetings took place to diagnose the problem and find solutions. The considered basic solutions to mitigate the liquefaction potential and improve the bearing capacity of the soil were: machine modifications or ground condition improvements. Minor machine modifications are usually less costly than ground improvement, but there is a higher potential for problems. Generally, the ground improvement techniques were slurry grouting, chemical (permeation) grouting, compaction (displacement) grouting, jet grouting, and fracture grouting ^[5]. The following paragraphs present the summary of six suggested solutions with their approximate cost estimates and probabilities of success to aid in the cost-benefit-risk analysis of the decision making process.

A. Arched barrel, hood, or plates (as shown in Figure 2) would increase the bearing area of the MTBM and redistribute the concentrated load at the front of the head. This solution was eliminated because the cutters would not have access to tunnel through the potentially harder material along the drive and through the receiving shaft wall. If the barrel, hood, or plate could extend and retract upon demand, this solution might have been a better solution.

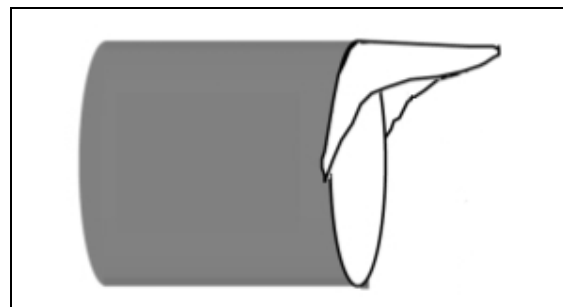


Figure 2 The barrel, hood, or plates ^[6]

B. Wings on the sides of the head to provide additional bearing area to distribute the weight of the heavier frontend of the MTBM. The wing solution involved the following actions:

1. Welding two plates along the sides of the articulated head as shown in Figure 3 to reduce the stress on the soil. The front edges of the plates would be serrated so it can cut through the coral if encountered.

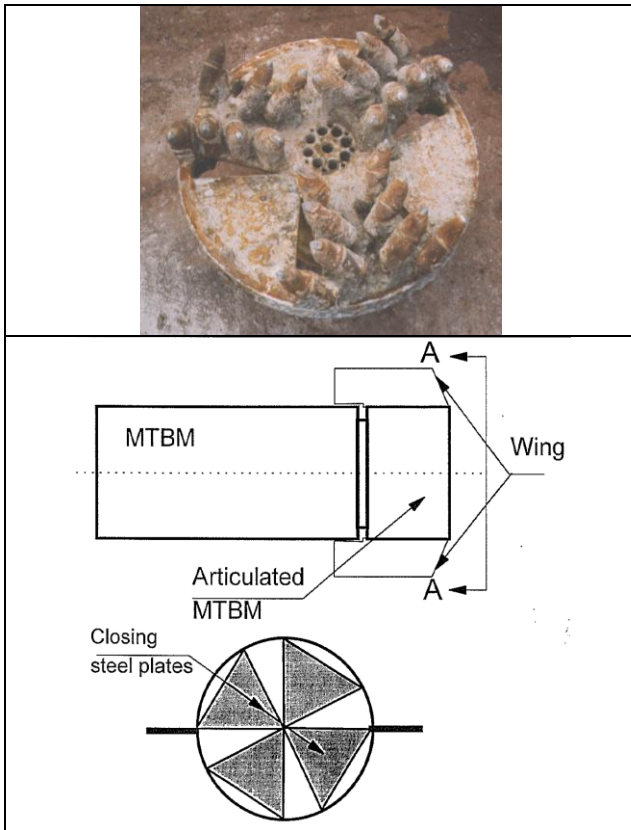


Figure 3 Wings modifications and choking the entrance of the MTBM

2. Choking the entrance to the crushing chamber as shown in Figure 3 and stop running the tunneling system.

3. Orienting the MTBM slightly upward to compensate for potential settlement

The estimated cost for the MTBM rework amounted to approximately \$8,500. There were other related miscellaneous cost (rubber ring, larger exit and entry rings, dewatering, etc.) making the total cost about \$25,000. In addition, the estimated cost for the machine retrieval amounted to about \$35,000 excluding the cost of retrieving the head in case of failure. This solution could be executed by the contractor without the need for subcontract from outside Oahu, which would reduce the delay in completing the project. This solution had never been tried before. There was also the potential risk of encountering coral along the path of the wings.

C. Compaction grout saddles would involve building a 1.50 m x 1.50 m x 1.50 m grout cube every 8 m on center. Compaction grouting is the injection of a very stiff and low slump grout under relatively high pressure to displace and compact soils in place. The grout usually

consists of a mixture of silty sand, Portland cement, and water sufficient to achieve a slump less than 75 mm. When the grout is injected into granular soils, bulbs of grout amass, displace and thus densify the surrounding loose soils ^[5].

The microtunneling machine would tunnel through the upper meter of the saddle as shown in Figure 4. It would go from cube to cube without falling substantially off grade. As the MTBM tunnels through the grout block, the microtunneling machine operator would steer the head against the harder body underneath it and correct the grade. The quoted cost from the grouting subcontractor was \$100,000; the total cost was estimated to be \$125,000 after adding the cost of the machine retrieval.

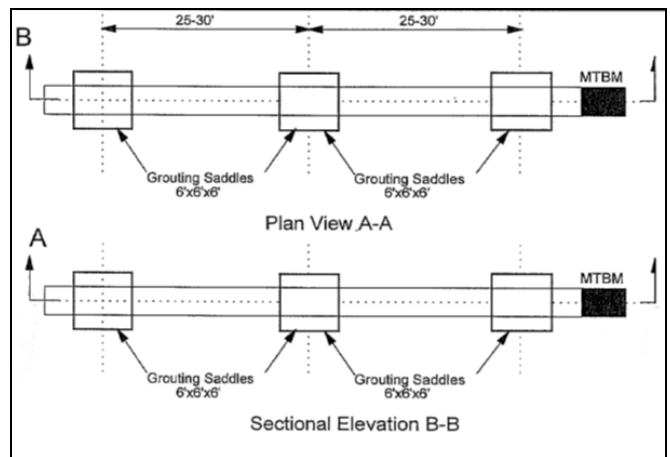


Figure 4 Compaction Grout Saddles Solution

The grout would be injected as a homogeneous mass with a distinct interface between the grout and soil. It would move into the weakest zones creating an irregularly shaped matrix of soil. Advantages were minimum site disturbance and risk, flexibility of scope, economy, applicability where the groundwater surface is high, and ability to lift settled structures to proper grade ^[7]. Compaction grouting is a proven method of stabilizing fine grain soils, but it has experienced a mixed history; technical success and cost control remain difficult to predict. It depends mainly on the permeability of the ground requiring stabilization ^[8]. The associated risks with this solution were the settlement and stability of the block, inconsistency of the block material, location, dimension, and permeability. It was uncertain if the 1.5 m length to the grouted block would be sufficient to correct the grade. Additional disadvantages were: limited number of suppliers, a long waiting time to procure a contract and to transport the required equipment to the Oahu, the risk of over spilling on the streets and sidewalks, which was not acceptable in Waikiki, and the experimental nature of the compaction grout in that microtunneling application.

D. Compaction grout pillars solution would involve grouting 750 to 900 mm diameter pillars extending upward from the harder layer at the bottom up to one foot below the invert of the casing pipe as shown in Figure 5. The pillars would support grouted cubes similar to the grout cubes of the previously mentioned solution. The pillars and the cubes would be constructed using compaction grout. The spacing

of the pillars would be 7.5 to 10 m. The quoted cost from the grouting subcontractor was \$140,000. This solution provides more stability to the saddle by carrying the weight of the saddle and the head to a more stable soil through the pillars. The disadvantages were inconsistency of the block material, inaccuracy of the block location, dimension, and permeability, and sufficiency of the 1.5 m length to correct the grade. This solution also has never been tried before.

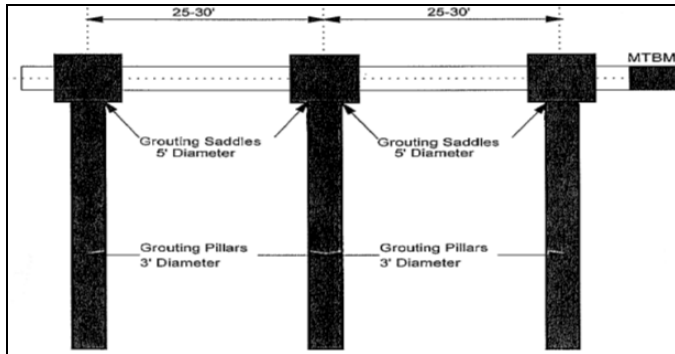
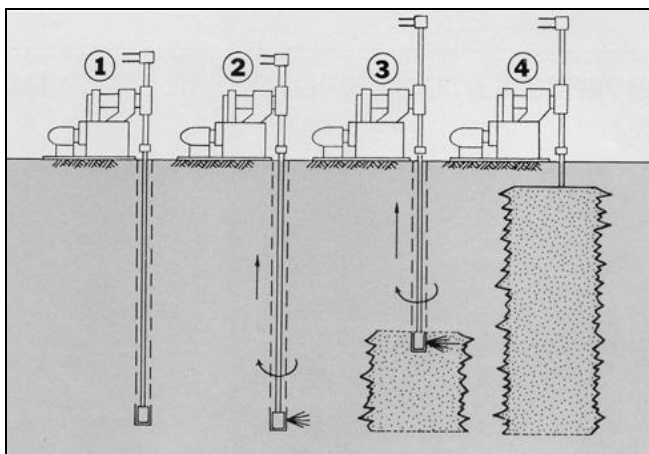


Figure 5 Compaction grout pillars solution

E. Continuous compaction grout block would create a continuous grout block approximately 1.50 m x 1.50 m wide for 300 m. The quoted cost from the compaction grout subcontractor was \$300,000. This solution would have a higher chance of success than the previous one, but at a higher cost.

F. Jet grouting pillar supports would create a total support for the pipeline by grouting pillars to the stronger coral ledge layer that lays 6.00 to 9.00 m below the pipeline invert. The pillars would support the saddles similar to those of compaction grout saddles in solution D or continuous block similar that of solution E. The jet-grouting-subcontractor's quotation was about a \$1,000,000.

In jet grouting, the soil structure is destroyed using high energy erosive jets while simultaneously mixing grout with the disturbed soil particles in situ^[8]. The jet grouting technique begins by drilling a hole; then, water jet, and in some cases an air jet, is activated. The drill rod, with the jet at the end, is rotated and withdrawn upward at a controlled rate pumping cement grout through the end of the drill stem. This process creates a roughly cylindrical column of mixed soil and cement as shown in Figure 6^[9].



JGP work sequence:	
1	Drill down to required depth;
2	High pressure cement milk emerges from the nozzle;
3	As drilling rod rotates and withdrawn, the cement milk mixes the soil
4	Cement ground column is formed

Figure 6 Jet Grouting procedures^[10]

Jet grouting was used successfully in a nearby microtunneling project on the Nimitz highway to provide permanent support for 915 m of 135mm RC pipe in lagoon deposits similar to one at Waikiki^[9]. This was the specified method for the second phase of the Nimitz highway sewer project which had similar conditions. This \$21-million project required the reconstruction of 2440 m of 900 mm diameter trunk sewer to replace aging, corroded, and sagging lines in downtown Honolulu under extremely complex conditions. The major challenge was unstable soil conditions which could have sunk the MTBM. The sewer line in these areas was supported by jet grout columns 24.50 m long. The project received the Grand Award from the American Consulting Engineers Council for overcoming this challenge along with other challenges^[11]. Jet grouting was also used successfully to improve soil condition in the horizontal direction in many tunneling and microtunneling projects; among them is the tunnel for the extension of the Metropolitan Atlanta Rapid Transit Authority (MARTA) under I-285^[12].

This solution was the most reliable solution; it had successful history in similar conditions. The disadvantages of this solution were high cost, limited number of suppliers, and a long waiting time to procure a contract and to transport the required equipment to Oahu. In addition, there was a risk of over spilling on the streets and sidewalks, which was not acceptable in Waikiki.

All the grouting solutions required a pilot test by the grout subcontractor to reach a workable combination of volume, pressure, cement-water-filling ratio, etc. All of the previously estimated costs for these grouting solutions did not include the cost of retrieving the microtunneling machine (about \$35,000) and the cost of ground surface restoration. The risk-benefit-cost analysis supported trying the machine modification by welding wings to the side of the MTBM. This drive was successfully completed using the modified machine.

III. FURTHER PROBLEMS IN THE NEXT DRIVE

The next drive (from the intersection of Paoakalani Avenue and Kalakaua Avenue towards Kapahulu Avenue) was one of the most critical drives because of the impact on hotels and businesses. After about 26 m in that drive, the MTBM encountered an electromechanical problem. After a week of investigation and consultation with the manufacturer in Germany, the problem was diagnosed as one or more of the connecting bins of the communication cord from the MTBM to the control panel were contaminated with grease. A worker crawled inside the pipe to the back end of the MTBM and cleaned the bins in the

connector cable. The machine resumed its normal operation for another 60 m where the MTBM started deviating to the left side of the target. Shortly after that the operator was not able to see the target on the target plate at the back end of the machine. Digging a rescue shaft in that location would have caused significant disturbance to the hotels and businesses in this area; therefore, every option to continue without a rescue shaft had to be fully explored. It was decided to continue drilling until we reached the shaft by steering the machine in the opposite direction. However, the MTBM reached the shaft with 4.75 m deviation from the centerline of the shaft.

After retrieving and checking the MTBM to ensure that everything was working properly, the MTBM was launched for the next drive (from the intersection of Paoakalani Avenue and Kalakaua Avenue towards Ohua Avenue). After about 20 m, the MTBM deviated significantly downward. At this point, construction was halted for more than a year to find alternative solution.

During that year, the contractor, engineer, construction manager, and the owner decided to abandon microtunneling as the installation method and install the pipe for the rest of the project using horizontal directional drilling (HDD) without any significant problems and with minimum impact on the businesses, hotels, and traffic as shown in Figure 8.



Figure 7 Retrieving the MTBM



Figure 8 Minimum impact of HDD operations on the businesses, hotels, and traffic

After more than 13 years beyond the conclusion of these events, reflections on the learned lessons from this experience and sharing them with industry may be beneficial. Some of these lessons, from the author's point of view, are presented in the following paragraphs.

The selection of the method of installation was critical in difficult soil conditions. The owner, construction manager, and the design engineer selected microtunneling as the method of construction to reduce the risk and increase the chances of success in this difficult soil and business/touristy environment. The previous microtunneling work in Oahu (Nimitz highway sewer project) involved a significant and messy amount of jet grouting work to enhance the soil conditions as mentioned earlier in the paper. However, grouting on Kalakaua Avenue would have had a significant negative impact on the tourist business.

Despite the many advantages that microtunneling offers, significant difficulties can be encountered when advancing in soils of a glacial origin especially when loose blocks are mixed within a clayey matrix with poor geotechnical characteristics. Also problems can be encountered in poor soils of a sandy-silty nature; the vibrations produced by the machine head during the excavation provoke a deterioration of the geotechnical characteristics of the soil inducing the machine to sink due to its weight and, therefore, cause deviation from the advancement direction (Ringen, 1998).

However, many projects have been successfully completed in poor soils with $N_{SPT} < 5$. Orestea, et al. (2002) found that ground with natural elastic modulus greater than 80 MPa would induce an allowable amount of MTBM settlements. For values lower than 80 MPa, it would instead be necessary to intervene either through integral structural works or through ground reinforcement. By adding lateral wings between 100 and 300 mm wide to the MTBM, it is possible to reduce the settlement to the tolerable level in soils having an elastic modulus above 20 MPa. In grounds with an elastic modulus lower than 20 MPa, it is necessary to intervene with preliminary reinforcement of the ground along the line of the micro-tunnel that has to be installed ^[6]

The experience of the contractor, engineer, construction manager, and the owner in the employed technology is crucial to the success of the project. The microtunneling experience of the above cited team at that time was limited compared to these difficult environments. The contractor and the Engineer hired different microtunneling consultants to make up for the shortage of experience. Later on, these members successfully completed microtunneling and trenchless projects in Oahu.

Both HDD and pipe bursting would have been less risky and less costly than microtunneling in dealing with these challenging conditions. The line was a force main; therefore, HDD, which can deliver pipeline within a few inches of accuracy, would have been sufficient. The cost of the project using HDD method would have been less than a third of the cost of the project using the microtunneling method ^[13]. Another alternative was bursting the existing 300 mm force main and replacing it with a 400 mm pipe. The cost of the project using this solution would have been

close to that of HDD ^[14]. The challenge with this solution would have been bypassing the flow from the existing forcemain.

Cooperation between the involved parties was critical for finding solutions to this problem. Willingness and commitment (from the project partners) to find and implement the optimum solution in terms of cost, risk, and benefits was crucial for making decisions and taking necessary risks. The participation in a partnering program assisted all the parties to be part of the solution.

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