BEFORE THE
MARYLAND DEPARTMENT OF TRANSPORTATION
BOARD OF CONTRACT APPEALS

Appeal of FRUIN-COLNON CORPORATION and HORN CONSTRUCTION CO., INC.	))	MDOT NO. 1002
Under MTA Contract No. NW-03-02	))	

December 5, 1980

Constructive Change -- The Mass Transit Administration's rejection of the contractor's support of excavation design and subsequent directive to correct it did not constitute a constructive change where it reasonably appeared that the design submittal failed to conform to minimum standards of good engineering practice.

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OPINION BY CHAIRMAN BAKER

This dispute concerns the sufficiency of a support of excavation plan designed by Appellant in accordance with certain contractually specified criteria. Appellant alleges that its design submittal was improperly rejected by the Engineer and that it was further directed to revise its support of excavation system at additional cost. This timely appeal has been submitted for decision solely on the issue of entitlement and the parties have stipulated the amount in issue at $79,435.56 should Appellant be successful on the merits of its appeal.

I. FINDINGS OF FACT

A. Introduction

On May 20, 1976 the Mass Transit Administration (MTA) issued a notice to contractors inviting bids on the construction of approximately 11,300 linear feet of single track earth tunnel, with associated cross passages, temporary bulkheads, two mid-line vent structures, and two access shafts. Bids were opened on July 27, 1976 and the lowest responsive bidder was identified as the joint venture of Fruin-Colnon Corporation and

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Horn Construction Co., Inc., the Appellant herein. On September 16, 1976, MTA Contract No. NW-03-02 was awarded to Appellant in the amount of $41,658,000, subject to approval by the Maryland Board of Public Works which was granted on November 12, 1976.

This contract was one of a series of contracts for construction of the northwest line of the Baltimore Region Rapid Transit System. It is anticipated that the tunnel line work which Appellant was to perform will connect the Bolton Hill Station structure with the Laurens Street Station to the north and the Lexington Market Station to the south. The subway station structures are not part of the contract.

Appellant's tunneling work was to commence at the north and south limits of the Bolton Hill Station contract. Prior to initiating this work, the contract mandated the excavation of two access shafts within the Bolton Hill Station contract right-of-way, one at each interface with the north and south tunnels. These structures, denominated the north and south access shafts, were to be designed, excavated, and supported by Appellant prior to the commencement of excavation for the adjacent Bolton Hill Station by another contractor.[1] Consistent with this work sequence, the contract Technical Provisions set forth the following pertinent performance requirements:

A. North Access Shaft: Design and construct the excavation end supports in such a manner that the south end support may be removed by the adjacent contractor without affecting the stability of the remaining three sides.

B. South Access Shaft: Design and construct the end excavation supports in such a manner that the north end support may be removed by the adjacent contractor without affecting the stability of the remaining three sides.[2]

In performing this contractual duty, Appellant was required to utilize certain design criteria shown on Contract Drawings S93-5 (Sheet No. 169) and S94-3 (Sheet No. 170) pertaining to, among other things, lateral soil pressures, surcharge and live loads. Appellant's design was also required to be submitted to the MTA Engineer for review and approval prior to the performance of any work and was to include "...complete computations, work sequence and working drawings."

B. <u>Initial Design of the Access Shaft Support System</u>

Appellant retained Mr. Ulf H. Werner, a consulting engineer, to design its support of excavation system for the two access shafts. The initial designs for the south and north access shafts were submitted on October 27, 1976 and November 14, 1976 respectively, and both were approved by the Engineer in December 1976. These initial submittals were not represented or shown as a complete design of the support of

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[1]Section 3.0 of the Contract Special Provisions apprised Appellant that the Bolton Hill Station structural contract was to be awarded on or about February 1, 1977. It ultimately was awarded to Peter Kiewit Sons Co.

[2]Tech. Prov. Division 2 - Site Work, Section 2.21 - Support of Excavation, Article 3.8 - Support of Excavation at Access Shafts.

excavation system but pertained only to the soldier piles.[3] However, Appellant did indicate in its initial submittals that it planned to use "thrust piles"[4] to serve as vertical footings to retain the unbalanced load which would be generated by the Bolton Hill Station excavation. Appellant further stated that its design analysis for the thrust piles, due to its complex nature, would be transmitted to the Engineer in a separate volume of computations. During the next eleven months, Appellant continued to submit various portions of its access shaft support design and proceeded to construct both shafts as four sided figures (i.e., having earth loads and structural support on all four sides). It was not until November 28, 1977, however, that Appellant's thrust pile analysis was first submitted to the Engineer for review.

On January 19, 1978, Appellant's thrust pile design was rejected based upon a review by Respondent's general engineering consultant, Daniel, Mann, Johnson & Mendenhall/Kaiser Engineers (DMJM/KE).[5] This rejection was due to concern over the proper coefficient to be used in computing the passive resistance of the soil behind the thrust piles. A resubmittal was required which would lower this coefficient in all design computations. Thereafter this issue was not a source of difficulty between the parties and does not enter into subsequent consideration of this matter.

Notwithstanding the role of DMJM/KE as primary structural reviewer, Respondent's general construction consultant, the Ralph M. Parsons Co. (RMP), also became involved in the review of the thrust pile design. Review responsibility was assigned by RMP to its Technical Services Group whose conclusions were discussed during an in-house meeting on January 26, 1978. It was observed at this meeting that the excavation progress by the Bolton Hill Station contractor in the vicinity of the south access shaft mandated expeditious resolution of the thrust pile design and thus it was recommended that the Resident Engineer[6] arrange a meeting between the MTA consultants and the contractor's design consultants to discuss the thrust pile submittals. The Resident Engineer scheduled this meeting with Appellant for February 17, 1978. In the interim, DMJM/KE engineers continued their analysis of Appellant's thrust pile

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[3] A soldier pile is a wide flanged beam having an H or an I shape. This structural member has great strength in a direction perpendicular to its flanges and is much weaker when loads are applied parallel to the flanges. (Tr 78) The soldier pile is placed vertically into the ground so that the earth loads on one side and the strut loads on the other side are acting perpendicular to the flanges.

[4] A thrust pile is a vertical footing that bears against the soil. Loads are applied against these piles with the orientation of the thrust load being horizontal and the structure that is resisting (i.e., the piles) being vertical. (Tr 12)

[5] Commencing in November 1977, specific responsibility for approval of working drawings and design computations had shifted from DMJM/KE to the MTA Construction Manager, the Ralph M. Parsons Co. However because the access shaft support design had been tendered piecemeal during the course of the previous year and was then within the technical cognizance of DMJM/KE, the latter retained design review responsibility for the thrust pile submittal.

[6] The Resident Engineer was an employee of RMP and generally had responsibility for the administration and supervision of the contract.

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design and on February 13, 1978 met with Dr. Ralph Peck,[7] DMJM/KE's geotechnical consultant. While no computations were made by Dr. Peck, he believed the thrust piles were adequately sized to receive the unbalanced bulkhead load.[8] However, Dr. Peck cautioned that the major portion of this unbalanced load would more likely be absorbed by the resistance of the soil against the sidewall soldier piles and thus would not be fully transmitted to the thrust piles. This opinion was conveyed to Appellant at the February 17th meeting and a restudy of the load transfer was requested by the Engineer. Appellant, on February 20, 1978, restated its conclusion that the major portion of the unbalanced load (61%) would be transmitted to the thrust piles and that no changes to its design were indicated.

C. Access Shaft Structure

Each access shaft was rectangular in shape and outlined by soldier piles located on approximate eight foot centers. As the shafts were excavated, lagging boards were placed between the soldier piles, against their front flanges, to prohibit the slippage of soil into the excavation. In order further to balance the soil loads behind the piles and lagging, internal bracing members were provided at three levels within the north shaft and two at the south shaft. The bracing consisted of "wales"[9] which were connected to the soldier piles and ran along the perimeter of the access shaft, and "struts"[10] which spanned the length and width of the excavation. The wales were connected to the soldier piles by welded steel angles denominated as packing.

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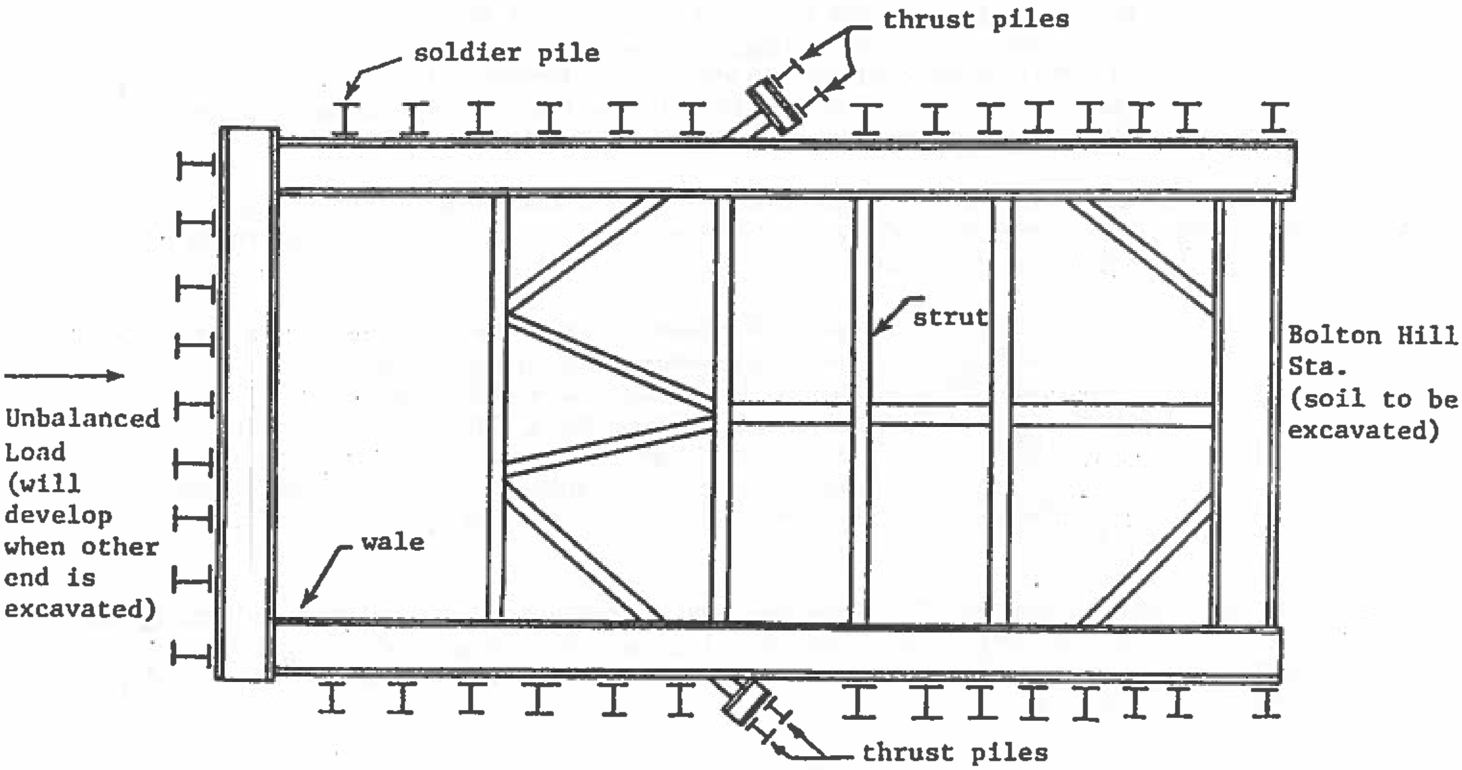
[7] Dr. Ralph Peck is a retired Professor of Civil Engineering and author of two textbooks dealing with soils and foundation engineering. He is presently a consultant and recognized authority in soil mechanics and geotechnics.

[8] In a subsurface structure with earth loads on each side, the support system balances the loads on one side against those on the other to prevent a cave-in. When the earth loading on one side of the structure is removed, the loading on the opposite side becomes unbalanced.

[9] Wales distribute loads from soldier piles to struts. (Tr 116)

[10] Struts are compression members which are preloaded by the use of jacks. They resist the forces imparted on the soldier piles by soil loadings. (Tr 118)

The relationship of each support member at a typical bracing level is depicted below:



D. Development of Dispute

On February 27, 1978, DMJM/KE engineers completed their analysis of Appellant's design. Their study concluded that 73% of the unbalanced load would be absorbed by the reaction of the sidewall soldier piles against the soil, while the remaining 27% would be transmitted to the thrust piles. Further even if Appellant's estimation of load distribution was presumed correct, it was determined that the structural connections between the soldier piles and wales were inadequate to receive the end bulkhead load. These conclusions were communicated to the MTA Engineer, Mr. Frank Hoppe, by memorandum dated March 7, 1978. This memorandum further recommended that Appellant's February 20, 1978 submittal be rejected and that tension tie members[11] be provided to reinforce the structural connections. The MTA concurred in these findings and, by letter dated March 13, 1978, the Resident Engineer rejected Appellant's thrust pile design stating:

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[11]Tension tie members are placed between the brackets supporting the wales and a soldier pile. In this case the recommended solution was to attach tension ties to adjacent soldier piles to resist lateral movement and thereby provide additional support. (Tr 563-64)

"...Your design indicated that 39% of the thrust loads will be carried by the soldier piles. We do not find evidence in your calculations to show that structurally 39% of the thrust can be transferred to the piles. Additionally, our current design review of the load distribution is indicating that more than 39% of the thrust loads must be supported by the soldier piles. On the basis of this, your design calculations are disapproved and you are hereby notified to revise and resubmit your design calculations for review and approval as required by the contract documents ..." (Rule 4, Tab N)

By letter dated March 16, 1978, Appellant's project manager submitted computations purporting to show that 39% of the end bulkhead load was capable of being transmitted to the side piles and stated that:

"...This assumption was made based on past experience of both of our consultants and is a common practice used in the design of end bulkheads. This assumption is that each side pile is capable of absorbing a portion of the end bulkhead load, this portion equal in magnitude to 10% of the normal load being transmitted into the pile. This analogy, when applied to all of the side pile, results in 39% of the end bulkhead load being transmitted into the side pile ..." (Rule 4, Tab 0)

This explanation along with the accompanying calculations were analyzed in detail by the RMP Technical Services Group. Their conclusions were transmitted to Appellant by letter dated March 22, 1978, under the signature of Mr. J. W. Maddox, the Construction Manager, and are summarized as follows:

1. The unbalanced load will be transmitted to the sidewall soldier piles rather than the thrust piles because the former path is the more rigid of the two. Maximum resistance in shear will develop against the soldier piles with little movement in the system. However, passive earth pressure against the thrust piles will fully develop only after the entire system moves several inches. Since the support system is not expected to move several inches, the unbalanced load will be taken by the sidewall soldier piles and particularly by the structural connections thereto.

2. The sit piles because the former path is the more rigid of the two. Maximum resistance in shear will develop against the soldier piles with little movement in the system. However, passive earth pressure against the thrust piles will fully develop only after the entire system moves several inches. Since the support system is not expected to move several inches, the unbalanced load will be taken by the sidewall soldier piles and particularly by the structural connections thereto.

2. The sidewall soldier piles will be able to resist more than 10% of the strut loads because of the interlocking action with the adjoining soil mass. This will help assure that the unbalanced load is resisted by the soldier piles and concomitantly will require strengthening of the structural connections.

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3. Existing connections are not adequate to carry even the 39% of the unbalanced load assumed by Appellant to go to the soldier piles.

4. Calculations submitted by Appellant on March 16, 1978 make certain assumptions as to the welding of star packing[12] which were not confirmed by field inspection of actual installation. Tee packing[13] was also found to be inadequate.

On March 28, 1978, the parties met to review the MTA findings. Appellant was represented at this meeting by its principal designers, Mr. Werner and Mr. James Wilton,[14] the developer of the thrust pile concept. Mr. Wilton explained to the MTA engineers that the 10% loading assumption was developed in San Francisco during the construction of the Bay Area Rapid Transit Project and was reasonable. He further challenged the MTA conclusion that the support system would have to move several inches in order to develop passive resistance behind the thrust piles. No additional computations were submitted by Mr. Wilton who stressed that it was difficult to accurately determine by calculation the precise movement which might be expected. However, empirical data was said to be highly significant in analyzing thrust piles and, in this regard, Mr. Wilton pointed to his experience with thrust piles on a number of other projects. During his direct examination, Mr. Wilton testified that the Stadium Armory Station constructed for the Washington Metropolitan Area Transit Authority (WMATA) under Contract 1D0081 involved a support of excavation system similar to the north and south access shafts. (Tr 69) The thrust piles installed on this project neither failed nor experienced undue distress. (Tr 72) However, on cross-examination Mr. Wilton admitted that the WMATA excavation support was designed as a three sided figure and that the dimensions of the shaft and levels of bracing were different than those existing at Bolton Hill. Further the method of preloading the three sided structure clearly resulted in the thrust piles being preloaded whereas the degree of thrust pile preloading at Bolton Hill was not as certain.

Following the March 28th meeting, the RMP Technical Services Group prepared additional computations with respect to the adequacy of the access shaft support system. After reviewing these computations and giving full consideration to Mr. Wilton's comments, Mr. Gus Leonard, the Manager of the Technical Services Group, prepared a memorandum to the MTA which again recommended both the rejection of

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[12]Star packing is also known as double angle or cross shaped packing. It consists of two L shaped steel angles packed back to back forming a star or cross. These angles are welded to the wale on one end and the soldier pile on the other end. (Tr 180-181, Exh 8)

[13]Tee packing was used only at south shaft. Tee packing resembles a T shaped length of steel turned on its side. (Tr 194)

[14]Mr. James Wilton is the President of Jacobs Associates, an international consultant engineering firm specializing in heavy construction projects. Mr. Wilton developed the thrust pile concept in conjunction with design work performed for the Bay Area Rapid Transit project in 1965. It was developed to reduce the number of bracing levels and labor intensive work commonly associated with traditional support of excavation systems which used rakers and tie-backs.

Appellant's design and the taking of immediate remedial measures. Thereafter, on May 4, 1978, the Resident Engineer directed Appellant to redesign its support of excavation system so as to assure stability. Enclosed in this letter were two suggested schemes for strengthening the design; the second of which involved the addition of two steel plates to the star packing at both the north and south access shafts. This scheme was thereafter directed by the MTA and this work was initiated by Appellant on June 2, 1978 under a written notice of claim submitted on May 12, 1978.

E. Welding of Packing

Appellant's original working drawings, submitted with its support of excavation design, provided for welding along one leg of each angle comprising the star packing. The angles each were welded along their vertical leg against the soldier pile and the horizontal leg against the wale. In an effort to resolve the design problem without litigation, Appellant agreed, in early April 1978, to weld the star packing along its unconnected legs at both the soldier piles and the wales. This work was performed between April 5 and April 20, 1978 at no expense to the MTA. The RMP Technical Services Group advised the MTA however that even the additional welding was not sufficient to enable the packing to withstand the shearing forces created by the unbalanced load. Consequently, the directives discussed heretofore were issued.

Prior to the performance of additional welding by Appellant, RMP had determined by computation that only 18% of the unbalanced load safely could be absorbed by the packing connections. Additional computations prepared by Respondent for this appeal on January 25, 1980 indicate that the complete welding of the packing doubled its strength and permitted 36% of the unbalanced load to be absorbed safely by these connections. Respondent's Mr. Leonard further testified that this welding would have satisfied the MTA concerns had it been convinced that only 39% of the unbalanced load would go to the soldier piles. The MTA's conclusion that a substantially greater percentage of the load would be resisted by the soldier piles resulted in the further strengthening of the packing connections and brought on the instant claim.

F. Monitoring of Pile Movement

In late April 1978, RMP instituted a procedure for monitoring movement in the soldier piles at both the north and south access shafts. Measurements were recorded on a weekly basis between April 25 and July 5, 1978. The maximum pile movements recorded were within one-half (1/2) inch. Inclinometer readings were also taken to measure the movement of the soil behind the braced excavations. These readings also revealed average movements of one-half (1/2) inch. Concomitantly, Appellant began a daily inspection of the packing inspections at the north and south access shafts. These inspections did not reveal any areas of distress. Survey monitoring of the north shaft also was initiated by Appellant at this time. This was limited to the north shaft because, unlike the south access shaft, it had not as yet been unloaded. No movements were recorded through August 18, 1978 when the monitoring was terminated.

In interpreting and considering this data the Board observes that all measurements occurred after the star packing had been completely welded. Consequently, these measurements are not considered as probative with regard to the propriety of Appellant's initial support design. However, Appellant contends that they are relevant to the question of whether additional steel plates were necessary to reinforce the packing connections after the welding of the packing had been completed.

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In this regard the monitoring data and other evidence of record indicate that the south access shaft was stable prior to the placement of steel plates which commenced on June 2, 1978.

G. Soil Support for City Temple

By letter dated April 4, 1978, Dr. Ralph Peck transmitted his conclusions to DMJM/KE concerning the adequacy of Appellant's thrust pile design. This letter in pertinent part stated:

> "I do not believe that the total displacement of the bracing system will be excessive; probably it will be no more than a fraction of an inch. Nevertheless, the displacements are likely to accumulate near the protuberances [thrust piles]. They would tend to put tensile strains in the City Temple near the junction of the tower and the remainder of the building."

When interrogated concerning this statement at the hearing, Dr. Peck testified that potential movements in the soil behind the thrust pile exceeded comparable movements at the nearby soldier piles. This differential movement might further have exerted tensile stresses on a portion of the church which contained a large stained glass window. By strengthening the wale to soldier pile connection, Dr. Peck believed that movement in the entire system would be minimized and prospective damage to the City Temple thereby avoided. On cross-examination Dr. Peck was asked whether his conclusions and recommendations to the owner would have been different had the City Temple not been located in close proximity to the north access shaft and replied:

> "They might have. I can't say for sure <u>because I would have wanted in any case to make sure that the connections would be adequate to transfer whatever loads they would actually see</u>." (Tr 507) (Underscoring added.)

Dr. Peck did admit however that the presence of the City Temple was an important consideration in arriving at his opinion.

Upon the foregoing, Appellant contends that the directive to strengthen the packing connections resulted solely from an "overabundance of caution" due to inordinate sensitivity to the stained glass windows of the City Temple. However, the evidence before this Board, particularly the studies performed by DMJM/KE and the Technical Services Group of RMP, clearly establishes that the reason for rejection of Appellant's design was concern over the adequacy of the packing connections when more than 39% of the unbalanced load is resisted by the soldier piles and the Board so finds. Dr. Peck's testimony when considered in conjunction with his April 4, 1978 letter is not inconsistent with this finding.

DECISION

While Appellant had responsibility to design and install the support of excavation system for the access shafts, it was not free to implement this design prior to receiving approval of Respondent's Engineer. This approval was to be granted subject to review of the intended design as evidenced by Appellant's working drawings, calculations and other information sufficient to explain its support of excavation system. The dispositive issue in this appeal is whether Appellant sufficiently detailed its load distribution assumption so as reasonably to have satisfied Respondent's Engineer that its underlying basis conformed to minimum standards of good engineering practice.

Appellant's support of excavation design ultimately was rejected because of Respondent's concern that as much as 75% of the unbalanced bulkhead load would be transmitted to the sidewall soldier piles. Appellant, on the other hand, assumed that only 39% of the unbalanced load would be absorbed by these soldier piles with the remainder being transmitted to the thrust pile system. The bases for these disparate views is essential to the Board's decision.

Appellant's determination that 39% of the end bulkhead load would be absorbed by the soldier piles is based upon a structural analysis of the load transfer. This analysis assumes that with a continuous wale and welded packing, all piles will contribute some resistance to the end bulkhead load. The extent to which each pile provides lateral restraint was further assumed to measure 10% of the loads each receives from the struts. As testified by Mr. Werner, this assumption was based upon:

> "...our judgment from past projects. It's based on our knowledge of the available resistance surrounding a pile, our general feel for the relative weakness of a soldier pile in the -- in this transverse direction. It's a value judgment based on many years of similar type work." (Tr 170)

Without reviewing the mathematics involved in computing the degree of loading resisted by the sidewall piles, we observe that the 10% figure is essential to Appellant's computation of load distribution. An increase in the 10% figure assumed by Appellant would result in a greater portion of the end bulkhead load being resisted by the soldier piles. However, as testified by Mr. Wilton, a prudent designer would not assume that a soldier pile could resist more than 10% of its normal load in the lateral direction notwithstanding that a soldier pile, under ideal conditions, could undertake as much as 30% without failing. Mr. Wilton further explained that the lateral loads on a soldier pile are generally applied to a single flange rather than at the center of the web resulting in a doubling of the equivalent load actually being placed on the pile. (Tr 79) If it is assumed therefore that a pile will resist 10% of its normal load laterally, it may, in actuality, be required to resist an effective loading of 20%. Hence a design assumption that more than 10% of the normal loads will be resisted laterally by a pile may result in the failure of the pile along its weak axis under actual field loadings.

Respondent's rejection of Appellant's design was based upon a study of the effect of the surrounding soil on the latter's support of excavation system. Dr. Peck, Mr. Leonard and Mr. Ziegler[15] all expressed the opinion that the end bulkhead load would be transmitted along the stiffest path of resistance. Pursuant to this analysis, any movement in the support system would be resisted initially by the shearing forces of the soil acting against the sidewall soldier piles and packing connections. The thrust piles would not absorb any part of the unbalanced load until sufficient movement occurred in those piles to develop the passive earth pressure of the earth behind them. Further

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[15] Mr. Ziegler has been employed as a civil engineer for over 30 years. He has substantial experience in soil and foundation engineering and is presently the head of the geotechnical engineering department at Rummel, Klepper and Kahl, one of the firms comprising the RMP joint venture.

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because the shearing forces acting on the soldier piles would develop fully with less movement in the system than was necessary for the development of passive earth pressure behind the thrust piles, the stiffest path was said to be along the sidewalls of the access shafts at the soldier pile and packing connections.

Underlying Respondent's analysis is a fundamental principle of soil mechanics which, as testified by Dr. Peck, provides "...that shear displacements develop the ultimate strength and shear at smaller movements than the displacements required to develop bearing capacities." (Tr 492) Quantitatively, Dr. Peck testified that one would have to compress several feet of soil to develop resistance in bearing whereas only a fraction of an inch of soil need be sheared to develop the full resistance in shearing force.

Respondent's Dr. Peck and Mr. Ziegler expected a total movement of the support of excavation system of approximately 1/2" or less assuming that the packing connections were adequately strengthened to resist the shearing forces placed upon them. The total movement necessary to develop bearing pressure behind the thrust piles in sufficient quantity to resist 61% of the unbalanced bulkhead load however was estimated at two to three inches.

Appellant's Mr. Wilton did not dispute the theoretical correctness of Respondent's analysis. However, Mr. Wilton opined that Respondent's analysis improperly assumed that the soil between the soldier piles was capable of developing the shear forces predicted. In excavating the access shafts Mr. Wilton testified to the inevitability that the soil between the soldier piles would be disturbed by placement of the lagging boards. Given this circumstance, the development of shearing forces was subject to the following chain of events:

> "...the wale has to transfer the load to the soldier pile, the soldier pile has to transfer the load to the soil between the soldier piles. It can't transfer it to the lagging because the lagging isn't tight, so it has got to transfer it to the soil between the soldier piles and that soil has to transfer the shear to the undisturbed soil behind. Since that soil between the soldier pile is disturbed, or we perceive that it is, we therefore can not develop the kind of shears talked about here in my opinion at least we can't depend upon doing it and that is why we don't." (Wilton, Tr 96-97)

Mr. Wilton also took exception to Respondent's contention that the thrust piles would have to move several inches to develop passive earth resistance. Since the thrust piles are designed and installed so as not to disturb the soil behind the piles, and consequently given the competent nature of the soil involved, Mr. Wilton testified that thrust movements of approximately 1/2" or less could be expected.

Mr. Wilton's assessment of the shearing force development was not concurred in by Mr. Ziegler who concluded that any disturbance in the soil would be exclusively behind the lagging while the shearing forces however would develop at the back flange of the soldier pile where the soil is undisturbed. (Tr 637-38) Dr. Peck also disputed Mr. Wilton's assessment of thrust pile movement. The soil mass settlement that is required to develop passive resistance was said to be unrelated to the amount of disturbance behind the thrust piles. Dr. Peck explained that it is only when the soil mass has been compressed to the point of deformation that it develops its full passive resistance and hence even undisturbed soil must be stressed in this manner.

Although both parties have agreed there is no rigorous mathematical method of analyzing the expected movement of thrust piles, there are methods of predicting such movement using standard engineering formulas. Respondent's Mr. Ziegler testified that thrust pile movement may be estimated with reasonable accuracy. (Tr 668) Mr. Wilton, on the other hand, testified that while such movement may be estimated, it is the kind of computation that could produce varying conclusions due to the difficulty in measuring the soil parameters that affect a support of excavation system.

Appellant prepared no computations pertaining to movement. Both Mr. Werner and Mr. Wilton were satisfied that the competency of the soil would result in very little movement in the thrust piles. Further their experience on other projects using identical design assumptions and analyses bolstered their belief that Appellant's initial design was contractually adequate.

Respondent's computations however were not supportive to Mr. Wilton's experience and empirical assessment of the thrust pile system. Mr. Edward Ziegler, on behalf of Respondent, presented a detailed exhibit analyzing the movement necessary to develop both the full passive resistance of the soil and resistance equal to 61% of the unbalanced load. This analysis concluded that 5.5 inches of movement would be necessary to develop the full passive resistance of the soil behind the thrust piles at the north shaft and 4.1 inches at the south shaft. If the thrust piles were to receive only 61% of the end bulkhead load, only about two-thirds (2/3) of this movement would be necessary. Mr. Ziegler also testified concerning a shear box test[16] which was performed on a sample of soil taken at the Charles Center Station area. This soil was said to be typical of the material existing, at the Bolton Hill access shafts. Test results indicated that maximum shearing resistance would be activated at displacements ranging from 1/10 of an inch to 1/2 of an inch. The results when compared to the movement estimates prepared by Mr. Ziegler's staff confirmed the MTA's conclusion that most or all of the unbalanced load would be resisted by the shearing forces of the soil.

Appellant challenges this analysis primarily due to the usage of a particular measure of subgrade reaction in the formula for computing movement. This measure of subgrade reaction was obtained from the geotechnical data prepared by an MTA consultant for use by designers. The table relied upon appears in pertinent part as follows:

Material Characteristics For Design

Principal Strata Designation	Subgrade Reaction Tons/Cu. Ft.	
	Vertical one sq. ft.	Horizontal one sq. ft.
c-1, c-2a	300 (below water table)	34 (constant below water table) per foot of depth

[16]Shear box is a laboratory piece of equipment which is used to measure the direct shear and angle of internal friction of soil.

113

Respondent used the 34 tons/cf figure in computing movement. This was said to be the appropriate factor for such computations by Mr. Ziegler. Appellant, in its brief, argues that the 300 tons/cf figure is the proper subgrade reaction for use with a vertical footing such as a thrust pile. Further when this measure is inserted in Respondent's computation, it reduces the movement necessary to achieve maximum passive resistance by a factor of approximately nine ($^{300}/_{34}$). However, Appellant presented no credible testimony supporting the materiality of the 300 tons/cf factor.

The thrust pile concept is an innovative one and, as such, Respondent was unable to avail itself of published data or professional articles explaining or measuring thrust pile support characteristics in different type soils. Aggravating this difficulty was the absence of a rigorous mathematical formula or accepted method of analyzing thrust pile movement. On the other hand, Respondent did apply well established principles of soil mechanics and attempted to verify Appellant's design assumptions. Respondent's conclusion both from a theoretical and quantitative standpoint was that the majority of the end bulkhead load would be resisted by the soldier piles thereby causing the packing connections, as originally designed, to become distressed and possibly fail. This conclusion resulted in the rejection of Appellant's design and a subsequent directive to strengthen the packing connections.

In reviewing Appellant's design, the MTA was confronted with accepting the detailed soils analysis of its own experienced engineers and consultants or relying upon the verbal assurance and experience of Messrs. Wilton and Werner. While the Board recognizes the expertise of Appellant's consultants, their testimony and statements must be measured against the accepted engineering theory which underlies Respondent's analysis. The Board finds that while Appellant and its consultants challenged the validity of the MTA soils analysis, they failed to identify any conceptual fallacy or demonstrate with specificity how other reasonable soil parameters might be utilized to compute smaller movements in the thrust piles than those calculated by Respondent's engineers. The preponderance of evidence before this Board also establishes that soil shear forces would develop quickly along the outer flanges of the sidewall soldier pile where disturbance of the soil would not be present. Consequently, the Board finds that Respondent's engineers were contractually entitled to reject Appellant's design submittal as it did not reasonably appear that the primary portion of the bulkhead load would be resisted by the passive resistance of the soil behind the thrust piles using conventional theories of soils analysis.

The utilization of monitoring data to measure the movement of the support structure after completion of the access shafts is inconclusive. Not only is the probative value of this data obscured by the on-going reinforcement of the north access shaft packing during the period of measurement, but it consists of measurements which were unavailable to Respondent at the time it reviewed Appellant's initial design and issued its directive. Further, Appellant's contractual duty was to demonstrate the stability of the shafts prior to construction rather than after.

Finally, the Board is compelled to consider the issue of burden of proof which it requested each party to address. After careful review of the record as a whole, the Board finds that the preponderance of the evidence supports Respondent's contention that Appellant's design did not conform to minimum standards of good engineering practice. Under these circumstances the risk of non-persuasion need not be allocated or discussed further.

For the foregoing reasons, this appeal is, in all respects, denied.