EXPERT OR LITIGATION "HIT-PERSON"?

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Abstract

The proliferation of litigation has resulted in legal experts who frequently have little if any actual preconstruction design experience. These experts frequently become advocates for their clients, who may be either the legal or insurance community. This advocatory approach to testimony can be in violation of the Engineering Practice Act, Paragraph 137.57, (a) "Engineers shall issue statements only in an objective and truthful manner" if the testimony is slanted to reflect the client's position.

Three case studies are presented where the expert appears to have taken a contrarian opinion relative to common practice and the evidence. An attempt was made to present each case study without bias to allow the reader to arrive at their own conclusion.

Introduction

Litigation is a fact of business, and as long as design and construction involves people making decisions, it will be a part of the construction industry. Not every decision made during the design or construction phase can withstand the intense scrutiny of hindsight. However, not every decision deemed "wrong" in hindsight represents negligence.

Negligence is defined by "Legal-Explanations.com" as:

> "An act or misconduct also called malpractice where professionals like medical practitioners, lawyers, accountants, architects, etc., failed to exercise their duties effectively and which results in damages to clients. It can be due to negligence, ignorance or intentionally. It cannot be proved just by the patient's judgment unless it is very obvious but a legal declaration has to be made by an expert of the same profession that the professional failed to meet the basic standards while performing the act."

There will always be gray areas of professional judgment, and as such, gray areas where the bounds of negligence are not well defined. However, when an expert begins to excessively "shape" his or her opinion to fit the client's needs or desires, the integrity of the expert should be questioned.

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Paragraph 137.57, (a) of the Engineering Practice Act, states "Engineers shall issue statements only in an objective and truthful manner." Three case studies are presented in the following sections. The reader is left to decide if the expert presented his or her case in an "objective and truthful manner".

The case studies occurred on a planet in a far, far galaxy. Any similarity to cases or individuals on planet Earth is pure coincidence.

Case Study 1: Pavement "Recommendations" versus "Design"

This case involves a new warehouse facility constructed for an undefined end user (i.e., speculative warehouse). The facility consisted of three, dock-high buildings, with footprints varying from 341,000 to 420,000 square feet. Based on the civil engineer's drawings provided by the developer to the geotechnical engineer, the facility had four access points from the main road to the facility, three of which were to service truck traffic. The fourth access point consisted of a fire lane traversing along the eastern site boundary.

The geotechnical investigation was performed in 1999 for the developer. Paving recommendations for rigid pavement were provided in the geotechnical investigation. The report had the following statements:

> "The specific pavement sections will be dependent upon the type and frequency of traffic. For drives and parking subject to cars and light trucks, a 5-inch thick, 3,000 pounds per square inch (psi) compressive strength pavement section constructed over a subgrade which has been scarified and recompacted as outlined in the **Earthwork** section should provide for unlimited repetitions over a 20-year life.
>
> For service areas subject to the equivalent of 10 or less loaded semi-trucks per day, and within fire lanes, a minimum 6-inch thick, 4,000-psi compressive strength pavement section is recommended. For access drives subject to increased truck traffic, a <u>minimum</u> 7-inch, 4,000-psi compressive strength pavement section is recommended."

The geotechnical report stated that stabilization of the subgrade could be performed; however, local experience indicated it was typically more cost-effective to increase the pavement thickness versus stabilizing the subgrade. The report did recommend stabilization of the subgrade if traffic speeds exceeded 30 miles per hour (mph). (The subgrade consisted of moderate to low plasticity clayey sand.)

The civil engineer incorporated the preceding recommended sections into the pavement design sheets. Five-inch sections were shown in areas to be used for car and light truck parking. Six-inch pavement was shown in the general heavy truck parking/loading areas and in areas of trailer storage. A minimum seven-inch section was shown in heavy truck drive lanes and entrances. The civil engineer was later directed by the developer to change the seven-inch heavy truck drive lane section to six inches. All sections were detailed to be lightly reinforced (#3's at 18" o.c.) for shrinkage control. Saw joints were identified at 20-foot centers.

During the bidding process, a "value engineering" design was provided by the contractor to the developer to use fiber mesh reinforcing within the pavement in lieu of the identified shrinkage control steel. The design section submitted by the fiber reinforcing sub-contractor included fiber within the concrete, with dowels and dowel baskets at saw joints. The dowel baskets and dowels were later "valued engineered" by the contractor and developer out of the contract to save cost. Neither the geotechnical nor civil engineer was aware that the dowel baskets and dowels were deleted from the fiber design.

To complicate the issues, the developer was in discussion prior to construction with a potential large distribution tenant to lease the entire facility. The tenant required that the facility be fenced, with one monitored entrance into and out of the facility. The tenant also provided estimated traffic frequency to the developer, who in-turn, provided the information to the contractor and fiber reinforcing sub-contractor. (The estimated truck frequency, which turned out to be reasonably accurate, was 150 to 180 per day. No specific truck load data was provided.) Fencing of the facility resulted in channeling all truck traffic through one pavement section and effectively doubling traffic on internal, designated drive lanes.

Within approximately three years following completion of construction, the developer notified the contractor and the geotechnical and civil engineers that the facility was experiencing failure of the pavement. Visual inspection indicated that failures were typically occurring at the contraction joints in heavily loaded traffic lanes. With repeated repetitions and an unsupported edge caused by the lack of dowels, pumping of the subgrade was occurring through the joint. Once sufficient material loss occurred and the pavement near the joint was essentially unsupported, the pavement cracked. Cracking was especially prevalent at joint intersections.

The developer hired a local geotechnical company to assess the condition and design of the pavement. As part of their analysis, a traffic load (number of trucks) versus pavement thickness table was developed for various subgrade conditions. Three subgrade conditions were evaluated:

- compaction of untreated clayey sand (existing on-site soils);
- two inches of asphalt over untreated, compacted clayey sand; and
- six inches of cement-stabilized flexible base.

This analysis is shown in Table 1. The analysis was based on 4,000-psi concrete with doweled contraction joints.

Table 1. Traffic Volume versus Pavement Thickness, inches			
Trucks/Day	**Effective Modulus of Subgrade Reaction, k, pci**		
	60[1]	150[2]	300[3]
10	4.5	4	4
100	6.8	6.3	5.8
120	7	6.5	6.1
150	7.2	6.8	6.4
200	7.6	7.2	6.7

1. Compacted, untreated clayey sand
2. 2 inches of asphalt over untreated clayey sand
3. 6 inches of cement-aggregate treatment

Analysis of Table 1 clearly illustrates that, for low-speed traffic, stabilization of the subgrade saves approximately one inch of concrete. Conversely, increasing the pavement thickness by one inch deletes the need for subgrade stabilization.

The consultant further analyzed the pavement sections for each subgrade condition for un-doweled joints. The consultant concluded that the pavement would have to be ¾ to 1-1/2 inches thicker to carry the same loading condition as the doweled section.

The developer, apparently not realizing what the report said, provided a copy of the report to the original geotechnical engineer. The original geotechnical engineer pointed out that the report supported the original design, and, that without the facility being fenced, would likely have provided a minimum 20-year life.

Frustrated, the developer hired a second geotechnical consultant from the planet Zebulon to analyze the pavement failure. The consultant, Dr. Smarterthanthou, P.E., inspected the facility and issued the following conclusions (and only these four conclusions) in an affidavit which became the basis of the lawsuit.

1. geotechnical engineer failed to recognize that the subgrade soils are subject to "pumping";

2. geotechnical engineer failed to properly identify the high level of truck traffic at the facility;

3. geotechnical engineer failed to properly evaluate the benefits of subgrade stabilization; and

4. geotechnical engineer failed to consider "loss-of-support" in evaluation the pavement thickness and the need for a stabilized subgrade.

With the exception of the second item, the interesting part of these conclusions is that they appear to be in direct conflict with the data shown in Table 1. It should also be stated that the developer's consultant who prepared Table 1 and Dr. Smarterthanthou, P.E., worked for the same geotechnical company, although on different planets.

When it was pointed out to Dr. Smarterthanthou that the high level of truck traffic was only applicable given fencing of the facility, the response was that the original geotechnical consultant should have counted the number of dock doors, multiplied by 2, and designed the pavement accordingly.

What do you think? Do general recommendations really represent design? By ignoring the role of the owner in the decision-making process, was Dr. Smarterthanthou really truthful?

Case Study 2: Pier Settlement

The second case involves settlement of belled piers on a new office/warehouse tilt-wall, dock-high building. The litigation was in conjunction with immediate settlement, and costs associated with correction thereof, of belled piers which occurred during placement of the tilt-wall panels. The geological conditions consisted of 30 to 40 feet of alluvial clay over weathered grading to unweathered shale.

The original geotechnical investigation recommended underreamed piers founded at a depth of 15 feet, proportioned using an allowable bearing pressure of 4.5 ksf. Estimated elastic settlement was ½ inch. The report cautioned that some re-shimming of tilt-wall panels may be required.

The allowable bearing pressure was for total load conditions. Considering dead load only, the actual bearing varied from 3.2 to 3.5 ksf considering design bells varying from 72 to 78 inches.

One hundred and fourteen piers were required along the perimeter of the structure for support of the tilt-wall panels. Upon placement of the tilt-wall panels, elastic settlement of the piers was noted. Eighteen of the 114 had total settlement of less than one inch. Eight others experience movement of less than 1-1/2 inches. The remaining 78 piers experienced settlement varying from 2 to 6-1/2 inches.

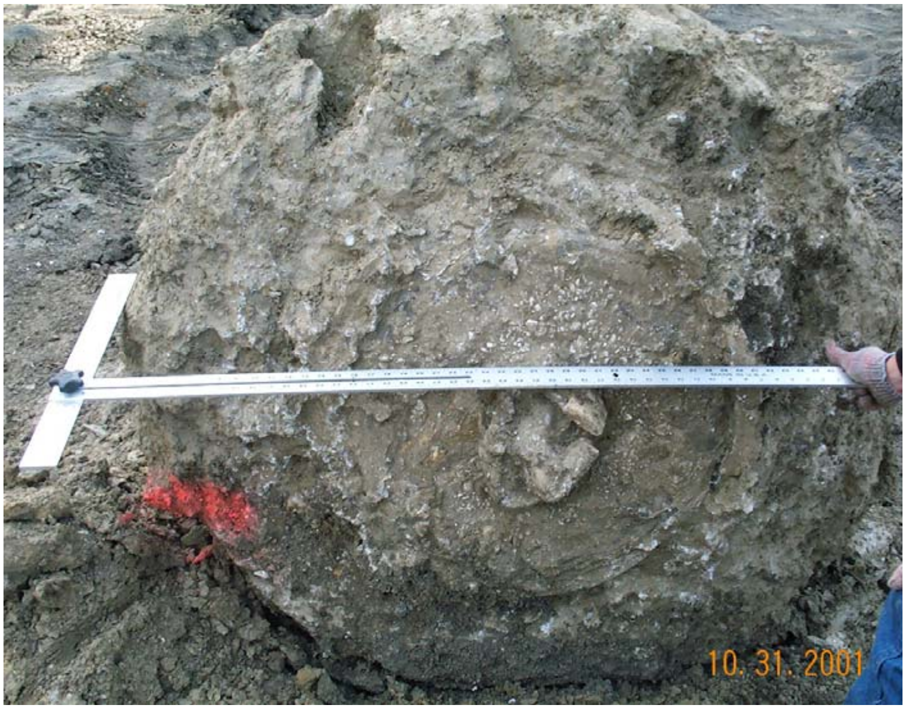
To evaluate the probable cause of observed differential settlement, 9 piers were partially excavated to expose the pier shaft and bell. This process allowed for measurement of the extension of the bell beyond the shaft. Three additional piers were completely exhumed to allow for detailed inspection of both the size and shape of the bell. Design diameter, measured diameter and contact pressure for 11 of the 12 piers are shown in Table 2. The calculated contact pressure in Table 2 is based on solid contact between the pier concrete and undisturbed soil. It does not account for lost area associated with any loose soil in the pier excavation at the time concrete was placed. (The specific dead load was not defined for one exhumed pier.)

Table 2.
Measured Bell Diameter versus Dead Load

Design Underream, inches/Dead Load, kips	Field Diameter, inches	Contact Pressure, ksf	Settlement, inches
72/90	60	4.6	2.6
72/90	52	6.1	5.8
72/90	60	4.6	2.6
78/115	72	4.1	2.2
72/90**	56	5.3	4.2
78/115	58	6.3	3.4
78/115	68	4.6	0.6
72/90	58	4.9	3.4
72/90**	60	4.6	2.6
72/90**	58	4.9	3.4
78/115	68	4.6	2.2

**Piers completely exhumed.

All three exhumed piers had similar shapes. The bottom of one of the three piers completely exhumed is shown in Photograph 1. By exhuming the piers, the bottom of the underream could also be inspected. It was noted by the poor quality of the concrete mold that the bottom had significant loose soil at the time the concrete was placed.



Photograph 1. Bottom of exhumed pier. Note how poorly the underream was cleaned of debris at time of concrete placement.

The geotechnical engineer of record also provided pier observation. It was noted that two separate pier technicians were utilized for inspection of the perimeter piers. One technician inspected 96 piers. The second inspected the balance of 18 piers. Fourteen of the 18 piers inspected by the second technician experienced settlement of less than one inch. Two additional piers inspected by the second technician experienced settlement of less than 1-1/2 inches.

There are numerous reasons why belled piers experience elastic settlement, to include bearing and delays associated with excavation and placement of concrete. It has been observed that elastic heave of the bottom of the pier can occur in larger diameter piers where there is a six plus hour delay between the time the bell is constructed and placement of concrete. This could account for some of the observed differential movement, but not likely any movement over about 1 to 1-1/2 inches.

Based on the information in Table 2, and extensive experience in the area on other projects, it was believed that the cause of excessive settlement (in excess of approximately one inch) was associated with incomplete extension of the bell coupled with insufficient cleaning of the excavation of loose soil prior to placement of the concrete.

Based on the information shown in Table 2, a load versus settlement curve was developed and is provided in Figure 1. The data appeared to show a clear relationship between approximate contact pressure and settlement. Figure 1 was also considered to be very conservative because the "contact pressure" calculation was based on the projected area of the smaller bell. Clearly, because of loose soil remaining in the excavation at the time concrete was placed, the actual bearing surface was smaller than the projected area of the bell.

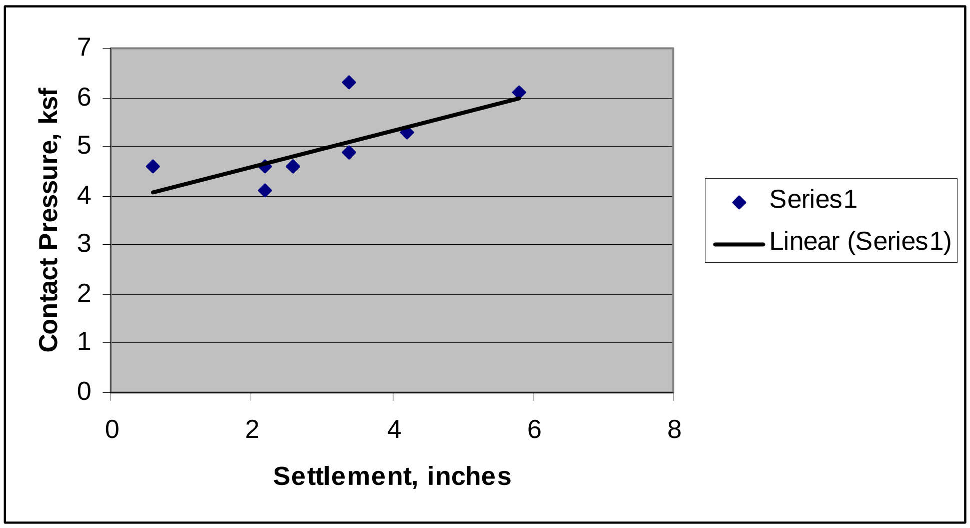


Figure 1. Contact pressure versus observed settlement.

The pier drilling sub-contractor hired their own geotechnical expert to evaluate the probable cause of movement. She recommended additional borings and analysis. As part of the investigation, three consolidation tests were performed. The consolidation curves for the three tests are shown in Figure 2.

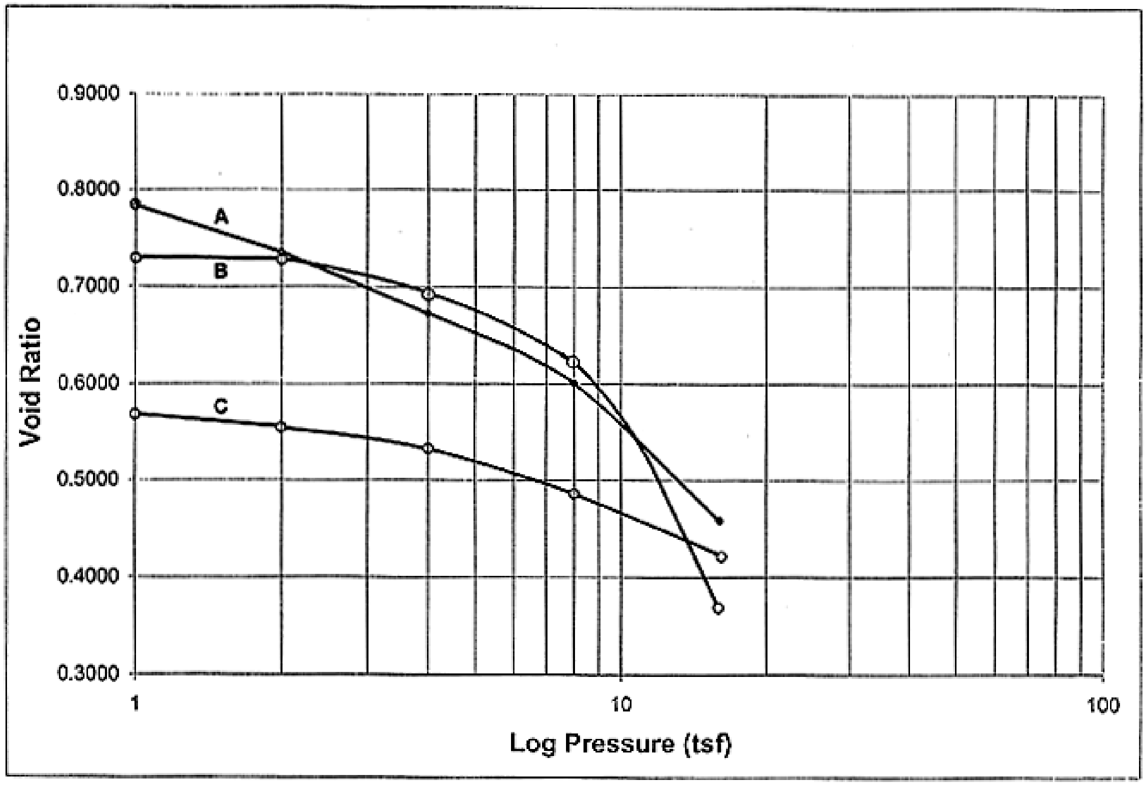


Figure 2. Consolidation tests

Curve “A” is a classic shape associated with a test performed on a severely disturbed sample (Schmertmann, 1953). For Curve “C”, the preconsolidation pressure was never determined because of termination of the test at 16 tons per square foot (tsf). Apparently not recognizing that consolidation Tests “A” and “C” were not valid, after extensive computer modeling, the consultant’s conclusion was that the original geotechnical investigation’s bearing pressure was in error. Clearly, if consolidation Test “B” was correct with an even higher preconsolidation pressure for Test “C”, then limited settlement at the design bearing should have occurred. Legal wrangling ensued.

Any analysis, be it the original geotechnical engineer or consulting expert, is only as valid as the data it is based on. Without sufficient experience to recognize sample disturbance, how valid is an expert’s opinion?

To arrive at a conclusion that the original bearing was in error, the consulting expert also had to completely discount the physical condition of exhumed piers, which were both undersized and clearly showed excessive loose soil was present at the time concrete was placed. Was she being truthful?

Case Study 3: Utility Backfill Settlement

This case involves gross settlement of utility line backfill below a city street. The utility line varied in depth from 20 to 25 feet below grade. The utility excavation was sloped at two vertical to one horizontal (2V:1H) to a depth of approximately six feet. Vertical excavation was then performed using a 30-inch wide bucket to required depth.

Native clays were used as backfill. Backfill specifications required 95 percent of ASTM D 698 density, at a moisture content of plus or minus 2 percent of optimum moisture. The majority of excavation and backfilling operations were performed during early to mid summer during relatively dry weather. Site paving was performed in late summer to early fall.

Field density and moisture content tests were performed on the utility backfill by a geotechnical engineering company hired by the city. Tests results were shown at approximate 100-foot intervals for each 12-inch lift. Reports indicated the fill met the project requirements. There was some question; however, about the validity of the tests since they were reportedly performed in a 20- to 25-foot deep, unsupported 30-inch wide trench.

Shortly after completion of site paving, seasonal rainfall occurred. Initially, settlement of the utility backfill was observed within portions of excavations which extended outside the area of paving.

As seasonal rainfall progressed, water ponded within excavated areas which had initially settled, thus exacerbating saturation of the fill. The settlement extended below paving, resulting in gross pavement failure. An example of the magnitude of settlement is shown in Photograph 2.

Photograph 2. Example of degree of utility line excavation settlement.

Borings performed in the backfill in areas exhibiting settlement encountered very soft (pocket penetrometer values of 0 to 1.0 tsf), very moist clay. The dry unit of the fill varied from approximately 88 to 95 percent of ASTM D 698 density. Because the samples were obtained after settlement had occurred, the pre-settlement densities were estimated to be below 90 percent required density.

The utility contractor hired a geotechnical consultant to evaluate the cause of settlement. After extensive investigation and analysis, the consultant concluded that the settlement was associated with water saturating the backfill, resulting in collapse. In other words, the fill was placed correctly; however, the addition of water caused the backfill to collapse.

The consultant stated that the fill was placed in accordance with plans and specifications, and that the source of water was associated with a design flaw in the civil engineering plans. The specific flaw in the civil plans is not relevant to this discussion. In any event, the consultant was able to transfer a significant portion of the liability for the settlement from the contractor to both the civil engineer and owner, since the owner hired the civil engineer.

Significant study has been conducted over the last 80+ years on the performance of clay used as fill, to include classic studies by Proctor (1933), and Turnbull and Foster (1956). A study of hydrocompression of deep fill (excess of 70 feet) was also reported by Brandon, Duncan and Gardner (1990). This study found that clay fill compacted to a minimum of 92 percent of modified proctor density (approximately 96 to 97 percent standard density) underwent heave or settlement of approximately one percent or less, dependent upon the compacted moisture, for fills of 30 feet or less. An abundance of literature throughout the last 50 years has reported similar results.

Was the utility contractor's consultant really being truthful or were they being an advocate for their client? Should geotechnical engineers anticipate settlement of 8 to 20 percent of the height of a fill where it is compacted to 95 percent of ASTM D 698 density if it gets wet during the life of the project? This opinion appears to be contrary to the observed performance over the last 80 years for properly compacted fill.

Conclusions

The case studies are presented in summary form, with a significant amount of detail deleted for the sake of brevity. However, it is believed the essence of each case has been preserved to allow the reader to reach their own conclusions regarding the consultant's role and opinions.

The consultant's role, in the writer's opinion, is to evaluate each case and develop probable or likely opinions as to the causes of the observed behavior or legal or insurance claim. If this opinion supports his or her client's position, so be it. If not, the consultant must be honest and forthright with the client. To be less so is simply selling the profession to the highest bidder.

References

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