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STATIC MODULUS OF SUBGRADE REACTION FOR LARGE STRUCTURES AT SAVANNAH RIVER SITE (SRS)

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1.0 INTRODUCTION

The term *modulus of subgrade reaction* relates the contact pressure below a foundation with settlement resulting from the contact pressure. It is sometimes referred to as the coefficient of subgrade reaction or simply subgrade reaction. It is generally expressed as the ratio of the contact pressure to deformation,

$$k_s = \frac{p}{\Delta} \qquad \text{(Equation 1)}$$

where p is the foundation contact pressure, Δ is the deformation (or settlement), and the subscript s refers to the foundation size. For example, k_1 refers to the subgrade reaction for a 1-foot square foundation. Where no subscript appears, the value refers to the subgrade reaction for the entire foundation. These values can either be estimated based on published data or they can be developed from field loading tests. Generally, approximate estimates of k serve the purpose.

2.0 COHESIONLESS SOILS (SANDS)

For cohesionless soils Terzaghi (1955) recommended values of k_1 as summarized below.

Table 1. Values of k_1 in tons/ft^3 for 1 ft^2 Plates or 1 ft Wide Beams Bearing on Sand

Relative Density of Sand	**Loose**	**Medium**	**Dense**
Dry or moist sand, limiting values for k_1	20-60	60-300	300-1,000
Dry or moist sand, proposed values	40	130	500
Submerged sand, proposed values	25	80	300

Terzaghi further recommended that for cohesionless soils the following relationship was to be used to relate k_1 to k_s for various foundation widths (Terzaghi, 1955),

$$k_s = k_1\left(\frac{B+1}{2B}\right)^2 \qquad \text{(Equation 2)}$$

where k_s represents the subgrade reaction for the foundation, k_1 refers to a 1-foot square plate, and B is the actual foundation width (in feet). Figure 1 depicts the relationship in equation 2 between foundation width and the modulus reduction factor for sands. For foundation widths greater than about 25 feet, k_s is approximately 25% of k_1.

3.0 COHESIVE SOILS (CLAYS)

For clays Terzaghi (1955) recommended the following values of k_1^*, based on a relationship with the unconfined compressive strength, q_u.

Table 2. Values of k_1^* in tons/ft^3 for Square Plates 1 ft x 1 ft or Beams 1 ft Wide, Bearing on Pre-compressed Clay

Consistency of Clay	Stiff	Very Stiff	Hard
Values of q_u, tsf	1-2	2-4	> 4
Range of k_1^*, square plates	50-100	100-200	> 200
Proposed values, square plates	75	150	300

For clays Terzaghi (1955) further recommended that k_1^* be reduced as follows to account for the size of the loaded area for foundations other than square,

$$k_1 = k_1^* \left(\frac{l + 0.5}{1.5l} \right) \qquad \text{(Equation 3)}$$

where l is the ratio of the foundation length to the foundation width. The relationship is depicted on Figure 2, which shows that k_1 for clays approaches 2/3 of k_1^* for continuous footings. In addition, as noted by Terzaghi, the value of k_s for clays decreases in inverse proportion to the foundation width, B (in feet):

$$k_s = \frac{k_1}{B} \qquad \text{(Equation 4)}$$

For normally consolidated and/or soft clays k_1^* is lower than given in Table 2 and becomes even lower as the foundation size increases, as shown on Figure 1. Thus, Terzaghi (1955) recommended that for design purposes, beams and rafts should be assumed to be rigid. Note that Terzaghi designated the values in Table 2 above for "pre-compressed" clays. This would indicate some degree of overconsolidation, however to what level is not known. These values are not recommended for normally consolidated clays without a more in depth examination.

It must be recognized that the modulus of subgrade reaction is not a property of the soil. It is a function of the soil stiffness, foundation shape and size, and the load being imposed. Thus, design values of k_s vary from foundation to foundation at a given site. Further, Terzaghi (1955) states that the above values (sands and clays) are valid only for foundation contact pressures which are smaller than ½ the ultimate bearing pressure.

Similar relationships appear in Navy Design Manual DM-7.1 (NAVFAC, 1986) for both coarse grained and fine grained soils, as shown below (adapted from a plot of k_1 versus density or stiffness). The ranges given in Table 3 are similar to those given by Terzaghi, except in the dense range where the suggestions by Terzaghi are somewhat higher. Historically the Savannah River Site (SRS) has relied on the suggestions of Terzaghi, recognizing that either set of suggestions (Terzaghi or DM-7) is a guideline and that actual measured settlement with applied load is the ideal.

Table 3. Modulus of Subgrade Reaction k_1 in tons/ft^3 (NAVFAC DM-7.01)

Relative Density (sands)	**Loose**	**Medium**	**Dense**
Coarse-grained soils	13-75	75-187	187-284
Submerged coarse-grained	7-38	38-95	95-142
Stiffness (clays)	**Medium Stiff**	**Stiff**	**Very Stiff**
Fine-grained soils	0-24	24-100	100-290

4.0 SUBGRADE REACTION BASED ON FIELD TESTS

For design, estimates of k_s are generally derived from field tests that relate stiffness or relative density for a given soil type. For example, the standard penetration test (SPT) blow count (N-value) has typically been used to determine the *in situ* soil relative density, in qualitative terms such as loose, medium, and dense. Tables 1, 2, and 3 (above) give guidelines with respect to unconfined compressive strength for clays and relative density (qualitatively) for sands.

For sands, standard penetration test N-values range from 4 to 10 for loose sands, 10 to 30 for medium sands, and 30 to 50 for dense sands. At SRS, typical SPT N-values in the upper 75 feet range from about 15 to 30. Assuming the groundwater elevation to be within ½ the foundation width, the submerged sand suggestions would apply. Thus, based on the suggestions of Terzaghi, a representative value of k_s for a large foundation at SRS bearing on medium dense submerged clayey sands would be approximately ¼ of 80 tons/ft^3 (tcf), or 40 kips/ft^3 (kcf).

5.0 SUBGRADE REACTION BASED ON MEASURED SETTLEMENT

It is recognized that the preceding discussion, although sound, is based on qualitative assessments and therefore the recommendations are simply guidelines. A better and more defensible approach would include actual measurements of load versus settlement for foundations of similar size and average contact pressures. For most designs this is impractical because the design obviously precedes any measurement of settlement. At SRS, however, settlement monitoring of large critical facilities has been ongoing since construction began in the early 1950s. Because the subsurface conditions from area to area are generally similar (in the upper 150 feet of the soil column), measurements from one area can be utilized to estimate subgrade reaction for another area. Therefore, actual SRS settlement measurements of existing structures are utilized to refine the determination of the subgrade modulus for new structures of similar size and loading. Although many structures have been monitored at the SRS, three are particularly useful because the loading is known. Each is discussed below.

5.1 Liquid Waste Storage Tanks -- H Area

During the period from the early 1950s to the mid-1980s, several liquid waste storage tanks have been constructed in the General Separations Areas (F and H Areas) near the center of the SRS. The size and dead load of each tank are known. The tanks in H Area are of particular interest because they have the most extensive and continuous settlement monitoring program. The subsurface conditions generally consist of medium dense clayey sands with SPT N-values generally in the range of 10 to 30 blows/foot and shear wave velocities (V_s) around 1,000 ft/sec (fps). The elevation of the groundwater table in this portion of H Area is approximately 275 ft.

This evaluation utilizes settlement data acquired for tanks 48-51 (also known as the "ITP tanks"). These tanks are located on what is called the "south hill" in the southern portion of H Area. Settlement monitoring has been ongoing since the beginning of construction (1980), and is summarized in WSRC, 1999. Although settlement monitoring has continued and the settlement has increased since 1999, the settlement utilized for the evaluation used only that fraction of settlement that was attributed to initial loading, i.e., it does not include secondary consolidation or creep-type settlement.

Each tank's dead load (empty) is approximately 1.75 kips/ft^2 (ksf); the operation load (i.e., tank partially full) is about 2.2 ksf. The four tanks (48-51) are each 95 feet in diameter and configured in a square pattern surrounded on all sides by embankment fill. The fill loading is approximately 4.8 ksf (40 feet of fill at a unit weight of about 120 pounds/ft^3 [pcf]), in an area of about 400 ft by 400 ft. It is clear that the embankment fill has the greatest affect in terms of settlement. However to assess the value of k, a weighted average load was determined based on the embankment load of 4.8 ksf over an area of 400 ft by 400 ft and the tank load of 2.2 ksf over an area equivalent to four tank diameters, each 95 feet. The resulting equivalent load is about 4.3 ksf.

Settlement versus log time for the four tanks is shown on Figures 3-6. Only the elastic portion of the settlement was used to calculate k_s. Thus, the secondary consolidation or creep-type settlement was ignored. In reviewing the data, it was determined the elastic settlement ended

near the end of March 1982. The measured settlements for each of the three risers on each of the four tanks are summarized below.

Table 4. Historic ITP Tank Settlement

Tank	Date	Riser 1 N Settlement (inches)	Riser 2 SE Settlement (inches)	Riser 3 SW Settlement (inches)	Average Settlement (inches)
48	March 1982	1.596	1.656	1.428	1.6
49	March 1982	1.668	1.476	1.572	1.6
50	March 1982	1.488	1.356	1.212	1.4
51	March 1982	1.464	1.284	1.315	1.4

Equation 1 is used to calculate k_s. The resulting back-calculated values of k_s for each tank are:

Tank 48: $k_s = 32$ kcf

Tank 49: $k_s = 32$ kcf

Tank 50: $k_s = 37$ kcf

Tank 51: $k_s = 37$ kcf

Utilizing the individual settlement measurements for each riser would result in a range of k_s between 43 kcf and 31 kcf. Thus, it appears for the ITP tanks and structures of similar size and loading, values of k_s between 30 and 40 kcf are appropriate. Note, if we convert k_s to k_1, the resulting values (120 to 160 kcf) are right in line with the recommendations of Terzaghi for medium dense submerged sands (see Table 1, noting the units are tcf).

5.2 Defense Waste Processing Facility (DWPF)

The Defense Waste Processing Facility (DWPF) is a vitrification facility, processing into glass much of SRS's highly radioactive liquid waste for eventual burial. It consists of several structures located in S Area, which is adjacent to H Area at the SRS. The facility was constructed in the 1980s and went into operation in the early 1990s. The main structure is the 221-S facility, which houses the vitrification process.

Settlement monitoring has been done continuously since excavation began in the early 1980s. The instrumentation monitored heave due to excavation relief as well as subsequent settlement as the facility was constructed. The subsurface conditions are similar to the conditions at nearby H Area; the site is underlain by medium dense clayey sands with SPT N-values ranging between 10 and 30 blows/ft and V_s of about 1,000 fps. The elevation of the groundwater table in S Area is approximately 245 ft.

The 221-S structure is approximately 362 feet long and 117 feet wide. The foundation mat is approximately eight feet thick. The elevation of the bottom of the mat is 270 feet, except for a portion on the northern end which is 256 feet. Original ground surface prior to construction was about elevation 278 feet. Based on design drawings, the load on the mat varies between 2.8 and 9.6 ksf (WSRC, 1995). The average mat load is about 5 to 5.5 ksf, with the center of gravity near the geometrical center of the building (WSRC, 1999). Thus, for computations an average load of 5.25 ksf will be utilized.

Settlement monitoring has been ongoing since construction began, and continues today. As with the waste tanks, the settlement that will be used to back-calculate k_s will be the elastic portion that occurred as load was applied and shortly thereafter. Secondary consolidation or creep-type settlement is ignored. Figure 7 presents the settlement versus log time for the four corners of the 221-S structure.

Construction was essentially complete in January 1988, however it appeared elastic-type movements occurred until February 1989, and therefore total settlements at that time will be used. Measured settlements at the four corners (represents the range of settlement at that time) of the 221-S structure as of February 1989 are summarized below (WSRC, 1999).

Table 5. DWPF Settlement 221-S

Settlement Marker	Date	Settlement (inches)
NW #3	February 9, 1989	2.58
NE #18	February 9, 1989	2.62
SW #14	February 9, 1989	1.63
SE #9	February 9, 1989	1.48

Utilizing equation 1, the back-calculated values of k_s are as follows:

Marker #3: k_s = 24 kcf

Marker #18: k_s = 24 kcf

Marker #14: k_s = 39 kcf

Marker #9: k_s = 43 kcf

Thus, for the 221-S structure, and structures of similar size and loading, k_s of about 20 to 40 kcf is appropriate. As above, if we convert the above values of k_s to k_1, the results (96 to 172 kcf) are also in line with the recommendations of Terzaghi for medium dense sands submerged sands (see Table 1, again noting units are tcf).

5.3 Tritium Extraction Facility Remote Handling Building (RHB)

The Tritium Extraction Facility (TEF) is situated in the northern portion of H Area adjacent to the Replacement Tritium Facility (RTF). It consists of several structures; however the structure under consideration here is the Remote Handling Building (RHB). The RHB was designed by WSRC utilizing subgrade moduli that ranged from 20 to 40 kcf. The basis for the range was the aforementioned discussion on the Waste Tanks and DWPF. Settlement monitoring and estimates of dead and live loads affords us the opportunity to back-calculate actual moduli.

The RHB is approximately 215 feet by 70 feet in plan and is embedded about 30 feet below original ground surface (approximately 290 feet elevation). Two sides of the excavation were supported with a tied-back soldier pile and wood lagging wall, while the other two sides were

sloped. Settlement monitoring was initiated prior to construction to measure heave and continues through today monitoring for continued long-term settlement.

The subsurface conditions are similar to the other two facilities described above; the site is underlain by medium clayey sands with SPT N-values of 10 to 30 blows/ft and V_s near 1,000 fps. The elevation of the groundwater table in this part of H Area is about 250 to 255 feet, approximately 5 to 10 feet below the mat foundation. The estimated pre-construction gross load was 4.2 to 4.5 ksf, resulting in a net load at the mat level of approximately 1 ksf (assuming an *in situ* soil moist unit weight of 115 pcf). Assuming a medium dense sand below the groundwater table would result in an estimated k_s (according to Terzaghi; see Table 1) of 40 kcf for a large foundation. Utilizing relationships published in Navy Design Manual DM-7.01 (NAVFAC, 1986), k_s, for a large foundation, would range from about 19 to 48 kcf for medium dense sands below the groundwater table. Therefore, based on subsurface conditions, literature recommendations, and back-calculated moduli from the Waste Tanks and DWPF, a range of subgrade modulus of 20 to 40 kcf was recommended for design.

As mentioned above, the RHB is unlike the previous two structures in that it is embedded about 30 feet below ground surface, bearing on a mat foundation six feet thick. The estimated net load for design was about 1 ksf. This value has since been revised down to about 0.5 ksf, based on quantity takeoffs and estimates by construction forces during construction of the facility.

Because of the deep excavation, the subgrade heaved upon soil removal and re-compressed upon loading. Figure 8 presents the relationship (unpublished) between deformation, time (calendar date) and estimated loading for the RHB. Table 4 below summarizes the heave and subsequent settlement monitoring for the RHB. The subgrade modulus, k_s, is computed based on a net load of about 0.36 ksf and settlements after recompression at the end of load application (January 2003).

Table 6. Remote Handling Building Deformation Measurements

Settlement Marker	Date	Heave (inches)	Date	Settlement (inches)
TEF-2	Oct. 2000	0.6	Jan. 2003	0.24
TEF-6	Oct. 2000	0.9	Jan. 2003	0.21
TEF-9	Oct. 2000	0.5	Jan. 2003	0.13

Utilizing equation 1, the back-calculated values of k_s are as follows:

Marker #TEF-2: k_s = 18 kcf

Marker #TEF-6: k_s = 21 kcf

Marker #TEF-9: k_s = 33 kcf

The measured settlements and estimated load confirm the range of subgrade modulus, k_s, used in design for the RHB, recognizing that the higher modulus results in a more conservative foundation design. The results further substantiate the use of k_s no higher than 40 kcf for large structures engaging soils to depths greater than about 50 feet.

6.0 CONCLUSIONS

Three structures have been reviewed that have good control on loads and deformations. Each structure is relatively large thus resulting in theoretical k_s values equal to 25% of predicted values for plates or beams of 1 foot (k_1). This allows direct comparison of the computed k_s values for each structure and allows comparisons to published recommendations for 1-foot plates and beams.

The subsurface conditions and location of the groundwater table are similar in all three cases, thus the resulting k_s values are a function only of loaded area and load intensity. The results demonstrate that for the granular nature of the subsurface soils at SRS's H and S Areas, k_s varies from about 15 to 40 kcf. Thus, for design of critical facilities of similar loading and foundation size it is appropriate to use a range of k_s not less than about 10 to 15 kcf and not greater than about 30 to 40 kcf. For these values of k_s, the corresponding values for k_1 would be 40 to 60 kcf

and 120 to 160 kcf, respectively, in agreement with published recommendations of Terzaghi and NAVFAC for 1-foot square plates and 1-foot wide beams bearing on submerged medium dense sand. It is fully expected that smaller foundations (widths less than about 20 feet) would have a higher k_s, approaching the published recommendations as the foundation width approaches 1 foot.

The preceding discussion and recommended values pertain to granular soils only. If site-specific investigations determine that thick deposits of clay exist, then a re-evaluation of the appropriate subgrade modulus would be required. Further, the original intention of Terzaghi's paper was for the computation of stresses and moments in foundations. It was never intended to use theories of subgrade reaction to compute foundation settlements. However, the use of actual measured structure settlement coupled with accurate estimate of foundation loading to compute k_s is the best possible method to determine k_s for a given structure. This is generally not relevant to design situations as, by definition, the measurements are taken after the fact. However, at SRS subsurface conditions are such that historic measurement of settlement and load for large structures result in k_s values that corroborate values used in design across the site. It is this experience that has led us to the conclusions given herein regarding k_s for large structures at the SRS.

7.0 REFERENCES

Terzaghi, Karl, 1955. "Evaluation of Coefficients of Subgrade Reaction," *Geotechnique*, Vol. 5, 1955, pp 297-326.

NAVFAC DM-7.01, Department of the Navy, *Soil Mechanics*, Washington, DC, September 1986, Fig. 6, p. 7.1-219.

WSRC, 1995. "Geotechnical Seismic Assessment Report for Defense Waste Processing Facility," WSRC-TR-00072, Rev. 0, February 1995.

WSRC, 1999. "Settlement of Defense Waste Processing Facility," K-ESR-S-00003, September 1999.

WSRC, 1999. "Settlement of H-Area Waste Storage Tanks and Structures," K-ESR-H-00011, October 1999.

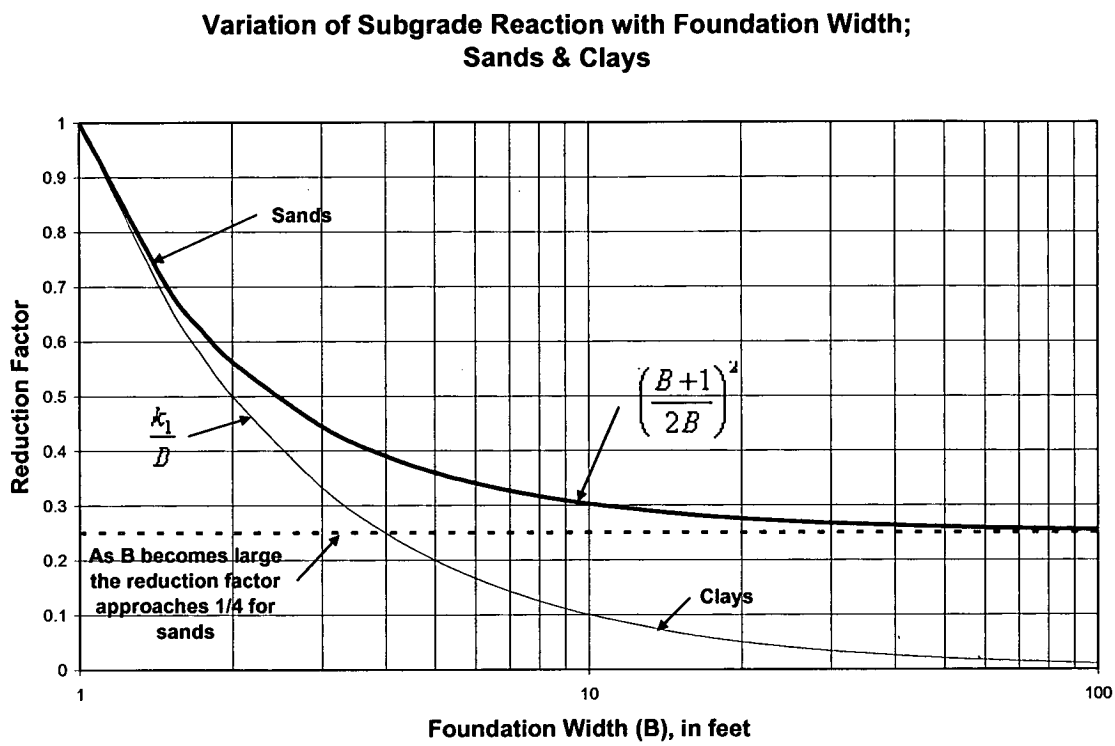

Figure 1. Variation of k_s with Foundation Width for Sands and Clays

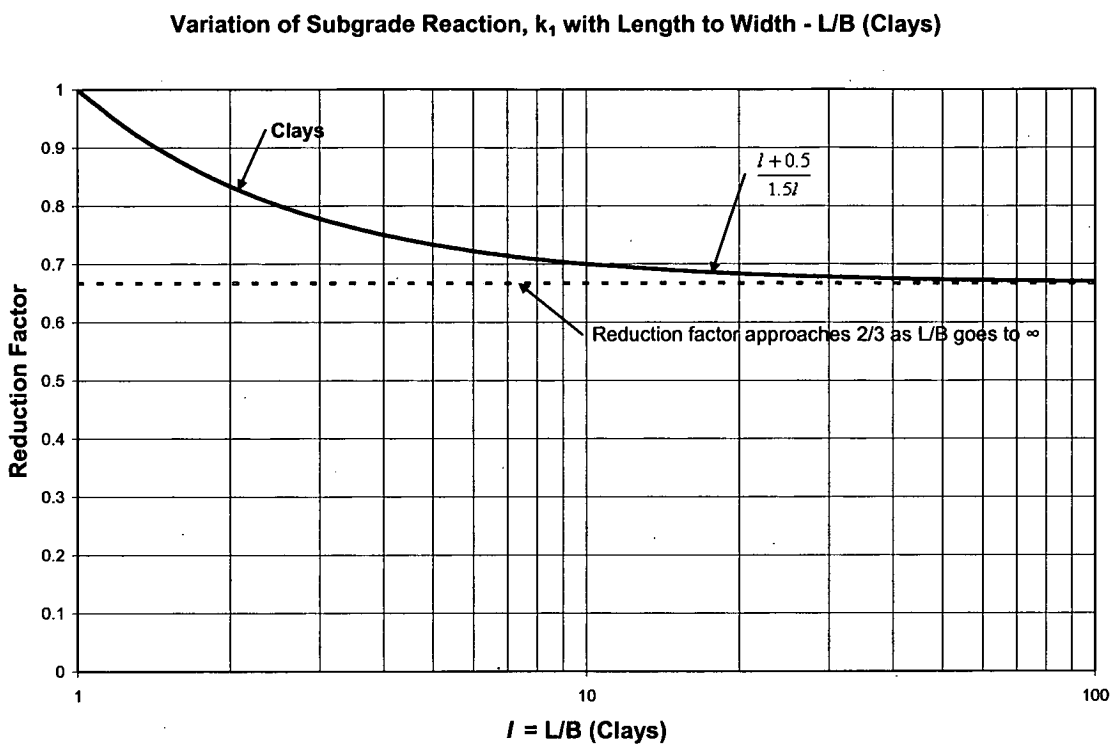

Figure 2. Variation of k_l with Foundation Size for Clays

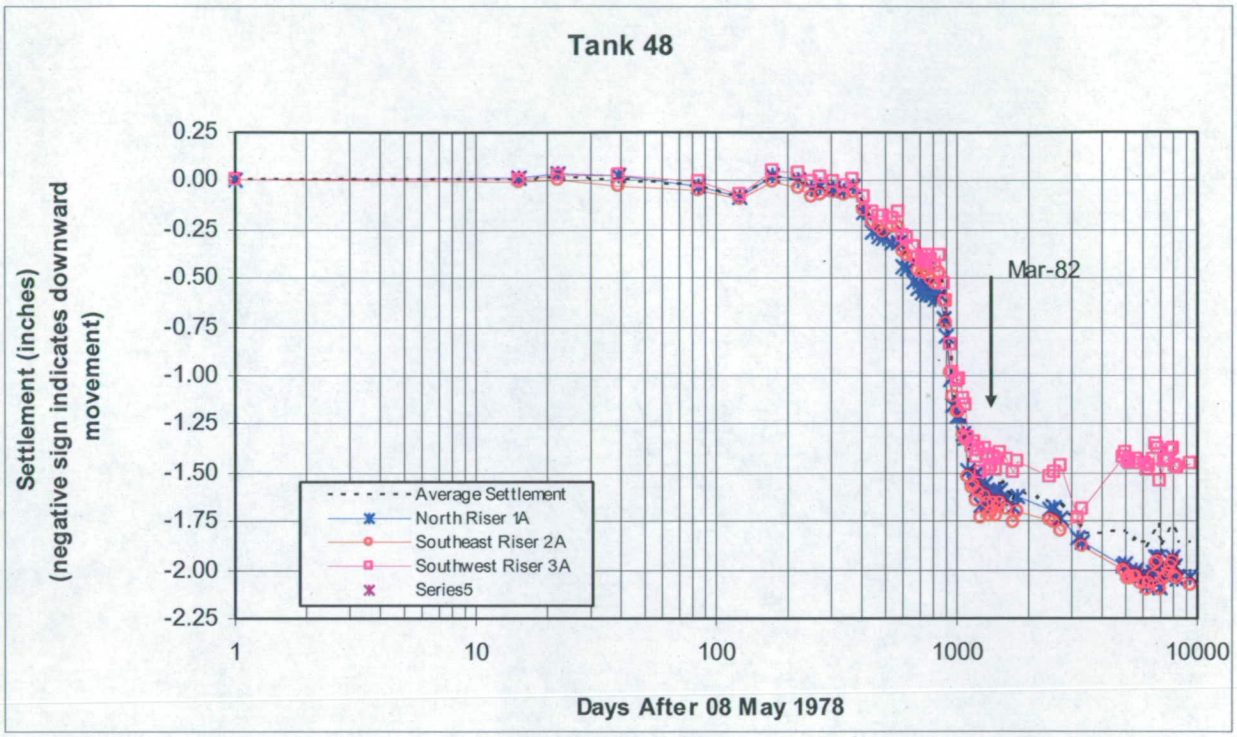

Figure 3. Tank 48 Settlement

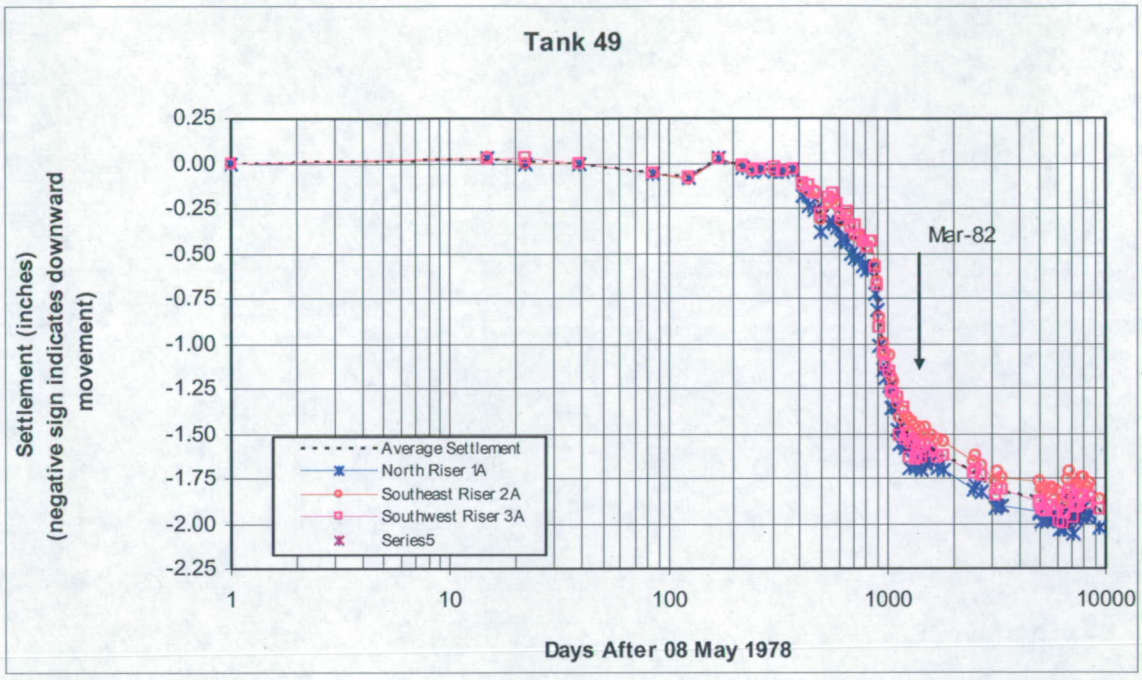

Figure 4. Tank 49 Settlement

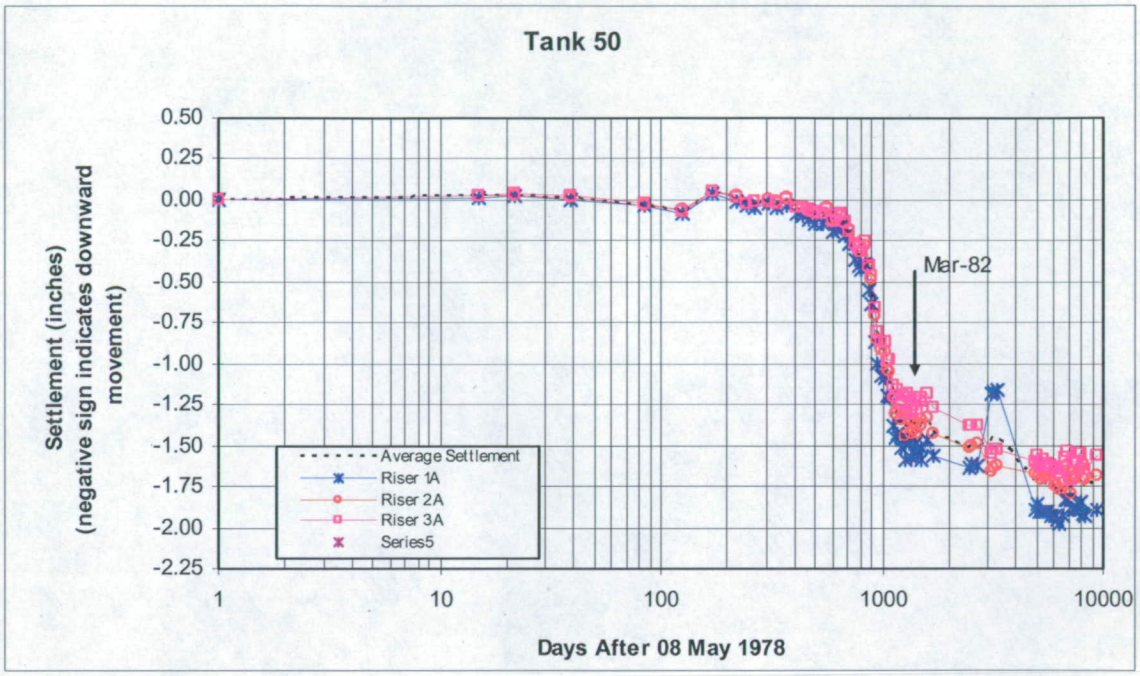

Figure 5. Tank 50 Settlement

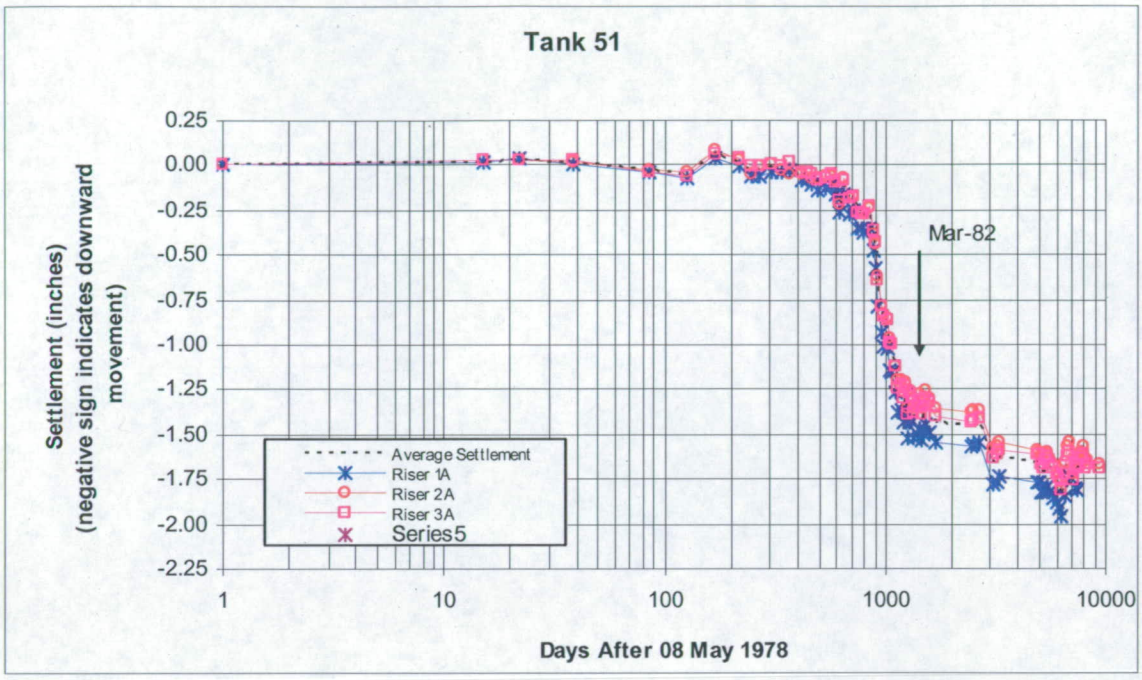

Figure 6. Tank 51 Settlement

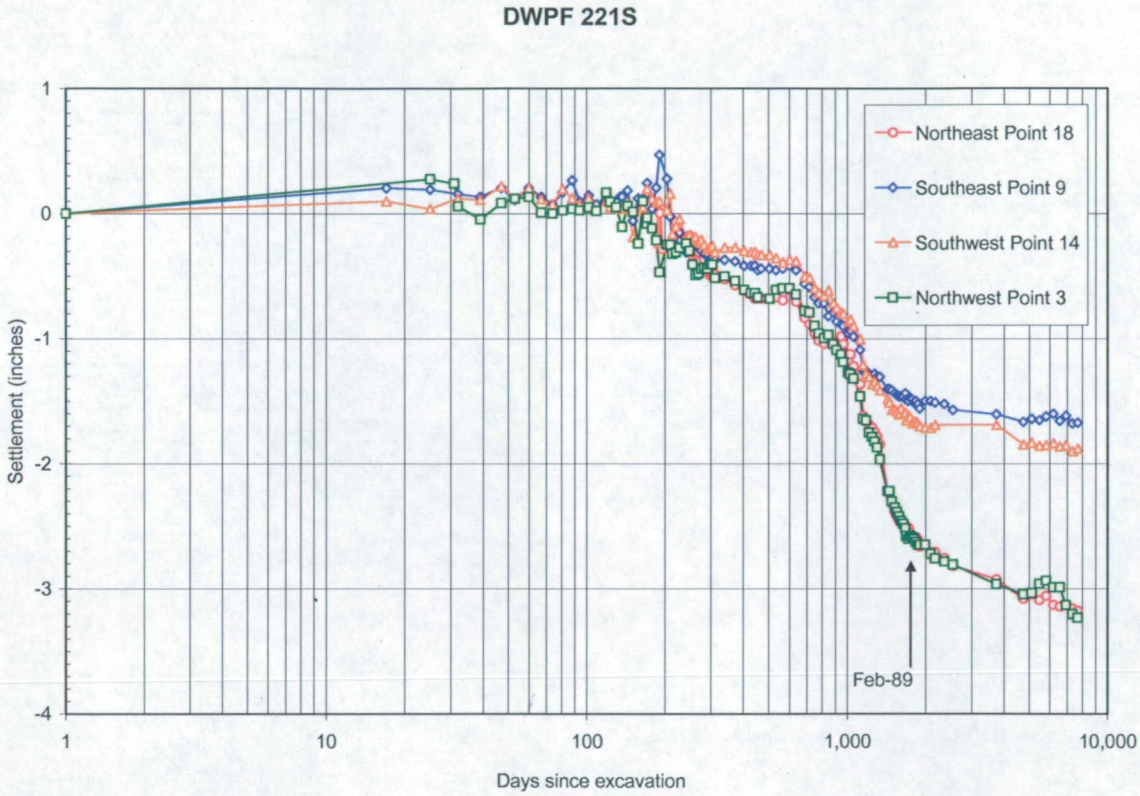

Figure 7. DWPF 221-S Settlement (4 corners)

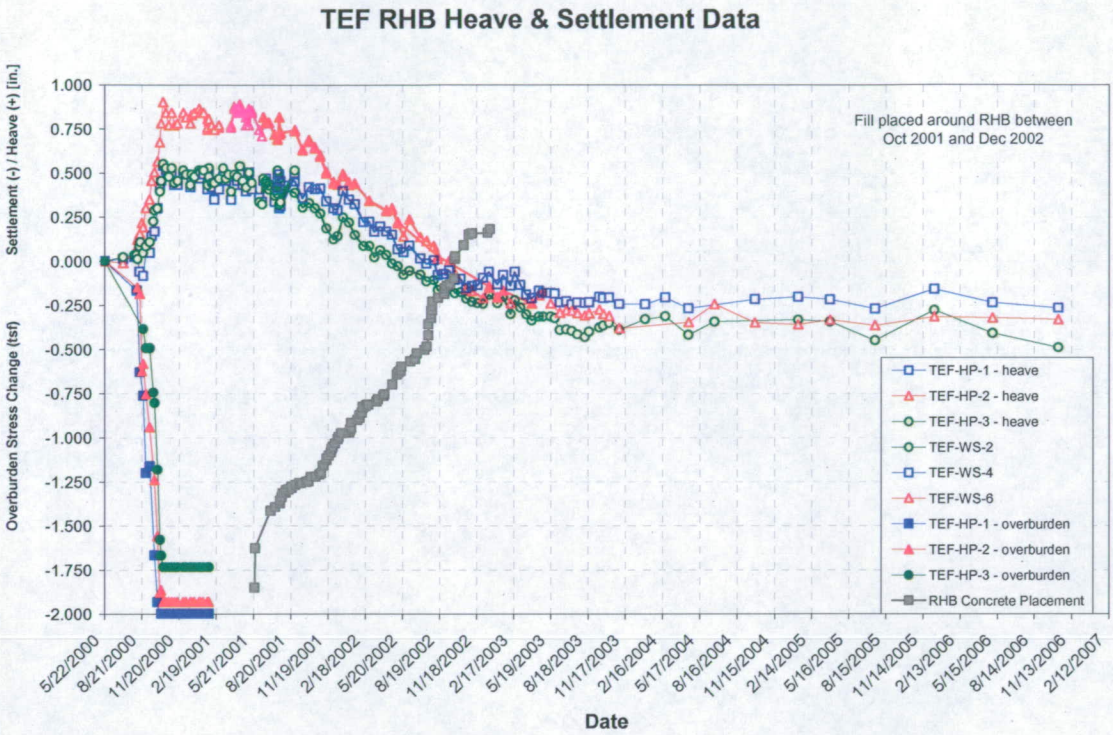

Figure 8. TEF Remote Handling Building (RHB) Deformation