CHAPTER 9 GEOTECHNICAL STUDY AND PAVEMENT DESIGN

9.1 SUMMARY RESULTS OF THE GEOTECHNICAL STUDY

Subsurface conditions were evaluated for this DCR by drilling 22 borings along the SR 303L corridor between Indian School Road and Bell Road between December 13, 2001 and December 21, 2001. This series of 22 borings consisted of a single, 60-foot-deep boring drilled in the vicinity of each crossroad intersection, and 10-foot deep borings drilled at approximately midway between the crossroads. Four additional borings were drilled in March 2004: two at the I-10/Cotton Lane traffic interchange, one at the southwest corner of the intersection of McDowell Road and Cotton Lane, and one in the vicinity of the proposed US 60/SR 303L Interchange. These four additional borings were each drilled to a depth of 99 feet below existing grade.

The soils encountered during the field investigation were generally granular, consisting predominantly of silty, clayey, and relatively clean sands, with lesser amounts of sandy and silty clays of low plasticity. The fines content of the clayey soils range from 35 to 75 percent, and is typically below 60 percent. The plasticity index of the clayey soils ranges from 5 to 30 percent, and is typically below 10 percent. The remainder of the soils is generally non-plastic with fines content ranging from 10 to 20 percent. The soils are generally medium dense to very dense, and firm to hard. No groundwater was encountered in any of the borings within the depths explored, and moisture contents of the soils were generally low.

9.2 PAVEMENT DESIGN

URS developed preliminary pavement designs for the proposed project alignment using geotechnical subgrade data generated during a limited field and laboratory investigation program, and using guidelines published by ADOT-Materials in the Preliminary Engineering and Design Manual (PEDM) and MCDOT Roadway Design Manual (RDM) – Chapter 10: Pavement Design Guide. These designs pertain to construction of new pavements for the project and do not address the existing pavement structure or any interim pavement rehabilitation projects that may be constructed by MCDOT. Several different pavement configurations were evaluated. Details of this evaluation process are included in the *URS, Preliminary Geotechnical and Pavement Report, SR 303L, 1-10 to US-60* submitted under a separate cover. Key points from this report are summarized below.

Preliminary Design Traffic (18-kip ESALs)

Traffic forecasts are presented in *Traffic Report, SR 303L, I-10 to US-60, dated January, 2006,* for the 2030 Design Year. These data were used to estimate design traffic loads for a design life of 20 years from 2015 to 2035. A truck volume of 10 percent of the total traffic was estimated for the future years and utilized in the pavement design calculations. The Equivalent 18-kip Single Axle Loads (ESALs) computed at different intersections of the project alignment are presented in Table 9-1:

Table 9-1 Design Traffic Loads

Location	Maximum Average Daily Traffic Volume*	Lane Distribution Factor	Estimated Rigid One-Way 18-kip ESALs	Estimated Flexible One-Way 18-kip ESALs
SR 303L Mainline	76,800 (One-way)	0.7	129,198,700	178,026,700
SR 303L Service Interchange Ramps	10,500 (One-way)	1.0	22,080,900	30,425,900
SR 303L/I-10 TI Ramps	27,900 (One-way)	0.8	46,935,300	64,673,500
SR 303L/I-10 Frontage Roads	21,400 (One-way)	0.9	40,502,000	55,809,000
SR 303L/Northern Parkway Ramps	14,200 (One-way)	1.0	29,861,000	41,146,300
SR 303L/Northern Parkway Frontage Roads	11,200 (One-way)	0.9	21,197,300	29,208,400
SR 303L/US-60 TI Ramps	33,400 (One-way)	0.8	54,762,300	75,458,600

* Maximum ADT Volumes for Year 2030.

The design traffic loads have been adjusted from total ESALs by applying the appropriate lane distribution according to the roadway lane configurations, as shown in Table 9-1. The two-way mainline design traffic loads have been adjusted by a directional distribution factor of 0.5 to obtain the one-way design 18-kip ESALs.

Preliminary Design R-value

Limited field investigations provided an assessment of the subgrade along the proposed alignment for SR 303L. Relatively shallow (5-foot) borings were drilled at approximately one-mile centers for this purpose. Sieve analyses and Atterberg Limits tests were performed on bulk samples collected from these borings. These data were used to estimate R-values based on guidelines provided in the PEDM. In addition, two laboratory R-value tests were also performed.

The correlated R-values ranged from 16 to 44 while the two laboratory tests yielded R-values of 20 and 25. An average R-value of approximately 27 was estimated by combining the correlated and tested R-values in accordance with a relationship provided in the PEDM. However a design R-value of approximately 20 was

utilized for the preliminary evaluation of pavement structures at or below existing grade, to account for the potential presence of poor subgrade soils.

The soils that exhibit R-values of less than 20 provide poor subgrade support. Further, these soils, may be expansive under some moisture and loading conditions, and may be corrosive to ferrous metals. Therefore, these soils could necessitate the utilization of reinforcement using geogrids or geosynthetics and/or comprehensive earthwork operations including but not limited to over excavation and/or remove and replace. It is recommended that a detailed geotechnical evaluation consisting of subsurface exploration and laboratory testing, be conducted to better evaluate and delineate these poor subgrade soils.

Pavement Design

The preliminary pavement design presented herein was performed in general accordance with PEDM pavement design procedures using traffic load and subgrade R-value discussed above as input parameters. The following additional pavement design parameters were used to develop pavement structural sections. Consideration was given to non-reinforced and jointed reinforced Portland Cement Concrete Pavement (PCCP) for the various locations throughout the SR 303L corridor, and also for asphaltic concrete at frontage roads.

- Pavement Design Life = 20 years.
- Reliability (Z_R) = -2.327 for a level of reliability of 99%.
- Combined Standard Error of the traffic prediction and performance prediction (S_0) = 0.25 for rigid pavements and 0.35 for flexible pavements, in accordance with current ADOT practice.
- Initial Design Serviceability Index (P_O) = 4.2, as determined from Table 202.02-2 of the PEDM.
- Design Terminal Serviceability Index (Pt) = 3.0, as determined from Table 202.02-2 of the PEDM.
- $\Delta PSI = P_O - Pt = 1.2$
- Seasonal Variation Factor for the project area = 1.0.
- Resilient modulus = 12,000 psi, as derived from the design R-value of 20.
- Loss of support = 0.
- Average modulus of rupture for Portland cement concrete (PCC) (S'_C) = 670 psi, compressive strength of PCC (f'_C) = 5,000 psi, and modulus of elasticity of PCC (E_C) = 4,000,000 psi.
- Modulus of Subgrade Reaction (k) = 619 pci as derived from Table 202.03-2 and Figures 202.03-1 of the PEDM.
- Load Transfer Coefficient (J) = 3.9 for plain jointed PCCP and 2.8 for jointed reinforced (doweled) PCCP according to Table 202.03-1 of the PEDM.

A summary of the recommended pavement sections for the various roadway elements is presented in Table 9-2.

Table 9-2 Preliminary Rigid Pavement Structural Sections

Roadway Element	Plain Jointed PCCP (inches)	Jointed Reinforced (Dowels) PCCP (inches)	Aggregate Base (AB) (inches)
SR 303L Mainline	0	14.5	4
SR 303L Service Interchange Ramps	13.5	0	4
SR 303L/I-10 TI Ramps	0	12.5	4
SR 303L/I-10 TI Frontage Roads	0	12.5	4
Northern Parkway TI Ramps	14	0	4
Northern Parkway TI Frontage Roads	13.5	0	4
SR 303L/US 60 TI Ramps	0	13	4

The thicknesses of the preliminary pavement sections presented above are significantly thicker than those normally constructed locally. These depths are a result of the low preliminary design R-value of 20. Therefore, additional geotechnical investigation is recommended to further evaluate the pavement subgrade, which may provide sufficient information to allow reduced pavement thicknesses to be used for the design of pavements for this project.

We recommend using AC (base mix) in lieu of AB in depressed and/or at grade locations for the rigid pavements constructed for this project.

If flexible pavements are used at the frontage roads, the preliminary pavement structural sections shown in Table 9-3 are recommended. Structural Number (SN) of 7.14 and 6.64 were computed for flexible pavements of SR 303L and I-10 TI frontage Roads and Northern Parkway TI Frontage Roads, respectively. These structural numbers were computed using structural coefficients of 0.44 and 0.14 for asphaltic concrete and aggregate base, respectively.

Table 9-3 Preliminary Flexible Pavement Structural Sections

Roadway Element	Asphaltic Concrete (inches)	Aggregate Base (Class 2) (inches)
SR 303L/I-10 TI Frontage Roads	10	15
Northern Parkway TI Frontage Roads	9	15

It is recommended that the rigid pavement sections presented in Table 9-2 include a one-inch-thick overlay of rubberized asphaltic concrete. Consideration should also be given to using rubberized asphaltic concrete in the upper one-inch of the flexible pavement sections presented in Table 9-3.

9.3 SLOPE DESIGN

Temporary slopes less than 20 feet deep should be excavated in accordance with OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P. In accordance with these standards, undisturbed native soils and existing fill should be considered "Type C" soils for preliminary evaluation, and excavations that are less than 20 feet deep should be constructed at maximum allowable unshored slopes of 1.5H:1V in these soils. Should steeper slopes be required due to the proximity of existing structures or other obstruction, a geotechnical engineer should evaluate the stability of the required slope by further testing and classifying the soils.

Heavy excavation equipment should be maintained at least ten feet away from the edge of any excavation, and the perimeter of all temporary excavations should be protected against surface water runoff to prevent raveling and erosion into the excavation.

Unprotected permanent cut and fill slopes should be constructed no steeper than 3H:1V. Permanent cut and fill slopes protected by erosion control measures, such as through the establishment of vegetative cover, can be constructed as steep as 2H:1V and 2.5H:1V, respectively.

9.4 CHANNEL DESIGN

Earthwork for open channels should be performed in accordance with Section 215 of the MAG Uniform Standard Specifications for Public Works Construction. Near-surface soils encountered during the DCR geotechnical investigation were typically medium dense clayey sand to stiff sandy clay. For preliminary evaluation, it may be assumed that channel excavations could be made so as to place concrete directly against the excavated surfaces provided that maximum unsupported side slopes of 2H:1V are constructed. It is anticipated that excavation along the project corridor will be accomplished with conventional equipment based on the soil conditions encountered.

Channel joints should be constructed so that they are water tight to prevent potential soil volume changes due to increased moisture content. To evaluate the potential for moisture induced soil volume change the following geotechnical laboratory testing procedures are recommended during the design phase:

- Collect relatively undisturbed near-surface soil samples and perform response to wetting tests (hydro consolidation) tests to evaluate volume change characteristics for the channel subgrade soils.
- Laboratory testing should be performed to determine the dispersive characteristics of clayey soils encountered along channel alignments.

9.5 SPECIAL GEOTECHNICAL ENGINEERING ISSUES AND GOALS

The proposed SR 303L alignment traverses an area of known and documented ground subsidence and earth fissuring. In fact, the greatest measured ground subsidence in Arizona (approximately 18 feet between 1957 and 1991) has been documented at Luke AFB, located 2 to 3 miles east of the SR 303L alignment. The proposed SR 303L alignment crosses the Luke AFB subsidence "bowl" or "trough." Furthermore, a documented earth fissure extends from (approximately) the intersection of Northern Avenue and Cotton Lane to the intersection of Peoria Avenue and Sarival Avenue and apparently intersects the SR 303L alignment at Olive Avenue.

In light of the documented ground subsidence and earth fissures in the vicinity of SR 303L, URS performed a preliminary assessment of these geologic phenomena. The scope of work within this DCR was limited to review of published information on ground subsidence and fissuring that could potentially impact the proposed construction. A summary of discussions and preliminary recommendations related to ground subsidence and earth fissures is provided in this report. Additional details are included in the Preliminary Geotechnical Assessment Report to be submitted under a separate cover.

The project alignment is located in the West Salt River Valley. This is an area of known ground water decline and associated ground subsidence and earth fissuring. In 1995, the United States Geologic Survey (USGS) in cooperation with the United States Air Force published an administrative report entitled *Investigation of Hydrogeology, Land Subsidence, and Earth Fissures, Luke Air Force Base, Arizona* (Study), by Herbert H. Schumann and Christie M. O'Day of the USGS. This report provides a comprehensive description of the hydrogeology and ground subsidence issues around Luke AFB. The following paragraphs related to ground subsidence and earth fissuring in the vicinity of the SR 303L alignment were largely summarized from the above-mentioned reference.

The Study indicates that between 1952 and 1977, groundwater levels in the vicinity of Luke AFB declined by as much as 300 feet. Between 1976 and 1983, the groundwater decline was generally arrested, and water levels rose by as much as 50 feet in some wells. However, the water levels remained well below their highs prior to large-scale pumping, and groundwater levels measured in 1991 were approximated those measured in 1977. The overall decline in groundwater levels has resulted in ground subsidence and the development of earth fissures.

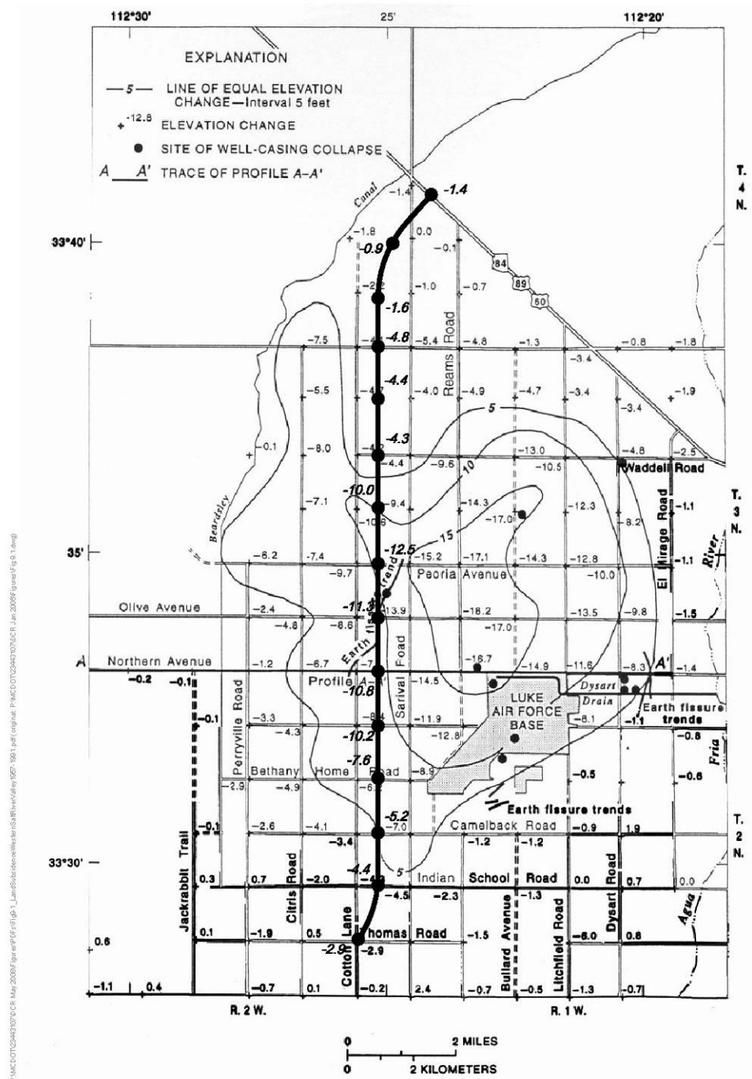
Figure 9-1 is taken from the report by Schumann and O'Day discussed earlier in this section. The figure shows estimates of ground subsidence between 1957 and 1991. These estimates were made by comparing elevation data from historic topographic quadrangles published by the USGS. Figure 9-1 shows that the maximum ground subsidence estimated for the area is approximately 18 feet and was measured at the intersection of Olive Avenue and Reems Road, approximately 1 1/2 miles east of the SR 303L alignment. This is the highest ground subsidence recorded in Arizona. Data from Figure 9-1 were interpolated to estimate ground subsidence along the SR 303L alignment, and a profile of ground subsidence along SR 303L is shown in Figure 9-2. The profile depicted in Figure 9-2 indicates that the greatest magnitude of ground subsidence along SR 303L occurs between Bethany Home Road and Waddell Road and generally tapers to significantly smaller levels toward the north and south termini of the alignment.

Figure 9-1 also depicts three zones of documented fissuring. The first zone is located east of Luke AFB along Northern Avenue between Dysart Road and El Mirage Road; a second zone is north of Camelback Road due south of Luke AFB; and a third zone is along Olive Avenue between Sarival Avenue and Cotton Lane. The third zone discussed above directly impacts SR 303L as it intersects the alignment at Olive Avenue. The final design phase should include a thorough assessment of the alignment for potential zones of fissuring. This is particularly critical because a grade separation structure (bridge) is planned for the Olive Avenue intersection.

Recommended Future Activities

The scope of work for this DCR was limited to general discussions on ground subsidence and earth fissuring along the SR 303L alignment, based on published information. As discussed in the preceding paragraphs of this section, SR 303L is located within a zone of significant, documented ground subsidence and earth fissuring. Careful and detailed assessment of these two geologic hazards and their impacts on the development of the proposed project should be a critical component of the final design phase. The final design phase of the project should include (but not be limited to) the following components:

- Obtain groundwater level information and prepare well hydrographs to assess groundwater trends post-1991.
- Evaluate estimates of future population growth in area and assess to the extent practical, the impact of projected water demand on future subsidence and earth fissure development.
- Contact and obtain (where possible) additional benchmark survey data in the project area in order to develop more realistic estimates of projected ground subsidence.
- Obtain and examine interferograms to assess magnitudes and patterns of subsidence along the alignment. The FCDMC is using this technique extensively to evaluate subsidence at flood control dams in the County.
- Examine aerial photographs to identify and study fissures in the area.
- Perform a detailed site reconnaissance to identify earth fissures.
- Perform low-sun angle aerial photography and attempt to identify fissure traces.
- Perform soil-structure interaction studies to evaluate the impact of ground subsidence and earth fissuring on the proposed construction.



Reference: Base map from U.S. Department of the Interior, U.S.G.S., Administrative Report; Investigation of Hydrogeology, Land Subsidence, and Earth Fissures, Luke Air Force Base, Arizona, 1995.

Figure 9-1 Land Subsidence in Part of the Western Salt River Valley, 1957-1991

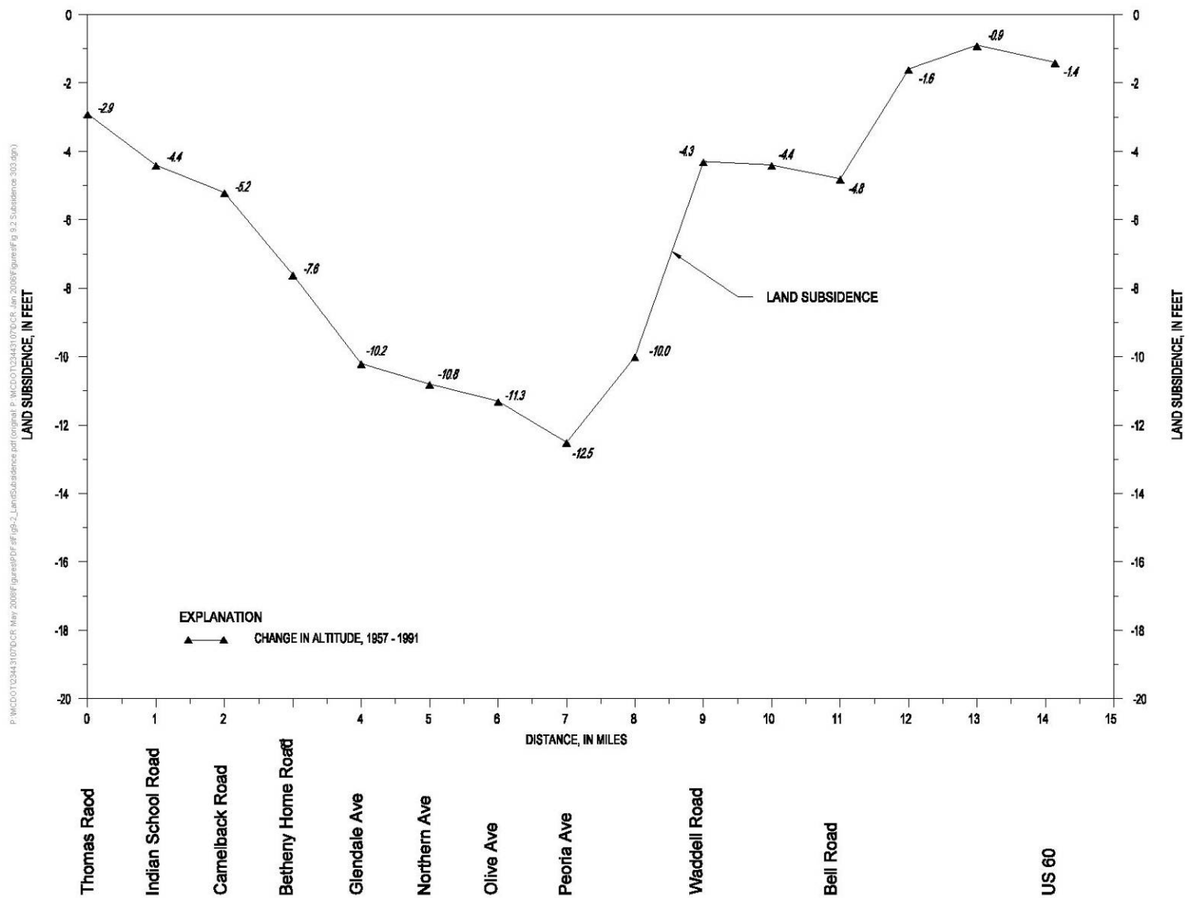


Figure 9-2 Land Subsidence Along Loop 303