Route 12 Sacramento River
Crossing at Rio Vista Preliminary
Geotechnical Engineering Review
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ROUTE 12

SACRAMENTO RIVER CROSSING
AT
RIO VISTA

PRELIMINARY GEOTECHNICAL ENGINEERING REVIEW

Caltrans

Prepared For:
City of Rio Vista
Rio Vista, California

Caltrans District 10
Stockton, California

Prepared by:
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Date:
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PRELIMINARY GEOTECHNICAL ENGINEERING REVIEW
SR-12/Sacramento River Crossing

Centennial Engineering Inc.

1P2/391/106
(38121-B6-120N:160W)

August 1993

Taber Consultants
Engineers and Geologists

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Preliminary Geotechnical Engineering Review
SR-12/Sacramento River Crossing
Rio Vista, California

1P2/391/106

Introduction

In accordance with our agreement with Centennial Civil Engineers, Inc. we have completed a preliminary geotechnical engineering review of the above site. The purpose of study is to develop limited available soils and geologic data into criteria for use in preparation of preliminary planning/evaluation of proposed alignments and bridge replacement alternates. The data and conclusions contained in this report should not be construed as criteria for use in developing project design past the general planning stage.

The primary element of study for this project consists of obtaining and review/evaluation of available data -- including published topographic and geologic mapping, test boring logs from such geotechnical reports as are available, plans/data for existing road/bridge projects and air-photos. A list of selected references is attached.

The discussion of geotechnical conditions, constraints and requirements concluding this report is based on preliminary project plan/profile sheets and specific project design/evaluation concerns generated in discussions with other design engineers. The available data appears to provide a reasonable picture of the range of earth materials and conditions likely to be encountered during design/construction of the proposed project. Along with a selected project alternate and defined structure type, support locations, etc., the data developed can provide a basis for generating the scope of an appropriate design geotechnical engineering study.

Site and Project Description

Original site topography is shown on the USGS "Rio Vista, California", "Isleton, California" and "Birds Landing, California" 7½' quadrangles (1952-3; photo-revised 1968). In general, the existing/proposed SR-12 alignment(s) crosses gentle/moderate terrain with numerous swales/drainages in the Montezuma Hills area west of Rio Vista from the intersection of SR-113 (west project boundary) at/near elev. 150 to the Sacramento River crossing at/near elev. 10. From the Sacramento River crossing eastward, topography is generally flat-lying and depressed between elev.-10 and elev.-15 to the Mokelumne River (east project boundary). On Brannon Island, existing SR-12 crosses Tomato Slough and Jackson Slough about one mile and three miles, respectively, east of the Sacramento River crossing.

Two alternate bridge/roadway realignments are being considered after elimination of several others in earlier stages of this study -- Alternates 2 and 6 as illustrated on available site topography on preliminary "Plan and Profiles" drawings received April 1, 1993 and prepared by Centennial Civil Engineers, Inc. The lengths of alternate structures range from 4250±ft to 5715±ft; deck levels range from 60±ft to 175±ft above the river channel water surface.

Associated roadwork as part of this project involves up to 33,400±l.f. of new roadway west of proposed alternate river crossings and up to 27,400±l.f. of new roadway east of proposed river crossings. Significant roadway cuts/fills in the Montezuma Hills area appear to be on order of 30-40±ft.

Geologic Setting

The project site is located within the southwestern Sacramento County and southeastern Solano County portions of the Great Interior Valley of California. The Great Valley is a gently-sloping to flat-lying area between the Coast Ranges on the west and the Sierra-Nevada on the east. The valley surface is underlain by a sequence of sediments up to several thousand feet thick. Depth to bedrock within the proposed project limits is on order of 1800-ft (USGS-OFR 82-737). The lateral distribution of surface geologic units is shown on attached Figure-1, along with superimposed project alternate alignments.

The dominant geologic unit west of the Sacramento River is composed of deposits associated with the Montezuma Formation (map unit Qmz, see Figure-1). This formation generally consists of alluvial deposits of poorly stratified, poorly consolidated gravels, sands and clays. Older alluvium (Qom) of the Montezuma Hills and vicinity forms slightly to moderately dissected fans on the northeast flank of the Montezuma Hills. Minor amounts of younger alluvium (Qym) of the Montezuma Hills are present but are very limited in extent and lie in and adjacent to presently active streams.

Natural deposits along the banks of the Sacramento River and eastward are associated with more recent near surface deposits of tidal wetlands and waterways (Qpm) and natural levees (Ql). These deposits are characterized by river flood plains and channels which lie along the Sacramento and San Joaquin Rivers. Materials may vary considerably in grain size from coarse sands along former stream channels to fine-grained silts and clays along the outer edges of flood plains. Interlayered to concentrated deposits of peat are typical of more recent materials in low-lying areas of the Delta and commonly are the material which has naturally backfilled old low-velocity stream channels. Normally consolidated fine-grained materials associated with Delta deposits generally have limited capability in direct bearing and are significantly compressible under more than light incremental loading. Peat, where present, is generally very weak and <u>very compressible</u> under incremental loading.

In the early 1900's hydraulic dredge soils (Qds) were deposited in an attempt to widen, straighten, and/or deepen the Sacramento and San Joaquin Rivers. Man-made hydraulic dredge soils consist of sand and subordinate silt, clay and peat. Within the project boundaries/alignments, these materials are shown to be banked against the base of the Montezuma Hills along the river southwest of Rio Vista and also extend to the north of Rio Vista as far as the new airport (approximately 2 miles). Man-made hydraulic dredge soils are generally considered unreliable for direct structure support, but may be capable of providing support for light superposed loads.

Older alluvial deposits underlying recent Sacramento River deposits and tidal wetlands (Ql and Qpm) within the limits of the proposed project are associated with Great Valley and/or Montezuma Hills sediments and typically consist of flat lying interlayered gravels, sands, silts and clays. Materials within such deposits are typically normally to over-consolidated and are generally stronger (more competent) and significantly less compressible than overlying Delta deposits.

The nearest major, capable/active faults (per CDMG OFR 92-01 and 92-03) are indicated to be the Antioch fault located 12±miles southwesterly and the Coast Range-Sierra Nevada Block Boundary seismic source zone located about 8-miles southwesterly. The Midland fault is also shown on published mapping (USGS OFR 82-737) to cross beneath SR-12 1.6±miles east of the Sacramento River crossing. The Midland fault is a suspected potentially active fault.

While the project area is considered susceptible to significant ground shaking from events on nearby faults, it is not in an Alquist-Priolo Special Study Zone and the potential for fault-rupture is considered remote. Secondary seismic effects typically associated with loose/weak sands/fills are liquefaction, rapid densification, lateral spreading and ground lurching.

There are no historical records of damage resulting from earthquakes in Rio Vista. However, historic ground failure associated with the 1906 San Francisco Earthquake has been recorded to have occurred at locations west and southeast of the current location of the Rio Vista river crossing. Approximately 19 miles west of the existing river crossing, ground settlement of up to 11±ft was recorded/observed along the Southern Pacific Rail Road tracks. Ground settlement and lateral spreading of up to 3±ft was experienced 20± miles southeast at the Santa Fe Rail Road Bridge across the Middle River between Point Richmond and Stockton.

Soils Profiles

A list of available sources for soils boring data is attached. Sources include Caltrans studies for the existing bridge, SR 12 and SR 160 roadway projects and several studies for other facilities scattered irregularly over the project vicinity.

Based on review of the geologic and soils data, "characteristic" soils profiles have been developed for various project elements/areas. These soil profiles are believed to present a reasonable picture of controlling soil conditions. However, the categories are very broad and details emerging from design studies will vary significantly from place to place.

For project elements located west of the Sacramento River and west of downtown Rio Vista, soils may be generally characterized as compact-very stiff gravel, sand, silt and clay. Locally along channels and flat areas, some soft-stiff clay and/or loose sand/gravel may be present to shallow depths. In general, soils in this area are expected to be capable of supporting moderate to heavy foundation and fill loads.

At/near the west approach to the existing bridge in Rio Vista, there is as much as 30-40±ft of peaty soils overlying more competent "mineral" soils (sand/silt/clay). The thickness of the peat soils is expected to thin to the west and south with increasing distance from the axis of a buried tributary channel. Underlying mineral soils are expected to be capable of generating support for heavy, concentrated foundation loads. Peat soils are weak and highly compressible even under very limited incremental loading.

In the river, a characteristic profile (based on exploration by Caltrans for the existing bridge) consists of mud and loose sand to elev. -55± (30±ft below low channel grade) underlain by compact-very stiff sand and clay to elev. -90±, in turn underlain by very dense sand and very hard clay (to maximum depth of Caltrans exploration = elev.-110±). The underlying soils are expected to be capable of generating support for heavy concentrated foundation loads with a significant contribution by all soils below elev.-55±. Mud and loose sand above elev.-55 are interpreted as very recent alluvium/bedload; they are potentially subject to scour and are not considered suitable or reliable for supporting any structure foundation loads.

East of the Sacramento River, a typical soils profile is expected to consist of 15-25±ft of weak, compressible peaty soils overlying another 5-10±ft of soft clay grading downward to compact-dense sand and/or very stiff-hard clay. Peat soils are very weak and highly compressible, although a shallow "crust" (say, 2-3±ft thick) is common at the surface owing to oxidation and desiccation of the organic soil. Soft clay immediately below the peat is expected to be at least moderately compressible and to have limited shear strength. Soils at greater depth are expected to be capable of supporting moderate to heavy directly applied and/or superposed fill/foundation loads without distress.

West of the river, free groundwater is expected to be present along swales at least seasonally. Other more erratic occurrences cannot be precluded but are not expected to have significant impact on project design or construction. East of the river and near the west approach to Alt. 2 bridge, free groundwater is expected at shallow depth and will be a significant factor in any earthwork or excavations made below existing ground surface.

<u>Site Seismic Conditions</u>

In accordance with current Caltrans Office of Structures Design site seismicity evaluation procedures "maximum credible rock acceleration" 0.32g is assigned the site associated with a controlling event of 7.0 magnitude on the Coast Range-Sierra Nevada Block Boundary seismic source zone located 8±miles southwesterly. From preliminary site data, depth to "rock-like" material is considered very likely to exceed 150-ft. Other than the distortion and local instability of embankment/levee fills/slopes and potential for liquefaction in loose sands, the risk of secondary seismic effects appears low.

Discussion

In general, materials anticipated to be encountered along proposed individual alignments and different alternates vary significantly and are expected to affect the alternates/alignments differently. Typical surficial materials are not considered suitable for direct support of bridge foundations but may be appropriate for support of limited embankment or roadway fill loading. Consideration is expected to be required for surficial soils compressibility/settlement under any significantly increased fill/embankment loading.

This study is very broad and preliminary and suitable only for use in preliminary planning of alignments/general schemes. Subsurface investigation directly linked to structure requirements will be required for use in setting design and construction criteria for foundations. The following discussion is keyed to "typical" soils profiles indicated above.

Bridge Foundations

Older deposits/materials at depth are expected to be capable of supporting heavy concentrated foundation loads without distress. It is anticipated that structure support may be achieved in such materials by means of driven piling achieving bearing through side friction and end bearing.

Soil conditions are consistent with the use of driven pile foundations penetrating through weak compressible surficial soils and/or channel bedload into relatively competent underlying soils. On land, the depth of very weak peaty soils varies form 30-40±ft in the vicinity of the west approach to the existing bridge (Alt. 2) and to 15-25±ft east of the Sacramento River; soft soils are likely absent at the west end of Alt. 6 bridge. Borings made for the existing bridge indicate the base of muck and loose sand in the channel at elev. -55±. Substantial and significant variations in soil profiles are likely to be encountered in design studies.

Preliminary discussions indicate that desired pile loadings/diameters for bridge foundations will include 100 ton/16±inch, 250 ton/30-inch, 300 ton/54-inch and 500 ton/54-inch. The 100 ton piling will tentatively consist of 14-inch square pre-cast concrete or 16-inch diameter steel pipe piling; CIDH piling may be suitable at the west abutment Alt. 6. The larger diameter piles are expected to be pre-stressed concrete tubes driven open-ended. Pile footings in the channel will tentatively be constructed at water level with piles acting as columns. Pile footings on land are expected to be embedded in the ground.

Based on available soils data, it appears that desired pile types can be driven to bearing, but jetting or drilling through cylindrical piles to aid in achieving required penetration may be necessary. Based on assumed "typical" soil profiles, estimated pile tips for most of these pile types is to elev. -110± (say, elev. -100± for 30-inch/300 ton piling). About 40-ft of original ground penetration is expected to be required for 16-inch CIDH 100 ton piling at Abutment-1/Alt. 6. With the exception of lateral loading, such piles are expected to be suitable for typical (Caltrans) service load combinations.

Settlement from embankment loading may be substantial and may apply significant incremental loading to foundation piling at abutments through "negative skin friction". Pre-drilling for pile foundations, driving piles to more than nominal bearing capacity to allow for negative skin friction and the use of approach slabs are methods which may also be used to accommodate settlement and associated loading conditions.

Piles with ground penetrations per above are expected to be "fully embedded" with respect to lateral loading. The effective column height of piling for piers in the channel will be on the order of 55-70±ft. Soft peaty soils are not expected to provide more than very limited lateral restraint and effective column heights for pier footings east of the river and west of the river for Alt. 2 may be as much as 40±ft. Piles at abutments (in embankment fills) are expected to have typical "firm" soil response to lateral loading, although the seismic response and stability of abutment fills founded on peat may not be acceptable.

Allowable loading and specified tip elevations of non-standard and/or large diameter piling will likely be controlled by settlement. Allowance should be made in schedule and budget for static load testing of selected initial piles.

Soil conditions in the channel are also likely to be suitable for construction similar to existing bridge -- i.e. piling driven with followers within sheet-pile cofferdam, tremie seal and de-watering, etc. However, future channel dredging may render this approach much more difficult owing to greater pier footing depth required. Soil support is also likely available for other approaches to foundation design/construction (e.g. "floating" caissons, caissons bearing as "footing"); additional comment can be provided on such approaches, but owing to their relative sensitivity to local soil conditions and to the uniformity of soil conditions across the pier areas, the present limitations in available soils data may be critical.

<u>Embankment Fills at Bridge Approaches</u>

Embankment approaches to the west end of Alt. 2 and to the east ends of both alternates will be constructed in areas with significant depths of very soft, weak, compressible peaty soils. It is not expected that any new embankment or foundations will be constructed in or adjacent to existing river levees; such construction would be a critical consideration. Previous construction for improvements to SR 160 and the SR 160/SR 12 intersection have incorporated provisions for wick drains, controlled loading rate (1±ft/week) and light-weight (wood-chip) embankment fill material.

Assuming no modification of fill foundation in areas of peaty soils, estimated settlement for various fill heights ranges 1±ft for 5-ft fill to 6±ft (or more) for 50-ft high fill. With the use of wick drains (and possibly surcharging), the time required for settlement can likely be kept to less than 180-days. Stability of embankment foundations is also a concern and slopes flatter than 2:1 may be required.

At either of the proposed east approach embankments, there can be sufficient horizontal clearance available to allow excavation of peat soils and replacement with engineered fill. Such a procedure would substantially reduce concerns regarding residual settlement/negative skin friction on piling, embankment stability and an extended construction/waiting period (although would not expect to entirely eliminate required waiting period. Full depth excavation of peat and replacement is recommended (for planning) within 150±ft of bridge abutments. Similar effects might be achieved with other ground improvement strategies (e.g. stone columns), but their effectiveness is difficult to assess within the scope of this study.

It is expected that, away from abutments, embankment construction to, say 25±ft high, will be generally feasible without special fill foundation preparation; the use of wick drains and sand blanket to accelerate settlement, light-weight fills, fill reinforcement, flatter side slopes and/or controlled construction/loading rate may be necessary to maintain reasonable construction stability. The risk of local embankment foundation failures would be an element of such work and significant post-construction settlement and residual differential settlement can be anticipated – particularly across buried peat channels.

The west abutment/embankment of Alt. 2 bridge will be in town, where settlement could have highly detrimental effects on underground utilities or buildings in the immediate vicinity. A specific approach to constructing embankment in this environment will depend on local soil conditions, clearances to structures/utilities, etc. For planning purposes, it may be reasonable to assume relocation of all buried utilities outside of embankment areas and careful monitoring of nearby structures as minimum construction requirements. The use of pile supported retaining walls, light-weight fills, mechanically stabilized embankments, ground improvement techniques (e.g. stone columns) could also be considered.

Roadway Construction

West of the river, cut and fill slopes as steep as 1½:1 are likely typically feasible -- although imported fill material requirements/availability and/or the desire to generate additional local borrow may indicate the desirability for flatter side slopes. Local soil conditions may dictate flatter slopes, but not likely flatter than 2:1. Fill foundation preparation requirements are expected to be nominal in this area, although the need for local overexcavation of soft soils from swale bottoms cannot be precluded.

East of the river, the great majority of new roadway construction is expected to have finished grade 3-5±ft above existing ground levels. Minimum embankment over original ground in this area is about 3½-ft to allow for a 1±ft thick fill foundation layer and 2½-ft of pavement structural section. Even with such low fills, settlements and differential settlements on the order of ½-1±ft can be expected. However, the use of geogrid reinforcement of earth fills could reduce the magnitude of differential settlements, particularly across any identified swale or buried peat channel areas. Fill foundation preparation is expected to be limited to scarification and compaction of surface soils.

Pavement structural sections for most of the project will depend on the quality of imported fill. Selective grading may be feasible west of the river if relatively high quality subgrade materials are identified in proposed cut sections. Subdrainage at/below subgrade level is not expected to be a general requirement.

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Attachments: "Selected References"
"Soil Boring Data Sources"
"Geologic/Alignment Map" (Figure-1)