REGULATORY INFORMATION DISTRIBUTION SYSTEM (RIDS)

ACCESSION NBR:8008070430 DOC.DATE: 80/08/05 NOTARIZED: NO DOCKET #
FACIL:50-361 San Onofre Nuclear Station, Unit 2, Southern Californ 05000361
50-362 San Onofre Nuclear Station, Unit 3, Southern Californ 05000362
AUTH.NAME AUTHOR AFFILIATION
BASKIN,K.P. Southern California Edison Co.
RECIP.NAME RECIPIENT AFFILIATION
SCHWENCER,A.A. Licensing Branch 3

SUBJECT: Forwards response to NRC 800722 request for addl info concerning design calculations for Unit 3 electrical tunnel. Info suppls 800121 calculations.

DISTRIBUTION CODE: B001S COPIES RECEIVED:LTR 1 ENCL 1 SIZE: 28
TITLE: PSAR/FSAR AMDTS and Related Correspondence

NOTES:Send all FSAR & ER amends to L Chandler. 05000361
~~Send all FSAR & ER amends to L Chandler.~~ 05000362

	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL
ACTION:	A/D LICENSNG	1	0	SCHWENCER,A.	1	0
	LEE,J.	1	0	ROOD,H. 05	1	1
INTERNAL:	ACCID EVAL BR	1	1	AUX SYS BR 18	1	1
	CHEM ENG BR	1	1	CONT SYS BR	1	1
	CORE PERF BR 17	1	1	DIR,SFTY TECH	1	1
	EFF TR SYS BR	1	1	EMERG PREP 22	1	0
	EQUIP QUAL BR	1	1	GEOSCIENCES	1	1
	HYD/GEO BR 11	1	1	I&C SYS BR 20	1	1
	I&E 06	3	3	MATL ENG BR	1	1
	MECH ENG BR	1	1	MPA	1	0
	NRC PDR 02	1	1	OELD	1	0
	POWER SYS BR	1	1	PROC/TST REV BR	1	1
	QA BR 10	1	1	RAD ASSESS BR12	1	1
	REAC SYS BR	1	1	REG FILE 01	1	1
	SIT ANAL BR 27	1	1	STRUCT ENG BR	1	1
EXTERNAL:	ACRS	16	16	LPDR 03	1	1
	NSIC 04	1	1			

AUG 8 1980

MA
4

60

TOTAL NUMBER OF COPIES REQUIRED: LTTR 50 ENCL 44

Southern California Edison Company

P. O. BOX 800
2244 WALNUT GROVE AVENUE
ROSEMEAD, CALIFORNIA 91770

K. P. BASKIN
MANAGER, NUCLEAR ENGINEERING
AND LICENSING

TELEPHONE
(213) 572-1401

August 5, 1980

Director of Nuclear Reactor Regulation
Attention: Mr. Albert A. Schwencer, Acting Branch Chief
Licensing Projects Branch 3, DPM
U. S. Nuclear Regulatory Commission
Washington, D. C.

Gentlemen:

Subject: Docket Nos. 50-361 and 50-362
San Onofre Nuclear Generating Station
Units 2 and 3

In a July 22, 1980 telephone conversation with SCE engineers, the NRC Structural Engineering Branch identified an additional question concerning the design calculations for the San Onofre Unit 3 electrical tunnel. Enclosed in response to this additional Structural Engineering Branch question are supplemental calculations to the original calculations which were submitted to the NRC by letter dated January 21, 1980.

If you have any questions concerning this matter, please let me know.

Very truly yours,

K. P. Baskin

Enclosure

B001
S
1/1

8008070430

A

Longitudinal Stress Analysis

The preceeding analysis and verification of the tunnel section to span over the 25 ft cavity considers only the transverse loading related to response from vertical earthquake and one horizontal earthquake applied in the direction transverse to tunnel axis. The longitudinal response due to soil motion derived from the passage of shear, compression and surface waves is calculated next.

An upper bound solution for the longitudinal response is derived from the interaction of the buried structure and soil subject to wave propagation at the various angles of incidence prescribed to maximize the response. The maximum longitudinal stresses due to axial and bending strains induced by the shear, compression and surface seismic waves are calculated and then combined with the flexural stresses derived from the transverse seismic loading. Such accounting of longitudinal stresses, while not exactly equivalent to the consideration of a second horizontal earthquake component along the longitudinal direction, it does give a conservative evaluation to satisfy the required 3-component earthquake analysis.

The maximum response for axial and flexural stresses are all combined by the SRSS method which is conservative since there is no definite basis for the total combination of stresses from different types of non-simultaneous responses, i.e: flexural response due to vertical and horizontal transverse earthquakes, and longitudinal response due imposition of axial and bending strains derived from non-concurrent shear, compression and surface seismic waves maximized at different angles of incidence.

CALCULATION SHEET

LAO 0513 8-73

SIGNATURE Alfredo Lopez DATE 7/22/80	CHECKED [illegible] DATE 7/31/80
	CALC. NO. C-260-5
PROJECT SONGS 2 & 3	JOB NO. 10079-003
SUBJECT Electrical Tunnel	SHEET 53 OF 78 SHEETS

The longitudinal stress analysis will be based on the formulations per Bechtel Seismic Topical Report BC-TOP-4A, rev. 4, section 6.0.

Strict application of the formulations implies complete anchorage of the buried structure into the soil, so that the structure undergoes loading resulting from the imposed soil strains. This type of loading is self-limiting upon achievement of the total soil deformation and clearly does not require the development of a resistance to carry an imposed load derived from gravity or seismic response. If the structure is flexible and sufficiently coupled to the soil it will track the soil motion resulting from the propagating seismic waves. The structure under consideration in this analysis is a rigid concrete box-section which will not necessarily comply with the motion unless sufficient anchorage exists.

The coupling or anchorage of the tunnel into the soil, as it is afforded by friction and by lateral soil bearing at ends and projecting elements of tunnel will be verified. If the available anchorage is not sufficient to develop the full axial thrust derived from the imposed strain, the axial thrust will be limited to the maximum possible from friction and soil bearing combined.

Differential movement of tunnel with respect to other connecting structures will be verified to be compatible with the flexible joints provided.

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/22/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 54 OF 78 SHEETS

Longitudinal stresses, maximum values (for differring angles of incidence for each type of wave and type of stress, per BC-TOP-4A, rev. 4)

1. compression wave

$$\sigma_{ap} = \pm \frac{E\, V_{mp}}{C_p} \quad \text{axial stress (for } \theta = 0°)$$

$$\sigma_{bp} = \pm\, 0.385 \frac{E R a_{mp}}{C_p^2} \quad \text{bending stress (for } \theta = 35° 16')$$

2. shear wave

$$\sigma_{as} = \pm \frac{E\, V_{ms}}{2 C_s} \quad (\text{for } \theta = 45°)$$

$$\sigma_{bs} = \pm \frac{E R a_{ms}}{C_s^2} \quad (\text{for } \theta = 0°)$$

3. surface shear wave

$$\sigma_{ar} = \pm \frac{E\, V_{mr}}{C_r} \quad (\text{for } \theta = 0°)$$

$$\sigma_{brc} = \pm\, 0.385 \frac{E R a_{mr}}{C_r^2} \quad (\text{for } \theta = 35° 16')$$

compressional component of wave

$$\sigma_{brs} = \pm \frac{E R a_{mr}}{C_r^2} \quad (\text{for } \theta = 0°)$$

shear component of wave

Longitudinal stresses (cont'd.)

Notation and numerical value for variables:

subscripts: a denotes axial
b " bending
p " compression
s " shear
r " surface shear wave
m " pertaining to ground particle

σ = stress

E = elastic modulus = 3.6×10^3 ksi; $f'_c = 4000$ psi concrete. (when compression on gross cross-section is considered)

= 29×10^3 ksi; $f_y = 60$ ksi steel reinforcing. (applicable when tension has exceeded cracking strength of concrete, the strain has been relaxed and net tension on rebar area becomes relevant)

v = ground particle velocity

Characteristic value of v_{ms} for sites with uniform soil, and maximum ground acceleration in the range of 1g, per Reference (1), sh 57, 100 cm/sec. For San Onofre, with 0.67 g, $v_{ms} = 67$ cm/sec is a conservative value. (adoption of this characteristic value is conservative when compared to the alternative of evaluating ground velocity by integration of the synthesized acceleration time-history record, which yields lower velocity values)

LAO 0513 8-73

CALCULATION SHEET

BECHTEL

SIGNATURE: Alfredo Lopez	DATE: 7/23/80
CHECKED: A.J.	DATE: 7/31/80
CALC. NO. C-260-5	
PROJECT: SONGS 2 & 3	JOB NO. 10079-003
SUBJECT: Electrical Tunnel	SHEET 56 OF 78 SHEETS

Longit. stresses (cont'd.)

$\therefore v_{ms} \approx v_{mr} = \frac{67}{2.54 \times 12} = 2.2$ ft/sec ($v_{mr} \approx v_{ms}$ is an accepted characteristic)

conservatively, in the absence of site specific data:

$$v_{mp} \approx \frac{v_{ms}}{2} = \frac{2.2}{2} = 1.1 \text{ fps}$$

C = wave propagation velocity

per Hall & Newmark:

$2000 \text{ fps} \leq C_s \leq 10\,000 \text{ fps}$ is the realistic range for shear wave velocity in the underlying competent soil.
It is emphasized that, unless rock or dense & stiff soils exist at shallow depths, the surface-measured wave propagation velocity is not applicable for this determination.

$C_s = 2500$ fps. is a representative value for San Onofre.
$C_r = C_s = 2500$ fps.

$$C_p = C_s \sqrt{\frac{2(1-\mu)}{1-2\mu}}$$ where: μ = poisson's ratio for underlying competent soil $= 0.35$

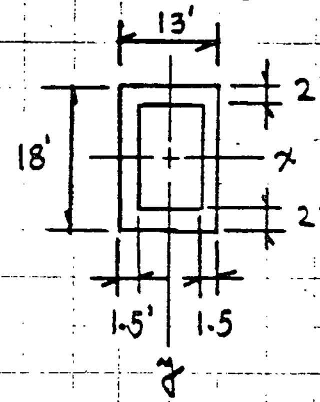
$$\therefore C_p = 2500 \overset{2.1}{\sqrt{\frac{2(1-.35)}{1-2(.35)}}} = 5250 \text{ fps.}$$

Longit. stresses (cont'd.)

a = ground particle acceleration, which appropriately can be taken as the maximum ground acceleration when evaluating maximum soil strain.

$$\therefore a_{mp} \approx a_{ms} \approx a_{mr} \approx (.67)\,32.2 = 22 \text{ ft/sec}^2 \quad \text{(for DBE)}$$

R = distance from neutral axis to extreme fiber of flexural cross-section of structure.



$R_x = 9 \text{ ft}$

$R_y = 6.5 \text{ ft}$

$A_c = 2(1.5 \times 18 + 2 \times 10) = 94 \text{ ft}^2$

$A_{s_{total}} = (.60)\,[2(18+10)] = 33.6 \text{ in}^2$ (in²/ft, sh. 24)

$I_x = \frac{13(18)^3}{12} - \frac{10(14)^3}{12} = 4030 \text{ ft}^4$

$I_y = \frac{18(13)^3}{12} - \frac{14(10)^3}{12} = 2130 \text{ ft}^4$

θ = angle of incidence of propagating wave (not used quantitatively, noted previously next to each stress formulation just to emphasize its variation when maximizing value for each type of stress and wave is considered)

Reference (1): Characteristics of Earthquake Ground Motions; Idress, I. Proceedings of the ASCE Geotechnical Division Specialty Conference, Vol. 3, June 1978, Pasadena, CA.

LAO 0513 8-73

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/23/80 CHECKED F.J. DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 58 OF 78 SHEETS

Longit. stresses (cont'd.)

Calculation of axial stresses due to wave propagation

$$\sigma_{ap} = 3.6\times10^3 \underbrace{\left(\frac{1.1}{5250}\right)}_{.00021} = \pm 0.8 \text{ ksi}$$

- If in compression, it is a moderate stress level for concrete.
- If in tension, it is higher than $f_r \approx 7.5\sqrt{f'_c} = .47$ ksi, therefore concrete strain is relaxed in cracked section and tensile stress on rebar area applies, see sh. 60 & 65

reinforcing steel stress: $f_{s_{ap}} = 29\times10^3(.00021) = 6.1$ ksi

$$\sigma_{as} = 3.6\times10^3 \underbrace{\left(\frac{2.2}{2\times2500}\right)}_{.00044} = \pm 1.6 \text{ ksi}$$

rather high compressive stress, see sheet 60 for evaluation and consideration of maximum total axial loads possible.

$$f_{s_{as}} = 29\times10^3(.00044) = 12.8 \text{ ksi}$$

$$\sigma_{ar} = 3.6\times10^3 \underbrace{\left(\frac{2.2}{2500}\right)}_{.00088} = \pm 3.2 \text{ ksi}$$

same as for σ_{as}, see sheet 60

$$f_{s_{ar}} = 29\times10^3(.00088) = 25.5 \text{ ksi}$$

by SRSS from three types of waves:

$$\sigma_a = (0.8^2 + 1.6^2 + 3.2^2)^{1/2} = 3.7 \text{ ksi}$$

$$f_{sa} = (6.1^2 + 12.8^2 + 25.5^2)^{1/2} = 29.2 \text{ ksi}$$

see sh. 60, 65 & 72 for verification and evaluation.

LAO 0513 8-73

CALCULATION SHEET

BECHTEL

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/23/80 CHECKED [initials] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 59 OF 78 SHEETS

Longit. stresses (cont'd.)

Calculation of bending stresses due to wave propagation

$$\sigma_{bp_x} = 0.385 \times \frac{3.6\times10^3 (9) \overset{\text{sh. 57}}{(22)}}{(5250)^2} = \pm .010 \text{ ksi}$$

low, not governing ∴ no need to calc. σ_{bp_y}

$$\sigma_{bs_x} = 3.6\times10^3 \times \underbrace{\frac{(9)(22)}{(2500)^2}}_{3.17\times10^{-5}} = \pm .114 \text{ ksi}$$

$$\sigma_{brc} = 0.385 \ (3.6\times10^3)(3.17\times10^{-5}) = \pm .044 \text{ ksi}$$

$$\sigma_{brs} = \pm .114 \text{ ksi}$$

$$\sigma_b = (.010^2 + .114^2 + .044^2 + .114^2)^{1/2} = .17 \text{ ksi} \ll .85 f'_c = 3.4 \text{ ksi}$$

∴ the combined bending stress in concrete is of distinct low order of magnitude; nevertheless, it will be accounted in total stress combination per sh. 71 & 73.

similarly in steel reinforcing:

$$f_{s_b} = \left(\frac{29}{\underset{8}{\cancel{3.6}}}\right) .17 = 1.4 \text{ ksi} \ll .9 f_y = 54 \text{ ksi}$$

Longit. stresses (cont'd.)

Axial stresses and equivalent total compression thrust, C_t
" tension ", T_t

sh. 58 — sh. 57

$$C_t = \sigma_a A_c = (3.7)(94)\,12^2 = 50{,}100\ k$$

$$T_t = f_{s_a} A_s = (29.2)(33.6) = 980\ k$$

Development of such high axial loads, as derived from the transfer of soil strains into buried structure requires sufficient anchorage of the structure into soil by (1) friction and/or (2) lateral soil bearing at ends and projecting elements. Conservatively, both friction and soil bearing will be considered as simultaneously effective and additive to obtain the limiting thrust transfer.

The friction considered will be (1) at underside of floor of tunnel and gallery at one end, subject to dead load and permanent live load, and (2) on sidewalls over a portion of tunnel length, subject to "at rest" soil pressure; see sketch per sh. 61.

LAO 0513 a-73

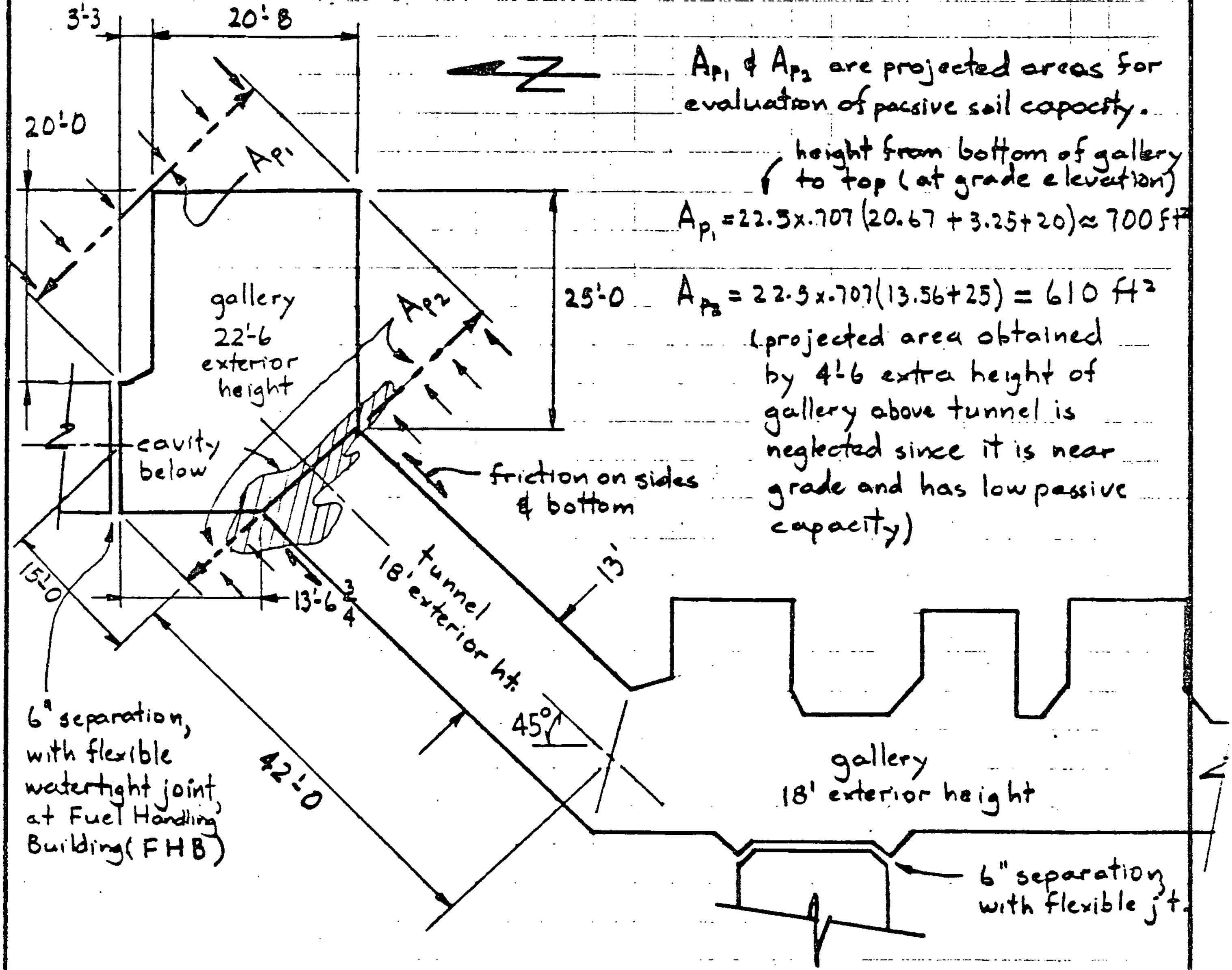
CALCULATION SHEET

CALC. NO. C-260-3

SIGNATURE Alfredo Lopez DATE 7/23/80 CHECKED [illegible] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 61 OF 78 SHEETS



A_{P1} & A_{P2} are projected areas for evaluation of passive soil capacity.

height from bottom of gallery to top (at grade elevation)

$A_{P1} = 22.5 \times .707(20.67 + 3.25 + 20) \approx 700\ ft^2$

$A_{P2} = 22.5 \times .707(13.56 + 25) = 610\ ft^2$

(projected area obtained by 4'-6 extra height of gallery above tunnel is neglected since it is near grade and has low passive capacity)

Plan - Underground Tunnel & Galleries (Unit 3 is as shown, opposite hand with respect to Unit 2 as shown in sheet 29)

scale:

0 10ft 20ft

LAO 0513 8-73

CALCULATION SHEET

CALC. NO. C-360-5

SIGNATURE Alfredo Lopez DATE 7/23/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2&3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 62 OF 78 SHEETS

Longit. stresses (cont'd.)

Anchorage to develop maximum axial thrust

For the purpose of evaluating the maximum longitudinal stresses in the tunnel at the location over the assumed cavity, the least anchorage on either side of the cavity will govern. The location of the cavity is well determined to be directly below the junction of the tunnel and the gallery at the northeast end of the tunnel, see sh. 61. That gallery at northeast end is smaller than the remainder of tunnel and galleries at other end, and it affords the least area for lateral soil bearing and the least weight for frictional resistance. Therefore, the least anchorage is as afforded by the NE gallery alone, without any real contribution from the tunnel since it actually lies on the other side of the cavity opposite to the side where the NE gallery is. Nevertheless, for the purpose of obtaining a conservative upper bound, a portion of the tunnel amounting to 1/4 th. of the tunnel length will be considered to develop frictional anchorage in conjunction with the gallery anchorage.

A_{p_1} is area at end, applicable for development of axial C.

A_{p_2} is area of projected elements, applicable for development of axial T.

Dead load + soil on top (only over tunnel, none over gallery whose top elev. is at grade) + permanent live load inside of tunnel or gallery are used for friction anchorage at underside of base mat.

W_g = 790 k is total weight corresponding to gallery.

W_t = 260 k " " " " tunnel (1/4 length).

LAO 0513 4-73

CALCULATION SHEET

BECHTEL

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/23/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 63 OF 78 SHEETS

Longit. stress (cont'd.)

Development of axial C

ultimate passive pressure:

$p_{pu} = K_{pu} \gamma h,$ where: $K_{pu} = 5.1$, $\gamma = .130\ k/ft^3$

avg. pressure over height of projected area

$\bar{p}_{pu} = 5.1(.130)\frac{22.5}{2} = 7.5$ ksf

(this is an upper limit of passive pressure introduced only to estimate the maximum axial thrust possible)

$C_p = \bar{p}_{pu} A_{p_1} = 7.5(700) = 5250\,k < 50{,}100\,k = C_t$

sh. 61 sh. 60

∴ not sufficient to develop C_t, so must include friction to obtain maximum possible

friction:

$C_f = \mu N,$ where: $\mu = 0.4$ (friction coeff. soil/concrete)

$N = (W_g + W_t) + P_s$

sh. 62

"at rest" lat. soil pressure, total load

avg. pressure

$p_s = K_s \gamma h$ $\bar{p}_s = 0.20(.130)\left(\frac{4.5+22.5}{2}\right) = .35$ ksf

$P_s = 2(.35)(18 \times \frac{42}{4}) \approx 130k$

$\frac{1}{4}$ x tunnel length, see sh. 62

2 side walls considered since cavity on one side is not a valid consideration, and besides, an upper bound for P_s is sought.

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 64 OF 78 SHEETS

Longit. stresses (cont'd.)

friction (cont'd.)

It is noted that frictional resistance on side walls of gallery is not a valid additive component since it is already implicitly accounted upon considering passive soil pressure on side walls which are oblique to axis of tunnel.

$$\therefore \quad N = (790 + 260) + 130 = 1180\,k$$

sh. 62 sh. 63

$$C_f = (0.4)\,1180 \approx 470\,k$$

$$\therefore \quad C_{t_{max.}} \approx C_p + C_f = 5250 + 470 \approx 5720\,k < 50100\,k$$

This is the upper limit of axial compression; higher loads are not postulated to occur due to yielding of passive soil wedge and/or sliding of buried structure with respect to soil (exact mechanism and sequence of soil failure is not important, only the estimate of maximum possible load transfer is important

$$\therefore \quad f_{c\,max.} = \frac{5720}{(94)\,12^2} = 0.42\,ksi$$

sh. 57

this is a low stress for concrete, see sh. 70 for combination with flexural compressive stress.

CALC. NO. C-260-5

CALCULATION SHEET

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 65 OF 78 SHEETS

Longit. stress (cont'd.)

Development of axial T

sh. 63

$$T_p = f_{pu} A_{p_2} = 7.5\,(610) = 4580\,k > 980k = T_t$$

sh. 67   sh. 60

$\therefore$ T_t as calculated from imposed strain is well developed by passive soil pressure alone, no friction.

Stress in tension reinforcing will be combined with flexural stress due to transverse loads, see evaluation per sh. 72 & 73

Evaluation of loading combinations will be on the basis of stresses since stresses for steel and concrete, rather than axial loads or moments, are the parameters calculated from the imposed strains from longitudinal response.

CALCULATION SHEET

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED F.L. DATE 7/31/80 CALC. NO. C-260-5

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 66 OF 78 SHEETS

Longit. stresses (cont'd.)

Calculation of flexural stresses due transverse loads on tunnel over cavity ~ 25' span.

gravity loads

dead load = 23.9 k/ft, sh. 8

live load = 2.8 sh. 8

26.7 k/ft

$$\therefore M_x = \frac{26.7 \times 25^2}{8} = 2086 \text{ k}'$$

$\frac{wl^2}{8}$ per sh. 33 & 34

active soil pressure on one side, none on side with assumed cavity (see sh. 72 for comments on assumption of cavity on side of tunnel)

$$\bar{p} = (.024)\left[4.5 + (.75 \times 18)\right] = .432 \text{ ksf}$$

considering as average the higher pressure applicable to lower half of 18' high wall

$$\therefore M_y = (0.432 \times 18)\,\frac{25^2}{8} = 608 \text{ k}'$$

LAO 0513 8-73

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 67 OF 78 SHEETS

<u>Longit. stresses</u> (cont'd.)

flexural stresses due to transverse loads (cont'd.)

<u>OBE</u>

1.5 × peak gr. acceleration, see sh. 9

$$M_x = (1.5 \times .34)\ 2086 = 1064\ k'$$

.51 sh. 66

for M_y:

lateral inertia from mass of tunnel D.L. & L.L:

sh. 8

$$\rho = 16.9 + 2.8 = 19.7\ k/ft$$

$$M_{y_i} = (1.5 \times .50)\ 19.7 \times \frac{25^2}{8} = 1154\ k'$$

lateral dynamic soil pressure on one side:

sh. 11

$$\bar{p} = (.021)\left[4.5 + (.75 \times 18)\right] = .378\ ksf$$

$$M_{y_p} = (0.378 \times 18)\ \frac{25^2}{8} = 532\ k'$$

conservatively, consider absolute summation of inertial & soil response:

$$M_y = 1154 + 532 = 1686\ k'$$

Longit. stresses (cont'd.)

DBE

$M_x = (1.5 \times .6)\ \overset{\text{sh. 9}}{2086} = 1877\ k'$

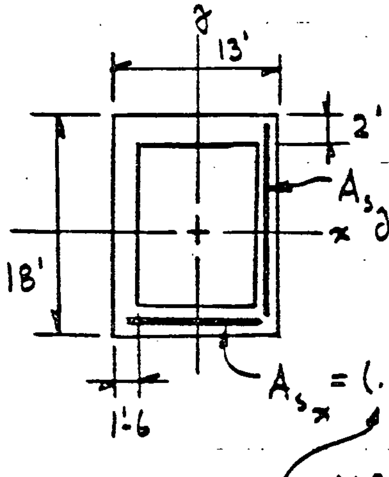
$M_{y_i} = (1.5 \times .9)\ 19.7 \times \frac{25^2}{8} = 2078\ k'$

$\bar{p} = (.\overset{\text{sh. 13}}{054})\left[4.5 + (.75 \times 18)\right] = .972\ ksf$

$M_{y_p} = (.972 \times 18)\ \frac{25^2}{18} = 1367\ k'$

$M_y = 2078 + 1367 = 3445\ k'$

The flexural stresses in steel reinforcing and concrete, as derived from foregoing moments will be combined with axial stresses obtained from wave propagation response.
The tunnel box section is "under reinforced", and the resultant flexural stresses in concrete are low (confirmed per sh. 71), therefore linear behavior for flexural stress analysis (WSD) of box section is adequate and conservative, particularly since longitudinal web reinf. is neglected.



$f_s = \frac{M}{A_s\, j d}$, $f_c = \frac{2M}{(kj)\, b d^2}$

$A_{s_y} = (.6)18 = 10.8 in^2$ $j = 1 - \frac{1}{3}k$, $k = \sqrt{2\rho n + (\rho n)^2} - \rho n$

expressions for j & k are for rectangular sections, but are adequate for box-sections if:
1. kd is within compr. flange thickness or nearly so, to be verified, sh. 69.
2. A_s' is conservatively neglected.

$A_{s_x} = (.6)13 = 7.8 in^2$

(.6 in²/ft from #7 @ 12 E.F.

Longit. stresses (cont'd.)

flexural stresses (coefficients as a function of M, based on WSD)

$$\rho_x = \frac{7.8}{(13\times 17)\,12^2} = .00025 \;\;\&\;\; n = 8 \longrightarrow k = .062,\; j = .98$$

$(kd) = (.062)\,17 = 1.05' < 2'$

$\therefore$ within compr. flg.

$$\therefore\; f_{s_x} = \frac{M}{7.8\,(.98\times 17)} = .00769\,M$$

(f in ksi, M in k-ft)

$$f_{c_x} = \frac{2\,M}{(.062\times .98)\;13\times 17^2\,(12)^2} = 6.08\times 10^{-5}\,M$$

$$\rho_y = \frac{10.8}{(18\times 12.25)\,12^2} = .00034 \;\;\&\;\; n = 8 \longrightarrow k = .076,\; j = .97$$

$(kd) = (.076)\,12.25 = .93' < 1.5'$

$\therefore$ within compr. flg.

$$\therefore\; f_{s_y} = \frac{M}{10.8\,(.97\times 12.25)} = .00779\,M$$

$$f_{c_y} = \frac{2\,M}{(.076\times .97)\;18\times 12.25^2\,(12)^2} = 6.97\times 10^{-5}\,M$$

CALCULATION SHEET

LAO 0513 8-73

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [illegible] DATE 7/31/80 CALC. NO. C-260-5

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 70 OF 78 SHEETS

Longit. stresses (cont'd.)

Concrete stresses:

1. From wave propagation: $f_c = 0.17 + 0.42 = 0.59$ ksi

bending, sh. 59 ; limited axial, sh. 64

The above stress (axial) applies to DBE as well as OBE since it is the maximum axial thrust limited by anchorage, which governs OBE thrust as well:

$$(C_t)_{OBE} \approx 0.5\,(C_t)_{DBE} = (.5)(\cancel{50100})\ 25050 \gg 5720\ k = C_{t\,max}$$

sh. 64

Appropriate ratio for OBE/DBE response pertaining to soil wave propagation effects. The lower structural and soil damping values associated with OBE, which often result in disproportionately higher OBE structural response (above grade), does not affect the governing wave propagation and ground particle velocities.
(The wave propagation velocities are the same for OBE & DBE; the corresponding ground particle velocities remain proportional to DBE or OBE ground accelerations)

2. From flexural loads:

stress coeff., sh. 69 ; M_x or M_y, sh 66, 67, 68

gravity & "at rest" soil: $f_{c_{M_x}} = (6.08 \times 10^{-5})\, 2086 = .13$ ksi

$f_{c_{M_y}} = (6.97\ \ )\, 608 = \underline{.04}$ ksi

.17 ksi

LAO 0513 8-73

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2&3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 71 OF 78 SHEETS

Longit. stresses (cont'd.)

2. Concrete stresses from flexural loads (cont'd.)

DBE :

$$f_{c_{M_x}} = (6.08\times10^{-5})(1877) = .11 \text{ ksi}$$

$$f_{c_{M_y}} = (6.97\quad)(3445) = .24 \text{ ksi}$$

$$f_{c_{(M_x+M_y)}} = (.11^2 + .24^2)^{1/2} = .26 \text{ ksi}$$

SRSS of two DBE seismic directions

OBE :

$$f_{c_{M_x}} = (6.08\times10^{-5})(1064) = .06 \text{ ksi}$$

$$f_{c_{M_y}} = (6.97\quad)(1686) = .12 \text{ ksi}$$

$$f_{c_{OBE}} = (.06^2 + .12^2)^{1/2} = .13 \text{ ksi}$$

3. Combination of wave propagation plus flexural :

flexural gravity, sh.70 — wave propg., sh 70 — flexural DBE, sh.71

$$f_{DBE} = .17 + \underbrace{\cancel{[(.59)^2 + (.26)^2]^{1/2}}}_{.64} = .81 \text{ ksi} < .85 f'_c = 3.4 \text{ ksi}$$

$$f_{OBE} = .17 + \underbrace{\cancel{[(.59)^2 + (.13)^2]^{1/2}}}_{.60} = .77 \text{ ksi} < .45 f'_c = 1.8 \text{ ksi}$$

∴ The concrete stresses are low. It should be noted that the biaxial compression that would result locally due to internal moments of box section, per sh. 42-51, does not represent an aggravation of state of stress.

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED F.J. DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 72 OF 78 SHEETS

Longit. stresses (cont'd.)

Reinforcing steel stresses

1. From wave propagation : $f_s = \overset{\text{sh. 59}}{1.4} + \overset{\text{sh. 58}}{29.2} = 30.6$ ksi

Above stress is for DBE, per particle velocity sh. 55. For OBE, based on proportionately lower particle velocity and commentary per sh. 70:

$$f_{s\,OBE} = .5\, f_{s\,DBE} = (.5)(30.6) = 15.3 \text{ ksi}$$

2. From flexural loads:

stress coeff. sh. 69 — M_x or M_y, sh. 66, 67 or 68

gravity & "at rest" soil : $f_{s\,M_x} = (.00769)(2086) = 16.0$ ksi

$f_{s\,M_y} = (.00779)(608) = 4.7$ ksi

DBE : $f_{s\,M_x} = (.00769)(1877) = 14.4$ ksi

$f_{s\,M_y} = (.00779)(3445) = 26.8$ ksi

OBE : $f_{s\,M_x} = (.00769)(1064) = 8.2$ ksi

$f_{s\,M_y} = (.00779)(1686) = 13.1$ ksi

For combination of seismic stresses, incorporating clarification of transverse loading (M_y's) related to postulated extreme of a cavity located towards one side of tunnel, see sh. 73.

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 73 OF 78 SHEETS

Longit. stresses (cont'd.)

The transverse lateral loading associated with a postulated cavity located next to one of the tunnel sidewalls, per sh. 35, was considered only to address the nominal flexural behavior of tunnel under lateral load in lieu of a more rigorous bending analysis. In reality, there is no basis nor physical evidence of the formation of such cavities at the shallow depths next to the tunnel sidewalls. Therefore, the postulated lateral loads, which imply gross abscence of soil and lack of lateral support next to the tunnel, do not constitute a realistic approach and need not be included in the rigorous stress combination accounting of the well founded loadings due to vertical flexure (cavity postulated below tunnel) and seismic wave propagation effects per this section of the calculation.

The current analysis of wave propagation affects includes evaluation of bending stresses (sh. 59). This is an appropriate accounting of secondary bending in addition to the primary bending due to the vertical loads spanning over the unlikely, but nevertheless realistically postulated, 25 ft cavity below tunnel.

Accordingly, flexural stresses in longitudinal reinforcing due only to transverse vertical loading (M_x's) will be considered in combination with axial and bending stresses due to seismic wave propagation.

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/24/80 CHECKED [signature] DATE 7/31/80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 74 OF 78 SHEETS

Longit. stresses (cont'd.)

Reinforcing steel stresses (cont'd.)

3. Combination of wave propagation plus flexural:

flexural gravity; wave propg.; flexural OBE; all per sh. 73

$$f_{s_{OBE}} = \underset{\text{flexural gravity}}{16.0} + \left[(\underset{\text{wave propg.}}{30.6})^2 + (\underset{\text{flexural OBE}}{14.4})^2 \right]^{1/2} = \underline{50 \text{ ksi}} \approx .9 f_y = 54 \text{ ksi}$$

$$f_{s_{OBE}} = 16.0 + \left[(15.3)^2 + (8.2)^2 \right]^{1/2} = \underline{33 \text{ ksi}} \approx .5 f_y = 30 \text{ ksi}$$

The stress levels stated above are regarded as adequate because of the following considerations:

1. The analysis and overall stress combination are very conservative, particularly as they pertain to the dominating stress component (30.6 ksi) derived from seismic wave propagation involving non-concurrent maxima of waves of different angles of incidence.

2. The abovementioned dominating stress component arises from the imposition of soil strain into the buried structure, therefore the stressing is (1) self-limiting upon reconciliation of the soil deformation within the structure, and (2) does not require development of a load carrying capacity to resist an imposed, sustained load. Clearly, additional reinforcing would not be effective to reduce 15.3 ksi (stress due to a fixed strain) in OBE case.

Differential movement at ends of tunnel

1. The northeast end of the tunnel (at NE gallery, see sh. 61) is the only boundary of the tunnel that is abutting onto a massive structure that extends above grade and exhibits appreciable seismic response with respect to the soil/tunnel system. The adjacent structure is the Fuel Handling Building (FHB) which undergoes the following displacement seismic response:

$$\Delta_{N-S} = \pm 1.1 \text{ in.}$$

$$\Delta_{E-W} = \pm 1.0 \text{ in.}$$

$$\Delta_{vert.} = \pm 2.1 \text{ in}$$ (more pronounced due to rocking amplification of edge of building)

The above response is based on the "SONGS 2 & 3 Power Block Analysis" (For the purpose of determination of differential displacements between structures from seismic excitation) dated 9/15/75.

2. The displacement at the ends of a short buried structure with respect to the soil can be estimated as follows: (per BC-TOP-4A, rev. 4, section 6.2.5)

$$\Delta_w = \frac{\epsilon_m L}{2} - \frac{f L^2}{8AE}$$

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/28/80 CHECKED T. Wod DATE 8-1-80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 76 OF 78 SHEETS

Differential movement (cont'd.)

where: ϵ_m = max. longitudinal soil strain

$$\epsilon_m = \frac{\sigma_a}{E} = \frac{\overset{\text{sh. 38}}{4.9}}{3.6 \times 10^3} \approx 1.4 \times 10^{-3} \text{ in/in}$$

(backcalculation from $\sigma_{a\,total}$)

L = length of tunnel = (42 + 15) = 57'
sh. 61

A = cross sectional area

E = elastic modulus

f = friction force per unit length

$$f = \frac{.4(W_t + P_s)}{42/4} = \frac{.4(260+130)}{42/4} \approx 15 \text{ k/ft}$$

sh. 62 & 63

(the above W_t & P_s are per sh. 63; conservatively only the least frictional resistance of tunnel length, neglecting soil bearing and friction at NE gallery, is subtracted in formulation for Δ_w per sh. 75)

<u>Differential movement</u> (cont'd.)

$$\Delta_v = \frac{1.4\times10^3\,(57\times12)}{2} - \frac{13\,(57)^2}{8(94)(3.6\times10^3)12} = .48''$$

Conservatively, this maximum axial displacement of tunnel with respect to soil will be added to displacement of FHB with respect to soil in order to evaluate maximum displacement of tunnel with respect to FHB:

$$\Delta_{max} = (1.1^2 + 1.0^2)^{1/2} + .48 \approx 2.0 \text{ in}$$

{ conservatively, the 45° direction resultant

The 6" flexible joint provided affords an allowable displacement of ± 3 in. which accommodates well the calculated $\Delta_{max} = 2.0$ in. Therefore structural interaction between tunnel and FHB is not applicable.

LAO 0513 8-73

CALCULATION SHEET

CALC. NO. C-260-5

SIGNATURE Alfredo Lopez DATE 7/28/80 CHECKED T. Dool DATE 8-1-80

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT Electrical Tunnel SHEET 78 OF 78 SHEETS

Transverse shear due to wave propagation

$$Q_{max} = \frac{9 E I a_m^2}{C_s^3 v_m} = \frac{9(3.6\times10^3)(4030)(.67\times32.2)^2}{(2500)^3 (2.2)} = 1.8k$$

rsh.

by inspection, that shear is insignificant for tunnel section.