UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

Owen's

Owen's
Turbine Bldg
Anchor Bolt Calculations

OCT 30 1981

MEMORANDUM FOR: Gus C. Lainas, Assistant Director
for Safety Assessment
Division of Licensing

FROM: James P. Knight, Assistant Director
for Components and Structures Engineering
Division of Engineering

SUBJECT: SAN ONOFRE 1 SEISMIC EVALUATION, STRENGTH OF COLUMN-TO-FOOTING CONNECTIONS IN NORTH TURBINE BUILDING EXTENSION

During the week of October 12, 1981, the Structural Engineering Branch (SEB) was asked to provide input to the Systematic Evaluation Program Branch (SEPB) concerning the ongoing seismic safety evaluation of the San Onofre Nuclear Generating Station Unit 1 (SONGS 1). SEPB was concerned that the seismic resistance of portions of the plant is such that it cannot be allowed to continue operation until structural modifications to the North Turbine Building Extension are made. Southern California Edison (SCE) had previously made a commitment to make such modifications by June 1, 1982, or shut the plant. However, SEPB indicated that certain structural components might be completely inadequate to resist the postulated earthquake and thus the plant would have to cease operation immediately in order to make the required modifications.

SEB was asked to review the available material and make a quick assessment of the adequacy of the structural components in question. The structural members considered weakest are several columns in the North Turbine Building Extension. Specifically, the resistance of column-to-footing connections against rotation was considered to be the most vital concern. More specifically, the capacity of the anchor bolts which connect the columns to the foundation was considered inadequate.

At a meeting on October 15, 1981, SCE presented an evaluation which indicated that the anchor bolts in question would survive the postulated seismic event if credit were allowed for the 8" thick reinforced concrete slab-on-ground, and the poured concrete surrounding the columns. SEB was asked to provide an evaluation of this analysis.

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XA

XA Copy Has Been Sent to PDR

SEB has performed a somewhat rough calculation (copy enclosed) to evaluate the column base connections. This indicates that the slab-on-ground can be considered to provide sufficient restraint for the column base connections based on the maximum loads postulated. Therefore, the adequacy of these connections should not be used as a basis for requiring immediate modifications to the Turbine Building structure.

Since SEB was not involved in this evaluation previously, our investigation was confined to the immediate area of interest, namely the adequacy of column base/slab-on-ground connections in the North Turbine Building Extension of SONGS 1.

James P. Knight, Assistant Director
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Enclosure: As Stated

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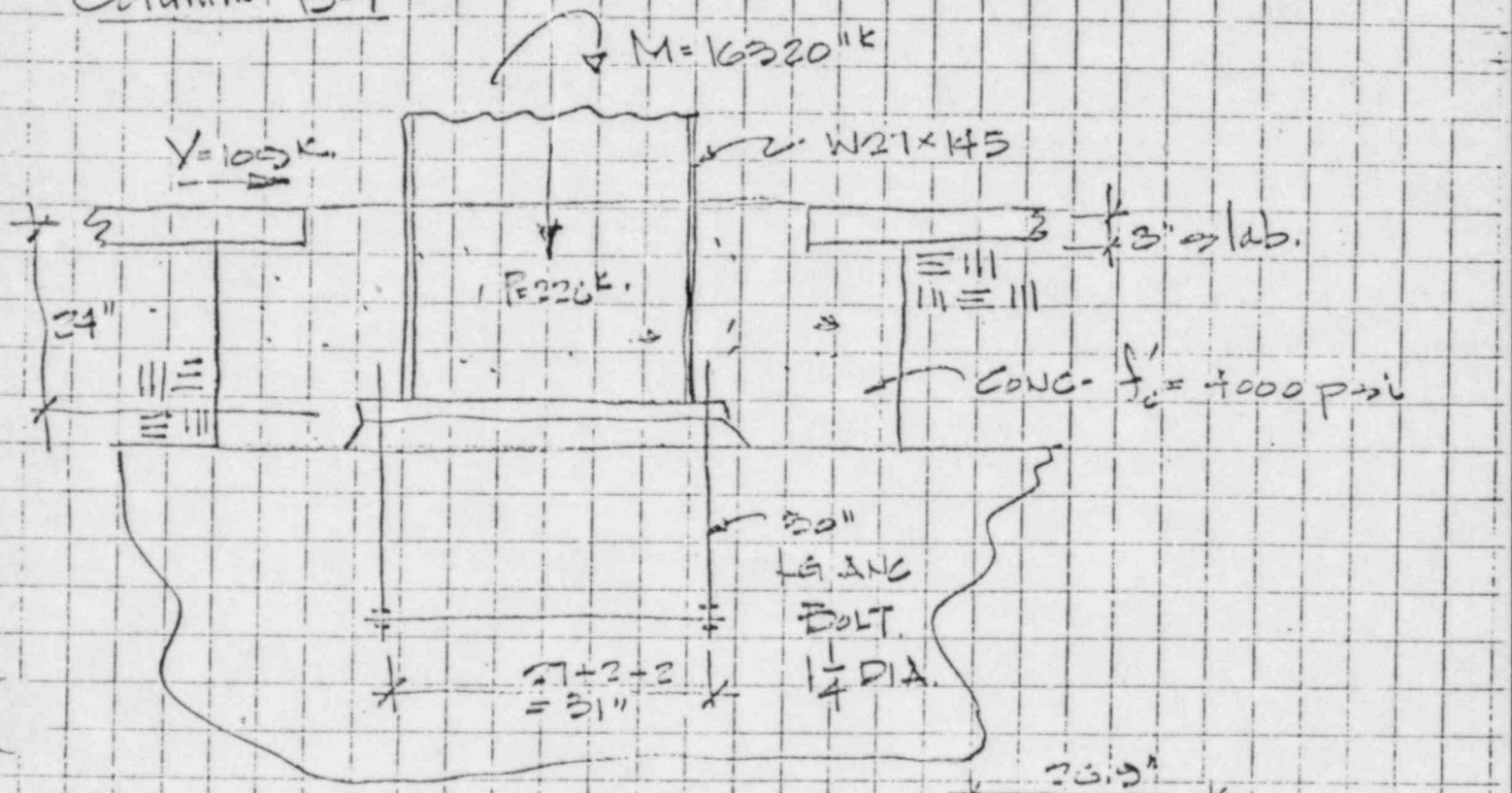
San Onofre #1 – North Turbine Extension

Investigation of Anchor Bolts for Columns

Column B-7

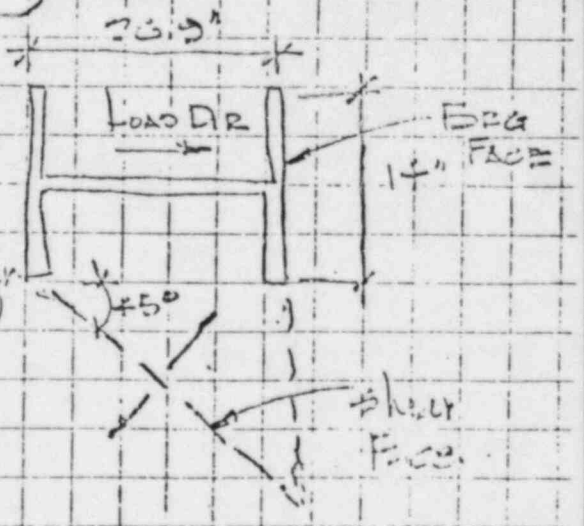


Max Capacity of 1 bolt

(Per ACI 318-71 & Commentary Sect 11)

Max allowable shear stress

$= 4\sqrt{4000} = 253$ psi (shear reinf in slab not considered)



Max allowable shear load around column =

$2 \times 26.9 \times 1.42 \times 8 \times 0.253 = 155^k$.

Maximum Bearing stress = 0.35×4000

$= 3400$

Max Bearing Load: $3.40 \times 14 \times 5 = 381$

Max Load in slab: $155 + 381 = \underline{536^k}$.

Max Load Carried By Slab

Ultimate strain = 0.003"/"

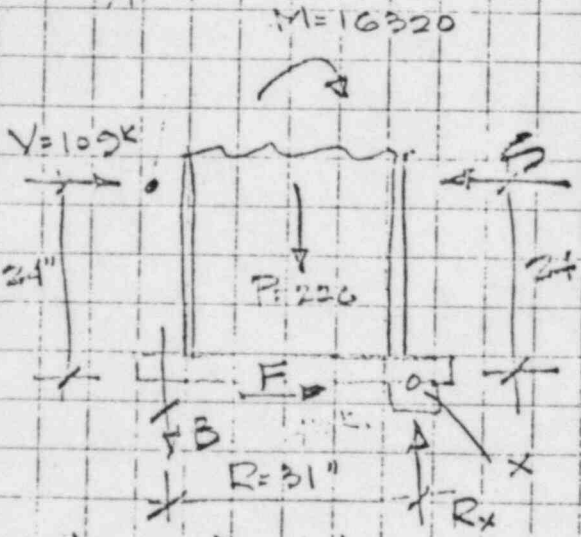
Say Max allowable ε_c = 2/3 × 0.003 = 0.002"/"

$E_c = 57000\sqrt{4000} = 3.61 \times 10^6$ #/□"

$\Delta = \frac{PL}{AE}$ so $P_{MAX} = \frac{\Delta}{L} \cdot AE$

$= 0.002\,(8 \times 14)(3.61 \times 10^3)$

$P_{MAX} = \underline{808^K}$ — ESTIMATED CRUSHING LOAD OF SLAB BEHIND COL. FLG. — MINIMUM



Max Bolt Load

105 × 0.97 = 102^K/BOLT

= 204^K on 2 BOLTS.

= B

Load into slab — $\Sigma M_x = 0$

$S = (109 \times 24 + 16320 - 226 \times 15.5 - 204 \times 31)/24 = 330^K$

330 < 536. O.K.

$(\Sigma F_x = 0)$ Therefore F Required = 330 − 109 = 271^K

$[\Sigma F_y = 0]$ $R_x = \frac{226}{2} + 204 = 317^K$; Friction Force available

= Bolt load

$= 317 \times 0.5 = 159^K$

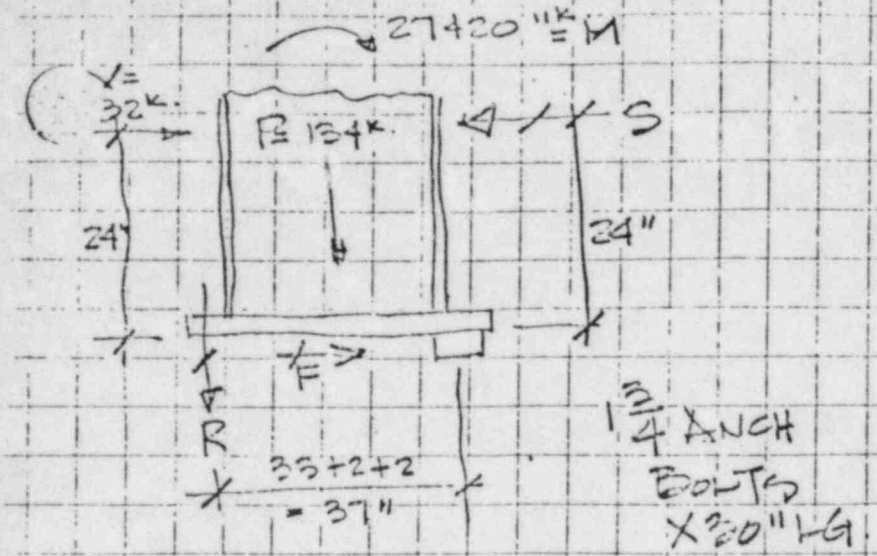
So about 271 − 159 = 112^K Must be taken in the concrete below slab.

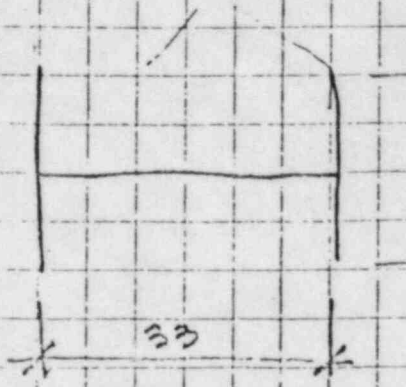
Assume 10 □' available Concrete parallel to plane of base plate

So shear resistance available in concrete =

10 × 144 × 0.253 = 364^K O.K., & Therefore there is no shear needed from bolts.

Column B-6





Slab Capacity

$\phi V_{nat} = 2 \times 2.5 \times 1.42 \times 5 \times 0.255 = 190^K$

Beam $= 3.40 \times 15.75 \times 5 = 428^K$

Max Slab Load $= 428 + 190 = 615^K$ allowable.

P_{MAX} @ 0.002 "/" Defl $= 0.002 \times 8 \times 15.75 \times 3.61 \times 10^3$
$= 910^K$ capacity at max. strain

Max Bolt Load $= 1.9 \times 105 = 200^K$ $\therefore B = 400^K$

$S = (27420 + 152 \times 24 - 134 \times 15.5 - 400 \times 37) / 24 = 604^K$
< 615
$- 2K.$

$\Sigma F_x = 0$ $F = 604 - 152 = 422^K$ required

Friction Force $= (134 + 400/2)(0.5) = 234^K$ (400 = Bolt Load)

$\therefore$ Need an additional resistance of $422 - 234 = 189^K$

From Concrete below slab. Shear resistance of (say) 10" of concrete in plane of base plate $= 10 \times 4 \times 1 \times 7.5$
$= 304^K >> 189$ & no shear wicked from bolts.

Owen Fairley 10/25/51