

2

ADA950137

6 DESIGN OF TURRET FOUNDATIONS.

DTIC ELECT S JAN 19 1981

A

APPROVED FOR public release? distribution unlimited MAY 12 1978

12 24

DTMB

U.S. EXPERIMENTAL MODEL BASIN Navy Yard, Washington, D.C.

DDC FILE COPY

11 Oct 28

14 DTMB 297

81 1 09 051

104650 mt

XX

DESIGN OF TURRET FOUNDATIONS

Studies of the strength of turret foundations have been made, the results of which are reported elsewhere. Attention is here given to specific procedure in design.

Only foundations of the accepted type are considered. In these the turning structure rests through roller or ball bearings on a fixed circular rollerpath which forms the free end of a vertical tubular cantilever, supported at its lower end by the structure of the ship. This cantilever is short in proportion to its diameter; it may receive support from attached structure above its base; and its design may be complicated also by other details; but it retains the essential character of a cantilever. The load is applied through the rollerpath and base connections. Other forces are secondary in importance.

As in every structure, we must consider both the strength of the whole unit and that of the detailed parts of which it is formed, the resultant forces and couples applied and the concentrations occurring in their distribution.

First let us get a clear picture of the resultants. With the ship and guns at rest and ship on an even keel the rollerpath receives only vertical forces whose resultant is a vertical force, equal to the weight W of rotating parts included in the turret, and passing through their C.G. The weight of oscillating parts is carried on the trunnions. During recoil the trunnions receive an additional load P which acts parallel to the line of fire. Its vertical component, P_y , combined with W , gives the total resultant

Availability Codes	
Dist	Avail and/or Special
A	
UNCLASSIFIED	

vertical load. The horizontal component, P , causes acceleration of the turret in motion which is eventually stopped by the reaction F at the fixed rollerpath. As a matter of practical convention we will take $F = 2P$ on the ground that F is suddenly applied to the turret to stop the motion acquired under the action of P .

Since the trunnions, the rollerpath, and the C.G. of the rotating parts are at different heights, the forces P and F and the dynamic actions involved will tend to produce tilting of the turret. If the C.G. were at the same level as the trunnions, say Y_t , the couple would be equal to F times height of trunnions above rollerpath, $(Y_t - Y_p)$; this assumption will ordinarily suffice for purposes of foundation design.

In design of a turret the quantities W , P , Y_t , Y_g , Y_p , are all known. The resultant load coming on the foundation consists of a vertical (axial) force W and P_y , acting centrally; a horizontal (radial) force $F - 2P_x$, acting in the plane of the rollerpath; and a tilting couple acting about a horizontal axis transverse to the line of fire. The magnitude of the couple equals the tilting moment of recoil $F (Y_t - Y_p)$ if there is no eccentricity of the vertical forces.

In any section of the foundation-cantilever at height Y above base three corresponding actions must be resisted; a total vertical pressure, W plus P_y ; a total shear $F - 2P_x$, and a couple whose magnitude is found by combining the couple at the rollerpath with the bending moments $F (Y_p - Y)$, to give a total couple $F (Y_t - Y)$.

The structure to be designed consists of a circular bulkhead forming the tubular cantilever, reinforced at the top by a ring of which the rollerpath forms a part, and supported at the base by connections to the ship's structure. Other connections to the ship structure may exist but the support which they may be expected to give is a matter of judgment rather than calculation. Such a structure may fail in three different ways: By excessive distortion of the ring, leading to jam in train; by buckling of the bulkhead plating at the rear of the base; by wrinkling in shear at the sides of the base. Model tests show that in mild steel, and aside from concentrations of stress, permanent set in the ring may be expected when bending stresses approach 40,000 #/sq. inch, buckling may be expected when compressive stress approaches 30,000 #/sq. inch, and wrinkling may be expected when average shear, over the whole cantilever section, approaches 15,000 #/sq. inch. In design each of these modes of failure should be considered and a safe margin provided. But in general it may be expected that the concentrations and other details will have a controlling influence.

THE RING

The behavior of the ring under concentrated radial load is clearly made out, and is set forth in the report on "Short Flanged-Tube Cantilevers". The load as actually applied is radial, since the bearing eliminates tangential components; but it is not concentrated. Simple spread of the area through which the load is transmitted to the ring

has only a small effect on stresses in the ring. Division of the load between front and rear, such as is accomplished by the flanged rollers used under battleship turrets has a powerful effect in reducing stresses in the ring. And the stresses in the ring are diminished through the support received from the turning structure along the transverse diameter.

These three circumstances; spread of load, division of load, support along transverse diameter, strongly influence the conditions under which the ring works. As no quantitative data exist on the actual extent of these effects we must be content with a note of the conditions in the only two designs which have been studied from this point of view.

In the CALIFORNIA the moving ring is very rigidly supported by a heavy and practically complete diaphragm, the "pan". The designed clearance in horizontal motion of the turret is $1/8$ inch. As the observed relative radial deflection of the moving ring with respect to the fixed ring at the rear point exceeds the amount of the clearance, the load must be somewhat spread through the fixed ring being forced to conform to the contour of the moving ring. The load is divided between the front and rear points, and the small deflections along the transverse diameters, which are less than the designed clearance, indicate that it is rather evenly divided; under these circumstances it appears that the moving ring comes under transverse compression very little, if at all.

In the PENSACOLA the interior of the moving ring is open, the radial clearance is small, and the load acts at the rear only, without division. On account of the small

clearance the spread is probably rather large, but since the load is undivided the transverse diameter is compressed more than in the CALIFORNIA, and at the same time the moving ring is not so well adapted to resisting this compression.

Although it is clear that bending moments and stresses in the ring calculated on the assumption that the load is concentrated will be considerably departed from, nevertheless such nominal stresses will serve to indicate the features in ring design which are necessitated by the resultant loads applied, leaving for judgment only such decisions as refer to the effects of spread, division of load, and transverse support. Since all three of these operate to reduce stresses, nominal values will be greater than actual values, and may even exceed the actual tensile strength of the material. This is due, of course, to reduction of load, not to increase in resistance of the material.

If the actual stresses were known the ratio of actual to nominal stress would be acceptable as a measure of the ratio of actual to nominal load. But the actual stresses not being known, it is not possible to evaluate such ratios and convenience leads us to deal in nominal stresses, even though these have only a fictitious relation to the strength of the material.

In the case of the CALIFORNIA another circumstance enters, namely, the support given directly to the ring by the massive vertical "stiffeners". These extend downward through two decks and act as individual cantilevers which relieve the ring of part of the load directly where it is applied. Considerations detailed in the "Theory of Design of Turret Foundations" lead to an estimate of the load thus taken off the ring at $1/3$ total.

On the other hand the free sliding of the turret has a tendency to increase the factor for impact above its nominal value. This increase was estimated at 7%, but in view of uncertainty it is considered better not to introduce any correction on this account.

In determining the moment of inertia of the ring a fair rule to follow would be to include only material above the connection with the bulkhead plating. But in the CALIFORNIA the heavy radial connections afforded by the top of the "stiffeners" lead to including all material down to the large lightning holes.

In details of the ring, as connections between parts, radial webs, etc., no special features are introduced by its curved form, and practice for straight beams may be taken as a guide. The connection with the bulkhead plating will be eccentric with respect to the load, and might be thought to require special attention to torsional stresses; no difficulty of this nature was found although the eccentricity of this connection in 8" mounts when proved was much greater than when installed in the ship. In this respect the circular form of the member strongly affects its action. The connection to the plating must carry the vertical load which is rather heavily concentrated at the rear point, and the horizontal load concentrated about the side points.

Doubt exists in the PENSACOLA as to the extent of material properly included in the ring section because the ring is attached to the deck. It is clear that this plating, which appears in the radial section of the ring as a very light projecting fin, cannot be effective in helping the ring to resist bending; where in compression it will buckle, and where in tension it will be locally overstressed.

In the calculation the deck plating was included in the ring section only out to the edge of the heavy doubler which surrounds the ring casting. Even this doubler seems thin, and the section resembles a T with a heavy flange and thin web. The deck beams were considered to be of assistance in holding this thin web to its work. Direct support is given the ring by the deck plating, and in turrets one and four, where heavy stringer plates and shear strakes are near by, this support must be considerable. In turret III, however, the structure adjoining the ring consists only of a light narrow deck house; the support which it affords is small and uncertain and, so far as the ring is concerned, is best neglected.

THE FOUNDATION-CANTILEVER

In the plating as in the ring, nominal stress will serve as a guide in adaptation of scantlings to dimensions and resultant loads; allowance for concentrations and special features must remain largely a matter of judgment. The horizontal load enters from the ring through the area adjoining the side points, and the vertical load is transmitted directly downward through the rear point. In these features there is little room for variation. But at the bottom connections in the PENSACOLA differ radically from those in the CALIFORNIA. In the PENSACOLA the circular bulkhead simply rests on the deck, held there, of course, by bounding angles. In the CALIFORNIA, it is continuous through the second deck and the cantilever is thus built in for a

full deck height. It is true that there are also deck connections, both at the top and at mid-height, in the PENSACOLA; but these are very insubstantial as compared with the connection at the second deck of the CALIFORNIA, where the narrow diaphragm takes hold of heavy barbette armor and armored deck outside of it.

In the CALIFORNIA, vertical forces are carried by the third deck, but the moment is carried partly by the horizontal reactions at the two decks and only partly by the vertical reactions at the third deck. The concentrations at the third deck are therefore less severe than if the connection at the second deck did not exist. The maximum bending moment in the foundation-cantilever is at the second deck, where the only concentration is that which gives, in a tubular beam, a maximum local value of shear along the neutral axis to twice the average shear taken over the whole section.

In the PENSACOLA, the deck connections at top and at mid-height undoubtedly assist the foundation-cantilever to resist the horizontal load, and the bulkhead connections help to absorb the vertical load. This assistance is least in turret III, which has been regarded as the most unfavorable case. The support to the cantilever, as distinguished from the support to the ring, though equally uncertain, is greater. In designing details at the base of the circular bulkhead it was assumed that $1/3$ the total load was taken off the cantilever by the adjacent structure.

The question of stiffeners has been reserved until the last because it is the most important.

It has been the practice hitherto to provide stiffeners for the circular bulkhead, not only such as a flat bulkhead would require, but, in the battleships, a great deal more. Analysis and tests both indicate these stiffeners to be superfluous; but as this may be regarded as somewhat radical the case against stiffeners is here summarized.

The function of a stiffener is to assist a slender member subject to compression by preventing it from buckling and so avoiding its load. Ordinary bulkhead stiffeners also act as girders between decks to resist water pressure, but that consideration is incidental in this case. To hold plating in place under compression simple angles, without end connections, are sufficient; this is standard practice, for example, in stiffening webs of deep built-up girders. The "stiffeners" in the CALIFORNIA, however, are themselves massive plate girders with substantial end connections, and although they do not act as integral parts of the foundation-cantilever, they have strength enough as individual cantilevers to absorb an appreciable fraction of the horizontal load.

The weight of these heavy members could be much more effectively used if applied so as to contribute to the strength of the foundation-cantilever as a whole. That they do not do this in the CALIFORNIA is shown by the fact that measured deflections are over ten times what they should be.

Of the sixty-inch models, No. 1 had stiffeners and No. 2 none. With less weight No. 2 carried easily with-

out failure a load 50% greater than the ultimate load on No. 1.

The fact is that the curvature of the plating has an enormous stiffening effect in itself. The stability of flat plating in compression may fairly be judged by application of the Euler formula, as the slenderness ratios involved are high, but as soon as the plating has even a slight, though uniform curvature, its resistance is of an entirely different order. The strength of cylindrical plating in axial compression, so far as is known, has not been accurately calculated; our best guide is therefore the test data.

Of the 16 models tested only three had axial stiffeners. The test data in general thus apply to unstiffened plating. The three stiffened models showed no exceptional features; if their cantilever stiffness was a little greater, their cantilever strength was a little less; and neither departure was large enough to have any significance.

Study of the mode of failure of the stiffened models confirms the ineffectiveness of the stiffeners. In each case the wrinkling or buckling began at the base, and most of the length of the stiffeners was therefore not applied where needed. On 60-inch model, No. 1 the faying flange of the stiffener simply pulled away from the plating for a length involving two rivets. A more effective connection of stiffener at the lower end would help, of course, but the most effective way to add material is to thicken the lower strake and omit the stiffeners altogether.

NUMERICAL APPLICATION TO THE CALIFORNIA
TURRET III

Data for Turret No. III are as follows:

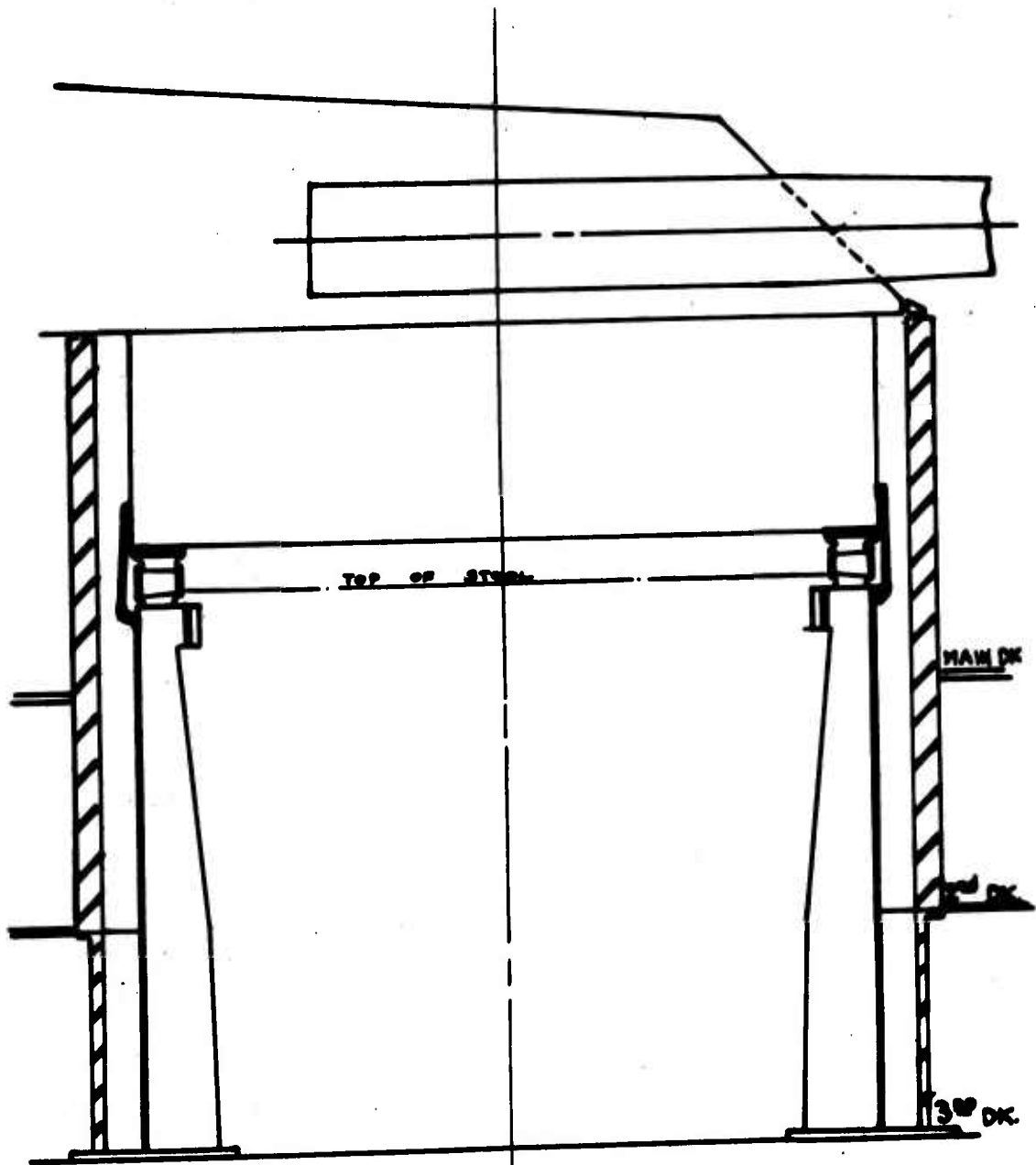
W	- Dead weight on rollers (C.G. at center of rotation)	2,130,000 #
P	- Pressure at trunnions due to recoil	2,070,000 #
Y_t	- Height of trunnions above 3rd deck	33 .07 ft.
Y_g	- Height of C.G. of rotating parts above 3rd deck	31.01 ft.
Y_p	- Height of fixed rollerpath above 3rd deck	20.48 ft.
Y	- Height of 2nd deck (base of canti- lever) above 3rd deck	7.77 ft.
F_o	- Horizontal reaction in ring at 0° elevation- $2/3$ ($2 \times 2,070,000\#$)	2,760,000 #
I	- Sectional moment of inertia of ring	23,500 in. ⁴
S	- Section Modulus	1,250 in. ³
R	- Mean radius of ring	160 in.
t	- Thickness of plating	1-1/4 in.

In calculating the three nominal stresses (bending of ring, compression of plating, shear of plating), such corrections as can be reasonably estimated will be introduced.

RING

If the full doubled recoil pressure of 4,100,000# were applied as a concentrated radial load to the fixed ring, the stress would be

207



GUN TURRET "II" FOUNDATION
U.S.S. CALIFORNIA.

$$\frac{.252 \times 4,100,000\# \times 160 \text{ in.}}{1,250 \text{ in.}^3} = 134,000\#/ \text{sq.in.}$$

If 1/3 be deducted from the load on the ring to allow for that taken by the "stiffeners" acting as individual cantilevers, the stress would be

$$2/3 \times 134,000\#/ \text{sq.in.} = 89,000\#/ \text{sq.in.}$$

If the load be assumed divided, $n = 2/3$ instead of $n = 0$, the stress would be

$$\frac{.125}{252} \times 89,000\#/ \text{sq.in.} = 44,000\#/ \text{sq.in.}$$

No correction is introduced for spread, for support along transverse diameter, or for increase due to motion of turret through horizontal clearance in rollers.

PLATING IN COMPRESSION

If "stiffeners" be assumed to act only to stiffen the plating and not to carry any load as separate cantilevers, or to work with the plating of the foundation-cantilever, section modulus is

$$0.8 \times 320^2 \times 1-1/4 = 102,000 \text{ inch}^3$$

If the support at the 2nd deck be neglected the bending moment at the 3rd deck is

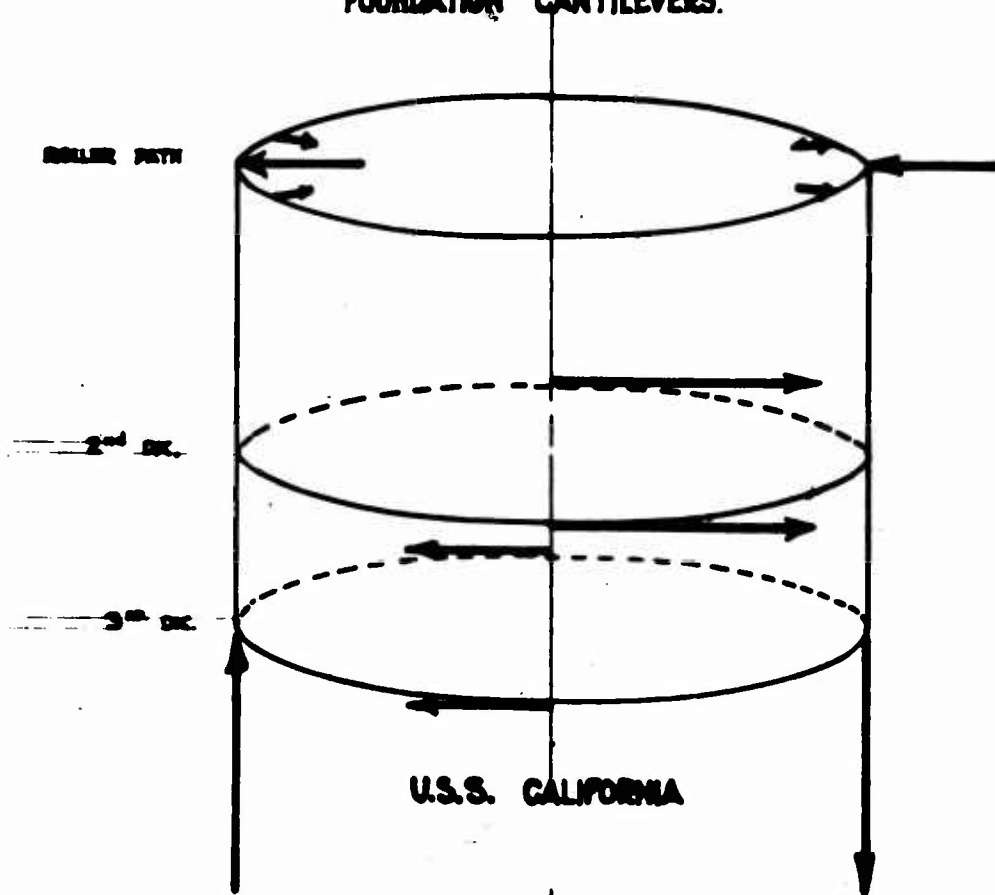
$$4,140,000\# \times 33.07 \text{ ft.} \times 12 \text{ in./ft.} = 1.64 \times 10^9 \text{ in.}\#$$

This leads to a nominal compressive stress in bending due to recoil of

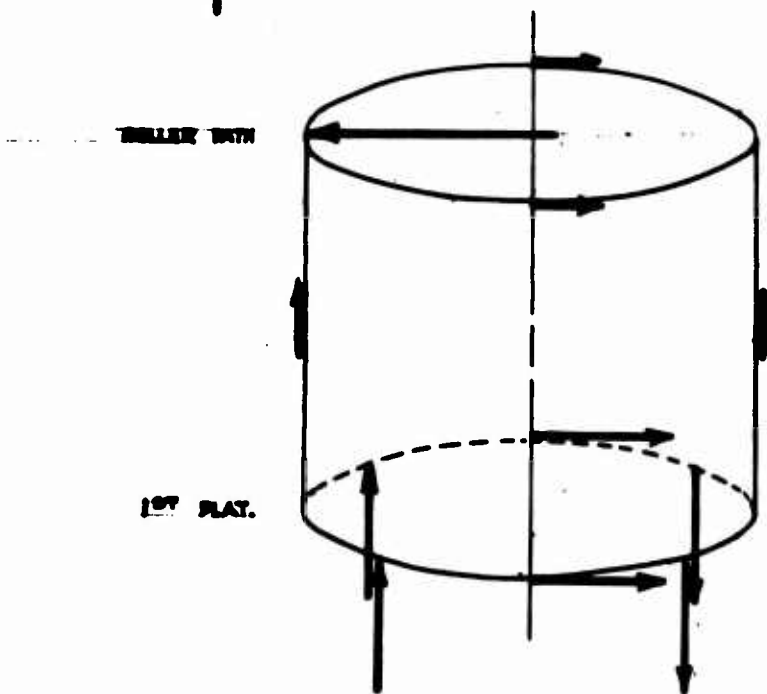
$$1.64 \times 10^9 \div 1.02 \times 10^5 = 16,100\#/ \text{sq.in.}$$

207

SCHEME OF EXTERNAL FORCES APPLIED TO FOUNDATION CANTILEVERS.



U.S.S. CALIFORNIA



U.S.S. PENSACOLA

The direct compression due to weight, assumed uniform, is

$$2,130,000\# \div (2\pi \times 160 \text{ in.} \times 1\frac{1}{4} \text{ in.}) = 1,700\#/sq.in.$$

giving a total nominal stress under the assumed conditions of

$$16,100 + 1,700 = 17,800\#/sq.in..$$

If $1/3$ the horizontal load be assumed to be carried by the stiffeners acting as separate cantilevers, this is reduced to

$$(2/3 \times 16,100) + 1,700 = 12,500\#/sq.in.$$

If $\frac{7.77}{33.07}$ or more of the bending moment be assumed to be absorbed by horizontal reactions at the 2nd and 3rd decks the maximum bending moment will occur at the 2nd deck and the stress there will be

$$(2/3 \times 16,100 \times \frac{25.3}{33.07}) + 1,700 = 9,900\#/sq.in.$$

PLATING IN SHEAR

Assuming no horizontal reaction at the 2nd deck, the nominal stress in shear under full doubled recoil load is

$$4,140,000\# \div (2\pi \times 160 \text{ in.} \times 1-1/4 \text{ in.}) = 3,300\#/sq.in.$$

Allowing $1/3$ of the load to be carried directly by the stiffeners leaves $2,200\#/sq.in.$

If more than $\frac{7.77}{33.07}$ of the moment be carried by the horizontal reaction at the 2nd deck, the horizontal reaction at the 3rd deck will be reversed and that at the 2nd

deck will exceed the load applied at the rollerpath.
Assuming 1/2 the moment so carried, the horizontal reaction
at the 2nd deck is

$$\frac{2}{3} \times 4,140,000\# \times 33.07' \times \frac{1}{2} \div 7.77' = 5,900,000\#/sq.in.$$

and the resulting shear is 4,700\#/sq.in.

NUMERICAL APPLICATION TO THE PENSACOLA
TURRET III

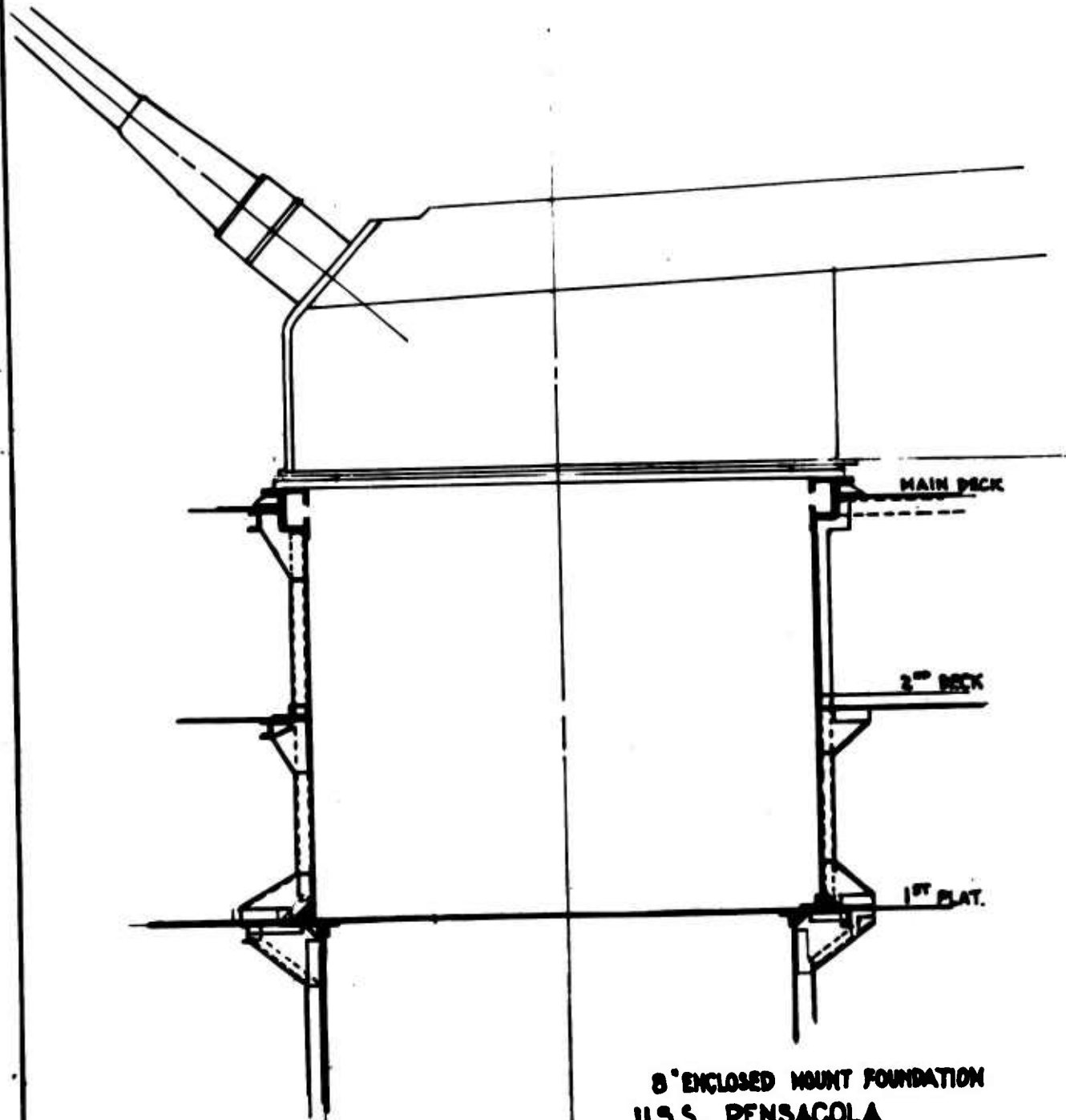
W	- Dead weight on rollers	560,000#
P	- Pressure at trunnions due to recoil	460,000#
Y_t	- Height trunnions above 2nd deck	26 ft.
Y_p	- Height of fixed rollerpath above second deck	20 ft.
F_o	- Horizontal reaction in ring at 0° elevation	920,000#
I	- Sectional moment of inertia of fixed ring, consisting of 2 castings, doubler, and adjacent deck	13,000 in. ⁴
S	- Section modulus of same	425 in. ³
R	- Mean radius of ring	120 in.
t	- Thickness of plating	3/4 in.

RING

Leaving out of account all assistance afforded
to the fixed ring by adjoining fixed structure and by the
moving ring, and applying the doubled recoil pressure as a
concentrated undivided radial load, the stress would be

$$\frac{.252 \times 920,000\# \times 120 \text{ in.}}{425 \text{ in.}^3} = 66,000\#/sq.in.$$

207



8" ENCLOSED MOUNT FOUNDATION
U.S.S. PENSACOLA

SCALE 3/8" = 1 FT.

The actual assistance from adjoining structure is small and the load is actually undivided. But the fixed ring does receive support from the moving ring along the transverse diameter due to the small radial clearance. The effect in the fixed ring is like that of forces applied radially outward at the side points. Moment at the rear point due to such support would be opposite to that due to the direct recoil, and is easily found from the ring formulae to be

$$.182 \times \text{radius} \times \text{force}$$

The accompanying deflection along the transverse diameter is

$$.113 \times \frac{R^3}{EI} \times \text{force}$$

as compared with

$$.051 \times \frac{R^3}{EI} \times F$$

due to recoil load.

Now suppose the transverse diametral deflection of the fixed ring to be reduced by half through support from the fixed ring. The transverse force required is that which would produce a deflection equal to half that due to the recoil load, or

$$920,000\# \times \frac{1}{2} \times \frac{.051}{.113} = 210,000\#$$

The relief in stress at the rear point due to this transverse support is

$$\frac{.182 \times 120 \text{ in.} \times 210,000\#}{425 \text{ in.}^3} = 10,800\#/\text{sq.in.}$$

leaving a net stress at the rear point of

$$66,000\#/sq.in. - 10,800\#/sq.in. = 55,200\#/sq.in.$$

For a margin between this figure and a safe working stress dependence must be placed on the uncertain effects of spread, the support afforded to the ring by the adjoining fixed structure, and such diminution of the effects of impact as may occur on account of the small radial clearance.

PLATING IN COMPRESSION

Again neglecting the support of all structure except the circular bulkhead, the modulus of its horizontal section is

$$.08 \times 240 \text{ in.}^2 \times 3/4 \text{ in.} = 3.46 \times 10^4 \text{ in.}^3$$

The bending moment at the 2nd deck is

$$920,000\# \times 26 \text{ ft.} \times 12 \text{ in./ft.} = 2.87 \times 10^8 \text{ in.}\#$$

and the corresponding stress is 12,000#/sq.in.

Direct compression gives

$$\frac{560,000\#}{2\pi \times 120 \text{ in.} \times 3/4 \text{ in.}} = 1,000\#/sq.in.$$

and the total is

$$13,000\#/sq.in.$$

If 1/3 the load be assumed taken off the foundation-cantilever by the adjacent structure, this is reduced to 9,000 #/sq.in.

The concentrations in way of hard spots under the second deck may readily double this figure to make 18,000#/sq.in. but as the plates are of Special Treatment Steel, with high elastic limit, margin of safety is ample.

PLATING IN SHEAR

Neglecting all support from adjoining structure,
nominal shear will amount to

$$920,000\# \div (2\pi \times 120 \text{ in.} \times 3/4 \text{ in.}) = 1,630\#/sq.in.$$

NUMERICAL APPLICATION TO THE PROVING GROUND GIRDER

W	- Dead weight on rollers	560,000#
P	- Pressure at trunnions due to recoil (proof salvo)	570,000#
F _o	- Horizontal reaction on ring at 0° elevation	1,140,000#
I _o	- Sectional moment of inertia as originally installed	13,477 in. ⁴
S _o	- Section Modulus of same	773 in. ³
R	- Mean radius	120 in.

RING

Assuming load to be concentrated and neglecting support from moving ring, stress would be

$$\frac{.252 \times 1,140,000\# \times 120 \text{ in.}}{773 \text{ in.}^3} = 45,000\#/sq.in.$$

This value is comparable with that of 66,000#/sq.in. obtained from the PENSACOLA under the same assumptions as to load distribution

By three stages the section modulus was reduced to 512, and the nominal stress thus increased to

$$45,000 \times \frac{773}{512} = 67,000\#/sq.in.$$

without signs of distress in the rings.

PROJECTED NEW DESIGN OF CALIFORNIA TYPE TURRET FOUNDATION

Assuming the same load to act at the rollers with the same division between front and rear as in the CALIFORNIA alternative design of foundation is suggested in some detail to serve as an illustration of what the model tests indicate to be practicable.

The principal change consists in omitting entirely the "stiffeners" on the cylindrical bulkhead. In addition to the direct saving of weight, this would permit reducing the diameter of the cylinder and hence of the barbette; or utilizing the space required further down. What follows is based on the latter alternative.

RING

Since the stiffeners are not available to act as separate cantilevers, the load applied to the ring is 4,140,000# with 3/5 at rear and 2/5 at front, $n = 2/3$. This gives a bending moment (maximum) of

$$.125 \times 4,140,000\# \times 160 \text{ in.} = 8.28 \times 10^7 \text{ inch \#}.$$

The tests on the Proving Ground Girder show that nominal stresses of 67,000#/sq.in. are acceptable; spread and support along transverse diameter not allowed for are sufficient to reduce actual stress enough below the nominal value to give safe working values.

The section modulus indicated is thus

$$8.28 \times 10^7 \text{ in.\#} \div 6.7 \times 10^4 \text{ \#/sq.in.} = 1,240 \text{ in.}^3$$

or about the actual value in the CALIFORNIA. Since the moving ring is much better adapted to resist transverse compression

than was the moving ring in the tests at the Proving Ground, this would provide a greater margin of strength than that found sufficient at the Proving Ground.

PLATING IN COMPRESSION

In 60-inch model No. 1, ultimate failure of the plating did not occur until the local stresses exceeded 30,000#/sq.in. although on account of concentration the nominal stresses were only about half as great.

In the CALIFORNIA concentrations in compression are well eliminated by the action of the 2nd and 3rd decks in absorbing the cantilever bending moment, although downward loads under the front point of the turret must be in part transmitted to structure under the 3rd deck, and certain concentrations will result. If the entire load came on these hard spots failure would be approached at nominal stresses of 16,000#/sq.in. A working stress at 12,000#/sq. in. would allow a definite margin even in this extreme case, which is further increased to an indefinite but certainly large amount by the action of the 2nd and 3rd decks.

A nominal stress of 12,000#/sq.in. leaves

$$12,000 - 1,700 = 10,300\#/sq.in.$$

for stress due to bending moments. This moment at the second deck is

$$4,140,000\# \times 26.3 \text{ ft} \times 12 \text{ in./ft.} = 1.31 \times 10^9 \text{ inch}\#.$$

and this requires a section modulus of

$$1.31 \times 10^9 \div 10,300 = 127,000 \text{ in.}^3$$

In a diameter of 28' this gives a plate thickness of

$$\frac{127,000}{.08 \times (28 \times 12)^2} = 1.40 \text{ inch}$$

instead of 1.25 inch as at present.

The compressive stress diminishes as the height above the 2nd deck increases. The height from the 3rd deck to the rollerpath casting would require 3 horizontal strakes; the two lower ones would be 1.40 inch, but the upper one, if 90 in. wide, could be reduced to a thickness of 1.12 inch.

PLATING IN SHEAR

Even allowing 1/2 the moment to be absorbed by the horizontal reaction at the 2nd and 3rd deck, the total reaction at the 2nd deck will amount to

$$4,140,000\# \times 33.07 \text{ ft.} \times 1/2 \div 7.77' = 8,800,000\#$$

This may require special attention to connections, but no undue shear stresses occur, since this load comes partly from above and partly from below. The reaction at the third deck under the assumed conditions would be:

$$4,660,000\# \div (2\pi \times 28 \times 12 \times 1.4) = 3,2000\#/sq.in.$$

In the lighter top strake it will be

$$4,140,000\# \div (2\pi \times 28 \times 12 \times 1.12) = 3,500\#/sq.in.$$

TURRET FOUNDATIONS FOR NEW SHIPS

The details which have been discussed are features which will control the choice of scantlings in a new design, but as those in the PENSIVOLA differ radically from those in the CALIFORNIA, so the details in the new design may differ from both.

The tests have been analyzed and inferences made, but these inferences cannot be regarded as finally established. The tests point the way, but do not tell how far to go. To obtain complete answers by experimental means for all the outstanding questions would prolong the tests unduly.

It is therefore suggested that when the occasion arises a new design for turret foundations be prepared, adapted as it must be to various necessities which cannot be foreseen; but that where strength of the foundation is a controlling feature an effort be made to work the material well towards its limits. After all details have been tentatively decided on, a scale model should then be constructed simulating all pertinent details of structure and loading as closely as possible. Test of this model will then indicate whether the design is adequate, and if not, what part needs to be reinforced.