R E P O R T O N

LINDA VISTA BRIDGE SITE

BY

LLOYD A WALLING

CALIFORNIA INSTITUTE OF TECHNOLOGY

PASADENA CALIFORNIA

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INFORMATION ON EXISTING BRIDGE

The Linda Vista Bridge spans the Arroyo Seco about a quarter of a mile above the Colorado Street Bridge, but serves an entirely different territory; as there is no road between them on the west bank. Los Angeles, Hollywood, and several of the beach cities can be reached by the way of the Colorado Street Bridge. The Linda Vista Bridge carries the traffic to the northwest of Pasadena, that is, Flintridge, Linda Vista, Montrose, Sunland. After leaving the bridge, the road follows the west bank of the Arroyo almost to the mouth of the canyon; then to the west along the foot of the mountains and into the San Fernando Valley.

The Linda Vista Bridge was constructed in 1910 by the County of Los Angeles. The traffic it carries is of an interurban character and only moderate at the present time; but will undoubtedly increase in volume as the population increases on the west bank of the Arroyo. Passenger cars and an occasional truck constitute the traffic over the bridge. The possibility of eliminating the bridge at this site was considered. This could be done by continuing the road along the west bank until it connects with the Colorado Street Bridge. However, the value of the land and certain topographical features would make this procedure costly.

The Linda Vista Bridge is of reinforced concrete beam and girder construction. There are six main spans of 51' 9" Five trestles support these main spans. They consist of two bents, spaced 17' 3" center to center. Each pair of bents is rigidly connected by reinforced concrete struts. The roadway is twenty feet wide, and has a two-inch asphalt surface laid on a four and one-half-inch slab. The girders are about nineteen feet apart, and a four foot sidewalk is cantilevered out on each side of the bridge from the girders. There are two stringers spaced six feet center to center. The floor beams are 17' 3" apart. There are two expansion joints each of which is located at the junction of one of the main spans and a trestle column. At these points the girder is not joined to the column, but rests on a ledge formed by increasing the size of the column. Iron plates provide the sliding surfaces. The maximum height of the roadway above the stream bed is about seventy feet. The trestle columns have been plastered over, so the quality of concrete in them could not be determined. However, the under side of the slab , floorbeams and stringers are exposed; and judging from these exposed surfaces the concrete was well mixed, and properly placed. Practically no honeycomb was visible.

INVESTIGATION OF EXISTING BRIDGE

In checking the stresses in the members the California State Highway Commission loadings were used. These are; 20 ton motor truck(6 tons on front axle, 14 tons on rear) or 125 lbs. per sq. ft. of road surface. This probably is somewhat larger than the bridge was designed for, but if the bridge is to continue in use, it should be able to stand the traffic new bridges are designed for. The commonly accepted stresses of: 650 lbs. per sq. in. for concrete in compression; 16000 lbs. per sq. in. for steel in tension and compression, and 40 lbs. per sq. in. for concrete in shear, were used. Bond stress was not considered, as in most cases sufficient information on the length of the steel was not available; so the assumption was made that the reinforcement was embedded far enough to develop the full strength of the steel.

The computations show that in all of the members either the concrete or steel is stressed over the allowable, and in some cases, both are. However, in practically all of the members the per cent the member is overstressed is quite small; and there is no cause for alarm. The weakest part of the structure is the road slab, which is far too light for the present truck traffic. Instead of a 4½"slab, it should be at least 8". That no failure has been noted

is probably due to the factor of safety used in the steel and concrete; and no truck of the kind used in the computations has probably ever crossed the bridge.

When even a medium weight passenger automobile crosses the bridge at a moderate rate of speed, the vibration is excessive. This is especially so on the spans where the expansion joints are located. So the vibration is probably due partly to the presence of the expansion joints, and partly to the relatively light construction of the trestles in comparison with their height. Altho they are strong enough to carry the loads, they have not enough weight to prevent vibration at the top.

In the original design, the expansion joint was supported by means of a corbel on the trestle column. The sectional area of this corbel at the base was about 200 sq. in. The end reaction of the girder is about 80,000 lbs., giving a shearing stress of 200 lbs. per sq. in. This weakness in the design was evidently noticed after the bridge was completed, as each of the columns which carry an expansion joint have been increased in size about six inches at the base, and up to about 18 feet from the top. From this point on up to the top it gradually increases still further in size until at the top it includes the corbel which protruded about 15 inches from the column. This makes a very

patchy looking job, but undoubtedly is an improvement as far as strength is concerned. One of the men in the engineering department of the City of Pasadena inspected the bridge, and reported that the end of one of the girders forming the expansion joint had cracked, thus reducing the bearing area considerably. The writer could not investigate this failure because of the extreme difficulty in getting to that particular point on the bridge.

The road surface of the existing bridge is on a grade of approximately 3%, and there is a steep grade to the east and in a direct line with the bridge. This causes the traffic to go over the bridge at a higher rate of speed than would ordinarily be the case. In approaching the bridge from the east motorists coast down the hill, and are usually going about 30 miles per hour by the time they reach the bridge. In crossing the bridge from the west they speed up in order to make the hill on the other side in high gear. This makes a very dangerous junction at the east end of the bridge, where a road branches off to Brookside Park in the Arroyo Seco. This road also is on a fairly steep grade.

Summing up the results of the investigation, we reach the following conclusions. All of the members of the bridge are overstressed to some extent, but not enough to warrant

condemnation. But in considering the floor slab we find cause for alarm. As noted before, it is far too light to withstand the traffic of today. The vibration is excessive and this causes a very serious condition. It puts an additional strain on the members, and one which cannot be accurately computed. Finally, the reduction of the effective bearing surface at the expansion joint is still another weak spot which should be eliminated. It is certainly advisable to limit the speed on the bridge to ten miles per hour, as is being done; and to build a modern structure at this point as soon as possible.

EARTH FILLED ARCH

As the main trouble with the present bridge seemed to be excessive vibration, a brief study was made to see if some means of rebuilding the bridge could be devised to eliminate this. We will see if it would be practical and economical to use the old bridge as a skeleton for a new one. With this in view, the earth-filled arch was first considered. It was intended that the two trestles containing the expansion joints should be eliminated, leaving two main spans of approximately 120 feet and a short span of 52 feet adjoining each abutment. The remaining three trestles could be made into piers of sufficient section to sustain the thrust of the arches. One of the plates attached to this report illustrates how the proposed arch bridge would appear when substituted for the present bridge. The present slab would be taken out, and the arch filled with earth to the present grade, and a new road surface put on.

In this way, the present site could be utilized, and three of the trestles would serve as skeletons for the piers. Some means could probably be devised to support, at least partially, the forms for the arch.

These few advantages are more than offset by the following disadvantages. By this method we are building a new bridge, and yet forcing clumsy-looking piers by trying to

cover up the old bridge trestles. Some difficulty would be experienced in pouring the arch if the present bridge continued in use during construction. On the other hand, if the roadway is torn out to facilitate construction, we might as well tear it down completely and design a new bridge without having the fixed location of the piers. The volume of the trestle is small in comparison with the size of the pier which would be necessary. The sidewalk should be replaced by a wider and stronger one; and a more artistic railing substituted. In taking even these few points into consideration, it can be seen that in attempting to incorporate the present bridge in with a new earth fill arch very little would be gained. The yardage of concrete saved would be small in comparison with the total required for the arch. It would also be rather difficult to make the design of this bridge harmonize with the artistic Colorado Street Bridge, only a short distance from it.

CANTILEVERS

The possibility of constructing cantilevers to support the present bridge was next considered. The cantilevers would extend out from piers as shown in the accompanying sketch. These piers would be the same as for the earth-filled arch previously considered. The bridge would have very much the same appearance whether rebuilt as an arch or as a cantilever. The results of the computations show that it would be impossible to put the amount of steel required in a beam of reasonable dimensions. The present girder would have to be embedded in the upper part of the cantilever greatly reducing the space for steel.

Even if it were feasible and economical to eliminate the present expansion joints and substitute cantilevers for the girders, we would still have the problem of strengthening the floor slab which is the weakest part of the structure. This could not be done without putting in new beams, girders and slabs. If all this is done practically a whole new bridge has been built. Such being the case, the present bridge might as well be torn down and a new one designed to replace it.

PROPOSED SPANDREL ARCH TO REPLACE EXISTING BRIDGE

It has been shown that it would not be feasible to attempt to reenforce the present bridge, so we will consider in a general way the features of a new bridge to span the Arroyo Seco at this point.

A spandrel arch was chosen as it could be modeled along lines similar to the Colorado Street Bridge which is one of the most artistic bridges in the United States. Also for the length of span which was chosen a spandrel arch would be the most economical. By changing the location slightly, it was found that the span of the new bridge could be made about sixty-five feet less than the present one. As shown on the sketch, the west end of both the proposed and existing bridges coincide, but the center line of the new one has been swung to the north with reference to the center line of the existing bridge. A profile taken along the center line of the arch shows that an arch of very pleasing proportions can be designed to span the Arroyo at this point. A drawing shows this profile and the proposed arch. At each of the points where the abutment is to come there is an outcropping of solid rock so there is assurance that no trouble will be encountered in obtaining a suitable foundation for the arch.

The grade on the arch will be about 2.5%, whereas, on the present bridge it is a little over three per cent. The danger-

ous, steep, straight-away approach to the east end will be eliminated.

It is planned to have a twenty-five-foot roadway and a six-foot sidewalk on each side. Columns are spaced twenty-three feet, center to center, and floor beams 11'6" center to center. Stringers at the third points of the roadway. The clear span of the arch is 224 feet and a 42-foot rise. The thickness at the crown was taken as five feet, and at the springing line 6' 2", the increase to be uniform from the crown to the springing line. There are to be two arch rings spaced approximately twenty-five feet apart.

The sidewalks will be supported by cantilevers from the girders. The arch could be constructed without discontinuing the traffic over the existing bridge except when converting the roadway to the west bank. When this was being done the most westerly span of the existing bridge could be torn down and a temporary wooden structure substituted which would carry the traffic around the west abutment. This would very easily solve the problem of taking care of the traffic to Linda Vista while the new bridge was being constructed

The method used for the preliminary solution of the arch ring was taken from the Engineering News Record of March 6, 1919. Each half of the arch was treated as a cantilever and the thrust, moment, and sheer obtained at the end of the cantilever or crown of the arch. The moment at any other point can then be obtained by adding to the moment at the crown the moment of all the

intermediate loads about the point. No attempt has been made to design the arch but a table has been compiled for the arch ring chosen, which gives the coefficients for moment, thrust and shear at the crown caused by a load of unity at twenty points on the arch ring. After obtaining these coefficients, the bridge can be loaded in any manner desired and H, M, and V at the crown easily computed.

It has been the intention in this report to first investigate the existing bridge and determine its points of weakness. Then to suggest any changes in the present bridge or select the general characteristics of a bridge to replace it.

COMPUTATIONS

All computations based on the following assumptions:-

Weight of asphalt..... 80 lbs. per cu. ft.

Weight of concrete.....150 lbs. per cu. ft.

Allowable stress in steel(tens.&comp) 16000 lbs. per sq. in.

" " concrete(compression) 650 " " " "

" " " (shear) 40 " " " "

Live load(California Highway Commission)
Uniform load.....125 lbs. per sq. ft.
20 ton motor truck-14 T rear axle 6 T front axle
Wheelbase--12 ft. Gage--6ft. Tread--18in.

DEAD LOAD ON STRINGERS(BEAM#1)

Asphalt 2 in. thick.

$\frac{74 \times 2 \times 12 \times 80}{1728}$ .....82 lbs. per lin. ft.

Slab 4.5 in. thick.

$\frac{74 \times 4.5 \times 12 \times 150}{1728}$ .....346.. " " " "

Beam Sec. area 120 sq. in.

$\frac{120 \times 12 \times 150}{1728}$.....125 " " " "

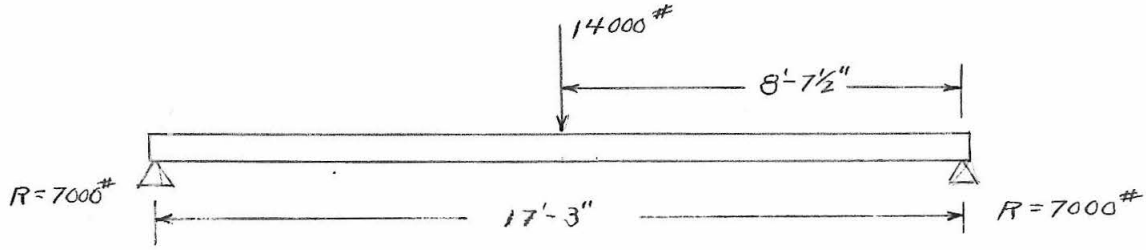
Total 553 lbs. per lin. ft.

LIVE LOAD(BEAM#1)

Uniform L.L. 125×6 .....750 lbs. per lin. ft.

Moment for a 17′ 3″ span

$M = 1/8\ wl^2 = \frac{750 \times 17.25^2 \times 12}{8}$ 335,000 in. lbs.



R_L = 7000 lbs.

$M_{max} = 7000 \times \frac{17.25}{2} = 60{,}400$ ft. lbs.

$= 724{,}000$ in.lbs.

Increasing 30% for impact

$M = 724{,}000 \times 1.3 = 940{,}000$ in.lbs.

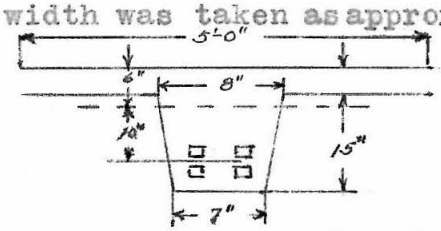
Since truck load gives largest moment on this beam, this moment will be used in the investigation of this beam.

$M(\text{dead load}) = \frac{553 \times 17.25^2}{8} = 20{,}550$ ft. lbs.

$= 247{,}000$ in. lbs.

Because of the fact that the beams have a trapezoidal section it was thought better to use the method of statical moments about the neutral axis rather than use cumbersome formulae derived for this type of beam.

T beam conditions were assumed where the slab thickness was not less than one-third the depth of the beam. The overhanging slab width was taken as approximately six times the slab thickness.



Assume k = .375

4-1 in. sq. bars

¼ ϕ stirrups

$I_c = (60 \times 6^3 \times \frac{1}{3}) - (52 \times 1.5^3 \times \frac{1}{3})$

$= 4320 - 58.4 \qquad = 4262$

$$nI_s = 15 \times 4 \times 10^2 = \frac{6000}{10262 \text{ in.}^4}$$

$$M(\text{dead} + \text{live}) = 940{,}000 + 247{,}000 = 1{,}187{,}000 \text{ in. lbs.}$$

$$f_c = \frac{Mc}{I} = \frac{6 \times 1{,}187{,}000}{10262} = 690 \text{ lbs. per sq.in.}$$

$$f_s = \frac{nMc}{I} = \frac{15 \times 1{,}187{,}000 \times 11}{10262} = 19{,}000 \text{ lbs. per sq.in.}$$

Concrete overstressed about six per cent

Steel overstressed about nineteen per cent.

Max. shear occurs with wheel at end of beam.

$$R = 14{,}000 + \frac{5.25 \times 6000}{17.25} \doteq 15{,}825 \text{ lbs.}$$

$$s_s = \frac{15825}{7.5 \times 20} = 105 \text{ lbs. per sq. in.}$$

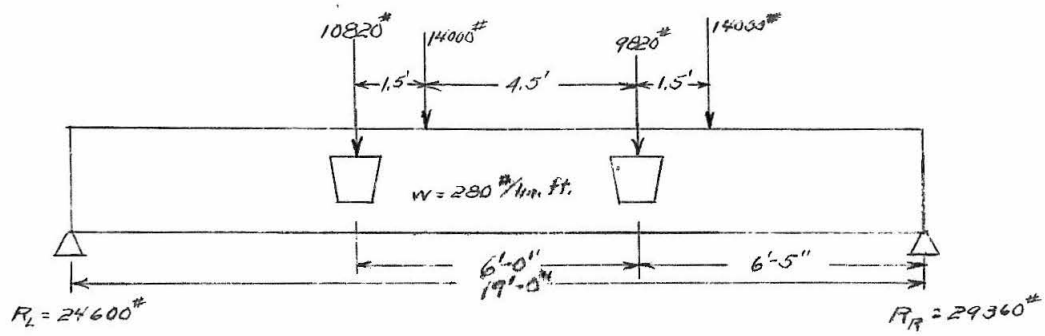
This is excessive so we will investigate the vertical stirrups.

$$s = \frac{3}{2} \times \frac{a_s f_s jd}{V} = \frac{3}{2} \times \frac{.05 \times 16000 \times .875 \times 16 \times 4}{15{,}825} \doteq 4.25 \text{ in.}$$

The plans did not show the spacing of the bars but even if it was in excess of the above the bent up bars would probably bring the shearing stress within safe limits.

FLOORBEAM(Beam #2)

Maximum condition of loading shown below.



Stringer reaction due to dead load.

$553 \times 17.25 \qquad = 9500$

Stringer reaction due to front wheels.

Left or right $\quad (6000 \times \frac{4.5}{6})\frac{5}{17} \qquad = 1320$

$\overline{10,820}$

$$R_L = \frac{(14000\times4.5)+(10820\times6.5)+(14000\times11)+(10820\times12.5)+(280\times19\times9.5)}{19}$$

$= 24,600$ lbs.

$$M(\text{center}) = (24600\times9.5)-(14000\times1.5)-(10820\times3)-\left(\frac{280\times19^2}{8}\right)$$

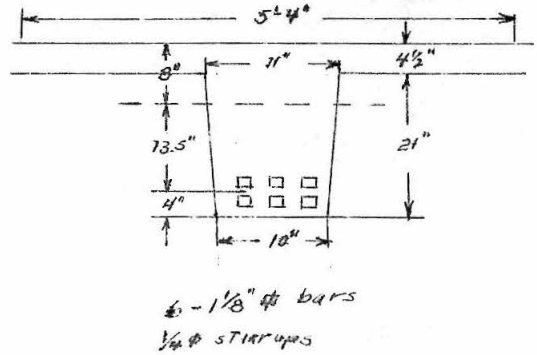
$= 233500 - 21000 - 32460 - 12600$

$= 167400$ ft. lbs.

$= 2,008,800$ in.lbs.

Increasing the total moment 15% for impact which is about the equivalent of increasing the live moment 30% gives

$M(\text{max}) = 2,008,800\times1.15 = 2,310,000$ in.lbs.



$$I_c = \left(64 \times 8^3 \times \frac{1}{3}\right) - \left(51 \times 3.5^3 \times \frac{1}{3}\right)$$

$$= 10920 - 728$$

$$= 10192$$

$$nI_s = 15 \times 6 \times 1.26 \times 13.5^2 = 20,650$$

$$I = 10192 + 20650 = 30842 \text{ in.}^4$$

$$f_c = \frac{8 \times 2310000}{30842} = 600 \text{ lbs. per sq. in.}$$

$$f_s = \frac{15 \times 15 \times 2310000}{30842} = 16,900 \text{ lbs. per sq.in.}$$

Steel overstressed about 5.5 per cent.

Shear:

Maximum occurs with one wheel one foot from end.

$$R = \frac{14000(12 \times 18)}{19} = 22,100 \text{lbs.}$$

$$W = 280 \times 9.5 = 2,660$$

$$R(\text{live} + \text{dead}) = 24,760 \text{ lbs.}$$

$$s_s = \frac{24760}{270} = 91.5 \text{ lbs. per sq. in.}$$

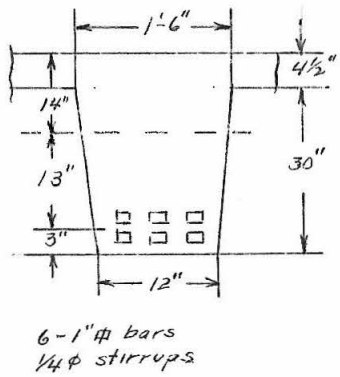
This is excessive so we will investigate the vertical stirrups.

1/4 rd. stirrups bent thus:

$$s = \frac{3}{2} \times \frac{a_s f_s jd}{V} = \frac{3}{2} \times \frac{4 \times .05 \times 16000 \times .875 \times 21.5}{24760} = 5.5 \text{ in.}$$

No spacing shown on plans but it problably was about 6in. and if any bars were bent up, the shearing stress was within safe limits.

FLOORBEAM(Beam #5)



$$p = \frac{a_s}{bd} = \frac{6\times1}{486} = .0123$$

$$k = \sqrt{2pn + (pn)^2} - pn$$

$$= \sqrt{2\times.0123\times15+(.0123\times15)^2}-.0123\times15$$

$$= .45$$

$$I_c = (18\times14^3\times\frac{1}{3}) - (2\times9.5^3\times\frac{1}{3})$$

$$= 16500-572$$

$$= 15928$$

$$nI_s = 15\times6\times13^2 = 24210$$

$$I = 15928 + 24210 = 40138 \text{ in.}^4$$

Weight of beam per foot $= \frac{405\times150}{144} = 420$ lbs. per foot.

Live load same as for Floorbeam #2

$$M(\text{center}) = (25930\times9.5)-(14000\times1.5)-(10820\times3) - \frac{420\times19^2}{8}$$

$$= 246000-21000-32460-18900$$

$$= 173640 \text{ ft. lbs.}$$

$$= 2,080,000 \text{ in. lbs.}$$

Increasing 15% for impact.

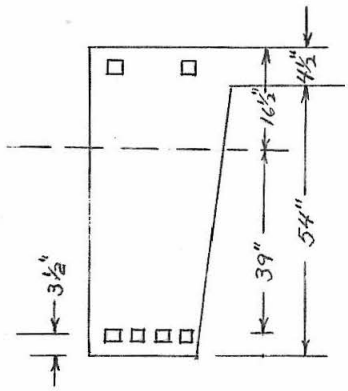
$$M(\text{max}) = 2080000\times1.15 = 2,395,000 \text{ in. lbs.}$$

$$f_c = \frac{14\times2395000}{40138} = 830 \text{ lbs. per sq. in.}$$

$$f_s = \frac{15\times17.5\times2395000}{40138} = 15700 \text{ lbs. per sq. in.}$$

Concrete overstressed about 27%

BEAM #4



6 #1 □ bars

Assume k = .30

$$I_c = \left(\frac{20 \times 16.5^3}{3}\right) + \left(\frac{4 \times 16.5^3}{4}\right) = 29900$$

$$I_{sc} = 15 \times 2 \times 14.5^2 = 6300$$

$$I_s = 15 \times 4 \times 39^2 = 91500$$

$$I = 127700 \text{ in.}^4$$

Load on beam

Weight of beam	=	1070 lbs/lin.ft.
Dead load(slab)	=	243 "
Live load	=	475 "
Total	=	1788 "

$$M = 1/10\ wl^2 = 1/10 \times 1788 \times 17.25^2 \times 12 = 637{,}000 \text{ in.lbs.}$$

$$f_c = \frac{M\ c}{I} = \frac{637000 \times 16.5}{127700} = 82.5 \text{ lbs./sq. in.}$$

$$f_s = \frac{15 \times 637000 \times 39.5}{127700} = 2950 \text{ lbs/sq. in.}$$

8

GIRDER (Beam #5)

Loading used 125 lbs. per sq. ft. of road surface and 50 lbs. per sq. ft. of sidewalk. 20 ft. roadway; 4 ft. sidewalk.

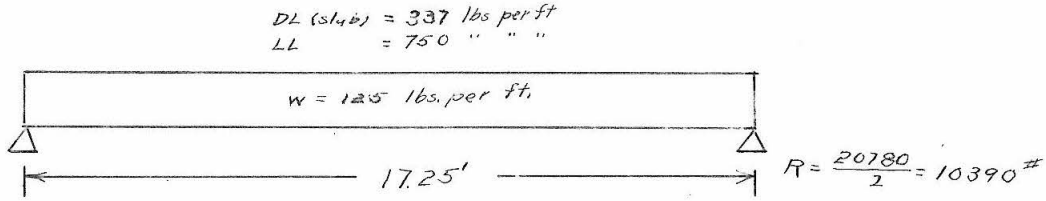
Stringer reaction

Weight of stringers $= 125 \times 16.5 = 2060$ lbs.

" " slab $= \frac{4.5 \times 12 \times 72}{1728} \times 17.25 \times 150 = 5820$ lbs.

Live load $= 125 \times 6 \times 17.25 = 12{,}900$ lbs.

The effect on the third point of the floorbeam would be the sum of the three quantities or 20,780 lbs.



Floorbeam reaction

Weight of floorbeam $= 280 \times 19 = 5300$ lbs.

R (floorbeam) $= 20780 + \frac{5300}{2} = 23{,}430$ say 23,500 lbs.

Weight carried direct to girder by road slab.

Weight of slab $= \frac{4.5 \times 12 \times 36}{1728}\ 150 = 168$ lbs. per lin. ft.

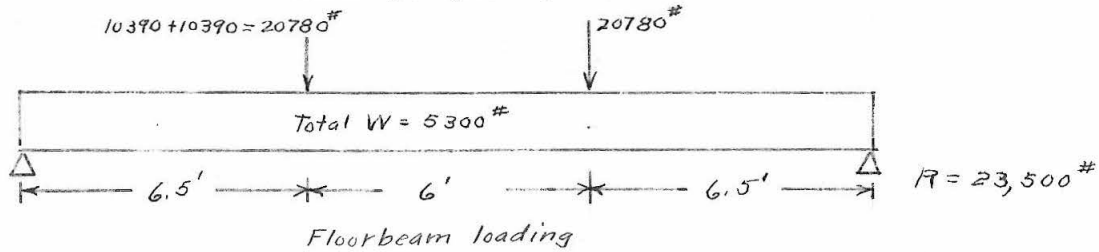
Live load $= 125 \times 3 = 375$ " " " "

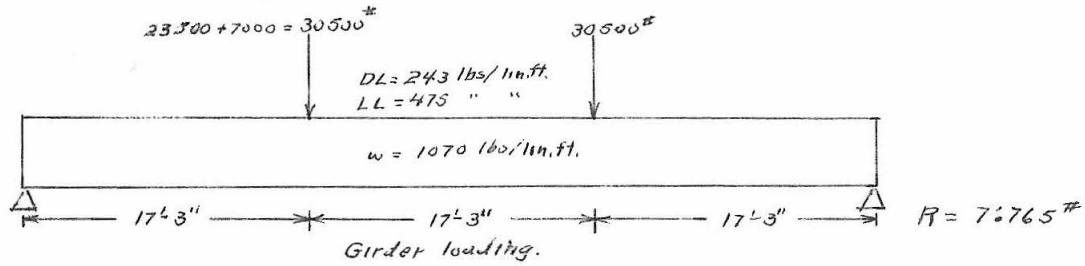
Sidewalk cantilever reaction = 7000 lbs. (See page 15)

Weight of sidewalk slab carried by girder direct = 75 lbs. lin. ft.

Live load " " " " " " = 100 " " "

BEAM #5 (con.)





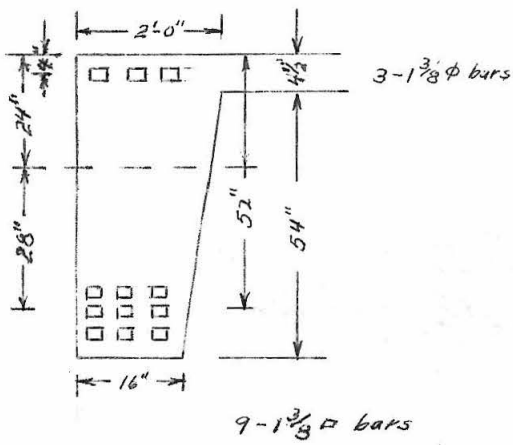
$$M\ (\text{center}) = 76765 \times \frac{51.75}{2} - 30500 \times \frac{17.25}{2} - 1788 \times \frac{51.75}{2} \times \frac{51.75}{4}$$

$$= 1{,}980{,}000 - 263{,}000 - 598{,}000$$

$$= 1{,}119{,}000 \text{ ft. lbs.}$$

$$= 13{,}420{,}000 \text{ in. lbs.}$$

. . . . . .



An approximate value for k will now be determined by using the rectangular beam formulae.

$$bd = 16 \times 52 + \frac{8 \times 48}{2} = 1024 \text{ sq. in.}$$

$$p' = \frac{3 \times 1.48}{1024} = .00433$$

$$p = \frac{9 \times 1.89}{1024} = .0166$$

$$\frac{d'}{d} = \frac{4}{52} = .077$$

BEAM #5(con.)

$$k = \sqrt{2n(p + p'\frac{d'}{d}) + n^2(p+p')^2} - n(p+p')$$

$$= \sqrt{2\times15(.0166+.00433\times.077)+15^2(.0166+.00433)^2} - 15(0166+.0433)$$

$$= .463$$

$$.463\times52 = 24\text{in.}$$

$I_c = (\frac{20\times24^3}{3}) + (\frac{4\times24^3}{4})$	$= 92160 + 13824$	=	105984
$I_{sc} = 15\times3\times1.49\times20^2$		=	26800
$nI_s = 15\times9\times1.89\times28^2$		=	200000
	I of section	=	332,784

$$f_c = \frac{M\,c}{I} = \frac{13420000\times24}{332784} = 970 \text{ lbs. per sq. in.}$$

$$f_{sc} = \frac{15\times13420000\times20}{332784} = 12100 \text{ lbs. per sq. in.}$$

$$f_s = \frac{15\times13420000\times30}{332784} = 18100 \text{ lbs. per sq. in.}$$

Shear (Beam #5)

Maximum at end reaction.

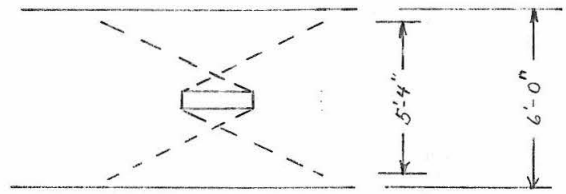
$$s = \frac{76765}{1024} = 75 \text{ lbs. per sq. in.}$$

3/8 rd. stirrups bent thus:

$$s = \frac{3}{2}\times\frac{a_s\times f_s\times jd}{V} = \frac{3}{2}\times\frac{4\times.11\times16000\times.85\times52}{76765} = 6.1 \text{ in.}$$

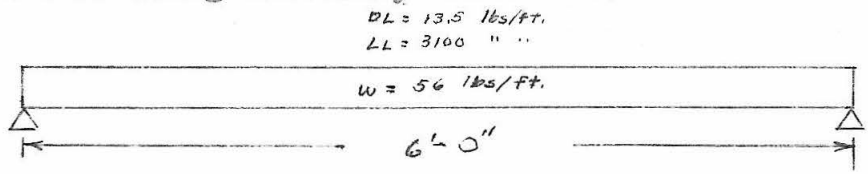
ROAD SLAB

Surface consists of 2 in. of asphalt. Maximum moment occurs with seven ton wheel load at the center of the 6 ft. span. But this need not be taken as a concentrated load. The diagram below shows the distribution.



$e = .6\,s + 1.7 = 5.3$ ft.

As this very nearly approximates the span, we will assume the truck load as being uniformly distributed.



Weight of asphalt $= \frac{2 \times 12 \times 12}{1728} \times 80 = 13.5$ lbs. per sq. ft.

" " slab $= \frac{4.5 \times 12 \times 12}{1728} \times 150 = 56$ " " " "

Live load $= \frac{14000}{6} \times 1.33$ (impact) $= 3100$ lbs. per lin. ft.

Total load = 3180 lbs. per lin. ft.

$M = \frac{1}{12} wl^2 = \frac{1}{12} \times 3180 \times 6^2 \times 12 = 114500$ in. lbs.

Reinforcement

3/8" ⌀ @ 9 c. c. = .19 sq. in. per ft.

$p = \frac{.19}{3.5 \times 12} = .0045$

ROAD SLAB (con.)

$M_s = pf_s jbd^2$

$114,500 = .0045 \times f_s \times .875 \times 12 \times 4^2$

$f_s = 151,000$ in. lbs.

This indicates a very critical condition. That there has been no failure thus far is problably due to several causes. The twenty ton truck used in the computations is larger than those in actual use and in the slab as constructed the steel might have been spaced closer than the plans show.

SIDEWALK SLAB

3 in. slab with $\frac{1}{4}$ bars @ 9in. c.c. 4 ft. span.

We will investigate for a uniform load of 75 lbs. per sq. ft.

The weight of the slab is 37.5 lbs. per sq. ft.

$M = 1/10\ wl^2 = 1/10 \times 112.5 \times 4^2 \times 12$

$= 2,160$ in. lbs.

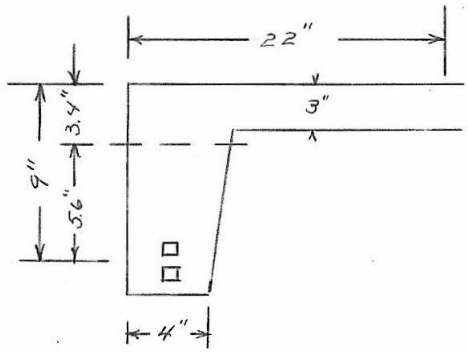
$p = \frac{.067}{2.25 \times 12} = .0025$

$f_s = \frac{M}{pjbd^2} = \frac{2160}{.0025 \times .875 \times 12 \times 2.25^2} = 16,300$ lbs. per in.^{2}

SIDEWALK BEAM

17.25 ft. span 2-$\frac{1}{2}$" ⌀ bars

Assume k = .375



2-½"□ bars

Loads

Weight of rail 4×3'-6" = 175 lbs/lin. ft.

" " beam

$\frac{(4\times10.5+5)}{1728} \times 12\times150 = 50$ "

Weight of slab

$\frac{3\times12\times12\times150\times2}{1728} = 75$ "

Live load = 150 "

Total 450

$I_c = (22\times3.4^3\times1/3)$

$= 290 \text{ in.}^4$

$I_s = 15\times.5\times5.6^2$

$= 236 \text{ in.}^4$

$I = 290+236 = 526 \text{ in.}^4$

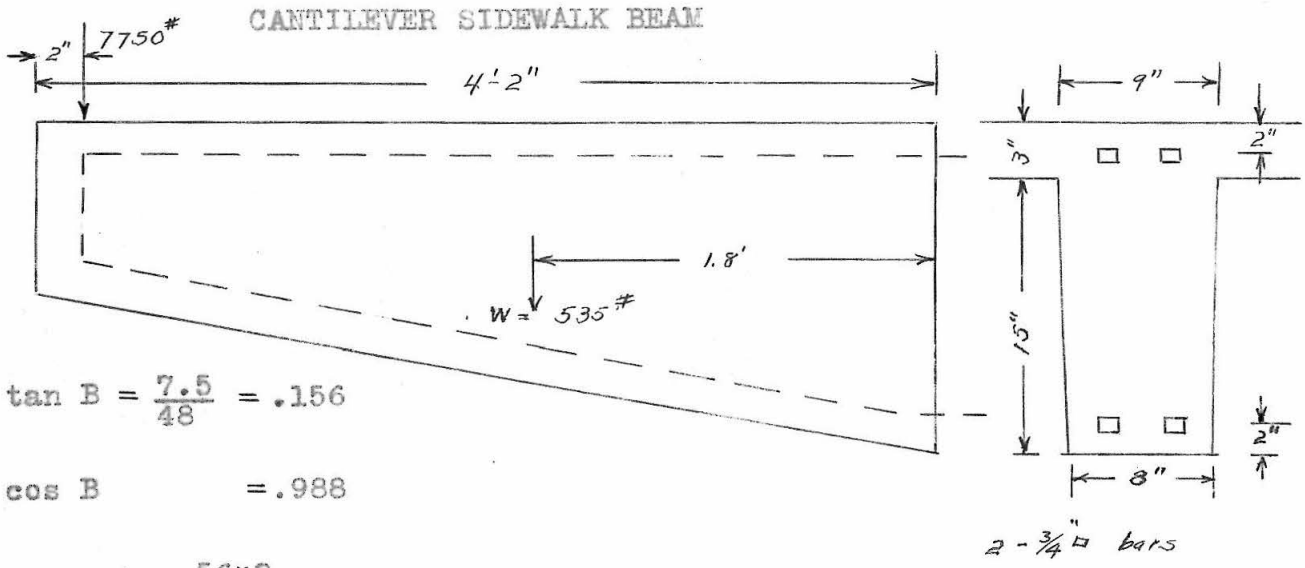
$M = 1/10\ wl^2 = 1/10\times450\times17.25^2\times12 = 165{,}000$ in. lbs.

$f_c = \frac{Mc}{I} = \frac{165000\times3.4}{526} = 1060$ lbs/in.2

$f_s = \text{"} = \frac{165000\times6\times15}{526} = 28{,}000$ lbs/in.^{2}

Shear $= \frac{450\times8.625}{.875\times4\times11} = 100$ lbs/in.^{2}

CANTILEVER SIDEWALK BEAM



$$\tan B = \frac{7.5}{48} = .156$$

$$\cos B = .988$$

$$p = p' = \frac{.56 \times 2}{155} = .007$$

$$M_s = bd^2 f_s K \qquad M_c = bd^2 fcL \qquad M_s = bd^2 f_s L$$

$$k = \sqrt{2n(p + p'\frac{d'}{d}) + n^2(p+p')^2} - n(p+p')$$

$$= \sqrt{2 \times 15(.007 + .007\ \frac{2}{16.5}) + 15^2(.014)^2} - 15 \times .014$$

$$= .318$$

$$K = p(1 - \frac{d'}{d}) - \frac{k^2}{2n(1-k)} \quad (\frac{k}{3} - \frac{d}{d})$$

$$= .007(1 - \frac{2}{16.5}) - \frac{.318^2}{30(1 - .318)} \quad (\frac{.318}{3} - \frac{2}{16.5})$$

$$= .00622$$

$$L = \frac{k}{2}(1 - \frac{k}{3}) + \frac{np'}{k}(k - \frac{d'}{d}) \quad (1 - \frac{d'}{d})$$

$$= \frac{.318}{2}\ (1 - \frac{.318}{3}) + \frac{15 \times .007}{.318}\ (.318 - \frac{2}{16.5})\ (1 - \frac{2}{16.5})$$

$$= .199$$

CANTILEVER SIDEWALK BEAM (con)

$$L' = p \times \frac{1-k}{k-\frac{d'}{d}}\left(1-\frac{k}{3}\right) + p'\left(\frac{k}{3}\cdot\frac{d'}{d}\right)$$

$$= .007\ \frac{1-.318}{.318-\frac{2}{16.5}}\ \left(1-\frac{.318}{3}\right) + .007\left(\frac{.318}{3}\cdot\frac{2}{16.5}\right)$$

$$= .022$$

Weight of beam

$a = 1/2 \times 7.5(8+9)+27 \qquad = 91$

$b = 1/2 \times 15(8+9)+27 \qquad = 155$

$\overline{246}\ 2 \qquad 123$ sq. in.

$$W = \frac{123 \times 50}{1728} \times 150 = 535 \text{ lbs.}$$

Load from sidewalk beam(live + dead) = 450×17.25 = 7750 lbs.

$$M = [(7750 \times 4) + (535 \times 1.8)]\ 12 = 383{,}000 \text{ in. lbs.}$$

$$f_s = \frac{383{,}000}{8 \times 18\tfrac{1}{2} \times .00622} = 23{,}700 \text{ lbs/in.}^2$$

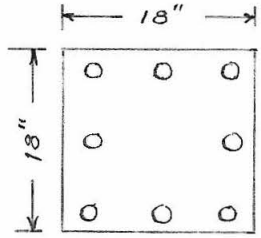
$$f_c = \frac{383{,}000}{8 \times 18\tfrac{1}{2}^2 \times .199} = 740 \text{ lbs/in.}^2$$

$$f_s = \frac{383{,}000}{8 \times 18\tfrac{1}{2}^2 \times .022} = 6{,}700 \text{ lbs/in.}^2$$

Shear at end of cantilever.

$$s = \frac{7750}{.875 \times 8 \times 9} = 125 \text{ lbs/in.}^2$$

TRESTLE COLUMNS



8-1¼"φ bars
Ties 15"c.c.

Total load per column

Beam #3		25930 lbs.
" #4		15400
" #5		76765
	Total	118095 lbs.

Allowing 1 1/2" for fireproofing

$A = 15 \times 15 = 225 \text{ in.}^2$

$A_s = 8 \times 1.25 = 9.84 \text{ in.}^2$

$p = 9.84/225 = .0437$

Assume $f_c = 450 \text{ lbs/in.}^2$ $\qquad n = 15$

$P = 450 \times 225 = 101{,}000 \text{ lbs.}$

$\frac{P'}{P} = 1 + (n-1)p = 1 + (15-1) \times .0437 = 1.61$

$P = 101000 \times 1.61 = 163{,}000 \text{ lbs.}$

The following computations were made to determine if it was feasible so far as the design was concerned to construct cantilevers under the main spans, and thus make a substantial bridge of the present structure. The intention was to make massive piers of some of the trestles by filling them with concrete; the others were to be eliminated as shown on the drawing. Those eliminated contained the expansion joint which weakened them considerably. The present girders were to remain and become an integral part of the cantilever. The cantilever on each side of the road to take the entire load. The top of the cantilever was to conform to the present level of the top of the girders, the lower edge to be the circumference of a circle 150 ft. R. Each cantilever to be three feet in thickness, and the depth at the center was chosen so as to be about 1' 6" below the present girder in order to facilitate construction.

Length of cantilever 58 ft.

Depth at center 18 1/2"+54"+4 1/2" = 6'5" or say 6'4"

This cantilever was then laid out to scale and areas obtained from which the total weight of the beam was computed.

Total weight 254,000 lbs.

Computation of Bending Moment at pier.

Live load

125 lbs/ft^2 on 10ft. of roadway		1250 lbs/lin. ft.
75 " on 4 ft. of sidewalk		300 "
	Total	1550 "

Dead Load

Slab 56 lbs/ft^2	560 lbs/lin. ft.
Road surface 13 lbs/ft^2	130 "
Stringers	125 "

Cantilever (con)

Girder		Total	1070 lbs/lin. ft.
			1885 "
Total uniform load	1550	1885	3435 or say 3500 lbs/lin.ft.

$$M_{max} = \frac{3500 \times 59^2}{2} + \frac{254000 \times 59}{3} = 133{,}000\ 000 \text{ in. lbs.}$$

$$a_s = \frac{M}{f_s jd} = \frac{133{,}000{,}000}{16000 \times .875 \times 193} = 49.2 \text{ sq. in.}$$

We need go no further with this solution as it would not be desirable to place 49 sq. in. of steel in a beam 3 ft. wide. We could increase the size of the beam, but this would greatly increase the cost as well as make the construction more difficult.

SPANDREL ARCH

Clear span 224 ft.

Span of parbola on center line of major axis 230 ft.

Rise of arch (1/2 minor axis) 42 ft.

There are to be two spandrel arch rings supporting a 25 ft. roadway and a 6 ft. sidewalk on each side.

Computation of coordinates for plotting.

$x^2 = ay$ when $x = 115$; $y = 42$

$a = \frac{115^2}{42} = 314.88$

$x^2 = 314.88y$

At crown $d = 5$ ft. At springing line $d = 6.16$ ft.

At intermediate points $d = 5 \frac{x}{1} 1.16$

x = 0	11.5	23.0	34.5	46.0	57.5	69.0	80.5	92.0	103.5	115
y = 0	.42	1.68	3.78	6.72	10.5	15.1	20.6	26.9	34.0	42.
d=5.0	5.12	5.23	5.35	5.46	5.58	5.70	5.81	5.93	6.04	6.16

The arch ring was then laid out to scale and the points of the column load located. The moment of inertia at these points and at points midway between were then computed. This moment of inertia is of a section of the arch ring taken at right angles to a tangent to the curve passing thru the mid-points of the arch ring. The sections were taken 11.5 ft. apart horizontally. "s" represents the distance between sections measured along the centerline of the arch ring. The width of the arch ring "b" was taken as 3.75 ft. for the first trial. Columns to be 32" 32"

SPANDREL ARCH (con)

All distances in the following table are in feet.

	1	2	3	4	5	6	7	8	9	10
d =	6.10	5.98	5.87	5.76	5.64	5.52	5.40	5.29	5.18	5.06
I =	70.93	66.83	63.21	59.72	56.06	52.56	49.21	46.26	43.43	40.45
s =	14.0	13.5	13.16	12.75	12.5	12.16	11.92	11.67	11.58	11.54
$\Delta = \frac{s}{I}$ =	.197	.202	.208	.213	.223	.231	.242	.252	.267	.285

The following table was computed according to the method shown in the Engineering News Record, March 6th, 1919.

	1	2	3	4	5	6	7	8	9	10	11	12	13
	X	Y	Δ	$\Sigma_0^n \Delta$	$[\Delta]$	$Y\Delta$	$\Sigma_0^n Y\Delta$	$[Y\Delta]$	$X\Delta$	$\Sigma_0^n X\Delta$	$[X\Delta]$	$X^2\Delta$	$Y^2\Delta$
1	109.25	37.9	.197	.197	0	7.47	7.47	0	21.5	21.5	0	2351.3	283.0
2	97.75	30.3	.202	.399	.197	6.12	13.59	7.47	19.7	41.2	21.5	1930.1	185.5
3	86.25	23.6	.208	.607	.596	4.91	18.50	21.06	17.9	59.1	62.7	1547.3	115.8
4	74.75	17.7	.213	.820	1.203	3.77	22.27	39.56	15.9	75.0	121.8	1190.2	66.7
5	63.25	9.5	.223	1.043	2.023	2.12	24.39	61.83	14.1	89.1	196.8	892.1	20.1
6	51.75	8.5	.231	1.274	3.066	1.96	26.35	86.22	12.0	101.1	285.9	618.6	16.7
7	40.25	5.1	.242	1.516	4.340	1.23	27.58	112.57	9.7	110.8	387.0	392.1	6.3
8	28.75	2.6	.252	1.768	5.856	.66	28.24	140.15	7.2	118.0	497.8	208.3	1.7
9	17.25	.94	.267	2.035	7.624	.25	28.49	168.39	4.6	122.6	615.8	79.5	.2
10	5.75	.11	.285	2.320	9.659	.03	28.52	196.88	1.6	124.2	738.4	9.4	0
C					10.819			211.14			800.5		
Σ			2.32			28.52			124.2			9218.9	696.0

	14	15	16	17	18	19	20	21	22
	$[Y\Delta]\Sigma\Delta$	$[\Delta]\Sigma Y\Delta$	$Q = 14-15$	$H_U = S\frac{Q}{K}$	$S[\Delta]$	$2H_U\Sigma Y\Delta$	$R = 18-19$	$M_U = \frac{R}{2\Sigma\Delta}$	$V_U = \frac{[X\Delta]}{2\Sigma X^2\Delta}$
1									
2	17.3	5.6	11.7	.028	2.3	1.6	.7	.015	.0134
3	48.8	17.0	31.8	.075	6.8	4.3	2.5	.54	.0391
4	91.8	34.3	57.5	.136	13.8	7.8	6.0	1.29	.076
5	143.4	57.7	85.7	.203	23.3	11.6	11.7	2.52	.123
6	200.0	87.4	112.6	.267	35.3	15.2	20.1	4.33	.178
7	261.2	123.8	137.4	.326	49.9	18.6	31.3	6.74	.241
8	325.1	167.0	158.1	.375	67.3	21.4	45.9	9.89	.311
9	390.7	217.4	173.3	.411	87.7	23.4	64.3	13.85	.384
10	456.8	275.5	181.3	.430	111.1	24.5	86.6	86.6	.461
C	489.8	308.6	181.2	.429	124.4	24.4	100.0	21.55	.500

$2\Sigma\Delta = 2 \times 2.32 = 4.64$

$2\Sigma X^2\Delta = 2 \times 9219 = 18.438$

$K = 2\left[(696 \times 2.32) + 28.52^2\right] = 4856$

$S = 11.5 \qquad S/K = .00237$

LOADS ON ONE ARCH RING

Live load

125 lbs/ft.^{2} on 12.5 feet of roadway	1562	lbs/lin.ft.
100 " " " 6 " " sidewalk	600	"

Dead load

8" slab - 12.5 ft. wide @ 150 lbs/cu.ft.	1250	lbs/lin.ft.
Inside stringer 8"×18" section	150	"
Outside " 24"×30" "	750	"
5" sidewalk slab	375	"
Balustrade	47	"
Coping, lamp-posts, etc.	403	"
Total (dead & live)	5137	"
say	5150	"

Loads at each column

Floorbeam (weight of 1/2 span)	3500	lbs.
Sidewalk cantilever beam	1000	"
Sidewalk beam (100 lbs/lin.ft.)	2300	"
Total	6800	"

Total load on a column

$$(5150 \times 23) + 6800 = 125{,}250 \text{lbs.}$$

In order that the longest column shall not be longer than 15 diameters, the section shall be 32" 32"

LOADS ON ARCH RINGS AT COLUMNS

	Col.#0	Col.#2	Col.#4	Col.#6	Col.#8	Col.#10
Live and Dead load	125250	125250	125250	125250	125250	125250
Weight of col.	46300	30350	17600	8250	3200	1100
Wt. of arch ring		45141	40850	36200	34960	32962
Total		200470	183430	169970	163410	159310

LOADS ON ARCH RING MID-WAY BETWEEN COLUMNS

	#1	#3	#5	#7	#9
Wt. of arch ring	47672	42890	37885	35775	33722