Zeitschrift: IABSE publications = Mémoires AIPC = IVBH Abhandlungen

Band: 6 (1940-1941)

Artikel: Modern design and construction practice for wide-span arches in U.S.A.

Autor: McCullough, C.B.

DOI: https://doi.org/10.5169/seals-7096

MODERN DESIGN AND CONSTRUCTION PRACTICE FOR WIDE-SPAN ARCHES IN U.S.A.

FORTSCHRITTE IM ENTWURF UND BAU WEITGESPANNTER EISENBETONBOGEN IN U.S.A.

PROGRÈS RÉALISÉS AUX U.S.A. DANS L'ÉTUDE ET LA CONSTRUCTION DES GRANDS ARCS DE BÉTON ARMÉ.

C. B. McCULLOUGH, Assistant State Highway Engineer, Salem, Oregon, U.S.A.

I. Modern developments in selection and control of materials

A. Utilization of special cements.

While portland cement is considerably over a century old, it has been manufactured in the United States for only about sixty years. The earlier American cements were low in lime and comparatively coarse, not more than 80 per cent passing a 200-mesh screen. They were therefore somewhat sluggish and inferior in strength. As construction necessities became more exacting, a better product was demanded, and there evolved a cement more finely ground, higher in lime, and, in general, more resistant to the elements. This was the "normal" or "standard" portland cement of today, designated hereafter in Table I as "Specification No. 1".

As structural knowledge and need developed, a widened field of utility for reinforced concrete forced the demand for cements characteristically different from this "standard" portland, and research in this field was stimulated. Foremost was the identification of the chemical compounds in cement clinker and principal in this field was the work of the Portland Cement Fellowship of the United States Bureau of Standards. Through this research the following compounds were identified, and methods developed for computing from the oxide analysis of any clinker, its approximate compound composition.

Name	Abbreviated Formula	Chemical Formula
Tetracalcium-alumino ferrite	(C_4AF)	$4\,CaO \cdot Al_2O_3 \cdot Fe_2O_3$
Tricalcium aluminate	(C_3A)	$3\,CaO \cdot Al_2O_3$
Dicalcium silicate	(C_2S)	$2\,CaO \cdot SiO_2$
Tricalcium silicate	(C_3S)	$3\,CaO \cdot SiO_2$
Magnesium oxide		MgO
Calcium oxide		CaO

A description of the method of calculating these compounds was published by Bogue [1]) and nomographs to facilitate the work are now frequently inserted in American specifications. Following this development, research

[1]) "Industrial and Engineering Chemistry", October 1929.

by many independent investigators demonstrated that compound composition had a pronounced effect on certain properties of resulting concrete, principal among which are the following ones:

a. Compressive strength is chiefly derived from the calcium silicates. C_3S acts earlier than C_2S but, weight for weight, their ultimate effect is about the same. C_4AF has little effect. It probably lowers slightly the compressive strength at all ages.
b. Rate of strength gain. Of the silicates, C_3S yields its major contribution during the first week, while C_2S acts principally after the first week. C_3A increases early strength but at later ages it may be detrimental.
c. Resistance to freezing and thawing is probably increased by the use of cements low in C_3A.
d. Resistance to sulfate waters is also greater for cements low in C_3A.
e. Heat of hydration. C_3A hydrates principally during the first day after placement, C_3S principally during the first week, and C_2S later. C_4AF is indefinite in its rate and its effect is slight.
   Weight for weight, C_3A liberates twice as much heat as C_3S at all ages. Reduction in total heat of hydration can therefore be obtained by decreasing the percentage of C_3A with a corresponding increase in C_3AF.

The above data, together with the well-known fact that finer grinding increases compressive strength of all cements at all ages, have lead to the development of a large number of special cements differing principally in fineness and in compound composition. Gradually these are being standardized. Today American practice is largely restricted to the four specifications given hereinbelow (Table I).

Specification No. 1 is the "standard" portland in common use; Specification No. 2 is a "high early strength" cement; Specification No. 3 is a "moderate heat of hardening" cement; and Specification No. 4 is a sulfate resistant cement. These are typical of a number of other special specifications recently used, but, as above stated, practice is being gradually standardized on these four. Most American engineers are still using Specification No. 1, or a similar one for arches. In California, several large arches (notably the Russian Gulch and Big Creek bridges) have been built with cement approaching No. 3 and "high early strength" cements have been used by several of the states for decks and sidewalks. The author in Oregon is using Specifications No. 1, No. 2, and No. 3 at the present time, and will probably utilize No. 4 for future sea-water concrete.

A recent innovation is the use of Specification No. 3 for cold weather locations. In Oregon, and in several other states, surface weathering of a number of older arches has caused some apprehension. In general, this weathering (which is concentrated principally on exposed horizontal surfaces) is confined to regions where many cycles of freezing and thawing are experienced each winter. It appears to start with a maze of hair-line cracks paralleling the longer dimensions of the members. These cracks permit moisture to enter, and subsequent freezing and thawing completes the disintegration to a point where the body of the concrete for a considerable distance from the surface becomes soft and lifeless. This disintegration is doubtless due to several basic causes. Unsound aggregates and improper curing have contributed in many instances, but the data gathered from extensive field and laboratory research in Oregon indicate the greatest trouble in those cases where the cement used was high in C_3A. Whether this be due to any quality inherent in this compound or to the difficulty in curing because of the high rate of early heat liberation for C_3A is not definitely known. As a precaution, however, it appears advisable to use cements approaching Specification No. 3 in regions of this character.

Table I
Condensed specifications for four Portland cements in current use in the United States

	Specification No.			
	1	2	3	4
Loss on ignition	3.00%	3.00%	3.00%	3.00%
Insoluble residue	0.75%	0.75%	0.75%	0.75%
Sulfuric anhydride (SO_3)	2.00%	2.50%	2.00%	1.75%
Magnesia (MgO)	5.00%	5.00%	5.00%	4.00%
Alumina (Al_2O_3)	7.50%	7.50%	6.00%	4.00%
Iron oxide (Fe_2O_3)	6.00%	6.00%	6.00%	4.00%
Tricalcium aluminate (C_3A)[2]	15.00%	15.00%	8.00% [3]	5.00 [3]
Fineness (Sp. Surface, Wagner) (square centimeters per gram)	1500.	1900.	1800.	1800.
Tensile strength (p. s. i.)				
1 day — 1:3 Mortar Briquets	—	275.	—	—
3 days — 1:3 Mortar Briquets	175.	375.	125.	—
7 days — 1:3 Mortar Briquets	275.	425.	250.	175.
28 days — 1:3 Mortar Briquets	350.	—	325.	300.
Compressive strength (p. s. i.)				
1 day — 1:2.75 Mortar Cubes	—	1250.	—	—
3 days — 1:2.75 Mortar Cubes	900.	2500.	750.	—
7 days — 1:2.75 Mortar Cubes	1800.	3500.	1500.	1000.
28 days — 1:2.75 Mortar Cubes	3000.	—	2500.	2200.
Maximum heat of hydration				
7 days (calories per gram)			70.	
28 days (calories per gram)			80.	

The use of autoclave tests[4]) to detect so-called "delayed unsoundness", the utilization of portland-puzzolan blends, and blends of natural and portland cements, are other developments which are gaining favor in the United States. Portland-puzzolan blends have been used by the author in several arch bridges. In general, they are somewhat more workable, slightly higher in both tensile and compressive strength, and more impermeable than the standard portlands. They are also said to have greater resistance to freezing and thawing and to the aggressive action of sulfate waters.

[2]) The expressing of chemical limitations by means of calculated assumed compounds does not necessarily mean that the oxides are actually or entirely present as such compounds. The tricalcium aluminate is calculated according to the following formula: Per cent $3\,CaO \cdot Al_2O_3 = 2.65$ (per cent Al_2O_3 — 0.64 per cent Fe_2O_3).

[3]) Ratio $Al_2O_3 \div Fe_2O_3$ shall be between 0.7 and 2.0. The per cent silica (SiO_2) shall be not less than 21.00 for Specification No. 3 and not less than 24.00 for Specification No. 4.

[4]) The autoclave test recommended consists of a steam bath for 5 hours, 3 of which are at 420° Fahrenheit. The maximum volume change specified in Oregon is 0.8 %.

In summary, therefore, it appears a safe prediction that future American arch practice will be characterized by an increased utilization of special cements, and perhaps of blends. Specification No. 1 will be replaced by No. 3 in cold weather areas, and Specifications No. 2 and No. 4 will be used as occasion demands. Cements, in general, will be selected for the optimum of strength and durability. It may be necessary in certain cases to consider also limiting the heat of hydration; however, this is ordinarily a problem which concerns heavy mass construction rather than bridge work.

B. Control of aggregates.

Recent progress in aggregates has been characterized chiefly by improved methods for insuring optimum size gradations. Concrete for earlier structures was developed from two separate aggregate sizes designated, respectively, as "fine" and "coarse". It is now common practice to split aggregates into three or more limiting size groups, deriving by recombination a product of greater uniformity and density.

Another development is the increasing use of an accelerated soundness test for both fine and coarse aggregates. In general, this method consists in determining the degree to which the aggregate particles are disintegrated by alternate cycles of wetting and drying in a saturated solution of either magnesium sulphate or sodium sulphate. This test has been successfully used for many years, but the wide range of results, particularly when applied to fine aggregates, suggests the desirability of a correlation of such test data with actual performance records in cold weather areas and with laboratory freezing and thawing tests on corresponding concretes.

A current need in connection with selection and control of aggregates is the development of test methods which will indicate more clearly the value of natural sands from a concrete-making standpoint since it frequently happens that of two or more natural sands, the one which shows up the best when subjected to the ordinary routine tests (mortar strength, grading, soundness, freedom from clay, silt and organic matter, etc.) may not produce the best concrete. In 1937 Oregon State College conducted a series of tests along this line on a number of sands of widely divergent characteristics, including therein mineralogical analyses, chemical analyses, physical analyses, mortar-strength tests, and concrete-cylinder tests. It was found that mineralogical and chemical analysis gave no significant indication of the suitability of sand for concrete. The results of certain of the physical tests such as porosity, organic matter, resistance to abrasion and silt content appeared to correlate fairly well with mortar strengths but not with corresponding concrete strengths. The same lack of correlation was apparent when accelerated concrete cylinder tests with high early strength cements were compared with similar standard cement tests at 28 days. It would appear, therefore, that as yet the only definite and certain method of predicting the value of any given sand in concrete is the actual casting of concrete test cylinders, utilizing the proportions, consistencies and gradations of aggregate (both fine and coarse) actually contemplated in the field. It is to be hoped that future research will develop some more expeditious method for predicting in advance the concrete-making value of natural sands.

C. Proportioning and batching.

Significant in this field is the present practice of proportioning by weight as against former volumetric methods. As early as 1916 this matter was receiving the attention of American engineers. In that year Professor Thos. R. Agg of Iowa State College, and the author published an "Investigation of Concrete Roadways", from which the following is quoted:

"Concrete as a paving material will never be used at its highest efficiency until all the materials are accurately proportioned as they enter the mixer and the proportions determined by weight."

Not until years later, however, was this method employed for bridge work. The Sidney arch constructed in 1922 by the New York Central Railroad over Great Miami River was the first large structure wherein concrete was proportioned on a strength basis, but here again volumetric methods were employed. However, proportioning by weight is now almost universal practice. The type of batching plant employed depends to a large extent on the size and importance of the job. The following, from the specifications for Rogue River Bridge (referred to hereinafter) may be considered as typical.

"Special methods for proportioning and control.

... The aggregate for the concrete work in this structure will be separated into four different size groups. The limiting sizes for each of the aggregate groups may be varied slightly in accordance with the results of tests subsequently to be made, but in general the four sizes shall be substantially as follows:

Size A — All aggregate passing a 1/4" screen [5]).
Size B — All aggregate retained on a 1/4" screen and passing a 3/4" screen.
Size C — All aggregate retained on a 3/4" screen and passing a 1 1/2" screen.
Size D — All aggregate retained on a 1 1/2" screen and passing a 2 1/2" screen."

"... The quantity of each of the above sizes of aggregate shall be measured by weighing in a cumulative weighing batcher. The arrangement of the batching plant shall be such that the quantity of each size of aggregate can be readily varied upon request of the engineer. The weight of aggregate shall be shown by a suitable mechanism within view of the operator.

The water used in mixing shall be measured in a tank equipped with a gauge glass or float indicating accurately the amount of water added to each batch. The tank shall be so arranged that the quantity of water may be readily varied upon request of the engineer.

The contractor shall furnish plans of the proposed concrete and screening plants and construction of these plants shall not be commenced until the plans of the same have been approved by the engineer."

II. Modern developments in design.

A. Trend as to unit working stresses.

Formerly maximum unit design stresses were limited to 600 or 650 p. s. i. with a 25 per cent increase for temperature and axial shortening effects. Current projects are employing unit stresses considerably higher. Typical instances are Russian Gulch and Big Creek Bridges in California wherein were employed working stresses of 1000 p. s. i. for dead load plus live load, and 1250 p. s. i. when temperature, rib shortening and shrinkage effects were included, and the Rogue River Bridge in Oregon, wherein working stresses of 1200 p. s. i. were utilized. For these stresses it is necessary, of course, to proportion concrete accordingly. For example, concrete in the arch ribs at Rogue River was specified to yield not less than 5000 p. s. i. at 28 days. Similar provisions are employed for other projects.

---

[5]) The intermediate grading for this size was specified elsewhere.

The above working-stress values represent about the maximum to which American engineers have been inclined to go, except for the case of specially reinforced details.

A few years ago the author conducted a series of tests on certain "Considère" type spirally reinforced hinges, the results of which are summarized hereinbelow in Table II.

Table II
Tests on spirally reinforced hinges
(Average core section 4″ × 4″; diameter of hooping 3½″)

Longitudinal bars		Spiral		60 to 90 day ultimate strength [6])
No. and size	Per Cent core area	Size and pitch	Per Cent core volume	pounds per square inch
4–3/8 round	6.2	1/8″ round at 1/2″ 1/4″ round at 1″	3.25 6.50	13,700 – 12,900 14,800 – 12,100
4–1/2 square	14.1	1/8″ round at 1/2″ 1/4″ round at 1″	3.25 6.50	11,700 – 13,900 13,800 – 13,700

For this type of hinge, since the loads are of short duration, design stresses as high as 5000 p. s. i. have been employed. Arch ribs similarly reinforced could doubtless be safely stressed to 3000 p. s. i., and perhaps higher, but as yet American engineers have not utilized the principle, possibly because the cost of the reinforcement more than offsets the saving in section.

B. Modern methods for control of deformation stresses in hingeless arches.

As the centering under any hingeless arch is released, stresses from the following causes are progressively induced.

Group "A", consisting of:

1. dead load proper,
2. elastic rib shortening (due to dead load),
3. temperature variation from initial to mean values,
4. initial or early shrinkage,
5. initial plastic displacement of supports, and
6. early plastic deformation of rib.

Group "B" consisting of:

1. live load and impact,
2. seasonal temperature variations, and
3. subsequent or periodic shrinkage and subsequent plastic flow.

Group "B" stresses are fixed. Group "A" stresses may be modified and controlled. In general, there are two methods of accomplishing this result, to-wit: the placement of temporary construction hinges (generally at crown and skewbacks) and the method of "rib compensation and adjust-

[6]) The 60 to 90 day strength of plain concrete test cylinders of the same mix as test hinges ranged from 6000 to 6400 pounds per square inch.

ment", which consists of the introduction of certain arbitrary compensating forces at one or more points in the rib by means of a system of hydraulic jacks, or otherwise.

In the utilization of these methods, American engineers are far behind their European contemporaries. In 1926, Mr. J. F. Brett, Designing Engineer for the Montreal Water Board, brought this matter to the attention of engineers in the United States in a paper entitled "The Reduction of Deformation Stresses in Fixed Concrete Arches", but to date neither method has been widely employed in America.

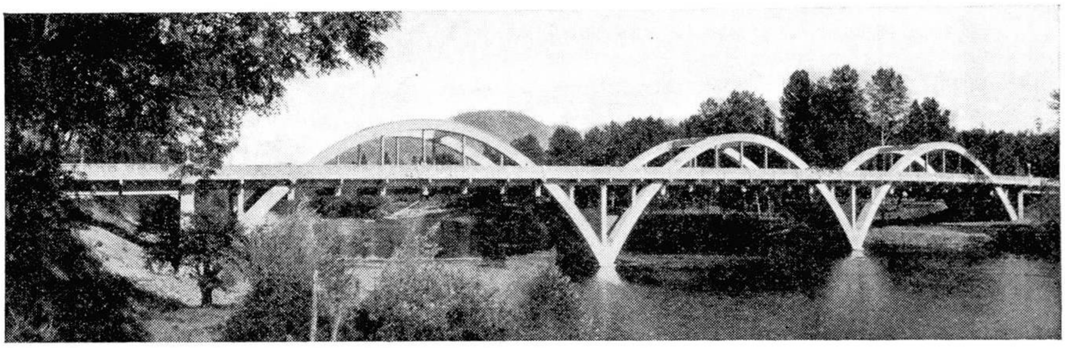


Fig. 1

First arch structure in the U. S. A. utilizing Considère construction hinge — Erste Bogenbrücke in den U. S. A. (Grants Pass, Oregon), wo provisorisch Gelenke nach Considère verwendet wurden — Premier pont en arc des U. S. A. (Grants Pass, Oregon) avec articulation provisoire système Considère.

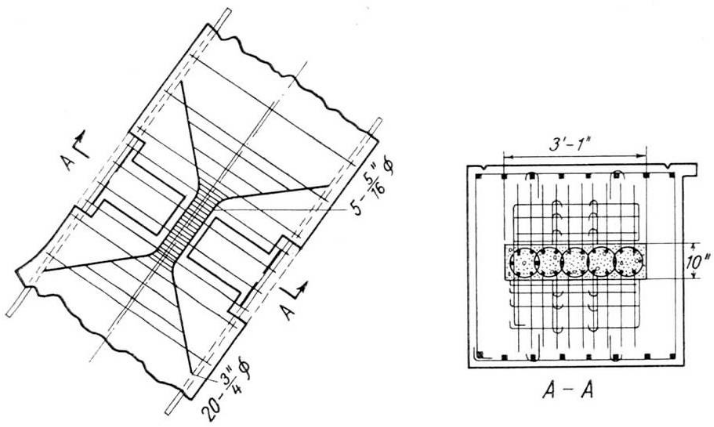


Fig. 2

Detail of Considère hinge at skewback, Grants Pass, Oregon.
Detail des Considère-Gelenkes im Kämpfer, Grants Pass, Oregon.
Détail de l'articulation Considère à la naissance, Grants Pass, Oregon.

In 1930 the author utilized construction hinges of the Considère type in a three-span arch at Grants Pass, indicated in Figures 1 and 2. Figures 3 and 4 indicate other construction views showing this type of hinge. Measurements of crown deflections at Grants Pass indicated an actual movement upon release of centers about 1/16-inch less than the calculated value, assuming complete freedom. The unit working stress utilized in this instance was comparatively low (only 3900 p. s. i.) but as above noted, there was little loss of flexibility. Table II hereinbefore indicates ultimate strengths obtained from contemporary tests of hinges of this type, and parallel tests on plain concrete cylinders of the same mix. Subsequent to this construction, a large number of structures employing this type of hinge have been built in Oregon.

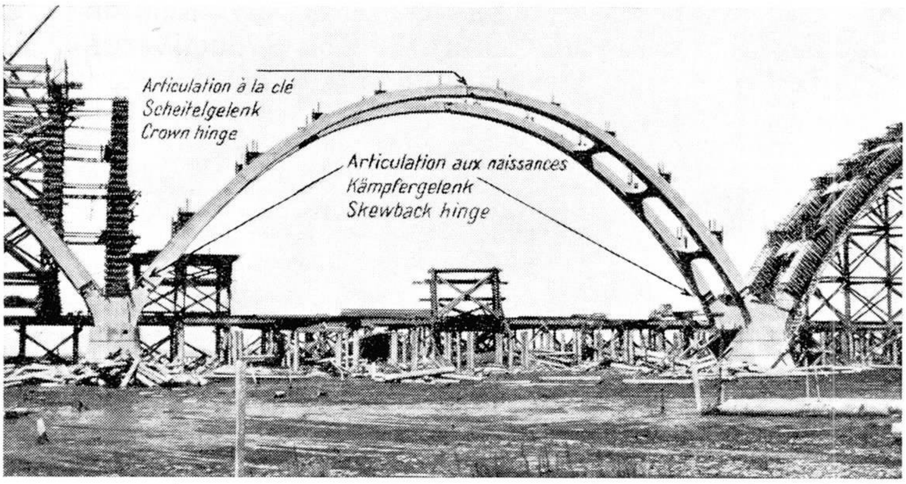


Fig. 3
Considère hinges at Coos Bay, Oregon.
Considère-Gelenke, Coos Bay, Oregon.
Articulations Considère, Coos Bay, Oregon.

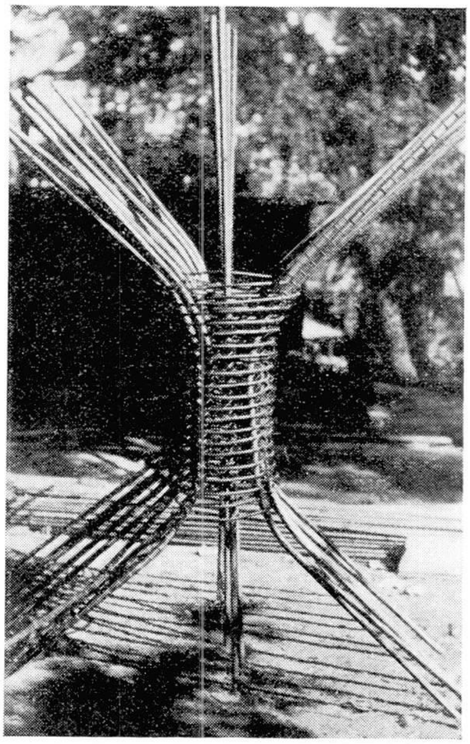


Fig. 4
Reinforcement of a hinge.
Armierung eines Gelenkes.
Armature d'une articulation.

In general, however, American engineers have been reluctant to utilize this principle, practically all of the other arches having been constructed by conventional methods. Just why American engineers have not utilized this principle to a greater extent is hard to understand. Its advantages are paramount, and particularly apparent for arches with low rise-span ratio or on foundations such that slight plastic displacements may be expected upon release of centers. Most of the deformation stresses vary directly as the moment of inertia of the rib, or roughly, as the cube of its depth, whereas the moment of resistance varies approximately as the depth squared. Such a situation renders deformation stresses particularly burdensome for long spans and for narrow deep ribs such as are often necessary for through or half-through arches because of space limitations.

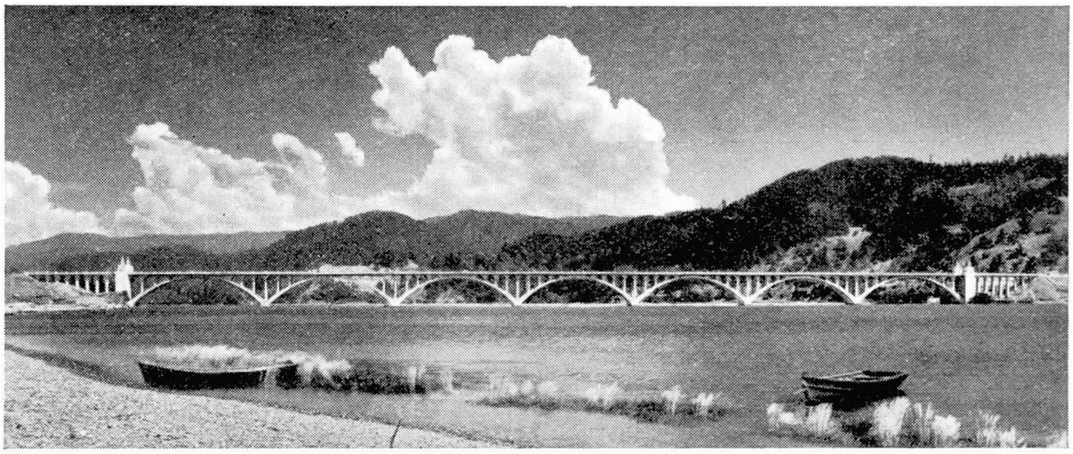
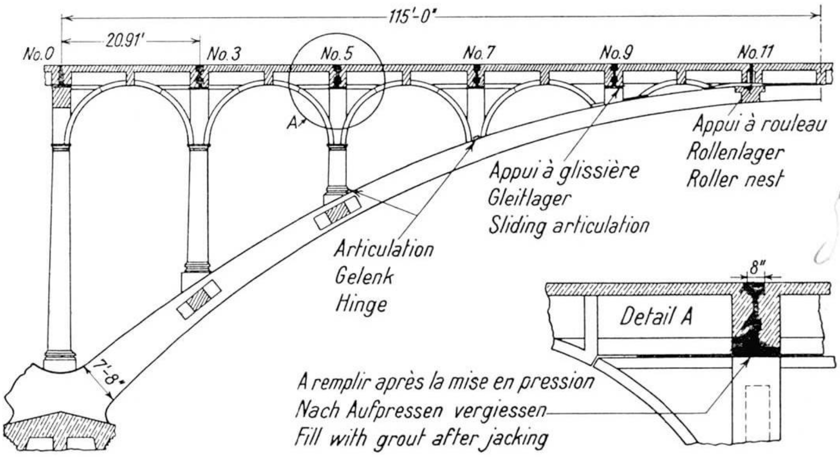


Fig. 5
Rogue River Bridge, first structure in the U.S.A. wherein the Freyssinet system was used.
Rogue River Brücke, bei der das System Freyssinet in den U.S.A. zum ersten Mal angewendet wurde.
Pont de Rogue River, où pour la première fois le système Freyssinet fut appliqué aux U.S.A.



gut

Fig. 6
Superstructure articulation of Rogue River Bridge.
Gelenksystem im Überbau der Rogue River Brücke.
Système d'articulation dans la superstructure du pont de Rogue River.

Notwithstanding the dearth of instances of utilization in arch bridges, considerable interest is now being evidenced in the carrying capacity of hinges. In 1937, Professor G. C. Ernst of the University of Maryland published a report of tests on 62 hinges of the Mesnager type, and this work is now being extended to include Considère hinges as well. These tests have particular reference to the use of hinges in rigid frame structures but the data will be of equal value for construction hinges in arch bridges.

The second method of deformation stress control bears the name of its inventor, the noted French engineer, E. Freyssinet. Again, as in the above case, American progress seems to be lagging behind that in Europe.

The first and about the only large structure to be built in the United States by this method was across Rogue River in Oregon (Fig. 5). This structure, designed and constructed under the author's direction, consists of a group of seven 230-foot symmetrical arch spans. There is a three-span central group with anchor piers and two intermediate elastic piers. At each end is a two-span group with an intermediate elastic pier. All piers are on wood piling. End abutments are on rock. Construction operations rendered it inexpedient to decenter without the spandrel structure, and in order to provide maximum freedom of movement, articulation was provided, as indicated in Figure 6. This consisted of sliding expansion plates at the tops of pier columns, temporary bar hinges at the tops of columns 3, 5, and 7, socket hinges at the base of columns 5 and 7, a sliding articulation at point 9, and a permanent roller nest at point 11.

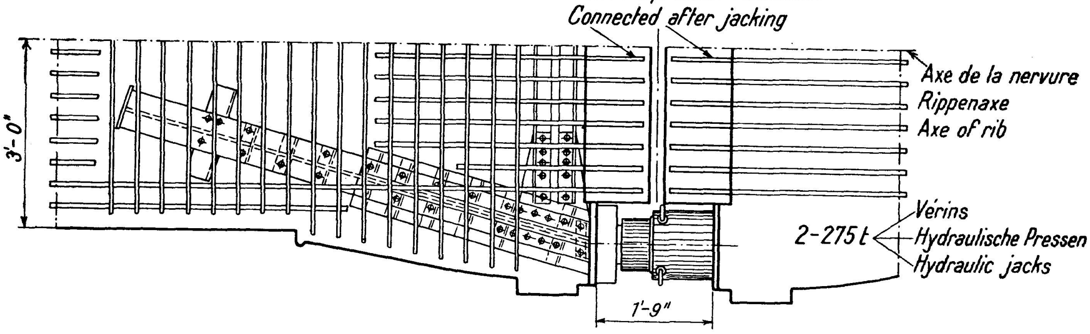


Fig. 7
Plan view of jack arrangement at crown.
Grundriß der Pressenanordnung im Scheitel.
Vue en plan des vérins placés à la clé.

Four 275-ton jacks were provided for each rib. These were so connected that the intradosal and extradosal jacks could be independently operated, thus introducing predetermined crown moments as well as thrusts. As a precaution against overstressing the concrete, structural steel jacking brackets were embedded on each side of the crown gap, as indicated in Figure 7. During decentering operations, the following observations of pressures and strains were made:

1. total jack thrusts and moments,
2. crown openings at extrados and intrados,
3. vertical movements at crown, spring line and intermediate points,
4. strains at extrados and intrados at five sections in each span,
5. temperatures on extrados, intrados and axis, at crown, quarter point and skewback,
6. rotation of rib axis at crown opening, quarter points, skewbacks and piers,
7. horizontal movement of piers at spring line,
8. measurements for determining thermal and shrinkage effects.

Jack pressures were measured by calibrated Bourdon gauges, crown openings with a sliding scale and vernier to nearest 0.01 inch, vertical movements by means of a system of water levels, horizontal movements by weighted invar wires, and internal strains by means of an installation of

McCollom-Peters electric telemeters. Rib temperatures were obtained by means of resistance coils and rib rotations were measured by means of 20-inch clinometers, equipped with a micrometer dial reading directly to 0.001 inch.

Table III indicates the effect on fiber stresses of this method of adjustment, and also the effect of spandrel structure restraint.

Table III
Effect on fiber stress of Freyssinet adjustment and also effect of spandrel structure restraint

Stress Condition	Maximum Fibre Stress (p. s. i.)		
	Crown	Quarter-point	Skewback
1. Fixed arch (immediate keying)	1491	968	766
2. Freyssinet adjustment (computed from observed jack thrusts)	874	1020	510
3. Freyssinet adjustment (computed from measured strains)	867	806	301
4. Stress reduction by adjustment	617	- 52	256
5. Effect of spandrel structure	7	214	209

Crown stresses were reduced from 1491 p. s. i. to 874 p. s. i. by adjustment, and notwithstanding the fact that the spandrel structure was articulated to the greatest possible degree, it still exercised enough restraint to lower rib stresses materially.

A complete report of this research was published by the U. S. Bureau of Public Roads jointly with the Oregon Highway Commission, and the following is extracted from the conclusions contained therein:

1. The restraint to free movement exerted by the spandrel structure of a concrete arch, even with the utmost degree of articulation possible, is so great as materially to alter stress distribution. This fact, together with the uncertainty regarding uniformity and continuity of such action renders the Freyssinet or some similar method of strain control particularly important, not only as an economy procedure, but also to insure certainty of stress action.

2. The effect and rate of action of shrinkage and plastic flow under actual construction conditions can not be predicted. This is another argument for this method of strain adjustment.

3. Spandrel structure restraint reduces the range of movement which may be safely produced by crown jacking so that whenever possible, a preliminary adjustment should be made on the free rib. The uncertainties regarding spandrel action, however, render it desirable to effect a final adjustment after such structure is in place. In order to eliminate the effect of form restraint, which was appreciable in this case even after wedges were removed and centering dropped, final adjustment should be made after all forms are stripped.

4. In view of the uncertainties involved, it is wise to provide for independent checks on adjustment processes, to-wit: a system of pressure gauges, a system of internal strain gauges, and a system of crown-movement measurements.

5. The mathematical analysis of arches with monolithic spandrel structure may be subject to some question as to reliability. In such a case, crown movements would be too small to constitute an accurate index so that de-

pendence would have to be placed upon pressure-gauge readings with internal strain-gauge readings as a check.

6. The effect of spandrel restraint may, in some cases, relieve rib stresses so that by a rigid system of strain control such as described herein, further economies may be realized.

7. The persistence of form-work restraint, even after centers were entirely free, indicates that construction hinges cannot be depended upon to permit free rib movement unless special precaution is taken to remove all such form-work restraint.

C. Modern design methods for multiple span arches on elastic intermediate piers.

There have been few multiple-span arches designed in the United States with consideration given to the effect of pier elasticity, intermediate piers in most cases being so heavily designed that elastic displacements were negligible. In order to study the effect of such pier elasticity, however, a group of three-span structures were analyzed by the author. These are indicated in Figure 9 [7]), and consist of three groups of three spans each, differing solely in the height of intermediate piers. These systems were analyzed by mechanical methods (the Beggs deformeter) and also by algebraic methods. This latter method, while tedious in application, is comparatively simple in principle. To illustrate, consider the single-span indicated in Figure 8 a. If at either end the rib were to be split into separate chords extending up to

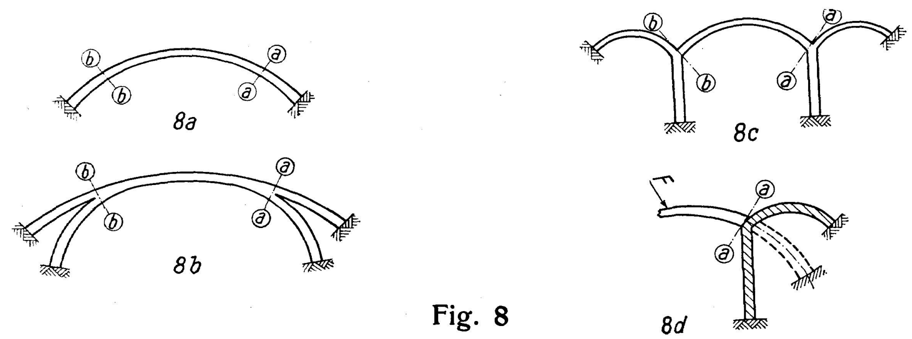


Fig. 8

sections a—a and b—b, ordinary methods of analysis would still apply. Fundamental methods would still remain unaltered if these bifurcated segments were extended to form an intermediate pier and an additional span (Figure 8 c). Obviously, if the elastic displacement of the pier and span combination (cross-hatched in Figure 8 d) is the same under any loading as the arch block from which it was developed, the elasticity of the central segment (a—a to b—b) is in no wise changed, and the stresses under any given load condition will remain the same. The method utilized in the above analysis was simply this principle developed in reverse order. In other words, the method consisted essentially in determining the elastic displacement of each pier and rib combination (beginning at either end) and the replacement of such combination by an "ideal" or substitute voussoir section. The complete analysis must obviously be carried a step further inasmuch as the above procedure suffices merely for the determination of rib stresses, induced by loads on that particular span. It becomes necessary, therefore, to determine what portion of the load on each span is transferred to adjacent

[7]) See also Figure 1 for a view of one of the combinations.

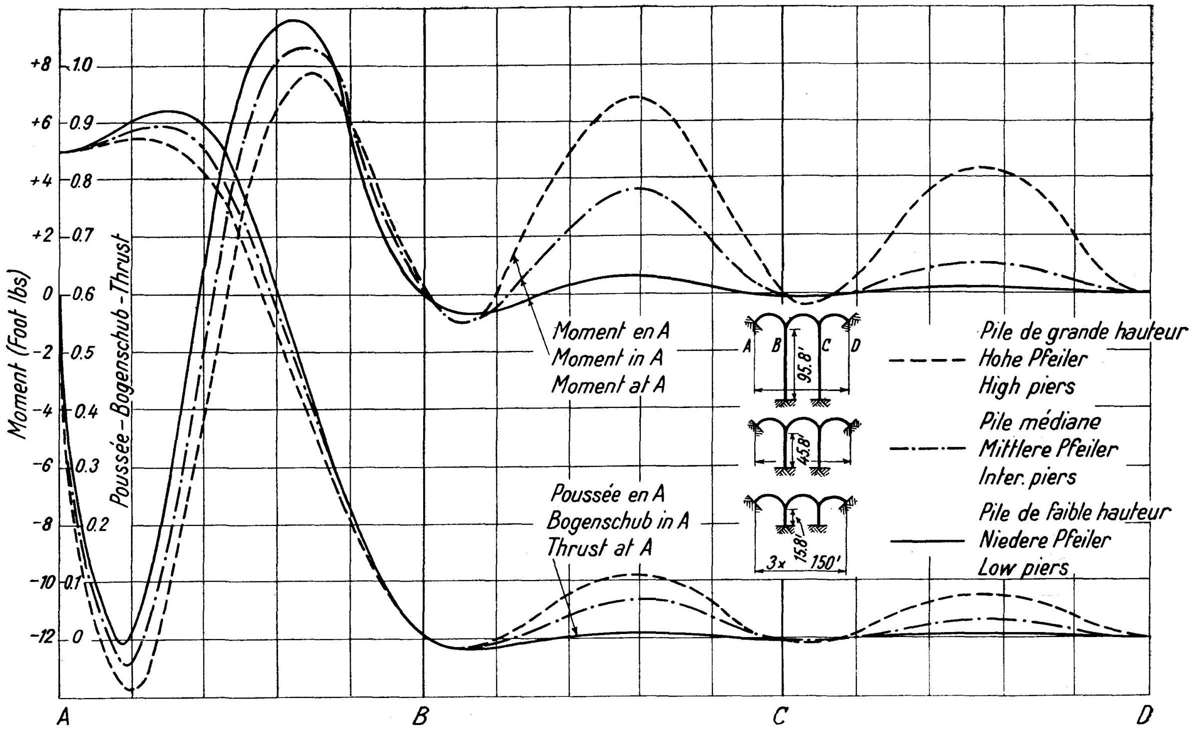


Fig. 9
Comparison of moment and thrust.
Einflußlinien für Moment und Bogenschub.
Lignes d'influence du moment et de la poussée.

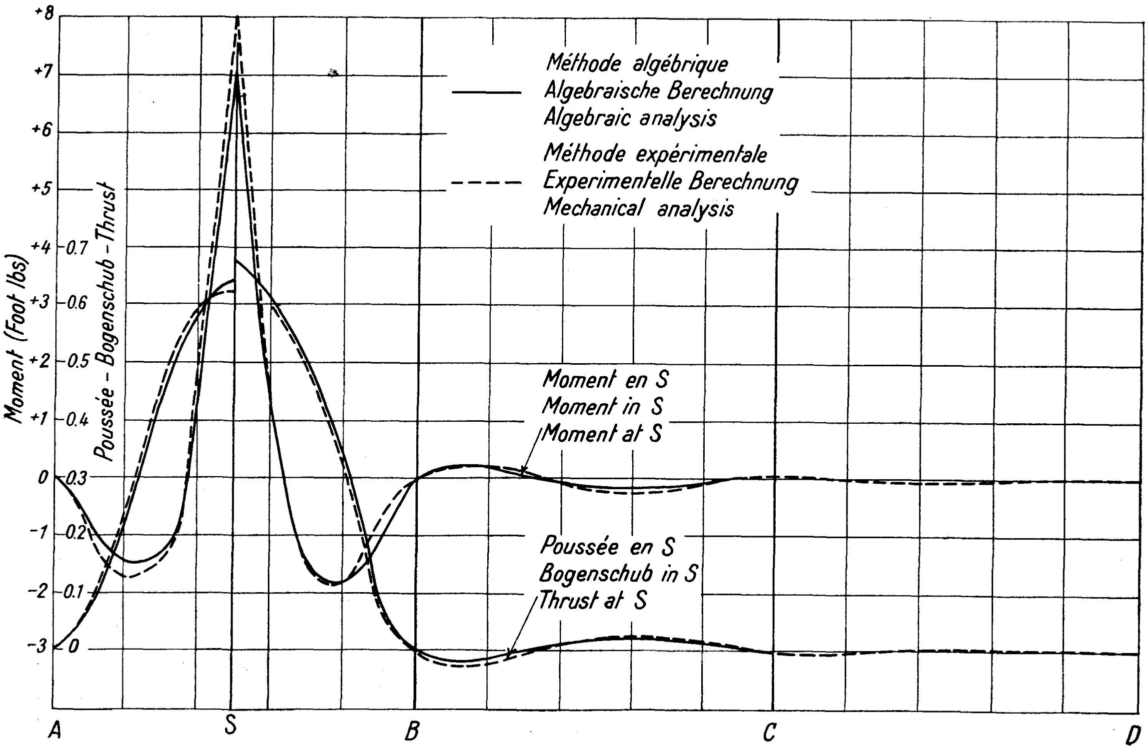


Fig. 10
Comparison of algebraic vs. mechanical method.
Vergleich zwischen algebraischer und experimenteller Berechnung.
Comparaison entre la méthode algébrique et la méthode expérimentale.

spans through the elastic piers. The mathematical derivations involved are comparatively simple, but, as above stated, somewhat lengthy, for which reason they must be omitted here [8]). Figure 10, a comparison of algebraic versus mechanical methods, indicates the close degree of correspondence.

Another method for the analysis of arch systems on elastic piers used to some extent in the United States is based upon the theory of the "Ellipsis of Elasticity". The theory was originally developed for this purpose by Ritter and Cremona, and first introduced into American literature by A. C. Janni in a paper presented in 1913 to the Western Society of Engineers. In 1924 Mr. Janni presented a more complete discussion of the subject before the American Society of Civil Engineers. Lack of space renders it impossible to outline this method, even in its briefest terms [9]).

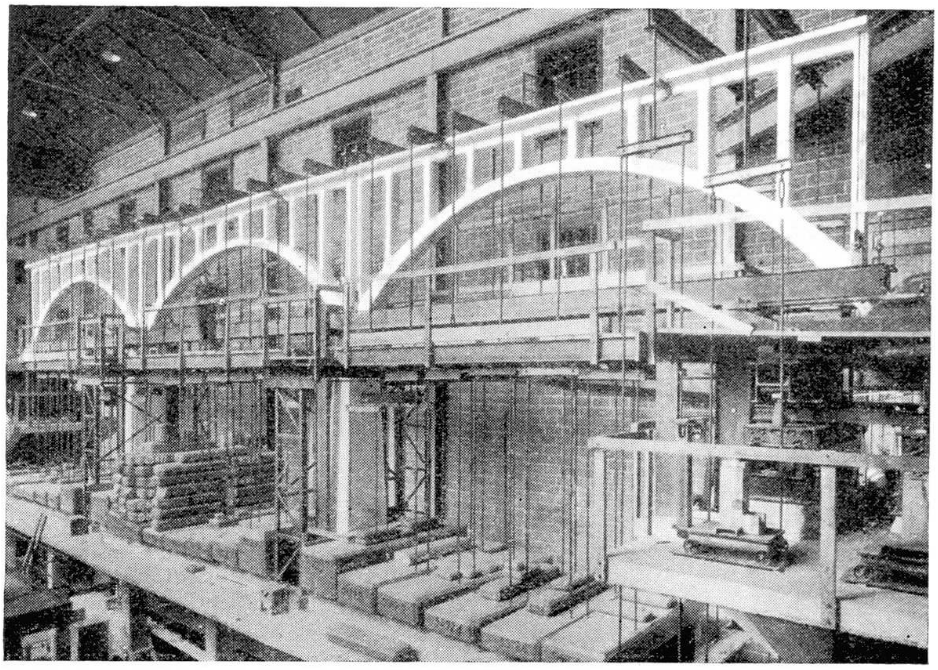


Fig. 11
University of Illinois tests.
Versuche der Hochschule von Illinois.
Essais de l'université d'Illinois.

In general, elastic pier displacements operate to increase positive crown moments. Crown thrusts are decreased. Skewback thrusts are also decreased but not to as great an extent, and both skewback moments are increased on the negative side. In the side spans, the greatest effect is for the skewback next so the elastic pier. It is thus seen that an elastic pier displacement acts very much like a drop in temperature or a shrinkage.

The above observations have reference to loads upon the span in question. In addition, pier displacements operate to effect a load transference from one span to another, thus resulting in an increased loaded length which, in general, will increase unit stresses.

[8]) For complete mathematical derivation, see "Elastic Arch Bridges", McCullough & Thayer, John Wiley & Sons, page 194, et seq.

[9]) "Elastic Arch Bridges", page 233, et seq.

In 1934, Professor Wilbur M. Wilson and Ralph W. Kluge of the University of Illinois, reported on certain laboratory tests of three-span arches on slender piers[10]. The test models consisted of three spans of 27 feet each, the general arrangement being indicated in Figure 11. The conclusions drawn by the authors were stated, in substance, as follows:

1. The mathematical theory, based upon usual assumptions, yields values which agree with actual measured values within the tolerance of the tests.
2. The analysis may, without material error, be based upon an assumption of constant elastic modulus since the value of "E" may vary considerably without materially affecting results.
3. The moment of inertia of any section may be computed on the basis that concrete takes tension.
4. Elastic deformation of piers increases maximum compressive stresses for dead load plus live load some 13 per cent.

Following this, the same authors published a further investigation extended to include test models complete with spandrel columns and deck. For one structure the deck was located at a considerable distance above the rib, for another the deck was so low as to be integral with the rib at the crown. Each structure was originally built, without expansion joints in the deck, except over piers. After these had been tested, joints were cut in the deck near each third point, and retests made. The principal conclusions drawn by the authors were, in substance, as follows:

1. The influence ordinates for fixed-end reaction are materially different for a structure having a deck from those for one consisting of a rib without deck. The effect of the deck is to reduce the load moment at the spring line where it is all resisted by the rib, and to increase the moment over the middle of the span where the deck acts with the rib. The effect of the deck is to increase the temperature moment at the spring line.
2. A deck without intermediate expansion joints increases the stiffness and the moment-resisting capacity of the central part of the structure. Intermediate expansion joints reduce both these effects.
3. There is considerable evidence indicating that the dead-load stress in a multiple-span structure may exceed the corresponding stress in a similar single span having fixed ends.

Notwithstanding the foregoing evidence of interest from a research standpoint, there are few instances of the application of these principles to actual arch designs, only one of the questionnaires indicating the use of the above or similar methods.

D. Plasticity and time yield.

Since this question is to be discussed in another paper it will be presented but briefly here.

In 1935 Wilson and Kluge published a report on "The Effect of Time Yield in Concrete upon Deformation Stresses in a Reinforced Concrete Arch Bridge"[11]. The test model consisted of a 27-foot open spandrel arch, with a rise of 6′ 9″. The concrete mix was 1:3:3 by volume, with a 1.2 water-cement ratio. The authors stated, among other things, that:

[10]) University of Illinois Bulletins Nos. 269 and 270.

[11]) Bulletin 275, University of Illinois Engineering Experiment Station. Report by Wilson and Kluge.

"It would appear safe to conclude that a considerable part of the early volume changes have little effect upon the stresses in the concrete of an arch rib." Also that, "If time yield in concrete diminishes with age, and with the number of applications of a load, it is possible that temperature stresses in the concrete of arches a few years old may not be greatly relieved by time yield."

The above conclusions are supported by the preponderance of opinion among American engineers. Mr. GEMENY, however, pointed out [12]) that an assumption of this kind is justified only under closely controlled laboratory conditions. Under construction conditions, this assumption might be far from the truth since there is apt to be no appreciable shrinkage for a considerable time after concrete is placed because of retention of water by forms and the

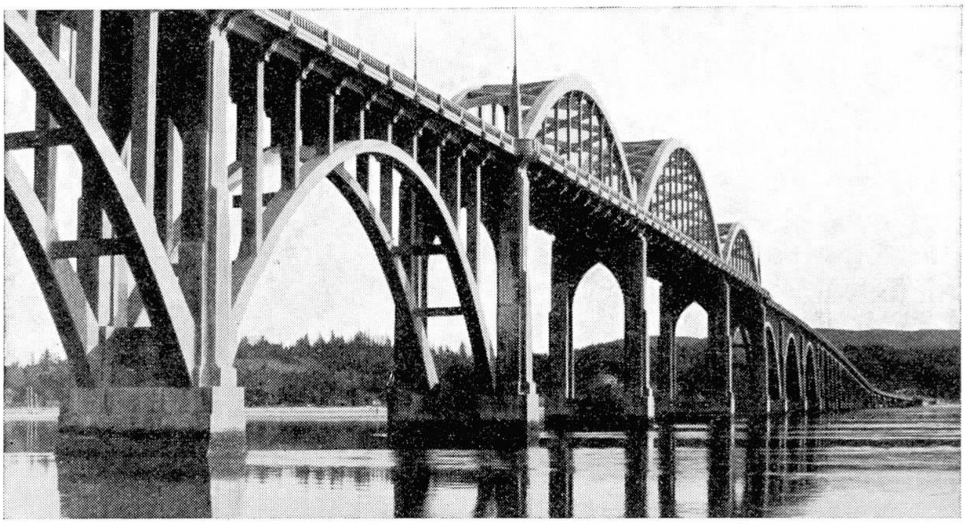


Fig. 12
Alsea River Bridge. The central arch is one of the longest tied arches in the U. S. A.
Alsea River Brücke mit einem der längsten Bogen mit Zugband in den U. S. A.
Pont de Alsea River. L'arc central est un des plus longs arcs à tirant des U. S. A.

addition of water by curing. When shrinkage finally begins, a large part of the plastic flow may have occurred, and its relieving effect thus be greatly reduced. In the meantime shortening of the axis itself, due to plastic flow, produces a residual stress which may more than counteract any stress relief from time yield.

It would thus appear that in spite of the fact that some relief from plastic flow is inevitable, such action can not be definitely relied upon under construction conditions, and special methods such as the Freyssinet system are a precaution entirely warranted for arches with deep ribs and long span.

E. Other design developments and trends.

Among these, mention may be made of the increasing use of the "bow string" or tied arch type wherein the horizontal thrust component is taken by a tension member at deck level. A number of these have been built both for railway and highway traffic. An interesting example is on the Atlantic

[12]) "Application of Freyssinet Method of Concrete Arch Construction to Rogue River Bridge". Report by Gemeny and McCullough.

Coast Railroad in South Richmond Yard. This structure, designed by A. C. JANNI, consists of a single 190-foot span on elastic piers.

In 1934 the author designed several structures of this type along the Oregon Coast Highway. The longest of these was at Alsea Bay (see Fig. 12). The central span group (which were raised to deck level, and designed as tied arches because of navigation requirements) consisted of three spans, the central one of which is 210 feet. Considerable economy was effected by designing the roadway deck integral with the reaction tie and as a part thereof.

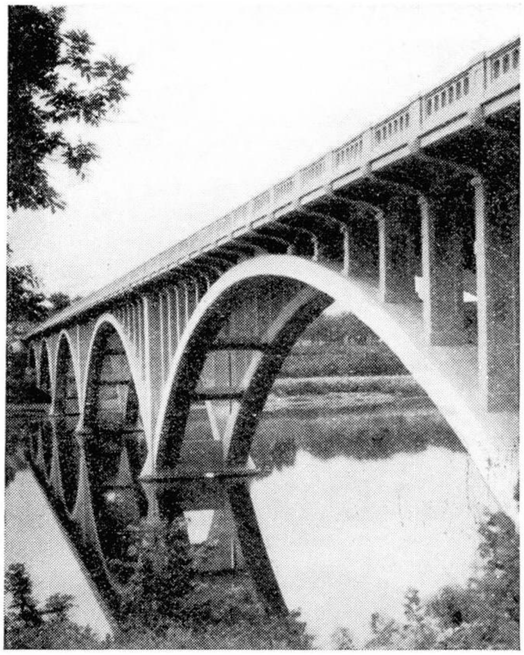


Fig. 13

The Bridge over Lake Taneycomo consists of five 195 foot spans on elastic piers — Brücke über den Taneycomo-See. Fünf Bogen von 195 Fuß auf elastischen Zwischenpfeilern — Pont sur le lac Taneycomo. Cinq arcs de 195 pieds sur piles intermédiaires élastiques.

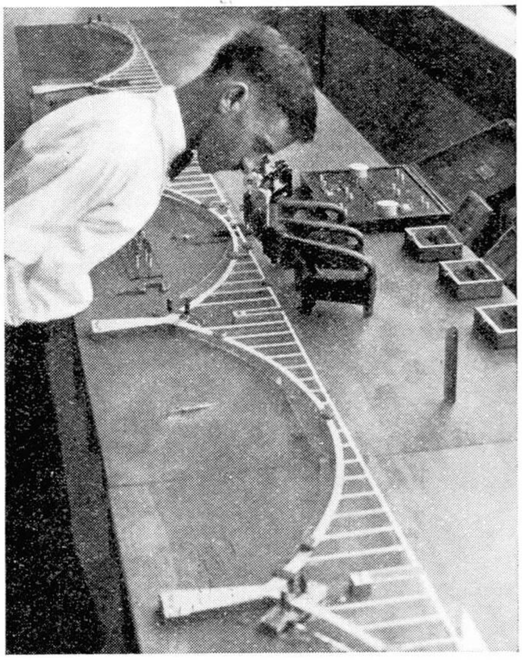


Fig. 14

Study of Lake Taneycomo Bridge by Beggs Deformeter — Untersuchung der Lake Taneycomo-Brücke mit Beggs Deformeter — Etude du pont du lac Taneycomo au moyen du déformètre de Beggs.

Another fairly recent innovation is the use of mechanical methods for stress analysis. Probably the most popular machine of this type in use in the United States is the Beggs Deformeter. Utilization of these methods has rendered it feasible in many instances to investigate stress effects which otherwise would involve such extensive and laborious mathematical calculations as to discourage all initial attempts. As an example may be cited the bridge constructed by the Missouri Highway Department over Lake Taneycomo (see Figure 13). The following description of this structure is quoted from a letter recently received from Mr. N. R. SACK, Bridge Engineer for the Missouri Highway Department.

"This is a multiple-span structure on elastic piers. Furthermore, the spandrel columns and floor were monolithic; expansion joints were provided over the piers only. As a result, the structure is highly indeterminate. In fact the mathematical analysis would require the solution of 270 simultaneous equations. As a result of this difficulty a preliminary analysis of the arch rings was made, by the conventional method, assuming the rings to be fixed at the springing and also assuming that the deck structure did not affect the deformation of the rings. After this preliminary analysis was made a celluloid model of the five arch spans was cut and the stresses, at various critical sections, were determined by the use of Beggs Deformeter."

Space will not permit even the briefest discussion of the various inter-relationships involved. Figure 15 is inserted for the purpose of indicating

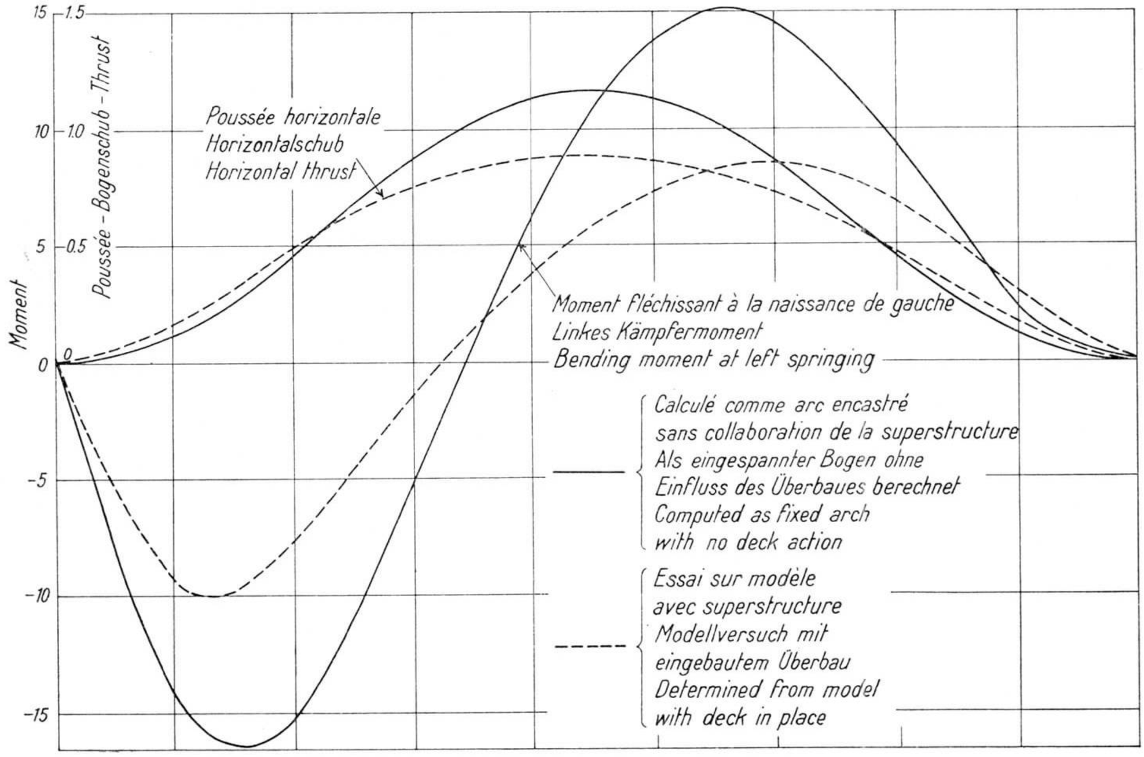


Fig. 15
Effect of spandrel structures restraint on bending moments and horizontal thrust.
Der Einfluß der Bogenaufbauten auf Biegemomente und Horizontalschub.
Influence de la superstructure sur les moments de flexion et la poussée horizontale.

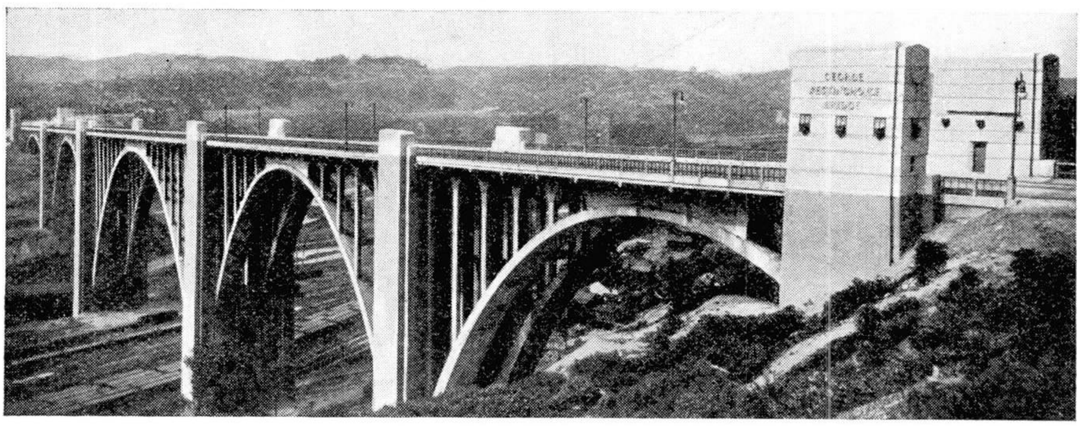


Fig. 16
George Westinghouse Bridge, Allegheny County, Pennsylvania. Longest reinforced concrete arch span (460′) in U. S. A. — George Westinghouse Bridge, Allegheny County, Pennsylvania hat den weitest gespannten Eisenbetonbogen (460′) in den U. S. A. — Pont George Westinghouse, Allegheny County, Pensylvanie, pont de béton armé le plus long des U. S. A. (460′).

the extent to which arch rib stresses were effected and also to illustrate the utility of the mechanical method as an auxiliary. As before stated, structural arrangements of this kind whose analyses involve tedious and difficult mathematics, are somewhat handicapped at the outset. The tendency of the American engineer has been to avoid such types and to adopt simpler types, even at the expense of economy. Mechanical methods are obviously not a substitute for mathematical analysis. Their effect in the United States, however, has been to invite investigation of the more complex structural arrangements.

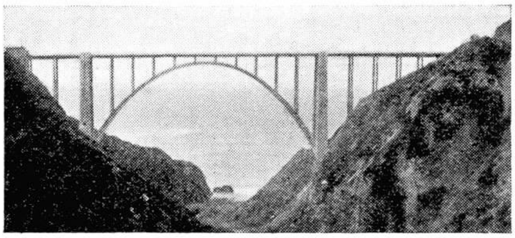


Fig. 17
Bixby Creek Bridge. The central span is 330′ — Bixby Creek Brücke. Mittelöffnung von 330′ — Pont Bixby Creek. Ouverture centrale de 330′.

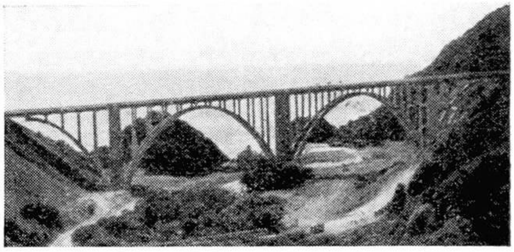


Fig. 18
Bridge across Big Creek has cantilevered half arches at either end — Brücke über den Big Creek mit auskragenden Halbbogen auf beiden Seiten — Pont sur le Big Creek avec demi-arcs en console à chaque extrêmité.

III. Modern examples of American arch construction.

There have been a number of comparatively important arches built in the United States during the past few years, but, with the exception of the

Abridged list of arch bridges recently built in the United States

Name	Location	Span Lengths
1. George Westinghouse Arch	Allegheny County, Pennsylvania	1 at 277.5′, 2 at 295′ and 1 at 460′
2. Bixby Creek Bridge	Monterey County, California	330′
3. Cedar Creek Bridge	Mendocino County, California	320′
4. Big Dann Creek Bridge	Mendocino County, California	320′
5. Yaquina Bay Bridge	Lincoln County, Oregon	1 at 204′, 1 at 232′ and 1 at 265′
6. Coos Bay Bridge	Coos County, Oregon	2 at 208′, 2 at 227′, 2 at 246′, and 2 at 265′
7. Russian Gulch Bridge	Mendocino County, California	240′
8. Nehalem River Bridge	Clatsop County, Oregon	231′
9. Gold Beach Bridge	Curry County, Oregon	7 at 230′
10. Cape Creek Bridge	Lane County, Oregon	220′
11. Alsea Bay Bridge	Lincoln County, Oregon	210′
12. Jughandle Creek Bridge	Mendocino County, California	210′
13. Key Bridge	District of Columbia	1 at 208′ and 2 at 204′
14. Meigs Bridge	District of Columbia	204′
15. Jefferson Bridge	Linn County, Oregon	3 at 200′

George Westinghouse Bridge whose central span is 460 feet (see Figure 16), none of these are of monumental dimension. The abridged list given hereinbelow includes most of the larger structures, and photographs of a few of these have been included.

Summary.

In the building of wide-span arch bridges of reinforced concrete, progress has been made in the United States with respect both to the quality of the material used and to design and calculation.

In the endeavour to improve the material, the introduction of special cements has above all had a favourable influence; of these cements there are principally four kinds in use. Their chemical composition and physical properties, on which the range of application depends, are described. The quality of the concrete as regards strength and durability is to a large extent dependent on careful choice of grain size and on mixing in the proper proportions; in the U. S. A., mixing by weight instead of by volume is now largely adopted.

In design and calculation the prevailing tendency is to make use of this improvement in quality by adopting higher permissible stresses. On the other hand, use has been made only in a few isolated cases of methods of removing unfavourable influences due to the shortening of the axis of the arch and to displacements of the abutments after removal of the centering, through introducing provisional hinges (Considère system) or through regulating the thrust of the arch (Freyssinet method).

In making calculations, the influence of the elasticity of the piers in a series of spans, as well as the effect of superstructures (regarding which, tests from the University of Illinois are submitted) have hitherto been neglected because of the great complications involved. Designers have only begun to consider these influences now that modern experimental methods are available (Beggs deformeter).

A mention of some of the latest arch bridges in the U. S. A. completes the picture of the present state of technic in this field.

Zusammenfassung.

Im Bau weitgespannter Bogenbrücken aus Eisenbeton sind in den Vereinigten Staaten Fortschritte zu verzeichnen, sowohl was die Güte des Baumaterials anbelangt als auch im Hinblick auf Entwurf und Berechnung.

Bei der Vervollkommnung des Baumaterials hat sich vor allen Dingen die Einführung von Spezial-Zementen günstig ausgewirkt, von denen vor allem vier Arten in Gebrauch stehen. Ihre chemische Zusammensetzung und physikalischen Eigenschaften, woraus sich der Verwendungsbereich ergibt, werden beschrieben. Die Qualität des Betons bezüglich Festigkeit und Dauerhaftigkeit wird weitgehend durch sorgfältige Kornzusammensetzung und richtige Dosierung bedingt; in den U. S. A. hat sich daher die Gewichtsdosierung gegenüber der volumetrischen Dosierung schon weitgehend durchgesetzt.

Beim Entwurf und der Berechnung herrscht die Tendenz, diese Qualitätsverbesserung durch Wahl hoher zulässiger Spannungen auszunützen. Hingegen wird von der Behebung der ungünstigen Einflüsse aus der Verkürzung der Bogenaxe und von Widerlagerverschiebungen nach dem Ausschalen durch

Einschalten provisorischer Gelenke (System Considère) oder durch Regulieren des Bogenschubes (Methode Freyssinet) bis jetzt nur vereinzelt Gebrauch gemacht.

Bei der Berechnung wird der Einfluß der Pfeilerelastizität bei Bogenreihen sowie das Mitwirken der Aufbauten (worüber Versuche der Hochschule von Illinois vorliegen) der umständlichen Berechnung wegen vernachlässigt. Erst durch die modernen experimentellen Methoden (Beggs Deformeter) finden diese Einflüsse die Beachtung des Konstrukteurs.

Eine Zusammenstellung der neueren Bogenbrücken in U. S. A. vervollständigt das Bild über den gegenwärtigen Stand auf diesem Gebiet.

Résumé.

Les progrès réalisés aux Etats-Unis dans la construction des grands ponts en arc de béton armé se rapportent tant à la qualité des matériaux de construction qu'à l'étude et au calcul de ces ouvrages.

L'introduction des ciments spéciaux a tout spécialement contribué à l'amélioration des matériaux. Quatre types de ciment sont principalement utilisés et l'auteur en décrit la composition chimique et les propriétés physiques ce qui permet de fixer leur domaine d'application. La qualité du béton quant à sa résistance et à sa durabilité dépend dans une large mesure d'une granulométrie choisie et d'un dosage exact; le dosage en poids remplace de plus en plus le dosage volumétrique aux U. S. A.

Cette amélioration de la qualité se traduit dans l'étude et le calcul par le choix de contraintes admissibles plus élevées qu'autrefois. Jusqu'à ce jour on n'a que rarement eu recours à l'introduction d'articulations provisoires (système Considère) ou au réglage de la poussée (méthode Freyssinet) pour remédier à l'influence défavorable du raccourcissement de l'axe et du tassement des culées que produit le décintrement.

Dans le calcul on néglige généralement, à cause de la complexité des calculs, l'influence de l'élasticité des piles intermédiaires et la collaboration de la superstructure (des essais sont en cours à l'université d'Illinois sur ce dernier point). Grâce aux méthodes expérimentales modernes (déformètre de Beggs), les constructeurs commencent à tenir compte de ces influences.

Un tableau des nouveaux ponts en arc construits aux U. S. A. complète cette étude sur l'état actuel de cette question.

Leere Seite
Blank page
Page vide