CHAPTER II

REVIEW OF THE DESIGN OF THE TACOMA NARROWS BRIDGE

The behavior of the Tacoma Narrows Bridge during the hour before the final failure of its center span floor was a severe test of the strength of the structure as a whole, as well as of its major component parts. Under the violent dynamic motions and during the final failure certain parts of the floor structure, the suspenders, the cables and the towers were subjected to stresses far beyond the safe stresses for which these parts were designed, and at many points to stresses under which failure had to result. That under these severe stresses the main carrying members, in particular the towers, did not actually fail attests to the excellence of their design, quality of materials and workmanship, and to the ample margin of safety which the structure would have had except for the severe dynamic motions. A review of the design of the structure on the basis of accepted requirements for strength and other structural qualities confirms that conclusion.

This review is based upon the design of the structure as actually built, and upon our independent calculations and judgment in respect to the intensity of the assumed loads and forces, the manner of their application and the permissible stresses in the various parts of the structure.

The review is limited to the more important parts of the bridge. In a structure of this type and size there are numerous details, the efficiency of which is subject to differences of opinion. Structural details which had evidently no bearing upon the failure of the structure
Besides the calculation of stresses produced in the structure by the assumed static forces, the major deformations, vertical, horizontal and torsional, from these forces have been determined with a view to examine their effect on the structure and on the grades, alignment and tilting of the floor and to compare them with similar deformations in other long span suspension bridges.

In order to explain the rupture of the floor and suspenders, and apparent overstressing of other parts, particularly the towers, an attempt has been made to evaluate the stresses which were developed at various critical points of the structure at the time of its failure. These are necessarily approximations based on the rather scant observations on the deformations of the bridge at the time of failure and on approximate evaluations of the accompanying dynamic effects.

**TYPE OF STRUCTURE, SPAN ARRANGEMENT AND TRAFFIC CAPACITY**

The suspension type of bridge was unquestionably the most suitable and most economical type for the purpose and the locality.

The arrangement of spans and the location of the anchorages were evidently chosen after careful study of the geological and topographical conditions for the pier and anchorage foundations and the economics of the structure, and could not be materially improved. Shorter side spans, and avoidance of the extension of the cables beyond them, would have furnished greater rigidity. It would not have been advisable, however, to effect such a change by moving the anchorages closer to the banks of the river because of danger of sliding, and a material increase in center span would have been more costly.
The capacity of the bridge, a two-lane roadway and two footwalks, appeared from the prospects of traffic development to be ample for many years to come, and as much as could be justified economically. A greater traffic capacity would have added to the cost of the project. To take full advantage of the economy of the narrow roadway, however, the designers adopted an extraordinarily small width of structure compared to its span. Their expectation that the bridge would have adequate lateral rigidity under wind pressure was unquestionably justified.

**ASSUMED STATIC LOADS AND FORCES**

*Dead Load.* Based on a detailed estimate made by the engineers of the Washington Toll Bridge Authority, the average weight of the suspended structure as built is 5,700 lb. p. ft. in the center span and 5,740 lb. p. ft. in the side spans. This compares with a dead load of 6,000 lb. p. ft. of bridge assumed for the design. This quite appreciable decrease of 300 lb. or 5% in the actual weight resulted principally to a modification of the design of the roadway slab and floor steel.

The weight of 5,700 lb. is made up as follows:

- Floor slabs, lighting, etc. 2,650 lb.
- Floorsteel and laterals 720 "
- Stiffening girders 300 "
- Cables with accessories 1,430 "

Total dead load of center span 5,700 lb.

*Live Load.* For the design of the floor system and suspenders the H-20 loading (20-ton trucks) of the specifications of the American Association of Highway Officials was assumed. This loading is now generally adopted for the design of bridges on first class highways. For the stiffening girders, cables and towers a live load of 1,000 lb. is assumed.
Fig. 11: Opening of 1/2 inch between Tower Shaft and base plate on Riverward side of Tower Shaft.
lb. p. ft. of bridge was assumed for the design. This is equivalent
to an average load of about 28 lb. p. sq. ft. of area of the 26 ft.
roadway and the two 5 ft. sidewalks. While this represents a load
condition which is not likely ever to occur over any considerable
length of the bridge, under actual traffic, it does not, in our opinion,
represent the intensity of load concentration which is possible, and
which should be made the basis of design of a structure such as the
Tacoma Narrows Bridge. In a bridge of this span highway live load is
relatively small compared to the dead load and an ample margin for
possible future increase in loading is warranted.

This review is therefore based upon a live load of 1,500 lb. p.
lin. ft. of bridge (42 lb. p. sq. ft. of floor area) which is more
nearly comparable to the loads assumed for other, much wider, recent
suspension bridges with long spans designed for highway traffic only
(Golden Gate Bridge, 4000 lb. p. ft., or 50 lb. p. sq. ft.; Bronx-
Whitestone Bridge, 3000 lb. p. ft. or 46 lb. p. sq. ft.; Triborough
Bridge, 4000 lb. p. ft. or 41 lb. p. sq. ft. This difference in
assumed live load has no bearing on the failure of the bridge, nor does
it affect materially the maximum stresses in the cables, towers and
anchorages, because it is largely offset by the aforementioned reduction
in dead load. Both reduction in dead load, indirectly, and higher live
load affect unfavorably the live load stresses in the stiffening girders,
but since the sections of the latter are largely governed by the wind
stresses the effect is not material.

For maximum live load stresses the load is placed in the most un-
favorable position. For torsion or transverse tilting of the floor it

-42-
is assumed distributed over part of the floor and in the most unfavorable position.

Wind Forces. The following static wind forces were assumed for the design.

On suspended structure, per lin. ft. of bridge:

- Wind on moving load: 200 lb.
- Wind on floor structure (1-1/2 x 8 ft. x 30 lb.): 360 lb.
- Total wind on floor: 560 lb.
- Wind on two cables: 60 lb.
- Total wind on loaded suspended structure: 620 lb.

On towers: 50 lb. p. sq. ft. of exposed area.

This horizontal wind force is equivalent to over 50 lb. p. sq. ft. of exposed area of the unloaded floor, the two cables, and the towers, as seen in elevation, is adequate as a static force for the proportioning of the wind system and has been adopted for the design review.

For the investigation of torsional effects the following assumptions have been made: (a) 200 lb. p. ft. of bridge acting on the moving load applied 6 ft. above the top of the roadway, combined with a vertical load of 400 lb. p. ft. of bridge located on the leeward lane, and (b) a uniform live load of 750 lb. p. ft. of bridge extending from one tower to center of span, applied 39 x 1/4 feet from the near cable and the same load extending from the center of span to the other tower applied 39 x 1/4 feet from the far cable. Assumption (b) causes the most severe torsion near the quarter points of the center span.

Temperature. A maximum change in temperature of plus or minus 40° F. from the normal was assumed for the design. This is entirely sufficient for the locality of the Tacoma Narrows Bridge.
The Floor System. The 26 ft. roadway is carried on a reinforced concrete slab 5-1/4 in. thick. It is flanked on each side by a concrete curb 15 in. high and by a 5 ft. sidewalk on a 2-1/2 in. reinforced concrete slab (Drawing 2). The roadway slab rests on five 21 in. I-beam stringers spaced 5 ft. 9 in. on centers. The stringers are framed into the floor beams which are spaced 25 ft. apart on centers and are plate girders 4 ft. 4-1/2 in. deep. The floor beams in turn are framed into the two main longitudinal girders which carry the loads to the suspenders.

The general arrangement and makeup of the floor system is exceedingly simple and, except for some details, meets the specifications of the American Association of Highway Officials. All material is structural carbon steel.

The curves of vertical deflections under maximum assumed live load at normal temperature are shown in Fig. 15. The maximum deflections from live load and temperature are 13.5 ft. near the quarter points and 13.4 ft. at the center of the center span. The greatest deflection in the side spans is 10.0 ft.

The floor slab and stringers are located near the neutral axis of the stiffening girders, and therefore are practically free from bending stresses from the vertical deflections imposed by the cables. To avoid stresses in the floor system from horizontal deflections and temperature changes the stringers were provided with sliding bearings and the concrete floor with expansion joints 150 ft. apart. In addition the concrete slab had asphaltic joints at every floorbeam. The main expansion joints of the floor structure and of the stiffening girders are located at the
**FIG. 15 - VERTICAL AND TORSIONAL LIVE LOAD DEFLECTIONS OF THE TACOMA NARROWS BRIDGE.**

MAXIMUM TILTING OF FLOOR AT 0.25

17.54%

9°57'
towers. Thus the floor system had great flexibility and adaptability to vertical deformations and probably at no time sustained severe stresses as long as the cables moved in phase.

The vertical deflections from live load and temperature cause an increase in grade in the side span near the towers of 3.93%, thus bringing the maximum grade of the roadway to 6.52%. On a highway bridge and under the extremely improbable load assumptions this grade cannot be considered excessive. It is not greater than on some of the other large suspension bridges.

In a wide and relatively rigid suspension bridge the transverse tilting of the floor and the resulting torsional stresses in the floor structure caused by non-uniformly distributed live load and eccentrically applied wind pressure are negligible, but in a narrow flexible bridge, such as the Tacoma Narrows Bridge, they may be appreciable and must be investigated. For maximum torsion the load condition hereinbefore mentioned was assumed (Fig. 15). Under this very improbable condition the floor at the quarter point of the center span would tilt sideways 17.5% or 6.84 ft. in the width of 39 ft. While such a torsion would probably never be produced under actual load conditions it is considerably in excess of similar tilting in other large suspension bridges (Table 4) and indicates the relatively great torsional flexibility of the bridge. The stresses resulting from this highly improbable but possible static load condition in the floor structure are, however, well within permissible limits.

Stiffening Girders and Wind Truss. The two main girders are located in the planes of the suspenders and cables 39 feet apart on centers.
They are hung from the rope suspenders spaced 50 ft. apart. They are solid plate girders 8 ft. 1/2 in. deep b. to b. of flange angles. Besides carrying the floor they act vertically as stiffening girders and horizontally as chords of the wind truss.

Each girder is made up of a web plate 96 in. x 1/2 in., 4 angles 8 x 6 x 1/2 in. and 2 cover plates 20 x 1/2 in. It has a sectional area of main material of 95.0 sq. in. and for stiffening an effective moment of inertia about the horizontal axis of 1283 in² ft.². Both web and flanges are of structural silicon steel.

The general arrangement of the stiffening system and the make-up of the stiffening girders are not unusual. Plate girders have been used in a number of suspension bridges. They are being adopted to a growing extent, and with increasing proportions, in bridges of all types because of economy and structural simplicity. When the required depth exceeds the practicable or economical depth of plate girders, open trusses are the proper solution.

The 8 ft. girders are so shallow and flexible that they exerted almost no stiffening effect upon the cables, except locally. Evidently the designers had relied on the weight of the long span of the bridge and on the relatively small cable sag to provide the necessary vertical rigidity. There is every evidence that insofar as rigidity under moving loads is concerned their judgment was justified. They had not expected that aerodynamic action could become so powerful as to put the great suspended mass of the structure into dangerous or objectionable motion.

Under the static live load, temperature changes and lateral wind forces assumed for this review the stiffening girders are subject to
the following maximum stresses in lbs. p. sq. in.

<table>
<thead>
<tr>
<th></th>
<th>Near 1/4 Point of Center Span</th>
<th>Near Center of Side Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>Compression</td>
<td>Tension</td>
</tr>
<tr>
<td>Dead load + live load + temperature</td>
<td>9,780</td>
<td>9,780</td>
</tr>
<tr>
<td>Wind only</td>
<td>23,350</td>
<td>19,840</td>
</tr>
<tr>
<td>Dead load + 1/2 live load + temperature + wind</td>
<td>26,890</td>
<td>23,730</td>
</tr>
</tbody>
</table>

These stresses are within permissible limits for structural silicon steel, the minimum yield strength of which is 45,000 and the ultimate strength 80,000 lbs. p. sq. in. The fact that the top flange of the girder is elastically braced at the floorbeams has been considered. The critical buckling strength of the flange is calculated at 40,000 lb. p. sq. in.

The K system of laterals between the two stiffening girders are located in a plane below the bottom flanges of the roadway stringers to which they are fastened at the points of intersection. All laterals are 8 in. WF beams. The stresses in the laterals have been determined by assuming the wind as a moving load placed in a position to cause maximum shear in the respective panels of the wind truss.

The largest stresses occur in the laterals at the towers, where they are 21,170 lb. p. sq. in. in tension and 17,150 lb. p. sq. in. in compression (1/r = 77). These laterals are of silicon steel. The largest stresses in carbon steel laterals are 15,540 and 12,570 lb. p. sq. in. respectively. These stresses are within permissible limits.

The width of the wind truss between centers of stiffening girders
is 39 ft. or only 1/72 of the span length. It is thus the most slender and flexible wind truss of any of the large suspension bridges, the nearest being the Golden Gate Bridge with a width to span ratio of 1/47. The lateral deflection of the truss at mid-span under full wind force on the center span is 20.0 ft. or 1/140 of the span length. This deflection is controlled largely by the cables to which the major part of the floor wind load is transmitted (Fig. 16).

The lateral deflection does not produce excessive stresses in the floor structure. In a suspension bridge the cables and, with them the wind truss, remain always in stable equilibrium and there is no danger of the wind truss buckling as a whole laterally in its plane, such as might be the case with a flexible wind truss in a simple span, cantilever or arch bridge in which compression chords of the main trusses form the chords of the wind truss. The wind truss of the Tacoma Narrows Bridge is discontinuous at the towers so there are no sharp curvatures in its elastic line and therefore no unusual secondary stresses in its members from the deformations.

Furthermore, it has been observed generally that the actual lateral deflections and therefore stresses from wind forces remain below those considered in the design. Wind pressure never acts over such great lengths with its maximum intensity. In the Tacoma Narrows Bridge the lateral deflection of the floor was observed as about two feet under the wind velocity of 42 mi. p. hr. on the day of failure. The calculated lateral deflection of 20 feet in the Tacoma Narrows Bridge would cause an horizontal rotation of the floor at the towers of about 2.8%, which would be noticeable. Again under actual conditions the angle would remain
FIG. 16—DISTRIBUTION OF 620 LBS PER LINEAL FOOT DESIGN WIND LOAD AND RESULTING LATERAL DEFLECTIONS OF THE TACOMA NARROWS BRIDGE.
below this value and it has therefore no significance in its effect upon traffic.

Neither the horizontal deflections resulting from wind nor the angular change at the towers are a measure of the allowable width to length ratio or of the required rigidity of suspension bridges.

THE CABLES AND SUSPENDERS

The Suspenders. The suspenders from which the floor is hung at points 50 ft. apart consist at each point of four parts of 1-1/4 in. diameter galvanized steel ropes with an average ultimate strength of a two part rope tested over a sheave of 298,300 lb. The average for a single rope was 165,400 lbs. The maximum axial stresses in a four rope suspender are: Dead load 106,000 lb.; live load 45,900 lb.; impact 10,200 lb.; total stress 162,100 lb. This is 27.2% of the ultimate strength of the four ropes combined and provides an ample margin under the assumed loads.

The Cables. The floor structure is suspended from two cables, one on each side of the floor in the plane of the stiffening girder. The cable sag of 232 feet at normal temperature, or 1/12 of the span length, is smaller than that of other modern suspension bridges in which it ranges generally between 1/9 and 1/11 of the span (Table 3). It is well known that a small cable sag produces greater rigidity against both vertical and lateral deformations under static loads, and it was for that reason and for the express purpose of producing greater rigidity that the designers adopted relatively flat cables, undoubtedly at some extra expense.

The analysis which is contained herein, corroborated by the behavior
of the structure, indicates that the effect of a flat cable on the vertical rigidity of the suspended system was overestimated, and was by far not sufficient to offset the flexibility permitted by other elements of the structure.

Each of the two cables consists of 6308 No. 6 galvanized cold drawn steel wires laid parallel in 19 strands of 332 wires each. The theoretical net area of wire in each cable is 190.32 (actual 191.296) sq. in. The diameter of the cable after wrapping is 17-1/4 in. The wire has a specified average strength for each heat of at least 225,000 lbs. per sq. in. The average of actual tests was 235,000 lbs.

The maximum axial stresses in the cable based on 190.32 sq. in. wire area are:

- From dead load and live load (1500 lb. p. ft.) and temperature 40°F. below normal, 84,500 lbs. p. sq. in.
- From dead load only at normal temperature, 67,700 lbs. per sq. in.

These axial stresses are not excessive and reflect a recent tendency toward higher unit stresses in wire cables which is justified by the greater uniformity of material and superior practice in splicing and laying the wires in the cable. The axial stresses in a suspension bridge cable include an allowance for secondary or bending stresses which are produced at the saddles, principally those at the tops of the towers, when the cables deflect under load. In the Tacoma Narrows Bridge these bending stresses are in the order of 5% of the primary stresses under maximum deflections from live load and temperature.

The Towers. The two towers as seen in side elevation are flexible shafts 425 ft. high from top of pier to center of cables, 19 ft. wide
at the bottom and 13 ft. at the top. As seen in transverse elevation they form simple rigid frames. The two shafts which have a uniform width of 13 feet from top to bottom, and on which the cables are centered, are connected by four deep horizontal struts, two above and two below the floor (Drawing 3). The two shafts are slightly battered so that the width between shaft centers increases from 39 ft. at the top to 50 ft. at the base.

Each shaft has a cellular cross-shaped section built of solid web plates, connected by angles. The sectional area varies between 898 sq. in. at the top to 1199 sq. in. at the bottoms. The bottom section is increased to 1879 sq. in. at the base. The maximum moment of inertia of each shaft is 2,061,000 in⁴ longitudinally and 3,250,000 in⁴ transversely. The struts are box shaped sections 3 ft. 11 in. wide and of depths ranging between 27 and 35 ft.

All materials in shafts and struts are structural carbon steel with a minimum yield strength of 33,000 lbs. per sq. in. and ultimate strength of 60,000 lbs. per sq. in.

The tower shafts are firmly anchored to the piers by a steel anchorage frame which extends to 16 ft. below the base into the concrete.

The general design of the towers and major details are in accordance with best recent practice.

The maximum unit stresses in lb. p. sq. in. in the towers from a combination of dead load, one half live load, temperature and wind are as follows:

<table>
<thead>
<tr>
<th></th>
<th>At Point of Maximum Axial Stress - 16 ft. above base</th>
<th>At Point of Maximum Bending Stress - 163 ft. above base</th>
<th>At Point of Maximum Combined Stress - 392 ft. above base</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Compression Stress</td>
<td>17,600</td>
<td>13,100</td>
<td>14,700</td>
</tr>
<tr>
<td>Bending Stress</td>
<td>2,200</td>
<td>9,500</td>
<td>8,800</td>
</tr>
<tr>
<td>Combined Stress</td>
<td>19,800</td>
<td>22,600</td>
<td>23,500</td>
</tr>
</tbody>
</table>
With the towers rigidly fixed at the base the value of $l/r$ can be taken at 60 and the permissible axial stress at 18,000 lb. p. sq. in. The permissible value for combined axial and bending can be assumed at 22,000 lb. p. sq. in. The maximum calculated stresses are therefore somewhat high, but in view of the excellent behavior of the towers under far greater stresses they cannot be considered unduly excessive.

The maximum longitudinal deflection of the towers at the top from live load (1500 lb. p. ft. of bridge) and temperature is 2.34 ft. riverward.

DAMPING INSTALLATIONS

The installations erected in the bridge prior to its opening to traffic, in order to dampen the oscillations from wind which had developed during erection of the deck structure, consisted of hydraulic jacks at each tower, acting longitudinally against the end of the floor structure of the center span, and diagonal stay ropes connecting the cables at the center of the span to the stiffening girders. These devices followed in general similar installations which had at that time been placed in the Bronx-Whitestone Bridge and, after various improvements, has proven effective. The amplitudes of the oscillations on the Bronx-Whitestone Bridge before damping devices were installed were, however, much smaller.

On the Tacoma Narrows Bridge these devices evidently reduced longitudinal motions of the floor, but had little effect in reducing the vertical oscillations, partly because the design of the devices was not entirely adequate and partly because the amplitudes were too great.

Hydraulic Jacks. The devices at the towers were designed primarily to steady the floor against dynamic longitudinal motions from wind action,
These stays were intended primarily to assist in checking longitudinal motions of the floor by tying the latter to the cables. Incidentally in combination with fixed ends of the floor they tend to dampen the vertical oscillations of the cables by restraining the longitudinal movement of the center of the cables. They could not, and were not intended to, prevent longitudinal motions of the cables entirely, because this would involve very large stresses in the stays and would materially alter the deformations and the stress condition in the cables. These center stays, because of their short length and therefore limited elastic stretch, can therefore be designed only for very moderate deformations of the cables. Within such deformations they exert a noticeable, but limited, damping effect. This was clearly observed on the Bronx-Whitestone Bridge. On the Tacoma Narrows Bridge it was apparently less marked. When the motions become as large as they did in the Tacoma Narrows Bridge, and the stays do not have a high initial tension, the cables impose alternately slackening of one and excessive strains with impact on the other of the two stays. The result is that under large distortions of the cables the stays, or their connections, may break or, as happened in the Tacoma Narrows Bridge, the cable band to which they were attached may slip when the stay rope stress becomes sufficient to overcome the frictional resistance of the cable band.

Similar experience on the Bronx-Whitestone Bridge in respect to alternate slackening of the central stay ropes, as at first installed, led to their replacement by heavier ropes, without turnbuckles, with more rigid connections, and with a large initial tension which prevented slackening.

From these experiences the conclusion may be drawn that central
stay ropes, when properly designed, exercise a limited damping effect on vertical oscillations, but they are largely ineffective when loose and may be a source of danger when the motions of the cables become large. Observations on the model of the Bronx-Whitestone Bridge showed that the damping effect of the central stays becomes overshadowed by the effect of stay ropes from the tops of the towers to points on the floor.

The Slipping of the Cable Band and its effect on the motions. It can be estimated that under certain possible force conditions, particularly torsional forces which tend to create out-of-step oscillations of the two cables, stresses approaching the strength of the center stay ropes or the frictional resistance of the cable band can be produced.

The stay ropes had a strength of at least 270,000 lb. The cable band was tightened by six bolts with a stress which may reasonably be estimated at 225,000 lb. The cable band friction was then probably at most 60% or 135,000 lb. It is possible that, under the rapidly alternating and eccentrically applied dynamic stresses from the stay ropes, the bolts, and therefore the cable band, became looser and the frictional resistance decreased.

After the north cable band slipped, the cable moved forward and backward through the band with a measured longitudinal amplitude of 40 in. The resistance of the band practically disappeared, thus bringing about, perhaps suddenly, a materially different dynamic condition of the suspended structure.

There can be little doubt that, as long as both cable bands were tight, and the stay ropes tied the cables to the floor, they were effective.
in preventing out-of-step motions of the cables with a node at the center. When one cable band became ineffective it rendered the structure more sensitive to torsional motions. Its sudden slipping induced a large torsional moment in the horizontal plane which in turn caused the cables to oscillate out of phase. This action was possibly aggravated by dynamic effects created by the sudden slipping of the cable band. Observations indicated that the change from the moderate parallel motions of the cables to the more violent out-of-phase motions was sudden.

This explanation of what happened appears to be strengthened by the results of torsional tests made by Prof. F. B. Farquharson on the model of the Tacoma Narrows Bridge. These tests indicate that without center stays the bridge was considerably more sensitive to torsion than with stays attached to both cables. Therefore the stays must have exerted considerable restraint on the cables to get out of phase and this accounts for the fact that no torsional motions were observed as long as the stays held both cables. The model tests show also that with stays removed at one cable the bridge became more sensitive to torsion although not as much as without stays.

Stay Ropes at Towers. Mention of the studies made by the engineers of the Washington Toll Bridge Authority with respect to the installation on the Tacoma Narrows Bridge of stay ropes attached to the tops of the towers, as an additional damping device, and comments on the effect such stays might have had on the behavior of the bridge, are contained elsewhere in this report. The study of these stays, although undertaken promptly, together with their proper design and installation was bound to consume months. The failure occurred before such installation could be made.
Tie-Down Ropes in Side Spans. Some time prior to the failure wire ropes attached to the floor in the side spans and firmly anchored were installed with an initial stress of 100,000 lbs. in an effort to dampen the motions. These ropes evidently had an effect in damping the side spans, but, inasmuch as the center span motions were to a large extent independent of the side spans, the effect of the tie-down ropes on the amplitudes of the center span oscillations was not appreciable. Their presence could have had no influence upon the failure of the floor structure.

**STRESS CONDITION OF SUPERSTRUCTURE AT TIME OF FAILURE**

The calculations of stresses made on the basis of the observed dynamic deformations of the structure show that where failure or permanent local damage in various members occurred, the stresses had reached at least the yield strength of the composite member. This indicates that neither faulty design, nor defective material or workmanship, contributed to such failure or to permanent local damage.

Under the alternating 45° torsion in the final one-node motion the floor system, more particularly the concrete floor slab, received its most severe stresses near the center of the center span. The steel floor had great flexibility to adjust itself to the large distortions, but the torsional shearing stresses in the concrete slab exceeded the ultimate strength and this explains the breaking down of the slab in that vicinity as one of the first failures. Severe shearing stresses were also caused between the floor slab and the stiffening girders, accounting for the longitudinal motions observed between these parts. Examination of remaining parts of the floor slab confirms its excellent qualities.

During the vertical parallel motions of the cables, with double
amplitudes up to 5 ft., and the mode of oscillation corresponding to a frequency of 38 p. min., the stiffening girders were subjected to vertical bending which caused stresses approaching the buckling strength of the flanges. Under the more severe torsional motions, the girders were subjected to breaking stresses near the center of the center span, where the first buckling was observed. The buckling of the girders near the quarter points of the center span can be explained by a combination of stresses from bending in the plane of the web, alternate 45° torsion and bending stresses in the flanges from the lateral component of the suspender pull.

The side span girders were most severely stressed when a portion of the floor structure of the center span tore away, thus causing the side spans to deflect at their center as much as 60 ft., six times the maximum live load and temperature deflections. This vertical deflection alone produced stresses which caused the buckling of the girders in that vicinity. The failure occurred at web splices where the horizontal Z stiffeners were discontinuous (Fig. 7). The buckling stress of the compression flange was reached at a deflection of about 40 ft. With increasing deflection the buckling progressed rapidly through the depth of the girder.

The type of failure clearly indicates that the floorbeams and the stiffener angles which connect them to the girder were rigid enough to limit the buckling length of the top flange, and of the whole girder, to the panel length of 25 feet. The top flange apparently had ample lateral support.

Under the severe tilting of the floor, each of the two parts of a rope was subjected alternately to severe increases in the axial dead
while permitting the slow but much larger motions from temperature changes. Hydraulic jacks acting between the ends of the floor and the towers (Drawing 3) were adopted for the Tacoma Narrows Bridge to act as bumpers or shock absorbers. Acting in this way they prevented longitudinal motion only partially because the structural details transmitting the force from the floor to the hydraulic jacks permitted considerable play with the result that the installation proved largely ineffective in checking vertical oscillations.

Experience at the Bronx-Whitestone Bridge proved that such damping devices at the towers were not fully effective until they had been made sufficiently strong and rigid to prevent play and elastic yield. This was possible only by fixing the ends of the floor to the towers by direct friction which permits motion only when, under the relatively much larger temperature forces, the frictional resistance is overcome. The effectiveness of this device in damping vertical oscillations was observed, not only on the bridge proper, but was subsequently demonstrated on the model of the Whitestone Bridge.

It is doubtful that even similarly effective friction devices in the Tacoma Narrows Bridge would have damped the structure sufficiently to prevent, or materially reduce, the much larger oscillations.

Stay Ropes at Center of Span. The stays installed at the center of the center span of the Tacoma Narrows Bridge consisted of a pair of 1-1/2" wire ropes at each cable, extending from the central cable band about 14 ft. on each side to points on the stiffening girder (Drawing 3). They were provided with adjustable turnbuckles and were erected with a nominal initial tension. Each rope had a specified minimum strength of 270,000 lb.
load stress, on account of the frictional resistance of the rope over the cable. This is evidenced by observations to the effect that at some suspenders one part of a rope was slack. In addition the ropes were subjected to sharp alternate local bending of 45° at their connections to the girders. The dynamic effect of the falling mass of the structure hung from the suspenders greatly exceeded the corresponding static stresses. Under these dynamic stresses the ropes failed. After failure of one suspender, that of the others progressed rapidly, resulting in the successive dropping of large portions of the floor structure of the center span.

During the large deflections of the side span the bending stresses in the cables at the top of the towers, together with the axial stresses, were much higher than the safe design stresses. There is not enough information available that would permit an even approximate appraisal of the stresses actually produced. While surface examination does not reveal breaking of wires, except where caused by the cable band slipping, it is not certain that incipient failure was not produced at other points.

The stresses in the towers reached their highest values when, as a result of the dropping of part of the center span suspended structure, the side spans were subjected to a vertical deflection of about 60 ft. The corresponding shoreward deflection of the towers at their tops was about 27 ft. or 12 times the maximum deflection from full live load and temperature. Throughout the height of the tower shafts the bending stresses reached the critical buckling stress in the shoreward cover plates, accounting for the buckles that are plainly visible in these plates. The stresses on the tension side also reached the yield
strength of the material. The splices failed probably by shearing deformation of the rivet shank. We do not have sufficient information to permit a definite statement as to the damage to the main material on the tension side. This severe bending also overstressed the upper part of the tower anchorage frame, lifting the tower base on the riverward side by about 1/2 in. above the pier surface without, however, damaging the pier except locally. During the torsional motions of the center span and the sagging of the side span the towers were subjected to torsion about their vertical axes. This caused severe stresses and consequent permanent deformation in the top strut between the two tower shafts.

DESIGN OF MAIN PIERS AND ANCHORAGES

Main Piers. The two main piers which support the steel towers are of rectangular section with beveled corners. The pier shafts are of cellular concrete construction 64.5 ft. wide and 117.5 ft. long. They extend from an elevation of 23.09 ft. above to depths of 168 and 217 ft. below mean low water, where they are founded on a firm bed of sand and gravel. At the bottom the caissons are sealed with a 25 ft. layer of concrete and the top is formed by a solid concrete slab 20 ft. thick.

In the pier design forces and dynamic action from tidal currents, waves and ship impact were considered. The resulting stresses in the piers and the soil pressures are found to be conservative. Under combinations of weight, live load, temperature and one-half the assumed wind force, or weight and full wind force, the extreme edge pressures range between 9.2 and 9.7 tons per sq. ft., and the stresses in the concrete are within 400 lb. p. sq. in. A combination of tidal currents,
waves and ship impact is estimated to increase the extreme edge pressure between 3 and 5 tons. Of the above pressures only about 1.9 tons are due to the weight and forces acting on the superstructure, and about 5.5 tons to the great mass of the piers themselves. Even under the severe deflections of the towers at the time of the failure the additional stresses imposed upon these massive piers were comparatively small and the piers could therefore not have suffered in any way from the failure except locally near the tower base. The margin of carrying capacity of the piers is such that they may be utilized for a new structure, with certain alterations near the top, even if the weight and live load of the superstructure should be doubled.

Anchorages. The cable anchorages are concrete blocks which rest on a sand and gravel formation about 100 ft. above mean low water and about 500 ft. back of the river banks. The layer of sand and gravel is about 80 ft. deep and overlies a bed of clay approximately 100 ft. thick. The maximum cable pull combined with the weight of the anchorage blocks, neglecting passive earth resistance in front of the anchorages, causes maximum bearing pressures of 3.8 tons per sq. ft. and horizontal forces equal to 44% of the resultant vertical reaction. This provides sufficient resistance against sliding of the anchorages. As would be expected, the anchorage suffered no damage as a result of the failure and they may be safely utilized in a new structure. It would not be advisable, however, to increase the bearing pressure nor to permit an increase in the ratio of horizontal pull on the anchorage to the resultant vertical reaction.