

HYATT-REGENCY WALKWAY COLLAPSE: DESIGN ALTERNATES

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ABSTRACT: The 1981 failure of walkways in a Kansas City hotel occurred because of inadequate connections between hanger rods and bridge beams. This paper briefly reviews the collapse and examines means of strengthening the connections, such as had been suggested in the aftermath. The additions of a heavy washer plate and of an end plate are analyzed by approximate plastic-design methods, and the results of simple tests on replicas are reported. Generally, good agreement among analytical implications, experimental results, actual loads, and conclusions by the National Bureau of Standards was found. The various indications of capacity are compared.

INTRODUCTION

On July 17, 1981 two walkways spanning the lobby of the Hyatt-Regency Hotel in Kansas City collapsed without warning (1). Although the failure took place under relatively low loading, 114 persons eventually died as a result of their injuries, making this the worst structural disaster, in terms of the number of lives lost, in recorded history. The incident generated much interest on the part of the profession, the news media, and the public.

The writer served as a consultant to a television station and to a newspaper and was privy to much of the available information. (The television station, Kansas City ABC-affiliate KMBC, made videotapes at the time of the collapse, and these proved to be an essential source of information for later studies. The newspaper, *The Kansas City Times*, together with its sister publication, *The Kansas City Star*, was later awarded the Pulitzer Prize for its coverage and analysis). Hard news was accompanied by a good deal of speculation as to the causes and possible means of avoidance. Subsequently, the causes were thoroughly investigated by the National Bureau of Standards (NBS) and explained in a February, 1982 report (3). As had been fairly evident from an inspection of the wreckage, the initial failure occurred when the nut and washer at the end of a hanger rod pulled through a double-channel box beam. The load on this connection was probably about 90 kN at the time.

As to possible avoidance of the failure, speculation has mainly centered around the question of whether this connection could and should

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have been strengthened by stiffening the webs or by using an over-size, plate-like, washer. This paper analyzes these two possible means of enhancing the connection and reports the results of some rather simple tests. The results are compared with the NBS findings.

The two walkways, about 36 m long, and about 2.5 m wide, had been constructed as shown schematically in Fig. 1. There were four spans, each about 9 m long, supported at the ends and by three rod-and-beam assemblies as shown. The hangers consisted of 32-mm rods, and the beams of two MC 200 $\times$ 12.6 (MC 8 $\times$ 8.5) channels butt-welded together along the center line so as to form a box. Both were made of ASTM A36 steel.

The working drawings called for continuous rods from the fifth-level ceiling through the fourth-level bridge and terminating at the second-level bridge. Apparently, because of construction difficulties, shop drawings were approved that called for using two sets of rods, one terminating at the upper bridge and one starting there, so as to suspend the lower bridge from the upper one (2). Figure 2 contrasts the two designs.

Based on weighings, NBS determined the weight of the two walks on one upper hanger, i.e. the correct design dead load, to be about 70 kN

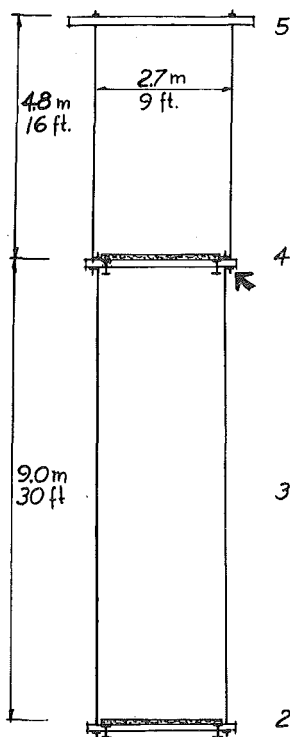


FIG. 1.—Arrangement of Walkways

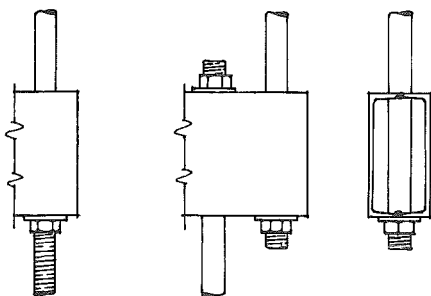


FIG. 2.—Modification of Connection

(2). The Uniform Building Code (6) used in Kansas City calls for 5 kPa on hallways, which results in a corresponding live load of 110 kN. The total design load consequently is about 180 kN. If load factors of 1.7 and 1.4 are applied to live and dead loads, respectively, a required load capacity of 285 kN can be inferred. The capacities reviewed in the following sections should be compared to this estimate.

ANALYTICAL ESTIMATES

No analytical inquiry into the capacity of such a connection can be very precise. The complicated shape with its fillets and holes requires greatly simplifying assumptions, and uncertainty regarding weld penetration, washer properties, and the like also impairs the reliability of the calculations. Nevertheless, the fundamental structural theory can provide some qualitative insights and some quantitative estimates.

The most obvious approach might be to consider the cross section of the box beam to be a portal, as shown in Fig. 3(a), with the load acting as a concentrated force at the center. A plastic mechanism, as shown in Fig. 3(b), could be expected to form. (Any strength added by the washer will be considered later.) The major uncertainties concerning this model include: (1) The magnitude of the plastic moment at the center (i.e. at the weld seam); and (2) the extent of the plastic hinges in the longitudinal direction (i.e. normal to the plane of the section). Concerning the first point it may be reasonable to assume the hinges in the webs extend to the end of the beam on the outside of the walkway and a distance of perhaps the depth of the beam, d , on the inside. These distances are 50 mm and 200 mm, respectively. Concerning the second point, one might consider the plastic moment developed to approximate half of the plastic moments in the webs, on the basis of estimated weld properties and dimensions. A cautious designer might wish to disregard the bending resistance of the weld entirely.

Equating external work to internal work then results in

$$F = \frac{3M_p}{b} \dots\dots\dots (1)$$

in which F = load producing a plastic mechanism; M_p = the plastic moment in the web; and b = width of each flange. The value of M_p is given by

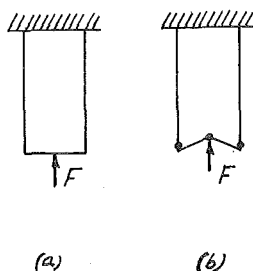


FIG. 3.—Simple Portal Mechanism: (a) Load Acting as a Concentrated Force at the Center; (b) Plastic Mechanism

$$M_p = \frac{1}{4} a w^2 \sigma_y \dots \dots \dots (2)$$

in which a = extent of the hinge; w = web thickness; and σ_y = yield stress of material.

The value of a was estimated previously, while b and w may be obtained from industry tables (4,5), and σ_y can be assumed to be the minimum for ASTM A36 steel. These values are: $a = 250$ mm; $b = 47$ mm; $w = 4.5$ mm; and $\sigma_y = 250$ MPa. Substitution yields a plastic moment M_p of 315 N·m and a collapse load, F , of 20 kN.

Such a simplified analysis model would obviously yield a capacity far short even of the small load that in fact produced failure. While it might serve as a preliminary guide to a designer, it does not provide an accurate estimate of capacity. The fault lies mainly in the assumption that one concentrated load acts at the center. In fact the load is transmitted, after some initial bending of the flange, by the two edges of the nut. The load thus acts not at the toes, but near the middle of the flanges, the external work is effectively halved, and the apparent capacity is consequently doubled.

Another phenomenon can be observed, if not predicted. During the initial stages of loading, before plastic behavior prevails, the box beam, under bending moments as diagrammed in Fig. 4(a), deforms approximately as indicated in Fig. 4(b). As the webs are also subjected to compressive normal forces, instability in a beam-column mode occurs, inducing the eventual formation of a plastic hinge somewhere in the lower half of the webs. The resulting mechanism [Fig. 4(c)] requires the formation of another hinge in each web, at the top.

Assuming the load is transmitted by the bolt and washer to the mid-points of the flanges, and assuming further the hinges in the web form at the quarter-points, the simple plastic mechanism analysis now yields a load of

$$M = \frac{26}{3} \frac{M_p}{b} \dots \dots \dots (3)$$

Using the same values for M_p and b as before, the load F becomes 58 kN. If one also considers the yield stress of the material is probably higher than the minimum—NBS found it to be about 300 MPa—a fair estimate

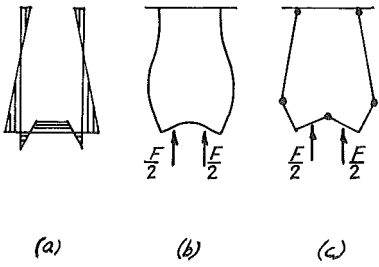


FIG. 4.—Observed Portal Mechanism: (a) under Bending Moments; (b) Deforming Approximately; (c) Resulting Mechanisms

for the load producing a mechanism becomes $F = 70$ kN. While this value is not unlike the load that indeed produced collapse, one should note that it based on assumptions, such as the one regarding the yield stress, that hardly follow conservative design practice.

In the aftermath of the disaster, one question frequently asked concerned the effect the possible omission of a washer might have had, or the substitution of a special, large washer plate would have had. Any washer, if allowed to slide against the flanges, would act rather as a simple beam inserted between the nut and the bottom flanges of the beam, and a hinge similar to that at the weld would be formed in it. Using the same approach as before, the additional load capacity is

$$\Delta F = 4 \frac{M_{pw}}{b} \dots \dots \dots (4)$$

in which M_{pw} = the plastic moment in the washer, that is

$$M_{pw} = \frac{1}{4} s t^2 \sigma_y \dots \dots \dots (5)$$

in which s = width of the washer in the longitudinal direction (less hole diameter); and t = thickness of the washer. A square plate, as wide as the beam, will illustrate the effect. If s is $100 - 33 = 67$ mm, t is 6.35 mm, and σ_y is 300 MPa the additional capacity might be 17 kN. For a normal washer of 75 mm $\times$ 3 mm the effect would be 1.1 kN; an insignificant amount in the context of the given precision. In each case, friction between washer and flange would increase the effect, but hardly by much.

Among other post facto suggestions was one to modify the box beam by welding a rectangular plate across its end, thereby hindering the bowing out of the webs and thus inhibiting the tearing of the weld seam. No simple yet reliable rational analysis of this configuration occurs to the writer, but the effect of such a modification might be likened to the one that would have been produced had the beams been extended some considerable distance beyond their actual cut-off points. One might as-

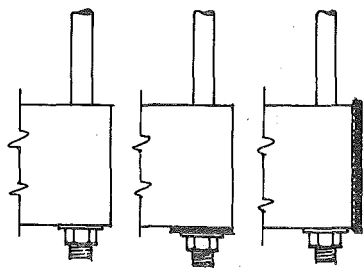


FIG. 5.—Test Specimens

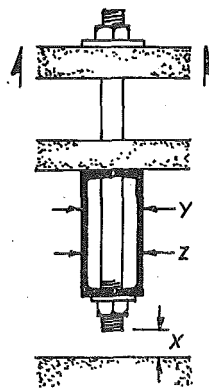


FIG. 6.—Test Arrangement

sume the total internal work to be twice that developed at the inside of the hanger rod. In that case, Eqs. 2 and 3 could again be used for an estimate, with a now being 400 mm rather than 250 mm. If the other parameters are as before an incremental load ΔF of 42 kN would result.

To summarize, the following loads producing the onset of distortion by plastic mechanism can be estimated:

1. Beam without washer: 70 kN.
2. Beam with washer: 71 kN.
3. Beam with washer plate and washer: 88 kN.
4. Beam with end plate and washer: 113 kN.

In each case nonlinear deformations can be expected at a lower load, as soon as the yield stress is exceeded anywhere. Final collapse will occur at a somewhat greater load after the material somewhere has passed through the plastic and the strain-hardening ranges.

An issue of incidental interest is the capacity of the hanger rods as such. For the cross-sectional area given, the minimum yield stress of A36 steel corresponds to a yield load of 200 kN. This value lies above the capacities of the connections considered, but below the required capacity.

TEST SPECIMENS, PROCEDURE, AND RESULTS

Three specimens were tested. One of these resembled those used in the actual structure and was intended to serve as a basis of comparison for the modified specimens, so as to be able to relate the test results to the behavior of the structure. The other two specimens were reinforced, each in a particular manner, but otherwise were made as similar to the control specimen as possible in all respects: materials, dimensions, and

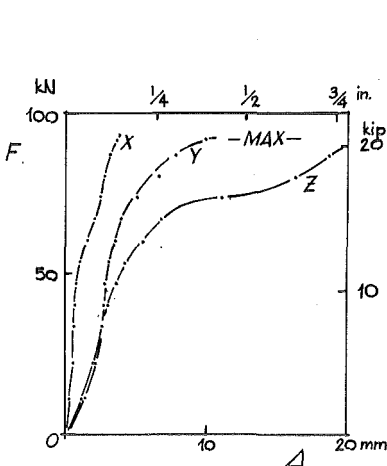


FIG. 7.—Load-Deformation Curves for Control Specimen

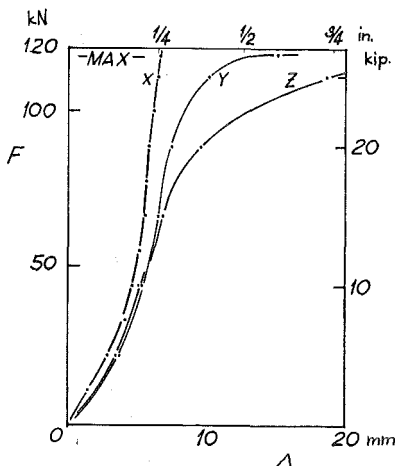


FIG. 8.—Load-Deformation Curves for Specimen with Bottom Plate

fabrication. One of these specimens was supported on the bottom by a square plate placed just above the washer. The other was fitted with a rectangular end plate slightly larger than the cross section of the box beam and fillet-welded to the outside of webs and flanges all around. The specimens are shown in Fig. 5.

The test specimens resemble the beams at the fourth-floor level in that the rods and holes are centered 51 mm from the ends. Also, they were made long enough (0.6 m) so that all stresses would effectively be dissipated short of the far (inside) end. The specimens, however, did not have holes to accommodate the rods supporting the lower walkway, and the downward force was applied concentrically to the upper flange, rather than by these rods and the stringers. Figure 6 schematically demonstrates the test setup in a hydraulic universal testing machine.

As indicators of deformation, the following measurements, shown in Fig. 6, were taken:

1. x = The vertical movement of the bottom of the pull-rod with respect to the platen (by dial micrometer). It should be noted this value is only about half the movement of the toes of the channels.
2. y = The lateral bulging of the flanges at the third-point from the top, at the lateral point of maximum deformation (by dial caliper).
3. z = The same as y , but at the third-point from the bottom of the flanges.

In the case of the control specimen, bulging along the center line was also measured, but these deformations invariably fell between those at

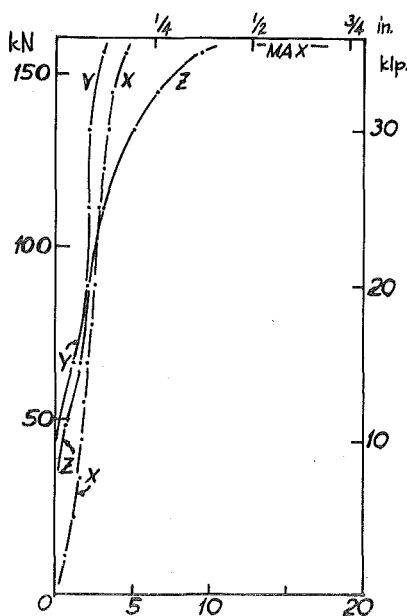


FIG. 9.—Load-Deformation Curves for Specimen with End Plate

the third-points and are not reported here. Maximum bulging, in fact, always occurred below the lower third-point, roughly at the lower quarter-point.

Test speeds were manually controlled and thus varied somewhat, but crosshead speeds were kept low, at about 0.2 mm/min. The loading was briefly stopped at regular load intervals for the purpose of measuring deformations.

The test results are shown in Figs. 7–9 in which the test loads are plotted versus the deformation parameters x , y , and z . The collapse mechanism assumed in the previous section implies a value of z almost four times greater than x (not quite, because a hinge would form slightly below the location of z). In the cases of the control specimen, as well as of the one with the bottom plate, this became true at about the load level when x started increasing plastically, i.e. even under constant load. In the case of the specimen with the end plate, this state was not reached, plastic movement occurred only near the maximum load, and the hinge was seen to form quite near the bottom (about 3 cm). Measurements were abandoned when the load started dropping off, which in each case proceeded stepwise as the bottom weld seam popped.

TABLE 1.—Comparison of Load Levels

Load Level, in		Explanation (3)
Kilonewtons (kN) (1)	Thousands of pounds (kip) (2)	
285	64	Factored load (safe design capacity)
·	·	
·	·	
·	·	
200	45	Minimum yield strength of rods
180	40	Dead load plus required live load
159	35.8	Failure of specimen with end plate
142	32	Half of factored load (required before design modification)
117	26.4	Failure of specimen with bottom plate
113	25	Collapse mechanism with end plate ^a
91	20.5	Failure of control specimen (replica)
90	20.2	ACTUAL FAILURE LOAD (NBS estimate)
88	20	Collapse mechanism with bottom plate ^a
71	16	Collapse mechanism by realistic analysis ^a (also: approximate dead load)
·	·	
·	·	
20	4.5	Collapse mechanism by simple analysis (reasonable estimate of minimum strength)
·	·	

^aAs estimated in this paper—not by conservative design methods.

The maximum loads reached were as follows:

1. Control specimen: 91 kN.
2. Specimen with bottom plate: 117 kN.
3. Specimen with end plate: 159 kN.

The capacity determined for the control specimen agrees quite closely with results obtained by NBS (2) and the apparent loading of the actual structure at the time of collapse.

In the various tests the deformations, as well as observations during the tests, suggest that mechanisms formed at load levels of perhaps three-fourths of the maxima.

CONCLUSIONS

The load capacities obtained by analytical estimation amount to only 70%–80% of the ultimate test loads. This discrepancy can be explained primarily by the fact the ultimate strength of A36 steel is appreciably higher than the yield strength. While the latter was used in the calculations, the former was surely reached or approached in some parts of the connections during testing, thus bringing some reserve strength into play. During the tests, plastic collapse tended to become visible at about the load levels analytically obtained.

The significant theoretical, test and actual load capacities are summarized graphically in Table 1. It is apparent that modifications of the box-beam-and-rod connections of the type considered here would have strengthened the structure somewhat, but not adequately. It is ironic, and indeed frightening, to conjecture what the possible result of such modifications might have been: The walkways could have collapsed under a much greater but still reasonably probable live load, with a yet more devastating effect.

ACKNOWLEDGMENT

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APPENDIX.—REFERENCES

1. "Hotel Disaster Triggers Probes," *Engineering News Record*, Vol. 207, No. 4, July 23, 1981.
2. "Hyatt Walkway Design Switched," *Engineering News Record*, Vol. 207, No. 5, July 30, 1981.
3. "Investigation of the Kansas City Hyatt Regency Walkways Collapse," NBSIR82-2465, U.S. Department of Commerce, National Bureau of Standards, Washington, D.C., Feb., 1982.
4. *Manual of Steel Construction*, 8th ed., American Institute of Steel Construction, New York, N.Y., 1980.
5. *Metric Structural Steel Design Data*, Canadian Institute of Steel Construction, Willowdale, Ontario, 1979.
6. *Uniform Building Code*, 1976 edition, International Conference of Building Officials, Whittier, Calif., 1976.