Failure of Stub-Girders due to Longitudinal Shear

Rupture de poutres «stub» due à un cisaillement longitudinal

Bruch von «Stub»-Trägern infolge Längsschub

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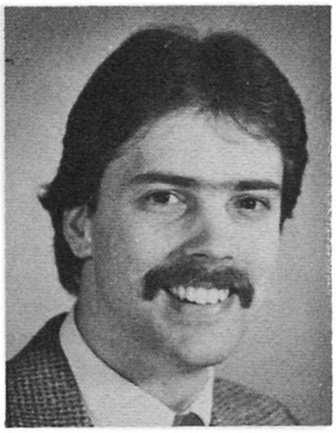

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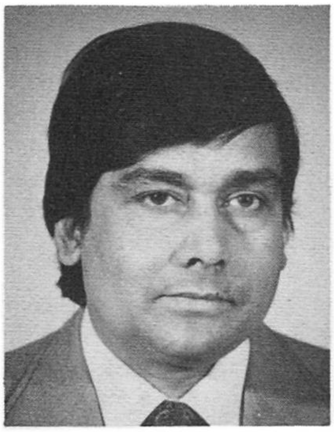

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SUMMARY

The paper summarizes the results of experiments performed on two full size stub-girders. In both beams, failure was caused by the longitudinal splitting of the concrete slab over the end stubs. A significant reduction in the shear strength of the stub-girders resulted from the prying and ineffective transverse reinforcement. The research reported here forms the first phase of a conntinuing investigation at the University of Saskatchewan.

RÉSUMÉ

L'article présente les résultats d'expériences réalisées sur deux poutres «stub», en grandeur réelle. Dans les deux poutres, la rupture a été causée par l'éclatement longitudinal de la dalle en béton sur les «stubs» d'extrémité. Une réduction importante de la résistance au cisaillement des poutres «stub», résulte d'un effet de levier et d'une armature transversale inefficace. Cet article est le résultat d'une première phase de recherches à l'Université du Saskatchewan.

ZUSAMMENFASSUNG

Der Artikel befasst sich mit Resultaten von Versuchen, welche an zwei «Stub»-Trägern in Originalgrösse durchgeführt wurden. Bei den Trägern wurde der Bruch durch ein Längsspalten der Betondecke über den äusseren «Stubs» verursacht. Eine beträchtliche Verminderung des Schubwiderstandes der «Stub»-Träger erfolgt aus einer Hebelwirkung und einer unwirksamen Querbewehrung. Diese Forschung ist das Ergebnis einer ersten Phase von Arbeiten, welche an der Universität von Saskatchewan ausgeführt wurden.

1. INTRODUCTION

The stub-girder floor system was first used in 1971 on the One Allen Centre [1], a 34-storey office building in Houston, Texas. Prior to the construction of this building and later the First International Building in Dallas, full size specimens were load-tested by the consulting engineers and results briefly reported [1,2]. In both cases, the test beams were of considerable length (approximately 12 metres) and the load tests reflected flexural behaviour. In 1980, an 8 metre stub-girder was tested by Buckner et al. [3] at the Louisiana State University in Baton Rouge, Louisiana. The failure mode of this girder was by longitudinal shear at the interface of the concrete slab and one of the end stubs as shown in Fig. 1. Based on test results, the investigators proposed a tentative design criterion to ensure adequate longitudinal shear strength of the slab and recommended that at least 50 percent of the required transverse reinforcement be placed in the bottom half of the slab.

The stub-girder tested by Buckner et al. [3] incorporated a solid concrete slab which permitted placement of the transverse reinforcement near the bottom surface of the slab. This configuration, however, is not typical of any stub-girder system used to date. A stub-girder system invariably utilizes a ribbed metal deck which prevents the placement of transverse reinforcement in the bottom half of the slab. In a typical floor slab consisting of approximately 75 mm of concrete on top of a 76 mm metal deck, the transverse reinforcement ends up at best in the upper half of the slab after allowing for the minimum clear cover requirements. Moreover, since the ribs are normally parallel to the girders, the effective shear transfer area of the concrete is considerably reduced. The design criterion proposed by Buckner et al. is therefore not applicable to a stub-girder system which utilizes composite metal decks.

This paper briefly summarizes the results of experiments performed on two full size stub-girders with slabs on ribbed metal decks. The main objective is to study the shear failure mechanism and the ultimate shear strength of stub-girders in relation to specific variables such as the length of the stub, the distance between stubs and other slab-related variables. As the first phase of a continuing investigation, this study considers the special case of stub-concrete connections with headed studs arranged in a single line as shown in Fig. 2.

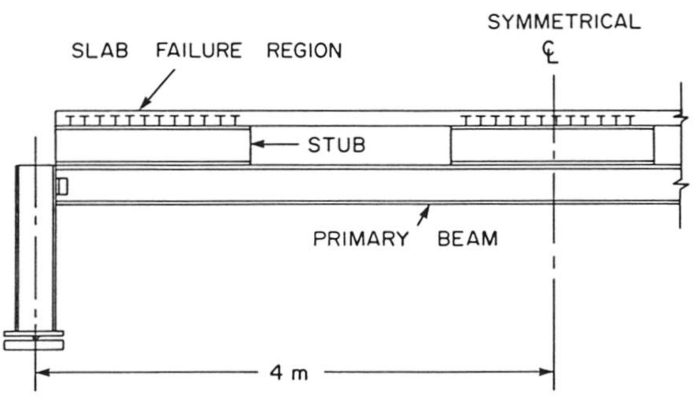

Fig.1 Longitudinal shear failure in test girder(Ref.3)

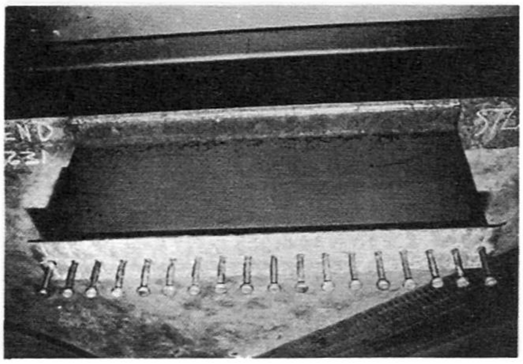

Fig.2 Headed stud arrangement used for phase one

2. EXPERIMENTAL INVESTIGATION

2.1 Test Program

A summary of the geometric properties of the two specimens is given in Fig. 3. The dimensions of the steel beams, the stubs and the concrete slab, which were identical for the two specimens, conformed to those for a stub-girder of 10 to 12 metre span. However, the span of the test beams was intentionally reduced to 7.31 m to induce failure through the stub-concrete connections. While the total length of the stubs was maintained at 3658 mm for both beams, the number of stubs was: three (1219 mm long with 24 studs) for specimen UO-1 and four (914 mm long with 18 studs) for specimen UO-2. Thus specimen UO-1 contained only two interior panels (1524 mm long) whereas three interior panels (914 mm long) were located within specimen UO-2. An 1829 mm long W360 X 33 section was placed at the midlength of each interior panel to simulate a floor beam. It was welded to the primary steel beam and connected to the concrete slab by means of eighteen studs. All headed shear studs were 76 mm long and 13 mm in diameter. For both specimens, the longitudinal reinforcement consisted of eight 13 mm diameter deformed bars whereas the same size bar at 305 mm centre to centre served as the transverse reinforcement. A clear cover of 25 mm was provided between the longitudinal reinforcement and the metal deck.

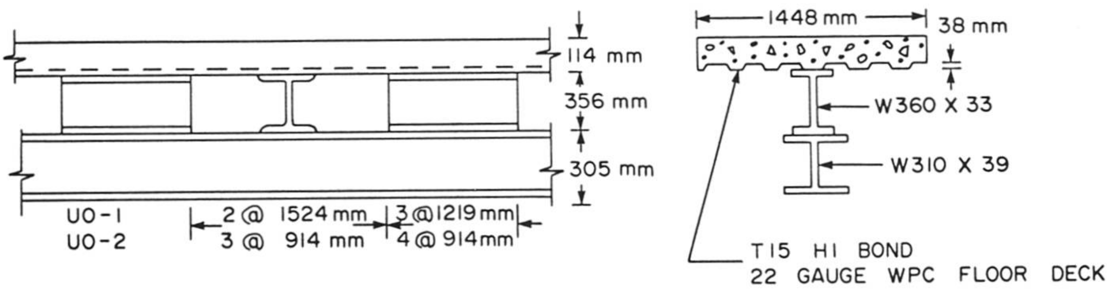

Fig.3 Specimen dimensions

2.2 Test Set-Up and Instrumentations

The test set-up used is shown in Fig. 4. Each stub-girder was mounted on load cells which were placed on concrete pedestals at each end of the stub-girder. The concrete slab was laterally braced at the supports and also at mid-span. The bracing system at midspan was based on Watt's mechanism [4] to permit free vertical movement. Vertical loads were applied through the floor beams in order to simulate the loading condition expected in an actual stub-girder system. This resulted in a 2-point loading arrangement for specimen UO-1 and a 3-point loading arrangement for specimen UO-2. The loads were applied by Enerpac hydraulic rams which were located below the test floor. Applied loads on each floor beam was monitored by four 25.4 mm diameter instrumented and precalibrated steel bars. Strains in the steel and in the concrete as well as deflections were recorded.

2.3 Material Properties

Material properties are included in the appendix. Table 1 summarizes the steel properties of the primary beam and of the stubs. The properties were obtained from laboratory tension tests on flat tensile specimens conforming to ASTM Specification A370-77. The properties of concrete and reinforcing steel are listed in Table 2.

3. RESULTS AND DISCUSSIONS

The load vs midspan deflection curves for specimens UO-1 and UO-2 are presented in Fig. 5. For both specimens, the deviation from the initial elastic behavior was due to longitudinal cracking on the underside of the concrete slab on both sides of the end stubs. With increasing load these cracks widened and, eventually, a sudden drop in the load was observed. It is believed that this rapid unloading was caused by sufficient widening of the cracks to allow bending of the headed studs.

After the initial rapid drop, the load-deflection curve exhibited ductile behavior with relatively small reduction in load. This behavior is attributed to the transverse reinforcement becoming effective in restraining further splitting of the concrete. Simultaneous resistance to splitting from the concrete and transverse reinforcement steel was not achieved because the transverse reinforcement was located too high in the slab (near the head of the studs). As indicated earlier, this placement of the transverse steel was dictated by the floor deck and the clear cover requirement for the longitudinal reinforcement. This practical problem needs further study especially if the use of wire mesh is envisaged. Fig. 6 shows the slab failure region over the end stub.

Eccentricity of the compressive force in the concrete slab induced prying forces [5] in the studs on the inner ends of the end stubs and produced compression between the concrete and the flange of the stub at the far ends.

The observed shear stress distribution across one of the end stubs is shown in Fig. 7. The deviation from the basic parabolic shape which occurs at higher loads is believed to be caused by local concrete failure at the tensile (prying) end of the stub. Localized failure could reduce the shear resistance of studs subjected to tension, giving rise to the skewed distribution in Fig. 7. The design equation for headed studs of a conventional composite beam therefore, would not seem to be applicable to the

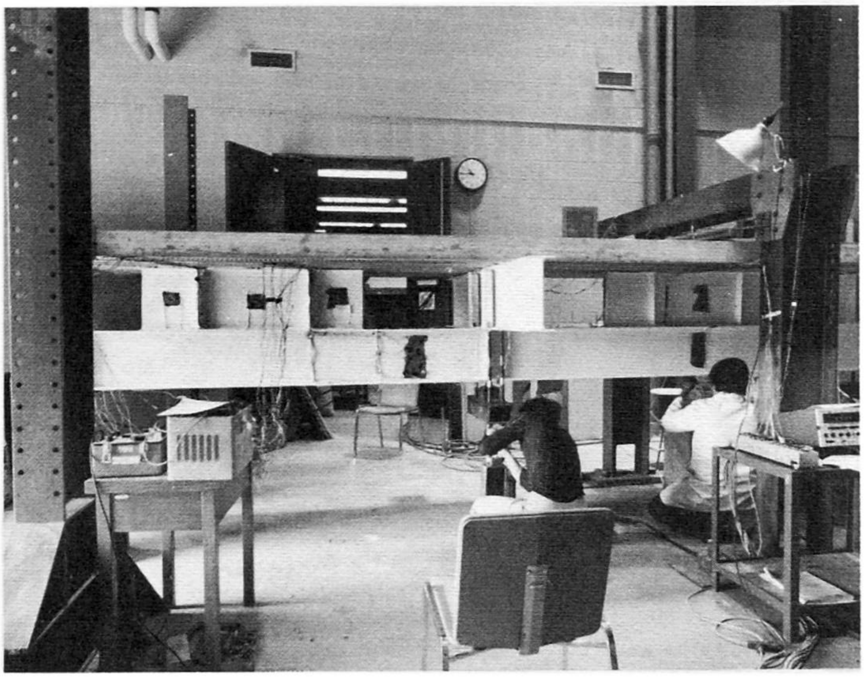

Fig.4 Test set-up

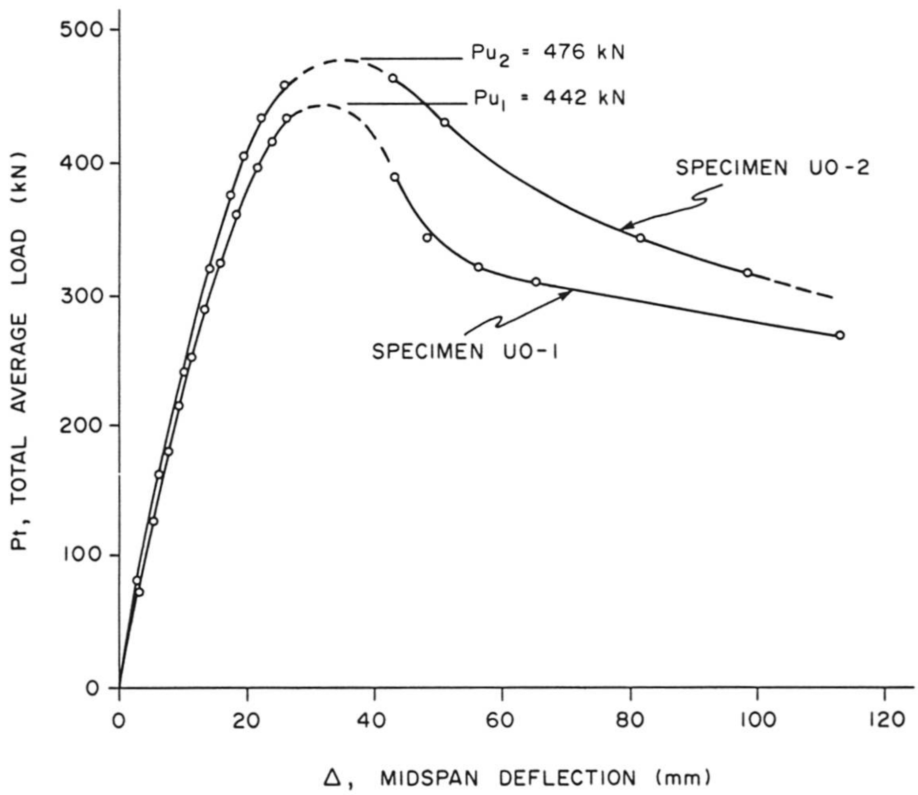

Fig.5 Load-deflection curves

case of stub-girders since the shear connectors in the conventional composite beam are subjected to less prying, if any.

The transverse floor beams, which are also connected to the concrete slab by the use of headed studs, influence the shear failure mechanism. In both specimens, ultimate failure was preceded by an early pull-out type failure of the connections between the concrete slab and the floor beam. This resulted in a redistribution of shear forces in the open panel with the steel girder carrying the entire load. The ensuing distortions further increased the prying effects mentioned earlier. At lower loads, when the connection between the floor beam and the slab was intact, approximately 55 percent of the total shear was carried by the concrete slab but, as the test progressed, this ratio was reduced to 40 percent due to the deterioration of the connection between the concrete slab and the floor beam and the resulting redistribution of the applied load to the primary girder.

Using the shear stress distribution across the end stubs at the ultimate load (similar to Fig. 7), the maximum horizontal shear force was calculated for both specimens. This gave an average shear resistance per stud of 26.7 kN for specimen UO-1 and 33.68 kN for specimen UO-2. The maximum capacity that can be developed in shear, based on the CSA standard CAN3-S16.1-M78 [6] for conventional composite design, is 64.2 kN for the high strength concrete used. The difference between the observed and calculated shear resistances is due to several reasons. First, the use of a single row of studs resulted in a wedge action that triggered a somewhat premature concrete splitting along the end stubs. Second, both specimens were subjected to considerable prying. Due to higher prying force induced by the larger open panel in specimen UO-1, the resistance per stud was smaller compared to that of

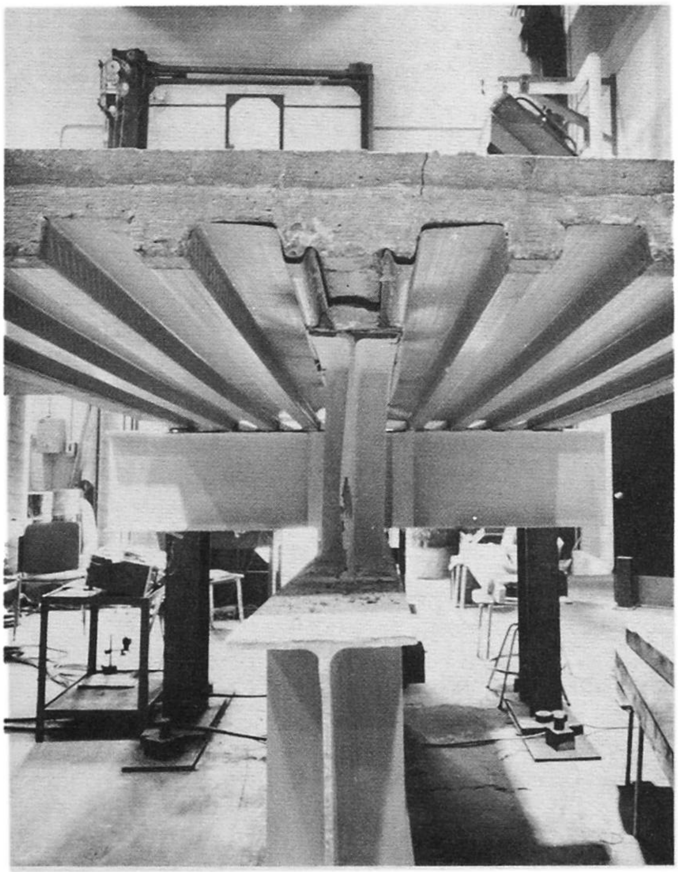

Fig.6 Slab failure region over the end stub

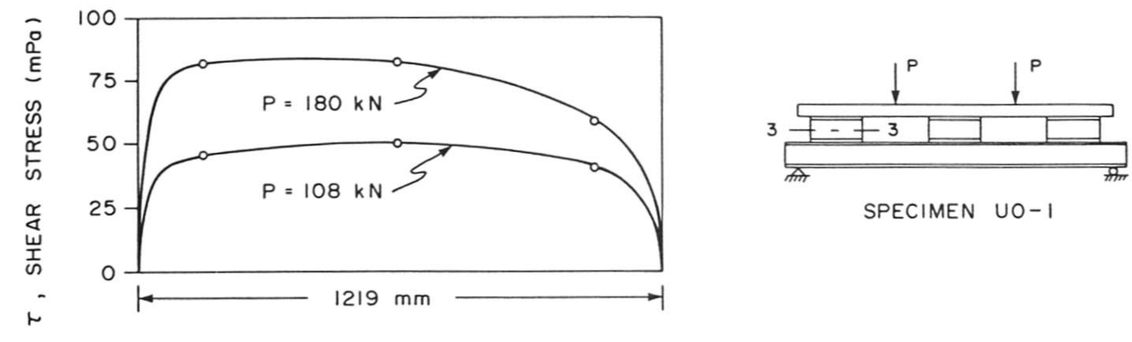

Fig.7 Shear stress distribution across section 3-3

specimen UO-2. Finally, the inadequate amount and ineffective placement of the transverse reinforcement also contributed to the premature failure.

4. CONCLUDING REMARKS

Investigation of the shear failure mechanism in full-scale stub-girders was limited to the testing of only two specimens. Yet, several important issues were resolved.

In comparison to conventional composite construction, the stub-girder composite floor system produced more severe loading on the stud connectors which transfer shear between the concrete slab and steel members. Prying of the slab from the steel stubs produced tension in some of the shear connectors, which led to local concrete failures and a reduction in the shear capacity of the connectors. In the tests conducted the prying was induced by two effects: eccentricity of the compressive force in the slab chord of the open panel, and direct prying due to transverse beam loads being applied entirely to the primary steel girder after connection of the transverse beam to the stub was lost at early loading. In some arrangements, severe prying can also be induced by the vertical shear force acting in the concrete slab along the open panel.

For the shear studs and concrete used in the tests, the ideal strength (q/ϕ) based on the CSA standard CAN3-S16.1-M78 [6] is 64.2 kN per connector. The observed average connector strength was 30.2 kN (47% of the ideal). Prying effects were responsible for part of the reductions. However, a more important consideration may be that placement of lateral reinforcement in both tests was ineffective in resisting concrete splitting and shearing. Though the transverse reinforcement area supplied was sufficient to satisfy the CSA requirement (transverse reinforcement ratio = 0.005), practical problems prevented placement in the bottom of the slab as required by the CSA. Although the CSA requirements do not depend on stud configuration, studs arranged in a single line produce a wedge action which would appear to make the concrete slab more susceptible to longitudinal splitting.

More full-scale tests should be conducted to investigate the effectiveness of different transverse reinforcement schemes such as varying the amount of reinforcment, widening the flute over the stubs or using irregular-shaped reinforcing bars.

ACKNOWLEDGEMENTS

This investigation was carried out with financial support from the National Research Council of Canada. The authors are indebted to Mr. Bill Watson and Mr. Claude Lavigne of the University of Ottawa for their help in conducting the tests. The assistance of Heather Larson, who typed the manuscript is also gratefully acknowledged.

REFERENCES

1. CALACO, J.P., "A Stub-Girder System for High-Rise Buildings," AISC Engineering Journal, July, 1972, pp. 89-95.
2. GUNNIN, BILL L., "The First International Building in Dallas, Texas," Acier Stahl Steel No. 3, 1976.
3. BUCKNER, C. DALE, DEVILLE D.J., MCKEE, D.C., "Shear Strength of Slabs in Stub Girders", Journal of Structural Division, ASCE, Vol. 107, No. ST2, Feb., 1981, pp. 273-280.
4. YARIMICI, E. YURA, J.A. and LU, L.W., Techniques for Testing Structures Permitted to Sway, Lehigh University, Fritz Laboratory Report No. 273.40, May, 1966.
5. LAM, Y.W., REZANSOFF, T. and HOSAIN, M.U., An Experimental Investigation of Stub-Girders, Proceedings of the International Symposium on Behaviour of Building Systems and Building Components, Vanderbilt University, Nashville, Tennessee, March 8-9, 1979, pp. 365-384.
6. Steel Structures for Buildings - Limit States Design, Canadian Standards Association Standard No. CAN3-S16.1-M78, Rexdale, Ontario, Dec., 1978.

APPENDIX

Table 1. Steel Properties

Section	Average F_y (MPa) Flange	Average F_y (MPa) Web	Average F_u (MPa) Flange	Average F_u (MPa) Web	Average Elongation %
Girder	332	346	481	476	26.4
Stub	334	374	482	487	25.6

Table 2. Properties of Concrete and Reinforcing Steel

Specimen	Average f'_c (MPa)	$*E_c$ (MPa)	f_y (MPa)
UO-1	34.0	27600	708.75
UO-2	31.5	26500	

$*E_c = 150\ f'_c$