Some Aspects of Stub-Girder Design

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Much research effort has been devoted to studies of the strength and behavior of the many types of structural members and connections that are commonly used in steel-framed buildings. The investigations have primarily focused on how to make the best use of the available materials, assuming that the end product would be safe, serviceable, and—as much as possible—economical structures. To that end significant advances have been made throughout the years, providing for an ever-improving quality of the work of the structural engineer. From his viewpoint, therefore, the attention of the researchers has been properly directed, because their studies have enabled him to perform his tasks better.

It is readily recognized, however, that the structural aspects of most building projects form but part of the total construction package. The completed structure must provide adequate service to its users in all respects, covering structural performance as well as all of the electrical, mechanical and environmental requirements that must be satisfied. This is probably best achieved by a systems analysis of the project design, which requires extensive cooperation between all of the parties that are involved in the work. Thus, not only must the architect, the structural engineer and the general contractor work together as early as possible, but it is increasingly important that the other groups in the design team participate fully in all of the decision-making processes. For a building this primarily refers to the mechanical and the electrical engineers, because many of the services that are planned by these professionals may influence the way the structure is designed and constructed.

The vertical transmission of utilities is commonly centered within service cores of the building and normally the structural difficulties that arise in this connection are only a result of the framing system. The primary problems that result from the integration of services within the structural frame therefore are tied to the floor systems. Although many structural configurations are acceptable, the most economical solution will be the one where electrical and mechanical installations can be done with a minimum of disruption. For a steel-framed building this implies not only that all construction trades can schedule their work efficiently, but also that the load-carrying members are located and formed such that duct work and other units are easily installed.

The development of the stub-girder system came about as a direct result of the preceding problems. In particular, J. P. Colaco, the originator of the system, noted that with conventional floor-framing schemes, the HVAC ducts, lights and ceiling either would be suspended below the steel beams, or in some cases web penetrations would be made to allow for passage of the ducts.[1] The latter is naturally a relatively costly structural detail. The layout of the stub-girder floor would eliminate these problems, as well as reduce the total steel weight because of higher bending stiffness of the composite girder. Construction time also was decreased, and the final result for the project therefore was that it could be completed at lower cost than a conventional system.

In the years following the first usage of the stub-girder system a number of buildings followed suit, and all of them experienced the advantages as well as the difficulties associated with the framing scheme. Many articles appeared in the trade press, but relatively few papers were written about the analysis and design methods. Some studies focused on the use of the stub-girders in certain buildings,[2,3] and some full-scale, but mostly proprietary experiments were run with girders that had been designed for particular structures.[4,5] Although all of these investigations appeared to substantiate the initial work of Colaco, questions were being raised as regards potential changes in the design procedure and alterations in the girder details and overall layout to effect further economies. More recent studies have

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This paper was presented at the 1980 Canadian Structural Engineering Conference, held at Montreal, Quebec, on February 25 and 26, 1980.

extended the use of the stub-girder to make it a part of the lateral load resisting frame for a building,[6] and theoretical analyses have provided some insight into the use of refined analysis techniques for the girders.[7]

Many questions remain unanswered, however, and it was the intent of the research work that will be described in this paper to attempt to resolve some of them. In doing so, particular attention has been devoted to providing practice-oriented solutions that will be of direct use to the designers as well as the structural steel fabricators. The advantages of the original framing scheme have been retained, and the changes that are recommended will lead to lower costs. It must be emphasized, however, that the project does not pretend to have answered all questions that have been posed, and other studies might be undertaken to refine the system further.

SCOPE OF STUDY

By reason of the questions and concerns that had been voiced, it was felt that the following items would benefit from further study:

1. Method of stub-girder modeling for analysis (Vierendeel representation).
2. Design and method of stub stiffening.
3. Design of welded connection between stub and bottom chord.
4. Design of stub shear connectors (stub to slab).
5. Analysis of slab behavior and strength, including concrete stresses throughout the stub-girder span.
6. Localized stress and strain effects, including "hard" points at end of stubs.
7. Cracking behavior of slab, particularly in the longitudinal direction of the stub-girder.
8. Design of concrete slab reinforcement (transverse and longitudinal).

It is pointed out that the above list represents a significant portion of the problem areas, but that certain questions were left out on purpose. For example, the dynamic characteristics of the stub-girder floor system are typically determined on the basis of the method that is given in Appendix G of the Canadian standard for limit states design of steel structures.[8] It is not clear whether that approach should even be applied to the stub-girder, since it was developed on the basis of simply supported, solid slab and wide-flange shape composite beams. This problem should be studied in detail, but is of such a magnitude as to warrant a separate investigation. Similarly, the method of shear stud design has been questioned, particularly in view of the possibility that the failure mode of closely spaced studs cannot be deduced from that of a single stud. Finally, the Vierendeel analysis and subsequent composite design use criteria that were developed for more "typical" composite beams. The state-of-the-art of composite truss analysis has not been sufficiently advanced to allow a rational application to Vierendeel-type members, but would seem to be the appropriate solution. Other concerns might also be addressed; for example, the behavior of the floor system under earthquake conditions. It is known that a building using the stub-girder system (in Mexico) survived an earthquake of Richter magnitude 6.0, exhibiting excellent ductility and strength.

In developing the plans for the research project, it was determined that the problems would be studied both experimentally and analytically. To that end the investigation was divided into the following four portions, to be completed in the order given:

1. Analysis and design of stub specimens and full-size girder.
2. Testing of stub specimens.
3. Fabrication and testing of the stub-girder.
4. Analysis of test data and correlation with the design.

In order to be as practice-oriented as possible, it was decided that all analysis and design work should be done in accordance with the methods that are currently used by structural engineers. This was particularly aimed at the need to obtain representative steel and concrete dimensions. Some finite element analyses of the specimens also were planned for, in order to tie strain-gage readings in the test specimens into detailed theoretical studies.

Among the primary difficulties with the stub-girder system are the method and type of stiffening of the stubs, as well as the design of the stub-to-chord welds. On that basis it was decided that full-size slab and *single* stub assemblies, utilizing the appropriate materials, sizes, shear connectors and welds, would be tested first. Loading conditions were made to simulate those in the real girder. Each of these assemblies would have different stiffening details, to examine their influence on the behavior and strength of the stub. The detailed plans for this portion of the study are given in a later section of this paper.

Based on the performance of the slab-stub assemblies, the one with the most desirable characteristics in terms of strength and behavior would be chosen for use with the stubs in the full-size girder. That is why the fabrication of the girder was delayed until all of the stub specimens had been tested. It will be shown that this was a very fortuitous decision, since it resulted in a stub-girder with stiffener and welding details that were significantly different and less expensive than those that have been used in actual girders in buildings. Similar knowledge was put to use for the slab reinforcement. All of these data are outlined in detail later in this paper.

As a final task, the test and theoretical data would be analyzed and compared. This is particularly important in light of the questions that had been raised about the Vierendeel analysis, and for the significant changes that were made for the stiffeners and the other details.

DESIGN OF THE STUB-GIRDER

Following extensive discussions with the staff of the Project Analysis Division of the Canadian Steel Industries Construction Council, as well as detailed surveys of the building projects that had utilized the stub-girder system, the following materials and members were chosen for the test specimen (refer to Figs. 1 and 2 for detailed drawings of the stub-girder):*

**Ed. Note: All units in the figures and text of this paper are in S.I. units; shape designations are Canadian standards. For conversion to U.S. units, refer to the 8th Edition AISC Manual, pgs. 6-14 and 6-15.*

Main girder (bottom chord of stub-girder): W310x86 (old des. W12x58)
Stubs and floor beams: W410x39 (old des. W16x26)
Steel deck: 76 mm Westeel-Rosco T-30V (1.22 mm deck thickness)
Concrete slab: 27.5 MPa semi-lightweight concrete (1760 kg/m^3) with 84 mm thickness over the deck ribs.

All steel in the main girder, floor beams, stubs and miscellaneous details was specified as CSA G40.21-M, Grade 300W. The concrete slab reinforcement that was used is shown in Fig. 2, and Fig. 3 shows a close-up photograph of the test specimen before the slab was cast. A

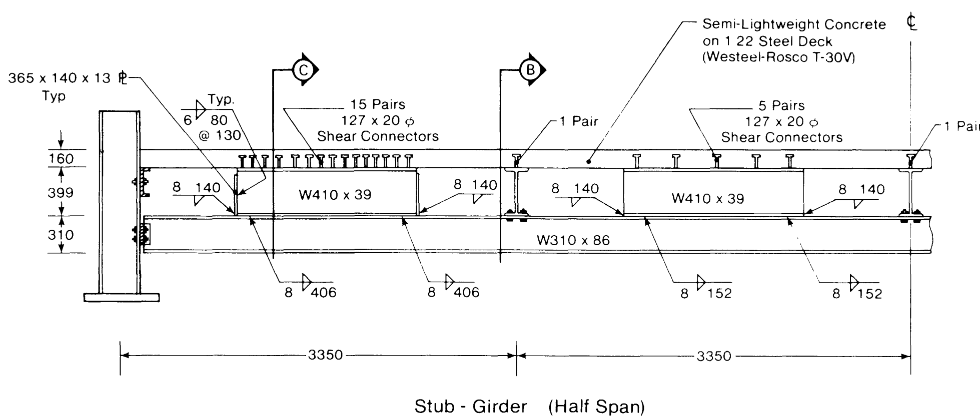


Fig. 1. Elevation of full-size stub-girder

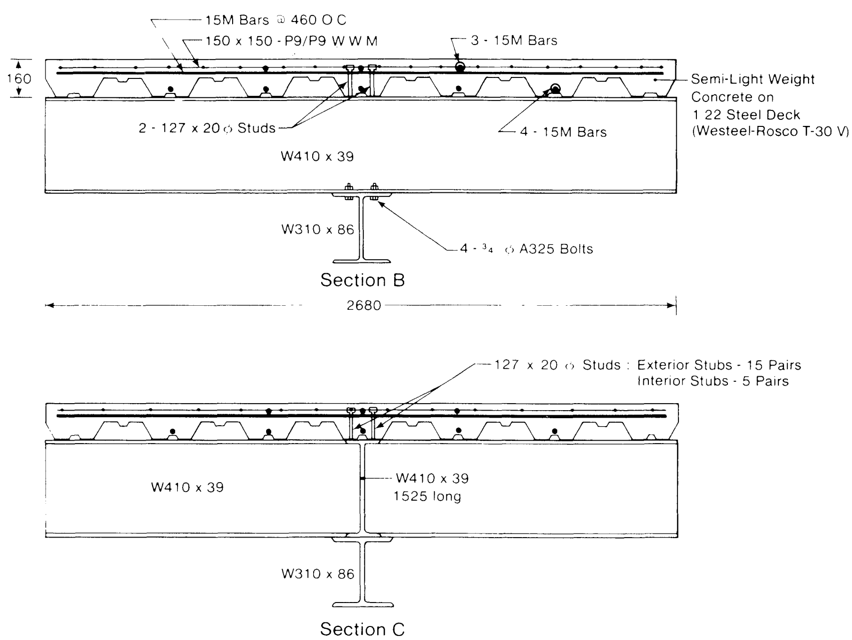


Fig. 2. Sections B and C of the full-size stub-girder

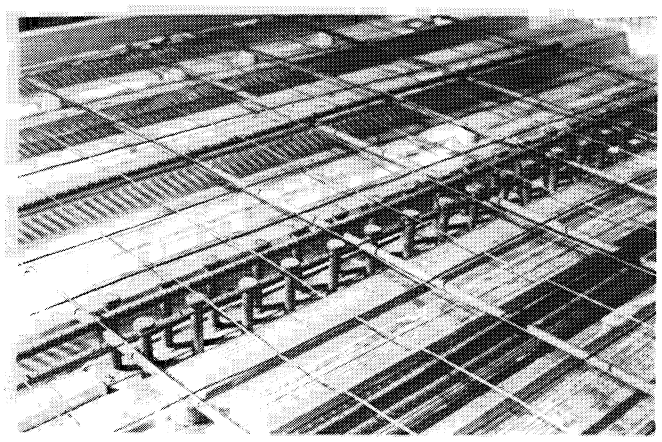


Fig. 3. Close-up view of stub-girder slab reinforcement (shown in area around exterior stub)

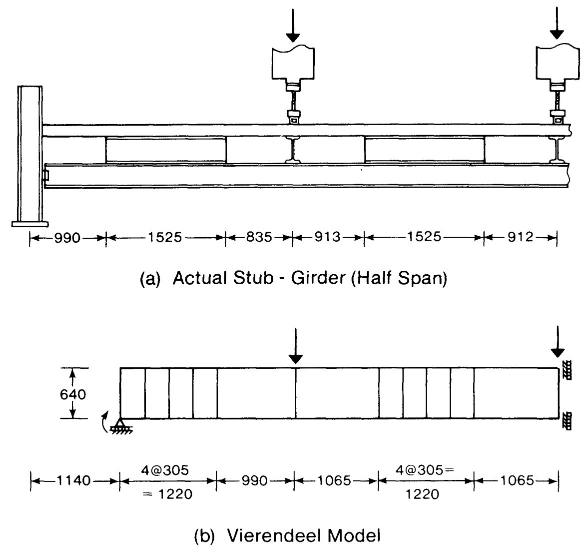


Fig. 4. Stub-girder and its Vierendeel analysis model

preliminary analysis demonstrated that the choices would provide a serviceable girder, without undue local problem areas that might lead to a premature failure. As will be seen later in the paper, the five stub specimens that were included in the project utilized materials and sizes identical to those chosen for the full-size girder, with the exception of the stiffener details, and the transverse reinforcement in the slab.

Analysis of the Stub-Girder—With the preliminary data chosen for the girder, a detailed analysis was carried out using the Vierendeel truss approach that was proposed by Colaco in his original designs,[1,2] and subsequently somewhat modified by Chien[9] to make it compatible with Canadian limit states design philosophy. Figure 4 illustrates the transformation from the real girder to the model. Because of symmetry, only one half of the girder was necessary to analyze.

Each stub was modeled as five vertical members, all rigidly attached to the top and bottom chords. Their moments of inertia were set equal to one-fifth of that of the stub itself. A vertical member also was placed at the location of each floor beam. The stiffness of the bottom chord was equal to that of the W310x86, and the concrete slab was transformed into an equivalent steel member using an effective width of 2680 mm (see Fig. 2) and a modular ratio of 12.

Boundary conditions are also indicated in Fig. 4. A roller support was used at the bottom left end, where a moment was introduced to account for the fact that the support was not located at the theoretical left end (column center line). At the right end (center line of actual girder) supports were prescribed that allowed only vertical displacements. It is noted that the portion of the slab between the left-most vertical member and the end of girder (i.e., between the outside end of an exterior stub and the column) was not considered an effective part of the truss, and therefore was not included with the model.

Concentrated loads were applied at the floor beam locations, as shown in Fig. 4, and the resulting axial force, shear force and bending moment distributions were determined, using a plane frame computer program. With an arbitrary value of each of the loads equal to P, the most critical axial force and bending moment data were as follows (note that all moments are expressed as P-meters):

1. Section at mid-span:
   Axial force = $8.81P$
   Slab bending moment = $0.27P$
   Bottom chord bending moment = $0.82P$
2. Section at exterior end of interior stub:
   Axial force = $6.87P$
   Slab bending moment = $0.28P$
   Chord bending moment = $0.83P$
3. Section at interior end of exterior stub:
   Axial force = $6.87P$
   Slab bending moment = $-0.15P$
   Chord bending moment = $-0.45P$

   It must be observed that the slab bending moment at this location introduces tensile stress in the top of the slab.
4. Section at exterior end of exterior stub:
   Chord bending moment = $1.49P$

   Axial force and slab bending moment are both equal to zero at this location.

It should be noted that the slab axial force always was compressive, and that in the bottom chord was tensile. Shear forces appeared to be less significant for the chord members, with the possible exception for shear in the slab at Section 3. However, subsequent shear stress analyses showed that this condition would not govern the strength and behavior of the girder.

The stub specimens that were to be tested were analyzed and designed on the basis of the stress resultants obtained for the exterior stub. As shown in previous studies,[1,4,5] this stub appeared to be one of the critical elements within the stub-girder. This also turned out to be the case in this investigation.

Some limited finite element analyses of actual stub-girders were performed in the past,[1,3] and theoretical research on this subject also was done.[7] Such evaluations were conducted in this study as well, but the results are of limited usefulness, because only two-dimensional, elastic data were obtained. However, the results did confirm the locations of the areas of force concentrations, and as such amplified the other computations. This subject is open for further research, and might prove to be of additional benefit to structural designers.

Design of Stub-Girder and Stub Specimens—With the materials and member sizes chosen, the data from the structural analysis were used to determine the design capacities of the stub-girder. The three potentially governing sections of the girder were identified as one of the following:

1. Section at mid-span.
2. Section at exterior end of interior stub.
3. Section at exterior end of exterior stub.

Proceeding on the assumption that no local failures would prevent the stub-girder from reaching its ultimate capacity, the ultimate values of each of the concentrated loads for the above three locations were found as follows:

1. Girder failing in combined bending and tension at midspan (tension predominant):

$$P = 223 \text{ kN}$$
$$\text{Total load:} \quad 3P = 669 \text{ kN}$$

Note that these values include the dead load of the girder itself, estimated at 71 kN.

2. Girder failing in combined bending and tension at the exterior end of the interior stub (both stress resultants of about equal importance):

$$P = 232 \text{ kN}$$
$$\text{Total load:} \quad 3P = 696 \text{ kN}$$

3. Girder failing in pure bending at the exterior end of the exterior stub:

$$P = 260 \text{ kN}$$
$$\text{Total load:} \quad 3P = 780 \text{ kN}$$

Sections 1 and 2 indicate loads that are almost the same, but in the strict sense, the computations showed that the governing failure mode would be one associated with a predominantly tensile failure in the bottom chord at mid-span. Of course, in an actual case it may turn out that local conditions induce changes in the behavior of the girder. The exact location of the governing section of the stub-girder therefore may be somewhat different.

The numbers of stud shear connectors for the exterior and the interior stubs were determined on the basis of the Canadian limit states design requirements,[8] but introducing some modifications to account for the effect of closely spaced studs.[9] The latter notes that the combined strength of two very closely situated shear studs should not be counted on as twice that of a single stud. Although research has not provided detailed substantiation of this phenomenon, the approach appears rational and realistic. Free body diagrams of the stubs showed the stress resultants that were necessary for equilibrium, based on the governing applied load of 223 kN applied at the three load-points. These internal forces governed the number of shear studs (30 pairs of 20 mm x 127 mm length for the exterior stubs; 5 pairs of the same type of studs for the interior stubs), as well as the amount of welding between the stubs and the bottom chord (see Fig. 1).

Present practice dictates a complete weld around the stub. However, the design forces did not require such extensive joints, and Fig. 1 therefore shows that only a fraction of the "normal" amount of welds was used for the stub-girder test specimen. The subsequent testing proved that this decision was sound.

The stub stiffeners were designed according to current practice, taking into account tensile as well as compressive forces. Due to the expense of using the fitted stiffeners that normally are seen in stub-girders, it was decided to test and analyze other configurations. This also covered the possibility that no stiffeners were needed in certain cases. Suffice it to observe at this point that the five stub specimens that were designed and fabricated had all details identical to those of the exterior stubs of the full-size test girder, with the exception of the stiffeners. The former was fabricated with the type of stiffeners that performed the best during the tests of the stub specimens, but also considering the amount of stiffening that was actually required. As a result, Fig. 1 shows that the interior stubs had no stiffeners at all.

The substantial differences between the stub-girder and the conventional composite floor systems warrant additional stress computations for the elements of the girder. Thus, the critical sections of the slab of the test specimen were analyzed, and it was found that the maximum shear, compressive and tensile stresses were all within the maximum values prescribed by the standard for structural concrete design.[10]

In the preliminary design phase, other stub-girder shapes also were investigated. This included utilizing other W-shapes,[11] as well as an HSS- (hollow structural section) shape for the bottom chord of the girder. The latter gave some interesting results, notably a 5% lowering of the steel weight, but a significant increase in the concrete slab stresses. The latter problem, along with apparent higher fabrication costs, caused the HSS-section not to be considered for the test girder.

Two independent analyses of the stub-girder test specimen gave ultimate loads (at each load point) identical to and 6.5 kN higher than that predicted by this investigation. Separate studies thus confirmed the design findings of the research team.

EXPERIMENTAL PROGRAM

Overall Testing Plan—In order to obtain as comprehensive data as possible, while considering the high cost of experimental research, the final testing plan called for one full-size stub-girder and five slab-stub assemblies. Although it would have been desirable to test more girders, economics dictated that some other avenue be sought, and this is why the stub specimens were conceived. Past experiments had shown that the behavior and strength of the exterior stubs were crucial for the full girder, and the intention of the specimen tests therefore was to simulate the actual in-girder conditions in the most important respects.

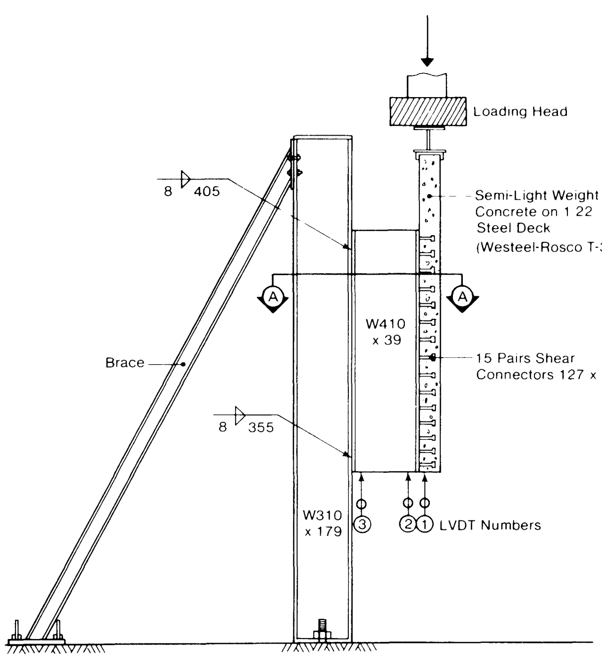
Originally, only two stub specimens had been planned, but subsequent developments demonstrated that it would be beneficial to perform additional tests. In particular, this allowed a detailed examination of many different stub stiffening schemes. Detailed data for these specimens and for the full-size girder are given in the following.

The materials and shapes that were chosen for the test specimens have been outlined in detail previously. Further comments will only be given to emphasize or explain experimental set-ups or test results.

Stub Specimens—Figure 5 illustrates the test set-up for the stub specimens, and Fig. 6 gives a cross section of a typical sample. Observe that no stiffeners are indicated on either; these will be described in detail below. The primary purpose of these tests was to investigate the behavior and strength of exterior stubs with different stiffening layouts. In addition to using a W410x39 as the stub, therefore, the shear connection was effected by 15 pairs of 20 mm x 127 mm studs, as had been decided on for the exterior members in the full-size girder. The welding that was used also duplicated that of the exterior stub in the girder.

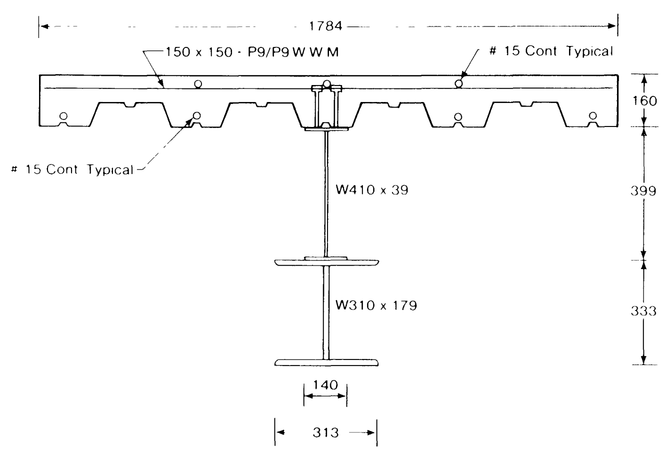
It should be noted that the 1784 mm width of the slab of the test specimen (see Fig. 6) was 896 mm less than what would be used for the stub-girder. This change was necessitated by the distance between the columns of the MTS universal testing machine which would be used in these experiments. However, it is believed that the smaller slab width did not affect the outcome of the tests, especially in view of what actually turned out to be the governing failure modes. Furthermore, the behavior of the stiffeners would not be altered.

Figure 6 indicates that the only transverse slab reinforcement was provided by the welded wire mesh, 150 x 150 – P9/P9 (same as 6 x 6 – 10/10 W.W.M.). This turned out to be very important, as the initial failure in the slab could be related directly to the amount of transverse reinforcement.



Single Stub Test Set-Up

Fig. 5. Test set-up for a stub-slab assembly (stub specimen). (No stiffening details are shown)



Section A-A

Fig. 6. Cross section of a typical stub specimen. (Stiffeners not shown)

The stub specimens were intended for study only of the corresponding areas within the stub-girder, and for that reason the W310x179 that is shown as part of the test set-up (see Fig. 5) was not meant to simulate the W310x86 bottom chord (see Fig. 1). It did ensure that failure would not take place in the chord area, but allowed the slab, shear connectors, stub, stiffeners, and stub-to-chord welds to behave as they would in the girder itself.

The original testing layout called for a slightly inclined (3.7° towards the left in Fig. 5) specimen, allowing the loading conditions to resemble very closely those that exist in a stub-girder. Thus, the load application point corresponded to the location of a floor beam in the girder. The first stub specimen failed prematurely during testing, however, and the cause turned out to be local bending in the slab, which became accentuated as the deformations increased. For this reason the tilted set-up was abandoned in all subsequent tests, but it is believed that this change did not have any significant impact.

Figures 7 through 11 show the details of the five stub specimens that were fabricated. The most important features are those given for the stiffeners, as follows:

Specimen I (see Fig. 7): Primarily intended as a control experiment, stiffeners were purposely omitted in this sample. This was done partly to investigate the behavior of an unstiffened assembly, but also to determine the relative strength and stiffness of the other types of stubs. The stub-to-girder welds that are indicated for Specimen I were used in all of the other assemblies as well. This includes the 8 mm transverse fillet weld that was placed across the end of the stub (see cross-sectional detail in Figs. 7 through 11).

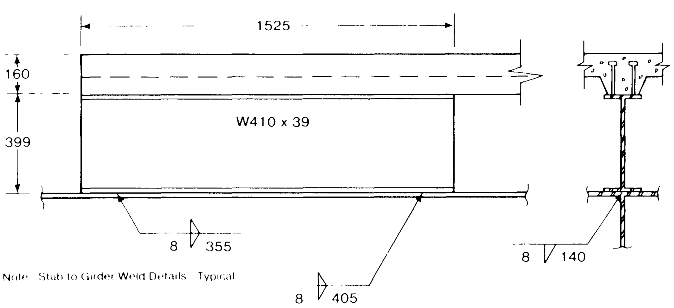


Fig. 7. Stub Specimen I: No stiffeners

Specimen II (see Fig. 8): The stiffeners used for this specimen are typical of what will be found in current stub-girders in buildings, although for the latter it is not uncommon to have a third stiffener, placed at the center of the stub. Note that the stiffener-to-web welds are intermittent (stitch) welds, which also differs from what has been used in many structures.

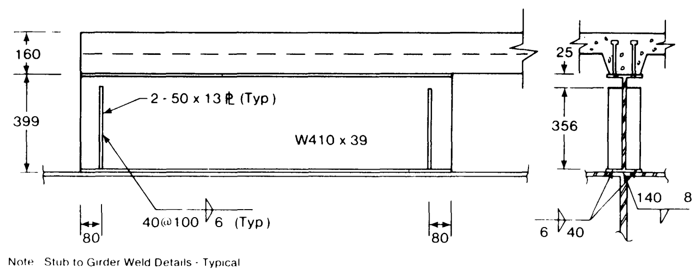


Fig. 8. Stub Specimen II: Fitted stiffeners at both ends of stub

Specimen III (see Fig. 9): Full end-plate stiffeners replace the fitted ones of Specimen II, again utilizing intermittent fillet welds. The advantages of this method of stiffening (as compared to the tradition indicated by Specimen II) are lower fabrication costs and less time, as well as a stiffer top flange of the stub. A potential disadvantage would be the development of stress concentrations at the top of the stiffener, where slab, stub and stiffener meet.

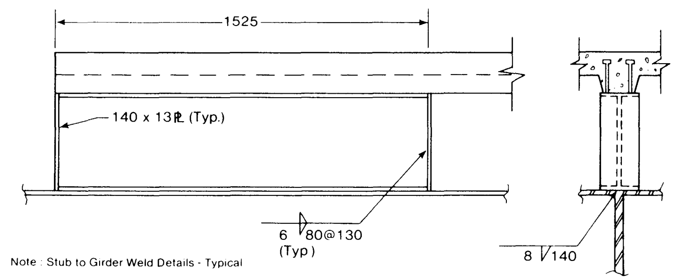


Fig. 9. Stub Specimen III: Full end-plate stiffeners at both ends of stub

Specimen IV (see Fig. 10): Although this stiffening detail does not provide the higher stiffness of the top flange (against local flange bending), it retains all of the advantages of the end-plate option. In particular, the subsequent tests showed that the stress concentrations that developed at the top of the stiffener for Specimen III would give rise to a premature failure. The partial end-plate of Specimen IV did not develop this kind of a problem, and therefore performed significantly better.

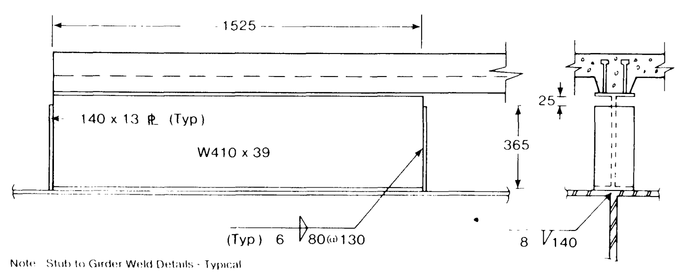


Fig. 10. Stub Specimen IV: Partial end-plate stiffeners at both ends of stub

Specimen V (see Fig. 11): Intended as a variation of Specimen II, recognition was made that stiffeners normally are needed only in compressively loaded areas of a web. The direction of the test load would induce compression in the area around the stiffener in Specimen V, and tension at the other end of the stub.

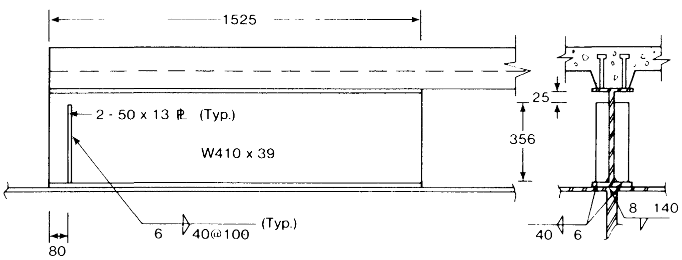


Fig. 11. Stub Specimen V: Fitted stiffener only in compressively loaded end of stub

Some of the instrumentation that was used to monitor the behavior of the stubs during the testing is indicated in Fig. 5. Displacement recorders (LVDT's) were used to measure the longitudinal deformations in the stub and in the concrete slab. Numerous strain gages were also placed on the stub in the anticipated high-strain areas (around stiffeners in web and flanges; close to welds; on top and bottom of concrete slab close to where it cantilevered out from the stub). All readings were made automatically through the use of the Nova 2/10 computer of the Structural Engineering Laboratory of the University of Alberta.

Load application would be done as indicated schematically in Fig. 5, using the 6675 kN (1.5 million lb) MTS universal testing machine in the laboratory. A distributing beam was placed between the slab and the machine crosshead to ensure loading on the full width of the slab.

Full-Size Stub-Girder—Detailed drawings of the full-size stub-girder are given in Figs. 1 and 2, and the materials and members that are shown have been discussed previously. A few additional comments should be made as regards the choice of stiffening details for the exterior and interior stubs, the transverse reinforcement in the slab, and the overall support conditions for the test girder.

Fabrication of the girder was delayed until all of the stub specimens had been tested. At that time it was obvious that partial end-plates (Specimen IV, Fig. 10) performed at least as well as the conventional fitted stiffeners, and the former therefore was specified for the exterior stubs of the girder. In similar fashion, analyzing the behavior and strength of the unstiffened stub (Specimen I, Fig. 7), and comparing it to the design forces for the interior stub, it was decided that the unstiffened alternative would be sufficient. Both of these choices reflect major departures from current construction practice.

As noted earlier, the stub specimens had no transverse slab reinforcement in addition to the welded wire mesh. This proved to be the cause of relatively early longitudinal cracking (due to Poisson's effect), and in turn led to a somewhat lower failure load. To avoid this problem for the stub-girder, ensuring that a proper girder failure would take place (rather than a slab failure), the transverse reinforcement that is shown in Fig. 2 was specified. This is clearly an excessive amount of reinforcement, but can be justified on the basis of the need to ensure the proper failure mode for the girder.

As indicated in Fig. 1, the stub-girder was tested as a simply supported member. A common double angle beam-to-column connection was used to fasten the bottom chord to a heavy test-floor column. The slab was rested on a channel, and no connection was made to the column.

A set of test frames was built around the girder, allowing hydraulic jacks of 535 kN (120 kips) capacity to be placed at the quarter-points (floor beam locations). Spreader beams were placed between the jack head and the slab, to permit a distribution of the load to a larger portion of the slab and the floor beam.

Extensive instrumentation was utilized to monitor the behavior of the stub-girder during the test. Thus, vertical deflection measurements were made at the quarter points as well as at the supports; the latter being done to obtain the reference data for the interior recordings. In addition, one of the exterior and one of the interior stubs were strain gaged, and this was also done at the quarter points in the bottom chord. Some concrete strain gages were placed on the slab in its most critical areas (at the interior end of the exterior stubs).

A final observation must be made with regard to the concrete that was used both for the stub specimens and the full-size girder. The material specifications called for a semi-lightweight (1760 kg/m^3) concrete of 27.5 MPa strength. The actual product turned out somewhat heavier, having a mass of 1960 kg/m^3. The average cylinder strength at testing was 28.2 MPa for the stub specimens, 29.4 MPa for the stub-girder. All of the concrete was supplied by a local ready-mix company. Time-constraints necessitated the use of high early strength cement for the stub-girder, which reached its value of 29.4 MPa after 10 days of curing. The stub specimens used normal cement, and obtained their strength after 28 days of curing.

With the higher unit mass of the concrete, the total factored dead load of the stub-girder was recomputed as 119 kN. This indicated that the net failure load per jack would be approximately 183 kN.

DISCUSSION OF TEST RESULTS

Tests of Stub Specimens—Figures 12 through 20 give some representative data from the stub specimen tests, including a few photographs that demonstrate important phenomena for some of the assemblies. Reference should be made to Fig. 5, which indicates the locations where LVDT-readings were taken in preparation for the load-deformation curves. This is reiterated here as:

LVDT No. 1: Concrete slab, close to bottom edge
LVDT No. 2: Flange of stub, close to the top
LVDT No. 3: Flange of stub, close to the bottom (girder)

These measurements recorded the logitudinal (i.e., parallel to the slab and the stub) deformations of the assembly.

Stub Specimen I: With no stiffeners, this test was expected to give the lowest ultimate load. This was also the first to be run, and therefore the tilted test set-up was used. As the load was applied, shear and bending strains developed in addition to the compressive ones. As a result, early transverse cracking appeared in the top of the slab directly above the end of the stub. This increased the effective angle of tilt, compounding the problems for the slab. Although the final failure came about as the "shear and compression" mode that was experienced by all of the other stub specimens as well, the ultimate load was well below what had been expected. Figure 12 shows that a load of 875 kN was reached at a total deformation of 4 mm.

The load-deformation data indicate very little difference in the displacements recorded for the slab and for the top of the stub. This turned out to be common to all of the stub specimens, and attests to good bonding and shear transfer mechanism between the two components. As expected, there was very little longitudinal movement at the bottom of the stub, and the welds exhibited no signs of distress. Some localized yielding took place in the web, as evidenced by flaking of the whitewash.

The "shear and compression" failure manifested itself as the sudden appearance of vertical cracks through the slab thickness above the flange tips of the stub and a separation between the steel deck and the concrete along the flange surface. This was accompanied by crushing of the concrete and buckling of the deck immediately adjacent to the top of the stub, at the point where the slab cantilevers beyond the end of the stub. Longitudinal cracks also developed in the slab, caused by Poisson's effect, but also as a result of the small amount of transverse reinforcement in the slab.

The slab of this stub specimen was subsequently broken apart to permit a study of the shear connectors. No visible distress could be detected, which was a result of the low overall failure load.

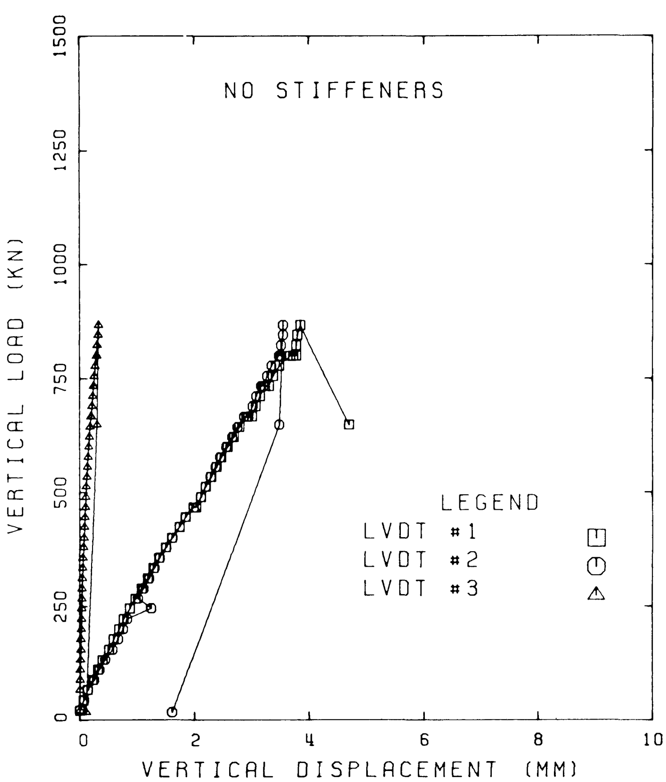


Fig. 12. Load-deformation results for stub Specimen I (no stiffeners)

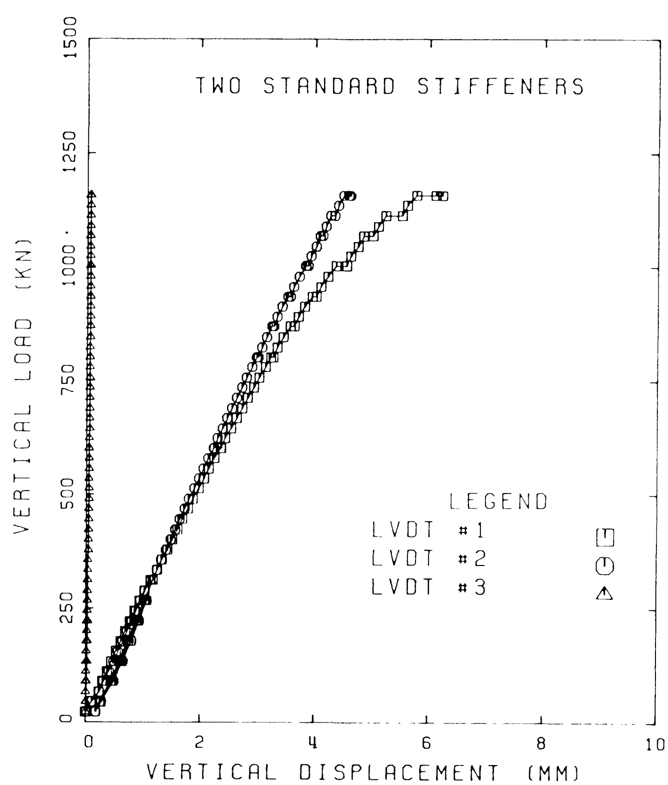


Fig. 13. Load-deformation results for stub Specimen II (fitted stiffeners at both ends)

Stub Specimen II: This and all subsequent stub specimens were tested with the assembly oriented vertically. The primary cause of the early failure of Specimen I thereby was removed.

Figure 13 shows the load-deformation data for this test. The maximum load of 1150 kN was reached at an average (between slab and top of stub) displacement of about 5 mm. Significant differences between the slab and the stub deformations were not observed until the load was at approximately 80% of the ultimate value. The response was linearly elastic in both slab and stub up to 50% of the failure load, and continued to be very nearly elastic in the stub until the end. Some local yielding could be observed around the ends of the stiffeners (in the web of the stub), but failure occurred before significant yielding took place.

The slab developed longitudinal cracking (Poisson's effect) at about 50% of the ultimate load, and subsequently much transverse cracking developed, especially directly above the end of the stub. Some secondary local bending therefore must have been present. The final failure came about as the "shear and compression" type that turned out to be typical for all of the stub specimens.

The failure load was less than expected, and again it was deduced that the lack of additional transverse reinforcement was the instigator of early distress. However, significant information was gained regarding the stiffness and ductility of the assembly.

Stub Specimen III: Figure 14 illustrates the load-deformation data that were recorded for Specimen III, and Figs. 15–17 show some typical and important phenomena that were observed in this and all other specimen tests.

The maximum load of 1025 kN was reached at a total displacement of approximately 8.5 mm. Some load/unload cycles were conducted to evaluate the initial and subsequent responses of the assembly, but going no higher than 300 kN (30% of ultimate). At this load level the responses were entirely linearly elastic, as expected.

The load-deformation data in Fig. 14 demonstrate the undesirable characteristics of this specimen, primarily a result of using full end-plate stiffeners. Strain gage readings as well as whitewash flaking indicated significant levels of distress throughout the stub, and particularly around the top of the end-plate (where stiffener, flange, and slab meet). Although the ultimate loads for all stub specimens were below the anticipated values, the result for Specimen III was significantly worse.

It is interesting to note that the stiffener forced the slab and top-of-stub LVDT-readings to remain almost identical throughout most of the test, as shown by Fig. 14. The "hard" point at the top of the stiffener thus was much in evidence.

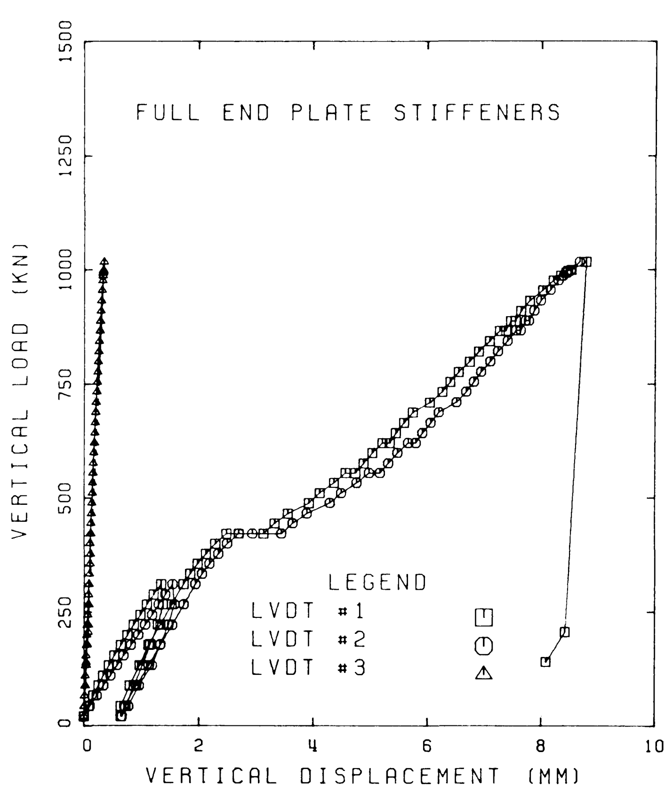


Fig. 14. Load-deformation results for stub Specimen III (full end-plate stiffeners at both ends)

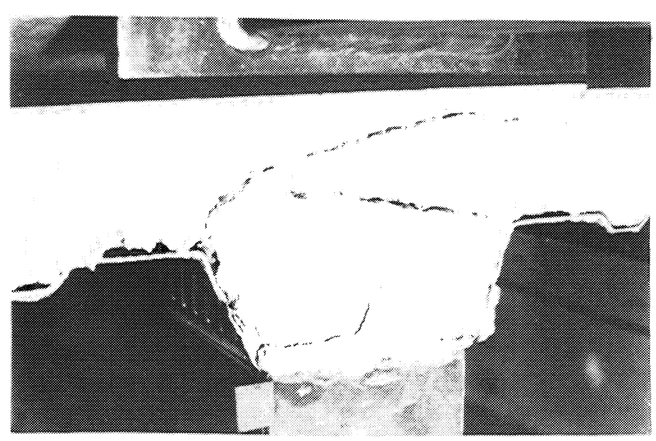


Fig. 15. Cracked end-section of concrete slab in stub Specimen III

Figure 15 shows the end of the slab after failure, indicating the typical cracking pattern of the "shear and compression" mode. Figure 16 illustrates the cracking pattern in the surface of the slab that was common to all of the specimens, including an area of spalled concrete which had left three shear connectors visible. Finally, Fig. 17 shows the buckled steel deck at the top of the stub.

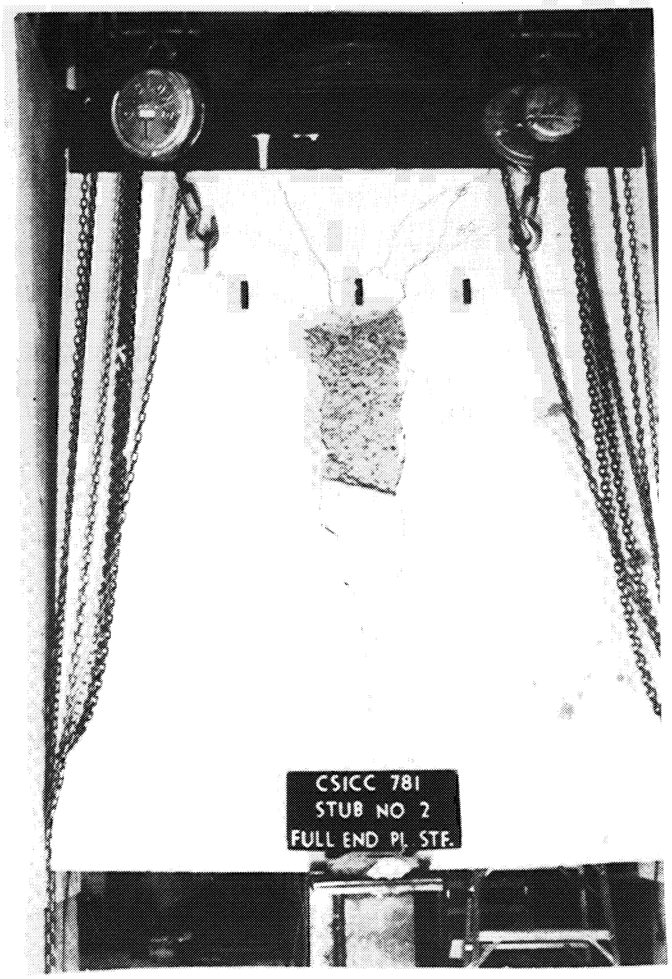


Fig. 16. Cracking pattern in top surface of concrete slab for stub Specimen III. (Notice spalled concrete, with heads of shear connectors showing)

Fig. 17. Buckling in steel deck for stub Specimen III (located at top of stub, where slab cantilevers). (Also notice yield lines in web of stub)

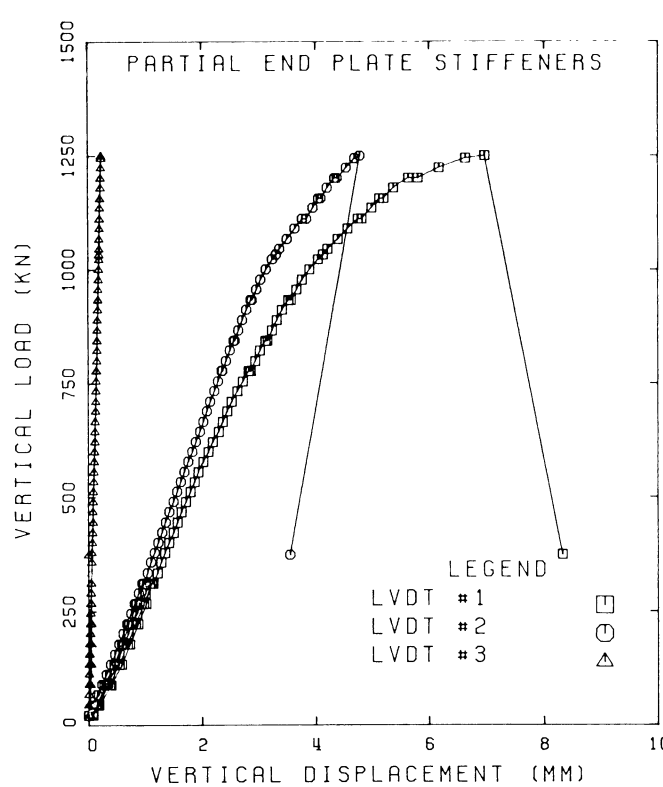


Fig. 18. Load-deformation results for stub Specimen IV (partial end-plate stiffeners at both ends)

Stub Specimen IV: Figure 18 gives the load-deformation results for Specimen IV, which utilized partial end-plate stiffeners. The maximum load of 1250 kN was reached at a top-of-stub displacement of about 4.8 mm, and a slab displacement of about 7 mm. It should be noted that this was the only one of the five tests that exhibited significantly different strains in the steel and the concrete. The response of the steel was linearly elastic up to a load of about 80% of the ultimate, and up to 50% for the concrete.

A comparison of Specimens III and IV provides some interesting findings. The two assemblies were identical in all respects except that the latter had its end-plate stiffeners end 25 mm below the top of the stub (see Fig. 10). This gave the partial end-plate specimen a 22% increase in ultimate load over that achieved by Specimen III. The overall ductility (in terms of total deformation capacity) appeared very similar, but the partial end-plate feature seemed to allow for a significantly better redistribution of internal stress resultants.

The final failure mode was the same as that of all the other stub assemblies, and no distress could be detected in the studs.

Stub Specimen V: Figure 19 shows the load-deformation characteristic for Specimen V, which utilized a single fitted stiffener in the compression region of the stub. The ultimate load of 1210 kN was reached at a displacement of about 5 mm, and there was very little difference in the deformations recorded in the steel at the top of the stub and in the concrete.

Of all five stub specimens, this one demonstrated the most extensive yielding throughout the steel, and it is the only one where the welds displayed signs of distress. Figure 20 gives an example of this by showing the yield lines that had appeared in the web and the flange (around the stiffener) at the time of failure. In the tension region, where no stiffener was used, the weld across the end of the stub flange developed a crack. The stub had started to pull away from its support ("chord"), and extensive flange bending (in a prying manner) as well as longitudinal weld yielding had taken place. The weld cracking occurred as a result of the relatively low ductility of a weld that is oriented perpendicularly to the primary direction of forces.

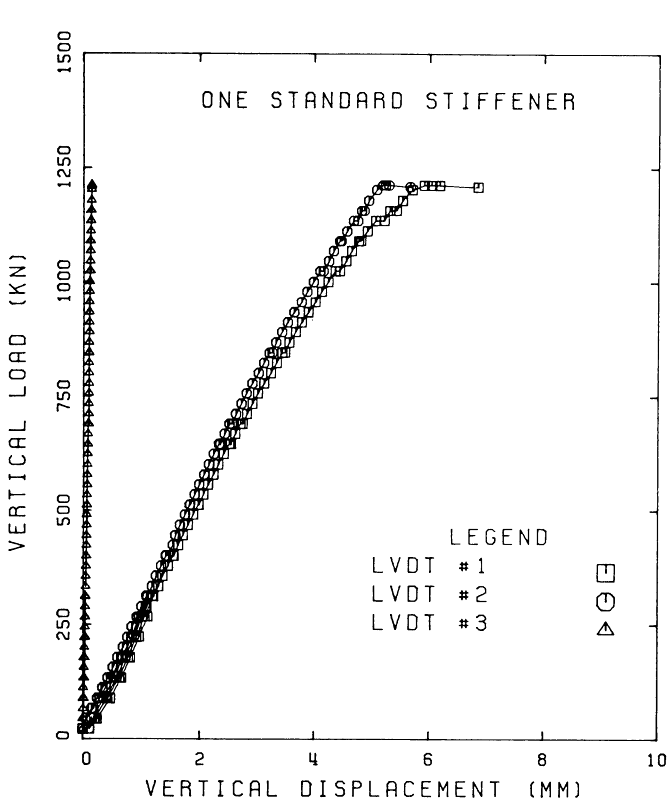


Fig. 19. Load-deformation results for stub Specimen V (fitted stiffener at one end of stub)

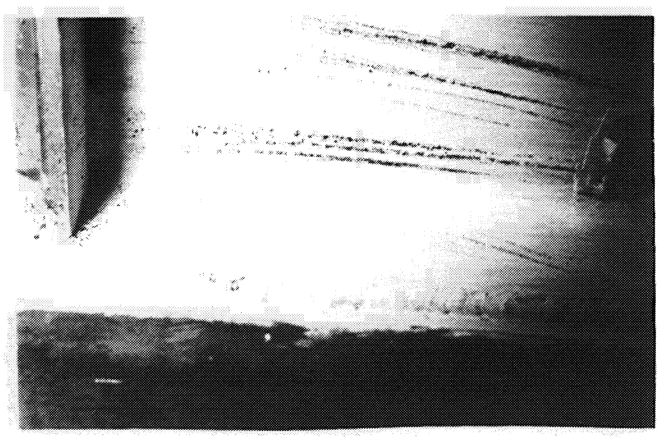


Fig. 20. Close-up view at toe of stiffener for stub Specimen V. (Notice yield lines in web and flange, especially in the flange around the toe of the stiffener)

The strength of the stub with only one fitted stiffener was slightly higher than that of the one with two (1210 kN vs. 1150 kN). The difference is too small to attach any significance to, but the type of distress that was exhibited by Specimen V is not desirable. A stiffener in the tension region therefore seems necessary.

The following recapitulation may be made as regards the results of the stub specimen tests:

1. All specimens failed at loads below the expected levels.
2. The same final failure mode was evident in all specimens, caused by a combination of longitudinal and transverse slab cracking. In effect, subsequent computations based on a combined shear and compression failure analysis demonstrated that the lack of sufficient transverse reinforcement prevented the desirable truss action of the slab to come into play.[12]
3. The top of the stub and the concrete slab exhibited very little differential displacements.
4. The shear connectors were largely undeformed.
5. Different amounts of yielding appeared in the stubs. It is important to note that identical patterns developed in the stubs of the full-size girder.

On the basis of these tests, it was decided to use the stub with partial end-plate stiffeners for the exterior stubs in the test girder, and an unstiffened element was prescribed for the interior stubs.

Stub-Girder Test—All relevant details regarding layout, materials, and so on for the test girder have already been given. In the following, the test itself will be described and observations will be made as to the implications of the results.

Three 535 kN capacity hydraulic jacks were used to apply the loads to the girder at its quarter points. All three were controlled from the same console. In the initial loading cycle, the loads were increased to the level of the service load (including service dead load of 32 kN per load point, and service live load of approximately 65 kN per load point). As Fig. 21 shows, the midspan deflection at this stage was 20 mm, which gives a deflection-to-span ratio of 1/670. In other words, the girder is extremely stiff under service conditions.

Following the loading to service conditions, the loads were reduced to zero, and all deflections and strains were recorded. The remaining (permanent) deflection at mid-span was 1.2 mm, which means that the stub-girder for all practical purposes behaved completely elastically. No distress was detected anywhere in the girder at service load.

The loads were then increased monotonically while readings were taken of all strains and displacements at approximately 8 kN intervals of jack load. The automated data recording procedure allowed a great many data-points, as the load-deflection curves in Fig. 21 shows.

Very minor yield lines started appearing in the exterior stubs at an applied load of 133 kN per jack, but significant amounts did not appear until the jacking forces had reached approximately 190 kN. By this time the design ultimate load of 183 kN per jack had been exceeded, with a mid-span deflection of 48 mm at design ultimate (= 1/279 of the span).

Extensive yielding had taken place in the south exterior stub by the time the applied loads had reached 210 kN, as the photograph in Fig. 22 shows. Figure 23 shows the yield line pattern in the north exterior stub for a jack load of about 230 kN (note that the black spot in the web of the bottom chord is electrical tape, covering strain gages). At this location yielding was also evident in the flanges of the stub (see Fig. 23). At a load of approximately 260 kN per jack, diagonal yield lines appeared in the bottom chord under the ends of the interior stubs, as shown in Fig. 24 (again, note that the large black area in the web of the chord does not represent yielding). Some yielding also had occurred at the toe of the interior stub, as can be seen in Fig. 24.

The hydraulic jacks ran out of stroke as the load reached 267 kN. The system therefore was unloaded, and the jacks reset. A permanent deflection at mid-span of 45 mm was recorded after these loads had been removed. The total deflection at mid-span for the 267 kN loads was extrapolated as something on the order of 120 mm. The LVDT's also ran out of stroke at this stage (however, it can be seen from Fig. 21 that the mid-span deflection was about 100 mm for a jack load of 250 kN).

The reloading response is also indicated in Fig. 21, and is clearly following the same kind of load-deflection characteristic that the original system exhibited. As the loads

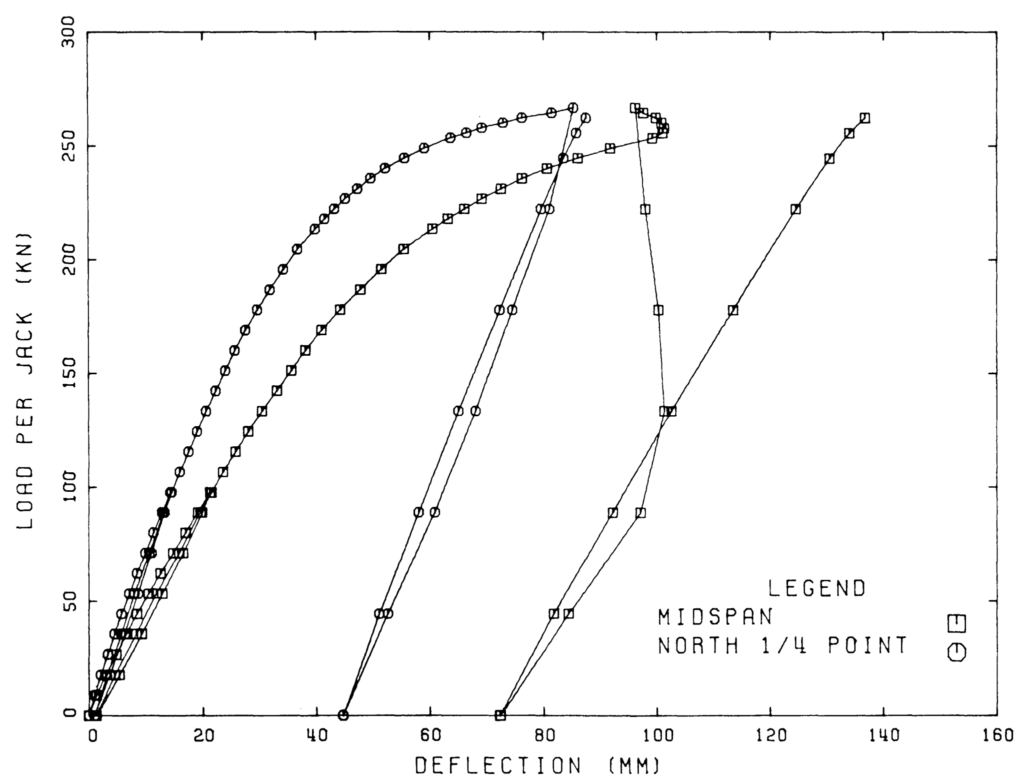


Fig. 21. Load-deflection measurements for mid-span and the north quarter-point of the full-size stub-girder

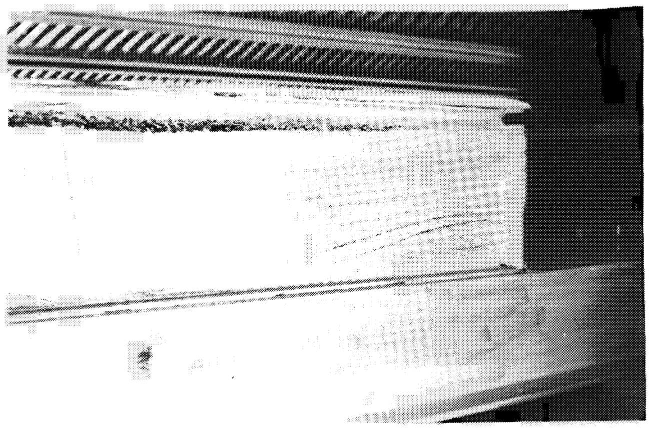


Fig. 22. Yield line pattern in south exterior stub at a load of about 210 kN per jack

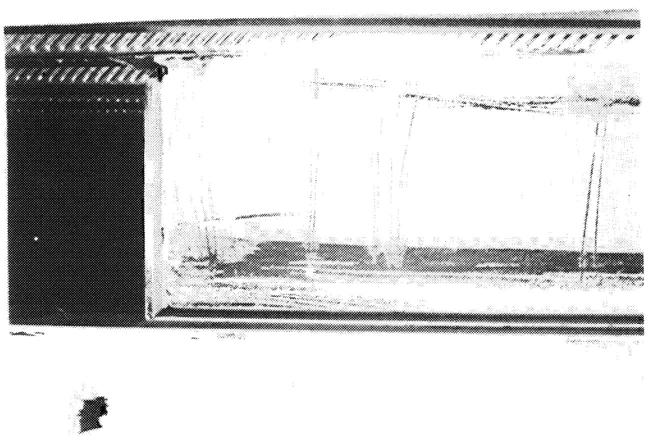


Fig. 23. Yield line pattern in north exterior stub at a load of about 230 kN per jack

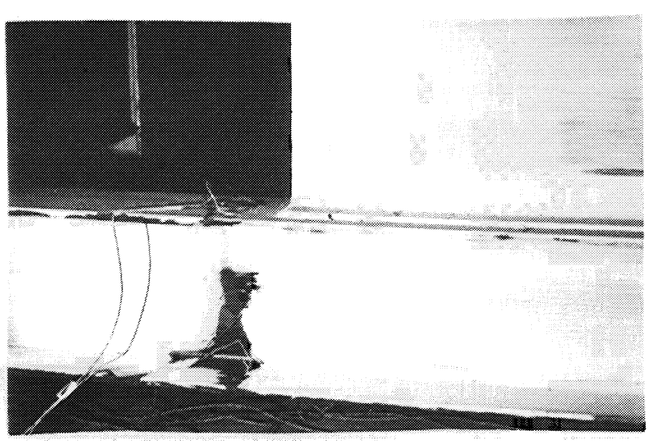


Fig. 24. Yield lines at exterior toe of north interior stub, and in web of bottom chord

reached 265 kN, a combination stud shear and stud pull-out failure occurred within the south exterior stub. The test was then discontinued, but it is noted that the mid-span deflection at this point had reached 135 mm, corresponding to a deflection-to-span ratio of about 1/100. At this stage almost all whitewash had disappeared from the exterior stubs; interior stubs had separated from the slab (by a small amount over the interior ends); and the bottom flanges of the interior stubs had separated from the chord flanges over the unwelded (central) portion. In spite of the extensive failure that had occurred, the stub-girder was still able to carry a load well in excess of the design service loads.

Examination of the top surface of the concrete slab revealed fairly extensive longitudinal and some transverse cracking, particularly in the area above the south exterior stub. The photograph in Fig. 25 shows some of the cracks.

Fig. 25. Crack pattern in concrete slab above south exterior stub

FURTHER COMMENTS ON STUB-GIRDER PERFORMANCE

Both the stub specimens and the full-size girder demonstrated that the floor system possesses a high degree of stiffness as well as excellent ductility. A great deal of reserve capacity seems to be available at service load, which can be supported even by a nominally failed girder.

Previous comparisons of ultimate-to-service load ratios generally indicated numbers around 2.0.[1,3,4,5] In this study the following ratios were obtained:

1. $\dfrac{\text{Design Ultimate Load}}{\text{Design Service Load}} = 2.30$
2. $\dfrac{\text{Actual Ultimate Load}}{\text{Design Service Load}} = 3.08$
3. $\dfrac{\text{Actual Ultimate Load}}{\text{Design Ultimate Load}} = 1.34$

Ratio No. 2 is 50% higher than what was found in other studies, and indicates a very comfortable margin of safety. Ratio No. 3 shows that the test specimen was able to carry 34% more load than the analysis had predicted.

The actual service load deflection was less than that predicted by the analysis, but the actual deflection at design ultimate was somewhat higher. The latter is understandable, since the design method utilizes elastic properties and concepts to determine the displacements at ultimate load.

The analysis of the test girder utilized the original Vierendeel approach, including the method of stub discretization, stiffener design, and shear connector design. Although the method certainly appears satisfactory, improvements could be effected in all of the above areas. These are subjects open for further research.

The changes that were made in the fabrication of the test girder appeared to make it behave significantly better than what should have been expected. Most of these changes will cause improved construction economies, notably those associated with the differences in stub stiffening and welding. The slab reinforcement seems to play an important role, particularly in the transverse direction. This has not been specifically addressed in other studies of the floor system, but would be of interest in future research.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the research that is presented in this paper, the following conclusions and recommendations can be made.

1. The Vierendeel approach to stub-girder modeling appears satisfactory, although changes in the manner of stub discretization, second order effects and method of slab representation could be the subjects of further research.
2. Deflection computations are conservative for service load; unconservative for ultimate load. However, the latter is based on elastic theory, and therefore improper in the first place. It is recommended that ultimate load deflections be eliminated.
3. The use of partial end-plate stiffeners is recommended instead of the traditional fitted stiffeners. The former gives at least as much strength and stiffness, and costs less.
4. The interior stubs can be left unstiffened in many cases. The test girder span was substantial; yet, the interior stubs showed very little sign of distress.
5. The current practice of prescribing an "all-around" stub-to-bottom chord weld is incorrect. The stub specimens as well as the stub girder utilized only a fraction of this, and still no welds were the cause of any failure. Forty-two percent of weld material thus was eliminated for each exterior stub; 73% for each of the interior stubs.
6. The amount and method of slab reinforcement appear important, particularly for the transverse direction. This should be studied further, but for the time being it is recommended that nothing less than what is prescribed by the CSA S16.1-M78 standard should be used.
7. The method of shear connector design appears to be conservative, but this is open for further study. The number of shear connectors that was used on the exterior stubs seemed excessive, even though the final failure was related to the studs. In particular, further studies are needed on the design of closely spaced studs.
8. The manner of applying the principles of the composite design to the stub-girder appears to give satisfactory results, although local stress computations must be carried out. This is particularly important for the slab.

ACKNOWLEDGMENTS

The authors wish to acknowledge the extensive analyses that were carried out by Alfred F. H. Wong during the early phases of the project. Furthermore, the suggestions and advice of Messrs. J. K. Ritchie and E. Y. L. Chien throughout the entire investigation were invaluable. The work of the technical staff of the Structural Engineering Laboratory at the University of Alberta, and particularly that of its Chief Technician, Larry Burden, is sincerely appreciated. The project was sponsored by the Canadian Steel Industries Construction Council as Grant No. 781, and the authors are very grateful for this support.

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