Current Steel Structures Research

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As has been the case for many years in all areas of the world, behavior and strength of connections continue to be among the most active research areas. It is also one of the most complex fields of study, in view of the large number of parameters that need to be incorporated into connection models and test specimens. Methods of analysis and detailed evaluation techniques have been significantly improved through state-of-the-art computer software, and many current projects have emphasized such theoretical efforts.

However, realistic physical testing has always been a key tool in the studies of connections, and the correlation between test and theory continues to be an essential component of the performance assessment. For the experimental research work, difficulties associated with component sizes and loading protocols, to mention just two of the primary considerations, are legion, as is the correlation to the response of connections in actual structures. Many laboratories are limited by their testing and measurement equipment and the extrapolation from individual tests to full-scale structures may be questionable. On the other hand, several research facilities in the United States and other countries are now capable of full-scale testing, and the results are very promising. A number of studies currently under way are examining the test performance of full- or large-scale structures with realistic connections and details. It is encouraging to note that some recent findings have demonstrated that certain long-established design approaches are in fact very acceptable.

It is also interesting to observe that structural stability, for individual members as well as framing systems, is seeing a resurgence of research activity. This has, in part, been prompted by the needs for enhanced design tools, but also by extended applications of thin-walled elements to housing construction, for example. State-of-the-art design specifications already reflect some of these efforts, and further improvements are expected as the studies continue.

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Improved materials, structural systems and connection details for composite construction, for buildings as well as bridges, are now under development in several locales. New elements and forms have especially been considered by European bridge designers, and economy of construction is addressed through novel but realistic applications of optimization techniques.

References are provided throughout the paper, whenever such are available in the public domain. However, much of the work is still in progress, and reports or publications have not yet been prepared for public dissemination.

CONNECTIONS IN STEEL STRUCTURES

High-Strength Steel Moment Connections: This is a major research project that involves physical tests and analytical evaluations of the deformation demands associated with moment connections in S690 (100 ksi yield stress) and S960 (137 ksi yield stress) steel (Girão Coelho, Bijlaard, and da Silva, 2006). The work is a collaborative effort between the Delft Technological University in Delft, the Netherlands, and the University of Coimbra in Coimbra, Portugal, with Professor Frans Bijlaard at Delft as the director. It is expected that the work will continue for some time, owing to the grades of steel that are used and the many design considerations that must be examined.

Based on the increased use of high-strength, high-performance steel in all types of structures and the perceived limited deformation capacity of some of these steels, it was decided to undertake a study of the deformation demands in certain moment connections. Three types of flush and extended end-plate connections as shown in Figure 1 were used, with end-plate steels as described earlier. For the welding details of connections Flush 1 and Flush 2, it is noted that the plate for Flush 1 extends beyond the shape, with the fillet welds placed accordingly. For Flush 2 the plate is smaller than the depth of the shape, such that the welds are placed between the edge of the flange and the edge of the plate. These are common details for such connections in Europe.

The primary goals were to determine whether these connections would qualify for PR-type analysis as prescribed by Eurocode 3, and whether they would be able to provide the necessary large inelastic deformations. From an American standpoint, the 100 ksi steel (ASTM A514) is only available in plate form. It is rarely used for anything other than selected applications in bridges and some transfer girders in buildings. Further, there is no American structural steel counterpart to the S960 European grade. Finally, hot-rolled shapes are not currently available in these two steel grades. Nevertheless, European steel design practice differs significantly from American approaches, in certain areas.

The end-plate connection and its tension region bolts were chosen for the first part of the study, specifically because the response characteristics are affected by the following factors:

- Connection geometry, especially the ratio between the end plate thickness and the bolt diameter
- Material ductility for the steel of the plate and the bolts
- Ratio of strength between the plate and the bolts
- Weld quality

The tests specifically aimed at validating the connection design criteria of Eurocode 3 (CEN, 2005) are detailed in Parts 1-8 and 1-12 of that code. But the primary focus of the project was to examine whether such high-strength steel connections have sufficient ductility and rotation capacity to qualify for global inelastic analysis of the structure.

Ten connections were fabricated and tested. The steel in the columns and the beams was the common HSLA Grade S355 (50 ksi yield stress). The special HS steel grades were only used for the end plates, and six of the connections had S690 plates; the other four used S960. Plate thicknesses were relatively small (by American standards), at approximately ⅝ in. (15 mm) and ⅜ in. (10 mm). These are common dimensions for such connections in European practice. All bolts were 24 mm in diameter, and all but one of the connections utilized the bolt Grade 12.9, with F_u = 175 ksi. Such a bolt grade is not available for structural applications in the United States. One of the specimens with an S690 plate had Grade 8.8 bolts, which is the same as ASTM A325. Clearly, the connection design focused on developing controlling limit states associated with plate deformation or fracture and avoiding limit states controlled by bolt behavior and strength.

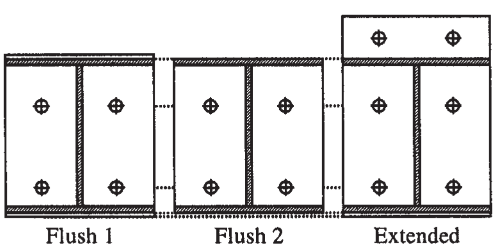

Fig. 1. High-strength steel end-plate connections for deformation demand tests.

The test results demonstrated that the strength of the connections was adequately predicted by the limit states defined by Eurocode 3; at the same time the researchers note that the code criteria overestimate the stiffness. But these findings are the same as those determined for connections in lower strength steels. Further, given the requirements for rotation capacity, it was also found that the test specimens satisfied the criteria, although it was apparent that the code guidelines can be very conservative. Finally, the study has concluded that the efficiency of high-strength steel connections is not related to the higher yield level, but also that the deformation demands of these connections are higher than those of lower strength steel. Additional research work along these lines is anticipated, specifically to attempt to develop practical expressions for the evaluation of the rotation capacity on the basis of material and geometric parameters.

Contact Splices for Columns: Recognizing that research on performance and design criteria for column splices is very limited, this study aimed at developing guidelines for certain types of splices. It was conducted at the Technical University of Berlin, in Berlin, Germany, with Professor Joachim Lindner as the project director (Lindner, 2006).

The earliest known study of column splices is that of Hayes (Hayes, 1957). No other known column splice research was conducted in the United States until Popov undertook a project at the University of California at Berkeley in the mid-1970s (Popov and Stephen, 1977). This was an AISC-funded study that looked specifically at the strength and response characteristics of heavy columns with splices, including splice imperfections in the form of lack of contact, and with filler plates to improve the bearing performance.

Professor Lindner considered three common splice details, with a view toward the column strength as well as the behavior of the splices themselves. Figure 2 illustrates the common European types of splices, and Figure 3 shows the imperfections that may be found in column splices. It is noted that the lack of contact shown on the left in Figure 3 is the type of imperfection used by Popov and Stephen, although their study also incorporated filler plates in accordance with the then-existing fabrication quality criteria of AISC (AISC, 1971).

For several reasons, the Lindner research team elected to study columns with the Type c splices shown in Figure 2 only, with imperfections as shown on the right in Figure 3. A total of 15 tests with varying amounts of imperfections were conducted. The column slenderness ratios were all approximately 55, within the middle range of inelastic column behavior. The steel grade was S235, with a nominal yield stress of 34 ksi (235 MPa). Extensive computer modeling was also done, with close correlation between the experimental results and the numerical data. The computations also included column splices of Types a and b, as shown in Figure 2.

The study recommendations include a number of dimensional and column size details. These include the use of at least four M24 (24 mm diameter) pretensioned Grade 10.9 bolts (equivalent to ASTM A490) for each side of the column [M27 bolts for columns deeper than 500 mm (20 in.)]. A separate column curve is also developed for use with columns that are not spliced at the ends. The curve lies slightly below Eurocode Curve c for slenderness values λ_c less than 1.5. The curve is essentially the same as Curve c for larger values.

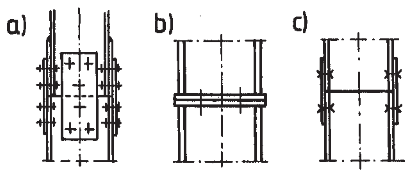

Fig. 2. Types of column splices of Lindner project.

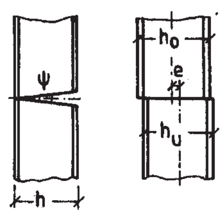

Fig. 3. Column splice imperfections.

As an aside, in comparison to United States column strength requirements, the AISC column curve for all practical purposes coincides with Eurocode Curve b for slenderness values between 0 and 1.0; it is very close to Eurocode Curve a for larger λ_c values (Bjorhovde, 2006).

Strength and Stiffness Requirements for Column Splices: It is interesting to note that without any work on column splices for many years, another study of has recently been carried out at the Eindhoven University of Technology in Eindhoven, the Netherlands, under the direction of Professor Bert Snijder (Snijder and Hoenderkamp, 2006). This work, like the project of Professor Lindner in Germany, aimed at providing rational criteria for certain splice types, with proper theoretical and experimental backing. The authors observe that the design criteria of Eurocode 3 appear to be very conservative in certain respects and without a proper technical base for some of the requirements.

The column splices that were selected for study are shown in Figure 4. These are very common European details. Splices A and B have certain advantages and disadvantages, but they are often the choice of designers and fabricators. Splice C was specifically chosen as one with a very limited stiffness. Finally, in addition to the tests of the spliced columns, full-length columns without splices were also tested to provide a direct measure of the effect of the splices. A total of 12 tests (4 × 3) were conducted with a column length of 11 ft (3.39 m) and splices as shown in Figure 4, including three unspliced specimens. Another six (2 × 3) tests with splice B and column lengths of 15 ft (4.54 m) and 28 ft (8.46 m) were also run. The steel grade was S235, with a nominal yield stress of 34 ksi (235 MPa), although the actual yield stress was found to be 43 ksi (298 MPa).

The end plate thicknesses of configurations A and B are ½ in. (12 mm) and ¼ in. (6 mm), respectively. The "a" measure shown for A and B represent the fillet weld size of 5 mm (approximately 3⁄16 in.). The columns were all of the European size HE100A, which is a hot-rolled shape with a depth slightly less than 4 in. (100 mm); it is comparable to the

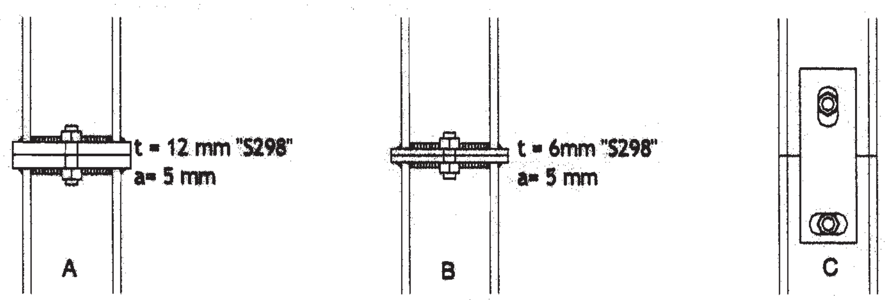

Fig. 4. Column splices used in the Eindhoven test program.

American shape W4×13. Obviously, the size of these members is not representative of American construction practice. The column size may have been restricted by the laboratory capacities at the university in Eindhoven.

The effectiveness of the splices was evaluated on the basis of the so-called "5% criterion." This states that a connection (in this case the splice) may be regarded as rigid if the strength of the member or frame into which it is connected is reduced by 5% or less (Bijlaard and Steenhuis, 2002). In this case the member is a column; the column splice may be regarded as rigid if it does not reduce the column strength by more than 5%.

Determining the column strength on the basis of the respective curve of Eurocode 3, it was found that the tested capacities of the spliced columns were approximately 33% higher than the code value. This is not necessarily surprising, since the column curve covers a range of different column types and sizes. However, what is interesting and potentially significant is that the test strengths of the Type B spliced columns were all within 4% of the capacity of the unspliced member. In other words, even a very flexible connection such as the splice with ¼ in. (6 mm) end plates effectively allowed the column to behave as a continuous member.

The researchers recommend that additional studies be undertaken to cover the full range of column sizes and lengths and all realistic splice designs.

SEISMIC RESPONSE OF STRUCTURES AND CONNECTIONS

Hybrid Experiments of Large-Scale Steel-Braced Frames and Steel Shear Walls: This is a major program that has been conducted at the National Center for Research on Earthquake Engineering (NCREE) and National Taiwan University in Taipei, Taiwan, with Professor K.-C. Tsai as the director. American support has been provided by Stephen Mahin of the University of California at Berkeley, Charles Roeder of the University of Washington, and Rafael Sabelli of DASSE Design in San Francisco.

Recognizing that buckling-restrained braces (BRBs) and steel plate shear walls (SPSWs) have seen increasing usage over the past several years, but also that full-scale testing of structures with such elements has been very limited, the research team developed plans for the testing of a three-story, three-bay braced frame with BRBs. Figure 5 shows the plan and the elevation of the frame that was tested, noting that it used square and circular concrete-filled columns, with 7-ft-wide (2.15 m) concrete slabs, 23.5 ft (7 m) bay sizes and 13.5 ft (4 m) story heights. These dimensions make it one of the largest frame tests ever conducted. The test was run with pseudo-dynamic load input, using data from the Chi-Chi (Taiwan) and the Loma Prieta earthquakes.

The frame had three different kinds of moment connections, with one type for each floor level. Similarly, three types of BRBs were used, with single-core, double-core, and all steel elements. In particular, the second-story BRBs were single-cored unbonded braces, with the unbonded ends connected to gusset plates with splice plates and high-strength bolts. The ends of the double-cored BRBs that were utilized for the first- and third-floor levels were also connected to gusset plates with high-strength bolts.

The first test series produced failures in the bracing members, including in the BRBs, the unbonded braces, and the gusset plates. The buckling failure mode of one of the gusset plates is shown in Figure 6a. In the second test series all buckled gusset plates were removed, and improved (stiffened) plates were fabricated, as illustrated in Figure 6b.

The tests have demonstrated that buckling-restrained braces are very effective elements in seismic-resistant braced frames, as is known and would be expected. The edge-stiffened gusset plates behaved extremely well, although the researchers observed that these elements introduced flexural deformation demands into the BRBs. This requires additional study on the efficiency of BRBs.

Finally, the gusset plates were designed on the basis of the well-known Whitmore Criterion, using Thornton's design procedure (Thornton, 1984). With appropriate edge-stiffeners, it was found that this design approach leads to gusset plates that can withstand cyclic loads as well as the kinds of bi-directional deformation demand that was demonstrated in the tests.

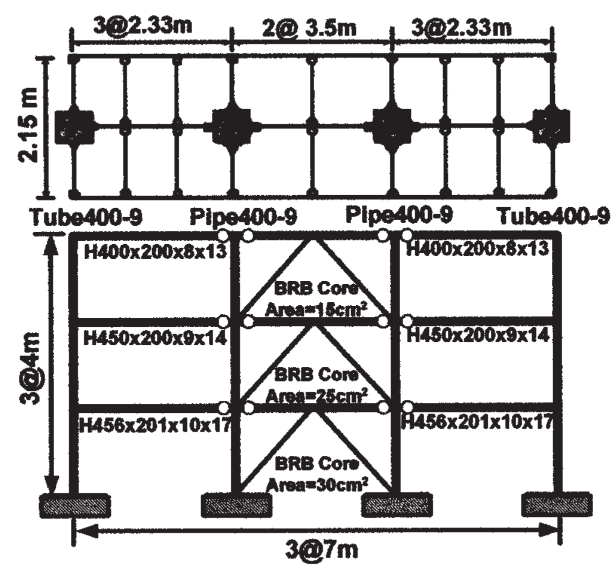

Fig. 5. Braced frame with buckling-restrained braces.

Reduced Beam Section Connections with Built-Up Box Columns: This study has been conducted at the Seoul National University and the Sungkyunkwang University in Seoul, Korea, with Professors Kim, Lee, and Park as the lead researchers (Kim, Lee, and Park, 2006).

Welded built-up box columns are commonly used for steel-framed buildings in Japan, Korea, and other countries. Some of this is based on long-standing tradition; other reasons may be attributed to the extensive utilization of concrete-filled construction and some of its advantages for strength and stiffness of framing members. On the other hand, some of the earthquake-related failures of box columns can be traced to inadequate rotation capacity of the moment connections that have been used. On this background the researchers decided to examine the potential of reduced beam section (RBS) connections between built-up box columns and W-shaped beam members, to shift the rotation demands from the box column to the RBS area of the beam.

The study included tests of three full-size, cantilever-type RBS connections, as illustrated in Figure 7. In addition, inelastic finite element analysis was used to model the response of the connections. The same column and beam sizes were used for all three specimens. The metric (SI) dimension designations are shown in the figure, indicating a 16 in. × 16 in. column with ¾ in. walls and 1 in. horizontal stiffeners (diaphragm); the beam is a 28-in.-deep H-shape with 12 in. × 1 in. flanges and a ½ in. web. The steel grades were SM 490 (70 ksi yield stress) for the column and horizontal stiffener materials; it was SS440 (65 ksi yield stress) for the beam. Normal complete-joint-penetration groove and fillet welds were used for the joints of the beam flanges and the web plate, respectively. Electroslag welds were used for the horizontal stiffeners to the inside of the column plates.

One of the specimens failed prematurely by fracture of the electroslag welds between the stiffener and the inside of the column wall. The other two connections displayed adequate rotation capacity and local deformation demands. Based on the analytical data, the researchers note that for unreinforced box columns, the response characteristics are very different from connections between wide-flange type columns and beams. With RBS connections, however, the tests and the analytical results showed that the differences are small, because the primary deformation demand is removed from the column face. This is as would be expected.

COMPOSITE CONSTRUCTION

Composite Action through Adhesive Shear Connection: This project addresses the potential of a continuous shear connection by using an adhesive between the concrete slab and the steel beam in composite structures. The work is being carried out at the University of Champagne-Ardenne in Reims, France, with Professor Yves Delmas as the project director. The study features full-scale physical tests as well as nonlinear analytical evaluations.

(a)

(b)

Fig. 6. Buckled and improved (stiffened) gusset plates.

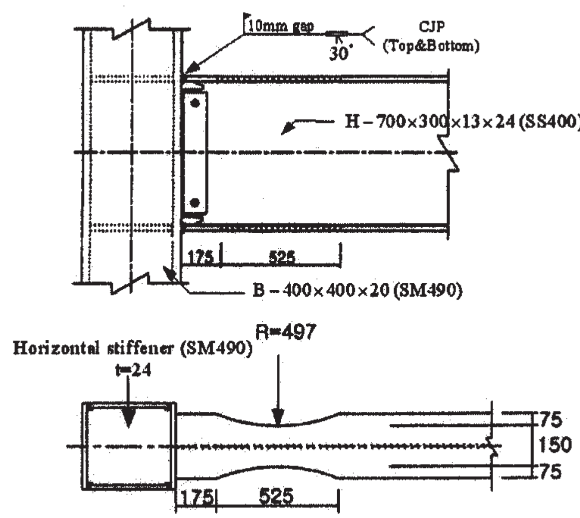

Fig. 7. RBS connection with welded built-up box column.

Some of the advantages of an adhesive connection are the removal of the stress concentrations that develop around the common stud shear connectors, the opportunity to use precast slab elements for the beams, and to improve onsite working conditions. Among the difficulties is the quality of the adhesive joints as well as their stiffness. The flexibility and hence the important redistribution characteristic of stud shear connectors have been one of the significant advantages of this form of composite construction, and this issue forms a major part of Professor Delmas' work. Different types of adhesives are used to examine these behavioral aspects, focusing on initial and subsequent distribution of stresses and deformation demands, and including the establishment of ultimate limit states.

The long-term stability and performance of the adhesive materials, including the effects of temperature and humidity, may be incorporated in future studies.

Cost Estimation, Optimization, and Competitiveness of Composite Floor Systems: This study is being carried out at the University of Maribor in Maribor, Slovenia. The project researchers are Uroš Klanšek and Stojan Kravanja.

The fabrication and erection costs of three types of composite floor systems are evaluated, including the potential for cost optimization. The systems are (1) floors with composite wide-flange shapes, (2) floors with composite trusses using hot-rolled channel sections, and (3) floors with composite trusses using cold-formed hollow structural sections. The evaluations incorporate all material, labor and fabrication costs, fire protection, painting (one top coat), slab formwork (if any), and power consumption for all types of operations in the shop and the field. The analytical solutions are developed by nonlinear programming techniques, providing "open form" expressions that are amenable for the use of the method under different economic and technological conditions. The cost objective functions are based on rigorous structural analysis and design criteria to ensure that the solutions incorporate ultimate as well as serviceability limit states. The design criteria are those of Eurocode 3.

The solutions demonstrate that under the present conditions, the composite trusses with cold-formed hollow structural sections are less cost competitive than the other two systems that have been examined.

BRIDGE STRUCTURES

Connections by Adherence for Steel-Concrete Composite Bridge Structures: This research project aims at developing cost- and time-effective shear connection systems for composite bridge structures, especially where precast slabs or slab elements have been designed for use as the bridge deck. The project is directed by Dr. Jean-Paul Lebet at the Federal Polytechnic Institute of Lausanne (EPFL) in Lausanne, Switzerland (Thomann, 2005; Thomann, Lebet, and Dauner, 2006).

Whether a new structure or the rehabilitation of an existing bridge, current construction often tends to be lengthy and very complex. This is especially the case when precast slab elements are intended for use as part of a composite structure. The common stud shear connectors do not lend themselves very well to the precast solution, and composite action by the adherence concept offers significant potential and advantages. Adherence provides resistance through friction between adjacent surfaces and delivers a continuous, as opposed to a series of, discrete shear connection. One example of such shear connections is the surface of the top flange of the special shape that was developed for so-called slim-deck construction in England, as developed by the British steel company Corus (previously known as British Steel).

Figure 8 illustrates one type of adherence connection that was developed by the Lausanne research team. The precast slab elements are prepared beforehand with a slot on the underside for the connection to the embossed steel plate, and assembly in the field is rapid and very accurate. The tests that have been run have shown that the connection meets the bridge code requirements of reliability, robustness, and economy. They exhibit high shear strength and stiffness under static load, both at the service and the ultimate load levels.

Additional work is currently under way, especially to address the issues of fatigue performance. But the analytical models that have been developed appear to be very practically oriented, including the approach to determine the plastic capacity of members with adherence shear connections. Figure 9 shows the load-deformation curves for a typical stud shear connection and a comparable adherence connection; it is clear that additional work is needed to establish the complete plastic performance characteristics.

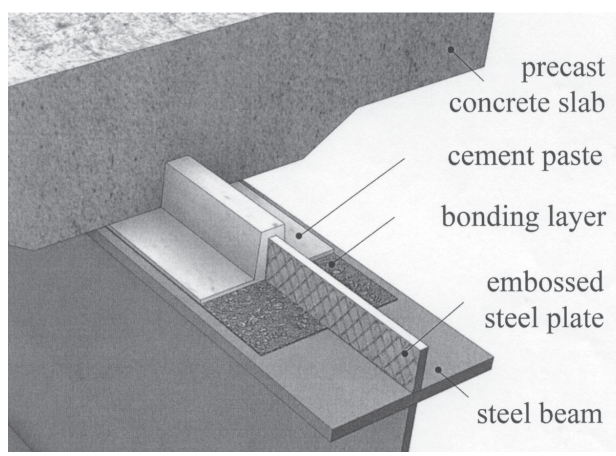

Fig. 8. Adherence connection for steel beam to precast slab.

Shear Connections in Composite Girders with Corrugated Steel Webs: Corrugated steel webs are currently being used in numerous bridges in Europe; they are being examined for use in American bridges. These elements offer significant advantages insofar as transverse web stiffness is concerned, since stiffeners and bracing diaphragms between adjacent girders can often be eliminated. The fatigue performance record in German bridges is very good. The project in question is being conducted at the University of Stuttgart in Stuttgart, Germany, with Professors Ulrike Kuhlmann and B. Novak as the primary researchers.

The study is looking at the possibility of replacing the steel girder flanges through the casting of the concrete slab as an integral connection with the corrugated web. A number of recent tests have shown that this solution offers certain advantages, especially with the high shear resistance of the interconnection between the slab and the web in the absence of mechanical shear connectors. The work is currently under way, and a total of 22 specimens have been tested. Further results are expected to confirm and expand on the initial findings.

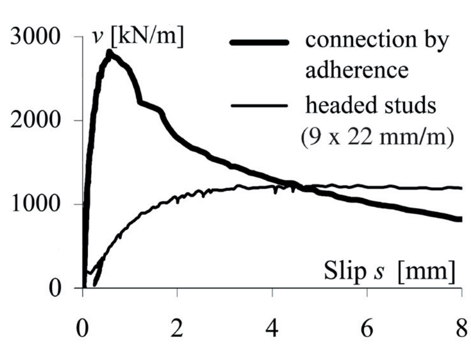

Fig. 9. Load-deformation curves for stud shear connector and adherence shear connection.

ACKNOWLEDGMENTS

Special thanks are due the following members of the International Structural Steel Research Advisors (ISSRA) who provided input to this paper:

Darko Beg, University of Ljubljana, Ljubljana, Slovenia

Dinar Camotim, Technical University of Lisbon, Lisbon, Portugal

André Colson, French National Contractors Association, Paris, France

Manfred Hirt, Federal Polytechnic Institute of Lausanne, Lausanne, Switzerland

Ulrike Kuhlmann, University of Stuttgart, Stuttgart, Germany

K.-C. Tsai, National Taiwan University, Taipei, Taiwan

Additional assistance has been provided by Joachim Lindner, Technical University of Berlin, Berlin, Germany; Bert Snijder, Technical University of Eindhoven, Eindhoven, the Netherlands; and Michel Thomann, Federal Polytechnic Institute of Lausanne, Lausanne, Switzerland.

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