ISSN 0394 - 5103

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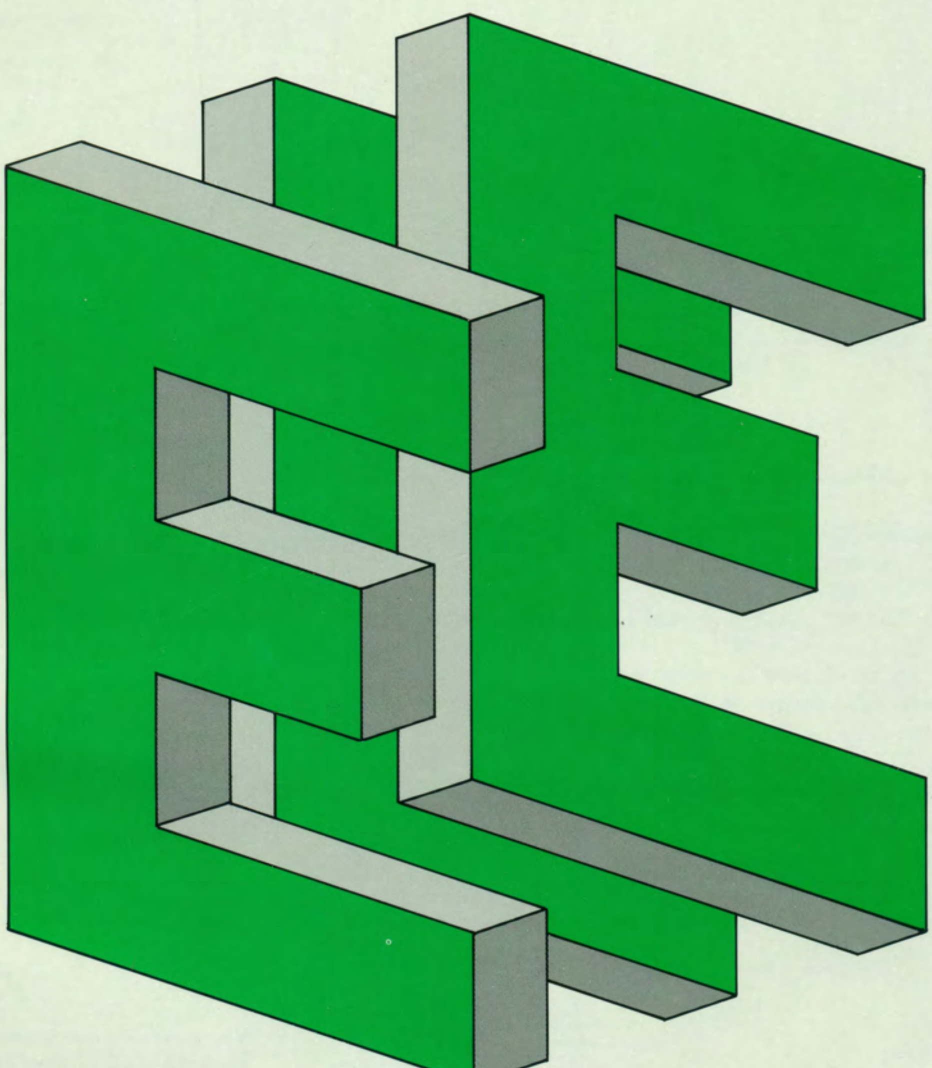
EUROPEAN EARTHQUAKE ENGINEERING

INTERNATIONAL JOURNAL OF EARTHQUAKE ENGINEERING AND ENGINEERING SEISMOLOGY



Composite behaviour under quasi-static and dynamic loading

A. Plakas*, J. Bouwkamp**, P. Carydis*, H. Mouzakis*

SUMMARY – This paper considers the influence of the loading-rate of different test procedures on the seismic response of composite steel/concrete cantilever beams. Full scale composite specimens with varying stud spacing were investigated both in the shaking table and through comparative displacement controlled quasi-static tests. The test results and their analysis show very good agreement between those obtained under low-rate pseudo-static displacement controlled forcing conditions and under high frequency real time shaking table input conditions. The influence of stud spacing is also evaluated.

KEY WORDS: Composite, Earthquake, Loading-rate, Shear studs.

Introduction

In order to evaluate the potential seismic response of structural elements, assemblages or entire structures through experimental tests, displacement-controlled quasi-static (also called pseudo-static) and pseudo-dynamic loading tests as well as dynamic shaking table studies are commonly used. Questions have been raised about the comparative validity of these different methods in as far as the test results – stiffness, load resistance and ductility – may be influenced by the loading-rate used in the different test procedures. Nevertheless, considering the fundamental frequencies of typical buildings and the test frequencies in general, this influence has been considered negligible in predicting the structural response under actual earthquake excitation.

On the other hand, considering the basically different characteristics of steel and concrete (being ductile and brittle, respectively, and having markedly different stress-strain relationships), the results of seismic tests on steel/concrete composite members and connections may possibly be affected by the different test methods used. Accordingly, several studies addressing this subject have been performed during the last decade. Ballio et al.(1990) focussed on an assessment of the influence of the concrete on the seismic behavior of composite connections under quasi-static load conditions. Constantinescu et al. (1992) presented the results of several pseudo-dynamic tests on composite steel/concrete joints with different levels of column-web ductility. Pradhan (1995) discussed the results of pseudo-dynamic tests considering the effects of various test parameters and the potential influence of experimental error propagation on the cyclic behavior of composite beam-column joints. In order to allow a comparison of the pseudo-dynamic test results reported by Pradhan, several shaking table tests on identical composite beam-column joints were performed by Carydis et al. (1995). Detailed comparative studies of the results of the above noted pseudo-dynamic and shaking table tests were presented by Pradhan (1998).

Pradhan (1998) reports that the test specimens for both the pseudo-dynamic and shaking table tests were fabricated and concreted at Darmstadt and consisted of partially encased composite (in-filled concrete) beam (HEA260) and column (HEB300) sections with both bolted and welded joints. The specimens for the shaking table studies were subsequently transported to Athens for testing at the Laboratory for Earthquake Engineering of the National Technical University of Athens.

Using for the pseudo-dynamic (PsD) tests the same time-history input, lumped mass quantity and damping as for the shaking table (ST) tests, the response time-histories of the PsD tests, carried out at the Darmstadt University of Technology, were compared with the corresponding ST test results. Comparisons were made for a total of 8 specimens covering welded and bolted connections with shear-panel stiffnesses of 7, 11 and 15 mm, respectively. Considering the response force-histo-

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Received june 2001. Revised october 2001.

ries for comparative tests, the results showed an excellent agreement, however, surprisingly, a comparison of the PsD and ST displacement histories for all tests showed a higher displacement response for the ST tests than for the PsD tests. This would indicate that the test specimens exhibited a higher stiffness in the PsD tests than under the real-time test conditions; a result basically contrary to the common phenomenon of increasing stiffness under increasing loading rates. Subsequent numerical simulations of both the ST and PsD tests, considering different lumped masses and various filtered table-top accelerations did not reveal possible errors in the shaking-table test set-up or table input excitations.

With the intent to clarify the earlier observations, it was decided to undertake a further study on the cyclic response of different composite steel/concrete beam-column assemblies, using both PsD and ST loading conditions. These studies were carried out at the Laboratory for Earthquake Engineering, National Technical University, Athens. A description of the test specimens, test set-up and program as well as the results are presented herewith.

In addition to assessing the influence of the loading-rate of different test procedures on the response of the composite steel/concrete slab cantilever beam under cyclic loads, the effect of the composite interface design, with shear studs spaced at 160 and 240 mm, respectively, reflecting a full and partial interface shear resistance has also been investigated. These studies were part of a general prenormative research effort carried out under the TMR-ICONS Research Project funded by the European Commission to develop detailed design guidelines for composite buildings within Eurocode 8: Design of structures for earthquake resistance.

Test Specimen and Test Set-up

The basic test specimen studied, was a welded steel beam-column assembly, consisting of a steel 2.0 m long IPE 200 beam section, with a 0.60 m wide and 0.10 m thick concrete composite slab, and a HEB 220 column stub. The overall dimensions of the steel members and concrete slab, together with the layout of the reinforcing steel, are presented in figure 1. Typically, the headed shear studs had a diameter of 13mm and a length of 75mm. The slab reinforcement consisted of a single layered mesh of longitudinal and transverse bars

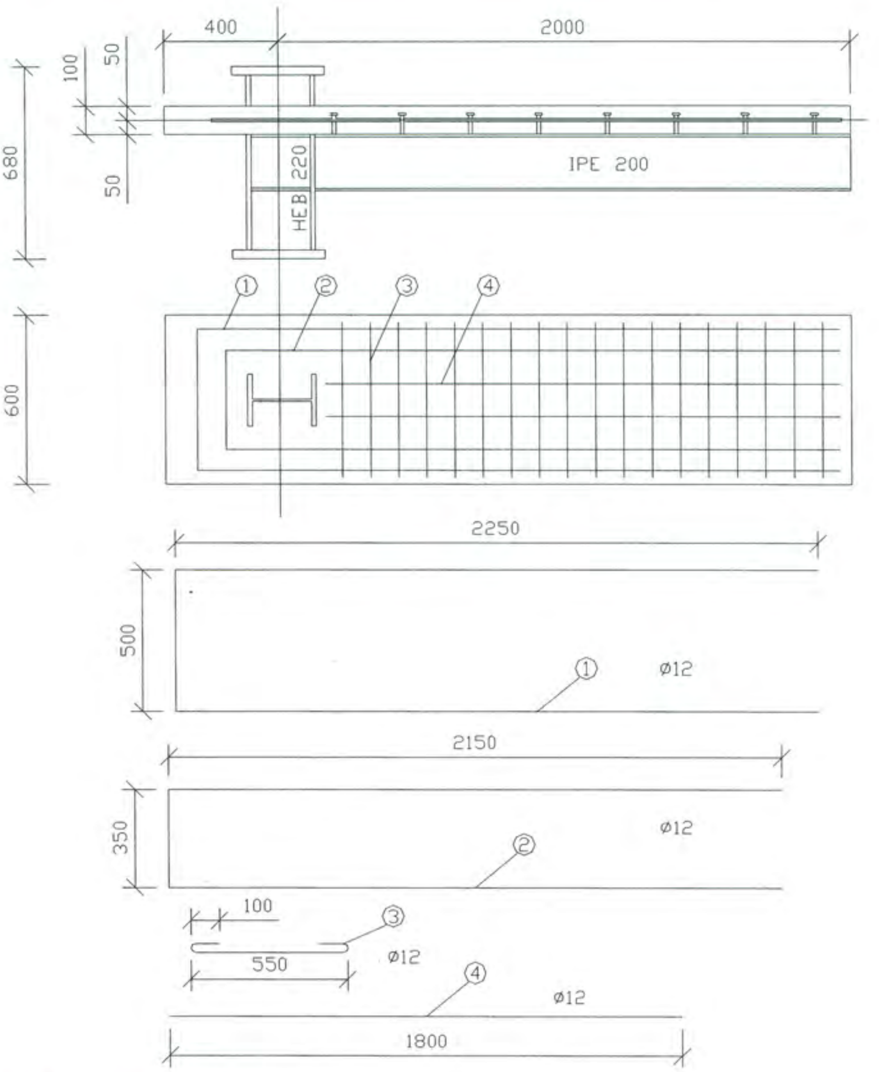


Fig. 1 – Test specimen and slab reinforcement.

positioned at mid-depth of the 100 mm thick concrete slab (see Fig. 2).

The specimens were designed to study the response of the composite concrete slab and steel beam section under cyclic cantilever beam moments and assess the possible influence of the loading rate by comparing the test results under both real-time (shaking table) and quasi-static test conditions. In order to develop the full composite slab capacity through a uniform interface shear resistance, a stud spacing of 160 mm was called for (see Fig. 3). This specimen was identified as C160. In order to study possible loading-rate effects under cyclic loading for a specimen with an interface design having a reduced (partial) shear capacity, a second specimen, identified as C240, was designed with a stud spacing of 240 mm (see Fig. 3). In order to permit an evaluation of the influence of the composite slab under both positive and negative moment actions, a third type of specimen with only a steel beam (without a composite slab) has been fabricated and tested. This specimen, identified as specimen S, would further allow an evaluation of the basic effect of real-time shaking versus pseudo-dynamic testing.

Considering the fact, that under bending the positive moment resistance of the composite cantilever section, with the concrete slab under compression, is larger than the negative moment resistance, and that yielding will occur first under a negative moment action, it was decided to use, for the shaking-table study, a symmetric test set-up with two coupled test specimens (see Fig. 4). Under those conditions, the combined resistance of the coupled beams would be the same, independant of the dynamic forcing direction. Otherwise, in case of a single specimen, first yielding in either direction would initiate a potential progressive cyclic deterioration in that particular (first-yield) direction and lead to early failure. As shown in Fig. 4, the test specimens were mounted vertically on the table through the sets of short, 0.85 m long, HEB 200 steel stubs. The specimens were connected to the stubs through eight M16 10.9 HS bolts; the stubs in turn were mounted to the table by four M27 10.9 HS bolts.

The dynamic test mass consisted of five interconnected steel plates, placed on top of each other, with a total mass of 5 Mg. The arrangement of the plates were such that the center of mass was located above the point of

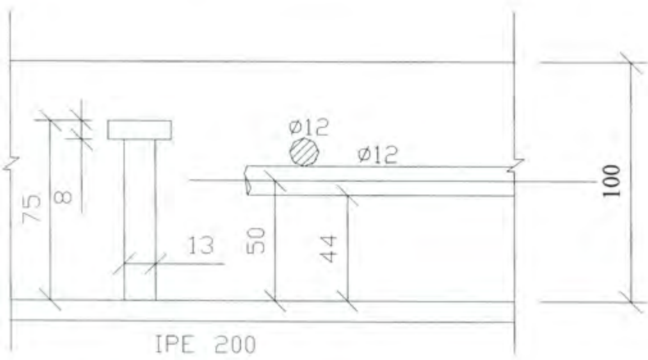
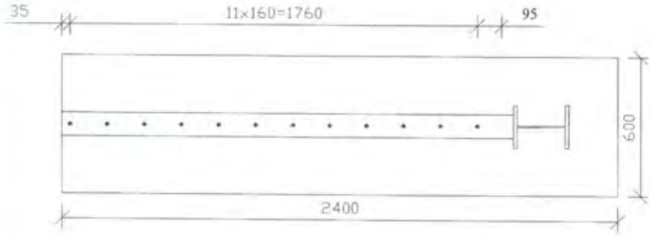
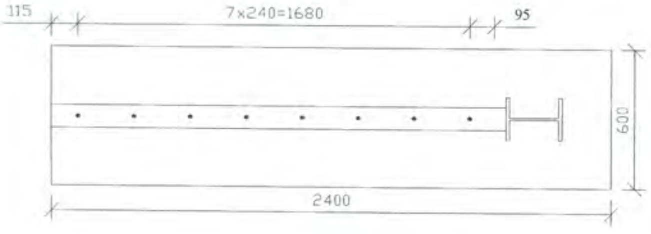


Fig. 2 – Detail of shear studs



(a) C160



(b) C240

Fig. 3 – Shear stud layout, specimen C160 and C240.

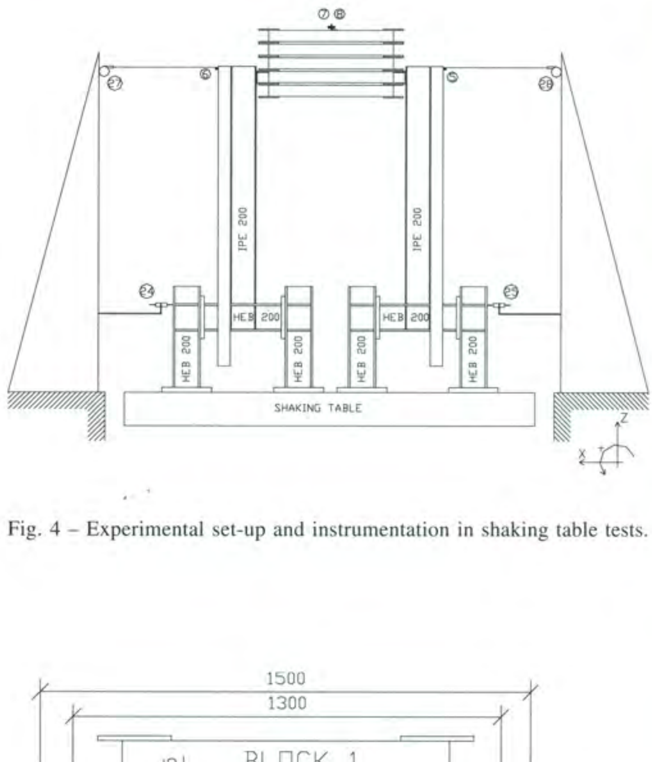


Fig. 4 – Experimental set-up and instrumentation in shaking table tests.

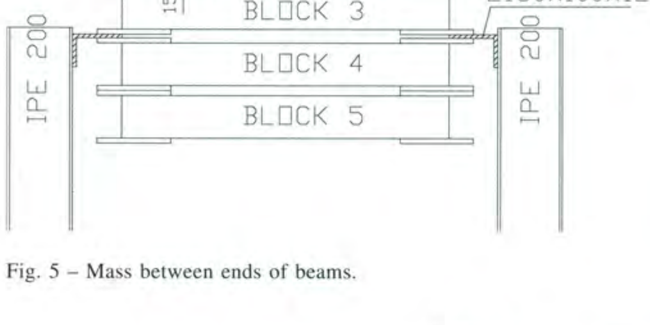


Fig. 5 – Mass between ends of beams.

connection to the test beams. This was achieved by placing three steel plates above and two steel plates below the connection level. At that position, the mass was bolted through connection plates to angles (L 150×100×12) which were welded to the end of the bottom flanges of the test beams (see Fig. 6). In order to ascertain that the angle support would not fail prematurely, the type of support was pretested by subjecting the outstanding leg successfully to over 2000 cycles of +/– 40 mrad.

For the pseudo-dynamic tests, a single test specimen, as shown in Fig. 7, was mounted vertically in a reaction test frame in the same manner as on the shaking table. A double-acting, displacement-controlled, hydraulic load cylinder, connected to the tip of the test beam, was used to introduce the same displacement history at the beam end, as had been observed in the real-time shaking table tests.

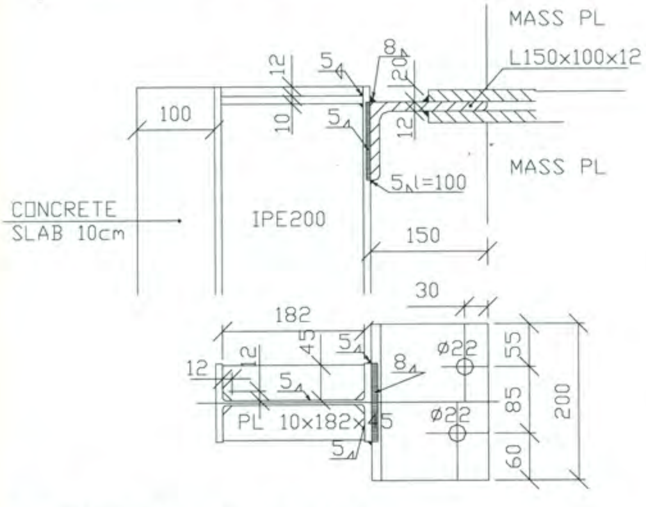


Fig. 6 – Detail of connection, beam to mass.

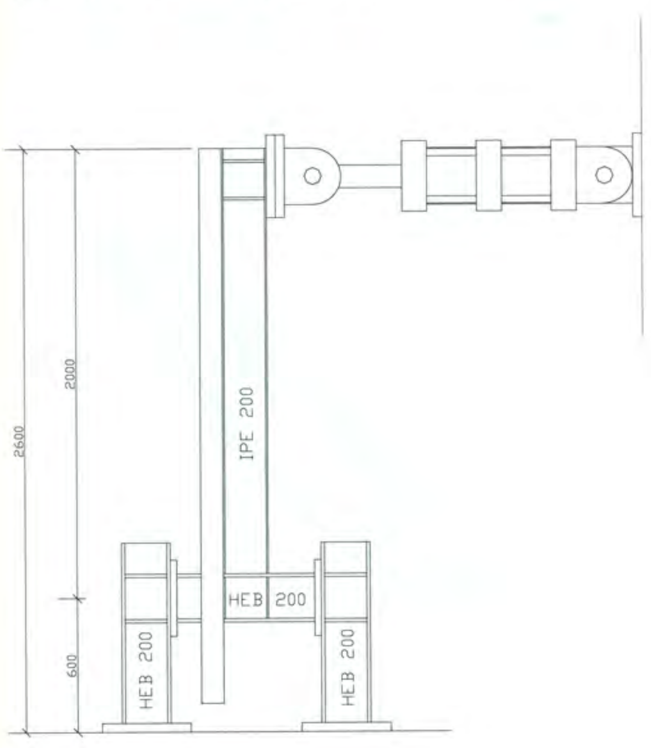


Fig. 7 – Test set-up for pseudo-static test.

Test Programme and Test Results

Test Programme

In order to develop a dynamic nonlinear response of the coupled test specimens, it was decided, to subject the table not to a particular seismic ground-displacement time-history but rather to a predefined cyclic displacement rampfunction with full displacement reversals (see Fig. 8). The cyclic test frequency was set constant at 80% of the natural frequency of the test set-up. This procedure had been used successfully by Pradhan in previous studies, as the onset of yielding basically leads to a reduction of the natural frequency of the test set-up and an increased dynamic amplification; thus resulting in an enhanced forcing efficiency of the table input. In these studies, the rampfunction selected had a uniformly increasing (10%) ascending branch of 10 full cycles, followed by 5 full cycles at the same maximum cyclic displacement level. After these 5 full cycles, the rampfunction called for 3 further, uniformly increasing (10%), displacement cycles, followed by 5 cycles of a constant magnitude. The test was concluded by an additional 5 cycles with linearly decreasing (20%) cyclic displacements, down to zero. The table displacement magnitudes were selected such that yielding of the test set up could be expected.

During the dynamic tests, the response of several accelerometers, placed on the test specimen both at the end of the cantilever beams (identified as numbers 5 and 6 in Fig. 4) and on top of the mass (numbers 7 and 8) were recorded. Also, the tip displacements of the cantilever beams (numbers 27 and 28) were recorded during the tests. In order to assess the efficiency (stiffness) of the column-stub mounting devices, additional displacements (numbers 24 and 25) were monitored and compared with other pertinent records.

For the pseudo-static cyclic tests, the specimens were subjected to displacement controlled forces using

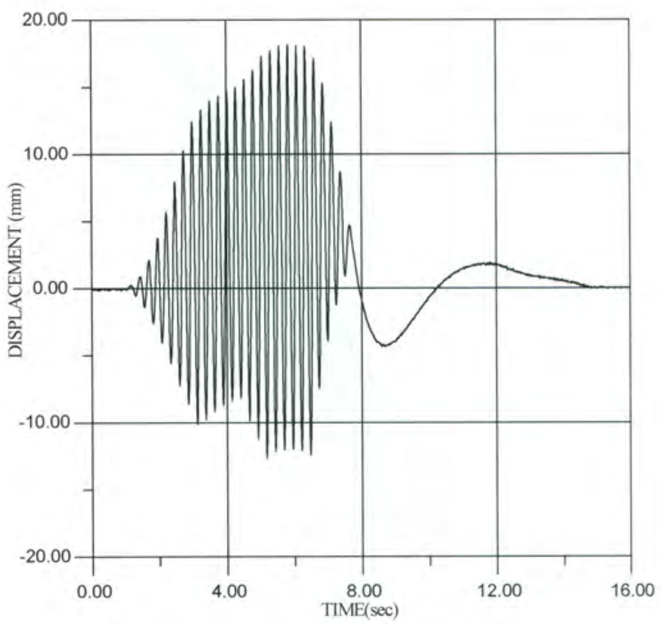


Fig. 8 – Table – displacement time history achieved.

as input the corresponding dynamic displacement record measured at the tip of the cantilever beams under the above described table excitation. Considering that the displacements of the mounting devices relative to the table displacements were negligible, the observed displacement history recorded at the tip of the cantilever beam of the pertinent dynamic test specimen was used as the displacement-control input for the corresponding pseudo-static test.

Test Results

Considering the above test procedures, first, the coupled specimens under real-time dynamic test conditions were tested. Subsequently, after having obtained the displacement response at the tip of the cantilever beams, the displacement controlled pseudo-static tests on the single specimens were performed. The results of the dynamic tests of specimens C160, C240 and S, presented as the «effective» beam moment at the face of the beam-column connection versus the induced tip displacement of the cantilever beam, are shown in Figures 9, 10 and 11, respectively. The «effective» beam moment is in fact half of the total moment (of both beam cantilevers) or, the average of the positive and negative moments of the two cantilever beams, at any given instance.

Specifically, Fig. 9 shows the moment-displacement record of the second dynamic test run (test 2) performed on the coupled C160 specimens. In fact, a total of 7 test runs were performed on this test set-up; while in test runs 1 and 6 the test specimen was subjected to only a few cycles of low-amplitude table displacements, in runs 2, 3, 4 and 5 complete table-displacement ramp functions, as described earlier, were introduced with a table frequency of 3.89 Hz and maximum cyclic table accelerations of 12.5, 17.5, 20 and 20 m/sec^2, respectively. In the last test run (run 7), the coupled specimen was subjected to more than 70 full cycles of table motion at a reduced frequency of 2.89 Hz and a constant peak table acceleration of 10 m/sec^2. In order to illustrate the response of the coupled C160 specimen under these changing test conditions, not only results of test run 2 (Fig. 9), but also the results of test runs 4 and 7, as shown in Figures 12 and 13, respectively, are presented. These figures show the «effective» cantilever beam-moment versus beam-tip displacement. In principle, the results of the several test runs allow evaluating the effectiveness of the closely spaced shear studs on the behavior of the composite beam-slab cantilever under repeated cyclic loading.

The results of the pseudo-static tests for specimens C240 and S, showing the true cyclic beam moment versus the corresponding cantilever tip displacement,

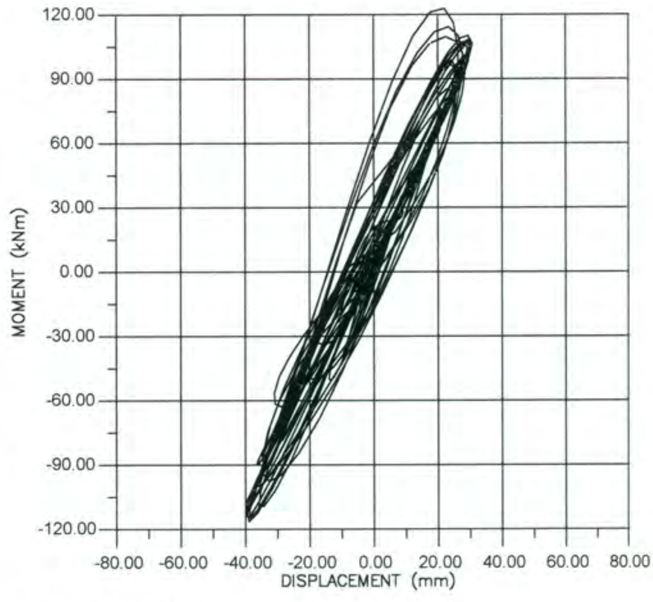


Fig. 10 – C 240 Dynamic.

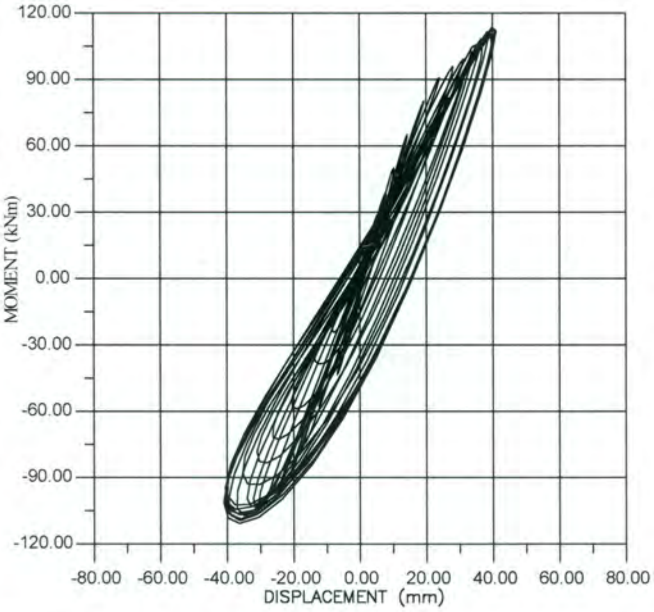


Fig. 9 – C 160 Dynamic (test 2).

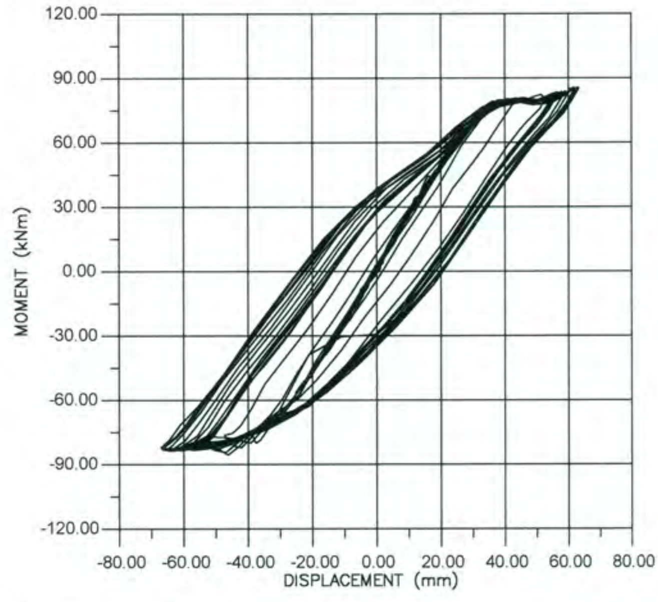


Fig. 11 – Steel Dynamic.

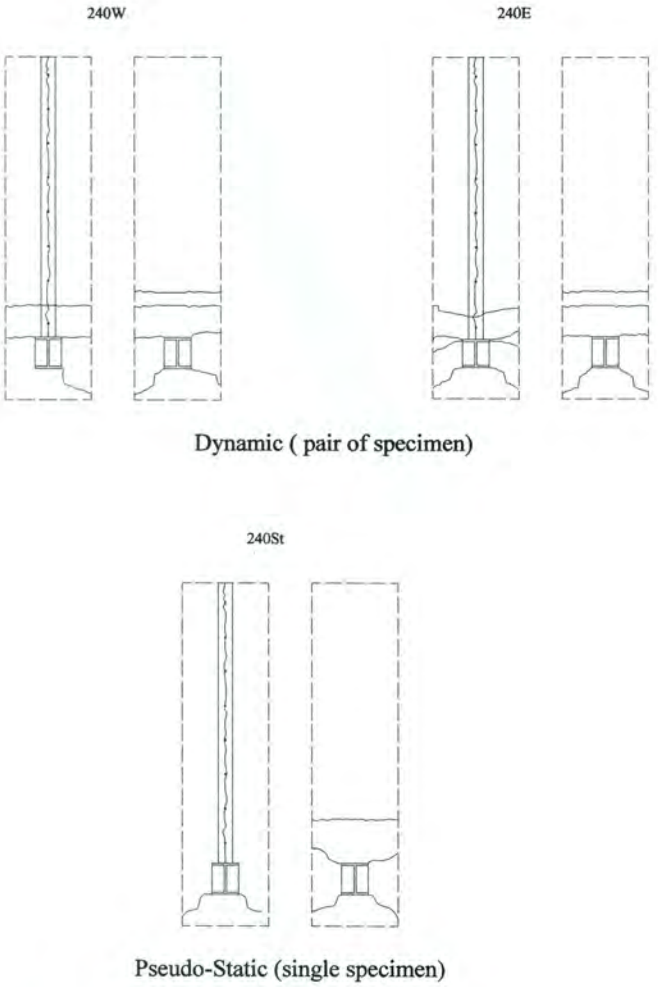


Fig. 16 – Concrete cracking of C240.

behavior reflects the effect of increased concrete cracking (see Fig. 16). Specifically, for displacements of –5 and –10 mm the «stiffness» changed from 5.4 to 4.4 kNm, respectively. Comparing these values with the «stiffness» of the steel beam only (specimen S), the composite section under a positive moment had an initial bending stiffness 2.31 times larger than the steel section. However, under a negative moment and tip displacements of –5 and –10 mm, respectively, the bending stiffness values for the composite specimen were only found to be between 1.86 and 1.52 times the bending stiffness of the steel cantilever beam specimen.

According to Fig. 14, the maximum positive and negative moments for the C240 specimen under pseudo-static loads were found to be +114 and –102 kNm, respectively. These values compare to calculated positive and negative resisting moment values (MRd) of +110 and –99 kNm, respectively. The positive moment resistance was based on values of fyd = 310 N/mm^2 for the steel section and a. fcd = 0.85 · 25 = 21.25 N/mm^2 for the C25 concrete slab. The effective width (under compression) was assumed equal to the width of the HEB220 column flange (beff = bc = 220 mm). Following the basic concept, developed by Plumier et al. (1998), but assuming an effective width of beff = bc + 0.7 hc = 374 mm, the positive moment resistance of the composite section tested, can be calculated at +128 kNm. Considering the test results, an effective width of 220 mm seems to be realistic. In calculating the negative resisting moment of –99 kNm, the slab reinforcing steel was assumed to have a yield stress of fs = 600 N/mm^2.

Dynamic Tests

Considering the moment-displacement curves for specimen C240 in Fig. 10, it is remarkable that the maximum displacements in the «effective» positive and negative moment regions are significantly different, namely, about 30 mm in the positive moment range and close to 40 mm in the negative moment range. As both the test set-up and the table-input rampfunction were symmetric, the noted response must be directly associated with a difference in the behavior of the two coupled specimens. The symmetry of both the test set-up and table forcing input was substantiated by the fact that a close observation of the first 7 to 8 cycles, with maximum tip displacements of up to + and – 10 mm, showed that the moment-displacement relationship was symmetric and linear. In fact, at the same tip displacement, the sum of the positive and negative cantilever moments, occurring simultaneously in the different beams, should be the same throughout the test. However, considering the noted differences in the maximum displacements, the test results seem to indicate a different shear-interface behavior of the two beams with a progressive failure in one direction of motion. This behavior is supported by the fact that observations after the test showed that several shear studs had failed (up to fracture).

The «effective» maximum moments for a single cantilever beam, as presented in Fig. 10, should in fact have been the average of the possible maximum positive and negative moments of a (a single) C240 composite beam. With maximum values of +114 and –102 kNm, as shown in Fig. 14, the «effective» maximum moment should have been 108 kNm for both the positive and negative moment values. However, in comparison, Figure 10 shows maximum moment values of +108 and –115 kNm (despite subsequent failure of some of the shear studs towards the end of the test). However, considering the overall hysteretic response of the pseudo-static (Fig. 14) and dynamic tests (Fig. 10) of the C240 specimens, the shape of the hysteretic loops are quite similar.

A comparison of the dynamic test results of the two coupled steel beam specimen S, presented in Fig. 11, and those of the pseudo-static test specimen (Fig. 15), shows a good agreement up to a cyclic displacement level of about 30 mm (at which premature flange buckling under a negative moment terminated the pseudo-static test). The first 7 cycles in both test cases indicate closely similar initial «stiffness» values of 2.9 and 2.6 kNm/mm for the pseudo-static and dynamic tests, respectively. The almost 10% lower «stiffness» for the dynamic test as compared with the pseudo-static test, is rather strange in consideration of the increased loading rate in the dynamic test case. However, because of possible test errors, the difference may not be significant. In fact, the acceleration data used in the data reduction process to calculate the forcing effect of the 5Mg mass block were measured at the top of the block and reflected in principle not only the translational motions of the block, but also, to some degree, the effect of rotational motions of the block. It would be speculative to evaluate either the extent of translational amplification of motion due to the fact that the accelerometer was placed on the top of the block, or the possible counter-rotational effects due to the differences of the vertical displacements and deformations of the steel brackets supporting the mass-block. On the other hand it should be mentioned here that the rotational response of the mass in the shaking table tests is not expected to affect the results significantly, as it has been assessed, for the particular shaking table used, and found to be at the utmost of the order of 4%.

The natural frequencies of the dynamic test set-ups of the coupled specimens, C160 and C240, showed that the initial stiffness of both types of composite beams was identical. However, as signified by the difference in the maximum cyclic displacements noted before, C240 seemed to have experienced an early loss of the shear resistance at the slab-beam interface in one or both of the coupled beams.

Considering the results of test runs 2, 4 and 7 of the coupled C160-specimen test set-up, as shown in Figures 9, 12 and 13, respectively, a rapid deterioration at the shear stud interface does not seemed to have occurred. Specifically, in test run 2 with maximum table accelerations of 12.5 m/sec^2 at a frequency of 3.89 Hz, the test results presented in Fig. 9, show maximum «effective» moments of +112 and –110 kNm and maximum cyclic tip displacements of + and – 40 mm. During test run 3, with increased maximum table accelerations of 17.5 m/sec^2 at the same table operating frequency of 3.89 Hz, the test results showed exactly the same «effective» maximum moments as in test run 2, but, because of the increased forcing level, increased maximum cyclic tip displacements of +49 and –51 mm. In principle, the results do not indicate any deterioration of the beam-slab interface.

In test run 4, during which the maximum table acceleration was increased further to 20 m/sec^2 at basically the same table frequency (3.88 Hz) as before in test run 3, the results shown in Fig. 12 indicate, as expected under the larger forcing input, a further increase of the maximum cyclic tip displacement to +60 and –55 mm. However, a reduction of the maximum «effective» moments from about +/– 110 kNm to values of +104 and –103 kNm, seems to indicate the onset of a shear-interface deterioration. In test run 5, with the same maximum table acceleration of 20 m/sec^2 at a table frequency of 3.89 Hz, a reduction of the «effective» maximum moments were observed with values of only +90 and –95 kNm. The associated maximum tip displacements also showed a significant reduction to only +48 and –51 mm. The later reduction is probably in part a result from a less effective table forcing input as the stiffness of the specimen had fallen by more than 30%; calling in fact for a table forcing frequency of about 3.2 Hz. The loss of moment resistance observed in test runs 4 and 5 are a clear indication of a distinctly non-symmetric loss of shear resistance at the beam-slab interface.

In the final test (test run 7), the table frequency had been adjusted to reflect the continuing loss of stiffness during the earlier test runs (which had reached, during a short exploratory run – test run 6 – a value of 55% of the initial stiffness). Consequently, a table frequency of 2.89 Hz was selected and the coupled specimen subjected to over 70 cycles with a constant peak table acceleration of 10 m/sec^2. Compared with the results of test run 5, the test results of test run 7, presented in Fig. 13, show a further reduction of the maximum «effective» moments to +81 and –90 kNm. Because of the improved table forcing input, the maximum tip displacements did not further decrease significantly and reached values of +52 and –47 mm. In fact, the moment values approach the moment resistance of the bare steel sections and indicate an almost complete loss of beam-slab interface resistance.

Conclusions

Considering the basic objective of studying the influence of the loading rate effect on the composite interactive behavior of a beam-slab cantilever section, the results show excellent agreement between the test results obtained under low-rate pseudo-static displacement controlled forcing conditions and high-frequency (3-4 Hz) real-time shaking table input conditions. In fact, it can be concludes that pseudo-static (also called quasi-static) or pseudo-dynamic test procedures are capable of capturing the real dynamic behavior of composite structures.

Furthermore, despite identical initial stiffnesses, specimens with a code-designed stud spacing (160 mm in this case), versus those with a 50% larger stud spacing (240 mm), showed a better than three-fold life expectancy under maximum, yield introducing, forcing conditions.

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