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Division V

IMPROVEMENTS IN NUMERICAL SIMULATION OF IMPACT INDUCED VIBRATION AND DAMPING BEHAVIOUR OF A REINFORCED CONCRETE STRUCTURE TESTED IN IRIS PHASE 3 PROJECT

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ABSTRACT

Impact induced vibrations and their transmission through a reinforced concrete building to floors and walls distant from the place of impact are influenced by the structural behaviour of the impact zone. Amplitudes and frequency content depend on the degree of energy dissipation due to local damage and the amount of damping of the structural elements on the transmission path. Phase 3 of the international benchmark project IRIS (Improving Robustness assessment of structures Impacted by a large miSsile at medium velocity) is dedicated to this research object. The project is organised by the OECD NEA (Nuclear Energy Agency) and, besides three other institutions, funded by ENSI.

According to the project description, see EDF (2017), the IRIS Phase 3 includes experiments with a box-shaped test specimen with open sides consisting of front and rear wall as well as floor and ceiling slab. The projectile impacts on the front wall. The rear wall is extended by an attic at the top. The test specimen is supported by four anchored feet made of steel pipe profiles. On the inside of the rear wall, two construction elements for the simulation of equipment components are mounted, which are fastened at anchor plates by welded resp. bolted connections.

In the last SMiRT conference, the authors already have reported on blind nonlinear dynamic analyses, which were carried out in phase A of the benchmark and afterwards compared with the measurements of the impact test series, see Borgerhoff et al. (2017). Subject of the current paper are the modifications implemented in the numerical model for the calibration calculations of benchmark phase B aiming at improvements in the numerical simulation.

INTRODUCTION

The OECD NEA (Nuclear Energy Agency) launched the activities for phase 3 of the IRIS project in August 2016. Subject of this project phase is the transmission of impact induced vibrations from the nonlinearly deformed impact zone to building structures outside the impacted area.

As experimental basis of the IRIS Phase 3 benchmark, three consecutive impact tests with a specially developed reinforced concrete (RC) mock-up were carried out. These tests, which were funded by the institutions ENSI, EDF, IRSN and STUK, were realised in October 2016 by the VTT Technical Research Centre of Finland in Espoo (Finland) and documented by Vepsä et al. (2016). The box-shaped

test specimen with open sides consists of front and rear wall as well as floor and ceiling slab, see Figure 1. The front wall is impacted by the projectile. The rear wall is extended by an attic at the top. The test specimen is supported by four anchored pedestals consisting of pipe profiles. On the inside of the rear wall, two construction elements for the simulation of equipment components are mounted, which are fastened at the anchor plates by welded resp. bolted connections.

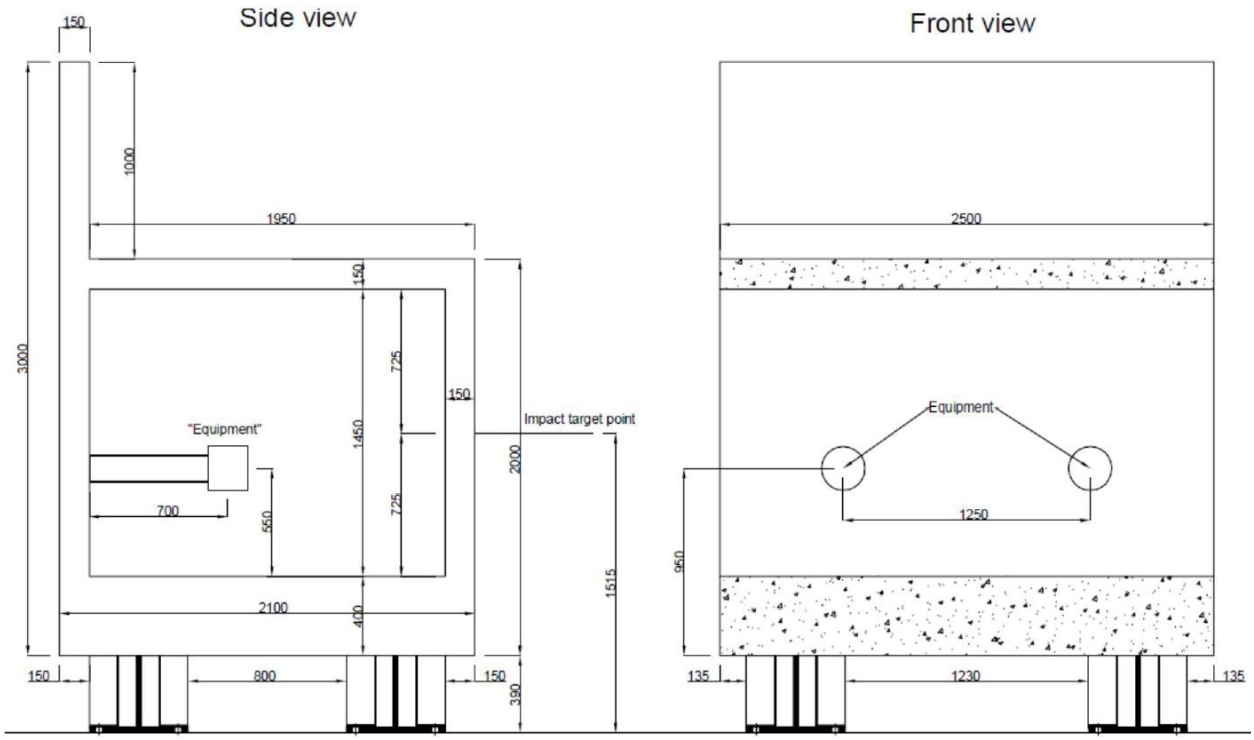

Figure 1. Geometry of the IRIS Phase 3 mock-up.

In the last SMiRT conference, the authors have reported on blind nonlinear dynamic analyses, which have been carried out in phase A of the benchmark and later were compared with the measurements of the impact test series, see Borgerhoff et al. (2017). In phase B of the benchmark, modifications were implemented in the numerical model within the scope of calibration calculations aiming at improvements in the numerical simulation. These modifications and their impact on the quality of the comparative analyses are reported in this paper by comparing calculated and measured test results.

DESCRIPTION OF THE TESTS

The dimensions of the box-shaped RC mock-up are given in Figure 1. The mock-up is supported by means of four steel pedestals, which are designed for transmission of vertical as well as horizontal forces to the anchorage in the floor of the test hall by Dywidag rock anchors with a diameter of 32 mm and material of St 950/1050. The steel equipments are mounted on the inside of the rear wall by plates with embedded anchors. The cantilever beams are attached to the anchor plates one with a bolted connection and the other with a welded connection. The finished mock-up structure and the pseudo-equipments at the rear wall according to Vepsä et al. (2016) are shown in Figure 2.

Figure 2. Mock-up structure (left), equipments as installed at the rear wall (right).

The concrete quality used for the mock-up is C40/50 with the maximum aggregate size being 8 mm. The basic reinforcement of walls and ceiling consists of steel bars B500B with diameter 6 mm and spacing 50 mm. The front wall is completely shear reinforced with diameter 6 mm hooked stirrups and spacing 100 mm/50 mm. The concrete covers are 35 mm (floor), 10 mm (front wall) and 25 mm (rear wall and ceiling).

The IRIS Phase 3 test series comprises three consecutive impact tests. Two types of deformable tubular projectiles with a wall thickness of nearly 2 mm were used. The projectiles used in tests 1 and 2 had a pipe length of 1500 mm. The pipe length of the projectile for test 3 was 2400 mm. The masses of both types of projectiles were about 50 kg. In the first two tests, the system response should be limited to mainly reversible deformations. The corresponding measured impact velocities were 91.8 m/s in test 1 and 93.5 m/s in test 2. Striving for significant nonlinearities particularly in the impact zone, the achieved impact velocity in test 3 was 167 m/s.

NUMERICAL MODELLING

Improved computer model of the mock-up

The dynamic analyses were carried out with a finite element (FE) model of the mock-up structure using the computer code SOFiSTiK (2014), see Figure 3. The reinforced concrete structure of the test specimen is modelled with nonlinear, layered shell elements. Slabs and walls are subdivided into 12 concrete layers, and the crosswise reinforcement at both sides of the structural elements is considered.

Compared to the blind predictions in phase A of the benchmark, the FE model is calibrated based on the test data with respect to several features. The modifications relate to the consideration of the measured material properties of concrete and steel, the modelling of the boundary conditions given by the anchoring of the steel supports and the definition of the Rayleigh damping parameters using the measurements before and after the single tests according to Vepsä et al. (2016). A distinction in modelling of the different connection types of the equipment components was waived.

Another modification, which has proved to be very relevant to the correct simulation of the mock-up behaviour, concerns the connection between the walls and the floor slab. The modelling of this detail was changed from shared connecting nodes in the intersection lines of the median planes of walls and floor slab to a rigid coupling of the lower wall nodes shifted to the level of the floor surface with the floor

slab nodes, see Figure 3. Due to the big thickness difference of walls and floor slab modelled by shell elements in their median planes, in case of shared nodes the height of the walls in the FE model is significantly larger than the free length between top of floor slab and bottom of ceiling slab. The related too low stiffness resulted in phase A of the benchmark in superelevated horizontal displacements and vibration periods.

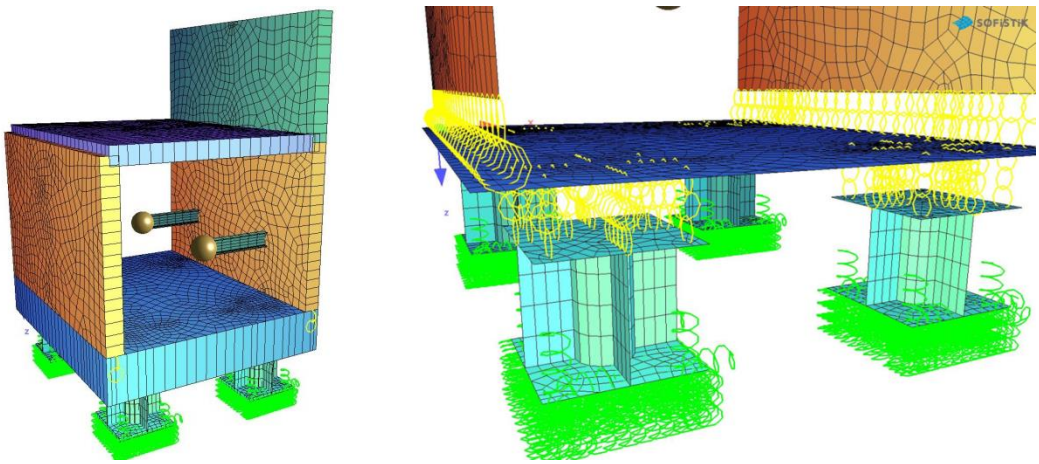

Figure 3. FE model of the mock-up (left), detail of support modelling (left).

Boundary conditions modelling

Modelling of the support structures was changed from fixed coupling to a flexible support on the test hall floor, see Figure 3. This modification pursues the objective of an adaption to the measured rigid body displacements and rotations. The ability of slippage due to the clearance holes in the base plates of the supports being larger than the diameter of the rock anchors is considered in the calibration calculations.

According to the model representation shown in Figure 4, the baseplates of the supports are nonlinearly bedded on the test hall floor without possibility of transmission of tensile forces. The springs at each node of the base plate are generated by the program. The sums of the spring stiffnesses of each base plate are specified as 10^8 kN/m in vertical and horizontal direction. The horizontal spring forces are limited by friction with a coefficient of friction of 0.5. The grouted anchorage bolts are represented by spring elements in each direction. The stiffnesses of the various springs are adjusted to the measured movements of the mock-up. The stiffness of the nonlinear vertical tension spring is 400 MN/m. The two linear horizontal springs have a stiffness of 20 MN/m.

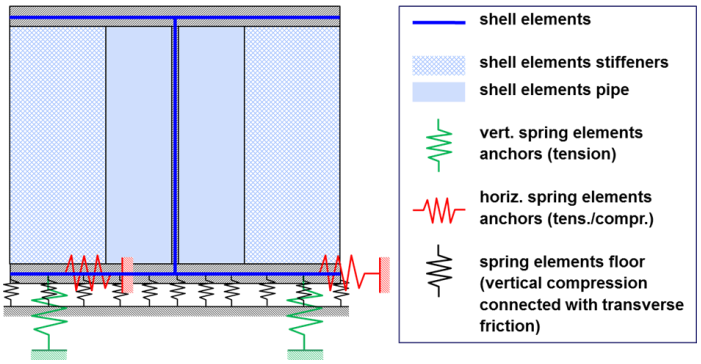

Figure 4. Model representation of the supports and the flexible couplings with the floor.

Damping

Rayleigh damping was used as damping model for all elements. The Rayleigh coefficients were calculated for 5% critical damping at 10 Hz and 1000 Hz. In the following diagram, the measured damping ratios are assigned to the related measured natural frequencies. The parameters of the damping matrix $C = \alpha \cdot M + \beta \cdot K$ fitting the measurements before and after the single tests best are $\alpha = 6.220976$ and $\beta = 0.000016$, see Figure 5.

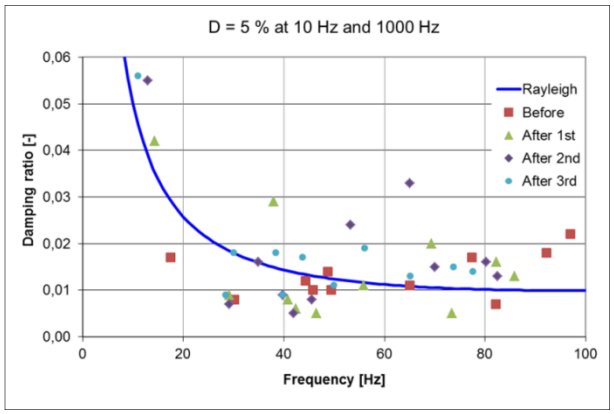

Figure 5. Rayleigh damping compared to measured damping ratios.

Load time functions

The impact loads are defined as time histories of the contact forces between the projectile and a rigid target, which were determined previously in a decoupled analysis with Abaqus FE models of the projectiles. The determination of the load time functions for the impact velocities 90 m/s and 170 m/s shown in Figure 6 is described in detail in Borgerhoff et al. (2017). For use in the calibration analyses, the amplitudes of the load time functions according to Figure 6 are scaled at the ratio of the test velocities (v_1 = 91.8 m/s, v_2 = 93.5 m/s and v_3 = 167 m/s) to the presumed velocities in order to take into account the impulse values transferred in the tests.

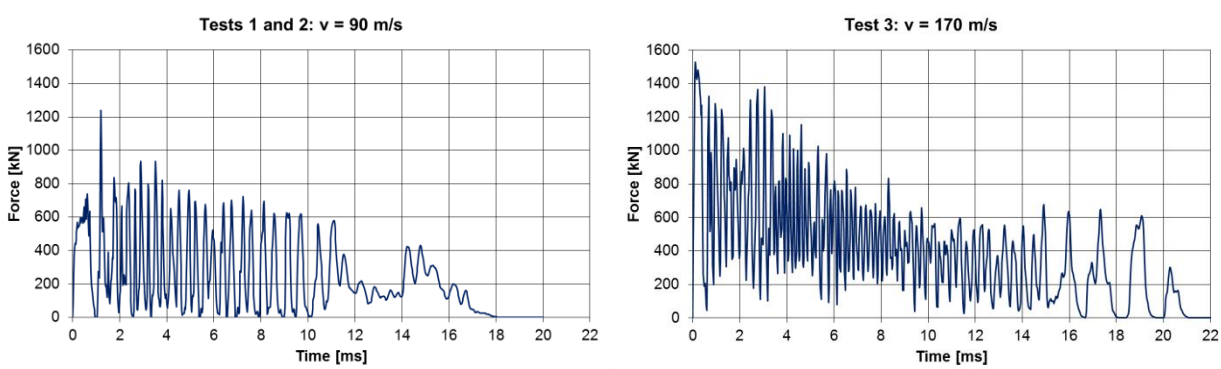

Figure 6. Calculated load time functions of tests 1 and 2 (left) and test 3 (right).

COMPARISON OF COMPUTED AND MEASURED RESULTS

The benchmark results are presented in form of diagrams containing time functions and response spectra of the computed and measured parameters. The installed monitoring system comprises displacement,

strain and acceleration sensors as well as sensors for measuring the reaction forces. Response spectra were derived from the computed and measured accelerations.

The locations of the displacement measuring devices are shown in Figure 7. The subsequent diagrams in Figure 8 illustrate the excellent numerical simulation of the measured test results in all three consecutive tests for the representative displacement sensors D01 (impact point), D3 (ceiling level), D7 (top of attic) and D10 (equipment with welded connection).

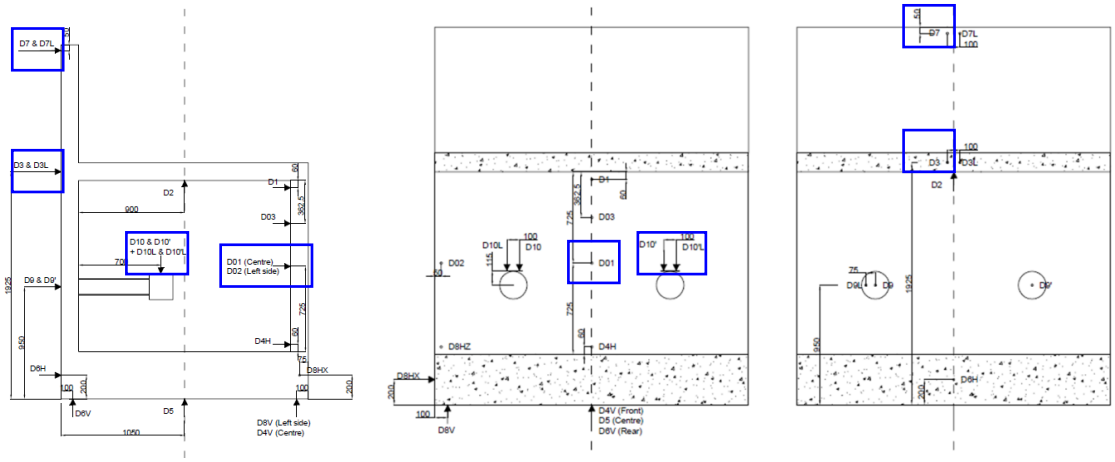

Figure 7. Displacement sensors, side view (left), front view of front wall (centre) and rear wall (right).

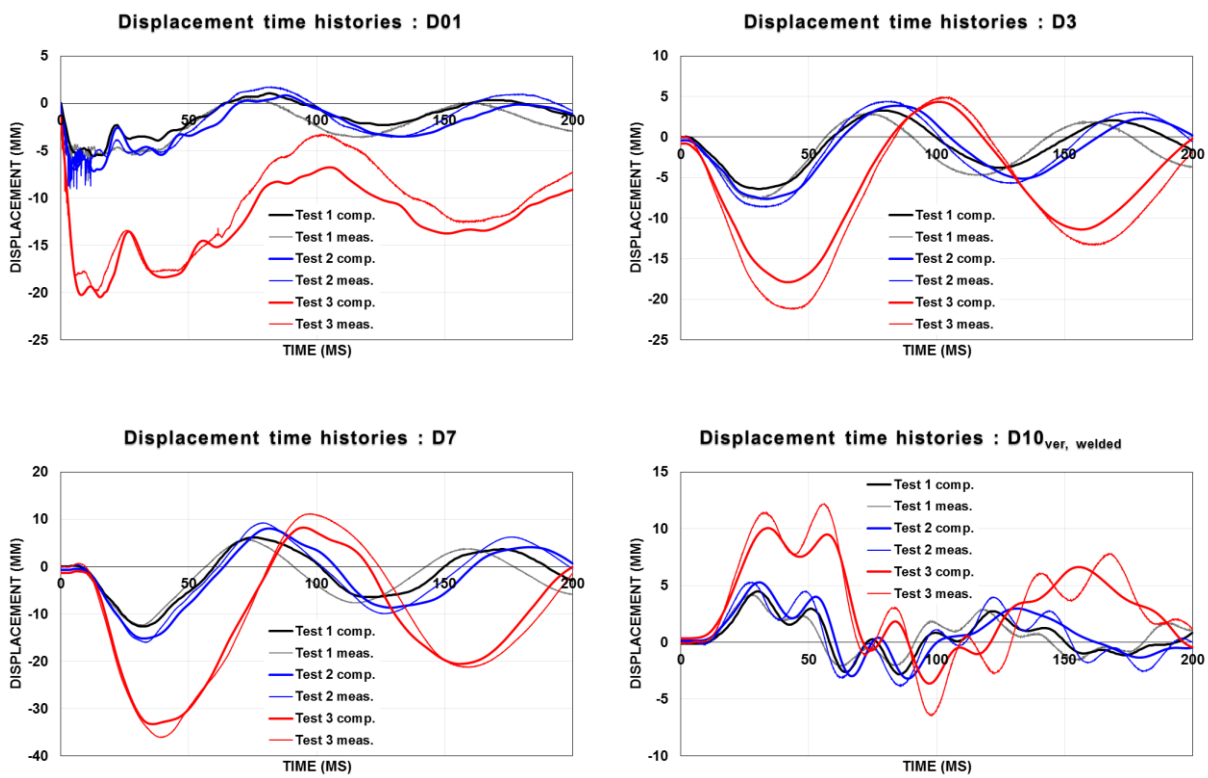

Figure 8. Horizontal displacements of sensors D01, D3 and D7, vertical displacements of sensor D10.

The locations of the acceleration measuring devices are shown in Figure 9. The presentation of the acceleration results is limited to the three selected representative accelerometers A7 at the top of the attic, $A10_{hor,\ welded}$ (equipment with welded connection, horizontally) and $A10_{ver,\ welded}$ (equipment with welded connection, vertically). The results of all three consecutive tests are shown in the diagrams in Figure 10, Figure 11 and Figure 12.

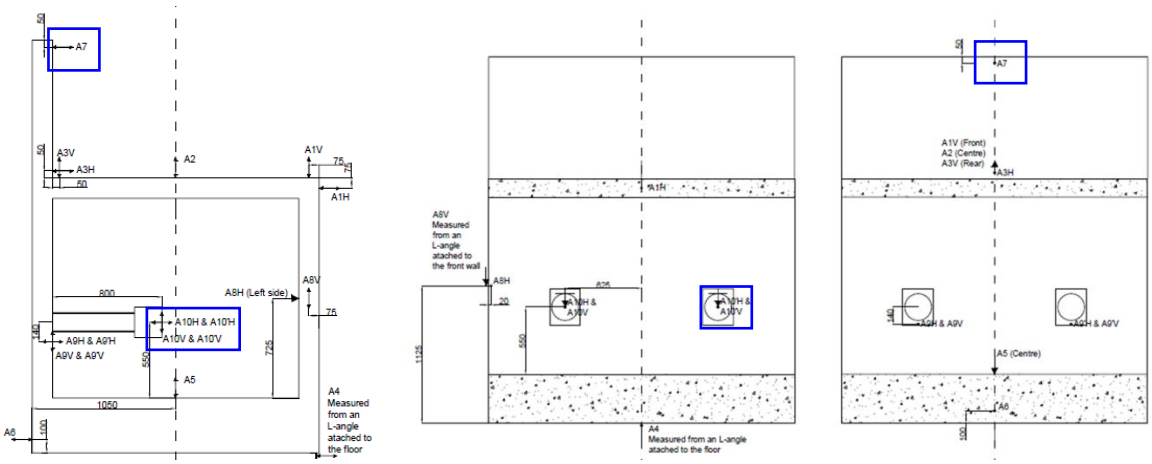

Figure 9. Acceleration sensors, side view (left), front view of front wall (centre) and rear wall (right).

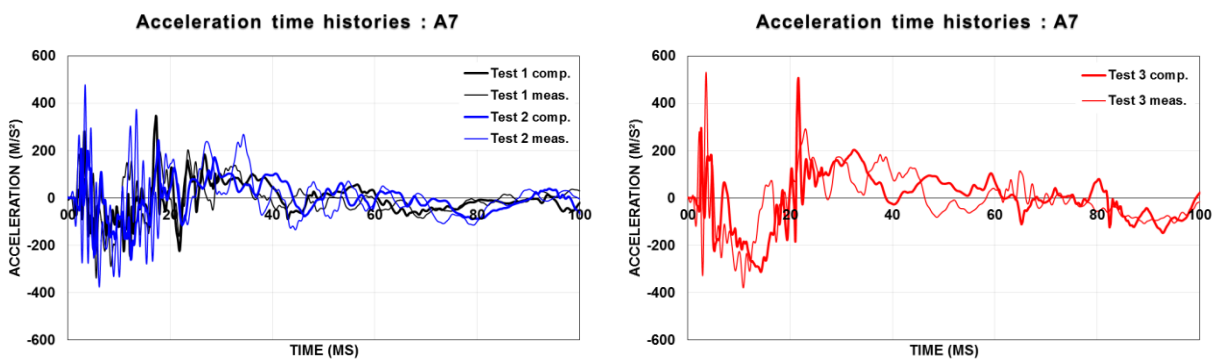

Figure 10. Horizontal accelerations at location A7, tests 1 and 2 (left) and test 3 (right).

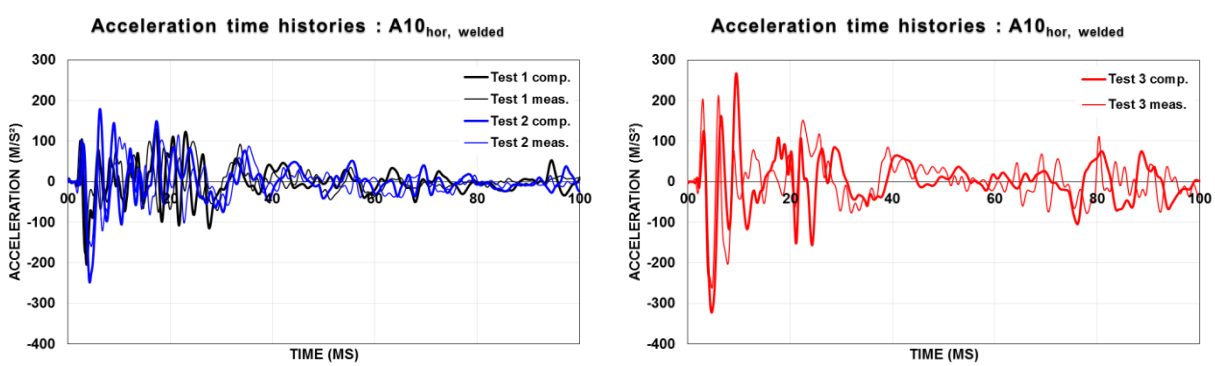

Figure 11. Horizontal accelerations at location A10, tests 1 and 2 (left) and test 3 (right).

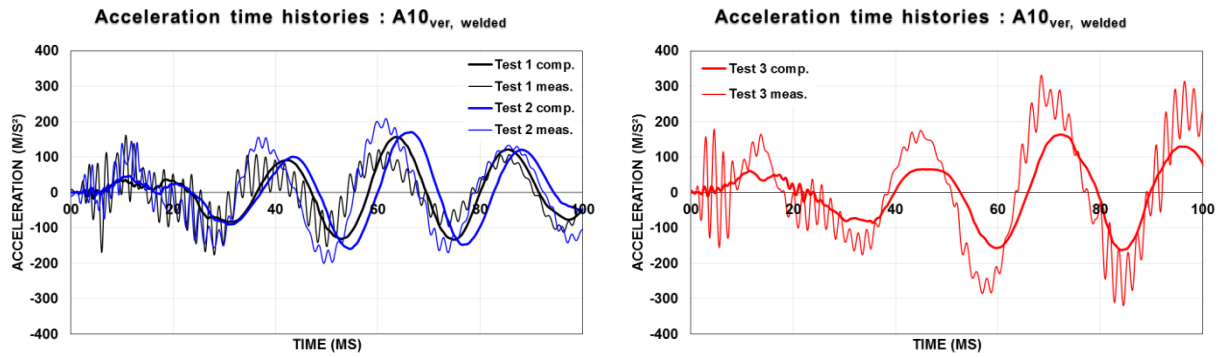

Figure 12. Vertical accelerations at location A10, tests 1 and 2 (left) and test 3 (right).

The measured acceleration histories are filtered in terms of the fraction above 1 kHz. Apart from the still present high-frequency share below this limit, the correlation between computed and measured results is reasonable. The higher frequency oscillations measured by accelerometer A10 in vertical direction are related to the natural vibration behaviour of the equipment. These oscillations are missing in the computed results due to modelling of the equipment as a concentrated rigid mass.

Acceleration and displacement response spectra associated to the measuring positions evaluated before are shown in Figure 13, Figure 14 and Figure 15. The correlation of computed and measured results is heterogeneous. The acceleration spectra at the highest level of the mock-up (A7) are closely related below 100 Hz, while computed as well as measured values at higher frequencies partly are questionable, see Figure 13. In contrast, the related displacement spectra are very well reproduced by the analyses in all tests.

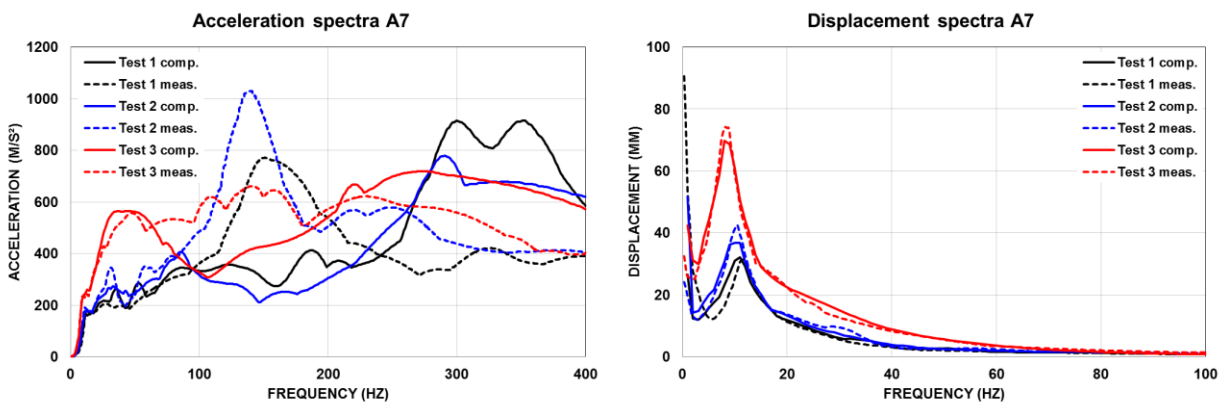

Figure 13. Horizontal acceleration (left) and displacement (right) response spectra at location A7.

The horizontal acceleration spectra of the equipment (A10) can be similarly assessed, although the numerical simulation on the whole is better even at higher frequencies, see Figure 14. However, the related displacement spectra show substantially higher measured values on a frequency range around 10 Hz. In vertical direction, measured acceleration spectra as well as displacement spectra exceed the computed results in particular on the basic frequency of roughly 40 Hz belonging to the steel beam supporting the equipment, see Figure 15. As already stated above with respect to the acceleration time histories, this deviation presumably is caused by the natural vibration behaviour of the equipment, which is not taken into account in the FE model.

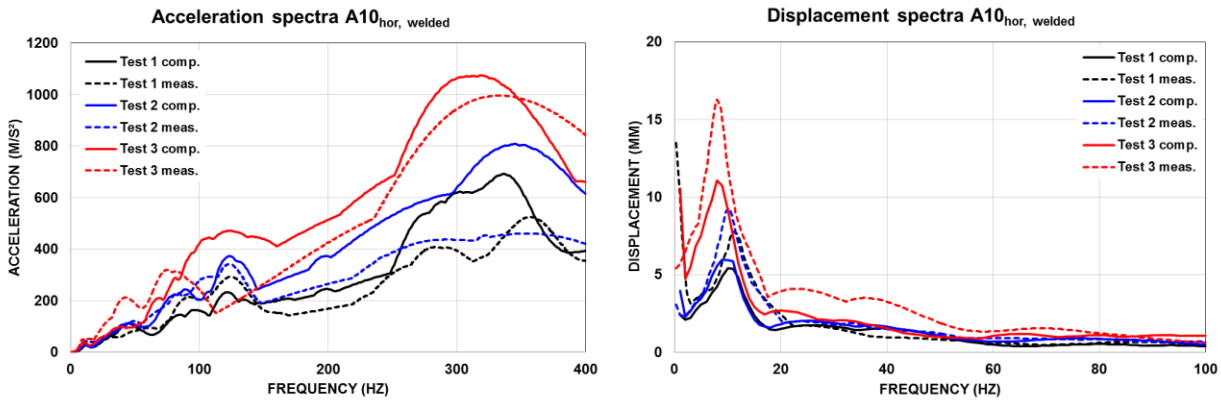

Figure 14. Horizontal acceleration (left) and displacement (right) response spectra at location A10.

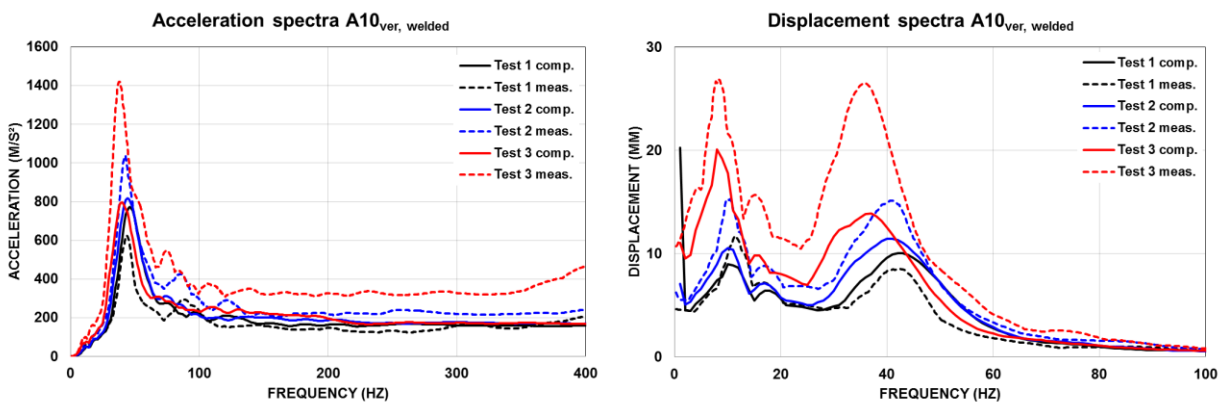

Figure 15. Vertical acceleration (left) and displacement (right) response spectra at location A10.

CONCLUSION

In phase B of the IRIS Phase 3 benchmark, significant improvements in the numerical simulation could be achieved by the modifications implemented in the numerical model for the calibration calculations. In the process of calibrating, the effects of the modifications on the FE model compared to phase A of the benchmark were investigated by a lot of parameter analyses.

The modified modelling of the connection between walls and floor slab proved to be of particular importance for the quality of simulating the vibration behaviour of the mock-up by use of a shell element model. Moreover, the successful adaption of the measured damping ratios by use of adjusted Rayleigh parameters is also worth highlighting. The good simulation of the damping behaviour in the deformation analyses moreover is an indication for the reliability of the measured damping ratios derived by the modal test data.

Overall, the results of the calibrated calculations with few exceptions show a good correlation with the measured data. Especially, the time histories of the displacements consistently agree well with each other both in amplitudes and in progression of their functions. And this agreement applies to the impact zone with minor nonlinear strains as well as for the overall structure, which behaves linearly apart from limited concrete cracking.

The best approximation of computed and measured acceleration spectra can be identified for frequencies below 100 Hz. From the associated displacement spectra becomes apparent that the extremely large accelerations due to the impact excitation are combined with very small displacement values, which are of minor significance for the structural integrity.

The calculated equipment results are also satisfactory, although both joint types (bolted and welded) were modelled equally as rigid connection, since no sufficiently good adaption of the bolted connection type to the test results could by obtained within a reasonable timeframe. The agreement of the displacement and acceleration time functions is mainly good. This assessment is similarly applicable to the acceleration and displacement response spectra in consideration of the simplified modelling of the equipment and its supporting steel beam.

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