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Seismic Investigations of the HDR Safety Program

Summary Report

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ABSTRACT

The primary objective of the seismic investigations, performed at the HDR* facility in Kahl/Main, FRG was to validate calculational methods for the seismic evaluation of nuclear-reactor systems, using experimental data from an actual nuclear plant. Using eccentric mass shaker excitation the HDR soil/structure system was tested to incipient failure, exhibiting highly nonlinear response and demonstrating that structures not seismically designed can sustain loads equivalent to a design basis earthquake (DBE). Load transmission from the structure to piping/equipment indicated significant response amplifications and shifts to higher frequencies, while the response of tanks/vessels depended mainly on their support conditions. The evaluation of various piping support configurations demonstrated that proper system design (for a given spectrum) rather than number of supports or system stiffness is important to limiting pipe stresses. Even with direct servohydraulic excitation at loads exceeding the DBE eightfold, piping still had significant margins and failure is improbable inspite of multiple support failures. The mean value for pipe damping, even under extreme loads, was found to be about 4%. Comparison of linear and nonlinear computational results with piping response measurements showed that predictions have a wide scatter and do not necessarily yield conservative responses underpredicting, in particular, peak support forces. For the soil/structure system the quality of the predictions did not depend so much on the complexity of the modeling, but rather on whether the model captured the salient features and nonlinearities of the system.

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EXECUTIVE SUMMARY

The Heissdampfreactor (HDR) Test Facility located in Kahl/Main in the Federal Republic of Germany, was used since 1974 by the Kernforschungszentrum Karlsruhe (KfK), in its HDR Safety Program (PHDR), to perform various experiments related to the design/safety of nuclear power plants. Prominent among these tests were those concerned with the seismic response of nuclear power plant structures, components and piping. The primary objective of the seismic investigations at the HDR was the validation of seismic calculational methods used in the design and safety evaluation of nuclear plants. Thus, accompanying all tests was an extensive calculational effort, performed independently by several participants from industry, consultants and research organizations.

The HDR Safety Program was a collaborative effort of many organizations in Germany, the USA, Switzerland and Great Britain. In particular, the U.S. NRC, Office of Research, under an agreement with the German Federal Ministry of Research and Technology (BMFT), collaborated in the HDR Safety Program and was extensively involved in the seismic investigations. The main contributors to the latter efforts, on behalf of the NRC, were the Argonne National Laboratory (ANL) and the Idaho National Engineering Laboratory (INEL).

The HDR reactor building has an overall height of 65 m, including the 13 m embedment, with a diameter of 22.4 m. Structurally it consists of an inner cylindrical concrete structure which is surrounded by a steel containment shell (3 cm thick), and of an outer cylindrical containment shell of lightly reinforced concrete (60 cm thick) with a hemispherical dome. The foundation of the HDR consists of a basemat and a massive cup-shaped concrete cylinder which supports the steel containment and the interior concrete structure. While the HDR building is more slender than modern power reactors, it is of comparable complexity and thus is very suitable for an investigation with the above stated objectives.

The seismic experiments conducted at the HDR between 1975 and 1988 included tests at low, medium and high levels of excitation. Since it was not possible to simulate a real earthquake excitation for the reactor building, various vibrational excitations were used to simulate different aspects of a seismic excitation. Primarily eccentric mass shakers were used to provide excitations at all levels in the frequency range of interest below 10 Hz. The capabilities of these shakers ranged from eccentricities of 147 kgm (40 kN force) to 67,000 kgm (10,000 kN force). Other means of excitation for the building included buried explosive charges and impact devices. Mechanical equipment, vessels and piping were also excited by snapback, mechanical and electrodynamic vibrators and programmable servohydraulic actuators.

The earliest low level tests on the reactor building served to define its dynamic characteristics and showed that its response is dominated by two fundamental rocking modes (at 1.52 and 1.57 Hz) and two out-of-phase bending modes between the inner and outer structure (at 2.63 and 2.81 Hz). An extensive calculational effort to predict the soil/structure system response was carried out in conjunction with these tests. The modeling ranged from simple beam models and half space theory to complex shell models and detailed finite element representation of the

soil. In general, all models substantially underpredicted the rocking frequencies and overpredicted the structural mode frequencies. It was also shown that more complex models do not necessarily yield better results.

The soil/structure system studies continued with intermediate level tests and additional calculational efforts, and culminated in the SHAG series of experiments in which the reactor building was tested up to incipient failure. A large coast-down shaker was used with startup frequencies between 1.6 and 8 Hz and eccentricities between 4,700 and 67,000 kgm. The SHAG tests attained the maximum possible loading without global failure of the reactor building, but significant local damage occurred, including cracking of concrete and failure of block walls. There was also very pronounced soil cracking and subsidence but no significant soil liquefaction or basemat uplift. Many nonlinear phenomena were observed including a nearly 40% drop in the rocking mode frequencies and an increase in the damping for the soil/structure system from 6% to 9%. Significant amplification and shifting to higher frequencies were observed in the response of equipment and piping due to nonlinear effects. The peak accelerations and displacement amplitudes at the top of the HDR building reached 0.4 g and 5 cm, respectively.

Extensive safety and predictive calculations accompanied the SHAG tests. Again, the intention was to validate various calculational procedures and their ability to capture the response nonlinearities. It was shown that linear calculations cannot adequately represent the nonlinear behavior, and that even properly formulated nonlinear models may yield poor predictions, unless the problem parameters, and especially damping, are well represented.

Various experiments were performed on vessels and tanks as part of the HDR seismic investigations. It was found that their response to a given loading is primarily dependent on the boundary and support conditions. Vessels that are well restrained and coupled to the building structure, such as the reactor pressure vessel, exhibited little independent dynamics. On the other hand free standing tanks were excited to very high response levels.

Many of the experiments and analytic studies were concerned with the seismic/vibrational response of piping. Both simple pipe runs with no supports and multi-branched piping systems with many supports were tested and analyzed. Predictions made with finite element models of the piping systems, in many instances, deviated significantly from the measured results, especially for complex piping systems and at low-level test excitations. The latter is primarily due to the nonlinear behavior of pipe restraints produced by clearances and friction which have a particularly pronounced effect at low excitation levels.

One piping system, called the VKL, was investigated at high levels of excitations. This included the SHAG tests, where it was subjected to indirect excitation from the reactor building, and the SHAM test series where it was subjected to direct simulated seismic excitation (up to 800% of SSE) by means of hydraulic actuators. These experiments were also used to evaluate the performance of a number of dynamic pipe support systems, which ranged from very stiff systems employing many struts and snubbers to a very soft system with only the minimal dead weight hangers. Also tested were snubber substitutes such as energy absorbers and seismic stops. The

results of these tests showed that the proper design of a support system (for a given spectrum) rather than the number of supports or system stiffness is important in limiting pipe stresses. The experiments also demonstrated the inherent ruggedness of piping systems. Even at loads exceeding the design basis earthquake eightfold, piping still has significant margins and failure is improbable inspite of multiple support failures. High margins were also shown for strut supports, none of which failed in the tests, inspite of being subjected up to seven times their rated capacity. On the other hand, the margins for mechanical snubbers were very variable. Some of them failed below their expected overload capacity (about 2.4 the rated load) while others were able to sustain loads up to 6 times their rated capacities.

A very careful evaluation of the damping of the piping system yielded a mean value of 4% even at load level of 400% SSE and substantial pipe plastification. This is a very significant result and contrasts the conclusions of other investigators, who have proposed much higher pipe damping values based on gross extrapolations from low level vibrational tests. These values are not confirmed by the SHAM tests.

Comparisons with linear and nonlinear computational results in both the SHAG and SHAM test series showed extensive scatter. In particular, predictions for piping system response with multiple dynamic supports, which exhibit nonlinear behavior (such as snubbers, energy absorbers, seismic stops), can be problematic. A statistical evaluation of the computational results shows that the predictions are not necessarily conservative, underpredicting, in particular, peak support forces but also accelerations. However, it is encouraging that the mean values of the design basis quantities, namely the stresses, are on the average estimated more or less correctly in the calculations. It should also be stressed that because of the inherent ruggedness of piping, the computational underpredictions of piping response are not a safety issue.

In a final test series with the VKL piping at high excitation levels (ERI), predamaged components were inserted in the pipeline. These included a straight section with a circumferential crack and a fatigue cracked elbow. Over one hundred test runs, including seismic tests up to 600% SSE and sine burst tests, were required to generate a leak in the straight section. No leak was ever developed in the elbow. This indicates that even cracked piping has substantial resistance to seismic loading, and failure in a single earthquake event is unlikely.

FOREWORD

The U.S. NRC Office of Research (RES), under an agreement with the German Federal Ministry of Research and Technology (BMFT), has extensively collaborated with the Kernforschungszentrum Karlsruhe (KfK) in the research program of the HDR Safety Project (PHDR). Argonne National Laboratory (ANL) on behalf of the U.S. NRC coordinated much of the collaboration in the area of seismic/structural investigations and participated in many of these studies.

The HDR program has provided valuable information on the vibrational/seismic behavior of structures and components. Similarly, much insight has been gained concerning the capabilities of various calculational methods and approaches. These results have direct applicability to the seismic design/evaluation of nuclear power plants. While some of this material has been presented in various conference publications, the only comprehensive overview of the results was prepared by the KfK in German ("Ergebnisse der Erdbeben-Untersuchungen des HDR-Sicherheitsprogramms," Technical Report PHDR 99-91, KfK, June 1992). To make this information more readily accessible to the U.S. technical community, NRC/RES decided to publish this material as a NUREG report, and thus directed ANL to prepare such a report in collaboration with the KfK. Except for some clarifications, the addition of some references and format changes, this report is an edited English version of the German report and faithfully represents its findings.

The report consists of three chapters, the first of which gives a general overview of the HDR seismic investigations and highlights of the results. The second and third chapters provide in depth coverage of the two most important seismic test series. In chapter two, the SHAG (Test Series T40) experiments and results are described. These tests and the associated analyses concern the high level excitation of the HDR soil/structure system. The final chapter describes the SHAM (Test Series T41) experiments and results. In these tests high level simulated seismic excitation was applied to an in-plant piping system and the performance of various piping support configurations was investigated.

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Over the past fifteen years, many colleagues from the USA, Switzerland, the United Kingdom and numerous German institutions have made significant contributions to the successful execution of the HDR seismic investigations. At the end of the project, the authors would like to thank them all for their support giving practical advice, making technical and scientific contributions, cooperating in working teams, and taking part in experiments and computer studies.

Without the dedication and willingness of all participants to make even extraordinary efforts when required, many of the objectives of the project could not have been met. In this project, teamwork was not just a metaphor, but the key to solving all encountered problems. It goes without saying that this cooperation has given rise to many friendships. Mentioning all contributors by name would exceed the framework of this report. Therefore, we would like to limit ourselves to listing the names of those who participated in many of the test campaigns:

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1 FIFTEEN YEARS OF SEISMIC TESTS AT THE HDR FACILITY

1.1 Introduction

The Heissdampfreaktor (HDR) Test Facility is located in Kahl/Main in the Federal Republic of Germany, 40 km east of Frankfurt/Main. It was built as a prototype Superheated Steam Reactor in the period of 1965 to 1969 and shut down in 1971 after only 2000 hours of operation. After extensive decommissioning and conversion work it was used since 1974 by the HDR Safety Project (PHDR) of the Kernforschungszentrum Karlsruhe (KfK) to perform vibrational/seismic, thermal hydraulic, blowdown, and other experiments related to the safety and design of nuclear power plants.

From the beginning of the HDR Safety Program, studies related to the seismic design of nuclear power plants were a major part of the overall program: in Phase I, as one of the five subprojects (EV 4000); in Phase II, with a total of three major test groups (STO, SHAG, SHAM) and one minor test group (RAU) and, finally, in Phase III, with experiments on crack propagation and fracture characteristics of predamaged pipes subjected to simulated seismic loads (ERI).

The objective of the seismic investigations conducted at the HDR was to verify the structural dynamic calculational methods for the reactor building and its interaction with the foundation soil, as well as those methods applied to mechanical systems, such as tanks and pipes. Here, the term calculational method does not only mean some computer program, but the entire process, from the problem-specific idealization of the structure, the choice of suitable boundary conditions and parameters, modeling of the idealized structure by finite elements, application of a dynamic solution approach, up to the interpretation of the results.

Such a task could not be addressed by making comparison calculations for analytically solvable academic problems or by means of laboratory tests, because in those cases, the modeling is already completely or largely predetermined. On the other hand, comparison calculations for experiments performed on a full-scale real plant, such as the HDR (Fig. 1.1), make it possible to investigate to what extent the assumptions and simplifications, which must be made in the dynamic calculations of such complex structures, are justified and where the limits of extrapolation of various modeling assumptions are.

The broad scope and objectives made it necessary that the HDR seismic investigations be performed as a joint effort of many institutions concerned with the seismic design of nuclear power plants. The KfK, on behalf of the German Federal Ministry of Research and Technology (BMFT) was responsible for project management, for planning and executing the experiments, evaluating the measured data, and for reporting. The main partners in those activities, who were responsible for the technical and scientific aspects, were the Battelle Institut, Frankfurt (between 1975 and 1982), the Fraunhofer-Institut für Betriebsfestigkeit (LBF), Darmstadt (between 1983 and 1989), and the Materialsprüfungsanstalt (MPA), Stuttgart for the experiments performed in Phase III. The comparison calculations for the experiments were always performed independently by several institutions; and the Project Management made certain to have a

balanced participation of vendors, expert consultants, engineering consultants, and research institutions. This also served the purpose of facilitating the direct transfer of the insights and conclusions from these studies into practice.

There has also been extensive participation in the HDR safety program from outside Germany and, in particular, from the U.S.A. Specifically, the U.S. NRC Office of Research, under an agreement with the BMFT, has collaborated extensively with the KfK in many of the research efforts at the HDR including the seismic investigations.

1.2 HDR Test Facility

From a structural dynamics point of view, four substructures can be distinguished in the HDR reactor building in the sectional view in Fig. 1.2, these are: the inner concrete structure, the steel containment, the outer concrete containment, and the foundation. The inner concrete structure is made up of two concentric cylinders interconnected by numerous floors and walls separating compartments for mechanical systems. In the center of the inner concrete cylinder there is a reactor pressure vessel which is 10 m high, with a 3 m inner diameter and a 14.2 cm wall thickness.

The steel containment has a wall thickness of approximately 3 cm. It encloses the outer cylinder of the internal structure, from which it is separated by a 2 cm thick styrofoam layer. The steel cylinder is 40 m high and has a hemispherical cap both at the top and at the bottom. The steel containment supports the polar crane at a height of 40 m. The materials transfer lock and two personnel entry locks are also attached to the steel containment.

The outer containment is a cylindrical shell of reinforced concrete with a hemispherical dome. Its total height is 65 m, including the embedded depth of 13 m. The outer containment has a wall thickness of 60 cm and, since the HDR was not designed to withstand airplane crashes, gas explosions or earthquakes, is only minimally reinforced. Only wind loads were considered as design basis loads acting in the horizontal direction.

The foundation of the HDR is made up of the basemat and a massive concrete cylinder which supports the internal structure and the steel containment like an egg in an egg-cup. Structurally, this is the only connection between the outer containment and the steel containment; otherwise, the two shells are independent. The annulus between them is 60 cm wide and can be entered.

The sectional view in Fig. 1.2 shows the hoist structure on the left of the reactor building, and the office/operations building on the right. Both structures can be considered to be decoupled from the reactor building. The hoist structure supports only part of the weight of the materials transfer lock on a movable bearing. The reactor building and the office building are interconnected by a double-deck bridge which has sliding supports on both sides.

The HDR plant was built on the refilled site of a former open-pit lignite mine. As a consequence, the building site had to be stabilized by subsoil consolidation before the start of

construction work. For this purpose, gravel was vibrated down in a column as far as the solid clay layer at 20 m depth. The ground water level is located at approximately -6 m. Figure 1.3 shows the result of soil tests conducted before the onset of seismic experiments in 1975.

Compared to the squat design of modern power reactors, the HDR is rather slender (about equal in height, but only approximately 40% in diameter) and is relatively deeply embedded in the soil (13 m for a foundation diameter of 22.4 m). The wall strength and reinforcement of the outer containment are significantly reduced in the HDR relative to modern reactor buildings, but the basic construction, with the decoupled outer and inner containments, is comparable. But most importantly, the complexity of the HDR is comparable to that of modern power reactors, and while the HDR was built as an experimental facility, its purpose was not the verification of structural dynamics calculations. This made the plant very suitable for a program of investigations with the above listed objectives.

1.3 Test Descriptions

As shown in Fig. 1.4, in the period between 1975 and 1988, a number of seismic/vibrational test campaigns were run at the HDR, these are:

- Seismic tests at a low excitation level, and experiments to determine the vibrational characteristics (1975 and 1982).
- Seismic tests at a medium excitation level (1979).
- Snapback tests of the reactor pressure vessel and piping at operating temperature and pressure (1983).
- Seismic tests at high excitation level:
 - Limit loads for the reactor building (SHAG, 1986).
 - Limit loads for a piping system (SHAM, 1988).
- Seismic tests with predamaged piping:
 - Crack propagation and pipe failure (ERI, 1988).

From the beginning of the investigations it was clear, that it was not possible to simulate an earthquake for the reactor building. In an earthquake, the reactor building is excited at the base by the seismic waves passing through the ground. While the excitation has a noise-like characteristic, its dominant frequency range is 1 to 10 Hz, and the strong seismic phase lasts only a few seconds. The response vibrations of the reactor building in turn constitute the base excitation for the mechanical equipment. Its frequency content and amplitude depend on the dynamic behavior of the building.

In the HDR tests, various methods of vibrational excitation were used to simulate different characteristics of seismic excitation. Mainly, mechanical eccentric mass vibrators, so-called shakers, were employed, because, at least in principle, the activation of various eccentric masses in the frequency range of interest below 10 Hz, permits the attainment of any desired excitation

level. Figure 1.5 is a plot of the excitation forces generated by the shakers as a function frequency. Figures 1.6 and 1.7 show two of the shakers used. Figure 1.7 also illustrates the mode of operation of the large ANCO coastdown Mk.16 shaker, with a peak force of more than 10 MN. The sequence is as follows: (1) the two shaker arms in the balanced position at the startup frequency chosen for the test run, (2) the closing of the arms after unlocking of a restraint, and (3) the instant at which the two arms become coupled, thus forming the preset eccentric mass of the system.

In addition to mechanical shakers, electrodynamic vibrators were used; also blast tests, snapback test, and tests with impulse-type loading were performed, see Fig. 1.4. For the VKL-piping, servohydraulic actuators were used to simulate earthquake time-histories with amplitudes up to eight times those of a safe shutdown earthquake.

As the HDR program progressed an ever more elaborate data acquisition and analysis system was installed at the facility. Ultimately, 400 fast (4 kHz) and 200 slow (2.5 Hz) measuring channels could be sampled simultaneously. The data acquisition system was connected through a data link with a data base at the PHDR/KfK in Karlsruhe. The data base also contained the results of calculations, which were performed in conjunction with all experiments. This provided a sound basis for data evaluation and for the verification of computer codes as well as of the mathematical modeling/assumptions.

Some 25 institutions in Germany, the USA, Switzerland, and the United Kingdom participated actively in the HDR seismic tests, partly in the performance of the experiments, but mainly by contributing precalculations and post-test calculations for comparison with the experiments. The test results were published in some 30 technical reports by the KfK. Since 1977, the studies have been reported regularly at all SMiRT (Structural Mechanics in Reactor Technology) conferences and at the annual status seminars of the HDR Safety Program (see Fig. 1.4).

1.4 Selected Results

This retrospective of fifteen years of seismic studies at the HDR can only provide a rough overview of the tests and the results that were obtained. The selected examples refer to the reactor building, the pressure vessel, a thin-walled vessel, and pipes.

1.4.1 Reactor Building

In the tests conducted at a low excitation level in 1975, the HDR was excited by explosive charges buried in the ground, and by shakers. The largest shaker available at that time, Mk.14 by ANCO, had a rotating arm with an eccentricity of 147 kgm, and produced a rotatory excitation force of up to 40 kN in the frequency range up to about 3 Hz (Fig. 1.6). The shaker tests consisted of sweeps (changing frequencies) for the identification of eigen frequencies and test runs with steady-state excitation frequencies close to the eigen frequencies. To determine the eigenmodes it was necessary to repeatedly reposition the few available acceleration sensors while keeping the excitation constant. The fundamental modes of the HDR reactor building were determined to be

two rocking modes at 1.52 and 1.57 Hz, and two structural modes at 2.63 and 2.81 Hz. At those frequencies, the internal structure and the steel containment vibrate out of phase relative to the outer shell. They were characterized as first bending vibration modes, although a considerable part of the deformations is caused by the flexibility of the foundation in which the two parts of the structure are coupled.

Six different institutions were commissioned to develop ten different computer models for the reactor building and the foundation soil. The foundation soil models included soil springs based on the half-space theory, semi-finite elements, and conventional finite element (FE) models, see Fig. 1.8. The simplest beam model of the reactor building had 42 dynamic degrees of freedom, while the most complicated FE shell model had nearly 10,000. The latter was used to compare the performance of the so-called static condensation, a mathematical procedure of simplifying a complex FE model, with the customary method, i.e., engineering judgment and choice of an appropriate idealization. The number of degrees of freedom in this model was reduced in two steps to 66, i.e. to a level comparable to that of the simplest beam model.

All modeling and all eigenmode calculations were carried out "blind," i.e. without any knowledge of test data, but using the original blueprints of the HDR building and the results of dynamic soil studies (see Fig. 1.3). Figure 1.9 is a comparison of calculated eigenmodes with the frequencies determined in the tests. It is obvious that the frequency of the rocking mode calculated in most models is too low. For the beam models, the deviation is approximately 30 - 40%, irrespective of the soil model selected. The variance of the frequencies calculated in the more complex models 6, 7, 8 shows that more complex tools do not necessarily guarantee better results. In the model with the highest resolution, the eigenmodes were barely comparable with those determined experimentally. The frequencies of the bending modes were overestimated by all beam models, because the rigid coupling of the inner and outer structures of these models does not correspond to reality. The flexibility of the foundation is better reflected in a shell model.

The tests run at a medium level of a excitation in 1979 were carried out with measurement and evaluation systems vastly improved over those used in 1975. At this point in time, it was also possible to run tests for vessels and pipes under operating conditions. The excitation forces in the shaker tests were increased by approximately a factor of twelve compared to the earlier tests. The shaker test runs consisted of computer controlled incremental step tests in various frequency ranges. At each frequency step, the shaker was run at a constant frequency until all transient signals had died down and the steady-state vibration could be recorded by a digital measurement system. Signals from 98 accelerometers on the building, vessels, and piping were recorded simultaneously over several vibration cycles and immediately reduced to the amplitude and phase information. After each test run, these values were normalized with the shaker force and analyzed with respect to eigenmodes, damping levels, and mode shapes.

The comparison calculations and the analytical studies of this test series, dealt with specific problems such as soil-structure interaction, damping dependency on load, or the applicability of simplified methods of calculation. These problems had been identified as priority subjects by an expert working group.

For the tests runs at a high excitation level (SHAG), performed in 1986 for the purpose of determining the safety margins of the reactor building, ANCO designed and built a very large shaker. In support of this effort, LBF performed numerous functional and design basis calculations. As far as is known, this shaker is the world's largest eccentric mass vibrator. Its basic operating principle is first to accelerate the two arms, with the eccentric masses in the balanced condition, to a preset start-up frequency which depends on the loading of the arms. Immediately after the drive system has been decoupled, an explosive bolt is fired which up to then had restrained the arms in the extended position. The so-called movable arm, pivoted in an eccentric bearing on the axis of rotation, swings around and is then coupled with the second (fixed) arm, thus forming a large eccentric mass (see Fig. 1.7). The shaker then coasts down, with decreasing frequency, sweeps through the resonance of the building and, in this way, transmits its energy to the building. Eccentric masses between 4 and 25 tons were attached to each of the two shaker arms for startup frequencies between 8 and 1.6 Hz, and eccentricities between 4,700 and 67,000 kgm. The maximum force was in excess of 10,000 kN.

In accordance with their objectives, the SHAG tests were to attain the maximum possible loading at which no global failures occur in the HDR-Reactor building. Local damage was expressly permitted. This objective was attained without taking full advantage of the design capacity-margins of the shaker. The extensive, detailed safety calculations conducted before the onset of the tests, and the immediate evaluation of the safety measurements during the tests, made it possible to exceed the computed permissible loads of individual structural components by factors of up to 10, compared to design basis calculations, or by factors of up to 3, compared to best estimate calculations; see Fig. 1.10.

Safety margins of this magnitude can only be expected in complex structural regions, such as the foundation region of the HDR, where the many walls and floor slabs are simplified in design calculations. Evidently, in some sense, such structures seem to behave in a fail-safe manner; i.e., when an element fails, the load is redistributed to another one. Simple structures, such as the outer shell of the HDR, have lower safety margins. It was also the outer shell which ultimately restricted the test loads during the SHAG tests at a level only slightly above its calculated (best estimate) limit; see Fig. 1.10.

In the tests conducted at the highest loading of the HDR building, pronounced non-linear behavior was observed. The rocking frequency dropped to approximately 1 Hz (which is about 66% of the value at low excitation) as a result of the pronounced reduction in shear stiffness of the soil; see Fig. 1.11. In parallel, damping increased greatly as a result of an increase in hysteretic soil damping which is superimposed upon the load-independent radiation damping; Fig. 1.12. This behavior was quite correctly predicted in some calculations.

As a consequence of the tests, concentric cracks were observed in the soil as well as some subsidence. Soil liquefaction occurred only to a very small extent, far below the hazard limit. Permanent settling of the HDR as a result of the tests amounts to 3 to 11 mm. Pounding was observed between the hoist structure and the reactor building. The bridge connecting to the office building was displaced slightly.

The excitation level in the SHAG tests was comparable to intensity 7 or 8 on the modified Mercalli scale. Consequently, the HDR reactor building withstood loads on the order of the severest earthquakes occurring in Germany, although it had never been designed for earthquakes.

1.4.2 Reactor Pressure Vessel

The dynamic behavior of the reactor pressure vessel (RPV) was investigated during the shaker and snapback tests performed at a medium level of excitation in 1979 and 1983. The "blind" pretest and comparison calculations using FE models all underestimated the frequency of the first bending mode of the RPV. The reason for this was the bellows expansion joint, which seals the annulus around the RPV against the flood water during refueling, and which unexpectedly exerted a stiffening effect in the horizontal direction. Figure 1.13 shows that this expansion joint is also responsible for the very different vibration behavior of the RPV in the cold and in the hot state. Due to thermal expansion with increasing temperature, the gap, between the components of the expansion joint which are firmly connected to the vessel and those components attached to the building wall, shrinks or is locally completely eliminated. This increases the horizontal stiffness and, since the cylindrical parts are not perfectly round, the damping effect is enhanced by friction. Damping values of about 40% were determined from the data measured in the snapback tests at operating temperature, i.e. the vibrations were damped out within one or two cycles.

1.4.3 Flood Water Tank

In contrast to the RPV, the investigated flood water tank, which is located in the auxiliary building of the HDR, is a thin walled tank. For the shaker tests and snapback tests conducted in 1979 and 1982 it was two thirds filled with water. The most important result of these tests was the incredibly low damping of the first bending modes of the tank. Figure 1.14 can only give an indication of the fact that the free vibrations after a snapback almost did not die down and remained measurable even after several minutes. It should be noted that the vibration amplitudes of 3 to 4 m/s², which follow the peak level of 19 m/s², correspond to strains of approximately 0.5% measured in the hemispherical bottom, where the tank is supported by an unfavorably shaped square structure which rests on four short beams.

1.4.4 Piping

Several piping systems were studied in the HDR seismic tests, see Fig. 1.4. These included very simple piping, such as a saturated steam pipe (SRL) and one of the primary steam pipe lines (PDL), as well as more complex, branched piping systems with pipes of various diameters containing fittings, valves, pumps, and other heavy components (URL, VKL).

Computational FE models, developed only on the basis of piping drawings, in general, produced results with major differences from the measured vibration behavior, especially for complex piping systems and at low-level test excitations. To some extent, these differences can be

attributed to differences between the nominal and the actual cross sections of pipes, and partly also to the non-linear behavior of pipe restraints produced by clearances and friction. In contrast to non-linear behavior in materials, this type of non-linearity has a particularly pronounced effect at low vibration amplitudes and becomes less pronounced as the amplitude increases. As a consequence, vibration studies conducted at low excitation levels yield little information about piping subjected to higher level loads. This holds in particular, for damping values which, due to friction effects, may be significantly higher at low excitation levels than for higher excitations of the system. Since damping is normally assumed to increase with the level of excitation, this may lead to drastic underestimates of the dynamic loads and stresses acting on the pipelines.

The VKL piping system was investigated at a high level of excitation in all three test campaigns (SHAG, SHAM, ERI). The system is made up of several pipe runs with nominal pipe diameters of 100 to 300 mm. The main branches are connected with the HDU (superheated steam converter) vessel and the DF 16 manifold, see Fig. 1.15. For this piping, several participants in the investigative program (NRC, EPRI, CEGB, KWU, see Fig. 1.16) designed dynamic pipe support systems using different concepts, but each in accordance with their respective design codes. The variation in the support configurations ranged from the very stiff NRC configuration, typical of US installations, to the so-called HDR configuration. In the latter the existing hangers, which support the dead weight of the piping, were only supplemented by horizontal restraints required for test purposes without performing any seismic design calculation. The investigations also included the replacement of typical snubbers (dynamically rigid, but statically flexible supports) with alternate supports such as energy absorbers and seismic stops; see Fig. 1.16.

The DF 16 manifold is located approximately 3 m below the 30 m operating floor of the HDR, where the large shaker used in the SHAG tests was mounted. Three shaker tests with different eccentric masses and startup frequencies of 8, 5.6, and 4 Hz provided indirect (base) excitation of the VKL test loop that was somewhat higher than the given HDR safe shutdown earthquake (SSE) spectrum, see Fig. 1.17. In the SHAM and ERI tests conducted in 1988, the VKL pipe was excited by two servohydraulic actuators acting in parallel at the DF 16 manifold and the H 5 hanger position; see Fig. 1.15. An earthquake time-history generated artificially from the given SSE spectrum was used as the excitation time function. It had a duration of 15 s and a maximum acceleration of 6 m/s^2 (for 100% SSE). Figure 1.17 shows the excitation spectra for the six investigated support configurations obtained with this time function.

To determine the safety margins of the piping, the excitation was raised considerably above the design load level in the course of the SHAM tests. The HDR configuration, which had not been designed for earthquakes, and which had only two horizontal restraints at the points of load application in order to limit transverse forces acting on the hydraulic cylinders, was subjected to 200% SSE, while the other configurations were loaded to at least 300% SSE. In two configurations, the flexible KWU system and the very rigid NRC system, the load was increased stepwise up to 800% SSE, i.e., up to the performance limit of the excitation system.

At the most highly stressed points of the piping, strain amplitudes of 1.5% were measured, while the residual strain amounted to 1.2%. For a Level D design of pipelines in nuclear plants

the permissible stresses for the austenitic material used here correspond to strain amplitudes of 0.44%. This level was exceeded at one point approximately a 100 times by a factor of up to 3; see Fig. 1.18. The fatigue curve of the material shown on the same diagram indicates that the actual capacity under cyclic loading is clearly higher. The allowable stresses for seismic loading conditions can be tolerated by the material for approximately 10,000 cycles. The stress limit at Level D is not based on the material resistance to cyclic loading, i.e., the fatigue curve, but on values that assure no global plastic failure under static loads. The application of these limits to seismic loading consisting of a few brief load peaks, is indeed very conservative.

The dependence of damping values on the level of excitation was studied in the SHAM tests by means of parameter variation calculations. The modal damping values of the employed computer model were changed individually until the response spectra at various locations in the piping agreed with the corresponding measured data. The results of this study are compared in Fig. 1.19 with the damping levels prescribed by the KTA codes (German nuclear codes), as well as new levels that are currently under discussion. Based on the experimental results, it can be said that the relationship between the modal stress and modal damping is not as unambiguous as is frequently assumed. For one of the modes in the piping (mode 2) even at load increases to 400 % SSE, no increase in modal damping was observed although the stress limits for Level D were exceeded.

In the light of the results of the SHAM tests, the present KTA damping values may be considered to be rather conservative. The application of the proposed values of 4% (KTA draft) or 5% (PVRC recommendation) appears feasible, provided one accepts that in one of the design steps, namely in the calculation of the vibrational response to a given excitation, one should not necessarily expect conservative results, but rather mean values.

For the ERI tests, which dealt with crack propagation and failure behavior of pipes under seismic loads, a straight pipe section and an elbow made of high-toughness ferritic 15 MnNi 63 material were welded into pipe sections A and B of the VKL test loop which had experienced the highest loading; see Fig. 1.15. Both components had been predamaged by fatigue tests in the laboratory. The straight pipe section contained a crack in the circumferential direction with a crack angle of 60° and a ratio of crack depth to wall thickness of $a/t = 0.4$. In the elbow, fatigue tests produced a crack field that spread over the entire length of the flanks, with crack depths up to $a/t = 0.17$. The ERI tests were conducted with pressurized water conditions (240 °C, 70 bar). The time variation of the nominal bending stresses at point A is shown, in a condensed form, in Fig. 1.20. A total of 58 seismic tests at 100, 400, and 600% SSE, and 50 tests with sinusoidal load (sine burst tests conducted to accelerate crack propagation) were necessary to drive the crack at this point up to leak development. Figure 1.21 shows that the crack angle of 60° did not change in the process.

The predamaged bend did not fail in the tests, not even in a subsequent second test series again with 108 tests at excitation levels of up to 800 % SSE. Fatigue-induced failures at other points in the pipe lead to the termination of the second test series. The results of these studies indicate that failure under seismic loads of piping made out of tough materials is extremely unlikely.

Fig. 1.1 HDR test facility

Fig. 1.2 Cross-section of the HDR building

Fig. 1.3 Shear moduli of the HDR site

TEST SERIES (Year)	EXCITATION	STRUCTURE	PEAK RESPONSE Acc. (Strain)	SMIRT-PAPERS
Low Level Tests (1975)	Ecc. Mass Shaker 40 kN Explosives 10 kg Snapback 25 kN	Reactor Building Steam Generator Recirculation Pipe URL Saturated Steam Pipe SRL	0.6 m/s ² 0.7 m/s ² 0.5 m/s ² (0.05 ‰)	SMIRT 4 (1977) K8/5, K8/6 SMIRT 5 (1979) K13/2, K13/3
Modal Vibration Tests (1982)	Servohydr. Shaker 13.3 kN El.-Dyn. Shaker 0.45 kN	Steel Containment Flood Water Storage Tank		SMIRT 7 (1983) K15/4, K15/5
Moderate Level Tests (1979)	Ecc. Mass Shaker 500 kN Explosives 5 kg Snapback 220 kN Rockets 400 kN	Reactor Building Reactor Pressure Vessel Flood Water Storage Tank Recirculation Pipe URL Primary Steam Pipe PDL	0.8 m/s ² 4 m/s ² 16 m/s ² (5‰) 16 m/s ² (0.55‰) 51 m/s ² (0.5‰)	SMIRT 6 (1981) K14/1 to 5 SMIRT 7 (1983) K2/8 to 12, K4/7, K9/7, K9/8, K12/12
Hot Snapback Tests (1983)	Snapback 400 kN	Reactor Pressure Vessel Primary Steam Pipe PDL	25 m/s ² 171 m/s ² (1.2‰)	SMIRT 8 (1985) K19/7
Seismic Margin Tests Building (1986)	Ecc. Mass 10,000 kN Coast-Down Shaker	Reactor Building Test Pipe Loop VKL	4 m/s ² (50 mm) 75 m/s ² (0.4‰)	SMIRT 8 (1985) K21/4, K21/5 SMIRT 9 (1987) K9/1, K9/2, K9/3, K14/5, K14/7, Sess.JK1 SMIRT 10 (1989) K32/1
Seismic Margin Tests Piping (1988)	Servohydraulic Actuators 800 kN	Test Pipe Loop VKL	395 m/s ² (9.12‰)	SMIRT 10 (1989) K 28/1, K28/2 SMIRT 11 (1991) K32/2, K32/3
Pipe Failure Tests (1988)	Servohydraulic Actuators 800 kN	Test Pipe Loop VKL with Predamaged Segments	300 m/s ² (6.5‰)	SMIRT 10 (1989) G 13(F)/1

Fig. 1.4 Vibration/earthquake test program at the HDR

Fig. 1.5 Shaker force and frequency ranges

Fig. 1.6 Eccentric mass shaker MK 14 (1975)

Fig. 1.7 MK 16 coast-down eccentric mass shaker (1986)

Fig. 1.8 Variations in soil and reactor-building models

Fig. 1.9 Ratios of calculated to measured natural frequencies

Structural component	Design limit load	Best estimate limit load	Observations during tests
Internal concrete structure	1.37	0.67	Minor cracks in stair case (not critical)
Basement			
Intermediate floor -8m	10.0	3.00	Large cracks through floor (not critical)
Foundation slab	3.00	1.75	No cracks detected in base slab
External concrete structure			
Basement region	5.26	3.30	No cracks detected in outer wall
Cylindrical shell	1.52	1.02	Test load limiting component
Sliding of steel containment in egg - cup	1.67	1.10	No sliding measured
Basemat - uplift	0.96	0.84	No uplift observed
Soil failure	1.61	0.94	Foundation settlement 3 to 11 mm

Fig. 1.10 Demand loads of reactor building structures during the SHAG tests as multiples of calculated limit loads

Fig. 1.11 Rocking frequency versus shaker force

Fig. 1.12 Rocking mode damping versus shaker force

Fig. 1.13 Vibration behavior of reactor pressure vessel

Fig. 1.14 Vibration behavior of flood water storage tank

Hanger No.	Hanger Configuration					
	NRC 3	EPRI/SS 5	EPRI/EA 4	CEGB 6	KWU 2	HDR 1
4	Strut	Strut	Strut	Strut	Strut	Strut
23	Strut	Strut	Strut	Strut	Strut	Strut
9	Strut	Strut	Strut	Strut	Strut	Strut
10	Strut	Strut	Strut	Strut	Strut	Strut
11	Strut	Strut	Strut	Strut	Strut	Strut
8	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut
7	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut
12	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut
3	Strut	Strut	Strut	Strut	Strut	Strut
22	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut
6	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut
2	Energy Absorber	Energy Absorber	Energy Absorber	Energy Absorber	Strut	Strut

▨ Strut ▨ Energy Absorber
 ■ Snubber ▨ Seismic Stop

Fig. 1.15 VKL piping with positions of dynamic supports during SHAM tests

Fig. 1.16 Arrangement of dynamic supports for the different configurations in the SHAM tests

Fig. 1.17 Response spectra ($D = 4\%$) of the VKL excitation accelerations during the SHAG and SHAM tests

Fig. 1.18 Frequency distribution of strain ranges during the SHAM tests

Fig. 1.19 Modal damping of the VKL piping as a function of modal stress

Fig. 1.20 Bending-stresses at predamaged pipe cross section

Fig. 1.21 Crack shape of failed pipe cross section

2 TEST GROUP T40, SHAG: SHAKER EXCITATION OF THE REACTOR BUILDING

2.1 Introduction, Overview, Objectives

In Phase II of the HDR Safety Program, shaker tests at a high level of excitation (Test Group T40, SHAG) were conducted in June and July, 1986 [1]. The test group completed the series of seismic studies of the reactor building, following the tests in Phase I, at a low (V2 - V5, 1975 [2-7]) and at medium level of excitation (V63 - V65, 1979 [8-18]).

These experiments, in which the U.S. NRC Office of Research, as well as many other institutions, participated, served to investigate the vibration behavior of a full-scale reactor building, with sizable stresses generated both in the concrete and in the soil, and with the maximum possible indirect excitation of the mechanical equipment. As in all HDR studies the objective was to verify the methods of computation with the aid of test results. In this case, verification was to extend into the range of non-linear vibration behavior of the building structures and mechanical equipment. The computational methods were also to be checked for their conservatism, and the safety margins were to be quantified.

At the same time, the test results furnished direct information about the vibration response and the behavior of components, such as pipes and other mechanical equipment, at high levels of dynamic loading. This in turn contributed directly to the understanding of the behavior of such systems in nuclear plants. Hence, one major program goal was the evaluation of a variety of pipe support concepts. This was accomplished through the systematic exchange of hangers under otherwise identical test conditions. As part of an Equipment Qualification Study, an American gate valve (25 years old, but overhauled) from the Shippingport reactor was investigated under operating loads and seismic loads.

A very large shaker, which had been developed by ANCO Engineers, Culver City, USA [19], was used for the excitation. Numerous calculations were performed by LBF Darmstadt on the design and functional characteristics of the shaker [20]. This shaker (Fig. 2.1) operates by the following basic principle: In the balanced condition, its speed is raised to a certain maximum level, which depends on the respective mass loading of the shaker arms. Immediately after the drive (440 kW) has been disengaged, the coupling between the two shaker arms is severed by an explosive bolt. An eccentric mass is generated in a matter of seconds, during which the movable and the rigid arms come together and are coupled. While traversing the building resonances, as its speed (frequency) decreases, the shaker transmits its energy to the building and the mechanical equipment installed in it (Fig. 2.2).

Masses of up to 25 tons were used per shaker arm. The startup frequencies were between 8.0 and 1.6 Hz, while the eccentricities varied between 4700 and 67,000 kgm. Peak forces in excess of 10,000 kN (1000 tons) were achieved (Fig. 2.3). A total of 25 tests were performed, ten of which served to study the response of building structures and the soil, while the other fifteen were devoted

to the VKL piping system. Of these, five tests were run under pressurized water conditions (210 °C, 70 bar) [21, 22].

The primary objective of the experiments was to excite the HDR reactor building, which was not designed for earthquakes, up to the limits of its load bearing capacity; whereby local damage (Fig. 2.4) was deliberately accepted, while global failure of the structure was of course excluded. Both objectives were met, without utilizing the maximum feasible eccentricity or force of the shaker. Excitation levels between the Operating Basis and the Safe Shutdown Earthquakes (for German sites) were achieved. The intensities can be classified between 7 and 8 on the modified Mercalli scale. The peak accelerations and displacement amplitudes, in the HDR building reached levels of 0.4 g and 5 cm, respectively. Non-linear behavior was observed in the building structure and the soil. Local damage occurred in the building, such as cracks in the concrete of intermediate floors in the foundation region and in the hemispherical bottom cap as well as damage in the block walls. During the tests, which especially excited the rocking resonance mode of the building, cracks (concentric with the building, see Fig. 2.5) as well as subsidences (approximately 10 cm) occurred in the soil. There were also impacts between the reactor building and the hoist structure and between the connecting bridge and the operations building. The strains in the outer containment reached or exceeded permissible limits.

For a better understanding of the loads reached in the tests CERRAM undertook a computational comparison of the most powerful shaker runs with a close-range earthquake in the Friaul region (24 km distance from the epicenter, 3 Hz central frequency), which represents an extreme seismic load for Central European conditions [23]. It was found that the loads and stresses, respectively, acting on the HDR building in the SHAG tests were up to 4.8 times (cross sectional moments for the outer structure) higher than those produced by the earthquake mentioned above (Fig. 2.6).

The VKL piping system was studied with up to seven different support configurations. They ranged from very stiff systems with snubbers and struts, which are typical of US plants, to very soft systems, which, aside from some struts, contained mainly spring- and constant-force hangers and are typical of conventional plants. As a substitute for snubbers, also energy absorbers (plastic dampers), seismic stops (displacement limiters) and visco-elastic dampers were used [21, 22]. The stresses in the VKL test loop which resulted from the seismic loads remained in the linear elastic range for all hanger configurations (even for the HDR configuration which was not designed for earthquakes and, hence, was very flexible).

Design calculations, safety calculations, pretest calculations, and blind comparison calculations were performed by a number of institutions in order to verify the analytical techniques and to quantify the safety margins inherent in the design methods. It was found that the calculations were in part very conservative; e.g., for the free field accelerations some of the measured values were more than a factor of 3 lower than the precalculated levels. The measured forces and stresses acting on the building structure also were clearly (approximately a factor of 3) lower than those calculated. For the piping system, which had been excited indirectly, i.e., via the

vibrations of the building structure, no satisfactory agreement was found between measurements and calculations.

In-depth analyses and assessments were performed by experts in the respective fields as part of focused evaluations. Starting with the action of the shaker on the 30 m platform of the reactor building, these evaluations can be linked into a cause-and-effect chain as follows:

- Loading of the reactor building structures.
- Damping and resonance frequencies of the building structures.
- Load or vibration transmission, inside the building and into the mechanical components.
- Safety margin relative to basemat uplift.
- Safety margin relative to soil liquefaction and pore water pressure.
- Vibrations in the free field.
- Assessment of piping supports.

Added to this are efforts to investigate the capabilities of various computational methods, such as

- simplified non-linear calculations of the vibration behavior and of the loads and stresses acting on the building;
- application of methods of probabilistic structural mechanics;
- calculation of the vibration behavior and of the loads and stresses acting on piping.

2.2 Summary of Evaluations for the Reactor Building, the Soil, and the Free Field

2.2.1 Building Loads and Stresses

The HDR reactor building was erected in 1965 to 1968. It is statically designed mainly for permanent (dead) loads and operational (life) loads. In accordance with the then existing state of technology, special load cases resulting from external effects, such as earthquakes, were not taken into account. Only wind loads were included as major horizontal loads. Consequently, the structure is not very strongly reinforced, i.e., even some of the regions important for the transmission of the horizontal forces contain only the minimum of construction reinforcement. Hence, prior to the shaker tests, in which the reactor building was to be excited to the limits of its load bearing capacity, safety investigations were performed by HOCHTIEF [24] and Zerna, Schnellenbach and Partners [25]. These calculations indicated that only modest shaker eccentricities were permissible (on the order of 10,000 kgm). Therefore, an overall experimental approach was developed which consisted of computational safety studies, measurements accompanying the tests and evaluations/assessments during the test campaign, i.e. after each individual test.

Because of its complicated geometry, the interior structure of the building is not really amenable to a detailed static calculation. Normally, in a conventional static design for horizontal loading, all loads and stresses would be assigned to the inner-most cylinder, because only there is a statically determined load transmission possible. However, strain measurements during the commissioning tests of the shaker showed that the inner cylinder carries only 10-15% of the total moment. The outer cylinder, on the other hand, carries about 15-30%, i.e., the walls and the steel containment, which are normally not considered in a static calculation, carry some 60% of the loads. Taking account of this load distribution it seemed possible, that as far as the load bearing capacity of the interior structure is concerned, tests with eccentricities up to 100,000 kgm could be run.

The safety calculations indicated that the foundation region is particularly critical. Consequently, strain gages were applied to the basemat of the HDR in the preliminary shakedown runs of the shaker in January 1986. The results of these measurements showed that the pretest calculations had been performed with oversimplified computer models. Therefore, HOCHTIEF developed more detailed models, which took into account the partition walls and floor slabs in the foundation region. Based on detailed analyses, it was shown that the basemat would now no longer be the critical test-limiting element. Instead, it was found that the outer structure contained the ultimate weak regions which needed to be kept under special surveillance during the tests. Specifically, these are the regions where intermediate slabs and floors are attached to it, and the region where the outer structure is restrained by its embedment into the ground. Therefore, an intensive measurement program was developed for these ultimately critical regions. In order to determine the relationship between the measured strain and the resultant stress, arising from the combination of a dead load and test load, numerous samples were taken from the endangered regions. From these the MPA Stuttgart determined the tensile rupture strength and the elastic modulus of the concrete.

The highest load acting on the outer shell in the shaker tests arises from vertical membrane forces in the bottom region of the shell between 0 and -11 m. That part of the building has only the minimum structural reinforcement (two R188 reinforcement mats). The tensile forces which can be sustained by the reinforcement are lower than the tensile stresses which can be accommodated by the concrete. Therefore, the highest permissible membrane tensile stress was set at 0.5 MN/m^2 based on the as determined tensile strength of the concrete. In defining that level, credit could be taken for the compression stress of 1.1 MN/m^2 generated by the dead weight of the shell. This maximum permissible membrane tensile stress was exceeded only slightly in the tests, although the shaker eccentricity was raised up to 67,000 kgm.

During the tests, at one location at the 0 m level, considerable bending contributions were observed in the shell in addition to the membrane forces. These were caused by the foundation of the hoist structure pressing against the outer shell (Fig. 2.5). After an approximately 10 cm wide gap had been excavated between the foundation of the hoist structure and the outer shell, these bending contributions disappeared. Although designed in the horizontal direction only for wind loads, the HDR reactor building tolerated horizontal accelerations of 0.4 g at the top of the building

without suffering major damage. This test load is equivalent to a seismic excitation of an intensity of 7-8 [26].

2.2.2 Structural Damping and Resonance Frequencies

Modal parameters of the building vibrations, i.e. basically resonance frequencies and the associated damping values, were determined by LBF from the measured results at the acceleration measurement locations. For the rocking modes of the reactor building, tests with shaker eccentricities between 4700 and 67,000 kgm were evaluated, while for the out-of-phase bending modes of the outer and inner structures, tests with eccentricities of 4700 and 8200 kgm were evaluated.

It was very difficult to identify the modal parameters because of the rotary excitation produced by the shaker, and also because both the rocking and bending modes always appear as a pair of closely spaced modes. Therefore, a complex exponential technique designed for multi-degrees of freedom systems was employed. In this technique the least squares method is used to determine a set of modal parameters which produces the optimum agreement between measured and identified transfer functions, simultaneously for all measurement points.

For the two out-of-phase bending oscillations of the inner and outer structures of the reactor building, the frequencies dropped from 2.32 to 2.23 Hz and from 2.41 to 2.33 Hz, respectively, when the load (eccentricity) increased from 4700 to 8200 kgm. The associated damping values increased from 2.7 to 3.5% and from 4.6 to 6%, respectively. Most of this modal damping was contributed by structural damping in the concrete structures of the foundation, where the inner structure and the outer shell are connected (Fig. 2.8).

The two rocking modes of the HDR reactor building are not simple rigid body vibrations of the rigid building on the soil, but also, to some extent, in-phase bending modes of the inner and outer structures. Therefore, both structural damping in the concrete as well as radiation damping and hysteretic damping in the soil contribute to their modal damping, which increased from 6% to 9% and from 5.2% to 8.8%, respectively, when the shaker eccentricity was raised from 4700 kgm to 67,000 kgm. For this load increase, the corresponding resonance frequencies dropped from 1.34 Hz to 0.98 Hz, and from 1.38 Hz to 1.1 Hz, respectively. This drop is mainly due to the decreasing shear stiffness of the soil under large shear deformations, which leads to a decrease in the integral value of the rocking spring stiffness to roughly one half of the original value (Fig. 2.8). Taking into account the results of earlier tests at low and medium levels of excitation, and plotting the values against the logarithm of the shaker force at resonance passage (see Fig. 1.12), the similarity with the corresponding curves for the shear modulus and the soil damping as a function of shear deformation becomes obvious [27, 28].

2.2.3 Load/Vibration Transmission

The stated objectives of the SHAG Test Program include investigations on the transfer of vibration energy from the shaker inside the building to the piping and large components (crane,

materials transfer lock, reactor pressure vessel), as well as to other parts of the building (foundation, outer structure), and to adjacent buildings (hoist structure, operations building). A significant number of measuring sensors was installed to address this aspect. The evaluation of the corresponding signals was undertaken by LBF for two tests (8200 kgm eccentricity, 4.5 Hz startup frequency; and 67,000 kgm eccentricity, 1.6 Hz startup frequency).

The evaluation showed that, inspite of the low excitation frequencies, vibrations at higher frequencies occurred at many measurement locations. These are due to non-linearities, mainly occurring as a result of collision/impact processes which cannot be handled with linear computational models. For instance, in the VKL piping, which is located in a compartment immediately below the operating floor on which the shaker is located, eigenmodes were still excited at 10-12 Hz. The vibration velocities of the VKL after the onset of the test were approximately a factor of 20 higher than those of the inner structure. One reason for this is that the piping is not only supported by the walls of the building, but is also connected to a large vessel, the HDU. The HDU vibrations also included strong participations in the frequency range above the startup frequency of the shaker. The latter had been excited by non-linear mechanisms at the HDU bottom support and its central restraint. Thus, the building-HDU-VKL chain may give rise to a twofold amplification of vibration responses.

On the other hand, the reactor pressure vessel of the HDR did not exhibit any characteristic dynamics of its own. This is probably due to its connection to the reactor building by means of a bellow expansion joint in the upper vessel area, slightly below the top flange. Already tests in Phase I had indicated that this expansion joint allows thermally induced axial displacements but, because of its construction, has a very high radial stiffness [17].

When the polar crane in the steel dome of the reactor building passed through a resonance, its response was a factor of 3-4 higher than that of the building. Based on the acceleration measurements, no slippage of the crane could be documented. However, from control measurements of the positions of the bridge and the trolley of the polar crane before and after the test, it can be deduced that the bridge was displaced by up to 23 mm and the trolley by up to 7 mm during one test. No dependence of the resultant shifts on the level of excitation was identified. While passing through an eigenmode the materials lock, which is welded to the steel containment, showed a vibrations response approximately a factor of two higher than that of the inner structure. The hoist structure, where the materials lock rests on a sliding support, does not appear to be excited through this bearing, but rather through the soil. The largest vibration amplitudes of the hoist structure were encountered when the rocking resonance of the reactor building was traversed. These amplitudes were about a factor of 5-6 higher than those of the building.

The operations building of the HDR is connected to the reactor building by a bridge which is supported in niches in both buildings. Evaluation showed this connecting structure to be an independent component, coupled to the two buildings mainly by friction. In the initial phase of the test with the highest eccentricity, this bridge was temporarily displaced by some 4 mm. Its rest position at the end of the test was some 2.5 mm away from its initial position. The operations building itself has a natural resonance at 3.1 Hz.

The rigid body motions for the basemat of the HDR building, were determined from measured acceleration signals (assuming a rigid plate). These measurements indicate that only minor torsional and vertical motions occurred. During traversal of the bending resonance, the horizontal translational motions were practically nil, whereas the rotations about the horizontal axes were quite considerable. From the ratio between the horizontal translational amplitude and the rotational amplitude, during traversal of the rocking resonance, a center of rotation can be determined at approximately 15 m below the basemat.

In the free field, the maximum vibrational amplitudes at higher frequencies and during traversal of the rocking resonance were measured primarily in the horizontal direction. On the other hand, when the bending resonance was traversed, vertical vibrations dominated [26].

2.2.4 Basemat Uplift

In 1984 - 1986, Elektrowatt Ingenieurunternehmung AG (EWI), a Swiss engineering consultant acting on behalf of the Swiss Central Department for the Safety of Nuclear Installations, conducted studies of the dynamic soil-structure interaction of the HDR reactor building. Assuming that no tensile stresses were transmitted at the interface between the foundation and the soil, a non-linear pretest calculation of the SHAG tests indicated, that at a shaker eccentricity of 100,000 kgm roughly half the basemat of the HDR building would experience uplift.

At the maximum shaker eccentricity of 67,000 kgm used in the test, no indications of uplift were found in the test, either from the stresses measured in the foundation area, or from the global accelerations measured in the reactor building. A comparison calculation of EWI, with a model unchanged from the pretest calculation and with the actual eccentricity used in the test, confirmed this finding. However, the calculated eccentricity ratio of 0.28 was only some 15% below the limit of 0.33, at which uplift occurs. Another recalculation, in which the soil stiffness and the damping values were matched such that the rocking frequency of the building and the acceleration and displacement of the building at rocking resonance traversal were in good agreement with the measured values, produced a much lower value of 0.12 for the eccentricity ratio and, consequently, a considerable margin from the critical value of 0.33.

The major reason for the differences between the pretest /comparison calculations and the recalculations, is the fact that the precalculations were non-linear only with respect to uplift. For the soil stiffness, the assumption was made that it remained unchanged up to uplift; while, in actual fact, a soil stiffness reduction by one-half was observed (see Section 2.2.2) [26].

2.2.5 Soil Liquefaction/Pore Water Pressure

To assess the possible hazard to the stability of the HDR reactor building as a result of soil liquefaction, the SHAG experiments were preceded by a safety evaluation performed by the Battelle Institute, Frankfurt. Following the KTA-Rule, the assessment was based on a comparison of the dynamic shear stress produced by the test load with the effective normal stress

in the foundation soil. The latter is calculated from the load imposed by the building and the overburden on the soil region under consideration. The study indicated that even at a shaker eccentricity of 125,000 kgm there would be no hazard of soil liquefaction.

During the tests, two pore water pressure sensors were installed at 14 m depth directly adjacent to the HDR foundation. The so-called semidynamic pore water pressure can be derived from the readings of these sensors; it is an indicator of the degree of soil liquefaction. The evaluation and extrapolation to the load case underlying the safety assessments, indicated a safety margin of at least a factor of two, for partial liquefaction with marked loss of load carrying capacity of the soil. This fully confirms the result of the safety evaluations based on the KTA-Rule (Figs. 2.9 and 2.10) [26].

2.2.6 Free-field Vibrations

The pretest calculations, also performed by the Battelle Institute, for the free-field vibrations generated by the SHAG tests, served to assess whether neighboring facilities would be unduly affected by the tests conducted in the HDR. A rotationally symmetric non-linear finite difference model of the soil was developed for the calculations and was coupled with a simple beam model of the building.

Since these were safety calculations, the assumed load cases for the computations corresponded to the performance limits of the shaker. The latter was deliberately designed to produce forces that significantly exceed the expected load bearing capacity of the HDR building. The calculations showed that even for these load cases no inadmissible vibrations were to be expected in neighboring facilities from the tests. In the SHAG tests, approximately one half that load was applied, and the accelerations, measured in neighboring facilities (corresponding to this load) reached roughly one-tenth of the permissible levels [26].

2.2.7 Calculations of the Building Response and Comparison with the Measured Results

Comparison calculations of the vibration behavior of the soil-structures systems were performed by five institutions with different models. The purposes for the calculations were as follows:

- Assessment of the vibration behavior and the loading/response of the reactor building under strong earthquake-like excitation.
- Validation of the procedures for determining load-dependent parameters for linearized calculations in the areas of:
 - soil-structure interaction,
 - materials behavior, steel-reinforced concrete, damping.
- Suitability of simple non-linear models.

The applied computational models can be characterized as follows [29]:

- BHZ model:** Beam model with elastic coupling of the inner and outer structures; non-linear soil springs derived from an earlier study of the soil-structure interaction with basemat uplift.
- CER model:** Beam model with elastic coupling of the inner and outer structures; linear soil springs.
- IMB model:** Shell model of the outer structure, rigidly coupled to the beam model of the inner structure; linear soil springs.
- KUH model:** Beam model with elastic coupling of the inner and outer structures; non-linear soil springs, decoupled in the two directions in a blind comparison calculation (CKUH), but coupled for a recalculation (DKUH).
- KWU model:** Rotationally symmetrical shell model, linear soil springs.

Figure 2.11 shows the measured data and also the calculated results of seven calculations for the acceleration at the apex of the outer dome of the HDR building. To provide a better picture of the motions, the acceleration responses were integrated twice and represented as displacements in a similar manner in Fig. 2.12. In both cases, it is not the time-histories of the response vibrations which are plotted, but only their envelope curves (i.e. the values of vibration amplitudes) versus the shaker frequency measured at the same point in time. Since the shaker frequency decreases during the test from the startup frequency, the start of the test is on the right hand side of the plot.

The differences between the computed results and the measured result, as well as among the calculations themselves, discernible in the two figures, can all be traced back to the specific characteristics of the models (linear, non-linear, choice of parameters). This makes it possible to draw conclusions as to the advantages and disadvantages of various approaches to the computational representation of such non-linear structural vibrations.

It is first necessary to recall the actual sequence of the test. Excitation of the building begins with increasing force, while the shaker arms are brought together, at a frequency only slightly above the (linear) resonance frequency of the building corresponding to a low level of excitation. With increasing displacement amplitude and soil deformation, the stiffness of the soil and thus the resonance frequency continue to decrease, i.e., the resonance frequency of the building is "pushed ahead" of the excitation frequency. Since the excitation force decreases with the frequency, only minor further increases in the displacement amplitudes are produced, until resonance is traversed. Because of the relationship between the acceleration and the displacement amplitudes in the frequency domain, the increase of the acceleration amplitude relative to the displacement amplitude is further reduced during resonance traversal. In the T40.13 test shown here, it even decreases from the beginning of the test (from right to left).

The non-linear precalculations (BBHZ), and the blind comparison calculations (CBHZ, CKUH), in principle reflect these events correctly. In particular, the frequency of the final

passage through resonance is captured very well. However, in the current problem, the building responses are clearly overestimated, because in all three cases the damping values ($< 6\%$) were assumed too low (see Fig. 2.8). In addition, in the CKUH calculation, the decoupled treatment of non-linear soil springs resulted in acceleration peaks at higher frequencies in the building response. This explains the deviation in the acceleration signal compared to the CBHZ and BBHZ calculations, while agreement in the displacement amplitude is good.

In a linear calculation, the simulation of the experiment must produce different results. If the highest load-dependent reduction in soil stiffness during the test run is predetermined accurately, as in CIMB, and if in addition the damping (9.1%) is of the right order of magnitude, then in the region of resonance traversal good agreement is obtained between calculation and measurement, as shown in the diagrams. However, above the resonance frequency, the building response is then inevitably underestimated by the linear calculation. If one uses the higher soil stiffnesses and lower damping values appropriate for lower level excitations, as in the CKWU case, resonance traversal in the simulation calculation occurs at a higher frequency and at higher excitation forces, so that the peak acceleration is overestimated.

The third linear calculation in these figures (BCER) deals with a real precalculation with an eccentricity lower than in the test (59,000 kgm). This, together with the fact that the load-dependent soil spring stiffness was assumed somewhat on the low side, explains the differences in the calculated results compared to CIMB. The high acceleration peaks at approximately 1.5 Hz in the two BBHZ and BCER precalculations are explained by the fact, that, in these simulation calculations with a coupled shaker model, the full excitation force is suddenly applied to the building at rest [26].

2.2.8 Probabilistic Structural Calculations

Probabilistic structural mechanics allows the consideration or treatment of "uncertain" (i.e., varying) parameters in a calculation, such as concrete strength or ground acceleration in an earthquake. Such parameters are considered to be random variables characterized by their probability distributions. The tests conducted at the HDR offered an opportunity to evaluate and improve, on the basis of test results, the models and procedures used by CERRAM in such reliability studies [30].

The load function of the shaker was assumed to be given (determined), while the masses, stiffnesses, and damping values of a building model, and the concrete strength data, were considered to be random variables. As far as possible, the parameters of the respective probability distributions were estimated on the basis of existing experimental data or data from the literature. The structural responses were determined by the response surface method. The outer shell and the basemat of the HDR reactor building were considered as potential failure sites. Exceedance of the concrete tensile strength was used as the failure criterion. The failure probability was determined by the importance sampling technique.

For the performed tests, the analysis indicated failure probabilities of 2.2% for the basemat and 16% for the outer shell. The normally accepted values are of the order of 10^{-6} for the loss of load-bearing capacity or 10^{-3} , for the loss of functionality. These results confirm that the HDR shaker tests achieved their goal of loading the building to its stress limits without loss of global load bearing capacity of the structure. This was only possible thanks to the approach, described in Section 2.2.1, i.e., performing measurements during the test and extrapolating the stresses at critical locations prior to each load increase [26].

2.3 Evaluation Summary for the VKL Piping System

2.3.1 Supports, Comparison of Test Results Obtained for Various Hanger Concepts

Approximately half of the shaker test runs performed as part of the SHAG test group were so-called "piping tests." These primarily served the comparative evaluation of various piping support concepts which were subjected to indirect excitation through the building with load levels on the order of magnitude of a design basis earthquake.

Five different configurations (with the same static hanger system) should be distinguished for the dynamically effective support systems (see Figs. 2.13 and 2.14). (Note, for the two configurations with visco-elastic dampers, Configurations 6 (Gerb) and 7 (ANCO), only a few comparison tests were run.) These are:

- HDR configuration, 2 articulated struts (very soft system).
- KWU configuration, 5 articulated struts (soft system).
- NRC configuration, 6 articulated struts, 6 snubbers (very stiff system).
- EPRI/EA configuration, like NRC, but with 3 energy absorbers instead of 6 snubbers.
- EPRI/SS configuration, like NRC, but with 6 seismic stops instead of 6 snubbers.

For these five configurations, three tests each were run at nominally the same loads, i.e., they all had the same shaker eccentricities and startup frequencies. A direct comparison of the measured responses for all these configurations is not very meaningful, because the frequency dependence of the shaker forces produces higher test loads in the upper frequency range of excitation (around 8 Hz) than in the lower frequency range. Thus the soft systems with low resonance frequencies might be judged more favorably in an evaluation based on direct response measurements. Therefore, all test runs were normalized, i.e., the test measurements were multiplied by factors equal to the ratio of their building response spectra peaks to the corresponding value taken from a reference spectrum (Fig. 2.15).

Even the comparison of the normalized measurements does not indicate any advantages for a rigid support design (NRC configuration) relative to a reasonably designed flexible support system (KWU configuration). On the other hand, the very soft HDR configuration which was not

designed for any dynamic loads, exhibits major drawbacks (inadmissibly large displacements and stresses). The energy absorbers and seismic stops, which should be considered as snubber replacements, proved themselves in the comparison tests, inasmuch as the pipe stresses for these configurations were comparable to those for the NRC configuration. However, due to the impacts occurring with seismic stops the piping was occasionally subjected to very high shock-type local loads (Fig. 2.16) [26, 31].

2.3.2 Recalculation of the Vibration Response of the VKL

Blind comparison calculations of piping behavior in these tests were performed by Wölfel Engineering Consultants and by the Rhineland Technical Inspectorate (TÜV Rheinland) for three configurations (HDR, KWU, NRC). Comparing the calculations among themselves and with the measurements, some very distinct differences in the dynamic responses were observed. To some extent, these differences were attributed to differences in modeling (masses of components, stiffnesses of support and restraint systems), but partly also to the idealization of the excitation in the particular computer models.

The TÜV Rhineland model was also used to recalculate the test cases with a modified model (relative to the blind comparison calculations). For the HDR configuration, much better agreement was found between measurement and calculation after the suspected impact of the thin piping section against an adjacent pipe was simulated by springs. For the KWU configuration, the differences relative to the measured results were smaller, while for the NRC configuration agreement between measurements and calculation was again unsatisfactory. This is attributed to the influence of non-linearities in the large number of supports, i.e., clearances in the articulated struts, and the function-dependent non-linear behavior of the snubbers.

2.3.3 Calculational Comparison of Various Hanger Concepts Under Operational and Seismic Loads

To provide a basis for a comparative computational evaluation of the three support configurations (HDR, NRC, KWU), both Wölfel Engineering Consultants and the TÜV Rhineland used the design approach for calculating the operational loads on the piping and the dynamic loads arising in a safe shutdown earthquake using a realistic spectrum and building floor accelerations of 1.4 g horizontally and 0.7 g vertically. They also computed the reference design stresses in accordance with the load combinations of the KTA 3201.2 Rule. Both calculations demonstrate that the operational stresses, i.e. the stresses produced by the dead weight, internal pressure and thermal expansion loads, differ only marginally in the three configurations, inspite of the different numbers of rigid supports. On the other hand, both calculations show a clear ranking of the stresses resulting from the safe shutdown earthquake load case: The lowest stresses were calculated for the stiffest system (NRC), the highest stresses, for the softest system (HDR). The two independent studies by TÜV Rhineland and by Wölfel Engineering Consultants differ slightly in the individual results and in their evaluations [26].

Fig. 2.1 Coast-down shaker-MK16 and its installation in the HDR

Fig. 2.2 Acceleration time-histories at shaker eccentricities of 8,200 and 67,000 kgm

Fig. 2.3 Shaker capabilities (design) and actual test excitation range

Viewing Level: -5.63m, -8.53m; -9.03m

Fig. 2.4 Cracks in a floor slab in the lower region of the building structure

Fig. 2.5 Location of highest bending stresses in the outer containment (at approximately 180°) and concentric cracks in the soil

		T40.13 Rocking	T40.37 Bending	Friauler E. quake	El Centro E. quake
Acceleration	(m/sec²)	1.42	1.22	1.05	0.85
Operating floor +30m	%	135	116	100	80
Displacement	(cm)	3.5	1.9	1.2	1.2
Operating floor +30m	%	292	148	100	100
Moment	(MNm)	391	335	183	173
Inner structure	%	214	183	100	95
Moment	(MNm)	400	485	101	107
Outer structure	%	396	480	100	106

Test parameter T40.13: Eccentricity 67000kgm, Starting frequency 1.6Hz
T40.37: Eccentricity 27800kgm, Starting frequency 2.1Hz

Fig. 2.6 Comparison of calculated building responses for shaker and earthquake excitation [23]

		Allowable shaker eccentricity for rocking resonance (In 1000 kgm)
Structural component	Min	Best Estimate
<u>Inner Structure</u>	49	100
<u>Floor -8.0 m</u>	7	22
<u>Foundation slab</u>	22	38
<u>Outer Structure</u>		
- undisturbed area	44	66
- Disturbance due to floor -8.0 m	13	20
<u>Slippage of steel containment (egg-cup)</u>	44	61
<u>Basement uplift</u>	70	80
<u>Surrounding soil</u>	41	- - -

Achieved in test: 67000 kgm

Fig. 2.7 Critical structural elements and allowable shaker eccentricities for the rocking resonance of the reactor building as determined by safety calculations [27]

Fig. 2.8 Increase in damping and decrease in rocking frequency at resonance as a function of shaker eccentricity

Fig. 2.9 Locations and results of pore-water pressure measurements

Fig. 2.10 Maximum semidynamic pore-water pressures and soil-liquefaction potential as a function of shaker eccentricity

Fig. 2.11 Comparison of calculations with measurement for the response at the apex of the outer dome (envelope of accelerations)

Fig. 2.12 Comparison of calculations with measurement for the response at the apex of the outer dome (envelope of displacements)

Fig. 2.13 VKL – piping system with hanger positions, SHAG tests – T40

HANGER CONFIGURATIONS							
HAN-GER-NO.	1 HDR	2 KWU	3 NRC	4 EPRI/E.A.	5 EPRI/S.S.	7 Anco	6 Gerb
1	_____	_____	S.A.	E.A.	S.S.	P.D.	_____
2	_____	_____	S.A.	_____	S.S.	P.D.	_____
3	_____	_____	STRUT				
4	STRUT						
5	STRUT						
6	_____	_____	S.A.	E.A.	S.S.	P.D.	_____
7	_____	_____	S.A.	E.A.	S.S.	P.D.	_____
8	_____	_____	S.A.	E.A.	S.S.	P.D.	_____
9	_____	STRUT					
10	_____	STRUT					
11	_____	STRUT					
12	_____	_____	S.A.	_____	S.S.	P.D.	_____
13	SPRING HANGER (NEW)						
14	SPRING HANGER (OLD)						
15	THREADED ROD						
16	CONSTANT-FORCE HANGER						
17	CONSTANT-FORCE HANGER						
18	CONSTANT-FORCE HANGER						
19	CONSTANT-FORCE HANGER						
20	_____	_____	_____	_____	_____	_____	P.D.
21	_____	_____	_____	_____	_____	_____	P.D.

S.A.: SHOCK ARRESTOR, "SNUBBER"

E.A.: ENERGY ABSORBER

S.S.: SEISMIC STOP

P.D.: PLASTIC DAMPER

Fig. 2.14 Arrangement of dynamic supports for the various hanger configurations, SHAG tests - T40

Fig. 2.15 SSE reference spectrum and SHAG original and weighted floor response spectra (3 test runs)

Fig. 2.16 Acceleration, support forces and bending stresses for the various hanger configurations SHAG tests – T40

3 T41 TEST GROUP, SHAM: SERVOHYDRAULIC EXCITATION OF MECHANICAL EQUIPMENT

3.1 Introduction, Overview, Objectives

The SHAM tests (T41) were performed in April and May 1988 as the final test group in Phase II of the HDR Safety Program. The behavior of a pipe line under very high seismic loads was investigated [32]. In contrast to earlier seismic studies in Phase I (V2, V3, V4, V63, V64, V65, V69, V70) [2-18], the load levels in the SHAM tests were many times higher than those of a safe shutdown earthquake (SSE). The earlier tests had been characterized by a deliberate restriction to linear vibration behavior of the excited structures and to a range of loads producing no material damage. Similarly in the SHAG tests (T40), the maximum strains measured in the mechanical equipment, which was indirectly excited through the building, remained largely in the linear-elastic range [21, 22, 26, 31].

Hence, the overall objective of the SHAM tests was to deliberately advance into the range of non-linear vibrations and materials behavior in pipes, by directly exciting a piping system up to approximately 8 times the SSE level. The tests and the accompanying theoretical studies served the following specific objectives:

- Verification of computational methods used for the seismic design of piping, and for the determination of safety margins.
- Evaluation of various dynamic pipe support concepts.
- Studies of non-linear effects (plastification, non-linear hanger behavior, damping).

The test object of the SHAM tests was again the VKL (Fig. 3.1), a multiply branched piping system with nominal pipe diameters between 100 and 300 mm. The piping was simultaneously excited at two locations by means of servohydraulic actuators, which produced displacement controlled artificial earthquake histories. This excitation simulated the earthquake motion of the building wall at the attachment point of a pipe support.

The SHAM tests were planned and executed jointly by an international group of institutions from Germany, the United States, and the United Kingdom. On the German side, the participants were the Fraunhofer-Institut für Betriebsfestigkeit (LBF) and the KWU Division of Siemens AG. The U.S. Nuclear Regulatory Commission (NRC), Office of Research, participated through the efforts of the Argonne National Laboratory (ANL) and the Idaho National Engineering Laboratory (INEL). Participating in the design work on behalf of the Electric Power Research Institute (EPRI) were the Bechtel Power Corporation and R. L. Cloud and Associates. The British Central Electricity Generating Board (CEGB) participated with its Sizewell B Project Management Team.

The seismic design of piping primarily requires the determination of those locations in the piping where it is necessary to constrain one or more components of the displacements and

rotations relative to the building, so as to prevent inadmissible deformations and stresses of the piping during an earthquake. The basic design for the SHAM tests, developed by INEL, was a very stiff support system typical of US plants. In addition to the hangers supporting the pipe weight and the struts for applying the loads, 12 other restraints were defined for this NRC configuration; see Fig. 3.2. Taking into account the thermal displacements, only six of the restraints could be designed as struts, while so-called snubbers had to be installed in six locations. Since one of the major objectives of the NRC was to study the consequences of individual support failures in tests with increasing loads, INEL dimensioned all supports as weak as possible.

Since snubbers are expensive to acquire and to maintain and, in addition, are considered to be not very reliable, all the other industrial participants wanted to qualify support concepts or design procedures, in which no snubbers were used. Both EPRI configurations served to demonstrate, in a practical application, that the restraints designed by Bechtel and Cloud (energy absorbers or seismic stops, respectively), can be used as substitutes for snubbers in existing piping. Cloud advocates a one-for-one replacement of snubbers by seismic stops, whose clearance is adjusted for the expected thermal displacement at the support point. Bechtel pursues the concept of replacing snubbers by a smaller number of energy absorbers, because the energy absorbed in those devices has a damping effect on pipe vibrations.

The design objective for the KWU configuration was to develop a flexible support system which exclusively uses struts, and which yields to the thermal expansions of the piping without a build-up of major stresses. A similar objective was pursued by CEGB. They used the SHAM tests to verify a support design optimization program, which, starting from the full complement of all possible restraining positions, determines the minimum number of supports that still satisfies the allowable loads and stresses for the pipe line. Finally, the HDR configuration, was to represent the bounding case of an extremely soft piping support system. This was achieved by using no other supports than the two struts, at the two excitation locations, required by the test to limit the lateral forces acting on the hydraulic actuators.

With the six support configurations, designed in accordance with the different objectives of the participating institutions, a total of 50 tests at various loads was performed on the VKL piping as part of the SHAM test group. The piping and the adjacent vessels were filled with water at ambient temperature and were held at a static internal pressure of 70 bar. Because CEGB had an additional support variant tested, and NRC modified the pipe system for the final test series, the test matrix in Fig. 3.3 shows eight different support systems (instead of the planned six). For each of the six basic configurations (Fig. 3.2) two tests were run with random excitation (2-40 Hz) for systems identification, each with loading only at one excitation location.

The reference load case was an artificially generated earthquake-time history with a given spectrum. The actual spectrum, the so-called HDR spectrum, see Fig. 3.4, was normalized to a zero-period acceleration of 6 m/s^2 (100% SSE). The generated time-history had a duration of 15 seconds. Three additional load-vs.-time functions were run for the CEGB support system (see Fig. 3.4), namely:

- Earthquake-time history (generated) with a spectrum applicable to the Sizewell B site, duration approximately 20 seconds, zero-period acceleration 4.4 m/s^2 .
- Earthquake-time history (generated) with a spectrum enveloping all British sites (All-Sites Spectrum), duration approximately 20 seconds, zero-period acceleration 7 m/s^2 .
- Sine burst excitation, frequency 4.2 Hz (a piping resonance), duration approximately 7 seconds, maximum displacement amplitude in the tests 60 mm.

In order to determine the safety margins of the systems and of their components, the SHAM tests concentrated on the investigation of the behavior and the strength of the piping and the supports under loading which clearly exceeded the design basis loads. The load increases up to a maximum of 800% SSE were obtained by linear amplification changes in the level settings of the hydraulic actuators.

Two configurations (the stiff NRC system and the flexible KWU system) were to be tested up to their limits or to the limits of the excitation system (800% load case). However, in order for all support systems to reach the maximum possible load increase, without jeopardizing the feasibility of the following experiments, an approach was chosen which is explained below by means of the strain measurement results at Cross Section 7 near the Tee, see Fig. 3.5. In this diagram, the ranges between the upper and the lower level of the measured strains in a test are indicated by the solid vertical bars. The horizontal thinner lines connecting those bars indicate the permanent strains accumulated during the test series. The sequence of bars from left to right corresponds to the test sequence.

For the first series of tests with the HDR spectrum, the loading was increased only so far that the support loads did not exceed four times the design-basis loads, and the strain amplitudes at straight pipe cross-sections did not exceed 0.2%, and at elbows, 0.4%. With these limits it was possible, for all configurations designed for the 100% load case, to run the 300% SSE load case without any problems. This was followed by comparison tests at 100% SSE performed with the HDR and CEGB configurations which were not designed for this spectrum. The next tests were conducted with the CEGB configuration; first with one and three times its design load corresponding to its design spectrum (Sizewell B), and then two tests at 50% and 200% of the load corresponding to the all-sites spectrum. Subsequently, this second series of tests was completed with the HDR configuration at 200% SSE (HDR spectrum).

In a slightly modified form, the NRC configuration was next tested up to eight times the design basis earthquake. At 600% SSE, three snubbers failed under overload, these were not replaced. In the 800% SSE load case, another snubber failed under overload without causing damage to the piping or permanently deforming it to an unacceptable extent.

The subsequent sine burst tests of the CEGB configuration were performed in order to induce "ratcheting" by the coupling with the existing static loads consisting of the internal pressure and the dead weight. This effect can be clearly observed in Fig. 3.5. The strains grow monotonically with each test, on the upper side of the pipe as tensile strains, and on the lower side as compressive

strains. Since the resultant global deformation of the pipe was still limited, the tests of the KWU configuration at loads of 400%, 600%, and 800% SSE (HDR spectrum) were run without necessitating any prior repair work. Again, in these tests the piping remained undamaged.

Figures 3.6 and 3.7 give an indication of the stress occurring in the VKL piping in the SHAM tests. The nominal stress amplitudes in Cross Section 7 (thin pipe) reached roughly 600 MPa (fictitious values, since they extend far into the plastic range), while the strains at this point exceeded 1%. The amplitudes of the local strains measured at Elbow 1 also reached nearly 1% [32].

3.2 Summary of Evaluations

3.2.1 Piping Loads and Stresses

The results measured for the 100% SSE (HDR spectrum) load case are shown in Fig. 3.8 as maximum bending stresses for six configurations. The bending stresses are used because they clearly dominate among the measured dynamic stresses. For nine out of twelve nominal-stress measurement cross sections, the stiff NRC configuration had the lowest stresses among all configurations; however, the differences relative to the KWU configuration and the two EPRI configurations are insignificant. In all configurations, the pipe region close to the DF 16 excitation point (cross section, QA 100) was subjected to relatively high stresses. Except for this region, the nominal stresses measured in the DN 200 (200 mm) piping (approximately 10-40 MPa) are quite low for all support configurations.

With respect to stresses, no advantage or disadvantage was identified in the tests for any of the support configuration specifically designed for this load case. The HDR configuration, which was not designed for earthquakes, and the CEGB configuration, which was designed for a different excitation spectrum, exhibited by direct comparison, clearly higher stresses in the DN 100/125 (100/125 mm) pipe section, see Fig. 3.8

No direct relationship was found between the number of supports and the piping cross-sectional forces and stresses. The same conclusions were made in the evaluations of the SHAG tests in which the VKL, with various support configurations, was indirectly excited by the vibrations of the reactor building [see Section 2.3]. Therefore, it is safe to say that the proper design of a support system for the applicable load spectrum is decisive for the piping stresses and not the number and type of supports, nor the overall stiffness of the support system [33].

3.2.2 Load-carrying Capacity of the VKL Piping

Stress limits in piping subjected to seismic loads are usually imposed based upon given strength criteria (stress allowables – minimum values). The realistic strengths are usually higher than these tabulated values. Tensile tests conducted at MPA Stuttgart determined the strength parameters of the two most highly loaded regions using samples of pipe material obtained

from those regions. For both materials, the real yield point is 25% above the specified minimum values. In addition, the materials are clearly tougher than required [34].

The design requirement, to limit allowable stresses to levels derived from the static strength of the materials, serves mainly to prevent plastification of large regions of the pipe and, thus, to avoid collapse or the ratcheting effect described earlier. For the high-toughness steel grades used in modern reactor construction, this stress limit is much stricter than necessary to exclude failure from crack formation under vibration-loading. This is evident from Fig. 3.9, in which the Level D allowable stress of an austenitic material (1.4961) is compared (top diagram) with the fatigue-stress curve (Wöhler diagram) for austenitic materials given in KTA 3201.2 [35]. Accordingly, the vibration amplitude of the allowable stress can be tolerated by the material approximately 10,000 times.

The bottom diagram in Fig. 3.9 presents the observed magnitudes and frequencies of the strain amplitude. These were determined from the strain measurements of all 50 SHAM tests, for the two most highly stressed regions in the pipe, using the "rain flow" counting technique. The diagram also shows the fatigue curve of the material derived from the Wöhler stress diagram, shown above, in accordance with the provisions in KTA 3201.2 [35]. The Level D allowable stress corresponds to a strain amplitude of approximately 0.44%. In the SHAM test this value was exceeded in the Elbow (QA 8112) more than three-fold for approximately one hundred times. The margin between the strain collective and the fatigue curve shows that, theoretically, the entire series of SHAM tests could have been repeated approximately 40 times in order to exhaust the fatigue life of the material. This implies that a single earthquake has no impact whatsoever on the fatigue life. Overall, the approach used in the design procedure with respect to stress limitations, can be considered to be highly conservative for the brief seismic loads that occur only a few times [33].

3.2.3 Damping

The damping values for piping used till now in design practice are considered to be conservative [36]. Indirectly they take into account the influences of external damping by allowing higher damping values to be used for pipes of larger diameter (>300 mm) than for pipes of smaller diameters (2% as against 1% for the OBE). The dependence of internal damping on the load level is expressed by the fact that the damping values applied to the SSE load case may be 1% higher than those applied to the OBE load case.

Since far higher damping values were determined in some vibration studies of piping systems, these prescribed damping values have repeatedly been the subject of discussions among those actively working on the applicable codes. However, the dispersion in the results is very pronounced, and in almost all cases, for the same piping systems, both vibration modes which were highly damped and which had little damping were observed.

In the new KTA 2201.4 draft code [37], a uniform value of 4% damping is stipulated for pipes of all diameters; the difference between the OBE and the SSE is generally abolished and replaced

by the Design Basis Earthquake. The reason given for this is, that the assumption of conservative damping for the hypothetical seismic accident, results in a stiff design of the system; this, in turn, has disadvantages for the stresses under normal operation. Moreover, based on a wealth of observations and analyses of plants subjected to seismic events, it is known that flexible piping systems have never been damaged by seismic vibrations. Lastly, the SHAM tests at the HDR have clearly demonstrated, that the flexible support system designed by Siemens/KWU exhibited no disadvantages under dynamic loading compared to the stiff NRC support system typical of US plants, and withstood eight times the design basis load without suffering any damage. This experience lead to the conclusion that the introduction of a "mean value concept" for damping in the seismic load case does not impair the overall safety of a piping system.

Against the background of this ongoing discussion, it seemed important to be particularly careful in determining the damping values during the evaluation of the SHAM tests. The SHAM tests are in many ways different from earlier piping vibration studies, namely:

- Excitation resembled a seismic event as closely as possible.
- The loads were increased stepwise far above the yield limit.
- The same piping was investigated with various support configurations, with differing support densities.
- The piping under investigation is sufficiently realistic (with branches, vessel connections, and various diameters).

In addition, the two techniques applied in the damping evaluation, by far surpass the one-degree-of-freedom evaluation methods still customarily used today, and are quite capable of producing robust results. These techniques are: (i) parameter identification directly from the measured data in random tests, with curve fitting simultaneously over all eigenmodes and measurement locations, and (ii) parameter variation calculations with individual fitting of the modal damping values at various load stages in the seismic tests.

Figure 3.10 shows the damping values for three support configurations of the VKL piping (HDR – most compliant; KWU – flexible; NRC – stiffest configuration) determined by parameter identification from the data measured in the random tests, as well as their mean values in the frequency range up to 10 Hz. The results indicate a trend toward stronger damping with increasing numbers of supports (see Fig. 3.11). However, the value of 3.92% for the flexible KWU system differs only slightly from the 4.15% of the stiff NRC system. Based on the experience in earlier piping studies, the values are as expected. Similarly the occurrence of individual either very strongly or very weakly damped modes (9.5% and 0.9%, respectively, of modal damping), was not unexpected.

The dependence of damping values on load level was investigated by parameter variation calculations of the seismic tests, in which modal damping values were individually fitted by comparison with the measured data. In Fig. 3.12, the results are compared with damping values given in or recommended for codes/standards. This representation is not quite as expected.

While in five out of six considered modes, a clear increase in damping is observed when the load increases from 300% to 400% SSE, damping still does not increase for mode 2, despite a maximum modal stress in excess of 500 MPa. There is no obvious relationship between the calculated modal stress and the damping. On the other hand, there is some relationship between the load level applied in the test and systems damping. On the average, for the six relevant modes, this value decreases (from 3.2 to 3%) with a load increase from 100 to 200%, remains unchanged with an increase from 200 to 300%, and increases more rapidly (to 3.7%) in a load increase to 400%.

This type of behavior has been observed in many experimental studies [12]. It is attributed to the fact that damping caused by mechanisms in which the damping force is independent of the vibration amplitude (e.g. friction forces) decreases proportionally with increasing amplitude, while the effect of other damping mechanisms (such as materials damping) only comes into play at higher amplitudes. The fact that, under this earthquake-like test excitation, the latter effect is only activated at loads at which the Level D stress limits were exceeded at several locations, is an unexpected result which merits further investigation. The possibility that these were locally limited overloads; i.e., that only a small volume plastified; must be eliminated because, in modes 2 and 4, four different regions of the piping always experienced stresses at least half as high as those at the most highly stressed locations. This implies that also in these regions the yield point had already been reached in the 300% load case.

Figure 3.12 shows the code damping values and the stress limits for their range of application. The current damping values in accordance with KTA Rule 2201.4 (old) [36] are clearly conservative lower estimates of the damping for the stress-relevant modes in the SHAM tests. The damping value of 5%, in the frequency range up to 10 Hz, proposed by the American Pressure Vessel Research Council (PVRC) [38] is clearly higher than the mean damping values determined in the same frequency range by the two types of excitation in the SHAM tests. In a study by Hadjian et al. [39] even higher damping values of up to 7.5% (for DN 200) are proposed for use in the American licensing procedure. These are derived based on a dependence of damping on the load level which was determined by regression analyses of experimental data. The database for these analyses is taken from various experiments in which, only in a few exceptions, the maximum stresses exceeded half the elongation limit. Consequently, from the given data an extrapolation was made to approximately fivefold higher stresses. The values derived in this manner are not confirmed by the SHAM tests.

On the other hand, in the light of the SHAM test results, the uniform value of 4% for all pipes, as proposed in the new German draft KTA 2201.4 (draft version 6/89) [37], may be considered to be a reasonable value. Admittedly, applying this damping value in a particular design step, namely the calculation of pipe stresses for a given excitation spectrum, will not necessarily lead to conservative results. However, the excitation spectrum itself is determined conservatively, and the load bearing capacity of the piping during seismic excitation is likely to be much higher (as has been shown by many observations including the results of the SHAM tests) than indicated by the allowable limits according to KTA 3201.2 [35] (see Section 3.2.2 and [32]).

3.2.4 Support Loads

The dynamically active supports used in the SHAM tests included articulated struts, snubbers, energy absorbers, and seismic stops. Because they mainly accommodate the static loads, the other existing supports, such as spring-hangers and constant-force hangers, are not considered under dynamic loading. In the tests, the supports were always investigated as a complete set. One would expect that a greater number of supports on a piping system should tend to reduce the loads acting on each individual support. However, in actuality, additional supports may also lead to load increases in existing supports

There is no direct relationship between the number of supports and the support loads. The design of a specific configuration is more crucial than the number and type of supports. Failure of individual supports during the tests, or their planned omission, did not necessarily increase the loads in the other supports or in the piping. In particular, the failure of snubbers under overloads was always observed to be an individual event without any direct repercussions for neighboring restraints; i.e., no "zipper effect," although INEL, in its test evaluation, arrives at exactly the opposite conclusion [40].

The systems replacing snubbers performed their functions quite satisfactorily. The impact forces in the new design seismic stops were effectively mitigated by elastic stops (disc springs), so that they were barely higher than the forces occurring when snubbers lock. The energy absorbers were designed in such a manner that they did not plastify at 100% SSE, and produced their actual beneficial effect, the absorption of energy, only at higher load levels.

Twelve different articulated struts were installed during the tests (three sizes, two manufacturers). No strut failed, although overloads up to nine times the rated load occurred in the test (see Fig. 3.13). The expected overload capacity (four times the rated load) of the struts was repeatedly exceeded by a significant margin for five of the struts. A detailed margins-evaluation by ANL gave similar results, with a maximum sevenfold margin for strut loading [41].

A total of 15 different mechanical snubbers were installed (three sizes, two manufacturers). Unlike the struts, four out of nine snubbers failed after having been loaded beyond their rated load bearing capacity. The expected overload capacity (2.4 times the rated load) was not attained by all snubbers. All snubbers that failed in the tests, did so under overload, without any externally visible symptoms. In all cases, failure implied that no support forces could be transmitted any more, and unimpeded motion was possible until internal stops were reached. Thus, these defective snubbers behaved like seismic stops (with large free displacements and without impact stops). Another possible malfunction, blockage also against slow motion (impeding thermal expansion), was not observed in the SHAM tests [32]. Again, INEL in its test evaluation concluded that one of the snubbers failed essentially in a rigid or blocked mode [40].

3.2.5 Comparison of Measurements with Calculations

Using design basis calculations and considering different objectives, the different support configurations were designed, prior to the test, for a specific loading spectrum. Based on the measured pipe stresses, no advantages or disadvantages can be discerned for any of the configurations designed in this manner. On the other hand, the configurations that were not at all seismically designed, or were not designed for the particular spectrum, experienced clearly higher stresses; see Section 3.2.1. A detailed comparison of calculations with measurements, of the dynamic portion of the piping stresses, indicates that the dynamic calculations contain only true conservatism when the appropriate damping assumptions or spectrum broadening is used. The same does not necessarily hold for the computational methods (be it Response Spectrum Methods–RSM or Time History Analysis–THA) or for the superposition of multiple excitation directions (3D excitation) [32].

The objective of the comparison calculations performed after the tests, but without any knowledge of the test results, was to show how well piping responses can be represented by linear models using the actual test excitation with realistic damping assumptions. Three different institutions participated in these comparison calculations with three different linear computational models and computer programs. This allowed a certain bandwidth in model development and in solution procedures to be taken into account. Also included in the comparisons were the pretest calculation by KWU, which used as input data the nominal excitation-time curve of the tests. This is also a meaningful comparison, because the excitations obtained in the tests differ only slightly from the nominal time history. In all three models, damping was assumed to be between 3 and 4% in the frequency range of 2-8 Hz, using a Rayleigh formulation with almost identical parameters.

The comparisons between measurements and calculations, as well as the comparisons among calculations, show a very heterogeneous pattern. In Fig. 3.14, the statistical data for all measurement locations (mean, standard deviation, and highest and lowest values) are plotted as ratios between the maximum calculated and the maximum measured values for four calculations and for three different quantities (accelerations, strut forces, and bending stresses). Ratios larger than 1 mean that the structural responses are overestimated by the calculation, while values below 1 imply that they are underestimated. The results from all measurement locations are equally included in the statistical evaluation, irrespective of their absolute magnitudes.

Clearly, accelerations and strut forces are generally underestimated in the calculations. The mean values of the calculations differ hardly among each other, except CKWU for the accelerations and CMPA for the strut forces. For the stresses, the mean values of the measurement/calculation ratios are close to unity for all calculations, i.e., the stresses are estimated more or less correctly in the calculations. The picture would be even more favorable if the ratios were weighted in accordance with the magnitude of the measured stresses. The large differences, between calculations and measurements, for the accelerations and strut forces can be attributed to the higher-frequency contributions in the measured structural responses, which are due to non-linearities in a real system. On the positive side is the fact, that the stresses, which are

the design basis quantities, are well estimated in the calculations. A similar pattern was found in the results of an independent investigation by ANL which compared both linear and nonlinear predictions to the SHAM test measurements [42, 43, 44].

Three of the support types used in the SHAM tests were deliberately designed for non-linear behavior (snubbers, energy absorbers, seismic stops). The influences of these non-linearities were studied by comparative linear and non-linear modeling. On the average, the results of the non-linear models are closer to the measured results than those of the linear models. This holds, in particular, for forces in the actual non-linear supports. For the forces in the other supports, and for the accelerations and stresses in the piping, no such clear improvements were found, as to justify the large effort involved in non-linear time-history calculations [32].

Pronounced non-linear materials behavior was observed in the tests only in the two most highly stressed regions of the piping. This implied that a non-linear calculation with a simplified method was possible, in which the component stress-strain curves are derived by static calculations describing globally the non-linear behavior of the respective regions. In this manner, non-linear dynamic calculations can be carried out with justifiable effort. Backed by parameter variation calculations, this approach may permit the determination of the main differences between the linear and non-linear structural responses [32].

The effect of randomness in the artificially generated earthquake-time history on the results obtained in the SHAM tests was investigated by probabilistic methods. This was accomplished by determining the parameters of stationary random processes from the structural responses and using them to determine the probabilities of exceeding the stress-allowables. For the design basis load case, this resulted in values on the order of 10^{-6} [32].

Fig. 3.1 VKL piping system with hanger positions, SHAM tests (T41)

Hanger No.	Hanger configuration					
	NRC 3	EPRI/SS 5	EPRI/EA 4	CEGB 6	KWU 2	HDR 1
4	Strut	Strut	Strut	Strut	Strut	Strut
23	Strut	Strut	Strut	Strut	Strut	Strut
9	Strut	Strut	Strut	Strut	Strut	
10	Strut	Strut	Strut		Strut	
11	Strut	Strut	Strut		Strut	
8	Snubber	Seismic stop	Energy absorber	Strut		
7	Snubber	Seismic stop	Energy absorber	Strut		
12	Snubber	Seismic stop		Strut		
3	Strut	Strut	Strut			
22	Snubber	Seismic stop	Energy absorber			
6	Snubber	Seismic stop				
2	Snubber	Seismic stop

Strut
 Energy absorber

Snubber
 Seismic stop

Fig. 3.2 Dynamic support positions for the different VKL hanger configurations, SHAM tests (T41)

Type and Location of Excitation		Hanger Configuration							
		HDR	KWU	NRC	EPRI/ EA	EPRI/ SS	CEGB	CEGB mod.	NRC mod.
		1	2	3	4	5	6	7	8
Random	DF 16	■	■	■	■	■	■		
	H-5	■	■	■	■	■	■		
Earthquake HDR - Spectrum	100 %	■	■	■ ■ ■	■	■ ■ ■	■ ■		
	200 %	■	■	■		■			■
	300 %		■	■ ■	■	■			
	400 %		■		■				
	600 %		■						■
	800 %		■						■
Sizewell B Spectrum	100 %						■	■	
	300 %						■		
All Sites Spectrum	50 %						■		
	200 %						■		
Sine Burst	10 mm						■		
	20 mm						■		
	30 mm						■		
	45 mm						■		
	60 mm						■		

Fig. 3.3 SHAM test matrix

Fig. 3.4 Response spectra for 100% earthquake excitation (4% damping)

Fig. 3.5 Strain amplitudes and residual strain accumulation at cross section 7 during SHAM test series

Fig. 3.6 Maximum strain at elbow 1 (Location B in Fig. 3.1)

Fig. 3.7 Maximum bending stress at cross section 7 (see Fig. 3.5)

Fig. 3.8 Maximum measured bending stress for the 100% earthquake tests – HDR spectrum

Fig. 3.9 Comparisons of allowable stress and of the measured strain-range frequency-distributions with the KTA fatigue curves

Fig. 3.10 Modal damping values as a function of frequency for three hanger configurations

Fig. 3.11 Dependence of mean damping values on the number of active dynamic supports

Fig. 3.12 Comparison of code damping values with damping derived from measurements as a function of modal stress or load

Fig. 3.13 Frequency distribution for the exceedance of the nominal design load (strut H3)

Fig. 3.14 Statistical comparison of linear calculations with measurements, 100% earthquake, KWU configuration

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