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Organisation de Coopération et de Développement Economiques
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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

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LESSONS LEARNED FROM HIGH MAGNITUDE EARTHQUAKE WITH RESPECT TO NUCLEAR CODES AND STANDARDS

English - Or. English

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ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT

Pursuant to Article 1 of the Convention signed in Paris on 14th December 1960, and which came into force on 30th September 1961, the Organisation for Economic Co-operation and Development (OECD) shall promote policies designed:

- to achieve the highest sustainable economic growth and employment and a rising standard of living in Member countries, while maintaining financial stability, and thus to contribute to the development of the world economy;
- to contribute to sound economic expansion in Member as well as non-member countries in the process of economic development; and
- to contribute to the expansion of world trade on a multilateral, non-discriminatory basis in accordance with international obligations.

The original Member countries of the OECD are Austria, Belgium, Canada, Denmark, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, the Netherlands, Norway, Portugal, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The following countries became Members subsequently through accession at the dates indicated hereafter: Japan (28th April 1964), Finland (28th January 1969), Australia (7th June 1971), New Zealand (29th May 1973), Mexico (18th May 1994), the Czech Republic (21st December 1995), Hungary (7th May 1996), Poland (22nd November 1996), Korea (12th December 1996) and the Slovak Republic (14th December 2000). The Commission of the European Communities takes part in the work of the OECD (Article 13 of the OECD Convention).

NUCLEAR ENERGY AGENCY

The OECD Nuclear Energy Agency (NEA) was established on 1st February 1958 under the name of the OEEC European Nuclear Energy Agency. It received its present designation on 20th April 1972, when Japan became its first non-European full Member. NEA membership today consists of 27 OECD Member countries: Australia, Austria, Belgium, Canada, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Portugal, Republic of Korea, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The Commission of the European Communities also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its Member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes, as well as
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information. The NEA Data Bank provides nuclear data and computer program services for participating countries.

In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of scientists and engineers. It was set up in 1973 to develop and co-ordinate the activities of the Nuclear Energy Agency concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations. The Committee's purpose is to foster international co-operation in nuclear safety amongst the OECD Member countries.

CSNI constitutes a forum for the exchange of technical information and for collaboration between organisations which can contribute, from their respective backgrounds in research, development, engineering or regulation, to these activities and to the definition of its programme of work. It also reviews the state of knowledge on selected topics of nuclear safety technology and safety assessment, including operating experience. It initiates and conducts programmes identified by these reviews and assessments in order to overcome discrepancies, develop improvements and reach international consensus in different projects and International Standard Problems, and assists in the feedback of the results to participating organisations. Full use is also made of traditional methods of co-operation, such as information exchanges, establishment of working groups and organisation of conferences and specialist meeting.

The greater part of CSNI's current programme of work is concerned with safety technology of water reactors. The principal areas covered are operating experience and the human factor, reactor coolant system behaviour, various aspects of reactor component integrity, the phenomenology of radioactive releases in reactor accidents and their confinement, containment performance, risk assessment and severe accidents. The Committee also studies the safety of the fuel cycle, conducts periodic surveys of reactor safety research programmes and operates an international mechanism for exchanging reports on nuclear power plant incidents.

In implementing its programme, CSNI establishes co-operative mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA), responsible for the activities of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health and NEA's Radioactive Waste Management Committee on matters of common interest.

FOREWORD

The CSNI Integrity and Ageing (IAGE) Working Group deals with the integrity of structures and components, and has three sub-groups, dealing with the integrity of metal structures and components, the ageing of concrete structures, and the seismic behaviour of structures. Ageing is also a primary consideration of the group.

In the last ten years, highly populated and densely industrialized regions in the world were strongly affected by high magnitude earthquakes. For each significant event, specialists have gathered relevant information with respect to design practice and given expert judgement regarding the nuclear design and the specific needs for codes and standards.

The complete list of CSNI reports, and the text of reports from 1993 on, is available on http://www.nea.fr/html/nsd/docs/

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Gratitude is also expressed to members of the IAGE Working Group on the seismic behaviour of Structures which contributions helped in building an complete review on the topic.

Members of the Task Group:

Mr. Renard	TRACTEBEL
Mr. Kitada	NUPEC
Mr. Sollogoub	CEA
Dr Duval	EdF
Dr Renda	EC/ISPRA
Mr. Werner	HSK

LESSONS LEARNED FROM HIGH MAGNITUDE EARTHQUAKE WITH RESPECT TO NUCLEAR CODES AND STANDARDS

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1. INTRODUCTION

In the last ten years, highly populated and densely industrialized regions in the world were strongly affected by high magnitude earthquakes. For each significant event, specialists have gathered relevant information with respect to design practice and given expert judgment regarding the nuclear design and the specific needs for codes and standards.

Four earthquakes have been selected. They have in common a magnitude equal to or larger than 7 and have caused casualties and important losses of private and public properties as well as in industrial facilities in regions that were recognized seismically actives and where exist modern codes and standards for seismic design.

The individual reports are given in Appendices. This preliminary note highlights the most relevant items that were identified by the experts.

Reports have been written about the following events and are reproduced in appendices A to D. Reports contains their own set of conclusions and recommendations. Only major conclusions are reported in this summary.

- Northridge (California) : January 1994 – magnitude 6.7 (Mw)
- Hyogoken-Nanbu (Japan) : September 1995 – magnitude 6.9 (Mw)
- Kocaeli (Turkey) : August 1999 – magnitude 7.4 (Mw)
- Chi-Chi (Taiwan): September 1999 - magnitude 7.6 (Mw)

For the three first events, the epicenter of the earthquakes occurred near dense urban or heavy industrialized zones that were built on the causative fault. Numerous records have been gathered, giving valuable data about peak accelerations and frequency content and their correlations with the damages due to near field earthquakes.

2. MOST SIGNIFICANT OBSERVATIONS

Among the damages or the causes of damages that were highlighted, one can identify:

2.1 Recorded acceleration and displacements

Large fault displacements have been observed (4 to 5 m for Kocaeli, 8 m for Chi-chi and 1 m for Hyogoken), causing important damages to constructions and viabilities in the faulted area.

For those high magnitude earthquakes, the horizontal accelerations measured on rock were in the range predicted by the codes; when measured on alluvial or soft soils large amplifications were observed (factor of 10 in Kocaeli). In general the vertical accelerations were less than 0.7 times the horizontal accelerations, but larger than 1 in some instances (Northridge).

The recorded acceleration response spectra are well enveloped by the design response spectra of the codes in the medium to high frequency range, but not in the low to medium range.

2.2 Site effects

Slopes failures and rock falls were observed in the mountains (Chi-Chi); liquefaction and settlements occurred in the soft alluvial areas near the seashore (Kobe, Kocaeli). Sea flood and subsidence as well as tsunami occurred in Kocaeli. Site focusing effects might have occurred in Kocaeli and Chi-Chi.

Settlement and liquefaction effects have been reduced by soil improvement techniques, like jet grouting or stone columns.

Site seismic effect prediction techniques should be improved

2.3 Buildings

Tilting of buildings, settlements due to exceedance of the bearing capacity or bulging of the surrounding soil have been observed in soft soil areas that were also submitted to liquefaction or in the vicinity of the fault. (Hyogoken, Kocaeli).

Buildings with more than three levels should be avoided in areas near active faults.

The “strong column weak beam” concept should be recommended for design of new framed buildings and should be emphasized in the codes.

2.3.1 Reinforced concrete frames

Masonry infill panels were very vulnerable, loosing their stability when they are not anchored or being crushed or breaking the adjacent RC columns due to differential deformation.

More attention should also be given to reinforcement details like adequate transversal rebar for confinement of the concrete and of the main rebar in splice zone and “hinge” zones (Kocaeli).

Specific training of controllers to construction practice and procedures in seismic areas should be encouraged as well as adequate quality controls and material testing and acceptance criteria should be required during detailed design and construction

2.3.2. Steel frames

There was a considerable damage to welded steel moment-resisting frames during the Northridge event and to a lesser extent the Hyogoken earthquake, were brittle fractures occurred with cracking in column panel zones and fractures in beam to column weld connections that developed in the elastic range from flaw in the low toughness weld metal and geometric conditions.

It has been recommended to:

- Ensure a better and more realistic estimation of the moment capacity of a beam section;
- Use a welding metal with reasonably high notch toughness, controlled by tests;
- Design in accordance with a proper and reliable seismic performance, e.g. by avoiding large stress concentrations in the beam column connections or avoiding yielding in high tri-axial restrained zones

- Improve the quality control and assurance in construction and fabrication for welding procedures, weld quality and welders qualification.

2.4 Industrial equipment

Attention should be given to equipment and components (tanks, racks, pipes, cable trays, supports, cranes,...) when their rupture or falling down can interfere with structures, equipment and components that are safety related (fire fighting, electricity emergency power, communication) or can increase the consequences of the earthquake by blockage of the access or by spilling of flammable, explosive or poisonous materials.

2.5 Lifelines networks

Due to settlement, liquefaction and differential displacements, most of the buried lifelines (water, sewers, electricity, communication ...) were damaged, mainly at their junction with buildings (Koaceli).

Because these earthquakes involved large areas, there was also an important blackout in electric power transmission due to damages to high voltage substation and plant tripping (Chi-Chi, Koaceli).

The generating plants behaved well as well as the transmission towers (except for Chi-Chi, due to ground instability in special locations) and lines but switch yards and in particular porcelain and transformers on poles or on rails suffered large damages (Northridge, Hyogoken, Chi-Chi and Koaceli). This problem is generic and should be addressed by adequate protection (specially designed, anchored and/or isolated equipment).

Telecommunication was also interrupted due to inadequate anchoring of electronic cabinets, collapse of hosting structures or rupture of underground transmission lines (Koaceli).

It is recommended to use seismically rugged equipment to ensure good emergency and back up power generation systems. For the associated networks, it is advised to take into account the consequences of the collapse or the interaction with non safety related and non-seismically designed equipment and structures.

3. CONCLUSIONS AND RECOMMENDATIONS: APPLICATION TO NUCLEAR SEISMIC DESIGN

The distances from the epicenters of the earthquakes to the closest nuclear power plants was larger than 120 km (Chi-Chi, Hyogoken, Northridge) and the amplitudes of the recorded seismic accelerations at the sites remained most often under triggering level and consequently no recording devices were activated neither inspection procedures specific to eartquakes events carried out. No seismic trip systems have been activated during the events

For the Chi-Chi earthquake, the recorded accelerations exceeded the triggering levels at Kuosheng NPP and Chinshan NPP (0.043 g at the reactor building foundation, well under design limits). These units scrammed down because of loss of off-site power. The earthquake emergency operating procedures were followed and these units were restarted after inspection of the plant and permission of the authorities.

The Japanese authorities reevaluated the guidelines applied for the seismic design of nuclear power plant with the information gathered from the Hyogoken earthquake to determine if:

- the method used to determine the design basis earthquake and the associated ground motion was adequate;

- the value of the recommended vertical acceleration was sufficient;
- the evaluation method of the activity of the faults and of the associated magnitude of the near site earthquakes was adequate.

The answers were positive, particularly considering the fact that Japanese nuclear power plants have to be founded on rock.

The codes should give more importance to the site conditions on the definition of the design accelerations and spectra: presence of active faults, the focusing effects, the accelerations greater than expected observed on soft layers in case of nearby earthquakes.

Because the nuclear codes does not allow the use of ductility for seismically categorized structures, the problem of the determination of the real limit values for the structural elements is of less importance, but this should be correctly evaluated for margins evaluations.

More importance should also be given on the construction details and on their check and controls in the field for reinforced concrete elements (splicing, anchorage of rebar, transversal rebar) and for welded beams and columns (weld material, procedures for welding and welders).

These earthquakes have shown the importance of the external power energy network and telecommunication system outside of the plant and the need for a realistic evaluation of their availability in case of high magnitude events. It was demonstrated with those events that outside power might not be back for a long period of time which might exceed safety requirements. Due to certain external conditions specific to the selected site, is the current required period of availability of emergency equipments and systems sufficient enough?

The interaction of non safety related structures, equipment and components with safety related equipment should be examined more accurately. In some circumstances, for non safety related elements, the stability criteria given by the non nuclear seismic codes may not suffice to guarantee serviceability or functionality conditions or any interaction effects.

APPENDIX A

LESSONS LEARNED FROM THE NORTHRIDGE EARTHQUAKE (JANUARY 1994) WITH RESPECT TO DESIGN AND CODE REGULATION ON THE EXAMPLE OF STEEL FRAME BEHAVIOUR

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1. INTRODUCTION

At the last meeting of the Subgroup on the Seismic Behaviour of Structures (SG-SB) in May 2000 /PWG 00/ it was decided to accepted the proposal of Mr. Schulz, chairman of the IAGE Principal Group, to generate a serie of small reports being entitles: “Lessons learned from high magnitudes earthquake with respect to nuclear codes and standards”. The intention of such a series of reports is to give the nuclear engineer some practical examples and insights on the seismic behaviour of structures in case of earthquakes with high magnitudes.

Each report should be directed to a single event and characterise the event with 1 to 2 pages followed by an expert judgement regarding relevant information with respect to design practices, with regard to nuclear design and specific needs for codes and standards (nuclear and non-nuclear) and explain why this is necessary. The single report should have max. 7 - 8 pages.

The following elaboration is a first contribution for the serie mentioned above. It consist of the topics as

- characteristic of the earthquake event
- observed behaviour of steel structures
- reasons for the specific behaviour of the steel structures
- consequences with respect to design and regulation practices of steel structures, which will be discussed in the following sections.

The informations will be keeping conscious short and are focused on the important items, because it do not make sense rewriting what had already been done.

2. CHARACTERISTIC OF THE EARTHQUAKE EVENT

The 1994 Northridge Earthquake was one of the largest earthquake to occur within a major U.S. metropolitan area since the 1906 San Francisco Earthquake /ELI 95/. This earthquake (with a magnitude 6.7) caused considerable damage to industrial facilities, life lines, and industrial buildings located within 40 km of the epicenter. The ground motion at most of the investigated sites had moderate to strong amplitudes which were between 0.35 g and 1.0 g peak ground acceleration (pga). In addition, several sites experienced vertical accelerations equal to or greater than the horizontal accelerations. The relatively high acceleration were due to the type of faulting which caused the earthquake. It has been determined that the earthquake occured on blind thrust fault which had not been identified in many geologic maps. As is typical with many earthquake, the frequency content was broadband between 1 and 10 Hertz. Most recording of the earthquake motion indicate that the duration of strong shaking was about 10 seconds in the epicentral region. Interestingly, many recordings show several acceleration peaks or spikes within the duration of strong shaking. It is important to note, that the seismic motion was not the strongest possible for most of the investigated sites. If a fault with larger seismic potential had moved, or if the duration of strong motion had been greater, there would have been more serious challenges to the variety engineered structures, systems and components (SSCs) within the epicentral region /ELI 95/.

San Onofre Nuclear Generation Station, located about 130 km from epicenter, is estimated to have experienced a pga (horizontal) of less than 0.02 g, and Diablo Canyon Nuclear Power Plant, located about 239 km from the epicenter, is estimated to have experienced a pga of less than 0.01 g. The earthquake caused no damage to these plants.

3. OBSERVED BEHAVIOUR OF THE STEEL STRUCTURES

Since for most structures the maximum seismic design value in building codes was 0.4 g horizontal, the Northridge Earthquake provided an opportunity to assess the performance of variety of engineered structures, systems and components (SSCs) subjected to design level and beyond-design level ground motion /ELI 95/.

The relatively large ground motions during the Northridge Earthquake caused several types of structured damage /ELI 95/. Surprisingly, there was considerable damage of welded steel moment-resisting frames (WSMFs). The frames were designed to withstand large seismic forces on the basis of the assumption that they are capable of extensive yielding and plastic deformation. The intendend plastic deformation consisted of plastic hinges forming in the beams, at their connections to columns. Damage was expected to consist of moderate yielding at the connections and localized buckling at steel elements /IAE 98/, /NRC 97/, /SAC 95/. Instead, the WSMF failures were brittle fractures with unanticipated deformations in girdes, cracking in column panel zones, and fractures in beam-to-column weld connections. Brittle fracture developed in the elastic range of response from flaws in the low toughness weld metal and geometric conditions.
Welded-flange, bolted-web connections of steel beams to steel columns suffered fractures and brittle failures in the joints due to cracks initiating at the welds with typical failure patterns of flange nugget pull-out, column fracture, and shear tab fracture. Federal Emergency Management Agency (FEMA) Publication 267 /SAC 95/ provides a detailed discussion of the WSMF damage (see Appendix the Figures 2-1 to 2-6 from /SAC 95/) and provides interim guidelines for the evaluation, repair, and modification of WSMFs (see also Supplement to FEMA 267 /SAC 97/).

4. REASONS FOR THE SPECIFIC BEHAVIOUR OF THE STEEL STRUCTURE

The information gathered at the sites /ELI 95/, /SAC 97/ and in additional experimental investigation /KAU 96/ provides indications about good and poor design details for the seismic performance of the steel structure such a WSMFs.
There are different reasons for the unexpected damage of WSMFs /KAU 96/, /SAC 95/, /SAC 97/, /NRC 97/.
The post-earthquake investigation /SAC 95/ indicated that **consistently higher yield strength** (25 to 35 percent higher than the minimum specified yield strength) **restricted the girder rotation** at the design moment. Thus, the restrained connections were required to dissipate the large amount of energy associated with the seismic event by fracturing. It was the inability of the girder to rotate that induced large unaccounted - for through - thickness forces in the thick-column flanges of the WSMFs.

The failure analysis have included the most common types of weld and columns fractures. Welded-flange, bolted-web connections in steel frames had potential detailing problems such as stress concentrations at the connection, connection strength less than the member strength, prescribed connection detail with no rotation requirements and a notch effect created by the back-up bar. Cracks in the welds of flange-web connections of steel moment-frame buildings were, in some cases, due to poor weld quality contributing to a lack of fusion between the girder flange and the column. In addition, back-up and dam bars served as crack initiation sites (see Appendix the Figures 2-1 to 2-6 from /SAC 95/).

The examination and testing performed /KAU 96/ revealed that the weld fractures were initiated form **porous weld roots** adjanced back-up bar, inadequate weld root penetration, and **that the fracture toughness of welds** made with E70T-4 weld electrodes used in the connections **was very low**, i. e. < 14 J at 21° C.

The use of the E 70T-4 electrode was associated with the FCAW (flux cored arc welding) process. It use (tensile strength / yield strength: 504/420 MPa and elongation of 22 %) is allowed by the AWS Code. However, the electrode need not be tested for notch toughness!
The performed tests indicated that **the notch toughness of the weld metal** used in the fabrication **had a strong effect** on the performance of the WSMF connections.
Another important aspect, as mentioned at the beginning of this section, **was the wide variation in strength properties of actual yield strengths**. This wide variation makes prediction of connection and frame behaviour difficult. Some have postulated that one of the contributing causes to damage experienced in the Northridge earthquake was inadverent pairing of overly strong beams with average strength columns /SAC 97/. Designer wish, in order to be able to economically design, to have strong column-weak beam criteria condition.
Investigations of structures damaged by the Northridge earthquake found some WSMF connections in which **beam yield strength exceeded column yield strength** despite the opposite intent of the designer. The practice dual certification of commonly used structural steels can result in mean yield strengths that are fifty percent higher than the specified yield (in current practice the specified properties, and controlled by the mills, include minimum yield strength, ultimate tensile strength and minimum elongation). **There are also other kind of imponderables in connection with the material testing:**

- In wide flange sections the tensile test coupons are currently taken from the web. The amount of reduction rolling, finish rolling temperatures and cooling conditions affect the tensile and impact properties in different areas of the member. Typically, the web exhibits about five percent higher strength than the flanges due to faster cooling.

- Coupon size and shape is specified based on the thickness of the material. In thick material, the yield strength will vary through the thickness, as result of cooling rate effects. If the full-thickness speciments are used, as is the practice in most mills, the recorded yield strength will be on average of relatively stronger material at the edges of the thickness and the lower material at the centre.

- Strain rate can affect the strength and elongation values obtained for material. High strain rates result in elevated strength and reduced ductility.

- There are two different ways in which the yield property for structural steel can be measured and reported. These include **yield point** and **yield strength**.
  The material specifications for structural steels typically specify minimum values for **yield point** but do not control **yield strength**.
  Although the yield point is the quantity controlled by the ASTM material specifications, it has little relevance to the plastic moment capacity of a beam section. Plastic section capacity is more closely related to the stress along the lower yield plateau of the typical stress-strain curve for structural steel.

For damaged WSMFs, a number of issues related to connection detailing and weld quality, such a fracture toughness, weld material, welding procedures, weld inspection and welders qualification, were addressed. In summary the factors for specific behaviour of the steel structure (WSMFs) could have been identified as:

(1) higher than specified yield strength of ASTM A36 steel
(2) very low fracture toughness of the weld metal
(3) the fabrication failures and construction inadequacies
(4) lack of adequate through-thickness strength of thick column flanges
(5) imponderables in connection with the material testing

This combination of factors was consistent with fractures that have occurred in other structures in the past, where similar geometric conditions and low toughness weld metal were both present.

5. CONSEQUENCES WITH RESPECT TO DESIGN AND REGULATION PRACTICES OF STEEL STRUCTURES

The consequences which ought to have been drawn, with respect to design and regulation practices of steel structures, results from the already described reasons of the behaviour of these items.

Following recommendations should therefore be put in the regulation, especially into technical specifications:

- realistic lower yield strength value (and not only minimum value for yield point) upon steel specimens should be taken from the edges of the thickness (particular from flange and not from the web) in order to assure a better estimation of plastic moment capacity of a beam section,

- the notch toughness of the welding metal (electrode) should be reasonable high and have to be controlled by test,

- the design practice should be in accordance with a proper and reliable seismic performance and should especially avoid the construction failure mentioned in the previously section. In connection with this the most important items include, e.g.:

  - avoidance of large stress concentrations in the beam-column connection as well as inherent stress risers and notches in zones of high stresses,

  - to avoid the inherent inability of material yield under conditions of high tri-axial restrain such as exist at the centre of the beam flange to column flange joins.

- the quality control and - assurance in the construction and fabrication is a important issue and should be addressed to

  - welding procedures (hydrogen included crack have to be avoided, the cooling rate of the welding material must be bounded) and

  - weld quality and welders qualification (avoidance of porous weld roots as well as lack of fusion between the girder flange and the column).

This recommendations are valid for items in both non-nuclear and nuclear (e.g. support structures) codes and standards.

Briefly, the lesson which could be learned from discussed behaviour of steel structure in case of strong earthquake as well as from the reasons for such behaviour has been that enhanced level of quality control and assurance is needed. This is true particular in such area as: specification of mechanical property got from test specimens of rolled shapes, design of WSMF connections and fabrications procedures.

The most of WSMF failures, however, could have been avoided if the well know technical design, fabrication - as well as quality - control and assurance rules had been pay reasonable attention.

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Paris, 18th - 19th May, 2000

Appendix: Damages of welded steel frames during the Northridge Earthquake according to /SAC 95/

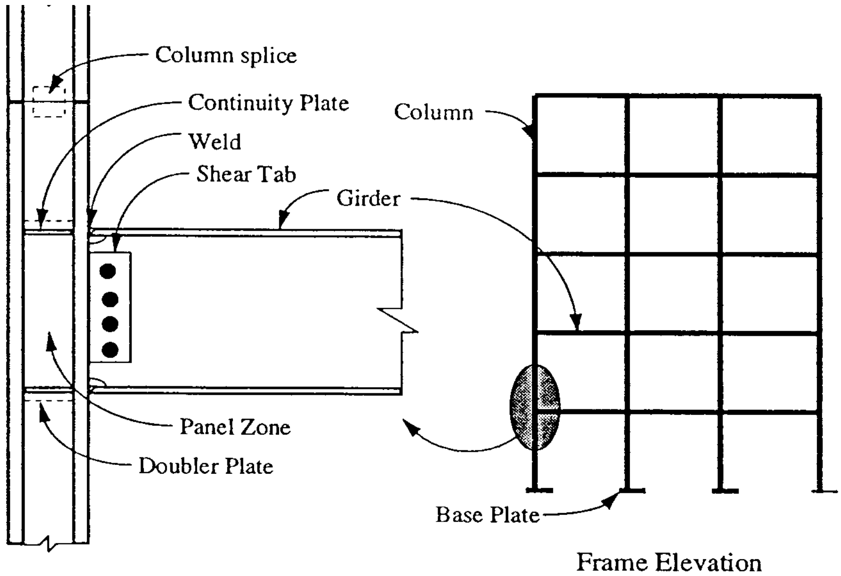
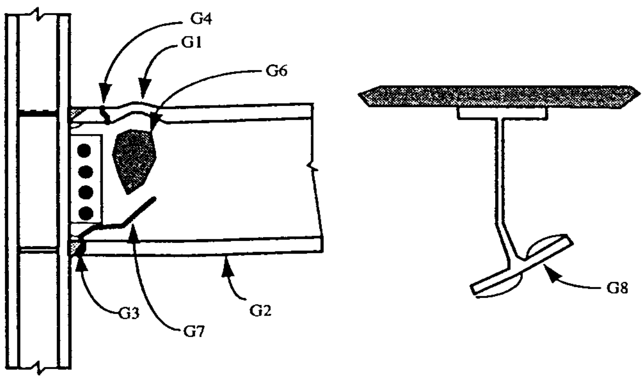


Figure 2-1 - Elements of Welded Steel Moment Frame



Note: condition G5 consists of types G3 and/or G4 damage occurring at both the top and bottom flanges.

Figure 2-2 - Types of Girder Damage

Table 2-1 - Types of Girder Damage

Type	Description
G1	Buckled flange (top or bottom)
G2	Yielded flange (top or bottom)
G3	Flange fracture in HAZ (top or bottom)
G4	Flange fracture outside HAZ (top or bottom)
G5	Flange fracture top and bottom
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsional buckling of section

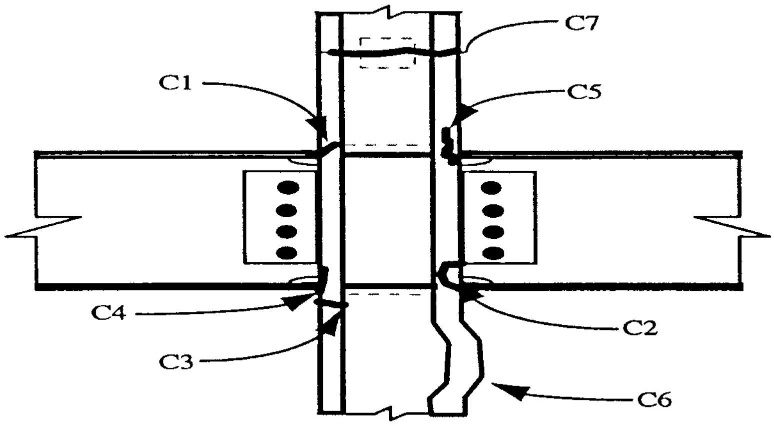


Figure 2-3 - Types of Column Damage

Table 2-2 - Types of Column Damage

Type	Description
C1	Incipient flange crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside HAZ
C4	Full or partial flange in HAZ
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column Splice Failure

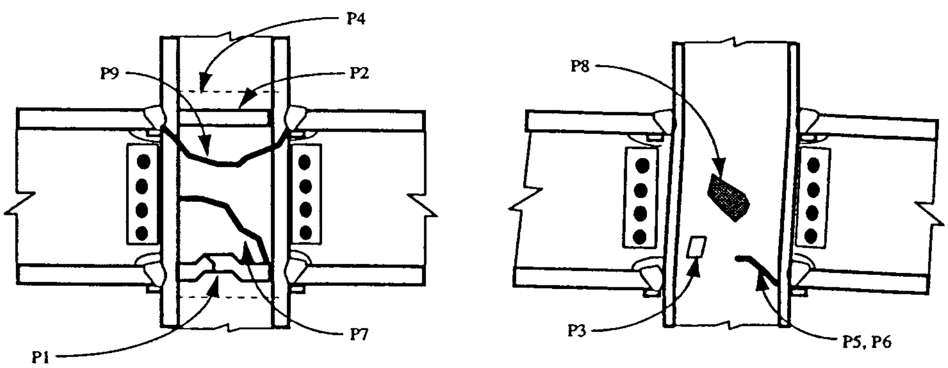


Figure 2-4 - Types of Panel Zone Damage

Table 2-3 - Types of Panel Zone Damage

Type	Description
P1	Fracture, buckle or yield of continuity plate
P2	Fracture in continuity plate welds
P3	Yielding or ductile deformation of web
P4	Fracture of doubler plate welds
P5	Partial depth fracture in doubler plate
P6	Partial depth fracture in web
P7	Full or near depth fracture in web or doubler
P8	Web buckling
P9	Severed column

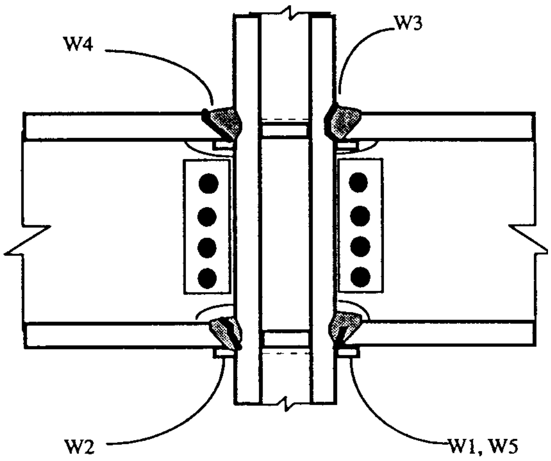


Figure 2-5 - Types of Weld Damage

Table 2-4 - Types of Weld damage, Defects and Discontinuities

Type	Description
W1	Weld root indications
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	UT detectable indication - non-rejectable

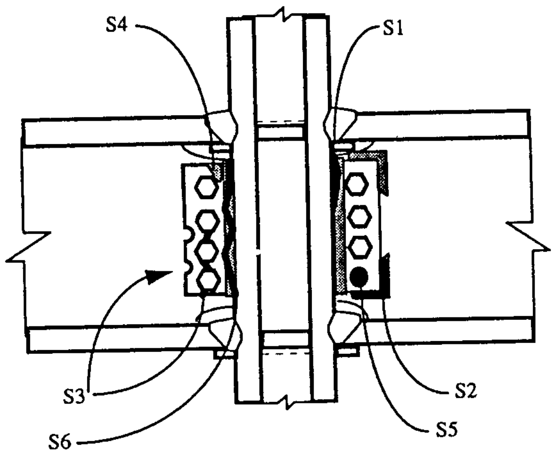


Figure 2-6 - Types of Shear Tab Damage

Table 2-5 - Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S2	Fracture of supplemental weld
S3	Fracture through tab at bolts or severe distortion
S4	Yielding or buckling of tab
S5	Loose, damaged or missing bolt
S6	Full length fracture of weld to column

APPENDIX B

OUTLINE OF
THE REPORT ON THE SEISMIC SAFETY EXAMINATION OF NUCLEAR FACILITIES BASED ON THE 1995 HYOGOKEN-NANBU EARTHQUAKE
(TENTATIVE TRANSLATION)

September 1995

1. INTRODUCTION

From the standpoint of thoroughly confirming the seismic safety of nuclear facilities, Nuclear Safety Commission established an Examination Committee on the Seismic Safety of Nuclear Power Reactor Facilities (hereinafter called Seismic Safety Examination Committee) based on the 1995 Hyogoken-Nanbu Earthquake on January 19, 1995, two days after the occurrence of the earthquake, in order to examine the validity of related guidelines on the seismic design to be used for the safety examination.

This report outlines the results of the examinations by the Seismic Safety Examination Committee.

2. BASIC PRINCIPLE OF EXAMINATIONS AT THE SEISMIC SAFETY EXAMINATION COMMITTEE

In order to proceed the examinations and discussions at the Seismic Safety Examination Committee, it is important to collect as much information as possible from the Hyogoken-Nanbu Earthquake. Thus, tremendous number of reports or related documents prepared by the regulatory bodies, research institutes or academic societies were investigated as well as field investigations were conducted to obtain various information on the Hyogoken-Nanbu Earthquake, namely. earthquake parameters, source mechanism, displacement of fault, earthquake ground motion, and the damages to buildings and civil structures.

Based on the collected data and various information on the Hyogoken-Nanbu Earthquake, which has been understood comprehensively up to now, the items to be examined were selected so that the validity of related guidelines on the seismic design for nuclear facilities were to be examined in detail.

3. OVERVIEW ON THE RELATED GUIDELINES OF THE SEISMIC DESIGN

3.1. Related guidelines of seismic design

Nuclear facilities include light water reactors for power generation, fast breeder reactors, advanced thermal reactors, research reactors, nuclear fuel facilities (nuclear fuel recycling and reprocessing facilities), radioactive waste management facilities, and radioactive waste repository.

With regard to the examination of the seismic design of these facilities, it is specified that the Examination Guideline for Seismic Design of Nuclear Power Reactor Facilities" (hereinafter called "Seismic Design Examination Guideline") is to be used or referred.

3.2. Overview on Seismic Design Examination Guideline

The Seismic Design Examination Guideline was systematically organized in 1978 by integrating the previously used conception on seismic design. Part of this guideline was revised in 1981.

Seismic Design Examination Guideline specifies the basic principle as follows: the nuclear power reactor facilities shall maintain its structural integrity against any hypothetic seismic force likely to occur at the site so that no earthquake brings about a major accident. Moreover buildings and structures shall be, in principle, of rigid construction and the important buildings and structures shall be supported on bedrock. Also it requires such severe seismic design as follows: the facilities of higher degree of importance shall be resistant to stronger seismic force than for usual commercial or industrial buildings and structures, the maximum design earthquakes shall be evaluated based on historical earthquakes, active faults, and vertical seismic force in addition to the horizontal seismic force shall be taken into consideration.

4. INFORMATION AND KNOWLEDGE OBTAINED ON THE 1995 HYOGOKEN-NANBU EARTHQUAKE

With regard to the Hyogoken-Nanbu Earthquake, many scientists, engineers and research institutes have reported investigations and studies. The knowledge that must be referred to examination on the seismic safety of nuclear power reactor facilities can be outlined as follows.

4.1. Parameters and source mechanism of the earthquake

The earthquake parameters of the Hyogoken-Nanbu Earthquake of January 17, 1995 were published.
The origin time: January 17, 1995 at 5:46 a.m.
Location of epicenter: 34 ° 36' N, 135 ° 03' E
Depth of focus: 14 km
Magnitude: 7.2 (The Japan Meteorological Agency(JMA) scale)

The Hyogoken-Nanbu Earthquake has been caused by right-lateral strike-slip displacement of the fault under the east-west compression tectonics. The earthquake accompanied surface rupture (prominent right-lateral strike-slip) on the known Nojima fault, which runs along the north west coast of the northern part of Awaji Island.

The distribution of aftershocks of the Hyogoken-Nanbu Earthquake is almost consistent with a complex system of known active faults (hereinafter called the "Rokko-Awaji fault zone") and extending over the whole fault zone from Rokko to Awaji.

On the other hand aftershocks of the first day after the earthquake (considered to be closely related to the earthquake source mechanism) are estimated to have been distributed in the range of about 40 km around epicenter which is clearly shorter than the entire length, approximately 60km, of Rokko-Awaji fault zone.

From the reasons mentioned above, the Hyogoken-Nanbu Earthquake is supposed to have been generated by the displacement of a part of Rokko-Awaji fault zone.

4.2. The damages of buildings and structures

The Hyogoken-Nanbu Earthquake having the magnitude of M 7.2 is a so-called "just underneath earthquake", which occurred at. the shallow part of the earth crust, and caused severe damages.

The Hyogoken-Nanbu Earthquake caused severe damages to buildings and civil structures, including collapsed, fallen and overturned railroads and overhead bridges on the roads, as well as damaged nearly 400,000 buildings, including wooden houses, buildings of steel construction and reinforced concrete construction.

Particularly some of the steel columns of the steel frame buildings seems to have brittlly fractured. In the coastal areas and the land-filled grounds, soil liquefaction occurred over the wide area. The areas of seismic intensity scale VII (on the JMA scale).are distributed in a belt zone extending from Suma ward of Kobe city to Nishinomiya city. With regard to the belt-shaped distribution of damages, some reports say that the damages were caused by buried faults, but in many reports, they blamed the soil conditions in surface layers for these damages. In the rock areas around the faults, the earthquake ground motions were relatively small. In the further south soil area where damages found concentrated, the earthquake ground ground motions were amplified greatly. Further, it is considered that the coincidence between predominant period of earthquake ground motion and natural period of wooden houses and low-and medium height reinforced concrete buildings caused the damage concentration.

With regard to the cause of the damage of buildings and civil structures, as the investigation proceeds, It was found that the horizontal seismic force rather than the vertical seismic force was predominant factor of the damages.

Many of the damaged buildings of steel frame or reinforced concrete construction were built before the enforcement of the current Building standard law was established in 1981.

4.3. Earthquake ground motion

4.3.1. Observation record of earthquake ground motion

In the Hyogoken-Nanbu Earthquake, a considerable number of strong motion of main shock were recorded in the epicentral area.

However, most of them are on the ground surface, and there is almost no strong motion record on the rock that can be directly comparable with the design basis earthquake ground motion on the free surface of the base stratum specified in Seismic Design Examination Guideline.

In the epicentral area, the strong motions that exceed 500 gal were observed, and the records over 800 gal were obtained at the sites in the central area of Kobe city, including 818 gal at the Kobe Marine Meteorological Observatory.

As for the vertical component, a peak acceleration of 556 gal was observed at the ground surface of Port Island of Kobe (landfill soil site).

4.3.2. Characteristics of earthquake ground motion

As a result of evaluation of maximum amplitude using an empirical distance-attenuation curves to estimate the maximum amplitude of ground motion, it was indicated that the earthquake ground motions are not especially strong compared with ’the past great earthquakes (Fig. 1) There were not a few observed records indicating that the peak acceleration of vertical component of ground motion was more than 0.5 times that of the horizontal component. However, many of these records were obtained on the ground surface near the seashore or of the flood plains site along the river. Thus it was pointed that these sites must have received greater influence of ground surface nonlinearity of surface layer subjected to the strong ground motion.

The frequency characteristics of the Hyogoken-Nanbu Earthquake can be supposed to have a relatively long period of predominant 1 second from the response spectra of earthquake ground motion observed at the Kobe Marine Meteorological Observatory.

5. EXAMINATION OF VALIDITY OF THE GUIDELINES BASED ON VARIOUS INFORMATION OF THE HYOGOKEN-NANBU EARTHQUAKE

5.1. Selection of items to be examined

After studying and examining various factors of the earthquake, main conditions listed include the very strong motion of M 7.2 directly underneath large urban city located along Rokko-Awaji fault zone, observation of strong earthquake ground motion near the fault, and not a few observation data indicating the peak acceleration of vertical component was more than 0.5 times as strong as horizontal component.

Based on these factors mentioned above, the items to be examined can be considered as the following three.

(a) If there is any problem or not in the evaluation method of the design basis earthquake and the earthquake ground motion,
(b) If there is any problem or not in the evaluation method of vertical seismic force, and
(c) If there is any problem or not in the consideration of evaluation method of active fault and magnitude of just underneath earthquake.

5.2. Evaluation of earthquake and ground motion based on the Seismic Design Examination Guideline

In examining the evaluation methods of the earthquake and the earthquake ground motion, it is necessary to set up some proper site. The site conditions must be located near the epicenter, must have obtained time-history records of ground motion, and must be a site not affected significantly by ground surface such as the liquefaction. Taking consideration of the site conditions mentioned above, Kobe University located at RokkoDai-cho in Nada ward of Kobe City, which is close to the epicenter, was selected as the evaluation site.

5.2.1. Evaluation of the earthquake

Magnitude of earthquakes in the Hanshin and Awaji area were estimated based on the studies on the historical earthquakes, active faults, and the seismo-tectonic structure, followed the Seismic Design Examination Guideline. (Table 1.)

As a result, the earthquake that gave the greatest influence to the Kobe University site was determined to be an earthquake of M7 3/u assumed to have occurred in the active fault system ranging from the southeast foots of Rokko Mountains to the northern part of Awaji Island from the standpoint of the seismo-tectonic structure. (Fig. 2)

Because the magnitude of the estimated earthquake is larger than the Hyogoken-Nanbu Earthquake of M7.2, the validity of the evaluation method of earthquake based on the Seismic Design Examination Guideline is concluded to be not impaired even by referring to the Hyogoken-Nanbu Earthquake.

5.2.2. Evaluation of earthquake ground motion

Ground motions (of earthquakes estimated in 10)above) on the free surface of base stratum at Kobe University site are estimated based on the standard method after OHSAKI, and the fault model usually used when the evaluation site is close to the hypocenter. As a result of this estimation, the response spectrum of the earthquake ground motion estimated for Hanshin Awaji area was found to have a larger value than that of observed ground motion at Kobe University site. (Fig. 3)

Although in the range of the long period, some value of the response spectra of ground motions obtained at Kobe University are larger than the estimated ones, the validity of the evaluation method of earthquake ground motion was considered not to be impaired for the following reasons: (a) because Kobe University site is not on hard rocks defined in the Seismic Design Examination Guideline and the influence of the amplification of subsurface layers could be possible, and (b) because nuclear facilities such as the buildings, structures, equipment, and piping systems which are important for the safety, are of rigid structure, as a principle, and their natural periods are designed in the short period range.

5.3. Evaluation of vertical seismic force

5.3.1. Observed vertical ground motions at the Hyogoken-Nanbu Earthquake

An analysis was made on the ratio of the vertical and the horizontal components based on the observed records of 126 sites, excluding those at the landfill soil sites and those supposed to have strong influence of structures. The result of this analysis indicated that the ratio of the peak acceleration amplitudes of vertical and horizontal components was less than 1/2 on an average. (Fig. 4)

5.3.2. Evaluation of the vertical seismic force in the Seismic Design Examination Guideline

The Seismic Design Examination Guideline requires that horizontal seismic forces shall be combined concurrently into most disadvantage mode with vertical seismic force that based on the value of 1/2 of the maximum acceleration amplitude of the basic design earthquake ground motion. In relation to above requirement, the committee investigated the ratio of the acceleration amplitudes of vertical component at the same time when the maximum acceleration of horizontal component occurred, using the data of 23 sites on which the time-history seismic waves were obtained. As a result, the average ratio was about 0.1 and the maximum value was about 0.3. They were much lower than 1/2. (Fig. 5)

In general, it is said that vertical seismic force is not dominant to compare with horizontal seismic force, and the effect of vertical seismic force is to be small in seismic design. Therefore, most seismic design standards, including the Building Standard law, do not specify the provisions for vertical seismic force. With regard to the damages to structures caused by the earthquake, the main factor of these damages by the Hyogoken-Nanbu Earthquake is reported to have been caused by the strong horizontal ,ground motion even though there might have been some influence of vertical motion.

The buildings of nuclear power reactor facilities are thick reinforced concrete shear resistant wall type structure due to the need of radiation shielding and strong horizontal seismic force for design purpose. These buildings are thus of more rigid construction compared with commercial and industrial buildings.

In addition, the vessels including the pressure vessels of nuclear power reactor facilities have high rigidity for vertical direction. The pumps are of rigid construction and the pipings are appropriately supported for vertical as well as oblique directions in addition to horizontal direction so as not to easily vibrate.

For these reasons, nuclear power reactor facilities are constructed with high rigidity especially for vertical direction. Thus the effect of vertical ground motion is considered to be small for nuclear power reactor facilities.

5.3.3. Summary

From the facts mentioned above, the validity of the evaluation of vertical seismic force in the Seismic Design Examination Guideline is concluded not to be impaired even though by referring to the Hyogoken-Nanbu Earthquake.

5.4. Consideration of the evaluation of active faults and magnitude of just underneath earthquake

Among the Rokko-Awaji fault zone, the earthquake return period for the active faults on Kobe side is estimated to be about 2,000 years according to the reports on these active faults. In addition, as a result of the excavation study after the earthquake, the Nojima fault has recorded another previous disturbance after the twelfth century. Because the earthquake return period for these faults is shorter than 50,000 years, the validity of the concept of Seismic Design Examination Guideline, which specifies the evaluation period for active faults as 50,000 years, is concluded not to be impaired by referring to the Hyogoken-Nanbu Earthquake.

Furthermore, the Seismic Design Examination Guideline requires the consideration of magnitude 6.5 (just underneath earthquake) as the design basis earthquake even when no active fault is recognized near the site. Because the Hyogoken-Nanbu-Earthquake (M7.2 just underneath earthquake) occurred in a region where a complex system of active faults had been previously mapped, and the earthquake having the magnitude greater than M7.2 of Hyogoken-Nanbu Earthquake could be supposed to occur from the length of fault zone, any findings is not obtained which impairs the validity of assuming that this kind of just underneath earthquake even by referring to the Hyogoken-Nanbu Earthquake.

5.5. Summary

As a result of understanding the various information of the Hyogoken-Nanbu Earthquake and by examining the matters to be discussed based on the concept of the Seismic Design Examination Guideline in detail and on the basis of knowledge obtained from the Hyogoken-Nanbu Earthquake, it is concluded that the validity of basic guidelines for securing the seismic safety of nuclear facilities of Japan is not impaired.

6. CONCLUSION

The Seismic Design Examination Committee surveyed the related guidelines on seismic design, selected the items to be examined, and examined on those items based on the knowledge obtained from the Hyogoken-Nanbu Earthquake. As a result, the Committee confirmed that the validity of the guidelines regulating the seismic design of nuclear facilities is not impaired even though on the basis of the Hyogoken-Nanbu Earthquake.

However, the people related to the nuclear facilities may not be content with the above result, but continuously put efforts in doing the following matters to improve furthermore the reliability of seismic design of nuclear facilities by always reflecting the latest knowledge on the seismic design.

(1) The people related to nuclear facilities must seriously accept the fact that valuable knowledge could be obtained from the Hyogoken-Nanbu Earthquake, try to study and analyze the obtained data, and reflect the results of investigations, studies, and examinations conducted appropriately to the seismic design of nuclear facilities referring to the investigations and studies of related academic societies.

(2) Research and test investigations are to be performed to further enhance the seismic design.

(3) Proving demonstration of seismic resistance of nuclear facilities are needed all the more.

(4) The information on seismic design must be provided to public in general and efforts must be put to the international exchange as well as to the joint study of international basis.

It goes without saying that it is important for the nuclear facilities to obtain public acceptance on the safety of facilities as well as to secure seismic design and sufficient safety standard. The people related to nuclear facilities are requested to put incessant efforts to the seismic safety of nuclear facilities and to develop much more national reliance.

Table 1 Earthquakes assumed around Kobe area

Note: X: Distance from hypocenter (in km)

	Assumed earthquake			
Type	**Symbol**	**Magnitude** (M)	**Distance From Epicenter** (km)	**Remark**
Historical Earthquake	E-1 E-2	7.5 8.4	34 180	* Earthquake in Kyoto and Kinai in 1596 * Earthquake in Goki and Shichido in 1707 .
Active .Fault	F-1 F-2	7.7 7.6	16 25	* Earthquake caused by the active fault system extending from the south-east foots of Rokko Mountains to the northern part of Awaji Island * Earthquake caused by the Arima-Takatsuki tectonic line
Seismo-Tectonic Structure	T-1 T-2	7 3/4 8 1/2	16 180	* Intra-plate earthquake ..* Inter-plate earthquake related to the Philippine Sea Plate ’
Just Underneath earthquake	N	6.5	- (X=10)	Just underneath earthquake’

APPENDIX C

SHORT REPORT ON LESSONS LEARNED FROM KOCAELI 1999, AUGUST 17TH HIGH MAGNITUDE EARTHQUAKE

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1. INTRODUCTION

This report is prepared as a contribution to the work of OECD IAGE seismic working group. It is part of a series of shorts reports aiming at sharing the lessons learned from the recent major damaging earthquakes. The ultimate goal of this work is to make sure that current and future design rules adequately ensure the safety of nuclear facilities towards recent lessons about seismic events.

The authors of this report were members of the French post-earthquake field reconnaissance team after of the Kocaeli earthquake of August 17th, 1999 ([1]).

The present reports also takes advantage of the extensive review published in [2].

2. SISMICITY, FAULT RUPTURE AND TSUNAMI

The Kocaeli earthquake is one of the most destructive recent EQ to strike Turkey. It occurred at 3.02 am (local time) on August 17, 1999 a time when most residents were asleep. Its magnitude was 7.4 (moment scale). Its focal depth was 17 km. A second event (Mw = 7.1) took place on November 12, 1999, in the east part of the first event faulting, increasing the damages in this area.

The event resulted from the relative movement of the Eurasian plate and the Arabian plate, forcing a westwards translation of Anatolian plate along the North Anatolian fault. It is considered a member of a series of events continuing the westwards propagation of sliding along the Anatolian fault, marked by a group of strong earthquakes since 1939.

It generated a surface rupture of about 120 km length from Lake Eften to The Hezek peninsula in Izmit Bay.

The surface rupture was a right-lateral strike-slip motion. The average slip length was about 1 meter. In several places the surface slip length was about 4 meters, with a peak value of about 5.5 meters.

Along the Izmit bay south side sea-shore, a wide area was submersed by the sea, as a consequence of a pull-apart phenomenon, leading to <u>subsidence of about 2.5 meters</u>. As a consequence, <u>complete parts of constructed city areas</u> were invaded by 2.5 meters depth of permanent sea water.

Along the surface faulting many building were obviously damaged, but, more unexpectedly, <u>numerous buildings immediately adjacent to the faulting suffered little or no damage at all</u>.

In some places, the faulting was distributed over a wide zone, with no well defined fault trace. Damage due to surface fault rupture appeared in these areas, but the damage tended to be much less severe.

The surface rupture occurred along a previously known active fault. In most places, the main fault traces could have been identified prior to the earthquake, based on detailed mapping and subsurface investigations.

The evidence of a potentially active fault must be taken into account in design of buildings and lifelines for which it is necessary. The following typical recommendations were expressed in [2] concerning the specific case of the Izmit area faulting :

- impose stronger foundation in areas with distributed faulting areas
- limit the height of building to one or two stories where the fault traces are not well defined
- for lifeline networks in the faulting areas, define emergency procedures to maintain functionality (proper placement of cutoff valves…)

A Tsunami wave was generated, more probably by subsidence along shoreline and underwater slope sliding South and North of the bay: evidence of the water elevation is witnessed by debris along the shore. In accordance with reported witnesses, the water movement had a period close to 1 minute corresponding probably to the natural period of the bay in the North South direction. Whereas no damaging effects was observed on the structures, the possibility of flooding extensive populated areas along the sea has to be taken into account in mitigation programs of the Izmit bay area

3. GROUND MOTIONS, SITE EFFECTS

Ground motion

A large amount of ground motion was collected during main shock and also during aftershocks. These data provide a substantial contribution to the understanding of short distance effects in large magnitude earthquakes.

The intensity of the ground motion expressed in acceleration in the vicinity of the fault rupture appeared to be less important than predicted by current attenuation relations.

The following effects were observed :

- rupture directivity effects associated to site response led to increase of energy content for longer periods.
- Permanent displacement was recorded in the near field area
- Intermediate field ground motions were bounded by the +/- 2 standard deviations of available attenuation laws.
- Rock sites fell well below median predictions, while most soil sites revealed amplification in the low frequency range, leading to levels above the median prediction
- In general acceleration response spectra were exceeded significantly by the UBC (ICBO 1997) design code.

Site effects

The heavy damage was concentrated in areas underlained by alluvium (Adapazari, Golçük, Degirmendere…). In Düzce, surprisingly, the damage was sparse. This can be related to the short duration of the motion in this area of fault rupture beginning process.

It should be noted that 80 km from the fault rupture, in Avcilar section of Istanbul, soft soils increased the level of ground shaking leading to enhanced damage.

Site effects were measured in Adapazari site during aftershocks by comparing the acceleration obtained on a rock site and on an soft soil site, revealing site amplification factor of 10 in the frequency range 1 to 4 Hz at rock PGA levels of about 0.05 g.

In addition engineering damaging indicators computed on a set of soil and rock measured signals confirm the amplification by soil sites.

Some directivity effects of the faulting displacement process could be responsible for the fact that, in Düzce, the second event (Mw = 7.1, November 12, 1999) provided strong motion more important that those resulting from the August 17 the event. Movement perpendicular to the fault seem to be the most important in median part of the fault. Whereas some influence of the direction versus the fault direction could be observed, it is difficult to anticipate directivity effect. Consequently, for design, an enveloping provision has to be done.

To improve the benefit of the recorded events some additional site investigation should be undertaken to describe more precisely the soil conditions in each measurement location : soil layering, influence of building adjacent to measurement devices.

Geotechnical effects, liquefaction.

Additionally to subsidence along Izmit bay the following findings are related to soil behaviour. Extensive liquefaction was observed in Adapazari and associated with the high level of damage to city buildings.

Damage was specifically most severe in the Hollocen basin portions of the city where loose sandy materials and soft clays were supporting the buildings. By contrast, there was relatively little ground failure and less structural damage in hilly areas of the city. The main shock was recorded in one story building on a stiff soil (Vs = 300m/s) at a level of about 0.3 to 0.5 g. 3 aftershock of about 0.1 to 0.16g were also recorded within one hour and could have contributed to the damage level.

The observed damages to buildings in Adapazari were the following :

- 20% of reinforced concrete building and 56 % of timber/brick buildings were severely damaged or destroyed/collapsed
- tilting, overturning or lateral translation of buildings
- bulging of the ground surrounding the buildings
- bearing failure of the ground and settlement of the buildings

The high level of damage can be associated to the following causes : liquefied or softened surface soil layers at the interface with building foundations, combined with basin effects.

Adapazari area remains an unique field for studying and understanding the site effects and the geotechnics limits of poor soil conditions. Research should be supported to clarify the physics of site effects in this area and raise criteria and mitigation rules for design codes.

Site improvement efficiency towards seismic hazard has been investigated :

- sites where soil treatment was used experimented less damage
- jet-grouted and stone column was revealed as effective to prevent liquefaction related damage

Five investigated dams displayed either no damage or very minor cracking : Kirazdere earth dam which is located near the epicenter, probably experienced 0.4 g with no damage.

4. DAMAGE TO BUILDINGS

Soft stories, P-Δ effect, and non uniform position and random behaviour of masonry infills, were found to be a common cause of observed damage.

The typical RC frame buildings filled with masonry blocks were proven to be quite vulnerable. The Regulations were rarely supplied.

The inherent strength of the masonry was often adequate to prevent substantial damage to one and two-story buildings.

The columns failures were attributed to the following causes

- beams strong enough to force the hinging to take place in the column
- non-sufficient lateral strength to limit ductility demands
- lack of sufficient transverse reinforcement to provide the hinge zone sufficient deformation capacity
- lap slices located just above the floors
- poorly confined concrete and inadequate length lap splices.

Damage to pre-cast concrete construction seemed related to flexibility of the roof diaphragm and the absence of complete lateral load path to the foundation.

Improvement in the building codes in 1975 and 1998 did not necessarily lead in practice to improved building design and details. It is not sufficient to improve the building codes in highest seismic zones to address deficiencies in existing construction.

There is a need in such situation for clear ready to use procedures to diagnose which buildings are still to be considered safe and functional.

Further the need is to be able to select cost effective retrofit techniques.

5. DAMAGE TO LIFELINES NETWORKS

The water, gas, electric power and telecommunications networks have been partially damaged by the EQ.

The most important damage was caused to buried water supply network in Adapazari, due to soil liquefaction and softening and related differential displacements . Brittle cement pipelines were disengaged and fractured. The entire distribution system was planned for replacement. In contrast, the modern water treatment plants sited on firm ground, and storage facilities sustained only minor and easily repaired damage.

There was also a countrywide blackout of the electric power transmission system due to high voltage substation damage and power plant tripping off. But also extensive damage occurred at several 380 kV stations, the power was re-routed by 380 kV transmission lines that bypassed the damaged stations permitting quick restoration of customer services except in areas of severe urban damage. The main observed behaviour and damages were relatively classical :

- generating plant generally behave well provided they are correctly founded
- transmission towers and lines are highly resistant even when displaced by surface fault rupture
- porcelain insulators used in high voltage are generally vulnerable
- unanchored equipment is seismically vulnerable, especially transformers on rails, and pole mounted transformers
- distribution network was damaged by falling of building, by failure of pole mounted transformers due to failure of transformer anchors or breaking of supporting piles caused by shaking or ground liquefaction.
- damage to underground cables are caused by the relative displacement of the building they are connected with. Such damage lead to long delays of restoration due to the relative difficulty in repairing underground cables compared to surface lines.

The natural gas distribution system in Izmit made of modern polyethylene piping was only damaged in locations where building collapsed.

The telecommunication interruption ranged from 3 hours to 3 days after the EQ. This performance was below expectation due to poor seismic protection of the backup power systems in central offices. Common classical cellular phones were not usable. Therefore, it has to be stressed that for major safety concern, only satellite transmission proved to be efficient during the very first hours following the event. The lessons learned are the following :

- hosting buildings should meet high standards for good construction practice, ensuring stability of the building and absence of brick wall falling on vital devices.
- adequate anchorage of electronic devices is a good means of seismic protection
- non network equipment should anchored or located away from the network equipment
- batteries should be tied together and separed by foam sheets to prevent shocks, and the supporting platform should be anchored to the floor
- the whole backup power generation system should be seismically designed including the generator, day tank, cooling lines, oil lines, exhaust systems, starter battery, and control system.

6. DAMAGE TO INDUSTRIAL FACILITIES

Two principal keys are examined : waterfront structures and industrial facilities

The Izmit bay is in an important industrial area. The epicentral area is home of about 40% of Turkeys heavy industry. Many industrial facilities were strongly dependent on their waterfront.

The main industrial facilities in the epicentral areas were petroleum plants (Tupras petroleum refinery, ISGAS fertilizer plant, Petkim petrochemical plant, BP petroleum plant, LPG plants,…), car industry factories (Hyundai, Toyota, Ford, Pirelli, Goodyear, BriSA, KordSA, DuSA …), electricity power plants, papermills, cement plants, pipe factories…

The most common damages to waterfront structures were the following :

- partial or complete collapse of piers making pipelines unusable, and preventing usual quay boarding operations,
- settlement of piers, leading to loss of functionality of quay cranes,

Additionally to loss of functionality of waterfront structures, the following usual damages to industrial facilities were observed :

- partial collapse of office buildings
- collapse of storage steel frame racks
- collapse of roofs due to differential displacement of supporting structures, collapse of precast concrete roofs
- collapse of cement tanks
- damage caused by collapse on non-reinforced brick walls
- transformers sliding and overturning
- falling of complete series of unanchored electrical cabinets
- falling of PC monitors
- buckling of steel leg leading to collapse of crane
- leakage of poisonous liquids at large storage tanks nozzles

Specific damage was observed at Tüpras refinery. The falling of a tall exhaust chimney, coupled with the collapse of fire water pipelines, made the fighting against fire very difficult and induced extended destruction of the plant due to long lasting (several days) fire including collapse of transformation stacks, collapse of burned tanks... The fire involved several large capacity storage tanks, for which the ignition process is not well known (propagation of fires along the piping, floating roof friction…).

It has to be noted several instances of good functioning of diesel emergency power supply systems

The industrial facilities had higher levels of engineering and much better quality control than the residential and commercial construction. However the observed damages was much more severe than usually observed in EQ with similar pga levels. This has probably to be related to the motion long period and long duration.

However the observed damages and induced situation are believed to be those which can be commonly encountered during EQ of these magnitude. The related hazards are typically those which might be faced in such situations : emergency response, fires, vital damages caused by falling of non critical structures, …

7. TRANSPORTATION SYSTEMS

The damages to the railway system were relatively low because quite no trains were in service early in the morning, and no train could run due to the power failure.

The main damages to highways and railroads were caused by surface faulting and settlement :

- S shape deformation or fracture of railways
- collapse of a bridge due to surface faulting
- collapse of the roof of a toll plaza due to poor shear reinforcement and damage to 5 other plazas

In general, despite short bearing seats, bridges unseating damage was relatively sparse, probably due to good longitudinal restraint associated with low induced displacement.

8. CONCLUSION

Kocaeli earthquake has killed about 20 to 30 thousands people and damaged a large number of industrial facilities around the Izmit gulf which concentrate 40 % of industrial capacity of Turkey.

The following conclusions are applicable to seismic design. They concern surface faulting, ground motion, site effects, building seismic design and retrofit, electrical networks and buried piping, and behaviour of industrial facilities.

1 Surface faulting, waterfront areas

Surface active faults should be identified. Construction should be avoided where surface active faults can be precisely located. More generally; the evidence of a potentially active fault must be taken into account in design of buildings and lifelines for which it is necessary.

In areas of potential subsidence (soft soils around gulfs…) the possibility of flooding extensive areas should be taken into account in mitigation programs for both industrial facilities and population protection.

2 Ground motion and site effects

The preliminary scaling of measured strong ground motion reveals that current attenuation laws largely overestimate the EQ motion at rock sites, whereas measurements on soft soils tend to give amplified motions right in the uncertainty ranges of attenuation laws. Globally the low frequency content was specifically important. These observations are consistent with recently obtained large magnitude EQ near field characteristics.

The particular observations made in Adapazari and Yalova area highlight the importance of site effects : amplification of the incident motion by basin reflections and soft soil-columns behaviour, and associated low geotechnical limits lead to instability of foundations and widespread collapse of buildings. Site effects could induce a significant amplification factor of ten as observed during aftershocks with relatively high level ground motions (rock soil pga of 0.05g).

The already collected data should be given additional support to complete the characterisation of soils at the precise location where the ground motion where measured. Development of site effects prediction methods should be encouraged.

3 Damage to buildings

Many RC frame filled with masonry blocks were damaged due to poor detailing constructional rules for steel arrangement, resistance to lateral forces, location of lap splices, concrete confinement…

Despite successive improvement of construction codes in the past decades, recent buildings did not demonstrate significantly improved behaviour. Therefore, in addition to improvement of construction codes, seismic design should include both appropriate design codes and appropriate construction controls from the design stage to the stage when building become functional. Extensive professional technical training should be encouraged.

The extensive damages to buildings after Kocaeli EQ raised the need for rules usable in post-earthquake inspection and retrofit process : ready-to-use procedures to diagnose which buildings are to be considered safe and functional, catalogues of cost effective retrofit techniques.

4 Damage to electrical network and buried networks

The loss of power supply was experienced as immediate initiator after the Kocaeli EQ. It was combined with the loss of telecommunications for a couple of hours due to inadequate design of the central offices emergency power supply systems.

A seismically rugged emergency power system implies that the whole system is seismically designed : the diesel generator, the related support systems, the control systems, the lines connecting them to the generator, the building hosting functional systems, the batteries...

Whereas such systems appeared to be functioning well in several instances of industrial facilities it was not completely demonstrative for telecommunication central offices.

The extensive damage to water supply network in Adapazari highlights the importance of choosing sufficiently ductile materials for buried pipes when EQ soil deformation/liquefaction are expected. Especially the connection with the buildings needs adequate design detailing to adapt the deformation imposed by the building behaviour.

5 Damage to industrial facilities

The Izmit bay area is hosting about 40% of Turkey industrial activity.

A common mode affecting most of the industrial facilities is the loss of functionality of their waterfront structures (submersed pipelines, impossibility to perform quay boarding operations), due to peer collapses.

Industrial facilities also suffered more usual damages in EQ of this magnitude : collapse of brick walls on essential devices, sliding and overturning of unanchored transformers, falling of complete ranges of unanchored electrical cabinets, falling of pc monitors, collapse of steel racks or cranes due to buckling of steel members, vulnerabilities of large storage tanks…

The collapse of the 120m high chimney on TUPRAS refinery stack possibly initiated long lasting fires throughout the site. It stressed the importance of considering the seismic safety as a complete system approach including the potential falling of non safety equipment or structure.

There is a need of expertise and rules to drive the process of appreciating the remaining ruggedness of existing building, and to select adequate retrofit techniques for damaged ones.

9. APPLICABILITY OF THE CONCLUSIONS TO NUCLEAR DESIGN AND SAFETY

All the conclusions could be applied in nuclear design and safety as summarised hereafter.

The evidence of a potentially active fault must be taken into account.

Building design should follow adequate construction codes with a specific care on construction detailing and relevant control procedures.

Site effects on soft soils sites should be evaluated and taken into account in the design. A specific effort should be made to improve the prediction methods.

The Kocaeli EQ revealed many instances of typical already well known failure modes of industrial equipment which should be avoided by appropriate design rules.

Not only the main structures and equipment should be seismically designed. Seismic safety has to be considered as a whole, involving essential support systems, and non-safety system the failure of which would impact the functionality of safety systems or equipment ("seismic interaction" concern).

Transmission networks and underground lifelines were unexpectedly largely damaged during Kocaeli earthquake. This should be addressed correctly to ensure the efficiency of immediate post-earthquake reactions.

10. REFERENCES

[1] French Association for Earthquake Engineering. "Le séisme de KOCAELI (Izmit-Turquie) du 17 août 1999. Rapport de mission".

[2] Earthquake Spectra, Supplement to Volume 16. "1999 Kocaeli, Turkey, Earthquake Reconnaissance Report"

SELECTION OF PICTURES

Surface faulting permanent displacements

Submersed areas along Izmit bay

Damage to waterfront piers


Extensive damage to buildings in Adapazari (left), poorly confined reinforcement (right)

Typical liquefaction effects : overturned building (left), settlement of buildings (right)


Rugged high voltage pylon (left), damage to electrical network (right)


Typical precast concrete, poorly tied warehouse (left), damaged oxygen silos supports (right)

TUPRAS refinery broken chimney (left), burned and collapsed tanks (right)

APPENDIX D.

CHI-CHI (TAIWAN) EARTHQUAKE OF SEPTEMBER 21, 1999

FINDINGS FOR THE NUCLEAR INSTALLATIONS IN SWITZERLAND: SHORT REPORT

Federal Office of Energy (Switzerland)
Swiss Federal Nuclear Safety Inspectorate

1. INTRODUCTION

The Swiss Federal Nuclear Safety Inspectorate (HSK) has a great interest to be rapidly informed and documented on relevant consequences of major earthquakes in industrialized countries in the world, in particular with respect to nuclear installations. The main goal is to recognize within a short time after such events, if measures should be taken at the Swiss nuclear installations.

Therefore, Basler & Hofmann have prepared a report on the Chi-Chi (1999) earthquake for HSK within a few weeks after the event. The information was obtained primarily from the Internet and by directly contacting local authorities and other selected organisations in Taiwan.

This short report is a summary representation of the HSK report of November 23, 1999 [1] (in German). No updated or additional information on the earthquake is included herein. The summary concentrates on the findings for the Swiss nuclear installations.

2. SEISMOTECTONICS AND DAMAGE

The Chi-Chi earthquake with its epicenter in central Taiwan had a moment magnitude of 7.6 and a thrust mechanism within a compressional stress field. Most of the past strong earthquakes occurred along or beyond the eastern coast of the island and at larger focal depths. Therefore, the damage effects of this event were unusually high.

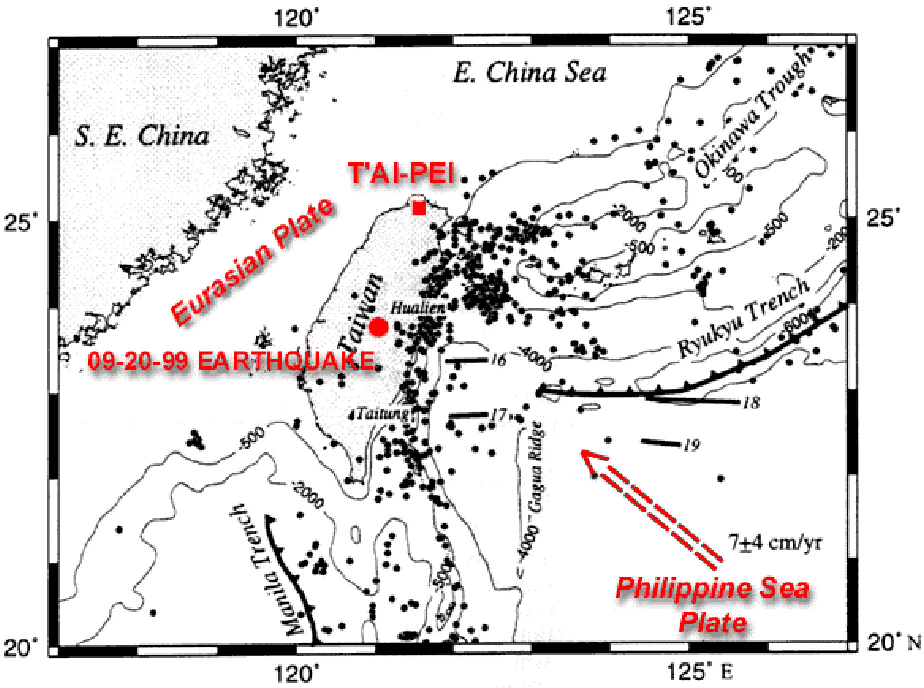

Figure 1 Seismotectonic situation [2]

Damage extent

The Chi-Chi earthquake was considerably stronger than the design earthquake of the local building codes. The damage at 17'500 buildings with more than 2350 deaths, 10'000 injured and 100'000 homeless persons was mainly caused in the epicentral district of Nantou and in three western coastal districts. Important damage was caused also by the very large relative ground displacements (up to 8 m) along known tectonic faults (in particular the Chelongpu fault).

Damage at industrial facilities

At a distance of about 100 km from the epicenter, in the Science-Based Industrial Park (important semiconductor wafer production), the earthquake caused damage at installations, but the main problem was the business interruption with enormous losses, due to lack of electric power for one week. Other interesting earthquake effects were damaged storage tanks and the relatively low consequences at three hydraulic power generation plants situated within only 15 km from the epicenter (with maximum accelerations of about 0.5 g).

Damage on lifelines

Important damage on hospitals, bridges, dams and on electricity and drinking water distribution systems has occurred.

Of special interest are the failures on electricity systems: Several 345 kV transmission towers failed due to ground instabilities at exposed positions. In the switch gears, damage on 53 transformers, on 46 lightning protectors, on many ceramic insulator columns, bushings, buses, and on other installations are reported. As an example, Figure 2 shows the failure of protective panels.

Figure 2 Toppled protective panels in the main control building of the 345 kV Chungliao substation near the epicenter of the earthquake [3]

Geotechnical effects

Very large, mainly vertical ground displacements occurred along the activated fault zone. A 5 m high water fall was created near the destroyed Shih-kang concrete water retaining and bridge structure. Large and numerous slope failures and rock falls occurred in the epicentral region. Soil liquefaction accompanied by large settlements resulted from the earthquake at the harbor of Taichung and on hundreds of buildings and on bridge foundations.

3. BEHAVIOUR OF THE NUCLEAR INSTALLATIONS

Overview of installations

The nuclear power plants in Taiwan (Table 1) are situated at the seashore and are cooled by sea water. Their seismic design is based on U.S. regulatory documents. The SSE design base accelerations are 0.3 g for the Chinshan site and 0.4 g for the other sites (Kuosheng, Maanshan, Lungmen). All sites have rock conditions and are protected against 5 m high Tsunami waves.

Name, Location	Reactor type, net power capacity	Reactor system	First concrete	Start of operation	Status during earthquake of 21.9.1999	Distance from epicenter of Chi-Chi earthquake
Chinshan 1 North	BWR-Mark I 636 MWe	GE	06/1972	12/1978	refueling outage	175 km
Chinshan 2 North	BWR-Mark I 636 MWe	GE	12/1973	07/1979	in operation	175 km
Kuosheng 1 North	BWR-Mark III 985 MWe	GE	11/1975	12/1981	in operation	175 km
Kuosheng 2 North	BWR-Mark III 985 MWe	GE	03/1976	03/1983	in operation	175 km
Maanshan 1 South	PWR 951 MWe	Westing-house	08/1978	07/1984	in operation	215 km
Maanshan 2 South	PWR 951 MWe	Westing-house	02/1979	05/1985	in operation	215 km
Lungmen 1 North	ABWR 1355 MWe	GE	1996	planned 2003	under construction	175 km
Lungmen 2 North	ABWR 1355 MWe	GE	1996	planned 2004	under construction	175 km
Lan Yu Island South	Intermediate storage facility	-			in operation	215 km
Research center INER Lungtan, North	Heavy water 40 MWe	Canada		70-ies		120 km

Table 1 Nuclear installations in Taiwan and distance from epicenter

Effects at the plant sites

The earthquake caused no damage to building structures and components at the nuclear power plants, since the distance from the epicenter of the earthquake was at least 175 km. However, at the Lungmen site, several erection cranes toppled over. At Chinshan I npp, the earthquake caused water to spill out of the reactor cavity and out of the equipment hatch. The nuclear research facilities south of Taipei remained without damage.

The seismic trip was not active during the event. However, the units Chinshan II, Kuosheng I and Kousheng II scrammed on loss of offsite power. The emergency diesel generators kicked in within 10 s and provided the power for the safety-related systems.

Strong-motion recordings

Taiwan has the densest strong-motion network in the world (more than 700 free-field sites). Within four weeks after the Chi-Chi earthquake, about 10'000 digital acceleration time-histories were recorded.

The seismic instrumentation at the Chinshan site recorded a maximum peak ground acceleration of 0.037 g. At the Kuosheng site, the free-field instrument failed, whereas the maximum acceleration at the containment foundation was 0.043 g. The Maanshan site instrumentation was not triggered.

In the epicentral region, ground accelerations up to about 0.6 g were recorded, in particular near the activated Chelungpu fault. Figure 3 shows the maximum East-West acceleration plot of the main shock.

Reactions by plant staff and authorities

The earthquake was weakly felt by the operators in the control room of Kuosheng npp. In the morning following the event, the plant engineers of Kuosheng and Chinshan inspected the most important structures, systems and components. After that action, the authority gave the permission to restart the units. Later-on, the authority had to respond to political rushes. The earthquake emergency operating procedures of these plants were used for the first time in this earthquake.

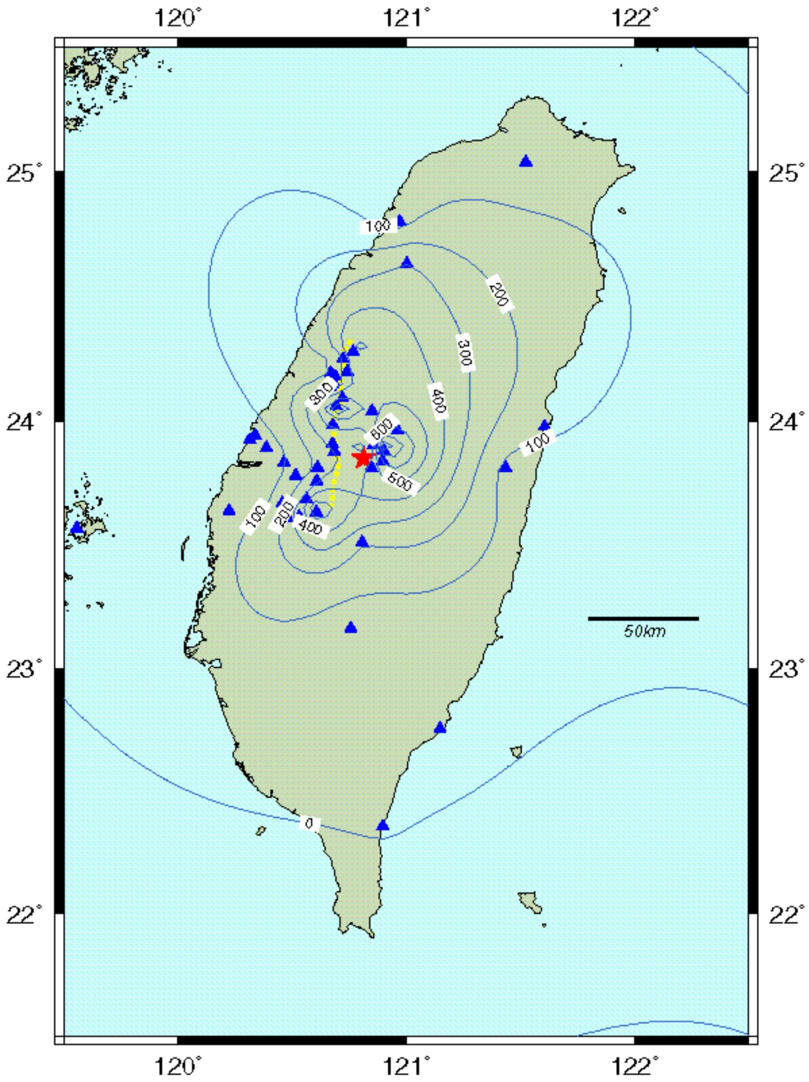

Figure 3Plot of maximum East-West accelerations of the main shock (unit: cm/s^2) [4]

4. CONCLUSIONS FOR THE SWISS NUCLEAR INSTALLATIONS

Seismotectonic aspects

Compared with Taiwan, Switzerland is situated in a seismically much less active region. However, similarities exist with respect to the focal mechanism (thrust), the compressional stress regime and the low focal depth. Thus, the pattern of the isoseismal lines and the maximum acceleration values (Figure 3) would be similar in Switzerland in the case of an equally strong event.

Maximum Magnitudes

Such a strong event has been considered as unrealistic to occur in Switzerland so far. In the PSA studies for the Swiss nuclear power plants, a maximum magnitude up to 7.5 was considered, but only with a low weight. Under the assumption of a magnitude 7.6 event in the seismically most active region in Switzerland (Valais), the SSE accelerations at the closest site (Mühleberg npp) could probably be exceeded.

Conclusions for components

Most effects at components observed during the Chi-Chi earthquake (at non-nuclear sites) are already known to HSK and they have been considered systematically at the Swiss npps. A rather sensitive point is the possible loss of power from emergency diesel generators, as observed at non-nuclear sites in Taiwan.

Problems caused by electricity distribution system

The problems with the electricity distribution system would be smaller in Switzerland than in Taiwan because the density of the grid is higher in Switzerland and connected to the neighbour countries. Furthermore, the Swiss npp sites are located within the major electricity user centers.

However, similar damage at the Swiss distribution system (transmission towers, switch gears, substations) is probable in a strong earthquake and can cause a sudden loss of power. Provident reflections would be beneficial in order to reduce the possibility of the trip of a npp caused by such external problems.

Strong-motion records

The evaluation of the strong-motion records at the Taiwanese npp sites took several weeks. This could be avoided by the use of a modern instrumentation and evaluation procedure. In Switzerland, not all npps are prepared in a best possible way in this respect, too. The large amount of records from the modern Taiwanese free-field network might be very useful for many regions in the world.

Information and response

The modern technologies allow a rapid worldwide information exchange after an earthquake. It is important for a regulatory authority to be well prepared to respond appropriately within a short time after a major event, in particular to the public media. The favourable site locations of the npps with respect to the epicenter of the Chi-Chi earthquake and the rapid information on maximum accelerations had a positive effect in this case.

5. REFERENCES

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