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NUCLEAR POWER PLANT CONCRETE STRUCTURES

Basu Prabir C.[1], Labbé Pierre[2], Naus Dan J.[3]

[1]Formerly in Atomic Energy Regulatory Board, India, and International Seismic Safety Centre, IAEA, Vienna
[2]Electricité de France, France
[3]Oak Ridge National Laboratory, USA

ABSTRACT

A nuclear power plant (NPP) involves complex engineering structures that are significant items of the structures, systems and components (SSC) important to the safe and reliable operation of the NPP. Concrete is the commonly used civil engineering construction material in the nuclear industry because of a number of advantageous properties. The NPP concrete structures underwent a great degree of evolution, since the commissioning of first NPP in early 1960. The increasing concern with time related to safety of the public and environment, and degradation of concrete structures due to ageing related phenomena are the driving forces for such evolution. The concrete technology underwent rapid development with the advent of chemical admixtures of plasticizer/super plasticizer category as well as viscosity modifiers and mineral admixtures like fly ash and silica fume. Application of high performance concrete (HPC) developed with chemical and mineral admixtures has been witnessed in the construction of NPP structures. Along with the beneficial effect, the use of admixtures in concrete has posed a number of challenges as well in design and construction. This along with the prospect of continuing operation beyond design life, especially after 60 years, the impact of extreme natural events (as in the case of Fukushima NPP accident) and human induced events (e.g. commercial aircraft crash like the event of September 11th 2001) has led to further development in the area of NPP concrete structures. The present paper aims at providing an account of evolution of NPP concrete structures in last two decades by summarizing the development in the areas of concrete technology and construction techniques, design methodology, maintenance and ageing management of concrete structures.

INTRODUCTION

The first nuclear power plant (NPP) was commissioned in early 1960s; construction of which might have started in middle of 1950s. Presently 437 NPP units are in operation in 37 countries with an installed electric net generating capacity of about 372 GW (*http://www.euronuclear.org/info/encyclopedia/n/nuclear-power-plant-world-wide.htm).* Further, 64 units are under construction and 113 units are planned for near future. These NPP reactors can be categorized into three generic groups – generation 1, 2 and 3. According to the Generation 4 International Forum (GIF), which introduced this terminology in 2001, generation 1 consists of reactors built before 1970, most of which are not in operation anymore, generation 2 to those built between 1970 and the end of the 90s, still in operation for most of them, and generation 3 to those designed to replace generation 2 in the coming years. Actually most of reactors constructed in last two decades and under construction as well as planned for future are generation 3 reactors.

. The evolution of reactors from generation 1 to generation 3 led to tremendous improvement in safety features, especially for internal events

An NPP involves complex civil engineering structures operating in demanding environments that have the potential to challenge the level of safety (safety margins) necessary throughout its life cycle. Civil engineering structures are significant items of the structures, systems and components (SSC) important for safe and reliable operation of NPPs. They perform their intended functions in two ways. Firstly, they house systems and components and provide required operating environment. Secondly, given an accident condition, they mitigate its impact containing/confining the radiological release within their structural boundary. Concrete is the commonly used civil engineering construction material in nuclear industry because of a number of advantageous properties it has; mould-ability, easy manufacturing process, usage of mainly locally available ingredients, relatively less production cost, good strength in compression, good shielding property against radiation especially gamma radiation, etc.

The distinct areas of NPP concrete structures dealt with in the nuclear industries are 1) material concrete including proportioning of concrete mix with chemical and mineral admixtures fulfilling the functional requirement, 2) design and construction of concrete structures covering behaviour of structural concrete, constitutive models, fracture phenomena and leakage, impact of operating environment, analytical and experimental response of structures against earthquakes and extreme loadings (both static and dynamic); and 3) operation and ageing management. These three areas of NPP concrete structures underwent a great degree of evolution along with the evolution of NPP reactors from generation-1 to generation 3. Survey of the transactions of international conferences on Structural Mechanics in Reactor Technology (SMiRT) provides a good account of the development in these areas over the years. Deliberation on the first area has been started in last two decades highlighting more on the concrete with silica fume and fly ash; while, work on the remaining two areas are being reported in the transactions of SMiRT Conferences since the beginning,

The present paper describes the evolution of NPP concrete structures in last two decades by summarizing the development in the areas of concrete technology and construction techniques, design methodology and approach for continuing operation through maintenance and ageing management of concrete structures.

MATERIAL CONCRETE

In nuclear power plant, concrete mixes having different requirements are used: concrete of different density – low, medium and high; strength – normal, moderate and high; desired durability – low permeability, wear resistance; etc.

The conventional ordinary normal strength concrete (NSC) for structural use was first developed with basic ingredients; coarse aggregates, fine aggregates, hydraulic cement or ordinary Portland cement (OPC) and water. Hydration starts immediately after mixing cement with water and produces cement paste or hydrated cement paste (*hcp*). Concrete at fresh state can be described as a suspension of aggregates in cement paste. The paste serves as binding agent that binds aggregates to work together as a composite material. The physical state of the paste changes from liquid with suspension to a complete solid during the process of hydration (Taylor 1990, Neville 1996, Mehta and Monterio 1997). Concrete is principally a three phase composite material consisting of a binding media within which particles or fragments of aggregates are embedded. Bulk *hcp* and aggregates constitute the first two phases, while the third one is transition zone between bulk *hcp* and aggregate. The transition zone is also *hcp* but of inferior quality than that of bulk *hcp*.

Structural concrete of various types, normal concrete, heavy concrete, and borated concrete were manufactured in early days of nuclear power plant construction using primary ingredients coarse aggregate, fine aggregate, cement and water; mix design being done by volumetric method. Strength was considered as the most important performance parameter in mix proportioning. Durability was believed to be directly proportional to the strength and workability. Workability was assessed in terms of slump.

The effect of high temperature on concrete has always special implication on the safety of NPP concrete structures. There are numerous publications on the effect of high temperature on conventional concrete. The work of Kanema et al. (2005) has good coverage in this area. They investigated the influence of the mix parameters and microstructure on the behaviour of concrete at high temperature (five heating - cooling cycles with maximum temperatures 150^0C, 300^0C, 450^0C and 600°C). Five normal concrete mixes without mineral admixtures having cement content of 325, 350, 400, 450 and 500 kg/m^3 with water-cement ratio 0.62, 0.55, 0.44, 0.36 and 0.29 respectively and a constant aggregate content were studied before and after heating – cooling cycles. Measurements of compressive and tensile strength, modulus of elasticity, and permeability were carried out on cylindrical specimens before and after heating – cooling cycles. Complementary measurements of concrete mass losses and temperature field at various locations in the specimens were also carried out during the heating – cooling cycles. The properties of the heated concrete at 150°C remained close to those of the non-heated concrete; there was even an improvement of the compressive strength for concrete mixtures with high cement content. All concrete samples underwent significant mass loss between 150^0C and 300 °C mainly due to escape of a great part of interstitial water and hydrates bound water, but maintained their mechanical properties on a high level. In the temperature range 300^0C to 600°C, major decrease in the mechanical properties, deterioration of concretes matrices and small mass loss were observed.

Demand generated from the increasing concern for safety, the advancement in structural engineering, the shrinkage of resources and the new economic considerations dictated the necessity of development of workable and durable high strength concrete. A limited success of improving the NSC was achieved using the chemical admixture of plasticizer/super plasticizer categories (Ramachandran 1995). Introduction of chemical admixture not only improved the quality of concrete, but also facilitated the mechanized construction by pumping the fresh concrete.

High strength concrete (HSC) was achieved with the introduction of mineral admixtures having pozzolanic characteristics (Mehta and Monterio 1997). HSC had issues related to durability. However, the mineral admixture along with chemical admixture opened the opportunity of engineering a concrete mix with improvement in its properties and as per the specified various performance requirements. This resulted in the development of high performance concrete (HPC) (Shah and Ahemd 1994, Neville 1996, Aitcin 1998). HPC mix is generally attributed to higher strength and durability than NSC. Improved properties of HPC is achieved by modifying the microstructure, particularly that of transition zone. Such modification in microstructure is engineered by using chemical and mineral admixtures, appropriate grading of solid materials starting from coarse aggregates to the finest one and low water binder ratio (w/b). The mechanism which leads to the desired modification of the microstructure has three components, (i) reaction mechanism among ingredients, (ii) physical process, and (iii) curing.

Reaction mechanism is basically related to chemical reaction among ingredients and related physical phenomena involved with the hydration of cement paste. Physical process results in creating conducive conditions for reaction mechanism to take place through different actions during construction. Curing maintains the satisfactory condition so that reaction mechanism can be completed to a desirable state by preventing moisture loss and maintaining appropriate temperature after the placement of concrete (Basu 2001a).

Further advances in concrete technology led to the development the self-compacting concrete (SCC), which is also known as self-consolidated concrete (Okamura and Ouchi 1999), Skarendhal and Paterson (2000), Ouchi et al. (2005). Unlike traditionally placed concrete, which is compacted with external energy input (e.g. vibrators, tampering, etc.), the SCC is compacted or consolidated under the action of self-weight. SCC is also a HPC mix with additional performance requirement of self-compaction or consolidation. The rheological characteristics of fresh concrete essential for self-compaction are filling ability (ability to fill all spaces within the form work), passing ability (ability to passing through the obstructions of reinforcement and embedment) and resistance to segregation (maintaining its homogeneity). The difference between the traditionally placed concrete and self - compacting concrete is the performance in fresh state; requirements are same in hard state.

With the development of HPC, strength, workability and durability are principal performance requirements considered as separate parameters in engineering the concrete mix for construction of NPPs for last two decades or so. Basu (1999) derived and summarized the performance requirements of HPC for Indian NPP.

HSC / HPC mixes with admixtures was introduced in the construction of NPP structure during the last phase of generation 2 and being now used in the construction of generation-3 NPPs. Generally two types of chemical admixtures, plasticizer/super plasticizer and viscosity modifying agent (VMA) (in case of SCC only) are employed in the concrete mix for NPP structures. Though HPC mixes had been developed using silica fume (SF) (Malhotra et al. 1987), fly ash (FA) (Malhotra and Ramezanianpour 1994), granulated blast furnace slag (GGBS), high reactivity metakaolin (HRM), rice husk ash (RHA), use of SF and FA as mineral admixtures is common in the construction of NPP concrete structures.

High Strength / High Performance Concrete

HPC was used in the construction of Civaux-2 NPP of France. This HPC was characterized by its low cement content (266 kg/m^3) and water content (161 liter/m^3), as well as by the use of silica fume and calcareous fillers (Larrard 1990). This formula resulted concurrently in high 28-day strength of 64.5 MPa and high workability. As compared to a conventional concrete, the temperature rise was cut by 25% in a 1.2 m thick wall, shrinkage and creep features were improved and the air-tightness was increased by a factor of 10.

Delamination of the under surface of primary containment dome of Kaiga Atomic power project, Unit-1 (Kaiga-1 dome) in India had occurred during construction. One of the key components for re-engineering of the delaminated dome was use of SF based HPC of characteristic compressive strength of 60 MPa and characteristic split tensile strength of 3.87 MPa (Basu and Gupchup 2004, BIS 2000). The concrete mix design and construction techniques with SF were improved taking into account the experience of Civaux NPP. The M60 grade HPC mix for the re-engineered Kaiga-1 dome was established by conducting a series of trial mixes (Basu and Mittal 1999). The laboratory and field tests undertaken to finalize the mix were reported by Mtiial and Kamath (1999). A detailed R&D program was undertaken to develop suitable HPC mix for Indian NPP by Chakraborty et al. (2001). The containment structures of Kaiga-2, Rajasthan Atomic Power Project, Unit-5&6 (RAPP-5&6) and Tarapur Atomic Power Project, Unit-3&4 (TAPP-3&4) were constructed with similar HPC mix Basu 2001b).

Noumowé and Gall (2001) studied the thermal gradient and mechanical behavior of silica fume based High Strength Concretes at raised temperature up to 200°C. Four groups of specimens (two HSC with and without polypropylene fibers, one lightweight aggregate concrete and one NSC) were subjected to identical testing conditions that determined compressive strength, modulus of elasticity and splitting tensile strength before and after a heating- cooling cycle at 200°C. Thermal gradients were significantly lowered by adding polypropylene fibers to concrete (2 kg/m^3). Polypropylene fibers did not modify the residual mechanical properties of the tested high strength concrete. Neither the compressive behavior nor the tensile behavior was significantly modified by the adding of polypropylene fibers. Thermal gradients were greater in lightweight aggregate concrete than in HSC with normal weight aggregate . NSC presented lower thermal gradients than HSC. The compressive strength was reduced but the splitting tensile strength appeared to improve when lightweight aggregates were used. Furthermore, the residual mechanical properties of NSC were better than that of the other tested HSC.

Janotka and Nürnbergerová (2003) investigated the effect of temperature on structural quality of SF based HSC at Temelin (Czech Republic), Mochovce (Slovakia), and Penly (France) nuclear power plants. The strength, elasticity modulus and deformation of concrete are irreversibly influenced by temperature elevation mainly to 100^{0}C and 200^{0}C. Concrete exhibits a pronounced “softening” as a result of the elevated temperature exposure. The pore structure coarsening, at the end of recovery periods after 100^{0}C and 200^{0}C exposures, results in significant strength decrease of cement paste and concrete. Rapid cooling after temperature elevations evokes equal and irreversible structural quality deterioration of concrete and cement paste. The “self-curing” of concrete and cement paste after short- and long-term recovery at 20^{0}C does not contribute to the structural integrity improvement.

Oh et all (1999) examined the influence of mixture proportion of HPC mixes having different types of cement and mineral admixture including SF, FA and GGBS on the durability of concrete. The durability of concrete mix was assessed by chloride ion penetration test. The chloride ion penetration for HPC mixes was low as compared to the concrete mixes without mineral admixtures. The degree of improvement in durability varies with the type of mineral admixtures and their quantity. The HPC mix with an optimum amount of silica fume (10%) exhibited very high resistance to chloride ion penetration.

Fly Ash Based Concrete

In fly ash based concrete, the siliceous fly ash (SFA) of IS 3812 (BIS 2003a, BIS 2003b), or Class F fly ash of ASTM-618 (ASTM 2003) is used as a supplementary cementitious material, incorporated as a cement replacement. The cement replacement level (CRL) is in the range of 10% to 30 % by mass in low volume fly ash concrete (LVFAC), while it could be as high as 50% in high volume fly ash concrete (HVFAC). The Advanced Concrete Technology Program at CANMET in 1985 developed HVFAC (Malhotra and Ramezanianpour 1994). This class of concrete has very low water content (water-binder ratio, w/b <0.4), that is possible due to use of superplasticizer, and more than 30% of the OPC by weight is replaced with fly ash. There are numerous publications on the mix design methods for HVFAC (Aitcins 1998, Malhotra and

Ramezanianpour 1994, CIDA-NRCCAN-CII 2003). Most of these methods derive the governing parameter, w/b, by trial and error method. Ganesh Babu and Nageswar Rao (1993) proposed a noble idea of HVFAC mix design by taking into account the efficiency of fly ash. Choudhury and Basu (2012b) proposed a new strength – cementitious material – water relationship for design of HVFAC mix taking directly into account the characteristics of OPC, FA and fine aggregates to be used in the production of the actual mix, Fig. 1.

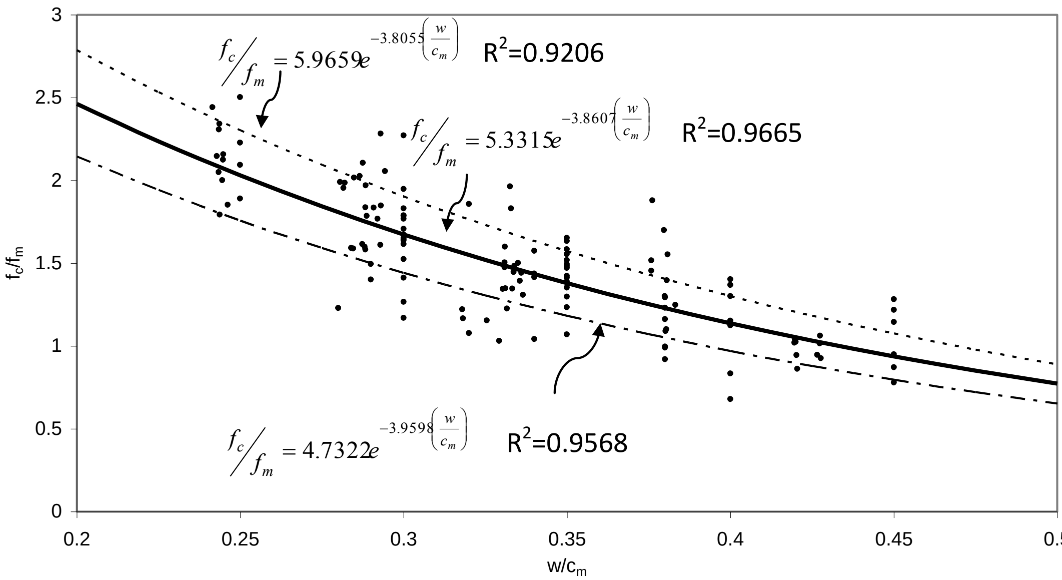

Figure1. Relation between f_c/f_m with $\frac{w}{c_m}$ at 28 days age

Note:
f_c = Target strength,
f_m = Strength of standard mortar, which is a mixture of cement, fly ash and fine aggregates with water cement ratio equals to 0.4,
w/c_m = Water - cementitiuos material (OPC plus fly ash) ratio, and
R^2 = Correlation coefficient.

The Prestressed concrete containment vessels (PCCV) of 3rd and 4th units of pressurized water reactor based Japanese Ohi nuclear power station were constructed using LVFAC mix having specified concrete strength of 45.88 MPa (450kgf/cm2). This was the highest specified concrete strength ever used in nuclear power station construction projects in Japan. No thermal cracking occurred and quality control was successfully implemented to obtain the structural concrete strength satisfying the specified strength by the specified age during actual construction of PCCVs (Sakurai 1991).

One important example of using HVFAC in the construction of NPP structures is Sizewell-B NPP, Soffolk, U. K. during the period of 1988 to 1993. About 6,20,000 m3 of fly ash concrete was used in the construction of Sizewell B NPP consuming nearly 137,000 T of ordinary Portland cement (OPC), 100,000 T of fly ash and 1300 T of sintered fly ash lightweight aggregates. The main power station buildings were constructed with a structural concrete mix having characteristic strength of 45 N/mm2. The cement replacement level by fly ash was 40%.

Also, concrete with fly ash as 50% of the cementitious component was placed as mass fill. Use of fly ash – concrete at Sizewell B is reported to result in significant cost savings. The concrete produced with fly ash was extremely workable and was well suited to pumping over long distances /heights. Properties of the concrete mixes, such as durability, heat of hydration, creep and resistance to alkali-silica reaction, were benefited by incorporation of fly ash. The lightweight fly ash aggregates helped in avoiding thermal cracking (Davies and Kitchener 1996).

Both HVFAC and LVFAC mixes were used in the construction of some of the concrete structures of Indian NPPs – RAPP-5&6, TAPP-3&4 and Kaiga-1&2. Structural FA based concrete mixes with varying CRL, 23.5% - 40%, were used for construction of some safety related structures and also a few non-safety-related structures. Concrete mixes for nonstructural applications had 50% CRL (Basu et al. 2007). Presently, all concrete structures of Kakrapar Atomic Power Project, Unit-3&4 (KAPP-3&4) and RAPP-7&8 are being constructed using HVFAC with 40% CRL. HVFAC with 50% CRL are employed for non-structural applications like mud mat, screed concrete, etc.

Woo et al. (2001) reported the study conducted in Korea to develop LVFAC concrete with 10%, 20% and 30% CRL for the construction of concrete structures having dense reinforcement area, wall, and slab of 30 cm or less in NPP. The study found that LVFAC with 20% CRL is the optimum mix for containment building and structural components with dense reinforcement when 2.0% fluidizing agent and 300 g/m3 of the super plasticizer respectively are added. Durability and time-dependent deformation characteristics of these LVFAC mixes are better than the mix without fly ash.

Two important observations were made from the investigation on the strength gain of high strength (specified strength 420 Kgf/cm^2) LVFAC concrete mix used in the construction of the PCCV of Japanese NPP Genkai, unit No.3. The strength gain of the concrete cast in summertime was predominant in initial stage then became insignificant till 1 year of age; the gain was noticeable from 1 year age to 5 year. On the other hand, the strength gain of concrete cast in winter time is significant up to the age of 1 year and remained almost constant till 4 to 5 years age. However the strength after five years was almost same (Mitarai 1991).

Tanaka et al. (1993) studied the prospect of using ultra-high strength concrete (specified strength 39.2 to 98.1 MPa) material as one of the two features in developing the optimal structural concept for nuclear reactor buildings for longevity of nuclear facilities and ease of decommissioning of NPPs. Material and structural tests using ultrahigh strength materials, and the subsequent analysis of those tests for reinforced concrete shear walls, were conducted. The positive results of this study showed a bright future for the use of ultrahigh strength materials for the reinforced concrete shear walls of nuclear reactor buildings.

Self-Compacting Concrete

The self-compaction characteristics of SCC is achieved by appropriate employment of superplasticiser, low water-to-powder (cement + mineral admixture + any ingredients having particle size not more than 125µ) ratio and use of VMA. Design of SCC mix is more complex than traditionally placed concrete mixes (Skarendhal and Paterson 2000). Choudhury (2008) summarized the mix design methodologies available for SCC. All these methods put emphasis on rheology; strength is taken care of indirectly by trials. Chowdhury and Basu (2012a) developed a new design methodology for self-compacting concrete mixes with high volume fly ash. The method proportions a SCC mixture with equal importance to achieve appropriate rheology and specified target strength directly with minimum trials. M25 to M40 grade (BIS 2000) SCC with high volume fly ash having CRL 40% and 50% were used in the construction of several concrete structures of RAPP-5&6, Kaiga-3&4 and TAPP-3&4. Structural components of larger dimension can be constructed in shorter time with SCC, Fig.2. The overall Indian experience of SCC in NPP concrete structures is encouraging (Basu et al. 2007). SCC with high volume fly ash has also been used in the recent construction of KAPP-3&4 and RAPP-7&8. Reporting on the use of SCC in the construction of NPP structures is very limited.

Figure2. A 10 m high column of turbine building, RAPP-5&6 constructed with SCC

CONSTRUCTION

Along with the beneficial effect, the use of admixtures posed a number of challenges in the areas of construction which led to further development in construction techniques. NPP concrete structures are complicated; critical structures have thick continuum elements of complicated shape and embedded parts / penetrations. The construction activities constitute the physical processes of developing HPC composites with SF and FA. Characterization of ingredients, mixing method to manufacture fresh concrete mix, transportation, placement and compaction has significant effects on the performance of NPP concrete structures. Working out appropriate procedures for these components of physical process and curing along with strict quality control are extremely important.

Mixing Method

The sequence of mixing and the time of agitation play a crucial role in governing the properties of the concrete mix especially in case of HPC. Mixing method of HPC should ideally be such that the cementiotious materials, particularly the mineral admixture is adequately mixed. Basu and Saraswati (2004) observed that the effect of mixing method has significant impact on workability, strength and durability of HPC composites with SF, FA, GGBS and HRM. According to Mehta and Aitcin (1990), the standard method (for HPC) is to obtain first a homogeneous mixture of the coarse and the fine aggregates in the mixer and then add cementitious materials followed by water and the superplasticiser. However, mixing method is to be optimized so that any further increase in agitation time does not affect the homogeneity or the workability of the concrete (Hoff and Elimov 1995). This is also necessary for economizing the construction time. Efficiency of mixing method depends on when and how the chemical admixture has to be introduced into the mix (Ronneberg and Sandvik 1990). Multi stage mixing method has been found suitable for manufacturing concrete mixes with admixtures.

Transportation and Placement

Detailed specifications need to be worked out for transportation, placement and compaction of concrete with mineral admixtures. Basu (2001a) reported the development of specifications that were used in the construction of containment structures of Indian NPPs using concrete with SF in hot weather. Two important aspects, in this respect, are maintaining adequate workability and use of cool concrete (with SF) for reducing the heat of hydration in the thick members. Use of retarder in the concrete mix was very effective in maintaining the workability for a longer duration. Addition of secondary dose of superplasticiser near the pumping point was found useful. Pre-cooling of aggregates in addition to replacement of water with ice flakes is suitable in controlling the temperature of fresh concrete. Also insulation of transit mixers and concrete pipelines helps in controlling the rise in temperature during transit. Mitarai et al. (1991) found that precooled concrete not only reduces the thermal stress in mass concrete due to heat of hydration, but also ultimately improves the strength.

Curing

Curing of concrete is defined as "the maintenance of a satisfactory moisture content and temperature in concrete during its early stages so that desired properties may develop". Curing has more influence on the development of properties in the harden states of HPC as compared to that of normal concrete (Wang et al. 1997) because of less water content in the former one. Two stage curing was adopted in the construction of Indian NPPs (Basu 1999b); initial and final curing. Objective of initial curing is to prevent moisture loss at the early stage and it should be carried out without applying water directly on the surface of fresh HPC. Curing compound is not efficient for the initial curing. Spreading of opaque colour plastic sheet over the fresh HPC contacting the exposed surface is observed to be an efficient approach. Wet burlap may be spread over the plastic sheet for reducing the temperature of concrete surface if FA and GGBS are not used. Initial curing should be continued about an hour more than the initial setting time of concrete. Final curing is wet curing by ponding water, or spreading wet burlap or spraying water on the exposed surface. 10 days of wet curing is sufficient for concrete with SF.

Concrete members of large depth, for example ring beam of the primary containment dome, are cast in a number of pours segmenting along the depth / thickness of the member. Exposed surface of the segmented pour with extended reinforcement forms the construction joints for the subsequent pours. Curing of this type of surface is an involved one when constructed with high strength concrete, especially for structures with leak tightness as a functional requirement. The exposed surface cannot be covered for initial curing and laitance needs to be removed in order to achieve construction joints of good quality. Providing a cover of opaque colour PVC sheet after spraying of surface retarder and subsequent green cutting after the final setting of concrete have been found to be an efficient approach for the initial curing, This also provides appropriate treatment of the surface for good construction joint. Surface retarder may be mixed with a dye in order to ensure uniform spraying by visual inspection. In hot and dry ambient conditions fog spray may be used (Basu 1999b). Final curing should be done adopting similar approach of wet curing of other surface.

DESIGN AND VERIFICATION OF CONRETE STRUCTURES

The concrete structures of generation 1 NPPs were designed using working stress method both for load cases of normal operation and design basis accident (DBA) conditions. The evolution of design approach witnessed from working stress method to ultimate strength method and then to limit state method in the design of generation 2 NPP. The load conditions for design remained same for this generation of NPP, i.e. normal and DBA conditions; though work on determination of ultimate load bearing capacity (ULBC) of containments structures for beyond design basis accident (BDBA) and seismic capacity against beyond design basis earthquake was initiated during this period. For generation 3 NPP, all countries excepting USA adopted the limit state method for design of concrete structures for load cases of normal as well as DBA conditions. Highlight of the design of this generation of NPPs is the safety assessment against certain BDBA / severe accident conditions, particularly for containment pressure, seismic ground motion and commercial aircraft crash. Practice of linear structural response analysis against the loadings of normal design as well as DBA conditions are prevailing from the time of generation 1 through generation 3. Non-linear structural analysis is conducted for safety assessment against BDBA or severe accident scenarios. A number of experimental works for confirmation of structural behaviour in nonlinear conditions were conducted during this period

Major evolutions from Generation 2 to Generation 3

In comparison to Generation 2 NPPs, design requirements associated to Generation 3 present tremendous evolutions which should be accounted for by both regulators and operators. Regarding their impact on concrete structures, the most significant evolutions are:

- The regular operational life has been extended by some designers to 60 years. It means also that, through the life extension process, actual operational life could be as long as 80 years or more. It should be pointed out here that, although they might be regarded as long or very long by some individuals, such periods of time are not exceptional in the field of civil engineering. Life expectations of dams and large bridges are even longer. There are numerous examples around the world of dams that have been continuously operated for centuries, for various purposes including irrigation.
- The severe accident occurrence is considered as a design condition of containment instead of being categorized as a beyond design occurrence. A significant consequence is higher design pressure for the containment. Taking example of two PWRs, the EPR containment design pressure is 0.65 MPa and the AES 92 one is 0.6 MPa, to be compared to the design pressure of Generation 2 standards which were respectively of 0.53 MPa for the French N4 standard (1400 MW) and 0.5 MPa for the WWER-1000. A comprehensive and deep review of advanced light water reactor design is presented by the IAEA (2004).
- After the September 11th 2001 event, the commercial aircraft crash is now considered as a load case. Basically it means that, as compared to the case of an accidental military aircraft crash such as considered in the Generation 2 design assumptions, the momentum of the missile has been increased by a factor larger than 10.

- After the Fukushima-Daïchi accident it is clear that beyond design extreme natural events should also be considered. In the wake of the accident, robustness of installations has been assessed under Beyond Design Basis Earthquakes (BDBE). In view of the increasing level of seismic input motion to be taken into account, a new interest is paid to seismic isolation techniques for which France has been a pioneering country (IAEA 2013) (Labbé 2010).
- A better attention should also be paid to Tsunami loads. The Indian NPP Kalpakkam had been flooded in December 2004 after the Sumatra Tsunami. In comparison, a feature of the Fukushima Daïchi case is that significant dynamic impact effects were observed due to the wave velocity, which should be considered in safety assessment.

Additionally, especially in countries that comply with the IAEA requirement of implementing Periodic Safety Reviews (PSR), the safety of existing nuclear facilities should also be assessed against these updated load cases, which is very challenging. Various approaches and methodologies have been deployed to cope with this difficulty.

- Hydrogen explosion risk has been mitigated in some countries by implementing hydrogen recombiners.
- More realistic estimates of containment structures capacities under high pressure have been made possible both in terms of leaktightness and of ultimate limit state.
- More realistic criteria have been used for assessing aircraft crash effects, such as developed hereunder.
- The Seismic Margin Assessment (SMA) method, such as developed by EPRI and endorsed by the NRC, and later by the IAEA, is now considered as an international good practice for dealing with BDBE issues.
- Seismic Probabilistic Safety Assessment (Seismic PSA) has been also used in several countries in order to assess the seismic risk and/or to optimize the decision making process of safety upgrading actions to be implemented.

In the following a focus is made on progress in assessment of containment pressure capacity and on those significant evolutions driven by extreme external hazards, namely commercial aircraft crash and extreme earthquake. In the knowledge of the authors developments on design basis principles against tsunami are not yet stabilized, although an IAEA document is anticipated on the subject.

Ultimate bearing capacity of containment structures

Although research on the integrity of containment structures or vessels for nuclear power plants has been conducted around the world, it is recognized that the most comprehensive experimental effort has been conducted at Sandia National Laboratories (SNL), primarily under the sponsorship of the US Nuclear Regulatory Commission (NRC), after the accident in 1979 at Unit 2 of the Three Mile Island NPP. A series of increasingly large and complex tests of scale models of containment structures and components were conducted at Sandia National Laboratories between 1983 and 2001 (USNRC 2006). The latest and most significant was constructed by Nuclear Power Engineering Corporation (NUPEC) of Japan, now Japan Nuclear Energy Safety organization (JNES).

NUPEC/NRC Prestressed Concrete Containment Vessel Model Test

The NUPEC/NRC model consisted of a 1:4-scale model of the Prestressed Concrete Containment Vessel (PCCV) of an actual nuclear power plant in Japan, Ohi-3. Ohi-3 is an 1127 MWe Pressurized Water Reactor (PWR) unit, one of four units comprising the Ohi Nuclear Power station located in Fukui Prefecture, owned and operated by Kansai Electric Power Company.

The design pressure, Pd, for the model and the prototype is 0.39 MPa. The features and scale of the PCCV model were chosen so that the response of the model would mimic the global behavior of the prototype and local details, particularly those around penetrations, would be represented. The model includes a steel liner anchored to the concrete shell. Details of the design and construction are reported in the PCCV test report (USNRC 2003-a). The overall geometry and dimensions of the PCCV model are shown in Figure 3.

Prestressing levels for the model tendons were selected so that the net anchor forces (considering all losses due to anchor seating, elastic deformation, creep, shrinkage and relaxation) at the time of the Limit State Test matched those expected in the prototype after 40 years of service. Construction started in 1997 and was completed in 2000. Concurrent with the construction of the model, Sandia installed nearly 1500 transducers to monitor the strain, displacement, forces, temperatures and pressures in the model.

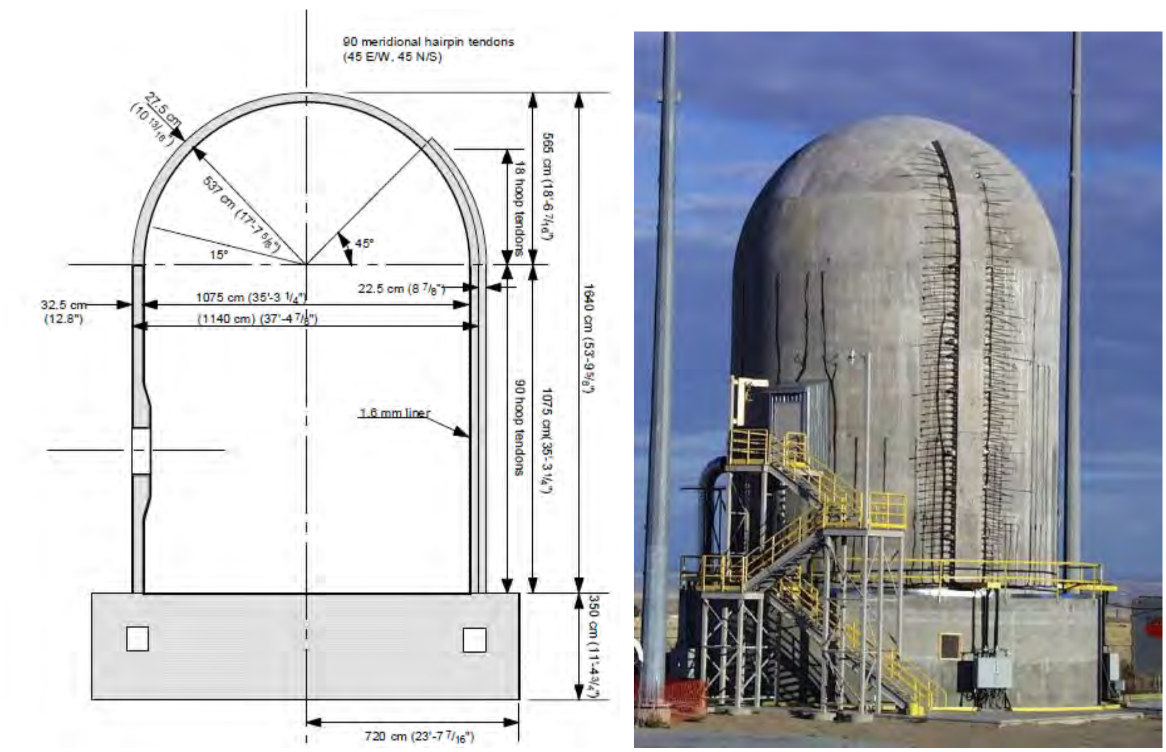

Figure 3. The NUPEC/NRC PCCV model tested by Sandia National Laboratory

Pressure testing of the model consisted of a series of static over pressurization tests of increasing magnitude, beginning with the System Functionality Test (SFT) to 0.5Pd. The series of tests combined Structural Integrity and Integrated Leak Rate Test (SIT/ILRT). The PCCV model was pressurized to 1.125Pd and, after holding pressure for approximately 1 hour, the model was depressurized to 0.9 Pd and held at this pressure for 24 hours. During this period a leak rate of less than 0.1% mass/day was calculated, essentially demonstrating that the model was leak-tight.

For the Limit State Test (LST), the model was pressurized in increments of approximately 0.2Pd to 1.5 Pd, then in increments of approximately 0.1Pd, to 2.5Pd. At this pressure, a leak rate of 1.63% mass per day was calculated, indicating that the model was leaking, most likely from a tear in the liner in the vicinity of the equipment hatch, and it was concluded that the model had functionally failed (was not as leak-tight as required) between 2.4 and 2.5 Pd.

In order to collect data on the inelastic response of the structure and to observe a possible structural failure mode, pressurization continued in steps of 0.05 Pd. The pressure was increased to slightly over 3.3 Pd before the leak rate exceeded the capacity of the pressurization system and the test was terminated. At maximum pressure local liner strains approached 6.5% and global hoop strains at the mid-height of the cylinder averaged 0.4%. Large tears were observed near the equipment hatch. Although extensive concrete cracking was reported at some locations, there was no indication of tendon or rebar failure and the data showed that no tendon strains exceed the elastic limit while only a few dozen rebar strain gages showed strains in excess of 1%.

With the objective of observing large inelastic deformations, for comparison with analyses, and witnessing the structural failure mode of the PCCV model, the interior surface of the liner was then sealed with an elastomeric membrane, the model was fill with water to 1.5m from the dome apex, and the remaining gas pocket re-pressurized. A wire break was detected for a pressure slightly higher than the 3.3 Pd peak pressure achieved during the LST, and at an effective pressure of 3.63 Pd the PCCV model ruptured suddenly and violently.

The main lesson learnt from this remarkable experiment is that a Prestressed concrete containment of this type can be regarded as having a pressure capacity around 2.5 times the design pressure in terms of leak-tightness and around 3.5 times the design pressure in terms of ultimate limit load (NRC 2006).

Associated Analyses

Pretest prediction analyses were completed and published prior to the LST (USNRC 200-a). A number of regulatory and research organizations were also invited to participate in a pretest Round Robin analysis to perform predictive modeling of the response of the model to over pressurization. Seventeen organizations responded and agreed to participate in the pretest PCCV Round Robin analysis activities (USNRC 2006). Luk (USNRC 2000-b) compiled the results along with the individual participant reports, which resulted in the following observations: Predictions of elastic response were, for the most part, very consistent up to the onset of global yielding (hoop) which appears to occur around 2.5 Pd. Predictions of response diverge significantly beyond this point with responses varying by a factor of three to five or more at a given pressure. For failure predictions based on material failure of the steel components (liner, rebar or tendons), the average predicted pressure at failure is 3.6 Pd. Approximately half the participants predicted failure based on structural failure, i.e., rupture of rebar or tendons, while approximately half the participants predicted functional failure from excessive leakage through a tear in the liner and/or cracks in the concrete.

The PCCV test showed that the response quantity driving the limit state of the vessel is radial expansion of the cylinder. This response must be predicted correctly in order to reliably predict vessel capacity and, at least approximately, the local response mechanisms that are driven by the cylinder expansion. Interestingly, a conclusion of posttest analyses (USNRC 2003-b) was that the average radial expansion or hoop strain of the cylinder was predicted very accurately by the axisymmetric analysis. Posttest analyses also gave evidence that an appropriate modeling of tendon friction behavior is crucial. A detailed analysis led to the conclusion that considering both the effects of a discrete crack and elevated local strains, a liner tear could have been predicted to occur as early as 2.8 Pd.

One question which remained following the PCCV test was how temperature loading might have affected the results of the test. In an attempt to answer this question, the Nuclear Energy Agency (NEA) of the OECD recognized it as an international standard problem (ISP) and sponsored analyses in view of its resolution. Eleven organizations (or teams), including several participants in the Pretest Round Robin, participated in the ISP, results of which are reported by the OECD (OECD 2005). Unfortunately there was no clear consensus regarding the effect of temperature on the failure mode and pressure.

Aircraft crash

A remarkable example of evolution is related to aircraft crash: for the generation 2 NPPs conventional design criteria for reinforced concrete were used, associated to load cases of business or military aircraft crashes. After the 2001 September 11 attack, it was recognized that a malevolent commercial aircraft crash should be considered in safety assessment of NPPs and taken into account in the design of NPPs to be built in the future. However it was also recognized that, when dealing with extreme loads such as generated by commercial aircraft crashes, more realistic criteria should be taken into account according to the ultimate capacities of reinforced/pre-stressed concrete structures.

Concrete wall capacities against perforation

Perforation of concrete walls by rigid missiles has been studied for decades. Different empirical formulae were proposed by different authors, a comprehensive review of which is presented by (Li et al. 2005). A widely used formula is known as the CEA-EDF formula, which was established in the seventies (Berriaud et al. 1978). An improved formula has been proposed by (Buzaud et al. 2007) with a significantly larger validity domain.

At the moment of Generation 2 NPPs design, rather small aircraft engines were considered. They were regarded as rigid missiles and the corresponding formula were used. For generation 3 NPPs, larger engines are considered. However researches carried out in the last three decades gave evidences that aircraft engines should not be regarded as perfect rigid missiles and, consequently, a larger interest has been paid to achieve a realistic estimate of their actual perforating capacities. A series of experiments was carried out in Germany in the seventies, known as the Meppen tests (Jonas et al. 1979). A famous experiment was also carried out by Sandia National Laboratories, consisting of crashing an actual military aircraft on a concrete wall (Sugano et al. 1993-c). Recently an experimental campaign was conducted by the Technical Research Centre (VTT) of Finland in the frame of an international cooperative program, which provided the matter for an international benchmark under an OECD-NEA initiative (OECD 2012).

Analyses carried out by Sugano (Sugano et al. 1993a & 1993b) led to amend empirical formulae for rigid missiles so that they are applicable to aircraft engine impacts. Such amended formulae are available in CEB (1990), DOE (2006) and NEI (2009)... For instance, in order to be protected against perforation, the required wall thickness is only 70% of what is necessary according to rigid missile formulae.

Concrete wall bending capacities

As opposed to the perforation issue, the assessment of concrete wall bending capacity is not addressed through empirical formula. The physics of phenomena can be accurately modelled. The loading force is generally established through the Riera's formula (1968) and the wall capacity is evaluated through usual ultimate limit state (ULS) acceptance criteria. About commercial aircrafts, it should be pointed out that they were initially aimed by Riera. Consequently the physics of their crashes fit the conceptual approach even better than the military aircraft crashes do, and not surprisingly the method still applies.

Regarding acceptance criteria, it was recognized that Design Basis Condition (DBC) criteria, such as adopted for the design against military aircraft crashes were not appropriate to assess effects of extreme loads such as generated by commercial aircraft crashes. For this purpose commercial aircraft crashes were categorised as a Design Extension Condition (DEC) and appropriate criteria were established, corresponding to a more realistic approach of the impact effects on the structure. A simplified comparison of DBC and DEC criteria is summarized in Table 1. National practices vary, so that it is not possible to establish here a comprehensive and accurate comparison. Values in the table should be regarded as representative of values that are retained by the following documents: DOE (2006), NEI (2009) and ETC-C (2010). An IAEA document is also under preparation on the subject (IAEA 2012).

Table 1. Comparison of DBC and DEC criteria for aircraft crash

	DBC	DEC
Concrete strength	f_{ck}/γ_c	f_{ck} or f_{cm}
Strain rate effect for concrete	1.0	1.1 – 1.25
Concrete confinement effect	1.0	1.2
Age of concrete	1.0 (28 days)	1.1 (100 days)
Concrete strain capacity	0.35 % - 0.5%	0.5 % - 1.0 %
Steel elastic strength	f_{yk}/γ_S	f_{yk}
Steel ultimate strength	f_{tk}/γ_S	f_{tk}
Steel ultimate strain	0.8 % - 1%	5 %
Strain rate effect for steel	1.0	1.05 – 1.1

Induced vibrations

Both military and commercial aircraft crashes result in in-structure induced vibrations that should be accounted for in safety assessment. The frequency content of the dynamic load generated by the crash is still an open issue (OECD 2012). Refined modelling of the aircraft structure crash indicates that high frequency content, ignored in the Riera's approach, should be taken into account in the load function. It does not mean that an additional input should be added on the top of the Riera's load curve, but that this load curve should not be as smooth as it appears (OECD 2012). The high frequency content of the loading force and the corresponding wave propagation in concrete structures (in particular energy dissipation along the wave path) are matters of research for the coming years.

The in-structure transferred vibrations are crucial for equipment assessment. A usual way of dealing with them is to set a comparison with seismically induced vibrations. Due to the rather small aircrafts that were considered for the generation 2 design, it was possible to substantiate that aircraft crash induced vibration were covered by seismically induced vibrations. With the aircraft that are now considered, the intrinsic high frequency content of aircraft crash induced vibrations should be addressed per se. A favourable feature of those high frequency vibrations is that they generate very small displacements. There is a consensus among experts on the fact that an approach based on induced displacement is much more appropriate than a conventional seismic approach based on accelerations when addressing equipment safety assessment issues under aircraft crash conditions.

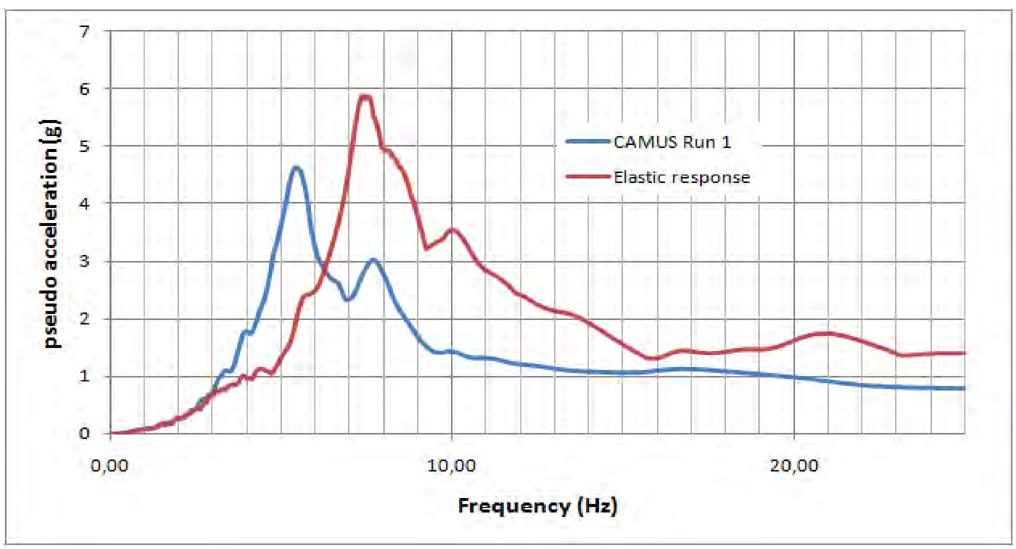

Figure 4: Floor response spectra at the top of the CAMUS specimen

Earthquakes

To a certain extent there is a similarity between aircraft crash and seismic input motion issues. In both cases, for existing NPPs, plant design was established under design basis conditions and it should now be assessed under extreme conditions or design extension conditions.

Experience and experimental feedback

Several cases of seismic motions that approach or exceed the Design Basis Earthquake (DBE) were reported before the Fukushima-Daïchi accident, in Japan (Onagawa 1993, 2003, 2005, Kashiwasaki 2007, Shika 2007, Hamaoka 2009) and to a lesser extent in USA (Perry 1986 and North-Anna 2011). As a matter of fact, the more challenging seismic input motion recorded in an NPP at the occasion of the 11 March 2011 Great East Japan Earthquake (GEJE) was not at Fukushima, but at Onagawa, much closer to the epicentre than Fukushima.

The feedback of experience of all these events is that NPP concrete structures are actually very robust against seismic input motions. This is due to the fact that nuclear industry design criteria encompass large margins resulting in the fact that when designed for a given input motion an NPP concrete structure is capable to sustain at least 3-4 times this design input motion without significant damages. An evidence of such large margins was provided by the Kashiwasaki-Kariwa NPP in 2007: Although the recorded motion of the raft of unit 1 was 0.68 g for a design value of 0.27 g, the only visible damage on concrete structures was a thin crack.

This feedback of experience is confirmed by experimentation outputs. A comprehensive review of experimental results available in 2003 on NPP concrete wall robustness against earthquakes has been presented by the IAEA (2011). The more significant experimental campaigns on typical nuclear walls were carried out in Japan, USA and Europe (JRC Ispra). More recently the SMART experiment was carried out in France, accompanied by an international blind prediction benchmark (Richard 2013).

Seismic Response Analysis

The current situation is that existing NPPs were designed for a given seismic input motion and that nuclear industry design practices provide large margins, principally implicit, beyond the design. Actually the situation is similar for the design of generation 3 NPPs. A challenge for the nuclear industry is now to make those margins explicit so as to identify a beyond design input level until which the plant is safe.

Under DBE conditions, the response of concrete structures is generally supposed to stay in the elastic regime (accordingly the conventional industry “behaviour factors” are disregarded), although the associated criteria are those of an Ultimate Limit State (ULS) compatible with cracking and even R-bars yielding. The inconsistency is sustainable because most of structural elements stay eventually in the elastic regime under DBE (very few of them reach acceptance criteria). Under extreme input motions the fiction of an elastic regime for computing the structural response is not possible anymore, because, by definition, structures will actually respond in the non-linear domain.

A practical approach of nonlinear response of concrete structures, such as widely used for seismic assessment of existing NPPs around the world is presented by the IAEA (2003). However this approach relies on elastic response computation and is therefore limited to relatively small DBE exceedance. When addressing significant DBE exceedance, non-linear behaviour should be represented. In this regard an example of universal engineering design practice that accounts for non-linear behaviour is given by geotechnical engineering such as it was established in the seventies on the basis of equivalent linear relationships (Seed and Idriss 1970). Development of a similar methodology addressing concrete structures was recommended by the IAEA before the Fukushima-Daïchi accident (IAEA 2010).

Concurrently, in order to address the aimed domain of non-linear response, it is necessary to have not only a realistic representation of the soil and structure behaviour but also a realistic description of seismic input motion. It means that the conventional design response spectra such as NRC (1973) or EUR (2012) or Eurocode (2005) spectra, which provide appropriate load for design purpose, are not appropriate for dealing with extreme events for several reasons:

- On soil sites, the site response is controlled by the soil behaviour. Consequently due to the non-linear soil response, the spectrum shape of the input motion is dependent on the seismic input level.
- Conducting non-linear analyses generally requires concurrently that it be transient analyses, meaning that time history input motion should be available. There are engineering tools that enable to derive accelerograms from response spectra, but the state-of-the-art is precisely that such signals should not be used for nonlinear analyses, except if generated with a special care (Boomer and Acevedo 2004, Hancock et al. 2006, Viallet and Humbert 2007).

Acceptance criteria

As mentioned above, there is a certain lack of consistency, at least in some countries, in the nuclear industry approach of seismic qualification of concrete structures because, on the one hand, the seismic response is computed by assuming an elastic behaviour of concrete, but on the other hand, acceptance criteria are those of ultimate limit states. Furthermore, these criteria are expressed in terms of forces or stresses, while according to the state of the art, the purpose of the evaluation of seismic capacity should be to analyse the strains induced by the postulated input motion and to compare them with ultimate acceptable strains. Basically, as pointed out by the IAEA (2003), it means that approaches orientated to strain evaluation (displacements approach) are more relevant than those based on stresses evaluation (forces approach)[1].

Such an evolution of criteria towards displacement or strain approach is in line with the current development of standards that go towards a wider use of performance based criteria. In place of acceptance criteria based on computation of stresses or forces to be compared to allowable stresses or forces, performance based criteria are preferably expressed in terms of strains, either global (drift under earthquake load) or local (strains in R-bars and concrete under aircraft crash). Criteria are modulated according to the required function of the concrete structure under consideration (leak tightness, supporting of equipment, stability ...).

An interesting and exemplary use of performance based criteria is presented in the ASCE 43-05 standard (2005). For instance, depending on the accepted damage in case of an earthquake, acceptable drift for a concrete moment resistant frame varies from 0.5% (essentially elastic) to 2.5% (stable, short to collapse). The 0.5% drift associated to an essentially elastic behaviour of the structure draws attention because, for such a drift, forces computed on the basis of an elastic response exceed acceptable forces according to the conventional criteria. It reveals once again that ignoring cracking effects that occur for even low levels of input motion is not realistic.

Induced vibrations

The purpose of seismic response analysis of NPP concrete structure is not only to discuss the structure qualification but also to derive the seismic input motions that are transferred to the

[1] However it should also be mentioned that the usual and simple Displacement Based Approach (DBA), such as codified for conventional structures on the basis of a push-over analysis, is generally not applicable to nuclear structure because of their complexity.

equipment. This transferred motion is generally represented in the form of Floor Response Spectra (FRS).

As pointed out by the IAEA (2010), the key point with FRS is that they are very sensitive to small structural non-linearity. The IAEA conclusion is that, even for input motions that do not exceed the DBE, those non-linear effects (related to micro cracking or cracking of concrete structures) should be taken into account in FRS generation. An illustration of this effect is provided by outputs of the CAMUS experiment carried out in France, on the CEA Azalée shaking table in the nineties and extensively discussed at the occasion of an international benchmark organized by the IAEA (Labbé & Altinyollar 2011). As presented in Figure 4, during the first run of the experimental campaign, under an input motion (0.24 g) that did not exceed the acceptable input level according to the nuclear practice (0.27 g), a significant frequency shift towards lower frequencies was observed (IAEA 2010). A fortiori, when dealing with BDBE, it is certainly necessary that structural non-linear effects are properly included in the finite element model.

CONTINUING THE SERVICE OF NPPS

Of the 437 operating units, approximately 83% have been in operation for 20 years or more and 46% for 30 years or more, which are mostly generation 2 NPPs. One key concern that could affect the continued operation and development of nuclear power relates to the impact of ageing on plant performance. Unlike mechanical components or electrical equipment that can be replaced, the concrete civil structures either cannot be replaced or would be very difficult to replace.

As noted previously in this paper, the NPP concrete structures are designed, built, and operated to standards that aim to reduce the likelihood of release of radioactive materials to levels as low as reasonably achievable. A NPP, however, involves complex engineering structures, sometimes operating in demanding environments, that potentially could challenge the high level of safety (i.e., safety margins) required of the plant throughout its service life. With respect to the concrete structures, age-related degradation may affect engineering properties, structural resistance/capacity, failure mode, and location of failure initiation that may in turn affect the ability of a structure to withstand challenges in service.

Degradation Considerations

Whether a concrete structure will degrade is a function of many factors including constituent materials, location (e.g., coastal or inland), climatic conditions (e.g., temperature and moisture), and presence of external agents (e.g., aggressive ionic species). NPP concrete structures are composed of several constituents (e.g., concrete, mild steel reinforcing (rebar), and prestressing systems) that, in concert, perform multiple functions (e.g., foundation, support, containment, and shielding) (Naus 2007). Some concrete structures (e.g., spent fuel pool and containment) also can include a metallic or polymeric liner to provide leak tightness or a pressure boundary.

Concrete material durability can be limited by adverse performance of either its cement-paste matrix or aggregate constituents under attack. In practice, these processes may occur concurrently to reinforce each other. In nearly all physical and chemical processes influencing concrete durability, dominant factors involved include transport mechanisms within pores and cracks and the presence of water. Physical attack mechanisms involve degradation of the concrete due to external influences and generally involve cracking due to exceeding the concrete tensile strength, or loss of surface material (e.g., salt crystallization, freezing and thawing, thermal expansion/thermal cycling, abrasion/erosion/cavitation, irradiation, fatigue or vibration, biological attack, and differential settlement). Chemical attack involves the alteration of concrete through chemical reaction with either the cement paste or coarse aggregate that generally occurs at the exposed surface region of the concrete (cover concrete), but with presence of cracks or prolonged exposure it can affect entire structural cross sections. Chemical attack may occur in several different forms (e.g., efflorescence and leaching; attack by sulfate, acids, or bases; delayed ettringite formation; and alkali-aggregate reactions). Degradation of steel rebar can occur as a result of corrosion, irradiation, elevated temperature, or fatigue effects, with corrosion being the most likely form of attack. Post-tensioning systems are susceptible to the same degradation mechanisms as rebar plus loss of prestressing force primarily due to tendon relaxation and concrete creep and shrinkage. Metallic liners are susceptible to corrosion (e.g., areas of moisture accumulation) and fatigue at locations of stress intensification (e.g., shape changes and structural attachments). Nonmetallic liners can degrade as a result of impact loads, stress concentrations, or reflective cracking (e.g., physical or chemical changes in concrete).

Operational Experience

In general the performance of NPP concrete structures has been very good; however several of these structures have experienced degradation that has required a remedial action. Initially many of the problems identified occurred early in life, were addressed at that time, and were related to errors in material selection, design, or construction (e.g., cracking of prestressing anchor heads due to stress corrosion cracking, containment dome delamination due to design and material deficiencies, low concrete compressive strengths, excessive voids or honeycomb in concrete, and misplaced rebar). As NPPs age, degradation-related problems are occurring due to environmental effects (e.g., cracking and spalling of containment concrete due to freezing and thawing; corrosion of rebar in water-control structures; cracking, moisture intrusion, and leaching of concrete; corrosion of metallic liners; prestressing tendon wire failures due to corrosion; concrete cracking due to alkali-silica reactions; and leakage of borated water from spent fuel pool and refueling cavity liners potentially causing concrete erosion and corrosion of embedded carbon steel). In order to ensure the continued safe operation of NPPs, it is essential that the effects of degradation of the plant structures, as well as systems and components, be assessed and managed.

Ageing Management

Operating experience has demonstrated that periodic inspection, maintenance, and repair are essential elements of an overall program to maintain an acceptable level of reliability over the service life of a NPP concrete structure. Knowledge gained from conduct of an in-service condition assessment can serve as the baseline for evaluating the safety significance of any degradation that may be present, and defining subsequent in-service inspection programs, and maintenance strategies. Effective in-service condition assessment of a structure requires knowledge of the expected type of degradation, where it can be expected to occur, and application of appropriate methods for detecting and characterizing the degradation. Basic components of a condition assessment program include: (1) a review of "as-built" drawings and other information pertaining to the original design and construction information so that information, such as accessibility and position and orientation of embedded rebar and plates in concrete, is known prior to the site visit; (2) detailed visual examination of the structure to document easily obtainable information on instances that can result from or lead to structural distress (e.g., crack mapping); (3) determination of the need for additional surveys or application of destructive and nondestructive testing methods; (4) analysis of results; and (5) preparation of a report presenting conclusions and recommendations. Techniques for establishing time-dependent change such as section thinning due to corrosion, or changes in component geometry or material properties, involve monitoring or periodic examination and testing. General guidance for developing a condition assessment program for managing aging of NPP concrete structures has been provided by organizations such as the International Atomic Energy Agency (IAEA 1998); the Electric Power Research Institute (EPRI 2005 and 2003); International Union of Laboratories and Experts in Construction Materials, Systems, and Structures (Naus 1999); and Nuclear Energy Agency Committee on Safety of Nuclear Installations (NEA 1995). Individual countries also have developed their own programs addressing the concrete structures (IAEA 1998).

Testing and in-service inspections are utilized to assess the condition of NPP concrete structures. In the United States (U.S.), as well as several other countries (e.g., Belgium and Spain) testing encompasses conduct of an integrated leakage-rate test of the containment structure to provide periodic verification of the leak-tight integrity of primary reactor containment as well as systems and components that penetrate containment (CFR 2011). Associated with the leakage-rate test is a general visual inspection of the accessible interior and exterior surfaces of the containment structure and components to uncover any evidence of structural deterioration that may affect either containment structural integrity or leak-tightness. Subsection IWL of Section XI of the ASME Boiler and Pressure Vessel Code provides regulatory requirements for inspection and repair of NPP reinforced and post-tensioned concrete containments (ASME 2010). Accessible concrete surfaces are examined visually (general and detailed) at 1, 3, and 5 years following the containment structural integrity test, and every 5 years thereafter, for evidence of damage or degradation. The acceptability of concrete in inaccessible areas (e.g., foundations) is to be evaluated when conditions exist in accessible areas that could indicate the presence of or result in degradation to such inaccessible areas. One approach is to sample the groundwater adjacent to the structure of interest (e.g., pH, and chloride and sulfate ion concentrations). The unbounded post-tensioning tendon system examination schedule is the same as for the concrete and involves: measurement of prestressing forces for selected sample tendons; removal and testing of tendon wire samples to determine yield strength, ultimate tensile strength, and elongation; analysis of tendon corrosion protection medium for alkalinity, water content, and soluble ion concentrations. Other countries (e.g., Canada, Japan, and Russia) have developed country-specific codes, standards, and documents that address aging management of the concrete structures that are based on testing and in-service inspections.

Several countries (e.g., Belgium, France, and United Kingdom), in addition to leakage rate testing and visual inspections, have included instrumentation at locations of interest to monitor long-term performance and thus are useful for condition assessment evaluations for continued service determinations (e.g., document concrete distress and provide reference data for future examinations to indicate changes from previous inspections). This instrumentation is typically embedded into the concrete at time of construction and has included: vibrating wire strain gauges, thermocouples, pendulums, extensometers, liquid level gauges, humidity gauges, dynamometers, and benchmarks. In NPP containments where grouted tendon systems have been utilized, the instrumentation also provides important data for assessing the condition of the post-tensioning system. Additional information on performance of the grouted tendon systems is provided in some cases by leaving a limited number of tendons ungrouted and installing load cells to monitor the prestress level.

Guidance for Long-Term Operation

Although some countries such as the U.S. have a specified initial licensing period (e.g., 40 years) for commercial NPPs, other countries do not have a defined period but conduct periodic safety reviews at defined intervals (e.g., 10 years) to provide confidence in the continued ability to perform required safety functions. Design life of most existing generation 2 NPPs was typically chosen as 30 to 40 years. However, economic benefits for utilities resulting from extending plant service life (with 60 or more years being a quoted target), means that existing concrete structures often will have to meet their functional and performance requirements for a time period significantly greater than considered during their initial design.

Guidelines for the production of durable concrete typically utilized for generation 2 designs were available in national consensus codes and standards that had been developed over the years through knowledge acquired in testing laboratories and supplemented by field experience. Serviceability of concrete was incorporated into the codes through strength requirements and limitations on service load conditions in the structures (e.g., allowable crack widths, limitations on mid-span deflections of beams, and maximum service level stresses in prestressed members). Durability was included in these designs through specifications for maximum water-to-cement ratios, requirements for entrained air, and minimum concrete cover requirements over rebar. As the generation 2 plants mature and their plant service lives are being extended, environmental factors are going to become increasingly important.

In conjunction with the evolution of plant designs there also has been an evolution in the concrete materials utilized to fabricate the plants as was noted previously. Generation 3 plants have utilized high performance and self-consolidating concretes incorporating chemical and mineral admixtures, and VMAs that have improved performance characteristics. The high performance concretes exhibit higher strengths and improved durability while maintaining or improving the capability of the concrete to be placed. Also at this time an increased emphasis was placed on durability-based design. These developments with respect to concrete materials and durability-based design in all likelihood will improve the capability of the NPP concrete structures to meet their functional and performance requirements for an extended period of time.

Although the U.S. limits initial commercial power reactor licenses to a 40-year period, regulations allow these licenses to be renewed for an additional period of 20 years, with no limit on renewals (CFR 2012). In support of license renewal, the U.S. Nuclear Regulatory Commission (USNRC) published the Generic Aging Lessons Learned (GALL) Report (USNRC 2010) that contains the USNRC staff's generic evaluation of the existing plant programs and documents the technical basis for determining where existing programs are adequate without modification and where existing programs should be augmented for the period of extended operation. Each structure and/or component is identified in the report as well as the material(s) of construction, environment, aging effects/mechanisms, acceptable programs to manage the effects of aging, and if further evaluation is required. Aging management programs (AMPs) related to NPP concrete structures include: ASME Section XI Subsection IWL (GALL AMP

XI.S2) and Structures Monitoring (GALL AMP XI.S6). AMPs related to the post-tensioning system include: GALL AMP XI.S2 and Concrete Containment Tendon Prestress Time-Limited Aging Analysis (GALL TLAA XI.S1). AMPs related to liners of reinforced concrete containments include: ASME Section XI Subsection IWE (GALL AMP XI.S1); 10 CFR Part 50, Appendix J (GALL AMP XI.S4); and Containment Liner Plate, Metal Containments, and Penetrations Fatigue Analysis Time-Limited Aging Analysis. Inspections are performed as part of the AMP review to verify that there is reasonable assurance that the applicant has adequately addressed all the identified passive and long-lived structures, systems, and components identified during an aging management review, and, through review of supporting documentation and walk down of selected systems, that the effects of aging can be adequately managed in the period of extended operation. A more detailed description of the U.S. license renewal process is available (*http://www.nrc.gov/reactors/operating/licensing/renewal.html*).

In a related activity, the International Atomic Energy Agency (IAEA) in September 2010 started preparation of a document to provide an international platform for discussion between regulators and utilities regarding implementation of acceptable NPP aging management plans [i.e., International Generic Aging Lessons Learned (IGALL)] *(http://www-ns.iaea.org/projects/igall/)*. The objective of this programme is to provide: a guide for ageing mechanisms and effects based on both research results and accumulated operational experience, and an international agreement on what an acceptable AMP involves for standard plant components, structures, materials, and environments. The overall task is aimed at facilitating the exchange of experience accumulated by Member States on identification, establishment, and implementation of AMPs, as well as providing a knowledge base for activities such as the design of new plants and design reviews of existing plants. The scope of the programme addresses: PWRs, BWRs, water-cooled water-moderated energetic reactors (WWERs), Canada deuterium uranium reactors (CANDUs), and pressurized heavy-water reactors (PHWRs). IGALL working groups are focusing on: mechanical components and materials, electrical components and I&C, and structural components and structures. Structures and components include: containment structures (concrete containments, steel containments, and common components); nine groups of building structures other than containment; and supports and anchorages for structures, equipment, and components. Under the structural components and structures working group ten AMPs have been prepared: AMP 301, ISI for Containment Steel Elements; AMP 302, ISI for Concrete Containment; AMP 304, Leak Rate Test; AMP 305, Masonry Walls; AMP 306, Structures Monitoring; AMP 308, Coating; AMP 309, Composite Liner; AMP 310, Ground Movement Surveillance; AMP 311, Containment Monitoring System; and AMP 312, Concrete Expansion Detection. Completion of IGALL is scheduled for 2013.

Candidate Concrete Research Topics in Support of Long-Term Operation

Based on operating experience related to existing NPPs, several areas have been identified where additional research would be of benefit to ageing management of NPP concrete structures to provide an improved basis for long-term operation (Naus 2012): (1) compilation of material property data for long-term performance and trending, evaluation of environmental effects, and assessment and validation on nondestructive testing methods; (2) evaluation of long-term effects of elevated temperature and irradiation on concrete properties and performance; (3) improved damage models and acceptance criteria for use in assessments of the current condition as well as estimation of future performance of concrete structures; (4) improved constitutive models and analytical methods for use in determination of nonlinear structural response (e.g., accident conditions); (5) nonintrusive methods for inspection of thick-walled, heavily-reinforced concrete structures and base mats; (6) global inspection methods for metallic pressure boundary components (i.e., steel containments and liners of concrete containments) including inaccessible areas and backside of liner; (7) data on application and performance (e.g., durability) of repair materials and techniques; (8) utilization of structural reliability theory incorporating uncertainties to address time-dependent changes to structures to ensure minimum accepted performance requirements are exceeded and for estimations of end of life; and (9) application of probabilistic modeling of component performance to provide risk-based criteria to evaluate how ageing affects structural capacity.

SUMMARY

Concrete structures are significant items of the SSC important for safety of a NPP and its reliable operation. Along with the development of nuclear power reactors from generation-1 to generation 3, the NPP concrete structures underwent a great deal of evolution. The evolution has taken place in all three principal areas of concrete structures - material concrete, design methodology and construction techniques, and approach for continuing operation through maintenance and ageing management. Present paper provides a brief account of this evolution witnessed in last two decades.

Nuclear industries have started deliberation on the 'material concrete' in SMiRT conference with the introduction of high strength / high performance concrete during last two decades. HPC is superior concrete with regard to workability, strength, and durability as compared to conventional NSC. A large number of NPP structures have been constructed successfully with traditionally placed HPC and SCC in countries like India, France, UK, and Japan. No adverse performance of NPP structures constructed with SF or FA based concrete has been reported. The effect of high temperature on concrete has special implication on the safety of NPP concrete structures. Behaviour of concrete without mineral admixtures at high temperature is better than that of concrete with mineral admixture. Temperature behavior of FA based concrete has edge over the concrete with SF.

Construction methods have significant influence on the strength, durability, and performance of structures constructed using HPC or SCC. Multi stage mixing method is suitable for manufacturing of HPC mixes. Detailed specifications need to be worked out for transportation, placement and compaction of concrete with chemical and mineral admixtures. Precooled concrete (when silica fume is used) reduces the thermal stress due to heat of hydration in structural component with large thickness / depth in hot weather and also improves the strength. The concrete structures with mineral admixtures should be cured in two stages; initial and final curing. The initial curing is dry curing and the final one is wet curing. Curing compound is not efficient for the initial curing. Spreading of opaque colour plastic sheet over the fresh HPC contacting the exposed surface is effective for initial curing. Application of surface retarder followed by high pressure air water jet green cutting was found to be an effective method for curing and preparation of prospective construction joint surface.

More realistic estimates of containment structures capacities under high pressure have been made possible both in terms of leaktightness and of ultimate limit state. Both the September 11 event and Fukushima-Daïchi accident have a significant impact on design of new NPPs and verification of existing ones, especially with respect to aircraft crash and earthquakes.

Malevolent commercial aircraft crash is now taken into account in the design of NPPs to be built in the future. However, when dealing with such extreme loads, realistic criteria should be adopted in place of conventional design criteria, according to the actual ultimate capacities of reinforced/pre-stressed concrete structures. Regarding Beyond Design Earthquakes, it is clear that the non-linear response of structures should be taken into account, in particular when computing seismic input motions transferred to equipment (floor response spectra). Conducting non-linear analyses implies that seismic input motion should be selected with special care. Acceptance criteria should evolve towards displacement or strain based criteria in place of force or stress based criteria.

As noted, nuclear power plant concrete structures are designed, built and operated to standards that aim to reduce the likelihood of release of radioactive materials to levels as low as reasonably achievable. A NPP, however, involves complex engineering structures, sometimes operating in demanding environments that potentially could challenge the high level of safety required of the plant throughout its service life. Operating experience has demonstrated that periodic inspections, testing, maintenance, and repair are essential elements of an overall ageing management program to maintain an acceptable level of reliability for these structures. In general, NPP concrete structure's performance has been very good with the majority of identified problems initiating during construction and corrected at that time. However, as plant service lives are being increased to 60 or more years, the potential for degradation due to environmental factors is likely to increase. Countries that do not have a specified licensing period conduct periodic safety reviews at defined intervals (e.g., 10 years) to provide confidence in the continued ability of the concrete structures to perform required safety functions. Others, such as the United States, have a specified initial operating licensing period, but allow these licenses to be renewed for an additional period of time, with no limit on the number of renewals. In support of license renewal for these plants, guidance has been prepared documenting the regulator's technical basis for evaluating the adequacy of existing aging management programs (e.g., GALL Report). In a related activity, the IAEA is developing a document to provide an international platform for discussion between regulators and utilities regarding implementation of acceptable NPP ageing management plans (i.e., IGALL).

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