ROLE OF CONCRETE IN SUSTAINABLE DEVELOPMENT IN IRAN

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ABSTRACT

In this paper, some important factors that have significant role in production and utilization of concrete are identified. These factors are analyzed and discussed as the main components that have established the role of concrete in civil, social, economic and cultural development of societies. Some points are raised in order to provide a clear base for sustainable development.

Keywords: concrete, development, properties of concrete, cement, education, research, standards, regulations

1. INTRODUCTION

Thought and efforts of man are two key elements that have the main role in the development process. This process would lead to fruitful results by making use of continuous research and education. Certainly, development enjoys several aspects and characteristics. None of these may be considered as the main factor of development due to the variety of different branches of science and technology. Not to mention, however, that each of these factors has their own specific part in this process.

Development rate may be evaluated by scientific methods in different countries. Evaluation of the extensive field of civil engineering and its several branches, may be considered as one of these methods. Various specialized and up-dated branches of civil engineering, along with other branches of science and technology, provide the main ground for socio-economic and cultural development. Another aspect that should be considered in the development process is that, by means of right thinking, challenge and hard working, it would be possible to convert the natural and potential capacities, to man-made products through industrial cycles.

In today’s specialized world, civil engineering branches and their relevant scientific, research and professional activities, in conjunction with environmental engineering branches, fulfill the requirements of man’s life in a standard, comprehensive and perfect way.

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One of the most significant aspects that should be considered effectively in the field of civil engineering is investigation on properties of materials and relevant technologies for their production and utilization. Concrete enjoys a specific place amongst other building materials and encounters several problems regarding its production and utilization.

Due to making use of natural resources like sand, gravel and cement as its raw materials, concrete is a main factor for damaging the environment and erosion the natural resources. On the other hand, the impacts of aggressive environment on concrete construction, reduce the service life of concrete structures. All these, form a problem of this complex building material. This specific character of concrete, strongly influences economic circumstances.

Therefore, optimization of concrete as a structural material has an important role in engineering economics. Another critical point is that a great number of different professions in industries influence concrete production and utilization, directly and indirectly.

2. CONCRETE: FROM PRODUCTION TO UTILIZATION

Although the basic concrete materials consist of cement, aggregates and mixing water, but addition of some other materials is required for development of special types. The purpose of this paper is not an analytical approach to engineering, physical and chemical properties of concrete or the interaction between concrete constituents, but it is oriented towards notification of the fact, that firstly the concrete materials as simple elements have their own specific characteristics and secondly, as a composite material concrete chemical reactions, at the time of setting, after hardening, and at the time of utilization, leave special effects. Therefore, those who are somehow involved in professions related to production of concrete constituents or their composition as the end product (concrete) should focus on the basis of appropriate research about different aspects and dimensions of this complex material.

Concrete materials consist of cement, natural and artificial aggregates, natural and artificial fibres, polymer materials, admixtures (natural and artificial pozzalanas), additives (plasticizers, super-plasticizers, accelerators, expansion materials).

Interaction of chemical industries, however, implies a logical and continuous interaction between concrete and other substances. Considering the production of different types of concrete, the following types may be put forward:

- Prefabricated concrete
- Prestressed concrete (Pretensioned, post-tensioned)
- Ready mix concrete
- Lightweight concrete
- Insultion concrete
- Special concrete
- Roller compacted concrete
- Heavyweight concrete
• Shotcrete
• Preplaced concrete

More accuracy and care in production, utilization and different levels of technological trends would resolve many problems. For instance employment opportunities will be provided, construction methods will be improved, construction rate will be accelerated and therefore necessary ground for development will be provided.

Among the concrete constituents, cement has a more dominant situation. Cement is usually regarded as the symbol of industrial productivity and by some others, the cement consumption is considered as the motivator of economic wheels. Cement consumption per capita is also regarded as an index for development. These two views are somehow correct and the significant evidence for development.

Production rate of hydraulic cement in the world, in 1999 has been 1.52 billion tons. 15 countries possess 70% of the whole product of this type of cement accordingly, and the remaining 30% belongs to the other countries of the world. The 15 countries are listed below:

2. Japan  7. Germany  12. Russia
4. India  9. Italy  14. Taiwan
5. South Korea  10. Spain  15. Thailand

If the export and import rate is added and deducted to the production rate, respectively then the rate of cement consumption in every country is obtained. By dividing this figure to the population of each country, the obtained figure is the cement per capita consumption, which is regarded as a relative index of constructiveness. Table 1 shows the production, consumption, and per capita consumption of cement in some countries, in the year 2000 [1].

As may be seen from table 1, Iran is situated at the lowest rate. This table may be regarded as a comparing tool for measurement of development from quantitative point of view. Based on another view, according to the quantitative objective of the Third Development Plan (TDP), [2] it is predicted that the household density would reduce from 1.61 in 1999 to 1.12 in 2004, which is the termination year of the TDP. It means that 3.114 million residential units, will be constructed, 216000 units in urban and 954000 in rural arrears. In total, 13 million households will reside in these units. According to the implemented studies, the required cement for residential units, considering various types of structures and the methods for construction is more than 43 million tons. Assuming that the average production rate of cement per year has a constant growth, it is required that in the TDP, 8.6 million tons of cement should be consumed, merely for construction of residential units.

In case it is supposed that the cement consumption is only allocated to civil engineering activities, now we have to determine the cement quantity for other types of civil works. For determination of this amount, we may use the cement consumption rate in housing sector, to the total amount of consumed cement in the country, in the Second Development Plan (SDP).

According to the data and statistics published by the Central Bank [3], Management and Planning Organization [4-7], National Land and Housing Organization and the Statistical Center
of Iran, the average cost of per m² of residential units in urban areas, built by the private sector and started in 1999 which is the termination year of the SDP, was 467,808 and the average cost within the period of the Second Development Plan was estimated as 358,238 Rls. These costs for the completed buildings at the end of 1999 was 459,676 Rls. and its average rate was estimated as 295,997 Rls. within the period of the Second Development Plan. Provided that the figure 358,238 Rls. is assumed as the base for estimation calculations in the TDP, and if the average growth rate in the SDP is considered for the TDP, the estimated average cost for per m² of buildings would be 551,684 Rls. Undoubtedly, this figure is quite conservative and underestimated.

Based on the inaccurate information and the implemented studies, also taking into consideration the experts’ approaches, if the cost of per m² of a rural residential unit is assumed as 50% of an urban residential unit, (this rate must be about 60-70%, actually) the expenditure for per m² of rural units will be 275843 Rls. The average surface area in urban and rural residential units to be built in TDP period, are 216000 m² and 76320 m² respectively provided that the average surface area per urban and residential unit is considered as 100 and 80 m² respectively. By these calculations, the sum of 140,216 billion Rls. would be required for construction of 3,114 million of residential units. Subject to the mentioned reference it is determined that the ratio of spent credits for production of residential units, to the total spent credits within the Second Development Plan, which is 94465 billion Rls. and 101.21 billion Rls., respectively, is equal to 0.95.

Now with the assumption that this ratio within the whole 5-year period of the TDP is constant, the expenditures of other civil projects will be estimated by having the cost of residential units in this period, which is calculated as 147,600 billion Rls. Thus the proportion of construction projects in the TDP will be 51.3%. Therefore, the required amount of cement will be estimated as 88.30 million tons, which results in total, the consumption of 131.3 million tons of cement during the TDP. It means that 26.26 million tons of cement will be required per year. By calculating the 1.04 ratio of production to consumption within the SDP, the average production of cement within the TDP would reach 136.56 million tons, meaning that the annual cement production rate must be 27.513 million tons.

It is now clear that for fulfillment of the TDP, so many efforts should be made for production of cement and consequently concrete, and relevant industries.

3. CONCRETE AND CLIMATIC CONDITIONS

Since concrete as a structural material should be durable and possesses the expected service life and the required durability in the specific environmental conditions, Therefore it is required that the environmental circumstances which may influence the strength gaining process and its texture, must be taken into consideration. Application of appropriate construction details, addition of other materials to protect concrete and concrete structures and accessibility to an optimum mix design, all are of great importance. For our country with its variable climatic conditions, it is necessary to use national regulations, standards and recommendations rather than regulations of other countries that may not be compatible. It is therefore necessary to provide the codes of practice, standards etc. for the mix design and other aspects of concrete
independently, taking into account the specific climatic conditions of different areas. This requires great and realistic challenges.

Amongst the various climatic conditions, Persian Gulf coastal regions as well as northern coastal areas of the Caspian sea are the most critical areas, since these areas may be considered as the gates of development. Construction of super structures must be based on the required durability and strength, as these structures are directly related to the topic of development.

4. CONCRETE AND EARTHQUAKES

One of the most severe and critical issues in our country is seismicity and the frequent occurrence of severe earthquakes. Other natural disasters like floods, landslides etc. are also obstacles of sustainable development in our country. A short review of the earthquake issue makes it clear that a large number of research and executive undertakings for an appropriate programming regarding earthquake and how to approach its related problems, e.g. strengthening of existent buildings, as well as provision of a comprehensive plan for disaster management in Iran are all left unattained.

In a country that every 5 to 10 years an earthquake with the magnitude 7, or every 2 to year an earthquake with magnitude 6.5, occurs it is certainly necessary to consider earthquakes, and strengthening of buildings against them in long-term development plans and budgeting programs, very strongly and significantly. Undoubtedly development of the country would not be fulfilled without concrete utilization and the relationship between concrete and earthquake engineering should be maintained through continuous research and educational programs.

One of the significant cases which is to be deliberated is the concrete behaviour in earthquakes regions. In other words, the seismic behaviour and determination of specifications of the concrete, which can absorb the vibrations of earthquakes to an acceptable limit, all, require their own specific software and hardware facilities. Provision and making of required equipment will involve a great number of industrial units. It is quite obvious that a great number of research studies should be implemented on all aspects of concrete structures as related to earthquakes, and many developmental activities are required to do these research studies:

On the other hand, since the production of concrete is not mechanized, like production of steel, several factors interfere in its production and utilization. If these factors are determined through accurate scientific research processes, one of the development steps has been taken. It is interesting to know that during the 5-year, SDP, 6 severe earthquakes with 6.1-7.5 magnitude were taken place. Therefore, prediction of such events in the TDP is vital

5. CONCRETE AND STANDARDS

There is no need for reasoning that one of the significant factors for progress and development is use of standards and regulations in all activities. As regarded concrete, not only provision and application of standards is a necessary action for achieving the sustainable development, but it should also be taken into consideration that the standards up dating, by means of continuous scientific research is vital.
Ever since, in many countries, numerous standards have been provided for concrete, its materials, constituents and special types, even on concrete structural design, and analysis. The followings have to be mentioned, regarding the situation of concrete standard in our country:

- **Iranian Concrete Code of Practice (ABA)**
  This code of practice has referred to 130 codes excerpted from 110 ASTM, 23 BS, 21 AASHTO and 44 other standards. The interesting point is that only 29 Iranian standards have been quoted [8].

- **National Standards (ISIRI)**
  Yet, 62 Iranian standards have been compiled on concrete, cement and aggregates, amongst which only 6 standards on Portland cement are mandatory. Standard 3132 entitled “hot rolled reinforcement bars in concrete” is one of them. Among 29 National Standards, only 16 titles have been compiled since 1986. Generally, 45 standards have been provided or revised in 1990-2000. Another 14 standards in the field of concrete are under preparation and publication.

### 6. CONCRETE; EDUCATION, AND RESEARCH

Development is not feasible without considering its required components. Is it possible to compare these components, to each other, or, consider more significance for one of them, for instance manpower? Undoubtedly, manpower is the main requirement for sustainable development. Efficient manpower needs education and research. Regardless of priority of research and education, vice versa, or their interaction within a specific process, it is important to know that research provides the suitable ground for growth and innovation. Some of the development indices were reviewed, in the past sections of this paper and now some facts on the financial credits for research and education are put forward.

#### 6.1 Education

According to the information obtained from the data-base of Scientific Information and Documentation Center, affiliated to the Iranian Ministry of Science, Research and Technology, 59379 titles of Ph.D. and MSC dissertations have been done since 1989. This figure may be less than actual, but since these dissertations are related to all of the branches of science, and majority of unregistered dissertations in that Center are those of medical science (and therefore not included in the above-mentioned information), thus, this figure may somehow be acceptable for the present survey.

The total number of Ph.D. and M.Sc. dissertations in the field of civil engineering is 1314, amongst which 114 titles (%8.7) are directly related to concrete, 178 titles to steel (%13.5) and the rest, to other subjects.

The number of dissertations on concrete is not considerable. It is therefore obvious that higher education as well as professional education bodies should pay more attention to such fields like building materials especially concrete.
6.2 Research Credits

In the TDP\(^1\) it has been predicted that at the end of this period, proportion of the spent credits in the field of research, to the Gross Domestic Product (GDP) in public sector (Government) should reach 1% of the total credits and in the private sector to 0.50% of the whole credit of the private sector, governmental companies and banks.

In the same plan, the government is obliged to spend 15% of this credit in implementation of fundamental research works, as well as research studies, which would lead to new technological trends. Such an approach and orientation in the TDP is quite entrusting. Based on the commitments of the SDP, the fulfilment of the TDP would be planned.

In table 4, the final expenditures of the year 1999, is estimated on the basis of the rate of average definite expenditures to approved expenditures [2]. Therefore, the definite expenditures for civil construction within the SDP are given in this table. If the rate of these expenditures is considered as one of the development indices, it would be seen that the proportion of the total research credits, to the Gross National Income (GNI) in the SDP was about 0.3%, and this occurred in circumstances where the objective was achieving 0.75% of the public budget and 0.75% of the “Other resources” budget, in the SDP.

Reference to table 4, makes it clear that the proportion of research credits, to the total budget in the SDP was about 0.46%, and its proportion to the construction credits was 4.57%.

Knowing about the figures of expenses and credits of housing, urban and rural development research, it would be more clear that the proportion of this item to Gross National Income, Total National Budget and construction credits, is not satisfactory. The interesting point in this relation is the proportion of research credits to actual construction activities, which is 0.06%. In other words, for every 10000 Rs. allocated to construction activities, only 60 Rs. is spent for research works in the same field.

One of the figures which is not reflected in the table is the proportion of credits of university, industrial and technical research (in the field of urban and rural development) to the total national credit, that is 0.33%. This figure also shows the inadequacy of credits in this field. Other view to the period subject of concrete and development indicates that within SDP, 114.52 million tons of cement has been produced. Assuming that 80% of this amount is used for making different types of concrete of average 250 kg/m\(^3\) cement content, therefore 293,200 m\(^3\) concrete has been used.

If the cost of per m\(^3\) average, is 100,000 Rs., the production value of this amount of concrete is more than 29317 billion Rs. The proportion of total spent credits for research works in the field of housing, urban and rural development during the SDP (which is 55454 million Rs.) to the cost of only one of the building materials (concrete), has been about 0.2%. In these calculations, if other materials and activities are included, this percent will seriously reduce. These calculations necessitate more accurate programs for construction activities of the country.

\(^1\)Third Development Plan
7. CONCRETE AND QUALITY CONTROL

The quality issue has two specific aspects; quality management and quality control. Generally speaking, quality control is a strategic process, with the main objective of stabilization of constructional development, by means of error prevention.

Since the properties of concrete are gained during specific period of time and under certain environmental conditions, therefore the quality control process will be quite significant. Some of the manufactured products have more satisfactory quality control procedures, but some other materials like concrete has different conditions. Although the quality control of cement and other raw materials are done in quite satisfactory circumstances, but it should not be forgotten that small changes in one of its properties, may influence the concrete workability. Therefore special care should be made regarding the quality control of cement.

Mixing, transportation, compacting, curing and concreting are the main processes that greatly influence the quality of concrete.

8. CONCLUSION

Some important points are raised having significant role in the production and utilisation of concrete. These factors are analyzed and discussed.

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CRACKING IN PRECAST, PRESTRESSED DECK PLANS IN TWO RTA BRIDGES: CAUSES OF CRACKING, CONCRETE CHARACTERISTICS AND REHABILITATION OPTIONS

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ABSTRACT

Precast, prestressed concrete (PSC) planks in the decks of two RTA bridges, referred to MC and BC bridges, have exhibited cracking in the soffit of the planks and alkali-aggregate reaction (AAR) was suspected. Two PSC planks from each bridge have been examined to identify the causes of the cracking and determine their remedial needs based on the residual strength and residual expansion properties.

Based on petrographic examination and scanning electron microscopy, strong AAR was found to be the cause of cracking for both bridges. Investigation of strength properties of the concrete cores showed that significant loss in strength properties had occurred, of the order of 30% in compressive strength and up to 50% in elastic modulus. Residual expansion of the cores was determined in the laboratory under conditions of elevated temperature and humidity and it was found that the expansion potential of the cores examined was relatively small.

The in-situ cast columns of MC bridge were also examined and found to be free of AAR, of adequate strength and in sound condition. Options for the rehabilitation of the structures have been discussed. Remedial action would need to consider the economy of replacement of the whole deck. Strengthening of the affected planks using recently developed retrofitting techniques may not be a desirable long-term option for these particular structures.

Keywords: alkali-aggregate reaction, prestressed concrete, cracking, concrete strength, expansion

1. INTRODUCTION

Recently some bridges under the control of the Roads and Traffic Authority (RTA) in New South Wales have been identified in which prestressed, precast concrete planks have shown signs of distress in the form of longitudinal cracking and exudation of white materials which cover parts of the soffit of the deck planks. Alkali-aggregate reaction (AAR) was suspected as a cause of cracking. The planks are typically about 11.8 x 0.6 x 0.35 m and are placed 10 mm

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apart over each span to form the deck. The spaces between the planks are filled with a sealer, and a layer of in-situ cast concrete (around 140 mm) forms the wearing course of the deck.

The cracking of the deck planks may influence the load-bearing capacity of the decks, and this paper reports on an investigation undertaken on two of the bridges to find out the causes of cracking and to determine the current properties of the concrete to assist with decision making on appropriate remedial measures for the decks.

Although columns in the two bridges are, at present, free of cracking, they were also included in the investigation to determine whether or not they would develop cracking in the future.

2. THE BRIDGES

Two bridges were investigated and are referred to MC bridge and BC bridge which were built in 1989 and 1977, respectively. They have 6 and 4 spans, and 16 and 17 planks per span, respectively. Figure 1 shows the cross section of the deck at a pier for MC bridge, and Figure 2 shows details of steel reinforcement in the planks, which are similar for the two bridges. The top surface of the deck, consisting of in-situ cast concrete, has developed longitudinal parallel cracking which is more extensive in MC bridge than BC bridge, and the soffit of the PSC planks also exhibit this type of cracking as seen in Figure 3. Other bridges of similar construction have also shown similar cracking features.

![Figure 1 Cross section of the bridge deck at a pier, showing arrangement of planks](image1)

![Figure 2 Cross section of a plank showing details of reinforcement and prostrating wires](image2)
Figure 3 Cracking of the wearing course concrete (top) and beam plank (center) of MC bridge, and beam plank of BC bridge

Documentation on the drawing of the PSC planks for MC bridge specifies a minimum 28-day compressive strength of 40 MPa with a transfer strength of 35 MPa. The calculated hog of 40 mm at 28 days was based on the following assumptions that had been made at the time of construction:
Density = 2600 kg/m$^3$; elastic modulus at transfer = 36.7 GPa; steam curing at 70°C for 8 hours; plank self weight = 6.5 tonnes (with no external load); storage after steam curing in open air at 20°C average temperature, and RH of 50-75%. Although no such documentation was found on BC bridge planks, it can be expected from the similarity in design that they had similar concrete mix designs and strength properties.

The columns in the bridges were all cast-in-situ reinforced concrete of 30 MPa strength grade, which is adequate for their non-aggressive exposure conditions.

3. CORE SAMPLES

3.1 Sampling of PSC Planks
In MC bridge, the soffit of many planks showed cracking which was worse at the two ends of the planks where they rest on the cross beam, and where there is more water leakage. The cracking of the wearing course concrete and the presence of joints at the cross beams may have caused enhanced AAR in the planks in these areas by allowing more water penetration to the planks. Two planks in the southernmost span were selected and two cores of 95 mm diameter drilled in each. These were designated cores M1-M4. Cores M1 and M3 were from the first plank west of the bridge centre-line and cores M2 and M4 from the second plank west of it.

In BC bridge, cores B1 and B2 were taken from cracked planks under a cracked area of the wearing course near the center-line of the bridge, and cores B3 and B4 from uncracked planks near the kerb.

3.2 Sampling of Columns
The columns of both bridges are free of cracking. However, it was suspected that similar materials must have been used in their manufacture, and it was necessary to identify whether or not the columns may develop distress in the future. Therefore two cores were taken from each
of the two representative columns in MC bridge for AAR determination and strength testing. These were designated MC-1 to MC-4.

4. VISUAL FEATURES OF CORES

4.1 MC Bridge
The aggregate in all the four cores from PSC planks in MC bridge is a hornfels or meta-sediment, and exhibits internal cracking, typical of AAR. Many aggregate particles show a wet-looking appearance due to AAR gel impregnation. The fracture surfaces of cores show AAR rims which vary in thickness from core to core. Some cracks and air voids appear to be filled with AAR products. Cracks about 25-30 mm deep are present in some cores. Strong AAR seems to present in the cores.

Cores taken from the columns of MC bridge showed no sign of AAR and no visible defect.

4.2 BC Bridge
The aggregate in all the four cores from BC bridge contain crushed quartz gravel with other sedimentary rocks and meta-sediments.

In cores from uncracked planks no unusual feature is present in the concrete. In cores from cracked planks, strong AAR rims are present at the fracture surfaces of the cores. Many aggregate pieces show internal fractures parallel to the core surface, and some are partially filled with AAR products. Strong AAR exists in the latter cores.

Examples of the visual features of cores are given in Figure 4 and Figure 5 for MC and BC bridges, respectively. These features suggest that strong AAR has occurred in the cracked planks of both bridges.

![Figure 4](image) Fracture surfaces of core M1 (above) and two segments of core M2 (right) showing strong AAR rimming. AAR gel covers the steel/concrete interface in M2, and fills some voids in M1.
5. INVESTIGATION OF CORE SAMPLES

Detailed examinations and testing were carried out on all the cores. Those from the PSC planks were subjected to petrographic examination, Scanning Electron Microscopy (SEM) combined with Energy-Dispersive X-ray (EDX) analysis, determination of residual alkali content, modulus of elasticity, ultrasonic pulse velocity, compressive-strength, expansion potential at 100% RH 40°C and in 1M NaOH solution at 40°C. Due to the small amount of core samples available not all the tests could be done on the samples available. Cores from the columns were also used for these tests, except modulus, pulse velocity and expansion tests. Results of the above examinations and tests are summarised below.

5.1. Petrographic examination

Cores M3 and M4 representing the two planks of MC bridge, and cores B1 (with AAR) and B4 (free of AAR) representing the two planks of BC bridge were examined petrographically. Representative cores from the columns of MC bridge were also examined, and the results are summarised below.

5.1.1 Cores M3 and M4

The coarse aggregate in these cores appears fissile and shows orientation and banding when seen by the unaided eye. Under the petrographic microscope, the aggregate is a fine to medium-grained hornfels with a non-uniform texture. Some quartz rich zones are separated by bands of micaceous zone giving the banded appearance. The size of the quartz grains varies from very fine to medium, and some aggregate pieces are entirely of a very fine mixture of quartz, micaceous materials with some iron oxide staining. The latter is present in most aggregate pieces giving them a slight brownish tinge. Feldspar is also identifiable in aggregate pieces which have a coarser grain size.
Quartz grains show some elongation in the coarser aggregate pieces, in which some quartz patches exhibit quartzitic features with welded joints and lenticular appearance, indicating the metamorphic nature of the aggregate.

The sand fraction consists of rounded to angular quartz grains of which some are monocrystalline and some polycrystalline. All show moderate to high undulose extinction angles. The sand also appears to have a metamorphic nature.

Both the coarse and the fine aggregate are considered to be susceptible to AAR. Microcracking is evident in the aggregate itself as well as partially around it and extending into the matrix. Fine microcracking is seen throughout the matrix. Some coarse aggregate particles have AAR gel on their periphery, and some cracks appear to be partially filled with the gel. Some of the sand grains also appear to have reacted.

5.1.2 Cores MC-1 and MC-3 from columns

The coarse aggregate in the columns was different from that in the PSC planks, but still of a metamorphic nature of gneissic type. Both the coarse and fine aggregates contain significant amounts of moderately to highly strained quartz which are considered to be susceptible to AAR. However, no sign of AAR could be detected by the petrographic examination. This indicated that the progress of any AAR would depend on the amount of available alkali in the concrete.

5.1.3 Cores B1 and B4

The coarse aggregate particles show a variety of features probably indicating different origins. Among these, some particles are heavily stained by iron oxide and have a very fine texture similar to chert. Some other particles are of medium-grained sandstone texture with tightly packed and well sorted quartz crystals and some iron oxide staining. A large proportion of the quartz grains show undulatory extinction. Some other particles show extreme deformation with highly strained, elongated quartz that have highly sutured boundaries and show considerable recrystallization of microcrystalline quartz at the grain boundaries. Yet other particles appear to be of a deformed granitic nature (gneissic), with highly strained quartz.

The fine aggregate is a natural sand probably of the same origin as the coarse aggregate and shows similar petrographic features. These features are indicative of strong alkali reactivity in concrete. The matrix of core B1 shows considerable microcracking which has resulted from AAR, and some of the cracks appear to contain AAR gel. Signs of reactivity are seen with many coarse aggregate pieces as well as with sand grains. No wide microcracking is seen in the section from core B4, although a considerable number of fine microcracks are present in the matrix running between sand grains and sometimes originating from the coarse aggregate. These microcracks may be due to thermal shock that may have affected the concrete element as a result of steam curing. No visual signs of AAR are present in core B4.

6. SEM AND EDX ANALYSIS

Examination by SEM and EDX analysis of the reaction products, clearly showed that strong cases of AAR exist in the concrete of the cracked PSC planks in both bridges. In addition, secondary or delayed ettringite (hydrous calcium-sulfoaluminate) was also detected, but the latter was much less prevalent than the AAR products. Moreover, the ettringite in MC bridge was largely in the pores and only in a few locations was indicative of expansive ettringite.
whereas ettringite in BC bridge was more similar to the delayed formation of ettringite, albeit in small amounts. No evidence of AAR was found in the cores from the columns of MC bridge.

A representative example of extensive formation of AAR gel is shown in Figure 6. This gel forms within the aggregate causing expansion and stress build up that eventually causes cracking in the aggregate and mortar phases of the concrete. The gel impregnates the surrounding mortar and part of the cracks that may be formed around the aggregate. Many sites of this nature were observed in the affected concretes of both bridges. The white reaction rims on the reacted aggregate contain crystalline AAR products (Figure 7), which is indicative of advanced reaction in both concretes. Some other Na-rich phases (Figure 8) are indicative of a high alkali, Na-rich cement in the concrete.

![Figure 6 AAR gel formation covering a large area around the reacted aggregate site. The composition of the cracked gel is rich in Na and Si as indicated by the EDX spectrum](image-url)
Figure 7 Crystalline (Rosette) AAR products in the white reaction rims seen within the aggregate boundaries
Other than the usual form of ettringite lining in some air voids, a few locations were seen in MC bridge concrete, where the form of ettringite at the cement/aggregate interface (Figure 9) may indicate expansive ettringite as judged from its radial growth at the reaction site into the interfacial paste. In BC bridge concrete, locations were seen where AAR products were associated with ettringite, as indicated by the composition of the materials seen in Figure 10. The ettringite appears to have been formed at a later stage than the AAR gel in cracks at the aggregate interface, as seen in Figure 11, where ettringite crystals are shown below the layer of AAR gel at the aggregate surface. Figure 12 shows another site of ettringite formation where it could indicate some contribution to concrete expansion, by its compact form that has filled a gap at cement/aggregate interface. Such locations were far less frequent than the AAR sites, in both bridges, and it appears that AAR is by far the main cause of the observed cracking.
Figure 10 Mixture of AAR and ettringite
Figure 12: Etching layer bound at aggregate interface and mixed with small amounts of AAR.

Energy (keV)

Figure 11: Etching Pyrex forming a layer of AAR gel at the aggregate interface.

Energy (keV)
Table 3. UVP and elastic modulus of core specimens

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<td>120</td>
<td>110</td>
</tr>
<tr>
<td>155</td>
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<td>165</td>
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<tr>
<td>175</td>
<td>160</td>
<td>165</td>
<td>170</td>
<td>150</td>
<td>140</td>
</tr>
</tbody>
</table>

7. STRENGTH PROPERTIES OF CONCRETE

For AAR not to have occurred, the UVP (or compressive strength) should not have been different and probably low in which case the compressive strength would not be 7.2. There is little or no evidence of a difference in the compressive strength. The UVP values on the table are the same as those found in the UVP for concrete and in the AAR produced by the AAR product. The AAR values on the table are for the AAR product and not for the AAR product alone.
stiffness by about 60%. This indicates a significant deterioration in the engineering properties of the AAR affected PSC planks.

Elastic moduli of Cores B2 and B3 and their compressive strength were also measured by the static method. Core B2 (with AAR) had a compressive strength of 41.5 MPa and an elastic modulus of 15.5 GPa, whereas Core B3 (no AAR) had a compressive strength of 57.2 MPa and modulus of elasticity of 24.5 GPa. Based on these values the strength and modulus for BC bridge planks have been reduced by about 28% and 37%, respectively. These significant reductions should be taken into account in making decisions regarding rehabilitation of the structures.

The cores from the column of MC bridge had compressive strengths ranging between 33 and 35 MPa with an average of 34 MPa. Considering the benign environmental conditions of this bridge, this strength is adequate for continued service life of the columns.

8. RESIDUAL ALKALI CONTENT OF CONCRETE

The soluble alkali was extracted from the concrete and analysed to determine the residual alkali content in the concrete. The results are given in Table 4.

The amount of residual alkali in the PSC planks is considered high and adequate to sustain further reaction in the concrete. Therefore, provided that reactive components are still present in the concrete (which is usually the case), further expansion and cracking is likely in the concrete. Core B3 which is free of AAR has the least amount of residual alkali, and at this level would not support development of AAR in the concrete. Therefore, this plank would remain free of cracking.

The amount of soluble sulfate in the concrete is relatively high, particularly if converted into ettringite, and it is likely that some additional ettringite precipitation may take place within the existing micro-cracks. However, this may have only a marginal effect on the magnitude of future expansion.

Table 4 Residual alkali and sulfate contents of cores

<table>
<thead>
<tr>
<th>Core</th>
<th>Na₂O%</th>
<th>K₂O%</th>
<th>Na₂O equiv. %</th>
<th>Concrete density kg/m³</th>
<th>Na₂O equiv. kg/m³</th>
<th>Corrected Na₂O equiv. kg/m³</th>
<th>SO₄ %</th>
<th>SO₄ kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>0.088</td>
<td>0.047</td>
<td>0.119</td>
<td>2399</td>
<td>2.71</td>
<td>2.36</td>
<td>0.052</td>
<td>1.25</td>
</tr>
<tr>
<td>M2</td>
<td>0.094</td>
<td>0.055</td>
<td>0.130</td>
<td>2374</td>
<td>2.93</td>
<td>2.58</td>
<td>0.053</td>
<td>1.26</td>
</tr>
<tr>
<td>M4</td>
<td>0.097</td>
<td>0.065</td>
<td>0.140</td>
<td>2400</td>
<td>3.20</td>
<td>2.85</td>
<td>0.034</td>
<td>1.30</td>
</tr>
<tr>
<td>B2</td>
<td>0.090</td>
<td>0.053</td>
<td>0.125</td>
<td>2367</td>
<td>2.96</td>
<td>2.81</td>
<td>0.057</td>
<td>1.35</td>
</tr>
<tr>
<td>B3</td>
<td>0.031</td>
<td>0.098</td>
<td>0.095</td>
<td>2376</td>
<td>2.26</td>
<td>2.11</td>
<td>0.026</td>
<td>0.62</td>
</tr>
<tr>
<td>M-C-2</td>
<td>0.032</td>
<td>0.022</td>
<td>0.047</td>
<td>2412</td>
<td>1.12</td>
<td>0.77</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M-C-3</td>
<td>0.027</td>
<td>0.013</td>
<td>0.036</td>
<td>2412</td>
<td>0.86</td>
<td>0.51</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* an estimated 0.35 kg Na₂O/m³ has been deducted for contribution from the metamorphic aggregate, and 0.15 kg/m³ for the gravel aggregate.
From the data in Table 3 it is clear that the cements used in the PSC plank represented by core B3 is different from the others. Firstly, the nature of alkalies in the concretes are different. B2 and all MC cores being rich in Na₂O while B3 containing more K₂O. Secondly, the other cores contain more than twice the amount of SO₄ in B3. The residual alkali values indicate that further deterioration could be expected in the affected planks, but the uncracked plank would remain sound.

The amount of residual alkali in the columns appears to be very low, indicating that although the aggregate is reactive, AAR is unlikely to develop due to the insufficient amount of alkali. The cement used for the manufacture of the columns must have been a low alkali cement.

9. RESIDUAL EXPANSION POTENTIAL OF CONCRETE

Residual expansion measurements were conducted on the remaining portions of the cores. Portions of cores M1 and M3 and a portion of core B1, (each 150 mm long), representing one of the planks from each bridge were separated and prepared for expansion measurement. After the initial conditioning, core M3 and B1 were stored at 40°C, 100% RH conditions and core M1 in 1M NaOH solution at 40°C, and their expansion was measured regularly. Expansion under the former storage conditions provides information on the residual expansion of concrete under the existing alkali levels, and expansion in 1M NaOH, 40°C indicates whether reactive components are still present in the concrete, and also the maximum expansion potential of the concrete.

Figure 13 shows the expansion curves obtained under the two storage conditions. Core M1 in 1M NaOH, 40°C continues to expand, indicating the presence of reactive components, and will continue until the maximum expansion potential is reached. Expansion of core M3 under 100% RH 40°C, has been slower and smaller, as expected, and has reached 0.07% at 340 days of storage, of which around 0.04% could be due to water absorption rather than AAR expansion. Assuming 0.03% residual expansion, each affected plank (600 mm wide) will expand laterally by 0.24 mm which could be accommodated by the sealed gap between the planks. However, this could cause further longitudinal cracking in the planks. Due to the prestressing effects, the longitudinal expansion may be less than the lateral expansion.

Figure 13 Expansion curves for core segments under the conditions indicated
Expansion of core B1 has reached 0.045% at 340 days of storage, of which 0.025% is probably due to water absorption, and the remaining 0.02% is the residual AAR expansion. This lower expansion compared to core M3 is probably because the BC plank has already undergone more extensive AAR than the MC plank. The expected lateral expansion of the plank (600 mm wide) of 0.12 mm could also be accommodated by the gap between the planks, otherwise the movement across the deck would be 2.0 mm if all the planks expand to this level. Actually, some of the planks are not expansive. These movements should be taken into account in any rehabilitation program.

10. OVERALL ASSESSMENT OF CONCRETE AND SUGGESTED REMEDIAL ACTION

10.1 MC bridge
Visual, petrographic and SEM examinations have shown that the PSC planks examined have significantly suffered from alkali-aggregate reaction and, in one core segment, the steel/concrete bond has been lost and AAR gel accumulated at the interfacial zone. Measurement of ultrasonic pulse velocity, strength and modulus of elasticity have shown that the strength properties of the AAR-affected concrete planks have significantly deteriorated as a result of the reaction.

A large number of the planks are affected by the AAR, and due to the method of construction their individual replacement is very difficult and costly. Considering the advanced stage of AAR, and existing potential for further expansion and cracking, strengthening of the planks using techniques such as composite fibre-epoxy bonding may not be advisable. In any case the wearing course of the deck would need major rehabilitation, including water-proofing and reconstruction. Another option may be replacement of the whole deck, which may be less expensive in the long term. The economy of each option needs to be considered before taking any remedial action.

The columns in MC bridge are sound and of adequate strength. No remedial action seems to be required for the columns.

10.2 BC bridge
A survey of the deck is needed to determine the number of the affected planks. If the number is large the remedial actions would be similar to those suggested for MC bridge. A few affected planks could probably be tolerated after in-situ strengthening.

11. CONCLUDING REMARKS

Two PSC planks from each of MC and BC bridges have been examined for AAR and residual strength and potential for further expansion. For MC bridge both planks were found to have suffered strong AAR, and their strength properties significantly deteriorated such that the compressive strength was reduced by 35% and elastic modulus by about 50%. For BC bridge, one of the planks was free of AAR and in sound condition, whereas, the other had undergone a strong AAR and its compressive strength and modulus of elasticity have been reduced by 28% and 37%, respectively, compared with the sound plank. The residual expansion of the affected planks appear to be small, but not insignificant.
It has been suggested that individual AAR-affected planks could be replaced if there are only a few of them, although they could also be strengthened. However, where many of the planks are affected, the rehabilitation should include placement of an impermeable layer on top of the deck, and application of epoxy-bonded composite fibre sheet to the soffit of the planks to increase their load-bearing capacity. This would require structural calculations to determine the amount and distribution of the composite materials. The disadvantage of this method is that AAR could progress and cause further damage over many years.

Another option would be to replace the whole deck, and an economic evaluation of these option would be needed before taking any remedial action. In any case, the columns appear to be sound and of adequate strength for supporting a new or a rehabilitated deck.

Disclaimer: The opinions expressed in this paper are those of the authors and not necessarily of their respective organisations.
SHRINKAGE OF CEMENT COMPOSITES REINFORCED WITH SISAL AND COCONUT FIBRES

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K. Ghavami, Civil Engineering Department, Pontificio Universidade Catolica, PUC-Rio, Brazil
M. A. Sanjuan, Institute Español Del Cement y sus Aplicaciones

ABSTRACT

The influence of sisal and coconut fibres on the free plastic shrinkage and long-term drying shrinkage of cement matrices is evaluated experimentally using factorial design. A special chamber was designed for the experimental investigation where the test specimen are exposed to a wind speed of 0.5 m/s and temperature of 40°C for a period of 280 minutes. The drying shrinkage was studied for a period of 320 days in the environmentally controlled laboratory with 21.0 ± 1.6 °C and 41.0 ± 8.6% relative humidity. In this paper the results of the research on the influence of mix proportions and the fibre types, length and their volume fractions on the drying shrinkage of cement composites with and without fibres are presented. The inclusion of 0.2% volume fraction of 25mm sisal fibres in cement mortar significantly reduces free plastic shrinkage and up to 3% volume fraction of 25mm of vegetable fibre reinforcement increases the drying shrinkage of the matrix from 0% to 27% depending on the variables studied.

Keywords: sisal and coconut fibres, cement mortar, composites, factorial design, shrinkage.

1. INTRODUCTION

Plastic shrinkage is the dimensional change in all fresh cement based materials within the first few hours after it has been placed in the formwork. This type of shrinkage is not unacceptable in itself but it is sometimes accompanied by the development of cracks, which are unsightly and objectionable. When the cement matrix is placed, the aggregates and cement start to settle and water rises or bleeds to the surface. The disappearance of the sheen from the surface of the concrete, mortar or paste indicates the time when the rate of evaporation exceeds the rate of bleeding water rising to the surface. The time required to attain this condition will be influenced by the temperature, wind velocity and relative humidity of the air, the temperature and bleeding characteristics of the cement.

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Plastic shrinkage cracks may develop when the surface of the cement matrix has some initial rigidity. At this stage, the matrix cannot accommodate the rapid volume change of plastic shrinkage by plastic flow because it has not developed sufficient strength to withstand tensile stress. The use of low modulus fibres at a volume fraction smaller than 0.3% is one of the effective methods to reduce plastic shrinkage and cracking [1-7].

Long-term drying shrinkage is an inherent property of all cement based materials. Hardened cement paste has a high drying shrinkage; concrete, on the other hand, shows relatively less shrinkage because the moisture movements are restrained by the rigidity of the aggregates [2]. The effect of fibres on the drying shrinkage of concrete, based on the few results available, are not yet conclusive [8-11]. For example steel fibre has very little effect on the shrinkage of concrete [8], and can reduce the shrinkage by up to 40% [8]. Glass fibre has reduced the shrinkage of cement mortar matrices between 20 and 30%[10-11]. The vegetable fibres such as sisal and coconut fibres are porous and can create moisture paths deep into the matrix and could increase the shrinkage as confirmed by the authors' investigations [1,6].

The main objectives of this paper are to determine whether low modulus sisal fibre might be useful for controlling the free early age shrinkage of mortar, to present a model for predicting the free plastic shrinkage of sisal fibre reinforced mortars and to study the influence of sisal and coconut fibres of various volume fraction and lengths on the drying shrinkage characteristics of the developed composites.

2. EXPERIMENTAL PROCEDURES

2.1 Materials
The sisal and coconut fibres used in this investigation were of Brazilian production. The sisal and coconut fibres of 25 mm long had a mean density, elastic modulus, and tensile strength of, respectively, 0.90 g/cm³, 19 GPa and 577 MPa and 0.80 g/cm³, 3.5 GPa and 174 MPa.

The sand and cement employed in the free plastic shrinkage tests followed the Spanish Standard with a maximum particle size of 2 mm and ordinary Portland cement “OPC” (CEM I 42.5 R), respectively. The fineness modulus of the sand employed in the drying shrinkage tests was 2.81. Tap water was used in all mixes. Chemical and physical properties of the used cements are presented in Table 1.

2.2 Methods
The method which enables to measure accelerated horizontal deformation of fresh mortar specimens of 150 mm x 1200 mm and 15 mm, using dial gage extensometers was used in this study to determine the free plastic shrinkage of the composites [12]. A conventional pan mixer was used to manufacture two specimens of each mix. Immediately after casting, the gages were located on the samples, the chamber closed and set to hold wind speed and temperature of 0.5 m/s and 40°C, respectively, in the interior. Free plastic shrinkage tests started at this moment and measurements were recorded at regular intervals of time up to 280 min when the free plastic shrinkage was nearly complete. A complete factorial design was used to define the experimental program. The factors studied in this investigation were: a) water/cement ratio (X₁); b) sand/cement (X₂); and c) percentage of fibre (X₃). These factors are the common variables considered in the design of fibre reinforced cement mixes. The lower and upper levels selected for each factor are presented in the axis of Figure 1. The mix proportions are numbered and shown at the corners of the cube.
Table 1 Chemical and physical properties of the cementing materials

<table>
<thead>
<tr>
<th>Property</th>
<th>OPC Blue Circle</th>
<th>CEM I 42.5 R</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Chemical properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CaO (%)</td>
<td>64.7</td>
<td>63.3</td>
</tr>
<tr>
<td>SiO₂ (%)</td>
<td>20.7</td>
<td>18.9</td>
</tr>
<tr>
<td>Al₂O₃ (%)</td>
<td>4.6</td>
<td>3.8</td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>3.0</td>
<td>3.9</td>
</tr>
<tr>
<td>MgO (%)</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td>SO₃ (%)</td>
<td>3.0</td>
<td>2.9</td>
</tr>
<tr>
<td>Na₂O (%)</td>
<td>0.13</td>
<td>0.15</td>
</tr>
<tr>
<td>K₂O (%)</td>
<td>0.65</td>
<td>1.05</td>
</tr>
<tr>
<td>Loss on ignition (%)</td>
<td>1.3</td>
<td>3.17</td>
</tr>
<tr>
<td>Soluble residue (%)</td>
<td>0.38</td>
<td>1.89</td>
</tr>
<tr>
<td>b) Physical properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fineness (m²/kg)</td>
<td>353</td>
<td>-</td>
</tr>
<tr>
<td>Setting time (initial – min.)</td>
<td>134</td>
<td>-</td>
</tr>
<tr>
<td>Compressive strength (MPa) at 7 days</td>
<td>47.2</td>
<td>-</td>
</tr>
</tbody>
</table>

The drying shrinkage test specimens were cast in a four-cavity plastic mould of internal dimensions 90 mm in diameter x 300 mm in length. Three pairs of studs were placed internally in each cavity before casting to allow fixing of the screws to be used as strain measurement points. Sisal and coconut fibres of 25 mm long were mixed randomly with cement mortar in a pan mixer. To produce a well homogeneous fibre cement mortar mix the following procedure was adopted. First all the sand was placed in the mixer and then 40% of the total required water was added to the running mixer. In order to avoid clumping of fibres and also to keep the mix wet enough, 35% of water and the fibres were slowly added. After placing all the fibres and the whole cementitious material the remaining water was added and the mixing process was continued for about 5 minutes to enhance fibre dispersion.

The specimens were cast in three layers using external vibration. The time of vibration was established according to recommendation made by ACI 544.2R [13]. Strains were measured over a gauge length of 254 mm using a mechanical gauge, which had a sensitivity of 2.5 x 10⁻⁶. Readings were taken on the three gauge lengths (120° spaced) of the specimens. The temperature and relative humidity of the concrete laboratory monitored during the period of test were, respectively, 23.13°C ± 1.58°C and 41.00% ± 8.60%.

The specimens were water cured at 18°C during the first 28 days. The influence of fibre type and volume fraction and matrix composition on the shrinkage of the material was studied. The experimental program is presented in Table 2. In this Table, the following abbreviations are used to represent fibre type, fibre length, volume of fibre, cement matrices and cure condition:

- M1 – cement mortar mix proportions by weight 1:1:0.4 (cement: sand: water);
- M2 – cement mortar mix proportions by weight 1:2:0.4 (cement: sand: water);
- S – sisal fibre;
- C – coconut fibre;
Number after the fibre type - volume fraction of fibre (%);
Figure 1 Factorial design $2^3$

Table 2 Experimental mixes used for studying the drying shrinkage of the composites

<table>
<thead>
<tr>
<th>Mix</th>
<th>Mortar mix proportions (by weight)</th>
<th>Fibre Type</th>
<th>Fibre Volume (%)</th>
<th>$l$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>1:1:0.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M2</td>
<td>1:2:0.52</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M1S325</td>
<td>1:1:0.4</td>
<td>Sisal</td>
<td>3</td>
<td>25</td>
</tr>
<tr>
<td>M1S225</td>
<td>1:1:0.4</td>
<td>Sisal</td>
<td>2</td>
<td>25</td>
</tr>
<tr>
<td>M1C325</td>
<td>1:1:0.4</td>
<td>Coconut</td>
<td>3</td>
<td>25</td>
</tr>
<tr>
<td>M1C225</td>
<td>1:1:0.4</td>
<td>Coconut</td>
<td>2</td>
<td>25</td>
</tr>
<tr>
<td>M2S225</td>
<td>1:2:0.52</td>
<td>Sisal</td>
<td>2</td>
<td>25</td>
</tr>
</tbody>
</table>

3. RESULTS AND DISCUSSION

3.1 Plastic shrinkage

Figure 2 presents the mean values and the coefficient of variation of the free shrinkage of the composites. The results can be combined in seven different ways to estimate the main effects and interactions of the variables. The average effect of fibre content ($X_3$) on the free plastic shrinkage of mortars is a reduction of $769.5 \mu\varepsilon$ in its value as given in Table 3 and Figure 2. This reduction is greater with w/c ratio of 0.5 than with w/c ratio of 0.45. This fact shows that the factors $X_3$ and $X_1$ do not present an additive behaviour and that there is an interaction between them.
An analysis of variance was performed to show which effects are significant. The significance of each effect was tested at confidence levels of 99% using the F test. The results indicated that the main factors and the interacting factor $X_2 \times X_3$ are significant. A multiple linear regression analysis was carried to the set of experimental data producing equation 1.

$$FPS = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \varepsilon$$  \hspace{1cm} (1)

<table>
<thead>
<tr>
<th>Main effects ($\mu\varepsilon$)</th>
<th>Interactions ($\mu\varepsilon$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$X_1$</td>
<td>$X_2$</td>
</tr>
<tr>
<td>178</td>
<td>-220</td>
</tr>
</tbody>
</table>

In this equation: FPS is the free plastic shrinkage; $\{\beta_i\}$ are the unknown regression coefficients; $\{X_i\}$ are the factors and $\{\varepsilon\}$ represents the random error. The fitted model is given by equation 2.

$$FPS = 2370 + 622 W/C - 110.25 S/C - 2200\% isisal$$  \hspace{1cm} (2)

Hyperplane in the three dimensional space of the independent variables $\{X_i\}$ of equation 2, is unlikely to be a reasonable approximation of the response surface over the entire space of the independent variables, but for the relatively small region of practical application studied, the errors observed were smaller than 15% and are well acceptable.

![Figure 2 Free plastic shrinkage test results](image)

3.2 Drying shrinkage

Shrinkage and loss of mass measurements were started at the age of 28 days. Influence of fibre type and volume fraction on the drying shrinkage of the matrices can be seen in Figure 3. The density of the specimens after 28 days of curing and the loss of mass due the drying shrinkage after 320 days are given in Table 4.
The shrinkage of the matrix increases when vegetable fibres are present in the mixture. The higher percentage of fibres added, the higher is the shrinkage. This trend could be attributed to the porosity of vegetable fibres, which create more moisture paths into the matrix. The porous nature of the vegetable fibres can be seen in Figure 4 from the scanning electron micrograph of the sisal fibre. Specimens reinforced with sisal fibre shrank more than those reinforced with coconut fibre. This difference increased with the increase of the amount of fibre present in the mix. For example, specimens reinforced with 2% and 3% of sisal fibre present values of shrinkage 0.3% and 8.2% higher than those observed from specimens reinforced with, respectively, 2% and 3% of coconut fibre. The higher water absorption of sisal fibre, 230% compared to 100% of coconut fibres, may be the reason for the higher drying shrinkage of the composites reinforced with sisal fibres.

Table 4 Specimen density after 28 days of curing and shrinkage and loss of mass after 320 days of drying.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Density after 28 days of curing (kg/m³)</th>
<th>Drying shrinkage After 320 days of drying (µε)</th>
<th>Loss of mass per specimen after 320 days of drying (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>2275</td>
<td>1285.00</td>
<td>216.25</td>
</tr>
<tr>
<td>M1C225</td>
<td>2180</td>
<td>1412.50</td>
<td>228.75</td>
</tr>
<tr>
<td>M1C325</td>
<td>2163</td>
<td>1511.67</td>
<td>235.75</td>
</tr>
<tr>
<td>M1S225</td>
<td>2175</td>
<td>1416.00</td>
<td>226.75</td>
</tr>
<tr>
<td>M1S325</td>
<td>2160</td>
<td>1636.67</td>
<td>242.25</td>
</tr>
<tr>
<td>M2</td>
<td>2266</td>
<td>935.00</td>
<td>252.50</td>
</tr>
<tr>
<td>M2S225</td>
<td>2118</td>
<td>1239.17</td>
<td>253.00</td>
</tr>
</tbody>
</table>

When sisal and coconut fibres were added to the mix the mass loss due to drying of the matrix slightly increased (Table 4). For example, composites reinforced with 2% and 3% of sisal fibre and cured in water increased the loss of mass of the matrix in, respectively, 4.4% and 12% after 320 days of drying. For the same condition, the increase in the shrinkage of the matrix was more significant. Composites reinforced with sisal fibres exhibited a slightly higher mass loss as compared with those reinforced with coconut fibres.

Figure 3 Influence of fibre type and volume fraction on the drying shrinkage of the matrices.
Aggregate content in concrete or mortar is one of the most important factors affecting drying shrinkage. Keeping the water/cement ratio constant, an increase in the aggregate content will result in a reduction of shrinkage strains. For the same cement:sand ratio an increase of the water/cement ratio is also known to increase the shrinkage of the material [14]. In Figure 5 it can be seen that the shrinkage of the mix M1 is considerably higher than that of the mix M2. At the age of 320 days, the shrinkage of the mix M1 was 27% higher than that observed for the mix M2. This is mainly due to the smaller volume of cement paste in mix M2. The presence of 2% of sisal in the mix did not change this trend but slightly reduced the difference. The mix M1S2S1 exhibited shrinkage 15% higher than the mix M2S225.

Figure 4 Porous nature of sisal fibre microstructure

Referring to Figure 6 which shows the shrinkage-loss of mass curves for the mixes M1, M2, M1S225 and M2S225 it can be seen that for the same loss of mass, the shrinkage of the mix M1 is higher than that observed for the mix M2. This is related to the amount of aggregate and cement paste in the mix. To have the same composite shrinkage, the mix with lower content of paste needs to produce more shrinkage in its paste and thus it needs to lose more water.

Figure 5 Influence of the mix proportions on the drying shrinkage of the material
Figure 6 Loss of mass–shrinkage curves for the mixes M1, M2, M1S225 and M2S225

4. CONCLUSIONS

The free plastic shrinkage of composites reinforced with sisal fibre has been studied using a factorial design of experiments. This technique saves time, resources and investigation costs and leads to a mathematical model resulting in good correlation with the experimental ones in the range of factors investigated. Low volume sisal was found to be extremely effective in reducing free plastic shrinkage of cement mortar mixes.

Drying shrinkage is increased up to 27% by the presence of up to 3% volume fraction of sisal or coconut fibres. The presence of vegetable fibres appears to create moisture paths deep into the mortar which enhance the route of moisture loss and aid the development of higher drying shrinkage strains.

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STRUCTURAL APPRAISAL AND REHABILITATION OF BRIDGES AND VIADUCTS

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ABSTRACT

A study is in progress with the targets of monitoring, aging management, repair and strengthening of a stock of bridge structures. For this purpose a review of the structural appraisal and restoration interventions of these structures has been performed. Among the numerous aspects of the problem, the following features have been individualized:

- Methods and procedures for the assessment of structural reliability;
- Planning on appraisal with regard to the refinement of the structural analysis and to the costs;
- Research on information regarding construction techniques, properties of materials and computational methods used in design of the structures;
- Materials and techniques for structural repair and strengthening;
- Arrangement of procedures to develop a data-base of the frequent damages, and of the techniques and materials for repair and strengthening;
- Development of a monitoring system.

On the basis of the acquired results a model of a bridge management system is in definition.

Keywords: appraisal, bridge management system, monitoring, rehabilitation, database

1. INTRODUCTION

The main aspects of the management of existing bridges and viaducts are life prediction and maintenance management, structural rehabilitation and restoration of initial serviceability, technologies and materials for this purpose.

The inspection of the studies carried out on these subjects shows that the fundamental aspects of the problem are:

- to answer the question of whether the structure is safe enough at present. It is made by means of definition of the appraisal path, that is cyclical and consists of measurements and tests;

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monitoring plan can be prepared, the information are studied and analysed and adequacy is checked;

- to propose typical repair and strengthening interventions on structural components in order to restore or to improve the initial performance conditions;
- structural and serviceability appraisal of the interventions performed;
- monitoring plan of structures after interventions, if it is necessary;
- upgrading of repair and strengthening technologies employed on the basis of evaluation of the obtained results;
- inserting all results in a data-base in order to handle above procedure more easily and to disseminate results among the interested people.

The problems that concern repair and restoration of serviceability of bridges are very important not only for specific aspects of scientific and technological research, but also for great costs requested to the owners of road-nets.

These bridges often have to be proportioned to the actual lane-width. Frequently no maintenance interventions were made on the latter and, after many years from construction, damages affects materials, structural or secondary elements. Actual loads include often seismic actions, frequently not included in the design codes at the construction phase.

The aim of this paper is to review the structural appraisal and restoration interventions of these structures. Among the numerous aspects of the problem, the paper individualizes the following features:

- Methods and procedures for the assessment of structural reliability; there are three stages:
  i) a preliminary, broad assessment of apparent physical condition, robustness and strength of the structure, including simple calculations, where necessary;
  ii) a complete assessment including, but not exclusively based on, numerical checks on stability of the whole structure as well as of the strength of each member;
  iii) if (a) and (b) are not conclusive, a deeper analysis is required. This should be based on best knowledge of loads and materials' strengths that can be practically obtained, either by measurements and tests, or if appropriate, by other investigations [1].

- Planning of appraisal with regard to the refinement of the structural analysis that is intended to develop and to the costs that anyone is disposed to support;

- Research of informations on construction techniques, mechanical, chemical and physical properties of materials, computational methods used in design of the structure and loads assumed when the structure was planned, especially for dated bridges and particularly for masonry bridges;

- Materials, techniques for structural repair and strengthening with particular regard to the durability and compatibility of new materials and those employed when the structure was made;

- For the most frequent typologies (arch masonry bridges, p.r.c. deck girder bridges) arrangement of procedures in order to realize a data-base of the most frequent damages, techniques and materials to repair and strengthening.
The paper tells of a bridge monitoring system, whose targets are monitoring, aging management, repair and strengthening of a lot of bridge structures (about two thousands), which is being developed on the basis of the results obtained from a review of the studies carried out in many countries on this problem.

2. OBJECTIVE AND STRUCTURE OF SYSTEMATIC BRIDGE MANAGEMENT

The problem of the management of a system of transports is tackled today under manifold points of view, whose common denominator is to assure the reliability of the system or to furnish the user the same performance with the time.

Bridge management is part of transportation management which in turn is part of infrastructure management. Infrastructure management is an important part of the entire economic structure.

A systematic approach to networks management involves many aspects within which reliability of all components is the priority.

Among these problems the paper attempts to tackle current trends in bridge management, whose reliability involves large funding.

Bridge management system (BMS) has became a fundamental issue of infrastructure management in all countries where it is today under various stages of development [1] - [6].

The aspect of the problem is double: one concerns the management of existing bridges, another the design and construction of new one with particular regard to durability design. Focal points of a systematic management of existing bridges are:

- Inventory
- Condition assessment
- Condition rating
- Determination of maintenance and rehabilitation interventions
- Economic evaluation of funding needs
- Economic efficiency considerations
- Planning of interventions
- Execution of interventions
- Management of repaired or rehabilitated bridges

Durability design of new bridges would be focused on these aspects:

- Inventory of most frequent defects encountered in existing bridges
- Determination of rules to be employed in design, execution, supervision, management and maintenance in order to extend service life
- Cost/benefit calculations in order to ensure optimal service life
- Development of codes for above operations to achieve foreseen targets.
The problems are very complex and not all above aspects have been developed today entirely: a systematic evaluation is in progress only for some of them.

3. MANAGEMENT OF EXISTING STRUCTURES

The stock of existing bridges differs notably from country to country. Bridges in Italy, for example, have been built since antiquity period: some in Roman age, others in Middle age, others in Renaissance, others in following age. Most are masonry bridges, some wood bridges, few steel bridges. Reinforced concrete bridges, prestressed concrete bridges and steel bridges were typical on road and railway network in the second half of the twentieth century, during the Post-World War II decades.

As far as bridge management is concerned, this means that most of the stock doesn't exhibit a homogenous structure aging and materials condition.

Masonry bridges have been realized up to the more recent years, with large rise in the years from first half of the XIX to the first decades of the XX. when the growth of r.c. structures yielded the progressive abandonment of masonry structures. Masonry bridges, built before World War II, are therefore present on all the national and provincial roads and they have to be adjusted to new volume of traffic (exercise loads and road dimensions).

Reinforced and prestressed concrete bridges of last decades begin to be subjected to destructive mechanism that are typical of reinforced concrete structures and that are due to maintenance lack.

As like in many countries as well as in Italy in order to ensure the structural stability, traffic safety and proper functioning of bridges in the long term, greater attention has been given to a bridge management system [7-9].

Because of the complex nature of the task involved, it is especially important to analyze other countries' experiences and use them in development activities.

In the USA since the early 1980s the individual states have used bridge management systems to support their maintenance planning. In the early 1990s FHWA has been developing a comprehensive BMS known as "PONTIS", which is characterized by conditions in the USA, e.g. damage model were geared to specific structural problems (steel bridges, suspended bridges) and to identically designed types of structure which, as a good experience had been gained with, were frequently used in the past [1].

Since the early 1990s in Germany, where bridge conditions were different in the West and in the East after reunification, a BMS is being developed whose structure has been subdivided into seven principal subject groups (modules). Bridges stock in Germany consists of a lot of types of structure and their mean age is low, because most of them were reconstructed after World War II. Today the deficiencies and damages relate mainly to traffic safety and durability problems [2].

Information on Danish system is discussed in [3], the Swedish experience is described in [4], organization model for operation of major state owned toll bridges in California (USA) is discussed in [5], the Polish bridge inspection system is described in [6].

Focal points above mentioned will be dealt with later on.
3.1 Inventory

It is not surprising that many owners do not know how many bridges are in their networks. Inventory of bridges is a necessary starting point for the appointment of BMS. The provision of basic data organized in a database is the fundamental feature of this module of the overall system. Data collected can be linked with other databases, in which traffic data, accident data, operating data, etc., are organized. This means to develop a comprehensive road information base and to make possible a link-up with other management systems.

It is necessary to take great care over the interpretation of collected data: a lack in basic informations, as that concern rehabilitation works performed, might cause erroneous results in even if sophisticated computer based analysis and suggests incorrect strategies in the final module of the overall system (cost-benefit analysis).

3.2 Condition assessment

Reliability of existing structures depends on:

- Material condition
- Efficiency of the static scheme
- External loads.

Carrying capacity and serviceability of materials are referred to:

- The carbonation front propagation or carbonation depth in concrete;
- Chloride penetration and content;
- Reinforcement corrosion.
- Loss of initial prestress due to corrosion of tendons and grouting problems.

This phase concerns provision of condition data derived from results of bridge inspections and the evaluation of the identified damage.

It is rather articulated: it is followed by intervention phase immediately if condition ratings determine bridge maintenance, or by consecutive systematic inspections in order to know structure aging and materials condition. In any case the action required may be taken in phases, each phase depending on the findings of the previous one and the appraising involved people may be more and more experienced as the investigations are deeper.

In the preliminary assessment only visual inspections are needed and they may be of a routine nature, supported by appropriate procedures (e.g. after rainfall scuppers need to be inspected and cleaned). If that is the case the appraising people may not be specialized. More exhaustive, periodic visual inspections, carried out at some specific dates, include control of most notable among the rapidly deteriorating bridge components (joints, wearing surfaces, paint, bearings, pedestals, decks and primary members). These measures and material conditions may be inspected by specialized engineers.

The inspections have a cost and the expense has to be in any case justified by an investigation planning, particularly when interventions have to be performed.

Evaluation of defects and condition ratings, to make evaluation independent of the subjectiveness of the operator, may be assessed by normalized procedures. It is important
particularly when assessment doesn't depend on measurements of some selected control variables and in situ-tests.

For such purposes it is useful to predispose atlases or databases that visualize the defects that could occur to the operator, so that the evaluation may be performed on the basis of ratings previously set and with which good experience has been gained.

The preparation of the atlases of defects with their ratings is preliminary to the use of these procedures and to such purpose, for identically designed types of structures, condition ratings may be appointed by international cooperation.

A schedule of bridge inspection and condition rating reports, after inventory, need to be drawn up and a database has to be developed to include all data collected. Thus it is possible to follow progress of degradation for each individual bridge and to analyze in a systematic way the collected data if we are dealing with a group of bridges.

Experience of BMS in various countries in recent years has shown that it is worst to handle all data of an existing network as a whole, because average value is often meaningless.

In case of homogeneity for typologies or for environmental conditions or for both the characteristics, the data collected can be handled by the probabilistic approach.

To this end, monitoring systems and damage development models are to be integrated at a later date. The choice of the fittest procedure of investigation must be the object of an attentive evaluation, in-situ tests must be organized in several phases, each depending on the findings of the previous one. It is often necessary to plan investigations taking into account cost/benefit relationship. In addition to the costs of the in-situ tests it is necessary often take into account other costs as the scaffolding to carry equipments and operators on bridge members e.g. in arch bridges on deep gorge.

In this phase of appraisal, calculations may be necessary and they may be simple, conventional design calculations or a deeper analysis is required.

There are three stages [10]:

1. A preliminary, broad assessment of apparent physical condition, robustness and strength of the structure, including simple calculations. If these checks are satisfactory no further investigation is required. If these checks indicate a dangerous situation, some temporary safety measures may have to be taken, pending further investigations.

2. A complete assessment including, but not exclusively based on, numerical checks on stability and integrity of the whole structure as well as of the strength of each member. Conventional design calculations will usually be used for these checks, although "working stresses", calculated using unfactored service loads, often give a better appreciation of the margin of safety, when compared to failure stresses of materials.

3. If (1) and (2) are not conclusive, a deeper analysis is required. This should be based on the best knowledge of loads and materials' strength that can be practically obtained, either by measurements and tests, or if appropriate, by other investigations. Such more precise knowledge may justify reduction of the (partial) safety factors used in the calculations.

3.3 Overall bridge condition rating

The experiences gained in many countries suggest the rating of bridges on the basis of damage and defects identified. In practice on the basis of inspection reports by means of a procedure independent from subjectivity of operator, overall bridge condition is averaged by a number, which is the product of condition ratings for all components and all spans. The experience and the
data by now available in bibliography and in the procedures already adopted by some countries enable to define an upper threshold under which interventions need and a lower one under which bridge fails and has to be rebuilt.

It is more advisable to integrate ratings based on condition data with a judgement on the bridge adequacy to the volume of traffic that is currently present on the road and the foresseen volume of traffic in the bridge residual lifetime. A bridge included in full maintenance program to extend its service period by measures applied to its most sensitive components may be insufficient with respect to the current volume of traffic; if that is the case decision must be adopted on the basis of a deeper and more comprehensive judgement.

With regard to materials and members condition bridge ratings will depend on the development of deterioration. Appraisal of annual deterioration rate is a very important task as it allows to foresee residual lifetime and to determine the plans for bridge maintenance, repair and reconstruction.

Many refined methods to predict the degradation of materials, e.g. the carbonation front propagation or carbonation depth in concrete and chloride penetration and content, have been performed [11-13]. These methods, undoubtedly for new bridges, will be useful, however their application may be associated with some difficulties if applied to existing one.

In practice, prediction methods are based on long-term experience and are unable to satisfy the immediate need for knowledge, because obtaining results takes long time.

If the service life of a structure is long, the useful information obtained from experimental tests must take into account the exposure conditions which in turn depends on environmental conditions that today could suffer from sudden and not predictable variations. Moreover the results obtained from field tests are doubtlessly valid in condition exposures similar to those of the test site and cannot be applied to other environmental conditions.

Degradation depends also on other factors e.g. condition of bearings, joints, scuppers, water erosion, soil movement etc.

Therefore it appears more advisable to include all these degradation factors by means of a comprehensive annual rate of deterioration which is a fraction of a rating point. In this way bridge annual deterioration rate averages all components annual one.

Deterioration rate of a point could be established on the basis of the results obtained either from network's owner or from other owners' experiences.

In literature it is pointed that undocumented rehabilitation and repair work could influence the choice of the annual deterioration rate of a point and that the behaviour of rehabilitated bridges differs from the new ones.

Typical values of annual deterioration rate are 0.1 and 0.2 of a point at full, no maintenance or rehabilitated bridges respectively; those at full maintenance (0.1) are speculative while those at reduced or no maintenance are based on many years of experience. Moreover rehabilitated bridges can never be restored to a higher condition rating [1].

On the basis of these values for annual deterioration rate of a point in 15 or 30 years bridge reach the threshold under which minimum maintenance is insufficient and the structure needs full maintenance and annual deterioration rate increases.

As above pointed it is necessary take into account the likely obsolescence from the insufficiency towards the volume of traffic.
3.4 Examination of the typologies of interventions to be performed

Maintenance interventions have been performed in all epochs, today principally reinforced concrete and steel bridges are involved in the maintenance programs.

This phase concerns products and technologies to be employed and the most important features are materials durability and interventions costs.

It must be pointed that in some cases the useful information obtained from experience-based tests on the durability of maintenance and rehabilitation materials and measures is restricted to those which are in use for tens of years and in many cases monitoring period is fairly brief. Therefore reliable judgements may not be provided from today available results.

Results obtained by laboratory tests simulate seldom real materials aging or environmental deterioration. The wisest thing is to perform in-situ tests using cast test specimens fixed to the real structure, exposed to the same environment and execution as the structure or structure itself; in the latter, a non destructive testing is usual and suitable to compliance with laboratory tests [12]. This procedure suggested for design of new structures appears suitable for rehabilitation or maintenance interventions too.

Therefore it is better to use products and technologies whose behaviour with the time is well known and documented, taking into account that the intervals between two successive interventions must be at least of 5-10 years e.g. for paint of steel or reinforced concrete members while a longer durability for other bridge components must be ensured.

In this phase of BMS it is essential to fix intended service period of bridge on which maintenance and or rehabilitation and the types of interventions depend consequently. As above mentioned above, not only structural reliability but also serviceability must be taken into account. In the latter and to this end BMS may be interfaced with other management systems and finally with a comprehensive systematic road management.

Sometimes problems arising from the use of materials in, the fast to their harmfulness only recently we are turning our attention, make more complex and expensive the choice of the fittest technologies and materials to be employed in repair or rehabilitation interventions [14].

All the procedures of this module of BMS could be computerized; the organization of databases in which various intervention typologies and their time depending behaviour are collected is an important tool for decision making.

The database has to be predisposed so that all results of inspections after every construction measures (repair, conversion, new construction) can be entered directly and new technologies and products can be enclosed.

3.5 Calculation of funding requirements

The above phases enable the calculation of funding requirements. The monetary evaluation for the maintenance of the total stock of structures could be drawn initially per square meter of bridge deck taking into account main features of bridge (predominant materials, type of design) and maintenance strategy (full maintenance, reduced maintenance, demand maintenance) [1].

These initial cost forecasts could be subsequently detailed by means of deeper investigations with experts being consulted and measures being selected in each individual case, using the appointed catalogues.

The results of this module combined with those of an overall road management system allow higher administrative levels, central government, to plan funding requirements, taking into account political, economic and technical conditions.
3.6 Cost/benefit analysis
This phase is crucial as it separates appraisal from intervention phase and to this end other conditions play an important part e.g. social, economic, political, strategic conditions, as above mentioned above. These conditions cannot be quantified and tackled through computerized analysis and procedures; the final decisions therefore could be taken on the basis of considerations that are beyond those here developed.
Nevertheless any decisional choice could not put aside the knowledge of all aspects of the problem and, therefore, purpose of any BMS is to define this form also.

3.7 Planning of the management measures
Planning of the interventions could be organized on the basis of the results obtained in the above phases, after cost-benefit calculations have been assessed. For this purpose if the local administrations develop one’s database, it is necessary that collected data are interfaced and organized with those obtained from other local administrations, so that central government may decide which network are to be maintained, rehabilitated or reconstructed and may assign funding to the local administrations which in turn are responsible of implementation of interventions. To this end procedures adopted by the USA and Germany and other nations could be an useful reference for the organization of BMS.

3.8 Implementation of interventions
Once decision of full or reduced maintenance or rehabilitation has been reached, the measures must be implemented. This phase must be organized so that the basic data of any intervention are entered into the database and constitute useful references for further interventions and for the management of bridge itself.

The most recent results on the development of materials deterioration must be applied as this phase is similar to construction stage and, therefore, all the results on durability design and on prediction and extending lifetime of a structure could be applied.

As above mentioned products employed for repair and rehabilitation interventions should be monitored and deterioration rate with the time should be carefully controlled and measured by programmed inspections on the structure by means of non-destructive tests or by means of in-situ performed one using cast specimens fixed to the structure, exposed to the same environment and execution as the structure.

The most recent theoretical results on lifetime of concrete structures that take into account realistic materials modeling, structural damage evaluation and nonlinear structural analysis with deterministic reliability analysis could be employed and validated by means of in-situ tests and measurements [13]. It must be pointed that, as above mentioned, deterioration rate of rehabilitated bridges is greater than new one.

Most recent design codes impose provision of a manual of use and of maintenance in the design phase of new structures. Manuals’ preparation could be performed on the basis of results acquired during development of bridge management system.

3.9 Management of repaired or rehabilitated bridges
Management of rehabilitated structures must be developed in a different way comparing to the past: basic data after any intervention must be entered in a computerized database, structures must be monitored by means of a program of systematic inspections in order to assure new intended
service period. So it is possible to collect in-situ data on the technologies and products employed and it is useful to enter all data in a computerized database to which each owner could approach for his management system. Moreover if all these databases are interfaced, central government, which has the responsibility for the road network and provides the required funds, can decide.

A group of authorities (municipal, provincial, regional etc.) manages road and rail networks each of one, often, acts autonomously. Data acquired are not interfaced and so they are not comparable: basic data as catalogue of structures doesn't exist often. If that is the case it is advisable that the central government develops BMS and imposes it to local administrations.

4. DESIGN OF NEW STRUCTURES

The essential features of durability design and construction of new structures doesn't substantially differ from the above discussed, but procedures must be organized so that peculiar problems of new structures are taken into account.

It is often impossible to deal with some questions in the same way and by means of the results collected in database appointed for the management of existing bridges. The latter, for example, could be examined by deterministic or probabilistic manner but the so called "golden rules of building operations" cannot be deduced from database.

In any countries, e.g. Italy, the reconstruction after World War II was performed privileging the isostatic schemes for the bridge in consequences of difficulties arising often from geotechnics and reinforced concrete structures were realized without any superficial protection. Degradation and deterioration after about 30 years result in a change of the above mentioned choice: new bridge privilege the static scheme of a continuous beam. So the number of structural joints is drastically reduced and it is possible to rely on the capacity of the structure to redistribute exceptional loads as seismic actions within its all components and the greater complexity in the construction phase is counterbalanced by the less need for maintenance.

For these reasons warnings have been appointed for construction of new bridges and collected in a publication with the aim of planning maintenance of roads [9].

5. MODEL OF BMS IN DEFINITION

As mentioned in the abstract, a study is in progress to define a BMS for a public provincial administration whose road network is about 2200 Km and on which a large stock of bridges exist. Many of these are masonry bridges, other are reinforced or prestressed reinforced concrete bridges and only few steel bridges. Today inventory and a computerized system to acquire bridge condition have been performed. The system appointed can acquire all data available from visual inspections and from information on bridges' construction and maintenance or repair interventions performed. To this end the staff of the Transportation Department of the provincial administration has been trained on aging of r.c. structures and particularly on bridges' deterioration. The next step of the program is to train the staff for application of the appointed system.

New modules of the BMS are in definition: module for condition rating and for determination of maintenance and rehabilitation interventions. A collection of the most frequent and utilized products and technologies for maintenance, repair and rehabilitation interventions is in progress:
the data acquired will be computerized by means of a program similar to the one just appointed for data on bridges conditions.

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THERMAL STRESSES IN MASS CONCRETE

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ABSTRACT

Due to hydration heat generation, the newly placed massive concrete structures are plausible to be exposed to severe thermal gradients as they solidify. These detrimental differential volume changes are the main cause for development of thermal cracks. The coupled thermo-elastic stress investigations shall impose proper proportioning among the factors affecting thermal gradients in order to avoid associated cracks.

Keywords: thermal analysis, thermal stresses, mass concrete, hydration heat

1. INTRODUCTION

During hardening of mass concrete, considerable hydration heat is generated. The heating of concrete leads to temperature rise and volume changes. As there are always internal and external restraints to constrain volume changes, the generation of thermal stresses is inevitable. The external restraints are due to the rock foundation or hardened concrete layer below newly placed one. The internal restraints occur when different areas of concrete have different temperature changes and accompanied volume changes. This occurs when the inside and outside of concrete block are exposed to different temperature rises during hardening. For example, inside of concrete is exposed to hydration heat temperature rise and the outside of it is exposed to ambient air temperature variations.

The induced thermal cracks could be controlled or reduced by several measures. The first remedy is to release thermal strains by intentionally placed artificial hyper cracks, well known as contraction or expansion joints. The second one is to resort to methods to reduce the temperature rise. Among the main factors affecting considerably the amount of temperature rise are the cement content, fresh concrete temperature, ambient air temperature, lift height, block width and time interval between lifts. The conventional pre-cooling and post-cooling methods could be classified in this category.

2. PHYSICAL PROBLEM

The present study contains the thermal behavior of a concrete gravity structure that has considerable geometrical dimensions and hence is probable to be exposed to severe thermal

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4.1 Thermal Properties

Isotropic homogeneous thermal properties are assumed for both of the concrete and rock mass. Considering temperatures encountered in practice, it is justifiable to assume independence of thermal conductivity on temperature variation.

<table>
<thead>
<tr>
<th>Thermal Quantity</th>
<th>Mass Concrete</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal Conductivity</td>
<td>( k = 220.0 )</td>
<td>( k = 1200.0 ) [kJ/day·m(^{-1})]</td>
</tr>
<tr>
<td>Coefficient of Heat Transfer</td>
<td>( h = 1200.0 )</td>
<td>( h = 1200.0 ) [kJ/day·m(^{-2})]</td>
</tr>
<tr>
<td>Specific Heat</td>
<td>( \rho_c = 2510.0 )</td>
<td>( \rho_c = 1850.0 ) [kJ/m(^3)]</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion</td>
<td>( \alpha = 1.0\text{E-}5 )</td>
<td>( \alpha = 1.0\text{E-}5 ) [1/°C]</td>
</tr>
</tbody>
</table>

Ambient Air Temperatures

- Extreme Hot Weather \( T_{\text{max}} = 42.0 \) [°C]
- Extreme Cold Weather \( T_{\text{min}} = 10.0 \) [°C]

4.2 Equation of Hydration Heat

The following theoretical formula, which obeys an exponential variation [11,16], is used to obtain the magnitude of the generated heat of hydration \( W(t) \) as a function of time:

\[
W(t) = W_{\text{max}} \left[ 1 - e^{-t/t_0} \right]
\]  

where
- \( W(t) \) - Heat of Hydration at Time \( t \)
- \( W_{\text{max}} \) - The Maximum Value of Heat of Hydration
- \( t_0 \) - The Characteristic Time

The magnitudes of \( W_{\text{max}} \) and \( t_0 \) which are unknown parameters of the Eq. (7), could be determined by application of appropriate numerical techniques of curve fitting, such as the Least Squares Method, subject to knowing the values of \( W(t) \) at specific times. Taking into account the available results of hydration heat tests carried out in adiabatic calorimeter apparatus on four specimens of local anti-sulfate type-V cement, the following averaged values are extracted for \( W_3, W_7, \) and \( W_{28} \), respectively:

\[
\begin{align*}
W_3 &= 222.934 \\
W_7 &= 240.318 \\
W_{28} &= 326.459 \text{ [kJ/kg]}
\end{align*}
\]

The set of three non-linear equations obtained in this manner are solved simultaneously by numerical methods leading to the following results:

\[
\begin{align*}
W_{\text{max}} &= 326.694 \text{ [kJ/kg]} \\
t_0 &= 3.869 \text{ [Days]}
\end{align*}
\]

The above equation provides estimation for the magnitude of hydration heat corresponding to per unit mass of the cement in the mixture. Therefore, the amount of hydration heat per unit volume of the concrete could be found directly by multiplication of \( W(t) \) by cement content.
Concerning the conduction heat transfer equation, not only the magnitude of generated hydration heat but also and of more interest is the time rate of hydration heat generated within the concrete. This rate is obtained by differentiation of Eq. (7) with respect to the time variable. Depending on the cement content of the mixture, using consistent system of units, the magnitude of \( q_{B \text{ max}} \) is determined. For example, for cement content of \( Z=300 \text{ kg/m}^3 \), \( q_{B \text{ max}} \) is found to be equal to 293.191 W/m\(^3\). The heat rate is:

\[
q_B(t) = q_{B \text{ max}} e^{-t/t_0}
\]  

(8)

where \( q_B(t) \) - Rate of Hydration Heat at Time (t)  
\( q_{B \text{ max}} \) - The Maximum Rate of Hydration Heat  
\( t_0 \) - The Characteristic Time

Figure 2 Concrete hydration curves

4.3 Finite Element Formulation

Using appropriate weighted residual methods or variational principles and adequate interpolation functions, we obtain the following finite element equilibrium equation in nonlinear transient heat transfer analysis [1,3]:

\[
(1+\Delta t)C(I)_{t+\Delta t} + (1+\Delta t)K^{(i-1)} + (1+\Delta t)k^{(i-1)}\Delta T^{(i)} = (1+\Delta t)Q_{t+\Delta t} + (1+\Delta t)Q^{(i-1)} - (1+\Delta t)Q^{(i-1)}
\]

(9)

with nodal temperatures at end of iteration (i):

\[
T^{(i)} = T^{(i-1)} + \Delta T^{(i)}
\]

(10)
where \([K^k]\) is the conductivity matrix,

\[
^{t+\Delta t}K^k(i-1) = \sum_m \int_{V^{(m)}} B^{(m)^T} k^{(m)(i-1)} B^{(m)} dV^{(m)}
\]

(11)

and \([K^c]\) is the convection matrix,

\[
^{t+\Delta t}K^c(i-1) = \sum_m \int_{S_{S_{(m)}}} H^{(m)(i-1)} H^{S(m)^T} H^{S(m)} dS^{(m)}
\]

(12)

and \([C]\) is the heat capacity matrix,

\[
^{t+\Delta t}C^{(i)} = \sum_m \int_{V^{(m)}} H^{(m)^T} (\rho c)^{m(l)} H^{(m)} dV^{(m)}
\]

(13)

The nodal heat flow input vector \(\{Q\}\) is given by:

\[
^{t+\Delta t}Q = ^{t+\Delta t}Q^B + ^{t+\Delta t}Q^S
\]

(14)

where

\[
^{t+\Delta t}Q^B = \sum_m \int_{V^{(m)}} H^{(m)^T} q^{b(m)} dV^{(m)}
\]

(15)

\[
^{t+\Delta t}Q^S = \sum_m \int_{S^{(m)}} H^{S(m)^T} q^{s(m)} dS^{(m)}
\]

(16)

\[
^{t+\Delta t}Q^{c(i-1)} = \sum_m \int_{S_{S_{(m)}}} H^{(m)(i-1)} H^{S(m)^T} \left[ H^{S(m)} \left( ^{t+\Delta t}T - ^{t+\Delta t}T_{(i-1)} \right) \right] dS^{(m)}
\]

(17)

\[
^{t+\Delta t}Q^{k(i-1)} = \sum_m \int_{V^{(m)}} B^{(m)^T} \left[ ^{t+\Delta t}k^{(m)(i-1)} B^{(m)} + ^{t+\Delta t}T^{(i-1)} \right] dV^{(m)}
\]

(18)

The matrices \([H]^{(m)}\) and \([B]^{(m)}\) are the element temperature and temperature gradient interpolation matrices, respectively, and the matrix \([H^{S}]^{(m)}\) is the surface temperature interpolation matrix. Namely, for element \((m)\),

\[
T^{(m)} = H^{(m)} ^{t+\Delta t}T
\]

(19)

\[
T^{S(m)} = H^{S(m)} ^{t+\Delta t}T
\]

(20)
Following establishment of the FE model for the real physical problem, the Eq. (9) above may be solved by application of direct numerical integration algorithms to find out the temperature values at any specific time. This is accomplished automatically using the ADINAT (Automatic Dynamic Incremental Nonlinear Analysis of Temperatures) which is a thermal analyzer software [7]. Although, Euler backward implicit time integration is unconditionally stable, but, in order to obtain satisfactory results from numerical analysis, finite time increment for the time variable is taken equal to $\Delta t=0.03$ and $\Delta t=0.30$ days for middle and long terms analysis, respectively.

4.4 Initial Conditions
The temperature of fresh concrete is taken as the initial condition and for all lifts is assumed constant and equal to 18 °C. The initial temperature of rock mass is taken equal to annual average air temperature, i.e., $T_{\text{avg}}=26$ °C. Also, the initial temperature of rock-concrete interface and exposed rock faces is assumed to be equal to ambient air temperature at the time of concrete placement, i.e., 39.7 °C.

4.5 Thermal Boundary Conditions
For the concrete faces, which are exposed to air, time varying boundary conditions are imposed. Simulating the real situation of upward movement of the slipforms, subsequent to placement of concrete and prior to moving of the forms, the boundary condition on the air faces are taken as insulated and following the shifting up and stripping of the fromworks, i.e., 7.0 days time interval between lifts, the boundary condition shall be altered to a free convection. Also, the top horizontal boundary of each lift is assumed to be exposed to convection boundary condition prior to the placement of the next lift. As is evident, on these types of boundary faces convection elements with birth and death times should be applied. Therefore, 2D line

![Concrete lift placement schedule](image-url)
convection elements with birth time equal to striping time of formworks are used to model this type of boundary conditions. Slip forms are assumed as a wood-steel type with steel frame combined with 3 cm thick plywood panels. The schematic view of boundary conditions and concrete lift placement schedule is sketched in the following.

4.6 FE Model and Element Types
An arbitrary vertical 2D section normal to the longitudinal axis of the guide walls, with dimensions as sketched on Figure 3, together with considerable amount of rock mass media, is picked out and used for FE discretisation. This model of structure has dimensions of 44.5 m in height and 37.15 m in base width (perpendicular to flow direction). In-plane two-dimensional conduction elements are utilized in heat flow analysis. The elements are eight noded isoparametric finite elements. Further improvements are made on this automatically generated FE model by performing bandwidth minimization process [9]. Thermal interaction between placed lifts is considered by progressive increment of number of lifts, which resemble the real consecutive placement of concrete lifts above each other. The element birth option of the ADINAT program is adequately utilized to resemble addition of FE conduction elements, i.e., concrete lifts to the whole model.

![Finite element model](image)

Figure 4 Finite element model

The birth time of the elements of specific lift corresponds the placement time of that lift. It is worth to mention that, the whole concrete of each lift is assumed be placed simultaneously. For the concrete and rock faces exposed to convection boundary conditions, three noded line convection elements with appropriate birth and death times are employed. The time interval between successive lifts is taken constant and for all lifts equal to 7.0 days.

4.7 Analysis Type
Considering addition and deduction of conduction and convection elements to the whole FE model during time integration, therefore, the total assembled conduction (stiffness) matrix should be reformed at each time step. Also, to assure an accurate solution, equilibrium iterations
are performed at each time step. In fact, a non-linear analysis is carried out and the system response is evaluated using an incremental solution of the equations of equilibrium.

5. THERMO-ELASTIC STRESS ANALYSIS

The stresses corresponding to temperature rise within body of concrete are calculated through a two-dimensional plane strain analysis. Only incremental strains due to temperature variations are included in the present study, i.e., the concrete dead weight and creep effects are ignored. The same finite element model is also applied for thermo-elastic stress analysis. The time history of thermal stresses for selected elements are calculated and also presented at the location of 2×2 Gaussian quadrature points. The locations of elements are crosshatched on Figure 3.

As is recommended in [7], in order to avoid any inaccuracy due to interpolation of nodal point temperatures to Gauss points, a very fine FE model is utilized in both thermal and stress analyses. Considering time-dependent non-linear variation of concrete elastic modulus and also element birth time option that identifies the time at which elements become active, therefore, again a non-linear analysis is performed by ADINA (Automatic Dynamic Incremental Nonlinear Analysis) program with stiffness reformation and equilibrium iteration on each time step.

5.1 Elastic properties

According to the guidelines of ACI 209R-92, time-dependent elastic and isotropic material properties are assumed for the concrete. The rock mass properties are assumed to be linear elastic, time independent. The elastic properties are taken to be homogeneous.

**Mass Concrete**
- Compressive Strength at 28 Days $f_c(28) = 21.0$ [MPa]

  - Static Cylinder Compressive Strength $f_c(t) = \frac{t}{a + \beta t} f_c'(28)$

  - Modulus of Elasticity for Normal Weight Concrete $E_c(t) = 4730\sqrt{f_c(t)}$

  - Apparent Static Tensile Strength $f'_c(t) = 0.44(f_c'(t))^{2/3}$

  - Poisson's Ratio $\nu = 0.20$

**Rock Mass**
- Static Modulus of Elasticity $E_r = 10,000$ [MPa]
- Poisson's Ratio $\nu = 0.2$

The constants involved in time-dependent compressive strength of concrete, i.e., 'α' and 'β' are functions of both cement type and curing method employed [15]. For normal weight and moist cured concrete, the values of α=4.0 and β=0.85 seems to be satisfactory.

In order to find out the apparent tensile strength of concrete, the above-mentioned formula after Raphael [10] is applied. As this tensile strength is an apparent value, it could be applied directly to the results of an elastic finite element analysis. It is worth to be emphasized that, this strength envelope is strictly for an elastic analysis of the structure for which it is desired to
accommodate tensile stresses without causing tensile fracture. This approach does not indicate the safety of the structure once fracture has initiated.

Figure 5 Concrete time dependent properties

6. FE RESULTS SUMMARY

6.1 Heat Transfer
Figure 6 shows the middle-term temperature time histories for a few specific nodal points that are positioned on the interface of two lifts. On the graph, the time \( t=0 \) coincides with the 1st of June. For nodal points positioned on the mid-width of blocks, as the block thickness decreases the temperature drop rate is increased. Vicinity to rock foundation also intensifies this drop rate. At these points, a sharp temperature drop is detected at the placing time of the forthcoming lift due to its lower fresh concrete temperature. For surface nodal points that are positioned just behind the slipform, following two weeks from placement, the temperature variations shall be coincident with the ambient air temperature.

Figure 6 Middle-term temperature histories
On Figure 7, the corresponding long-term temperature time histories are shown. For points positioned on the middle of blocks, as the block thickness decreases the amplitude of temperature variation is increased. The fluctuation of temperatures is about annual average air temperature, i.e., $T_{\text{avg}}$ = 26 °C. The temperature curves follow a sinusoidal variation and has annual period. A snapshot of heat flux vectors is shown on Figure 8.

![Figure 7 Long-term temperature histories](image1)

![Figure 8 Heat flux vectors](image2)

6.2 Thermo-Elastic Stress Results
Figures 9 to 13 show the middle term horizontal ($\sigma_1$) and vertical ($\sigma_2$) stress time histories for a few specific elements that are mainly positioned on the mid-width of blocks and on interface of two lifts. For elements that are located on the lift top, ($\sigma_1$) stress variation from compression to tension is detected. Prior to placement of the next lift, the stresses are compressive and thereafter they tend to tension. The magnitude of tensile stresses is increased as the block width is decreased. For elements located on the bottom of lifts, the scenario is quite different. They undergo compressive horizontal stresses during whole solution time. If only uni-axial tensile
criterion is taken into consideration, generally the stresses are within tolerable limit of apparent tensile strength. The vertical ($\sigma_2$) stresses are mainly in tension. For elements on the lift top, they are of negligible value prior to placement of the next lift. However, the magnitudes of ($\sigma_2$) stresses would decrease due to existing gravity stresses.

![Graph](image)

**Figure 9** Stresses versus tensile strength

![Graph](image)

**Figure 10**

Since below and above the lift interfaces, concrete with different ages are placed, therefore, considerable difference in elastic modulus and hence stresses would be expected around this zone, i.e., the gradient of stress variations is very high. Therefore, fine FE mesh should be used in these zones to achieve reasonable results. For elements located at the half height of lifts, the stresses ($\sigma_1$) and ($\sigma_2$), are mainly fair and more justifiable.
Figure 11

Figure 12

Figure 13
7. CONCLUSIONS

This technical article summarized our experience acquired during the past few years on the subject of thermal stresses in mass concrete structures. There are many parameters that affect results outcome and this augments problem complexity. Inclusion of creep effects, model refinement, gravity stresses and also utilization of biaxial failure envelope [17], are among the primary recommendations for correct assessment of safety factors in future studies.

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REFERENCES

A STUDY ON PULL-OUT STRENGTH BETWEEN
LIGHTWEIGHT AGGREGATE CONCRETE AND REINFORCING
BARS

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Ferdowsi University, Mashad, Iran

ABSTRACT

This paper discusses the results of an experimental investigation into the bond strength between reinforcing bars and lightweight aggregate concrete (LWC). The parameters studied are unit weight of concrete and thickness of the concrete cover over the reinforcing bar. Normal-weight control concrete had a compressive strength of 24 MPa, and lightweight aggregate concrete had a compressive strength of 20 MPa. The study included tests of twenty pullout specimens in two series. The test specimens consist of concrete blocks with a single 80-mm reinforcing bar embedded horizontally. Each series comprised two groups with different concrete covers over the reinforcing bars. Each group included five similar specimens. Test results show that the pullout strength of lightweight concrete is generally higher than that of normal concrete. It is also concluded that the reduction factor recommended by the provisions of different codes for the bond strength of lightweight concrete is unnecessary conservative.

Keywords: bond strength, lightweight aggregate, lightweight concrete, reinforcing bars.

1. INTRODUCTION

Adequate bonding between reinforcing bars and concrete is a fundamental requirement for the satisfactory performance of reinforced concrete structures. It affects many aspects of the behavior of reinforced concrete such as cracking, deformation, internal damping, and instability. The findings of previous research into the bond strength of lightweight aggregate concrete are contradictory. Some studies showed that the bond strength of LWC was larger than that of normal concrete [1,2]. On the other hand, there are some experimental results showing that the bond strength of LWC is comparatively smaller [3]. Martin [4] and Berge [5] using standard RILEM pull-out test specimens found that the bond strength from the pull-out test was approximately the same for both types of concrete using lightweight and normal weight aggregates. Similar contradiction is seen in the studies on the influence of silica fume on bond strength [6,7]. In a discussion [8], the author has shown that the contradiction between the results of different studies on bond strength is caused by the following problems: (i) The concrete compressive strength of different specimens has generally been different in each study. That is why researchers have used the normalized bond strengths with respect to the square root of concrete strength and compared them in different tests. However, as previously shown [8], the normalized bond strength may not be independent of the concrete compressive strength. (ii)
In full-scale beam tests, the bond stress distribution influences the bond strength [9]. Many researchers have not considered this effect. (iii) The scatter in the bond strength test results is naturally significant. Therefore, the number of specimens tested by past researchers is not adequate to reach an exact conclusion.

To study the influence of different parameters on bond strength it is appropriate to keep the concrete strength constant in different test specimens. Otherwise, an equation that could appropriately account for the concrete strength is needed. Esfahani and Rangoon [9, 10] studied the bond strength in the cases of normal strength and high strength concrete in two types of specimens, pull-out and splices in beams. Using the test results and the bond strength theories, they proposed equations to calculate the bond strength in the cases of NSC and HSC. They showed that the normalized bond strength with respect to the square root of concrete compressive strength \( u'(f'_c)^{0.5} \) was not independent of the concrete strength in different pull-out and splice tests. In short length pull-out specimens in which the bond stress distribution over the embedded length is almost uniform, \( u'(f'_c)^{0.5} \) versus \( C/d_b \) relationship was obtained for NSC and HSC. It was seen that, in short lengths, the normalized bond strength increased with increasing the concrete strength. Equations 1 and 2, proposed by Esfahani and Rangoon [9], determines the bond strength of short embedded lengths in the cases of normal strength and high strength concrete.

\[
   u_c = 2.7 \frac{C/d_b + 0.5}{C/d_b + 3.6} \sqrt{f'_c} 
\]

\[
   u_c = 4.7 \frac{C/d_b + 0.5}{C/d_b + 5.5} \sqrt{f'_c} 
\]

\( u_c \) is local bond strength in MPa, \( d_b \) is the bar diameter, \( C \) is minimum concrete cover. \( f'_c \) is the concrete compressive strength in MPa. Figure 1 shows the normalized bond strength versus \( C/d_b \) relationship based on Equations 1 and 2.

![Figure 1 Normalized bond strength versus C/db relationship](image-url)
2. EXPERIMENTAL PROGRAM

Twenty specimens in two series were manufactured and tested. Each series comprised two groups with different \( C/d_p \). Each group included five similar specimens. These specimens were cast in one set of formwork at the same time. The concrete strength of the two groups of each series was the same. Each specimen was a concrete block having a reinforcing bar of 20 mm nominal diameter with a short embedded length (Fig. 1). Specimens of the first series were made of normal strength. Second series included specimens made of lightweight aggregate concrete. The concrete mixture of Test Series 2 was carefully designed in order to attain a compressive strength close to that of Series 1. The admixtures of silica fume and superplasticizer were added to the concrete mixture in order to increase the compressive strength of lightweight concrete.

The values of \( C/d_p \) for the first and second groups of each series were 2 and 4, respectively. In all specimens, the ratio of the side cover to the bottom cover \( C_x/C_y \) was 1, and the embedded length of bars was 80 mm (Fig. 1). The dimensions of specimens in the first group were 80 × 100 × 240 mm and in the second group were 80 × 180 × 240 mm. Other details of test specimens are given in Fig. 1. All reinforcing bars were bottom cast bars.

2.1 Materials

Tensile test was carried out on 20-mm diameter reinforcing bar. The measured yield and ultimate strengths of the bar were 370 MPa and 610 MPa, respectively. The reinforcing bars were longitudinally sliced in order to measure the rib geometric. The relative rib area, \( R \), for the bar was approximately 0.12.

\[
R = \frac{\text{projected rib area}}{\text{nominal bar perimeter}} \times \frac{\text{normal to the bar axis}}{(\text{center to center rib spacing})}
\]

![Concrete block](image)

**Figure 2** Details of specimens

Cement type 2 was used for the concrete mixtures. The maximum size of LWA and normal aggregate was 9.5-mm. Expanded shale LWA was used for lightweight concrete. The physical properties of LWA and fine aggregate are given in Tables 1 and 2, respectively.
Table 1 Physical properties of LWA

<table>
<thead>
<tr>
<th>LWA (expanded shale)</th>
<th>standard</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal maximum size</td>
<td>ASTM C330</td>
<td>9.5 mm</td>
</tr>
<tr>
<td>Absorption, 24 hr submerged</td>
<td>ASTM C127</td>
<td>14.2%</td>
</tr>
<tr>
<td>Bulk specific gravity 24 hr submerged</td>
<td>ASTM C127</td>
<td>0.80</td>
</tr>
<tr>
<td>Bulk specific gravity (SSD)</td>
<td>ASTM C127</td>
<td>0.84</td>
</tr>
<tr>
<td>Unit weight (dry rotted)</td>
<td>ASTM C29</td>
<td>527.4 Kg/m³</td>
</tr>
<tr>
<td>Unit weight (dry loose)</td>
<td>ASTM C29</td>
<td>437.9 Kg/m³</td>
</tr>
</tbody>
</table>

Table 2 Physical properties of fine aggregate

<table>
<thead>
<tr>
<th>Fine aggregate</th>
<th>standard</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>grading</td>
<td>ASTM C33</td>
<td>OK</td>
</tr>
<tr>
<td>Absorption, 24 hr submerged</td>
<td>ASTM C128</td>
<td>3.2%</td>
</tr>
<tr>
<td>Bulk specific gravity, 24 hr submerged</td>
<td>ASTM C128</td>
<td>1.99</td>
</tr>
<tr>
<td>Bulk specific gravity (SSD)</td>
<td>ASTM C128</td>
<td>2.07</td>
</tr>
</tbody>
</table>

Both concrete series were made in the laboratory. Concretes were vibrated thoroughly during casting. After two days of casting, the moulds were opened and the specimens were cured wet until a day before testing. Table 3 shows some details of mixtures. For each casting, ten 100 mm x 200 mm concrete cylinders were made to determine the concrete compressive strength.

Table 3 Details of concrete mixtures

<table>
<thead>
<tr>
<th>series</th>
<th>$f'_c$ MPa</th>
<th>unit weight Kg/m³</th>
<th>W/C</th>
<th>cement Kg/m³</th>
<th>SF Kg/m³</th>
<th>SP Lit.</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC-24</td>
<td>24</td>
<td>2350</td>
<td>0.50</td>
<td>500</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>LWA-20</td>
<td>20</td>
<td>1460</td>
<td>0.32</td>
<td>500</td>
<td>80</td>
<td>9</td>
</tr>
</tbody>
</table>

* liquid (based on 40% solution)
† series label: a-b: “a” stands for the concrete mixture, NC for normal concrete, LWA for lightweight aggregate concrete. “b” is the concrete compressive strength.

2.2 Test Set-up and Test Procedure
A tensile test apparatus was used for applying the load. A steel plate located between the specimen and test apparatus was used as a bearing plate (Fig. 2). The dimensions of the plate
were $100 \times 100 \times 20$ mm. The interfaces between the plate and the specimen were greased slightly to reduce the frictional forces between them. The specimens were loaded at the rate of about 7 kN/minute. All specimens failed due to splitting of concrete.

2.3. Test Results
The test results are summarized in Table 4. Bond strength, $u$, was calculated by the relation

$$u = \frac{P}{(\pi d_b L)}$$

where $P$ is the measured load at failure, $d_b$ is the bar diameter, and $L$ is the embedded length of bar. To compare the bond strength of specimens in the two series, all bond strengths were normalized with respect to the square root of concrete compressive strength. Figure 3 shows the relationship between $u/(f_c')^{0.5}$ and $C/d_b$ in different tests.

![Test set up](image)

**Figure 3 Test set up**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_c'$</th>
<th>$f_c'(0.5)$</th>
<th>$L$</th>
<th>$d_b$</th>
<th>$C/d_b$</th>
<th>$P$</th>
<th>$u_{test}$</th>
<th>$u/f_c'(0.5)$</th>
<th>meanSD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-NC24/2</td>
<td>24.0</td>
<td>4.9</td>
<td>80</td>
<td>20</td>
<td>2</td>
<td>33.8</td>
<td>6.73</td>
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<td></td>
</tr>
<tr>
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<td>80</td>
<td>20</td>
<td>2</td>
<td>29.8</td>
<td>5.93</td>
<td>1.21</td>
<td></td>
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<td>3-NC24/2</td>
<td>24.0</td>
<td>4.9</td>
<td>80</td>
<td>20</td>
<td>2</td>
<td>26.0</td>
<td>5.18</td>
<td>1.06</td>
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<tr>
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<td>20</td>
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<td>4.40</td>
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<td>20</td>
<td>4</td>
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<td>6.39</td>
<td>1.30</td>
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<td>4</td>
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<td>4</td>
<td>31.4</td>
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<td>80</td>
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</tr>
<tr>
<td>5-LWA20/4</td>
<td>20.0</td>
<td>4.5</td>
<td>80</td>
<td>20</td>
<td>4</td>
<td>33.5</td>
<td>6.67</td>
<td>1.49</td>
<td>0.18</td>
</tr>
</tbody>
</table>
To be able to compare the bond strength of lightweight concrete with that of normal concrete, the average normalized bond strength of 5 specimens in each group was calculated (Table 5) and plotted against C/d_b as shown in Fig. 4.

3. CALCULATION OF BOND STRENGTH

Equation 1, previously proposed for normal strength concrete, is used to calculate the local bond strength u_calc for normal and lightweight concertos. The results are given in Table 5. Equation 1 and the test results are also shown in Figure 4. It is seen that Equation 1 correlates well with the test results.
Table 5 Average normalized bond strength for different test groups

<table>
<thead>
<tr>
<th>Series</th>
<th>f'_c (MPa)</th>
<th>Groups*</th>
<th>C/d_b</th>
<th>P (kN)</th>
<th>u_{test} (MPa)</th>
<th>u/(f'_c)^{0.5}</th>
<th>u_c (Eq. 1)</th>
<th>u_c/u_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC-24</td>
<td>24</td>
<td>NC-24/2</td>
<td>2</td>
<td>27.1</td>
<td>5.40</td>
<td>1.10</td>
<td>5.89</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NC-24/4</td>
<td>4</td>
<td>36.4</td>
<td>7.24</td>
<td>1.48</td>
<td>7.82</td>
<td>0.93</td>
</tr>
<tr>
<td>LWA-20</td>
<td>20</td>
<td>LWA-20/2</td>
<td>2</td>
<td>27.0</td>
<td>5.72</td>
<td>1.28</td>
<td>5.38</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LWA-20/4</td>
<td>4</td>
<td>31.6</td>
<td>6.29</td>
<td>1.41</td>
<td>7.14</td>
<td>0.88</td>
</tr>
</tbody>
</table>

* Group labels A/B: A stands for the test series, B is the ratio of C/d_b in each group.

4. CONCLUSIONS

An experimental investigation to study the steel-concrete bond strength of lightweight concrete was conducted. Twenty short length specimens were manufactured and tested. Bond strength normalized with respect to the square root of concrete compressive strength was calculated in each test. Using the comparison of the test results, the following conclusions are drawn.

1. Local bond strength of lightweight aggregate concrete is generally larger than that of normal concrete, especially for smaller concrete covers. It seems that the reduction factor recommended by the provisions of different codes for the bond strength of lightweight concrete is unnecessary conservative.
2. The equation previously proposed for local bond strength of normal concrete could be used for calculation of the steel-concrete bond strength in the case of lightweight concrete.

Acknowledgements: The experimental study was conducted at the concrete laboratory of Freedoms University, Mashed, Iran.

REFERENCES


APPLICATION OF THE MODIFIED ART2 ARTIFICIAL NEURAL NETWORK IN CLASSIFICATION OF STRUCTURAL MEMBERS

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ABSTRACT

In this research, the basic algorithm of ART2 neural network has been modified for proper and efficient classification of vectors. In the basic architecture of ART2, the length of vectors is neglected. This causes error in sorting; parallel vectors with different length are classified in the same category. To overcome this deficiency, a virtual input neuron is added to consider vector length. The modified architecture not only considers the similarity of vectors direction but also considers the magnitude of vectors in sorting. ART neural networks are classified as unsupervised learning nets, a method is presented for supervised learning of ART2 without general changes in the basic algorithm.

Keywords: artificial neural network, adaptive resonance theory, ART2, structural element classification, supervised learning

1. INTRODUCTION

An artificial neural network is an interconnected assembly of simple processing elements, units or nodes, whose functionality is loosely based on the human neurons. The processing ability of a network is stored in the interunit connection strengths, or weights, obtained by a process of adaptation to, or learning from, a set of training patterns [3]. There are many kinds of artificial neural networks that are different both in architecture and their ability. In this paper, attention is paid to the use of ART (Adaptive Resonance Theory) networks and their ability in sorting of structural elements.

2. ART NEURAL NETWORKS

ART networks are configured to recognize invariant properties of a given problem domain; when presented with data pertinent to the domain, the network can categorize it on the basis of this features. This process also categories when distinctly different data are presented, including the ability to create new. ART networks accommodate these requirements through interactions between different subsystems, designed to process previously encountered and unfamiliar events, respectively [12]. Two kinds of ART networks have been studied and can be

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distinguished essentially on the basis of the form of input information. They can accept binary or continuous inputs and on the approach used to process only this information. An ART network can process only binary input data and ART2 networks designed to process continuous input pattern data. A special characteristic of such networks is the plasticity that allows the system to learn new concepts and at the same time retain the stability that prevents destruction of previously learned information. ART networks accommodate these requirements through interactions between different subsystems, designed to previously encountered and unfamiliar events [2].

3. ART2 NETWORK

Adaptive Resonance Theory (ART) networks which were developed by Grossberg and Carpenter are self-organizing neural networks, and can be classified in unsupervised learning nets. ART nets automatically detect clustering and form classes of the data structure. A typical architecture of ART2 is illustrated in Figure 1. In this figure $s_i$ is the $i^{th}$ component of input vectors. $W_i$, $x_i$, $u_i$, $v_i$, $p_i$, $q_i$, and $y_i$ are called short term memories (STM). $b_{ij}$ and $t_{ji}$ are long term memories (LTM) of ART2 net [1].

![Figure 1. Typical ART2 architecture](image-url)
The update F1 activations are:

$$u_i = \frac{v_i}{c+V}$$  \hspace{1cm} (1)$$

$$w_i = s_i + au_i$$  \hspace{1cm} (2)$$

$$p_i = u_i + dt_{ji}$$  \hspace{1cm} (3)$$

$$q_i = \frac{p_i}{c+P}$$  \hspace{1cm} (4)$$

$$v_i = f(x_i) + bf(g_i)$$  \hspace{1cm} (5)$$

So that $c$, a small parameter introduced to prevent division by zero, $\|V\|$, vector length, $a$ and $b$ fixed weights in the F1-layer, $d$, activation of winning F2 unit, $\theta$, noise suppression parameter and $\alpha$ is the learning rate.

The activation function is:

$$f(x) = \begin{cases} 
  x & \text{if } x \geq \theta \\
  x & \text{if } x < \theta 
\end{cases}$$  \hspace{1cm} (6)$$

Input signals to F2-Layer are:

$$y_i = \sum_i b_{ij} p_i$$  \hspace{1cm} (7)$$

and weight updates for winning unit $J$ are:

$$t_{ji} = \alpha d u_i + \left(1 + \alpha d (d - 1)\right)t_{ji}$$  \hspace{1cm} (8)$$

$$b_{ij} = \alpha d u_i + \left(1 + \alpha d (d - 1)\right)b_{ij}$$  \hspace{1cm} (9)$$

4. MODIFIED ART2 NEURAL NETWORK

An ART2 network categorizes vectors by the similarity between them. In the basic architecture of ART2 network that introduced by G. Carpenter and S. Grossberg [6] the similarity is based...
on the direction of the vectors. If the angle between two vectors is zero, the similarity of this two vectors is 100% and in this condition ART2 network places this two vectors in one category by a vigilance parameter equal to one. In the basic architecture of ART2 the length of the vectors is neglected. This causes error in sorting of vectors when the magnitude of sorted vectors is important for user. For example, two parallel vectors that have a very different length can be placed in a similar category, but two vectors having a little difference in length and direction could be placed in two different categories. The following example will clarify the discussion: In matrix A each column represent a three-components vector.

\[
[A] = \begin{bmatrix}
1 & 2 & 1 & 2 & 1 & 5 & 4 & 5 \\
3 & 6 & 3 & 6 & 3 & 3 & 3 & 3 \\
5 & 10 & 5.2 & 10.4 & 6 & 7 & 7 & 7.5 \\
\end{bmatrix}
\]

Here three categories are defined for sorting of eight vectors in Matrix A. The result of sorting based on the basic ART2 net is shown in Table 1.

Table 1. Sorting of vectors based on the basic ART2 algorithm

<table>
<thead>
<tr>
<th>No. of categories</th>
<th>Category</th>
<th>Category 2</th>
<th>Category 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classified Vectors</td>
<td>1, 2, 3, 4, 5</td>
<td>6, 8</td>
<td>7</td>
</tr>
</tbody>
</table>

Vector 1 is parallel with vector 2, and vectors 3 are parallel with vector 4. Vectors 1, 3, and 5 are almost the same, but vectors 2 and 4 are completely different from the previous vectors. As it observed, the basic ART2 architecture places the first five vectors in one category. In the modified version of ART2 net, by adding a virtual input neuron to the net structure the above deficiency is removed, but also it considers the magnitude of vectors in sorting. In the modified version, for a vector containing n components, n+1 neurons for the input layer are defined. The last input component is an additional component that is called a Virtual Neuron, and its magnitude is equal to the square length of the vector, and can be determined as follow:

\[
S_{n+1} = \sum_{i=1}^{n} S_i^2
\]  

(10)

By using this additional component for the input vectors, ART2 network considers the magnitude of the vectors in addition to their direction, and a vigilance parameter is applied simultaneously for both magnitude and direction of input vectors. By this modification the ART2 network categorize vectors correctly, as for the previous example the results of the sorting based on the modified ART2 is shown in Table 2.

Table 2. Sorting of vectors based on the modified ART2 algorithm

<table>
<thead>
<tr>
<th>No. of categories</th>
<th>Category 1</th>
<th>Category 2</th>
<th>Category 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classified Vectors</td>
<td>1, 3, 5</td>
<td>2, 4</td>
<td>6, 7, 8</td>
</tr>
</tbody>
</table>
5. APPLICATION OF ART2 NEURAL NET IN SUPERVISED LEARNING

Basically ART2 neural net is categorized in the group of unsupervised neural nets. In this discussion a method is presented in which ART2 operates as a supervised net without total changes in the basic algorithm.

In this approach the net training is accomplished in the following two stages:

The first stage that is called the *training mode*, ART2 net is trained by the input training vectors. Each input vector has an additional component, which carries a significant number; vectors that are categorized in the same group have an equal number in the last component. Actually this last component is the modification which has been made on the basic ART2 network. At the beginning of the training process all vectors are applied to the net by vigilance parameter equal to one. With this vigilance parameter each vector is categorized in its own category and vectors with equal components will be in the same category. Then ART2 net calculates top-down and bottom-up weights for every category disregarding the last additional component of vectors, which described earlier.

The second stage of the proposed method that is called the *test mode*, a vector is applied to the net with a desirable vigilance parameter. ART2 net calculates top-down and bottom-up weight for this vector without any modification in the last calculated weights. Then ART2 net compares this vector to the categories with this new vigilance parameter. If similarity between this vector to a category is greater than vigilance parameter this vector will be classified in this category. But the actual classification belonging to the vector, is the number of last additional component of the first vector in this category. In the test mode stage, the calculated top-down and bottom-up weight does not change, because the first classification did not change in any level of the testing mode.

6. APPLICATION OF ART2 NET IN STRUCTURAL ENGINEERING

The modified ART2 neural net can be applied in sorting and categorization of structural elements. The approach can be used in sorting steel structural elements such as beam-columns, columns, beams and bracing system etc. without direct design of elements. As an example, the net is trained to categorize different non-standard latticed steel beam-columns made of the following elements:

- 2IPE140 latticed with PLs 160×8 mm @150 mm c/c
- 2IPE160 latticed with PLs 160×10 mm @150 mm c/c
- 2IPE180 latticed with PLs 220×8 mm @200 mm c/c
- 2IPE200 latticed with PLs 220×10 mm @200 mm c/c
- 2IPE270 latticed with PLs 300×10 mm @300 mm c/c

For this purpose a FORTRAN based program is used to prepare the training pairs. In this program the maximum axial force and bending moment considering different lengths for the proposed beam-columns are calculated. Each output training vector has four components: length of elements, axial force, bending moment and the type of column section.

The following limitations have been imposed on the training pairs: axial force is taken between 0 and 700 KN with 2 KN increment in each step, bending moment is taken between 0
and 200 KN.m with 2 KN.m increment in each step and the length of elements is taken between 2.5 m to 4 m with 0.1 meter increment in each step. By imposing the above limitations the outputs of the mentioned program was 8391 vectors. The net is trained based on the prepared training vectors, top-down and bottom-up weights are calculated in the training mode stage. In the test mode stage, the design results of an eighteen floors steel structure building were presented to the net and modified ART2 net classified all beam-columns in the correct categories with 0% error. To demonstrate the efficiency of the net, the mentioned steps in generating the training pairs are altered and the net results are summarized in Table 3.

<table>
<thead>
<tr>
<th>Length of element (meter)</th>
<th>Axial force (KN)</th>
<th>Bending moment (KN.m)</th>
<th>Percentage of error in classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min 2.5 Max 4 Incr. 0.1</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 2</td>
<td>0.0</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.2</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 2</td>
<td>0.44</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.1</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 2</td>
<td>0.26</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.2</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 5</td>
<td>0.71</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.1</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 5</td>
<td>0.61</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.2</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 5</td>
<td>0.79</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.1</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 5</td>
<td>0.79</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.2</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 5</td>
<td>0.97</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.5</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 10</td>
<td>23.83</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.5</td>
<td>Min 0.0 Max 700 Incr. 2</td>
<td>Min 0.0 Max 200 Incr. 20</td>
<td>25.41</td>
</tr>
<tr>
<td>Min 2.5 Max 4 Incr. 0.5</td>
<td>Min 0.0 Max 700 Incr. 20</td>
<td>0.0 Max 200 Incr. 20</td>
<td>33.28</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

In this paper ART2 artificial neural network has been modified for proper sorting of the input vectors. First a Virtual Neuron is introduced to the basic algorithm to consider length of vectors. This input Virtual Neuron improves the efficiency of the ART2 neural net, where the length of vectors is considered in sorting. Second, an additional component for each input vectors is introduced. This additional component is a number that carry a significant information about the categorization of vectors. This component helps the network in arrangement of vectors in the final categorization. By this second modification ART2 neural network operates as a supervised ART2 network is used as a structural steel or concrete element classifier for design purpose in a minimum time and effort especially for built-up members. This procedure can be summarized in the following two steps:

In the first step that is called the training mode, the net is trained based on the input vectors. In this mode network learns to categorize the vectors for the next application. Then in the second step that is called the test mode, network can categorize any input vector that is applied to the net with previous learning. Choosing small increments in the training mode will result in an exact classification of vectors.
REFERENCES

CONCRETE QUALITY WITH MIX WATER AND ENVIRONMENTAL IMPACT OF CONCRETE

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ABSTRACT

The requirements to produce and specify more durable concrete in aggressive environments are increased to satisfy the demand for long-term economy and serviceability of concrete structures. These in turn need better quality concrete, which makes construction and the finished structure better. Considerable work has been done investigating various parameters, such as, water-cement ratio, cement content, admixtures including corrosion-inhibiting admixtures, air-entrainment, condensed silica fume (CSF), and has been reported in many places. This paper contributes through work done on the influence of mix water conditioner (MWC) on concrete behavior and includes the chloride permeability. Its effect on hydration product in the mix is emphasized, better concrete will be produced if proper hydration is accomplished. In addition, the paper deals with the environmental impact of concrete production. It has become increasingly important for the concrete industry to minimize the impact of a ready mixed concrete industry on the environment. The main idea is that concrete industry should be cognizant of such practice, which will need to be followed in the future if we want to expand the facilities and not get short-chained due to restrictions of Regulatory requirements to impact the production and therefore the use of concrete.

**Keywords:** Concrete, durability, hydration, admixtures, chloride permeability, environmental management, solid waste, sater discharge, air pollution

1. INTRODUCTION

Attempts have been made to develop a natural, economical, uncomplicated and consistently reliable method, which would significantly enhance Portland cement's overall quality criteria. Such development generally begins with the experience of the current technology prior to the development of synthetic admixtures, such as air entrainment, water reducers, plasticizers, super plasticizers, micro-silica, etc. But unlike many of these admixtures, which are added to already-mixed cement, the inventors determined that the most promising media for enhancing Portland cement was concrete's mixing water. The mixing water is important since certain reactions during hydration - when the water initially comes into contact with Portland cement, can be modified with benefits.

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Experiments with natural ingredients capable of enhancing the hydration abilities of Portland cement were conducted to determine their interaction. It was found that certain ingredients provided water with extraordinary molecular binding power to allow larger volumes of so-called water of convenience in concrete to be utilized, rather than evaporated, for additional hydration of cement. This action reduced the capillary void sizes, making them comparable to gel pore sizes, and created a significant increase in the concrete's impermeability. This research resulted into a mix water “pre-conditioner” that would provide extraordinary benefits to Portland cement concrete through such improvement of mixing water.

2. UNIVERSAL NATURE OF CONCRETE

Concrete is one of the leading universal materials of construction. It is therefore of interest to the users and designers all over the world to improve its qualities. In the colder climate, there are more handicaps of making concrete. On the other hand, very high temperatures also create problem. Both of them result in developing admixtures, which have been mentioned earlier. This paper deals with an approach, which can be used in either condition in any country so that better concrete can be made and its application be further enhanced. Cost is another parameter, which has played role in the use of any material of construction and concrete is no exception.

Design based on the cost parameters needs to be developed and not only the strength criteria. It will then be truly economical to design and build concrete structures better than any other material of construction. If good concrete is made in the beginning and such concrete needs much less maintenance cost, it will be even more attractive material. This good concrete comes mostly from the ready-mix concrete plants and creates the environmental awareness in the public, which needs consideration.

3. ENVIRONMENTAL IMPACT FROM CONCRETE

Ready mixed concrete has reached such a level of production that one needs to think about its impact on the environment. This impact takes various forms. If minimized, it would help the industry in a long-term basis to achieve the most of it and at the same time, it would create a much better image of the industry in the society. Recommendations [1] are now available, which represent good industrial practices which are practical, realistic and economically viable from the US perspective, although they are applicable in any urban setting, where the public is becoming more cognizant of the concrete industry as part of their life. Eventually, Regulatory requirements may dictate plant upgrades regardless of the economic consequences. In short this may be considered as a wake-up call for the industry. Some of these aspects of environmental impact and its management are presented in appendix A. Thus, one would have considered all bases related to a good concrete practice leading to a construction material, which is not only universal, but is good and also environmentally friendly.

4. QUALITIES OF GOOD CONCRETE

Good concrete means durable concrete. The durability of a material is a property, which indicates whether or not the material will endure, even though it may not be subjected to loads
sufficient to destroy it. Durability of Portland cement concrete, then, is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete retains its original form, quality, and serviceability when exposed to its environment.

Durability of concrete is its most important property since it is essential that concrete be capable of withstanding conditions for which it was designed, throughout the useful lifespan of the structure. Durability of concrete can be affected by various factors including alternate wetting-drying cycles, freezing-thawing cycles, aggressive sulfate exposure, heating-cooling cycles, capillary water invasion, abrasion, corrosion of imbedded steel reinforcement/other imbedded materials, chemical contaminant reactions, alkali-aggregate reactions, salt deposition by percolating water, dissolving of calcium hydroxide and/or certain other constituents by percolating water, dissolving of cement by certain acids, etc.

To make concrete durable, its permeability must be reduced. Ideally, one should produce defect-free finely textured cement paste, which is capable of improving concrete paste-aggregate contact zone areas. By accomplishing this, the microstructure around the aggregate is modified and thus, the cement contact zones from the weakest part of the concrete to the strongest. Current methods of improving the performance and durability of concrete (such as High Performance Concrete) are both expensive and complicated. However, by preconditioning the water with the right ingredients, you can achieve these goals simply and with minimal expense. It was found that concrete could be produced with no separation between the aggregate and water. This concrete had virtually no plastic shrinkage cracking and virtually no curling. Extensive laboratory work showed that the mix water conditioner’s (MWC) conceptual goals were consistently achievable, without necessitating use of outside mineral or chemical admixtures, synthetic agents, water-reducers, or plasticizing agents.

5. ROLE OF WATER IN CONCRETE

Water is essential to Portland cement concrete performance to make the cement a good bonding Agent. There are two ways in which Portland cement and its compounds react with water. The first is "hydration", which is the reaction created when water molecules contact dry cement. The second is "hydrolysis", which is the reaction as a result of hydration. Hydrolysis is directly responsible for the production of new products or compounds that are essential in freshly hydrated concrete. Both these products and compounds, which are created as a result of hydration and hydrolysis, are usually referred to as hydration products. They include calcium hydroxide, formed from released lime and freed through hydrolysis, as well as several other calcium hydrates, often referred to as calcium silicate hydrates.

The quality of internally produced products of hydration may vary greatly, depending on the quality and impurity content of the cement being hydrated, as well as the quality of mix water used. In concrete, the rate of hydration decreases continuously following initial hydration, and continues even after 28 days while amounts of cement still remain unhydrated. Tests show that cement grains in constant contact with water may only hydrate to a depth of 4 m, and even after a year may only hydrate up to 8 m. Water enhanced by mix water conditioner to improve its cement hydration abilities, consistently improved cement grain hydration percentages. Under
field conditions, concrete utilizing mix water conditioner has shown a 6 to 12 percent increase in cement hydration to prove the role of mix water in hydration.

During the hydration of Portland cement concrete reactions occur as the mix water initially contacts the dry cement and split off molecular portions of the dry cement’s compound components. These reactions result in some essential byproducts, such as Calcium Silicate Hydrates (C-S-H) and calcium hydroxide. These reactions, in combination with water dilution, temporarily lower Portland cement’s potency, causing the initial poor quality cement paste. This cement paste is initially absorbed by or coats the exterior of aggregates. The resultant concrete integrity is affected by weak paste-to-aggregate bonding quality, as well as the deposition of paste, which contains poor microstructure. It in turn affects paste-aggregate contact zone areas in concrete affecting permeability and thus its durability. However, the MWC effectively alleviates some of the reactions associated with hydration, and beneficially lowers heat evolution while promoting a smoother, gentle transition into the continuing phase of hydration. The MWC also greatly improves the hydration/hydrolysis byproduct quality (i.e., calcium hydroxide, calcium sulfoaluminate, and others). This allows an efficient use of the byproducts in production of beneficial C-S-H, leaving only minimal unused portions in the resultant concrete, as an end result.

6. ROLE OF ADMIXTURES IN CONCRETE PROPERTIES

Designers using admixture materials can tailor and adjust mixes to meet a wide variety of performance requirements. Most mix designs today include additional cementitious materials and admixtures. Natural pozzolans fly ash and slags supplement replace a portion of the Portland cement. Air entraining admixtures are used to improve freeze-thaw resistance. Chemical admixtures are used to accelerate or retard set, improve workability, reduce mixing water or increase strength. Adding these ingredients requires a thorough knowledge of mix designs.

In a much simpler way, MWC in water readily utilizes an additional 6 percent or more of already-included cement content in the mix, which in turn effectively increases the cementitious material content volume. It does this without increase of the originally designed dry cement volume (per-cubic-yard), which in turn lowers the water/cement ratio. This extraordinarily beneficial effect is due to the property of the preconditioned water in the mix to absorb more deeply the cement particles, posturing each cement particle to more readily and more often shed its mix water generated hydrate envelopes. It then allows new envelopes to be regenerated, which effectively causes the utilization of significantly increased percentages of each cement particle. It also uniquely generates additional volumes of cement paste, subsequently C-S-H or tobermorite gel per cement particle.

It is a known fact that when water is added in any mix, it reduces its strength. The objective, then, is to transform the water into a better product, such as cementitious material that produces even a lower water-cement ratio and not influencing the strength. Introducing selected natural ingredients into the mix water will generate increased utilization of water of convenience, in the mix producing virtually no bleed water. Following example will illustrate the use. Let’s assume that 5 gallons of water fully hydrates 100 pounds of cement with a water ratio of 0.42. Only 2.88 Gallons of normal mix water actually combines with the cement, while the remaining 2.1 gallons of water occupies capillary spaces following the surface finish until it is used for
production of the hydration products or is later evaporated, which leaves the void capillaries. When the mix water is preconditioned, however, virtually all of the normal mix water is used due to the increased cement particle saturation schematically shown in Figure 1.

![Figure 1: 100 Pounds of Cement Hydrated by 5 Gallons of Water](image)

Using the ACI standard that in a given mix design of 470 pounds of Type 1 Portland cement, only 80% (376 pounds) of the cement is hydrated. Mix water conditioner produces a minimum of 6-8% (28-38 pounds) more cementitious material from that remaining 18-20% (84-94 pounds) of unhydrated cement. This results in a very homogeneous ready mix as shown in Figure 2.

![Figure 2: Mix design: 470 Pounds Type 1 Portland Cement](image)

7. PROPERTIES OF CONCRETE

Based on the extensive testing, several MWC showed the following enhanced properties from utilization of increased percentages of the already-included Portland cement content in the concrete mix.

1. The usual (leftover) cement particle cores were much smaller, which increased impermeability and density. These particle cores became different sized through increased cement utilization resulting in the leftover cores to act as sand/aggregate binder. This was tested extensively [2].
2. The cement paste was finely textured and homogenous. It provided with the same charged-particle effect and greatly reduced the potential for internal voids, shrinkage, excessive external bleed water, internal micro-cracking, crazing, plastic/settlement cracking, etc.

3. The concrete was extremely homogenous with the increased workability due to increased lubricity [2].

4. Cementitious material waste was reduced and water utilization was increased. This resulted in the increased volumes of cementitious material utilization per cement particle, effectively lowering water/cement ratio of finished concrete.

5. It generated more finely textured cement paste consisting of smaller-size gel pores with excellent uniformity and smaller than usual capillary pores/voids. This is due to total mix water utilization, which results in increased hydration product volumes. These actions significantly lower total void percentages and permeability and thus increasing durability shown in Figure 3.

6. It produced true shrinkage-compensating concrete, compensated by production of increased volumes of C-S-H, the hydration product, which also virtually eliminated curling.

![Sample #1 - From Each Core, One Inch From Top of Deck.](image1.png)

![Sample #2 - From Each Core, Three Inches From Top of Deck.](image2.png)

Figure 3

8. DISCUSSION

From the durability standpoint, it is of crucial importance to achieve the lowest possible permeability in the shortest period of time possible. This was achieved when the MWC was used. If less pollutants or contaminants were allowed to penetrate in the interior of concrete, it would be less permeable [1]. The degree of permeability dictated whether these pollutants/contaminants would readily - or sparingly - allow ingress. Therefore, permeability effectively and directly affected concrete’s durability and translated into a longer service lifespan. For concrete made using normal weight aggregate, permeability is governed by porosity of the cement paste and pore-size distribution. In fact, its permeability is generally controlled by the capillary porosity and not by the gel porosity. Paste capillary porosity size is governed by water-cementitious material ratio in the mix and the degree of hydration. Mix water conditioner was designed to significantly enhance increased cementitious material
volumes and improved hydration rates and processes through re-organizing the mix water's hydration abilities.

9. EXAMPLES

Examples presented here from different jobs to illustrate a wide variety of applications of MWC in practice.

Figure 4

Figure 4 shows the Dally Center in Chicago, in which a 4.5 x 96 meters section was poured during the rehabilitation work at the center. The MWC treated mix was poured over the irregular bitumen surface. Bonding was excellent and there was no cracking except in one corner of the inset, which was probably due to deflection.

Figure 5
Figure 5 shows an Exxon filling station, where the whole area was decked with 28 MPa concrete. It used 227 Kg. of cement, 1-in. aggregate and 23-cm. slump. The yard after several years is in very good condition with no distress (cracking) and the cores taken at our random location at 28 days showed the strength of over 41 MPa.

Figure 6

Figure 6 shows a subway system floor in Chicago Transit Authority in 1995. The floor used another form of MWC, concrete identifier-sealer (CDS) which was used for protection in the rehabilitation of concrete. The CDS was spray-applied, which penetrated to a substantial depth. In this manner, it reduces significantly rust producing reactions. It did not alter concrete surface or appearance or its physical characteristics. The traffic was opened within a few hours without impairing traction or bonding.
10. CONCLUSIONS

It is shown that the mix water pre-conditioner added to Portland cement concrete mixing water, prior to concrete manufacturing, significantly eliminates all of the deterioration factors in concrete and more. Water in the mix, which has been preconditioned with the right amount of MWC will consistently produce concrete that is very strong, hard, impermeable and durable. It should be stressed that we can learn from our past experience to develop better concrete for the future. While doing so, one must recognize the need to preserve the environment and a good relationship with the community to improve the image of our industry as well as engineer in general.

Environmental impact should be considered during the production of ready-mix concrete. This will allow a better image of concrete and more useful applications of concrete.

Another point must be made related to concrete as universal material and its industry needs to promote it from developed world with all the good experience with new materials to the to developing world. This will bring the two worlds together to benefit everyone.

Acknowledgment: The late David Johnson, President of Applied Concrete Technology, Inc. of Arlington Heights, Illinois Heights, prepared original draft of the paper. His contribution to the state of the art of mix-water in concrete is gratefully acknowledged. Thanks are also due to him and his associate for making the materials available for testing and photographs from their fieldwork used in this paper.

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APPENDIX A: DETAILS OF ENVIRONMENTAL IMPACT AND MANAGEMENT

National Ready Mix Concrete Association (NRMCA) (Ref. 1) has prepared recommendations for environmental management, which represent good industrial practice. They are practical, realistic and economically viable from the US perspective, although they are applicable in any
urban setting, where the public is becoming more cognizant of the concrete industry as part of their life. Eventually, Regulatory requirements may dictate plant upgrades regardless of the economic consequences. In short this may be considered as a wake-up call for the industry.

Other available environmental management tools include corporate mission statements, corporate policies and procedures, and internal plant audit inspections. The NRMCA Plant Audit Checklist was developed to serve as an internal guide for concrete plant managers. It should also be mentioned that Canadian Environmental code of Practices (1993) is available, which should be referred to as more detailed background version of this presentation.

Ready mixed concrete industry should be responsible environmentally and be nuisance-free and operate without causing such problems; it is then imperative that concrete producers maintain a positive image of their plants and equipment. As a manufacturing facility, ready mixed concrete plant operation is not a significant threat to the environment. However, main concerns may stem out of discharges due to the use of water and other ingredients, which may find their way into public facilities, and some pollution due to air and noise disturbances to the community. Thus, we need to address such issues and let the public be aware of our share of the problem and to make our industry a model to others in the vicinity.

Concrete producers can significantly impact the perception of their neighbors and others in their community. This perception, whether positive or negative, can be improved by the company’s involvement in community affairs. From technical perspective, this involvement can take several forms, such as the contribution of goods or services for worthwhile community projects, supporting local student projects etc. The other sore problem is with concrete truck mixers, which are the advertising billboards for the concrete industry. They must be kept clean, freshly painted and well maintained to present a positive industry image. Use attractively designed company logos on truck mixers and other company vehicles. Consider using community logos or slogans on truck mixers, for example “Just Say No to Drugs.” Concrete plants and related buildings must be well maintained and present an attractive appearance. Truck and equipment parts, tires, empty drums, solid waste and other debris should not be in public view at the plant. Concrete has a number of components, which are known to every one; it is time to see which of these in one form or the other may create a problem environmentally that one needs to manage.
- Cementitious material (cement, fly ash etc.)
- Sand
- Aggregate
- Water
- Admixtures (water-reducer, air-entertainment agent etc.)

These are familiar, but once they make "Concrete" and most of the concrete is used for the purpose it was made for, the remaining part, which needs to be discarded need to be managed properly. The various management aspects from the concrete plant operation of interest, are specified as follows:

Water Management
Noise Management
Solid Materials Management
Admixture, Chemical and Fuel Management
Air Quality Management
Plant Closing (either temporary or permanent)

**Water** is used in the operation of a ready mixed concrete plant in several ways from mixing it in the batching concrete to rinsing the truck after loading and cleaning it at the job site and at the end of the day. It also includes dust suppression in the yard, on roadways, on stockpiles and at the point of loading mixers, heating, cooling and soaking aggregates and finally rinsing the reclaimed aggregates from leftover concrete.

Water is an irreplaceable commodity in any of these applications, but it can be even more expensive resource unless its use is properly controlled. It can be of short supply in some regions and can make a significant impact on operations and cost efficiency of the plant. Wasting water in the operation of plant during mixing it for batching should be avoided by eliminating the overflow during filling and reducing it for rinsing trucks after loading. On the other hand, the exterior of the truck mixer must be rinsed free of aggregates and cement dust after loading so that dust is not tracked out of the yard, which may give rise to dust or air pollution problem. Rinsing should be done on a paved area with slopes toward the water collection basin so water may be captured at the end of the day. Some of this water can be reused.

Rinsing chutes and the mixer drum at the job site can be a problem in some areas. Producers should make sure that the clean up take place in an area approved by the customer/owner. Mixer trucks should never wash into catch basins or streams, as this is a criminal violation of the Clean Water Act. A bag or diaper can be placed over the chute to prevent spillage and allow the truck to return to the plant to cleanup.

If possible, recycled water should be used for rinsing truck mixers at the end of the day. The rinse water should first be drawn from a settling pit, holding tank or reclaiming equipment. An effective alternative is to use recycling admixtures to stabilize the concrete residues in the drum. Use only 40 to 50 gallons of fresh water and the suggested dosage of recycling admixtures to rinse mixer drums. The resulting slurry is then held on the truck and then incorporated into the first batch the following day.

In some states the EPA is the sole regulator. In other states the EPA has delegated its responsibility to the state government. Regardless of the minimum standards set by the EPA, State or local governments may enact higher standards for compliance. The minimum standards for discharge of industrial process water were enacted in the 1972 amendment to the Water Pollution Control Act (referred to as the Clean Water Act). The Clean Water Act prohibits discharge of industrial process water into waters of the state without the issuance of an NPDES permit.

Water Management also includes other waters, such as industrially processed water, storm water, etc., which are discussed here.

**Noise** is a common complaint of neighbors located near a concrete plant. Ready mixed concrete plants and related equipment sometimes generate noise that may be unacceptable to neighbors. Plant managers must take steps to minimize noise levels due to plant operation or due to truck, so as not to create a nuisance.

**Solid** waste generally is concrete in the hardened or plastic state and reclaimed solids from mechanical recycling equipment or rinse water settling systems. These solids may be used for landfill but there are less and less sites available and one may have to find an alternate to reuse it as byproduct rather than disposing it. Some methods of using returned waste include:
- production of precast products, such as highway barriers, etc.
- paving yard surfaces or use it around the plant as fill
- if hardened, then breaking it and re-cycling it
- re-cycling it into cement manufacturing
- discharging into wash water collection system

Admixtures are widely used in concrete industry as ingredients. Most of them are liquids and are supplied in bulk. Fortunately, they have more than 50% water and inorganic salts, wood sugars or resins. Most of these are non-toxic from environmental point of view, but there is a trend to have a secondary containment to bulk storage tanks, which have foundation on concrete base slab. CaCl₂ is one of the admixtures, which is in powder form and delivered in bags. It is easier to deal with them by storing them indoors, without causing any environmental problems.

Air quality problem is generated through the dust through the normal plant operation. The principal concern is the release of small particle dust (less than 10 μm in diameter). They pose a health and safety risk to persons who may breathe these particles. While there are several sources of dust, such as cement and fly ash during filling, stockpiling of aggregate and their use in different places truck mixer loading and charging, etc. It is best to minimize dust, and if not it should be suppressed. Often dust can be collected, (filtered) and bagged and stored in a safe place for final disposal. Concrete Plants are usually required to have "air quality" permits for the entire plant to ensure that emissions from plant falls below the minimum threshold limit.
ASSESSMENT OF INTEGRITY OF CONCRETE STRUCTURES
APPLICATION OF INNOVATIVE NDT-METHODS

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ABSTRACT

The assessment and upgrading of structures are engineering disciplines which are gaining significance both from a technical and economic point of view. Important is the correct application of the adequate inspection and diagnosis techniques to get the required information for a quality assured rehabilitation. Helpful are standards, guidelines and evaluation reports concerning the applied methods.

Key words: concrete structures, NDT methods, rehabilitation, assessment

1. INSPECTION AS A QUALITY INSTRUMENT IN REHABILITATION

Inspection techniques are useful for the durability and the maintenance of existing structures. They give information about the condition of buildings, materials and their interactions with the environment. Inspection techniques must detect defects or deviations, which affect the building in its load capacity or its durability in an early state, e.g. corrosion, fractures, voids and moisture penetration. Table 1 assigns traditional and well-tried inspection techniques to protection principles, because defects or deviations must be detected before the rehabilitation [1] can start. The European standard ENV 1504 part 9 summarizes 11 protection principles related to defects in reinforced concrete (Table 1).

Figure 1 Quality circle for rehabilitation works. The dark zones indicate at which points or at what state of the process physical testing must be carried out.

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In a competitive world quality is not the only factor that determines success or failure, but still it is one of the most important ones according to our present high standards. The fact that quality cannot be tested into a product, but must be developed as an integral part of it has become widely accepted by now. The construction industry therefore focuses on improving the sequence of operations and thus on optimizing the production process. In this process-related approach quality stands for meeting the specifications and for the ideal of doing things right "the first time round". Based on these considerations, a "quality circle" specifying the individual steps of quality management can be established. The quality circle for preventive maintenance works is made up of zones that indicate when or at which point in the design, execution and service life physical examinations must be effected. The present paper points out the different tests used nowadays to determine the projected state, the actual state as well as the divergence between these two states.

Table 1: Principles and methods related to defects in concrete or reinforcement corrosion in accordance with ENV 1504-9.

<table>
<thead>
<tr>
<th>Principle No.</th>
<th>Principle and its definition</th>
<th>Inspection Techniques</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principle 1</td>
<td>Protection against ingress</td>
<td>Porosimetry Ultrasound Covermeter</td>
</tr>
<tr>
<td>(PI)</td>
<td>Reducing or preventing the ingress of adverse agents, e.g. water, other liquids, vapor, gas, chemicals and biological agents.</td>
<td></td>
</tr>
<tr>
<td>Principle 2</td>
<td>Moisture control</td>
<td>Radar NMR Microwaves Measurement of Conductivity Measurement of Capacity</td>
</tr>
<tr>
<td>(MC)</td>
<td>Adjusting and maintaining the moisture content in the concrete within a specified range of values.</td>
<td></td>
</tr>
<tr>
<td>Principle 3</td>
<td>Concrete restoration</td>
<td>Rebound Hammer Ultrasound Measurement of Carbonation Depth Measurement of Chloride Profiles Thermograph</td>
</tr>
<tr>
<td>(CR)</td>
<td>Restoring the original concrete of an element of the structure to the originally specified shape and function.</td>
<td></td>
</tr>
<tr>
<td>Principle 4</td>
<td>Structural strengthening</td>
<td>Rebound Hammer Measurement of Pull-Out Strength Ultrasound Impact-Echo Reemergence-Magnetism-Method</td>
</tr>
<tr>
<td>(SS)</td>
<td>Increasing or restoring the structural load bearing capacity of an element of the concrete structure.</td>
<td></td>
</tr>
<tr>
<td>Principle 5</td>
<td>Physical resistance</td>
<td>Porosimetry Rebound Hammer Abrasion Resistance Tests</td>
</tr>
<tr>
<td>(PR)</td>
<td>Increasing resistance to physical or mechanical attack.</td>
<td></td>
</tr>
</tbody>
</table>
| Principle 6  (RC) | **Resistance to chemicals**  
Increasing resistance of the concrete surface to deterioration's by chemical attack. | **Porosimetry**  
**Covermeter** |
|------------------|-----------------------------------------------------------------|-----------------|
| Principle 7  (RP) | **Preserving or restoring passivity**  
Creating chemical conditions in which the surface of the reinforcement is maintained in or is returned to a passive condition. | **Potential Measurement**  
**Covermeter**  
**Reemergence-Magnetism-Method Radar**  
**Porosimetry** |
| Principle 8  (IR) | **Increasing resistance**  
Increasing the electrical resistance of the concrete. | see Principle 7 |
| Principle 9  (CC) | **Cathodic control**  
Creating conditions in which potentially cathodic areas of reinforcement are unable to take part in the corrosion reaction. | see Principle 7 |
| Principle 10  | **Cathodic protection** | see Principle 7 |
| Principle 11  (CA) | **Control of anoxic areas**  
Creating conditions in which potentially anoxic areas of reinforcement are unable to take part in the corrosion reaction. | see Principle 7 |

Moreover, recent progress will be reported with regard to non-destructive test methods that are considered state-of-the-art and which allow the competent design engineer, consultant or contractor to gain a deep insight into the structure. The following methods will be discussed:
- The ultrasonic method to measure thickness and to locate defects
- The radar method to detect cavities and to locate tendons
- The reemergence magnetism method to locate fracture of steel
- The lithoscopical method to determine the concrete cover over a large area

2. ASSESSMENT OF CONCRETE COVER, A FREQUENT CONFLICT CASE

To ensure corrosion protection of reinforcement in reinforced concrete structures standards define minimum concrete cover depending on environmental conditions.

In a specific case prefabricated concrete elements had areas where the concrete cover did not come up with $c = 20$ mm, which is the minimum value prescribed by the German standard for an exposed concrete, strength class at least B 35, C 30/37 respectively. If spot checks indicate the lack of quality concerning concrete cover – what can be done further to save the construction, in this case a some hundred meters long concrete wall of prefabricated elements? Besides the application of NDT-techniques specific engineering skills are requested in the process of decision making. Here the example:
To assess the corrosion protection of the reinforcement information about the distribution of the concrete cover and information about the tightness of the concrete are required.

2.1 The concrete cover distribution
The distribution of the concrete cover is gained from a sufficiently large number of measurement values of a sufficiently large spot-check of concrete elements [2]. For the measurement a computer-Covermeter used (Figure 2) with the advantage that the measurement values are stored automatically available for a following statistical evaluation.

In a statistical approach the minimum concrete cover is considered as the 5 %-fractal of the concrete cover distribution following the German DBV-guideline: “Concrete Cover and Reinforcement” [3].

![Figure 2 Principal sketch of the Covermeter. A measurement curve on the right shows a threshold value (dashed line) of 20 mm and the concrete cover of single bars.](image)

It turned out that neither the normal distribution (Figure 3) nor the lognormal distribution (Figure 4) fitted the measured data set very well. However a linear combination of both distribution functions led to useful results (Figure 5). The statistical evaluation turned out that the 5 %-fractal was only 1.5 mm smaller than the value of \( c = 20 \) mm demanded by the standard.

![Figure 3 The relative frequency \( h(c) \) of the measurement values of concrete cover. The curve shows the best fit of the normal distribution. The Approximation is insufficient.](image)

![Figure 4 The relative frequency \( h(c) \) of the measurement values of concrete cover. The curve shows the best fit of the lognormal distribution. This approximation is also insufficient.](image)
Figure 5 The relative frequency $h(c)$ of the measurement values of concrete cover. The curve shows the best fit of a linear combination of the normal distribution and the lognormal distribution. It is a fairly good approximation.

2.2 The tightness of concrete cover
The next step of investigations focussed on the tightness of the concrete cover. A possible corrosion process of the reinforcement in an environment free of chlorides is starting only, if the front of carbonation reaches the reinforcement. The carbonation velocity depends on the diffusion coefficient of the concrete and thus on its total porosity and its pore size distribution. The concrete that had to be assessed belonged to the strength class B 45, which is corresponding to the strength class C 40/50. It is fair to assume, that the tested concrete is less porous than concrete of the strength class B 35. The total porosity was measured using a helium pycnometer. The result was $p_w = 13.3$ Vol-%. Under normal environmental conditions a concrete with a total porosity $p_w < 16$ Vol-% is considered as sufficiently durable. Also the pore size distribution (Figure 6), determined by a mercury pressure Porosimetry, showed its maximum in the range of small pores with radii between 0.01 mm to 0.1 mm. There were almost no large pores in the range between 10 mm and 100 mm.

This procedure led to a rationally established assessment, which allowed to the owner to accept in this case the small deviation of the target.

Figure 6 The pore size distribution and the cumulative volume of the assessed concrete.
3. A SEVERE DANGER: FRACTURES IN PRESTRESSING STEEL – HOW TO DETECT THEM?

Until the Reemergence-Magnetism-Method (RM-Method) has been developed, the only possibility to examine the condition of prostrating steel wires in prestressed concrete members was their visual inspection after a local destruction in order to remove the concrete cover. The disadvantage of this method is the local limitation of the inspection zone. A further disadvantage lies in the fact that after the metal sheathing has been opened only the first layer with three or four wires of a bundle of 16 up to 48 wires can be inspected.

![Figure 7](image1.png)

Figure 7 If a bar magnet is broken into two parts, a new magnetic dipole is forming (left). A similar dipole-distribution is also forming in the region of a fracture of a magnetized prostrating steel bar in a tendon. At the concrete surface a characteristic magnetic signal is measurable. The transverse component of the magnetic flux density is schematically shown in the right Figure.

![Figure 8](image2.png)

Figure 8 Magnetic measurement of the prostrating reinforcement in an I-shaped girder of a factory hall.

The transirradiation method (e.g. x-ray) was supposed to be appropriate to locate fractures of prostrating steel bars. This detection method requires a detectable fracture width. The fracture is not detectable with the transirradiation method, if other steel bars screen it. A magnetic method coping with the above-mentioned difficulties has been developed [4], [5], [6]. The magnetic field resulting from a magnetized tendon or a magnetized steel wire is comparable to the
magnetic field of a bar magnet. In the region of a fracture a magnetic dipole-distribution is forming and accordingly a magnetic leakage field (Figure 7). This characteristic leakage field is measurable at the concrete surface.

The method has been applied on various full size units. The installed measurement device is shown in Figure 8, fixed at the side surface of a girder for inspection. Girders and transverse tendons of bridges are very often a target of inspection.

The German federal highway research institute BAST has evaluated the method [7].

4. AN EVERYDAY PROBLEM: DETECTION OF CRACKS AND HONEYCOMBS IN CONCRETE

The Ultrasonic Pulse-Echo- or Pulse-Velocity-Methods is the right tools for this type of detection. The technique is traditional and well tried, so a new German guideline is just available: “Ultrasonic impulse method for NDT of mineral building materials and building parts”, published by the German Society of Non-Destructive Testing DGZfP [8].

When the load capacity of a concrete unit is in doubt, because poorly compacted areas are becoming visible after formwork removal, it is desirable to know whether internally of the concrete air voids, gravel pockets and areas of low concrete strength are existing, and if so, where they are located exactly. Ultrasonics is not a well tried method when the concrete reaches almost 1 meter thickness. But nevertheless it works by implementing engineering extra skills: Ultrasonic measurements were used to identify areas of low quality concrete in more than 0.80 m thick columns by correlating pulse velocity with the compressive strength of sample cores drilled. It is shown that grid measurements and 3D-visualization are essential tools to obtain an easy-to-read picture of the interior of the columns (Figure 9).

Another problem is shown in Figure 10, where cavities and cracks with a main extension orthogonal to the concrete surface, had to be detected. Also in this case ultrasound was the right procedure. Ultrasonic is frequently used, when cavities in concrete are to be located. However, if defects expected having only a small extension parallel to the concrete surface, then the reflected signal is too weak and in particular too blurred, in order to detect them. Cavities of this type can occur for example due to unsatisfactory compaction within the range of construction joints and also in brickwork between stone and mortar. The testing problem was solved by an arrangement of the ultrasonic transmitter and the receiver on the concrete surface in a way that the possible cavity is covered (Figure 12). This method uses a wave property of the ultrasound, i.e., that the energy does not transmit straight-lined, but that each point of the wave is the starting point of an elementary wave (Hugenness). So-called exponential horns, which possess an almost radial symmetrical characteristic, are used as transmitter and receiver.

A cavity that is mainly extended orthogonal to the concrete surface leads to the fact that a part of the energy, that would reach the receiver in the case of a defect free material, is not transmitted to the receiver.

The received signal is time-dependently represented after a numeric operation on an Oscilloscope (Figure 11). A defect becomes apparent by the fact that the transmitting intensity is weakened (Figure 10). That part of the pulse is missing. The transmission of this part is impeded by the defect.
Figure 9 A column with interpolated levels of ultrasound velocity. The darker zones represent a relative low pulse velocity. In a second step these areas are inspected with conventional techniques.

In the Basilica San Francesco in Assisi the procedure was used in order to detect cavities in the joints of the brick-work of the vaults. Drillings acknowledged the non-destructively detected cavities and serve at the same time as holes for injections in the context of the repair. The success of the injection work can be proven afterwards with the same procedure non-destructively.

Figure 10 Schematic representation of sound propagation. If no defect is present (on the left), a higher intensity is transmitted, compared to the case in which a defect is situated between transmitter and receiver (on the right).
Figure 11 The representation of the transmitted intensity on an oscilloscope. On the left the transmission is clearly stronger. No defect is present. On the right the transmission is weakened by a defect.

Figure 12 In prefabricated columns of a balcony construction hollow zones in construction joints attracted attention. 2000 columns had been inspected with ultrasound. Transmitter and receiver were placed in such a way that they cover this area.
5. THE RADAR METHOD

The radar method (Figure 13) allows detecting boundaries of materials with different dielectric properties. This method is able to locate moisture inclusions but also metal items like ducts.

Figure 13 Principal sketch of geo radar. The impulse is partly reflected at boundaries where the dielectric constant changes. Radar signals caused by ducts and reinforcement are shown on the right.

6. MEASUREMENT OF THICKNESS AND CONDITION OF TUNNEL WALLS

To find the adequate method to test the condition of tunnel walls and to measure their thickness some theoretical considerations are helpful [9].

Tunnel walls are normally accessible only from inside the tunnel. Therefore only methods that use the reflection of waves are applicable. The amplitude of the reflected waves is mainly determined by the change of the characteristic impedance at the boundary layer between two materials: The bigger the difference in the characteristic impedance of the two materials, the stronger the reflection.

The amplitude $A_R$ of the reflected wave as a function of the amplitude of the incident wave $A_I$ is calculated for the case of orthogonal incidence on a plane boundary layer between material 1 and material 2 by

$$A_R = R_{12} A_I$$

With the reflection factor

$$R_{12} = (-1)^n \frac{Z_1 - Z_2}{Z_1 + Z_2} \quad \text{with} \quad n = \begin{cases} 1 & \text{for nonmetals} \\ 2 & \text{for acoustic waves} \end{cases}$$

which is determined by the characteristic impedance’s $Z_1$ and $Z_2$.

The characteristic impedance $Z$ for electromagnetic waves is given by
\[ Z = \sqrt{\frac{\mu \mu_0}{\varepsilon \varepsilon_0}} = \begin{cases} \frac{\text{const}}{\sqrt{\varepsilon}} & \text{for nonmetals} \\ \frac{1}{\sqrt{\mu}} & \text{for metals} \end{cases} \]

With
- \( \varepsilon \): Dielectric constant of the material (real part)
- \( \varepsilon_0 \): Influence constant
- \( \mu \): Magnetic permeability of the material
- \( \mu_0 \): Induction constant

For acoustic waves the impedance is given by

\[ Z = \frac{p}{c} \approx \sqrt{pE} , \]

With
- \( p \): Density of the material
- \( c \): Sound velocity in the material
- \( E \): Modulus of elasticity

The knowledge of the material parameters (e.g. from literature) and their deviations enables us to judge the principal capability of methods using acoustic or electromagnetic waves to detect boundary layers, for example the layer between the tunnel (concrete) and soil (e.g. sand or quartzite) as shown by Figure 14 and Figure 15.

The curve in Figure 14 indicates, whether acoustic or electromagnetic waves lead to better results of e.g. thickness measurements. The curve can be used universally. Only the quotient

\[ k = \begin{cases} Z_1 \quad \text{for acoustic waves} \\ Z_2 \quad \text{for electromagnetic waves} \end{cases} \]

has to be calculated to find the reflection factor \( R_{12} \).

To measure the thickness of tunnel walls acoustic waves are obviously more adequate than electromagnetic waves. Acoustic waves have a further advantage in this case: The influence of the reinforcement of the tunnel wall on acoustic waves is much weaker than on electromagnetic waves like RADAR. There is a total reflection of electromagnetic waves at metals. This theoretical consideration can save a lot of time, money and trouble. It helps to find the adequate method in an early stage of solving a measuring problem.
Figure 14 The relative amplitude of the reflected wave is expressed by the reflection factor $R_{12}$, which is a function of $k$ the quotient of the characteristically impedance's of the two materials at the boundary layer. The ranges for $k$ are marked for electromagnetic and for acoustic waves. The marked ranges are valid for boundary layers between concrete and rock (in this case: quartzite).

Figure 15 The same curve as in Figure 14. The marked ranges concern a boundary layer between concrete and sand. For electromagnetic waves the reflection at the bound-ray between concrete and sand is weak or does not even occur.
REFERENCES


TECHNICAL NOTE

HEAT-CONDUCTING PROPERTIES OF SMALL-POWER-HUNGRY CELLULAR CONCRETE

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ABSTRACT

One of the most effective thermal insulation materials is unautoclave cellular concrete in modified cement. The laboratory scientific research on composition developments and technology of unautoclave cellular concrete fabrication were carried out. In the composition of elaborated concrete, the modified cement mix, gas making substance, structure formation additives, setting accelerators, polymeric admixtures and filler from coarse-ground quartz sand were used. Elaborated material has the density 180...400 Kg/m³, compressive strength 0.3...0.53 Mpa, coefficient of heat conductivity 0.07...0.105 W/m. K.

Keywords: unautoclave cellular concrete, heat conductivity, thermal insulation, compositions, polymeric admixtures

1. MIX DESIGN

The creation of new thermal insulation materials on the basis of mineral binding remains one of the main directions, enabling to solve the problem of thermal insulation of buildings under the conditions of power crisis in Russia.

The diminution of the density of especially lightweight concrete (ELC) up to 150-300 Kg/m³ favors to hold coefficient of heat conductivity within 0.05-0.08 W/m. K. Constructions, made of ELC, equally with high thermal-insulating properties should meet the requirements of non-combustibility, toxicology and longevity.

In our laboratory the scientific research of ELC compositions with the bindings on the basis of modified Portland cement and gypsum-cement-molding flask (GCF). The elaborated compositions represent high porous (the pores volume to 94%) concrete stones, making by means of air curing of specially mix design, consisting of binding fine-ground quartz sand, water and adjusting admixtures.

Depending on the kind of pores-forming admixture, the elaborated ELC compositions are gas or foam concretes. In order to get especially lightweight gas concrete we used the standard gas-forming admixture-aluminium powder and alkali additions regarded as gas formation and setting accelerators. In the process of making the ELC the fine-ground quartz sand and binding mixture, containing the foam-forming admixtures were mechanically air-entrained. The peculiarity of all elaborated compositions is air curing (t=+18-20°C, relative humidity φ=90-98%).
The optimum ELC compositions with the use of Portland cement, tested at the age of 7 days have the compression strength 0.25-0.6 Mpa and density in dry state 150-300 Kg/m³. Under favorable conditions the rise in strength for investigated materials continues even after 7 days. Their density-strength relation is shown in Figure 1.

![Graph showing density-strength relation](image)

**Figure 1** Density-strength relation  
1- Strength of gas concrete; 2- strength of GCF binding; 3- strength of foam concrete.

### 2. APPLICATIONS

In regard to structure formation peculiarities and physical-mechanical properties of elaborated ELC one can suppose that their most preferable field of application is cast-in-situ thermal insulation of the buildings.

In the course of researches the possibility to use GCF binding for the making of ELC was revealed. The composition of that binding is a special mix design on the basis of gyps (A), portland cement (B) and molding flask admixture (C). The most valuable properties of GCF binding for the manufacturing of ELC are rapid hardening (due to gyps hardening) and high water-resisting property (is secured by cement component).

As active mineral admixture locally procurable rocks (Sureck molding flask) were used. The molding flask, that is deposited in Penza region, consists on the whole of opal silicon dioxide (SiO₂ₙH₂O) with clay slip additions. The activity of used admixture in respect to calcium oxide, determined in conformity with the recommendations TY21-32-62-89 «Gyps-cement-molding flask binding», is within the limits of 210-240 mg/gram.

On the initial stage a work on the binding proportioning was accomplished with a view to find the optimum composition of ELC on the basis of GCE binding. Compression strength and water-resisting property served as the criterions of the optimum binding composition. In Figure 2.3 the relations of compression strength-coefficient of water resisting Kₜ (after 48 hours curing by ponding) component parts (on mass: A-gyps mark "Г-5," B-Portland cement mark "ПЦ-400," C-Sureck molding flask) are shown. These relations were defined by the use of equation (1) with coefficients, determinated with the help of statistic treatment of empirical data.
\[ R_{com} = 10.8A + 12.0B + 13.6C - 2.4AB - 18.4A_1C + 3.2BC \]  
where \(0.5 \leq A \leq 0.6); (0.2 \leq B \leq 0.3); (0.2 \leq C \leq 0.3)\]

Figure 2 Compression strength (Mpa) – component parts relations of GCF binding

Table 1 Physical – mechanical data of cellular concretes

<table>
<thead>
<tr>
<th>Kind of ELC</th>
<th>Density, Kg/m³</th>
<th>Porosity, %</th>
<th>Compression strength, Mpa</th>
<th>Coefficient of heat conductivity, W/m.K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas cement concrete</td>
<td>200</td>
<td>93</td>
<td>0.11</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>90</td>
<td>0.19</td>
<td>0.069</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>88</td>
<td>0.25</td>
<td>0.075</td>
</tr>
<tr>
<td>Foam cement concrete</td>
<td>200</td>
<td>93</td>
<td>0.27</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>90</td>
<td>0.38</td>
<td>0.069</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>88</td>
<td>0.45</td>
<td>0.075</td>
</tr>
<tr>
<td>Foam GCF binding concrete</td>
<td>260</td>
<td>90</td>
<td>0.21</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>88</td>
<td>0.32</td>
<td>0.078</td>
</tr>
</tbody>
</table>

The results of laboratory investigations point out that the majority of tested GCF binding compositions, containing Surek-molding flask, can be regarded as the water-resisting materials \((K_{w,r} > 0.8)\).

The optimum binding composition using the above-mentioned criterions has A, B, C ratio 2:08:1:1 (on mass). The GCF binding test samples, possessing such a ratio, assume compression strength 11MPa at the age of 5 days and a quantity of \(K_{w,r}\) equal 0.82. In order to raise the tensile strength by bending, this GCF binding composition was modified by means of polymeric admixtures-carbamide resin and dispersion polyvinylacetate. As it was found out, polyvinylacetate dispersion admixture is more effective and also its optimum quantity makes up 5% of GCF binding mass (it ensures the rise of bending strength by 41%). As further increasing polyvinylacetate admixture content, the considerable diminution of compression strength (by
60 %) as well as bending strength (by 7.5 %) takes place. On the basis of elaborated GCF binding composition, the cellular foam concretes were made and researched (table 1). The relation compression strength-density of ELC with the use of GCF binding is shown in Figure 1 (curve 2).

One of the principal quantitative factors relating to the thermal insulation materials is the coefficient of heat conductivity \( \lambda \), therefore it is important to secure the process of ELC designing and composition development by the prognostication of their heat-conducting properties. The calculating heat conductivity value of two-ingredient (air + binding) cellular materials can be determined by consideration of a number of proposed heat conductivity models [1,2]. Regarding as an elementary accounting ELC cell the most simple two-ingredient model with closed air space of cubic form, there was received for ELC the following relation

\[
\lambda = \frac{\lambda_{\text{bin}} \lambda_{\text{air}}}{\lambda_{\text{air}} \left( 1 - 3 \sqrt{V_{\text{air}}} \right) + \lambda_{\text{bin}} \sqrt{3} V_{\text{air}}} + \lambda_{\text{bin}} \left( 1 - \sqrt{3} V_{\text{air}} \right) \quad (2)
\]

Where \( \lambda_{\text{bin}} \), \( \lambda_{\text{air}} \) – the coefficient \( \lambda \) of binding and air

\( V_{\text{air}} = 1 - \frac{\rho_{0}}{\rho} \) - material porosity

Assuming, that for unaustoclave cellular cement concrete the value \( \lambda \) of air by bore of pore to 2 mm is within the limits of 0.025-0.027 W/m. K and that of cement (for lack of pores) – 0.85 W/m. K, the equation (2) takes on an air:

\[
\lambda = \frac{0.0212 \left( 1 - \frac{\rho_{0}}{\rho} \right)^{2/3}}{0.025 \left( 1 - 3 \sqrt{1 - \frac{\rho_{0}}{\rho}} \right) + 0.85 \sqrt{3} \left( 1 - \frac{\rho_{0}}{\rho} \right)} + 0.85 \left[ 1 - \left( 1 - \frac{\rho_{0}}{\rho} \right)^{2/3} \right] \quad (3)
\]

According to our laboratory research coefficient \( \lambda \) of GCF binding without pore formation make up the mean quantity 0.75 W/m. K. The equation (2) for ELC on the basis of GCF binding assumes an air

\[
\lambda = \frac{0.0187 \left( 1 - \frac{\rho_{0}}{\rho} \right)^{2/3}}{0.025 \left( 1 - 3 \sqrt{1 - \frac{\rho_{0}}{\rho}} \right) + 0.75 \sqrt{3} \left( 1 - \frac{\rho_{0}}{\rho} \right)} + 0.75 \left[ 1 - \left( 1 - \frac{\rho_{0}}{\rho} \right)^{2/3} \right] \quad (4)
\]
By equation (3.4) $\lambda$ calculation values, were determined which have shown a quite good similarity with that of empirical findings for ELC with the use of different cement density. (Figure 4)

![Graph showing coefficient of heat conductivity of cellular concrete](image)

**Figure 4** Coefficient of heat conductivity of cellular concrete  
1- calculating $\lambda$ values of cement ELC; 2.- empirical $\lambda$ date of cement ELC; 3. calculating $\lambda$ values of ELC on the basis of GCF binding.

Presently the materials studied in this paper are being put into production at one of Penza precast. Their techno-economic efficiency and competitiveness with heat insulators made form expanded polystyrene and mineral wool are evident.

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