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Estudiante: Harald Bjørntvedt

Tutor: Enrique DAVID LLÁCER

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Fiber reinforced polymers used in constructions

Concrete is the most widely used building material in the world. In the last century there was much effort focused on improving the strength of this material for use in new building applications. However, up to know the focus was mainly concentrated on how to strengthen concrete with steel based reinforcement. Unfortunately, steel has a high density ratio and is very vulnerable to corrosion. In the last two decades especially, researchers have investigated the possibility of combining laminates with concrete. This combination has given engineers and architects new opportunities to make traditional concrete buildings even stronger and allowed for new and creative design concepts to flourish.

This first chapter is focused on two specific types of fiber reinforced polymers products which can be used to strengthen concrete members. The first one is fiber reinforced polymer sheet (FRP-sheets) than the second one is fiber reinforced polymer bars (FRP-bars). Both FRP-bars and FRP-sheets can be produced from continues glass (G) and carbon (C) which is adhered with an epoxy resin, polyesters or vinyl ester, and can therefore be distinguished as GFRP and CFRP products. However, in this article only CFRP-sheets, CFRP-bars and GFRP-bars will be presented.

CFRP (carbon fiber reinforced polymers) used to strengthen concrete is a well-known and researched material, it is used to strengthen concrete and it has already been used in many existing structures. With the development of CFRP's the versatility of concrete has greatly improved. It is characterized by having low density, high tensile strength, a high modulus of elasticity (usually lower than that of steel) and a high resistance to various corrosive factors. The tensile strength is typically 10 times higher than that of mild steel.

Having a lower modulus of elasticity causes a higher levels of deflection of the beam during loading compared to that of reinforced steel beam applications. CFRP's also exhibit a linear stress-strain response to failure. But CFRP's do not have the same yield-strain property as steel. Because of this there is an increased risk of suddenly failure without prior warning. It is therefore critical to have accurate mathematical equations to avoid brittle fracture failures.

Reinforcement of concrete with carbon-fiber sheets

Many older buildings suffer from concrete deterioration causing cracks in the structure that can lead to structural instability. Often in an attempt to fix this problem some cement grout has been injected into the cracks. This solution can lead to an unexpected result. The old and new concrete separate and fail to stabilize the existing cracks. There has also been an attempt to fix the problem using a combination of epoxy and steel connections. This solution has often resulted in the deterioration between the concrete and steel interface. Placing an interface between the CFRF sheet and the concrete prevents this type of deterioration. The technique is simple; it consist in wrapping a sheet around a concrete member. The sheet will seal the member and contribute to keep the alkaline level from dropping, thereby preventing the corrosion of the steel and the carbonating of the concrete.

Confining concrete with CFRP laminate gives the same advantages as applying steel spiral rebar for internal transverse reinforcement. But there is one fundamental difference between members confined with CFRP instead of steel. CFRP strengthens the whole cross section area and the composite material works elastically until failure. Stirrups, on the other hand, strengthen only the core of the cross-section. When steel has reached its yield strength, strains increase under the constant level of the load.

The usual technique used to strengthen concrete is to attach a carbon fiber sheet above the concrete surface with an epoxy resin in situ. Before applying the fabric, the surface of the cement should be cleaned to ensure a good bond between the concrete and the FRP. You can for example sand-blast the surface to remove the outer layer of cement [1]. Cleaning the surface of the outer layer of the cement is important in preventing the premature failure of the reinforced sheet. The CFRP sheet could be wrapped completely all around the beam. This procedure will in most cases give the best increased strength. Another common strengthening method used is U-jacketing; the fabric is wrapped around the underneath part and up on two sides of the beam. You can also apply the fabric just on two sides of the beam (side bonding). Which method you choose depends on the occurring forces in the structure. The different methods of how to strengthen a concrete member is showed in figure 1, [1].

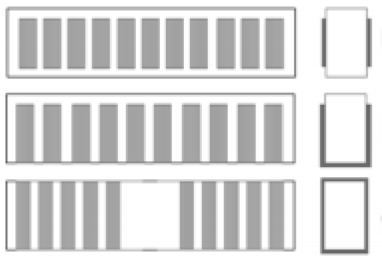

Figure 1, side-bonding, U-jacketing and complete wrapping

To calculate the strength of concrete reinforced with CFRP-sheets you need to know what the weakest link of the member is. Concrete reinforced with CFRP- sheets has more complex failure models than concrete reinforced with steel rebar.

In an article written [2] by Huy Binh Pham and Riadh Al-Mahaidi some of the different failure models of concrete reinforced with CFRP-sheets are discussed. Figure 2 [2] shows the different failure models you can expect to see. As shown, the strength of the member depends on many variable quantities, for instance how many layers of sheets are used, the strength of the sheet and the type of adhesive applied. The types of failure models we can expect to see are as follows:

- Mode 1 is called interfacial debond failure. This is one of the most commonly observed. The surface failure appears a few millimeters beneath the concrete-adhesive surface and could potentially dislodge a sliver or wedged section of concrete.
- Mode 2 shows tension failure, in which the main failure crack penetrates into the concrete from the FRP end.
- Mode 3 FRP shows tensile rupture.
- Mode 4 shows cohesion failure through the adhesive.
- Mode 5 shows FRP delamination.

The last three modes are rare because normal strength concrete usually has lower shear strength than that of the adhesive and FRP.

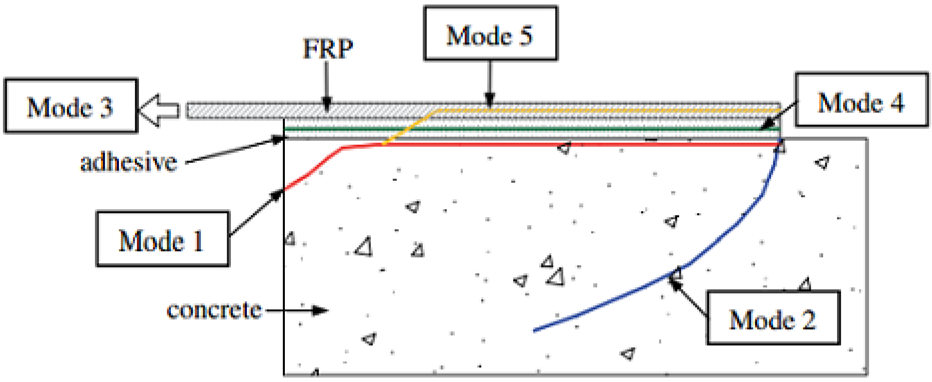

Figure 2, Failure models

Fiber orientation

Strengthening a concrete column implies the increase of the load capacity when it is done correctly. The fiber orientation within the sheet plays an important role when constructing a structure. The orientation should be done according to the axis orientation the forces acting into the structural element. Using a sheet with the wrong fiber orientation could contribute to early rupture thereby rendering the use of reinforced sheets ineffective. An article written by J. Piakarczyk, et al. [3] shows a study of a concrete column reinforced with CFRP-sheet. Their goal was to find out how different fiber orientation within the laminate, different sheets with different mechanical properties and different thickness of the laminate would influence the properties of a medium-strength concrete column.

It was notable that all the laminates that were tested exhibited properties of both higher tensile and modulus strength as opposed to those that had only had their cylinder surface wrapped. This type of application caused localized deformations due to the fracturing of the concrete cylinder.

When the axial stress load was relatively low, both the confined and unconfined columns behaved quite similarly. At the same stress level the unconfined concrete broke whereas the confined concrete was intact and able to carry more weight. The laminate supplied the lateral expansion of the concrete with compressive stress, causing cracks within the column. All the confined concrete systems behaved more ductile in comparison to the unconfined once.

Failure took place when the value of stresses loading the laminate (Lateral widening of the cylinder) exceeded the tensile strength of the laminate. After failure you could observe many small fragments in the central cylinder zones and solid fragments at the cylinder base. This indicates that the confining pressure was non-uniformly distributed. The failure occurred in the central part of the cylinder when the hoop rupture strains were reached.

The specimen with $0^{o}/90^{o}$ fiber orientation of the sheet gained the best increase in compressive strength. Half of the fibers were orientated in the hoop direction of the cylinder, and the other half orientated along the cylinder height. This gives us information that it is disadvantageous to have all the fibers orientated in the same way. In that case the cracks can easily propagate along the brittle epoxy matrix. A fabric with the fibers orientated in different directions will inhibit this splitting action. However, the column with the best compressive strength did not develop the best increase in axial strain. The specimen with 45°/45° fiber orientation developed the best increase in axial strain. A combination of 0°/90° and 45°/45° fabric could possibly give a column with high compressive strength and high fracture energy.

It was also interesting to find out how laminate thickness affects the axial compressive strength of confined concrete columns. The test showed that covering the concrete cylinder with M-300 type of laminate was effective up to a certain limit. Further reinforcement showed no significant increase in the effective strength. It should also be noticed that the calculated thickness for this level of strain was about 1.2 mm. This gives us a volumetric of M-300 laminate to concrete ratio of 5 %, while the usual volumetric is about 0.1-1 % for concrete strengthening.

Article [3] shows that using fabrics with fibers orientated in different ways results in a more favorable outcome. It makes the structure more ductile and not as vulnerable to early rupture of the fabric.

Only the results of round columns tested without the use of rebar were shown. The structures are often more complex in reality, that is, longitudinal rebar with stirrups attached to it, eccentricity of the force and so on. It is also known that it is it is easier to strengthen circular columns rather than quadrangular once. And it is more beneficial to have round corners but this is often very difficult to achieve. In most cases it is only possible to camphor a little of the corner. T. Trapko and M. Musial have also investigated concrete columns confined with CFRP-sheets and presented their results in an article [4]. They studied more complex columns (quadrangular containing rebar) and the load was applied with an eccentricity. The purpose for their research was to estimate the influence of changeable longitudinal CFRP intensity on the loadbearing capacity of the columns.

The columns in this test were applied with three different strengthening identities: 2.10%, 1.68% and 1.26% of CFRP in the longitudinal direction (figure 3 [4]). The strengthening identity was calculated as the ratio of the area of CFRP strip A_L to the area of concrete A_C. A single layer of transverse strengthening CFRP was applied around the concrete member. Only strengthening columns in the longitudinal direction of the height axis is disadvantageous; it will cause the debonding of the strips from the concrete. Transverse strengthening will also prevent premature delamination of the composite. The load during the test was applied on the end core cross-section of the concrete member with three different eccentricities of (0 mm, 16 mm and 32 mm).

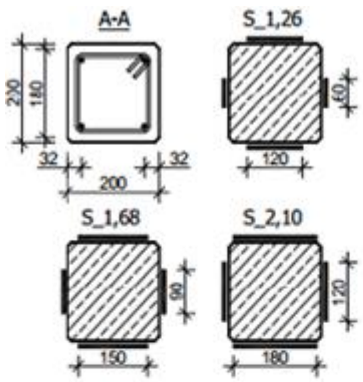

Figure 3, strengthening identities

The aim of the test was to calculate the load bearing capacity N_u of the column. The longitudinal strains $\varepsilon v2_{,lim}$ and $\varepsilon_{v1,lim}$ at the more compressed and less compressed side of the cross-section of the concrete were measured. This two values were used in an equation made to calculate the eccentricity e_1. This eccentricity is considered as accidental eccentricity and could be explained by the effect of non-homogenous concrete in the process of vertical loading. Moreover, it could also occur because of faults in load application and arrangement of the steel reinforcement.

All the confined columns experienced damage in the same way, by the rupture of CFRP at the edge of the column with the sound of carbon fibers fracturing. The rupture of the jacket occurred immediately when the columns first broke. Concrete underwent crush and fragmentation in the area of failure. All the confined columns experienced a much higher strength capacity compared to the unconfined ones. The columns got almost a constant and regular decrease of load bearing-capacity N_u as the eccentricity increased.

Applying the CFRP sheet on column had a very advantageous impact in reducing the undesired eccentricity. In other words, it improved the plastic properties of the column. The investigation showed us that unintended eccentricity decreased when the strengthening intensity increased.

In the last article [4] the authors mainly used longitudinal reinforcement to strengthen the columns. Longitudinal fiber orientation was more favorable on quadrangular columns with no significant rounding of the corners. The increase of load-capacity is mainly independent of the longitudinal reinforcement in quadrangular columns.

The two last articles I have presented in this paper concluded that it is disadvantageous to use fibers orientated in one direction only. The fibers should be supported with a unidirectional layer which will prevent premature delamination of the fabric and debonding of the composite from the concrete.

Mathematical equations to calculate shear strength and moment capacity of concrete members reinforced with CFRP-sheets

CFRP-sheets are commonly used to enhance the shear strength of concrete members. But because it`s a relatively new application in the civil engineering field, there exist no clear guides on how to calculate the shear strength of a reinforced concrete structure applied with the fabric. The existing guidelines provide a variety of numerical methods, but no one method is generally universally acknowledged. Several approaches can explain the behavior under stress, but no one behavior has been directly transformed into any formulas. Understanding how forces occur in a CFRP strengthened concrete is quite complex. The outcome of adding the CFRP fabric could reduce the amount of steel required in the structure. But on the other hand, the fabric could restrain the steel from reaching its maximum strength. Steel application can as well restrain the fabrics from reaching its maximum strength. An article written by A. Alzate et al. presents a study that was done in evaluating the different methods that can be used to calculate the shear strength of concrete reinforced with CFRP-sheets.

Many of the existing guidelines think of the shear strength as a sum of strength contributed by the concrete, inner tie bars and the fabric ($V=V_C+V_S+V_F$). But this equation is questionable because it is very difficult to get the complete strength advantage of all the materials at the same time. The strength provided by the rebar and the concrete can be calculated with aid of the Eurocode. It is more difficult to calculate the strength provided by the CFRP sheet. The suggested models struggled with two major problems. The first problem is the maximum stress withstood by the externally bonded sheet before failure. The second one is the problem of how the steel and the fabric interact with each other and how they might hinder each other from reaching their maximum strength.

The numerical methods based on the truss analogy approach have been proven to provide the best prediction of the shear strength contributed by the CFRP. Some of these models are evaluated in this article by analyzing 16 reinforced concrete beams which were loaded until failure.

As expected, all the confined beams developed higher strength than the unconfined ones. The increases ranged from 30 to 112 % depending on the amount of fiber reinforcement and application pattern. The failure of the U-jacked beams was a consequence of debonding, resulting in the detachment of the outer layer of concrete. The fully wrapped beams failed due to composite or bending failure. Shear played no role in this failure model.

No one of the investigated models which were evaluated in this experiment could predict accurately the failure load of the beam. Their results were widely scattered. In some cases, the equations even calculated a to maximum load capacity which could give a catastrophic result. The conclusion of the article is that more work needs to be done before reaching a reliable model to calculate shear strength.

There exist more better defined and known guidelines for calculating the bending strength of concrete members reinforced with CFRP-sheets, compared to the ultimate shear strength. In an article by IIker Fatif Kara, and F. Ashraf Ashour, [5] they presented a model which can predict a CFRP reinforced concrete's bending capacity quite well.

They collected data from previous experimental investigations to validate the proposed numerical method. All the beams which had been tested were reported to have failed in flexure, either by concrete crushing or FRP rupture. However, the ultimate load calculated with the presented method showed good agreement with the failure load experienced in the collected data.

It seems that more work needs to be done before we have mathematical equations to calculate the ultimate shear strength of concrete members confined with CFRP-sheet. The mathematical equations for calculating the ultimate bending capacity of concrete reinforced with CFRP-sheets at this time are better known.

FRP-bars in constructions

Steel has traditionally been used to reinforce concrete. As a construction material, it is functionally efficient and relatively cheap. However, steel is vulnerable to corrosion when it is exposed to salts, aggressive environments and moisture. This weakness causes costly repairs and maintenance. FRP rebar, on the other hand, does not suffer from deterioration caused by corrosion. It is considered immune against road salts and other deicing chemicals [6]. Although the costs of composites are generally higher than steel, considering the lifecycle costs (LCC) it could be economically more favorable to use FRP rebar instead of steel. It is more durable and needs less maintenance in the long run. Also, composite rebar weighs about one quarter that of comparably performing steel [6], which will result in less injury to the workers. In addition, FRP-laminates are inherently nonconductive; they will not transmit an electrical current. This makes it a much safer choice for applications near electrical devices such as power plants.

FRP-bars are characterized with many of the same properties as FRP-laminates, such as their elastic behavior until failure. The stress-strain relationship of GFRP, CFRP and steel rebar is shown in figure 4 [7]. It should be mentioned that only the characteristic properties of one type FRP-bars and steel are shown, but not the general stress-strain relationship. FRP-bars do suffer from creep rupture (Failure due to bearing loss capacity, loss under long term loadings) compared to steel [7]. FRP-bars should not be bent after the resin is cured due to the inflexibility or rigid nature of the FRP-bars. One main difference also worth mentioning between materials with reinforced FRP-bars compared to that of steel reinforcement is that they behave less ductile [7].

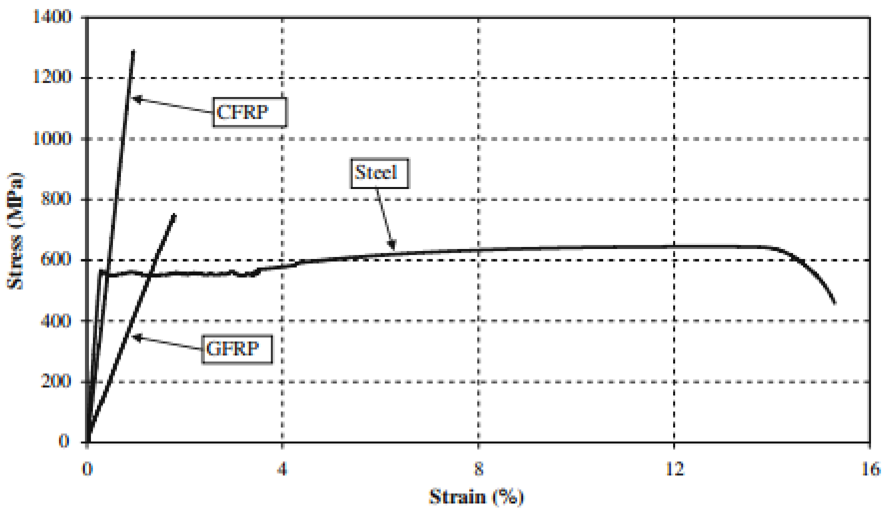

Figure 4

FRP-bars can be used as a replacement for traditional steel rebar. However it could also be mounted near the surface of the concrete structures. In an article [8] written by Laura de Lorenzis and Antonio Nanni they give us information about FRP-bars mounted near the surface of concrete members. The technology could, for instance, be used to strengthen the flexural capacity of deficiently reinforced concrete members. It can also give the possibility of anchoring the FRP-bars into adjacent reinforced concrete members. The method used in applying the rods is to first cut a little groove into the concrete surface. The groove is then filled half-way with epoxy paste before the FRP-rod is placed in the groove and lightly pressed into the groove. The groove is then filled with more cement grout before it is leveled.

Design guidelines for FRP reinforced concrete structures

Composite rebar got its start in Japan in the 1980s. However, it didn`t really take off before specifications were developed and published in the late 1990s [6]. American Concrete Institute`s Committee 440 gave out their first guideline in 1999. The guideline was updated in 2006 and 2012. Today a couple of hundred bridges in Canada and the U.S. have FRP-bars incorporated into some aspect of their construction. Floodway Bridge over the Red River in Winnipeg used FRP-bars in all the concrete members above the girders [6]. The bridge was completed in 2006 and used a total of 140 600 kg of GFRP-rebar. The bridge is shown in figure 5 [6].

Figure 5

As mentioned earlier, lack of formal design standards have been a barrier for extensive use of FRP-bars. A paper [9] written by Kypros Pilakouta et al. presents the general philosophy underlying the design of reinforced concrete elements reinforced with FRP and also discusses the various other important design issues. The design guidelines they present are based on already existing Eurocodes for steel reinforcement. As they have written in their article: "*There are no fundamental reasons why the principles behind the certification of SLS (service limit states) for FRP RC elements are not similar to those ready established in the codes of practice for steel RC elements*".

One of the challenges with the calculations when adopting the framework of Eurocodes is that the mechanical properties of FRP-bars are different from steel reinforcement. FRP-bars offers less rigidity than steel and their elastic modulus could be as low as 30 000 N/mm^2. However, the strain limits developed for steel rebar in the Eurocode can be adjusted to fit the mechanical properties of FRP rebar. The adjusted equations can then be used for calculating the strength of concrete reinforced with FRP-rebar. By adopting the framework of the Eurocode we also need to consider stresses in the materials. We can, for instance, expect to have a much larger deflection in a FRP reinforced concrete member because FRP`s have a low elastic module. This larger increase of deflections will result in more wide cracks appearing in the concrete.

The anchoring process of the FRP rebar could also be more difficult due to the high strains developed in the tensile reinforcement bars. Another fundamental difference between calculating the steel reinforcement and FRP reinforcement is the approach. The FRP reinforced concrete structures will be over-reinforced and that ultimate failure will be concrete crushing rather than by reinforcement failure. It is notable that steel becomes over-reinforced if the percentage amount of reinforcement p_f is around 3.2 %. In the case of GFRP and CFRP they remain over-reinforced for p_f above 0.5 % (assuming strength of 850 and 1350 MPa for GFRP and CFRP). For ratios below 0.5 rupture of the rebar occurs, depending on the strength of the FRP.

Because of the lower elastic modulus of FRP bars compared to steel, larger areas of reinforcement are required to achieve the same moment capacity whiteout presenting higher deformations of the structure, due to their lower elastic modulus. Because the strength of the FRP-bars is not utilized it is meaningless to use a FRP partial safety factor. Limitation on FRP stress is more complex and important for steel due to cracking of the resin and stress corrosion (of glass fibers).

Their article also presents some mathematical equations to calculate the flexural strength to a FRP-reinforced concrete member. In addition, it reveals a few equations that could be used to calculate the shear capacity. All the design approaches assume that the shear capacity could be expressed in terms of a concrete contribution and additional contribution from the FRP-bars.

We can experience bond-slip behavior between the FRP-bars and the concrete. There is a fundamental difference between the interaction for FRP-bars and concrete compared to steel rebar and concrete. The bond slip behavior in steel reinforced concrete occurs by crushing of the concrete in the vicinity of the lugs. The bond slip behavior of FRP rebar occur because of partial failure of the concrete surrounding the FRP rebar and as well as damage within the surface of the FRP-rebar.

Flexural behavior of concrete beams reinforced with FRP-bars

The paper [7] written by Zorislav Soric et al. presents a study done on the flexural behavior of concrete beams reinforced with FRP-bars. They were interested in exploring the mechanical properties, bond-slip behavior of FRP-bars within the concrete, and the force-deflection behavior of concrete beams reinforced with GFRP, CFRP and steel bars up to failure.

The concrete carried the entire load until the first cracks appeared regardless of the type of reinforcement that was used to strengthen the concrete. Beams reinforced with FRP-bars failed after shear cracks propagated from one of the span`s causing the compression zone of the concrete to crush. Immediately after that, the reinforcement failed. The failure was followed by a strong banging sound.

The members confined with FRP-bars only developed half the deflection compared to steel bars before failure. The beam’s load capacity increased as well. The concrete confined with GFRP and CFRP had strength of between 2.4 and 2.75 times greater than that of the ones with steel reinforcement.

They also presented mathematical expressions which describe how FRP reinforced beams behave during loading. These expressions were used to describe the behavior of the reinforcement between two adjacent cracks of the concrete member. This information was then used to calculate the bond-slip behavior of the reinforcement, the appearance of new cracks and the softening of the concrete between two adjacent cracks. The proposed equations were able to predict the type of behavior the FRP reinforced concrete experienced during the test. The bond stress and the slip of the reinforcement, as well as the appearance of new cracks, showed good agreement with the mathematically proposed methods.

High strength concrete

The American Concrete Institute defines high-strength concrete as concrete with specified compressive strength f_c` of at least 8000 psi (55) MPa. The strength quality of the ordinary concrete used in most part of structures is between C 15 to C C65 MPa [10]. As it was reported, in the last few decades we have seen concrete investigated on a large scale. All these investigations have provided us new information about how to make traditional concrete even stronger. The construction is currently possible with super high strength concrete with a compressive strength greater than 200 MPa in its constructed environment [11], what suppose more than four times the strength of regular concrete.

Using high performance concrete in structures (HPC) is not a completely new idea. HPC has already been used to build more durable structures in aggressive environments such as marine structures, highway bridges, nuclear structures, tunnels, etc. Higher strength concrete allows us to increase the available space by reducing column sizes. In other words it can allow us to build the same buildings with a decreased material consumption. Another great property of some types of high strength concrete is that we can put the concrete into service at earlier an age.

Factors that influence concrete`s strength

Water-Cement (W/C) ratio has a huge impact on concrete`s strength. Traditional concrete have W/C ratio higher than 0.40. This is the bottom level you need to get all the cement-paste to react in traditional concrete mixture. A higher W/C ratio will lead to more voids in the concrete with a weaker concrete as a result. But on the other hand, it will be more workable. It is therefore important in HPC to keep the W/C ratio as low as possible to prevent unnecessary voids.

A study done by J Baroninsh et al. [12] investigates how the influence of the polycarboxylates based super plasticizer (SP) (high-range water reducer) influenced the strength of HPC. In the article they tested concrete with different SP ratios and W/C ratios. The SP is added to obtain the workability of the concrete when the W/C ratio decreases. The strength of the concrete increased when the W/C-ratio decreased. The volume of porosity decreased in the concrete when the W/C-ratio dropped. And the workability of the concrete was also best in the test with lowest water ratio (W/C was 0.28). This was because this test was the one with the most SP-liquid (2.5 % cement weight). The use of SP-liquid can make concrete stronger because it allows us to reduce the water-cement ratio.

Another investigation done by Rukzon and Chindaprasirt [13] has looked at how bagasse ash could be used to strengthen concrete. Bagasse ash (BA) could be used as a pozzolan which contains a property allowing it to react with the calcium hydroxide produced through cement hydration. The result of their reaction is additional gels which contribute to higher strength of the concrete. They replaced Portland cement with 10 %, 20 % or 30 %. All the specimens with BA developed better strength than the ones without. But the increase in compressive strength was not that significant. Of these the one with a 10 % BA developed the best strength. The strength was increased with a variation of 102-107 % depending on when it was tested (7, 28 or 90 days after production).

A method described in an article [14] by Joachim Seehusen shows how the way to produce prefabricated high strength concrete. The idea is to cast the concrete in a round framework and then spin it around its longitudinal axes. The fresh concrete get pushed to the sides of the frame equal to a force of 20 G, that is about 600-900 Revolutions per minute. The revolutions produce a hollow in the middle of the columns. A picture of the rotation process of the column is shown in figure 6 [14]. The concrete gains much higher strength during this process and will be almost free of voids. This will also increase the durability of the column. A company in Germany named “Euro Poles” produces these columns with strength C 140 [15].

Figure 6

It is currently also possible to produce high-strength-concrete with lightweight aggregates. In an article [14] written by W. C. Tand et al. he presented a study on the fracture properties of high strength concrete (HSC) with normal aggregate (crushed limestone) and lightweight aggregate. The fracture properties that was of interest in this article was the critical stress factor (K_{IC}^{s}), the fracture energy (G_F) and the critical crack tip opening displacement($CTOD_c$). The concretes were mixed with almost the same recipe, except that the lightweight concrete added sintered fly ash instead of limestone. The silica fume replacement of concrete was 10 % (S/(C+S)). This was about the same level of replacement as Rukzon and Chindaprasirt discovered in their investigation [13] that would give the concrete mixture the best compressive increase in strength. The lightweight concrete mix weighed 469 Kg/m^3 less than the regular concrete which weighed 2390 Kg/m^3.

The experimental results are shown in table 1 [14]. The compressive concrete strength of the lightweight HSC is about 80 % of the normal aggregate HSC. This decrease is similar to the one that is found in the density. The splitting tensile strength of the lightweight HSC is reduced to 75 %. The reduction in the dynamic module was expected because of the dynamic module of the sintered fly as aggregate is only about one third of the limestone aggregate. Moreover, the failure model of the lightweight HSC in both compression and split tensile test behaved more ductile.

CONCRETE:	DENSITY: KG/M^3	COMPRESSIVE STRENGTH: MPA	SPLITTING TENSILE STRENGTH: MPA	DYNAMIC MODULUS OF ELASTICITY: GPA
NORMAL-WEIGHT HSC	2430	115	4..5	49
LIGHTWEIGHT HSC	2013	90	3.4	29
RELATIVE RATIO (LW/NW)	0.83	0.80	0.75	0.59

Table 1, Basic material properties at 90 days after casting

The fracture toughness of the material (K_{IC}^{s}) was quite similar for both light-weight HSC and normal HSC. The difference was around the same as we can expect between different specimens of normal concrete.

The critical crack tip opening displacement $CTOD_c$ was higher in the lightweight HSC. This might be related to higher tensile strain capacity and higher strength/weight ratio characteristics owing to air voids in the lightweight aggregate.

The fracture energy values (G_F) of the lightweight aggregate HSC are comparable to those of the normal HSC. The lightweight HSC behaved more ductile than the normal HSC. The difference between the elastic modulus of the cement mortar and the lightweight aggregate is large. As a result of this high stress concentrations can be found on the matrix-aggregate interface. The difference between the elastic modulus of the normal-aggregate and the cement mortar is less. The structural behavior of normal HSC is quite similar to a homogenous material. Because of the high stress concentrations within the lightweight HSC, cracks propagate around rather than across the aggregate as it occurs for the normal HSC. This results in a more ductile failure module and more non-linear behavior.

As a short conclusion it can be proved that it is possible to get high strength concrete with lower density. However, the mechanical properties will not be dramatically changed compared to that of traditional HSC.

Ultra high performance concrete

Ultra high performance concrete is a specific type of concrete with compressive strength higher than 150 N/mm^2 [15]. Through many years UHPFs have been researched resulting in some general guidelines of how UHPC could be produced:

- Enhancement of homogeneity by eliminating the coarse aggregate. Coarse aggregate and matrix are often the source of micro cracks in the concrete due to their different mechanical and physical properties [15].

- Enhance the packing density of the powder mixture. This means powder mixture should be composed of number classes of granular powder [15].

- Application of pressure before and during setting of the concrete mixture [16].

- Enhancement of the microstructure by post-set-heat-treating [16].

- Enhancement of ductility by incorporating small steel fibers. Makes it possible to obtain the required level of ductility [16].

An article [16] written by Pierre Richard, and Marchel Cheyrezy describes how they succeeded to produce concrete, with compressive strengths greater than 800 MPa. The concrete was designated RPC800 (Reactive Powder Concrete) and is suitable for pre-casting.

The concrete was made by an optimized selection of granular components. To get a high density, the fresh concrete was subjected to high pressure (50 MPa). This phase considerably reduces air bubbles within a few seconds. After the concrete has set, the concrete gets heated in ambient pressure. The RPC800 was heated with temperature variation from 250 to 400^{0}C.

The compressive strength using quartz sand as aggregate was about 490 to 680 MPa, by adding steel fibers and steel aggregate the result varied from 650 to 810 MPA. In addition to the high strength the specimens developed, they also developed an ultra-dense microstructure. This gives us excellent waterproofing and durability characteristics. This type of concrete could perhaps be used for industrial and nuclear storage.

The UHPC described above can only be used in pre-casting because of its production procedure. It is also possible to increase the strength of concrete whiteout using pressure curing and heat treatment.

Pre-casting is often more expensive than casting on-site. An article [17] by Kay Willie et al. describes a study of ultra-high performance concrete fiber reinforced concrete (UHPC-FRP) which could be casted on-site. The main goal of their research was to design a UHPC-FRP with more than 200 MPa in compressive strength, more than 10 MPa in post-cracking tensile strength and associated post-cracking strain of at least 0.3%.

The concrete was made by optimizing the packing density, by adding high strength steel fibers. The steel fibers were adapted in different shapes to improve the mechanical bond within the concrete. The best concrete components available on the US market were selected. The selection was based on investigations by others. However, nothing revolutionary was used to make this concrete. It was made of cement, pozzolanic reactive material (silica fume) and filler (glass powder), fine sand as aggregate and super-plasticizer.

The optimization of the mixture's components and properties were based on a flow cone test. The goal was to maintain the same level of flow ability while reducing the water. This phase increased the packing density and reduced the water voids within the concrete. By reducing the W/C ratio from 0.30 to 0.17 while maintaining the flow-ability resulted in increased strength from 122 MPa to 177MPa. The mixture without silica fumes (SF) had strengths of 158 MPa. By adding the silica fume the strength increased to 229 MPa. The third step was replacement of SF and cement with glass powder. The strength increased from 221 MPa to 240 MPa. Another thing that was quite positive was by optimizing the mixture, the added constituents lead to a substantial reduction of super-plasticizer (only about 1 % of cement weight is what was then needed). The UHPC developed in this article got a maximum compressive strength of 246 MPa.

High strength steel fibers were also added to some test specimens. The highest compressive strength developed for UHPC-FRP specimen was 291 MPa (by using 8 % volume fraction of fibers). However, it increased other properties as well compared to the ones without fibers. By using 8 % volume fraction of steel fibers, the strain-peak stress increased up to 1.1 %.

However, the test also showed us that it was possible to get post-cracking tensile strength of 13 MPa by simply adding a 1.5 % volume of deformed steel fibers. The tensile strain-peak developed was about 0.6 %. This level is about two times the allowed yield stress strain in commonly used steel reinforcement.

An article [15] written by D.B. Zadeh et al. presents the mechanical properties and autogenous shrinkage of UHPC.

Their concrete mixture used quartz sand with the size of 0.3-0.8 mm as aggregate. A quartz powder with a diameter smaller than 10 µm was used as micro filler. The powder was added to fill the space between the cement particle and the silica fume.

UHPC is characterized by the great amount of cement content, about 950 kg/m^2. But due to the low W/C ratio, only a part of the amount of the cement has hydrated. Un-hydrated aggregate lies within the concrete mixture as fine aggregate. In this mixture cement was step by step replaced by inert quartz powder. The results showed us that even if 30 % of the cement were replaced, the compressive strength did not suffer (table 2 [15]). Another positive consequence was that the flow-ability improved. This could be explained because adding the fine quartz powder reduced the voids in the mixture.

TEST NUMBER:	CEMENT:	SILICA CEMENT: (%)	CEMENT REPLACEMENT: (%)	W/C	COMPRESSIVE STRENGTH: (28D/20°C)	COMPRESSIVE STRENGTH: (14D/20°C + 3D/90°C)
1	950	25	0	0.187	149.2	189.7
2	893	26.6	6	0.199	141.6	195.5
3	836	28.4	12	0.213	145.9	186.3
4	779	30.5	18	0.229	115.1	177.5
5	722	32.9	24	0.247	143.0	190.1
6	665	35,7	30	0.268	148.1	201.7

Table 2, all of this test specimens were casted without vibration,

The minimum and maximum load in the load-unload cycles was 5 N/mm^2 and 65 % of its compressive strength. After the first cycle, the loading and unloading curves were almost identical. This result indicates that only a few new micro cracks were generated during the loading cycles. The measured Poisson`s ratio was about 0.18 which is quite low. This could be explained by the strong bond between the fine quartz sand and matrix. However, after the load exceeded 70 % of the compressive strength, the Poisson's ratio rose abruptly. This indicates a quick propagation of micro cracks.

Autogenous shrinkage was generally higher in the UHPC than conventional HPC. That is a bit of a setback because it can contribute to the development of micro cracks at an earlier age within the concrete.

However, article [15] show us that it is possible to produce concrete with a strength greater than 200 MPa under normal curing processes by reducing the air content less than 1 %. The Elastic module is also quite different from what we find in conventional concrete. In this case, a new empirical equations for UHPC need to be developed.

Self-compacting concrete

To build durable concrete structures with conventional concrete it is needed proper compacting by skilled workers. Because of the lack of skilled workers the investigation and interest in to self-compacting concrete (SCC) has been of great importance in these last three decades [18]. The utilization of SCC will also give better working conditions to the workers. Without the need for vibrating the concrete, the damaging health effects suffered by workers will be reduced. SCC can also be more cost effective because it can shorten the total construction time. Analysis done on the anchorages of Akashi Straits (world's longest span bridge) concluded that the usage of SCC had shortened the construction process from 2.5 to 2 years [18].

The idea of self-compacting concrete is that it should be possible to compact it into every corner of the framework, purely by means of its own weight and without any need for vibration or compacting [18]. Conventional buildings are currently constructed in a way so it is possible to vibrate the concrete. Without need of vibration, new construction methods could be developed that could provide us new possibilities on how to build smarter, more creative and cost effective structures.

General facts about self-compacting concrete

When segregation of concrete occurs it becomes weak and unstable which indeed is a very costly, highly dangerous and unfavorable outcome. One of the most important basic principles of how to avoid segregation of the concrete, are use of super-plasticizer combined with relatively high content of powder materials in term of Portland cement, mineral additions, ground filler and/or very find sand [19]. A partial replacement of Portland cement by fly ash is also favorable, because it will better the compressive strength and decrease risk of segregation [19].

W/C ratio is well known to have a great influence on the flowing property of SCC. But with increasing the W/C ratio, the mechanical properties get weaker. To make a SCC without a higher W/C ratio we use super-plasticizer which increases the flowing energy of the concrete.

Research has shown that the energy required for flowing gets consumed by increased internal stress. Coarse aggregate works like a barricade to the aggregate particles. Its internal stress consumption is also particularly intense [18]. The amount of coarse aggregate is therefore often limited in SCC.

New Applications in self-compacting concrete

An article written by Ali Nazari and Shadi Riahi [20] reports the research in concrete incorporated with TiO_2 nanoparticles. The aim of their work was to investigate the compressive strength and water permeability of high strength self-compacting concrete. TiO_2 nanoparticles were added to combine with the liberated lime during the process of hydration.

Adding Tio_2 nanoparticles up to 4.0 wt% increased the strength of the concrete, adding more resulted in decreasing compressive strength. This could be explained because it is disadvantageous to add more Tio_2 to the mixture than what is needed to combine the liberated glue from the hydration process. Thus the excess of silica leaching out causes decreased strength as it replaces part of the cementitious material. In case of flexural strength and split tensile strength, the concrete which contained 4wt% TiO_2 nanoparticles performed the best ratio. Tio_2 also affected the concrete to develop strength faster due to faster consumption of $Ca(OH)_2$.

The water permeability was also improved (after 7 days of curing) by adding TiO_2 nanoparticles. At 2 days of curing, the water consumption was greatest, since the particles require more water due to the rapid development of hydrated products. The microstructure of the concrete was increased by adding TiO_2. It works like filler enhancing the density of the concrete.

Properties of self-compacting concrete

An investigation conducted by A. Leeman and C. Hoffman, [21] looked at the properties of SCC. The purpose of their study was to compare the physical properties of SCC and conventional self-compacting concrete (CVC). All their test specimens were made with different mixture to check how the constituents influenced the properties of the concrete.

All the test specimens developed compressive strength greater than 44 MPa. The SCC and the CVC showed similar values when they had the same w/b- ratio. There were no significant differences in the flexural strength either. But it should be mentioned that the SCC developed a relatively high standard deviation. Also the oxygen permeability coefficient of any given strength was the same even though the SCC was added more past. This could be explained by the effect introduced by fly ash changing the microstructure of the cement. Adding fly ash to the mixture makes a denser pore structure than plain cement does. Only the SCC was added fly ash, this could explain why we did not get higher oxygen permeability in the SCC than the CVC. But this does not apply all types of fly ash, because some types may contribute to a higher level of porosity.

However, the SCC displayed lower value of E-modulus, higher shrinkage and higher creep rate at identical compressive strengths. The lower E-modulus (about 16 %) could be explained because of the larger amount of paste. Consequently the relation between compressive strength and E-modulus of SCC and CVC was the same when the grain size of the aggregate and volume of paste were identical. Increasing the use of super-plasticizer resulted in a large gradient. The maximum grain size of the aggregate also influenced that relationship. The shrinkage and creep rates were higher (about 30 %) of the SCC compared to the CVC. The difference seemed to be linear with the increase of paste. So the high paste volumes seem to be the ultimate reason of the higher shrinkage and creep rate.

The investigations done in the article mentioned above describe the properties of a medium strength concrete. An article written by S. Assiè et al. [22] presents a study of the durability of low-resistance self-compacting compared to vibrated concrete given the same strength.

The concrete specimens tested in their investigation were made with almost the same recipe. The mixes were made with the same components in similar proportions. The only difference between the components was that the SCC was made with limestone filler. The SCC were also added more super-plasticizer, paste (380 compared to 310 l/m^3) and had higher w/c ratio (0.65 compared to 0.60).

The SCC specimens had a greater compressive strength (21 %) than the VC. This could be explained by the use of limestone filler and greater quantity of super-plasticizer which could reduce the air voids in the concrete. However, the SCC experienced a decrease of elasticity modulus (-5 %). The SCC`s higher percentage by volume of paste could explain these effect.

In case of durability properties, the SCC`s oxygen permeability is lower than for VC. There was no significant difference in case of Chloride diffusion, carbonation and ammoniums nitrate leaching either. In other words, the durability of SCC in this test can be regarded as equivalent to the VC.

It could be interesting to look why we got almost the same properties of SCC and VC. The article explains a reason for every durability property. For instance, why do the two different types of concrete have similar value of Chloride diffusion when SCC has a denser microstructure? The test showed that having the denser microstructures did not result in better resistance against chloride permeability. This proves that that the transport properties depend also on other material properties such as tortuosity or interconnection network.

The conclusion of this article is that the investigated SCC was durable and did not suffer from any durability problems.

High-strength lightweight self-compacting concrete

An article written by Y.W. Choi et al. [23] describes research done on the fluidity and mechanical properties of high-strength lightweight self-compacting concrete. The goal of their investigation was to design a lightweight concrete with good viscosity in its fresh condition and a compressive strength greater than 35 MPa.

Both lightweight artificial fine and coarse aggregate were used within the concrete mixture. Natural fine and coarse aggregate were used as well. Using conventional mixing methods would give a rise in the material segregation and lower compressive strength due to the lightweight aggregate. To avoid such problems a method to mix high-performance self-compacting concrete was used. The mixing process of concrete was proportioned with various volume constituents. The different mixtures recipes are given in the table 3 [23].

GROUP NO.	MIX. NO.	PF	S/A (%)	W/C (%)	$\frac{LC}{(LC+NC)}$ (%)	$\frac{LF}{(LF+NF)}$ (%)	UNIT WEIGHT (KG/M³)					
							W	C	NC	LC	NF	LF
A	1	1.18	53	38	0	0	175	460	810	0	861	0
B	2	1.18	53	38	25	0	175	460	608	117	861	0
	3	1.18	53	38	50	0	175	460	405	234	861	0
	4	1.18	53	38	75	0	175	460	203	352	861	0
	5	1.18	53	38	100	0	175	460	0	469	861	0
C	6	1.18	53	38	0	25	175	460	810	0	645	158
	7	1.18	53	38	0	50	175	460	810	0	430	316
	8	1.18	53	38	0	75	175	460	810	0	215	473
	9	1.18	53	38	0	100	175	460	810	0	0	631
D	10	1.18	53	38	75	50	175	460	201	353	433	315
	11	1.18	53	38	75	75	175	460	201	353	217	773

Table 3, Mixture proportions of concrete

Increasing the amount of lightweight fine aggregate (LF) resulted in higher viscosity of the mixture. Use of more than 50 %LF did not satisfy the requirements of the V-funnel test.

The decrease in compressive strength was not linear as the volume of lightweight aggregate increased. The results of their investigation are given in table 4 [23]. With a use of 75 % lightweight coarse (LC) aggregate, the strength only decreased with 6 % compared to mix no 1 (control concrete). It should also be mentioned that the concrete with LF aggregate with more than 50 % had 8-20 % increased strength compared to the once with less than 50 % LF aggregate. It should also be noticed that the lightweight coarse aggregate has a 63 % higher crushing ratio than natural coarse aggregate and it also has a 28.09 % higher absorption rate than the NC. Why we get a higher strength when the LF aggregate increases are due to the fact that more micro particles are produced which result in a decrease of voids in the concrete. Mix no 9 was 100 % LF and it showed an increased strength of 33% compared to that of the controlled concrete

GROUP NO.	MIX NO.	DENSITY (KG/M^3)	COMPRESSIVE STRENGTH (MPA)	STRUCTURAL EFFICIENCY $(\times \mathbf{10^{-3}}\ \boldsymbol{MPa \bullet m^3/kg})$
A	1	2306	49	21.2
B	2	2221	46	20.7
	3	2135	45	21.1
	4	2051	46	22.8
	5	1965	34	17.0
C	6	2248	45	20.0
	7	2191	46	21.0
	8	2133	53	24.8
	9	2076	59	28.3

Table 3, calculations of structural efficiency

In case of the elasticity modulus all the specimens satisfied the demands of the ACI standards. The range of the elastic modulus of the LC and LF was approximately 71-81 % and 88-97% that of normal concrete.

The article also presents the results of the tested structural efficiency $(MPa \times \frac{m^3}{kg})$. The test specimens with 75% and 100 % LF improved the values of structural efficiency compared to that of conventional concrete. At the end of the article they concluded by providing us with some numerical equations which could estimate an expected result. The values were within 10 % of the differences for both the compressive and structural efficiency.

FRP rebar

Steel has been used as a complement to concretes limited tensile strength through many years. However, insufficient concrete cover, poor design or workmanship and exposure to the environment can lead to excessive cracking of the concrete and therefor corrosion of the steel rebar. The Federal Highway Administration (FHA) in the U.S. reported that in 2004 [24] 162 869(27.5 percent) of U.S. bridges are deficient due to either structural or functional causes. Of these there are 81 304 that are structurally deficient and 81 565 that are classified as functionally obsolete. In the last decade the utilization of FRP-materials has mainly been used to repair many of these functionally obsolete bridges bringing them up to compliance with today's safety standards.

Hall`s Harbor wharf

Figure 7 [25], Picture of Hall`s Harbor after rehabilitation

A paper [25] written by John P. Newhook presented the detail of the Hall`s Harbor Wharf Rehabilitation Project where GFRP reinforcement was used in the concrete deck and pile cap of the wharf. The predominant structure is the outer timber crib/stone breakwater originally constructed in 1904. The structure is built in a region of Canada which has the highest tidal range in the world at 14 m. The structure is also exposed to temperature ranges from between -35 and 35 degrees and frequently splashes of salt water.

In this extremely aggressive environment, the existing timber structure required major repair work every 30-40 year. In 1998 the, the middle section of the wharf collapsed during a winter storm. Building a new structure was proposed to avoid further costly repairs. This resulted in a construction project where 40 meter of the 120 m long section of breakwater was replaced with a precast steel-free concrete deck system.

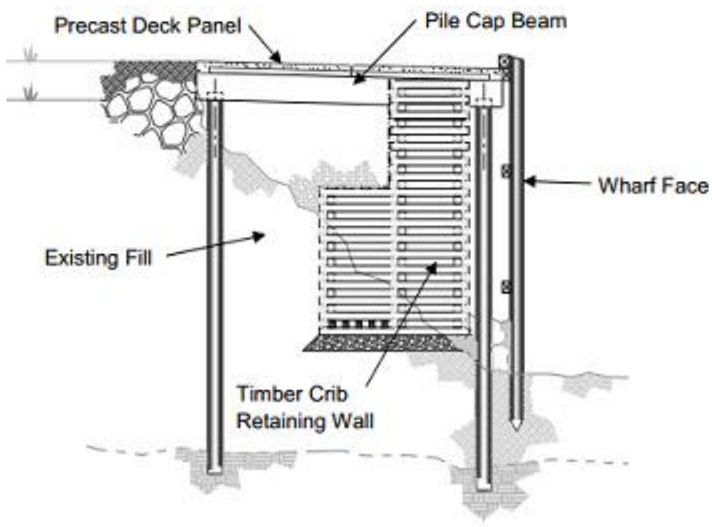

Figure 8

First of all, the collapsed rubble from the existing wharf was removed and new timber crib retaining was built to stabilize the earth backfill (showed in figure 8 [25]). The new concrete was then casted over the remnants of the existing structure to achieve the planned increase in height. The pile caps were supported on concrete-filled steel pipe piles. An important criteria in building this new structure, was that the stability of the structure should not be affected by any part by the old wharf. This was avoided by building the new section in a C-shape (seen in figure 7).

The basic design of the concrete deck was based on systems originally designed for highway bridges whit some new modifications. The design of the deck relied on enhancement of internal compressive forces and uplift forces (due to the arching action). The absence of internal steel reinforcement eliminated the corrosion problems and thereby greatly improved the longevity and the durability of the structure.

The external steel straps were encased in grout-filled PVC tubes. A high performance concrete with Portland cement, silica fume and fly ash mix was used. In addition a 2% volume of short synthetic polymer fibers was also added to the mixture.

The edges of concrete slabs are highly vulnerable to breakage and chipping. In order to avoid failure in this critical zone the edges of the panels were made thickened and reinforced with FRP-rebar consisting of 3 – 15 mm diameter in the bottom of the thickened sections of slab. The decks panels of the structure were precast and they were fastened with the pile caps. GFRP-bars were added to the top of the deck to provide strength against the flexure forces from the uplift forces. Near the back of the wharf, 15 mm FRP-bars added with spacing of 300 mm. In the front of the wharf the spacing was just 150 mm due to the greater uplift forces. In the longitudinal direction, only 3 -13 mm were used as positioning bars. The pile cap was also reinforced (seen in figure 9 [25]) to resist uplift forces. The outer layer of piles consisted of 5 – 25 mm GFRP bars with a concrete cover of 40 mm. The inner layer consisted of 4-25 mm steel bars with an 80 mm concrete cover. The hybrid reinforcing design was selected as a compromise between several of the design criteria CHBDC (Canadian Highway Bridge Design Code) required and the desire to advance the use of GFRP as a primary reinforcement application. For FRP reinforcement in general, CHBDC specifies that the section should be over-reinforced. To over-reinforce the pile cap in accordance with CHBDC, at least twenty-five 25 mm diameter FRP-bars would be required. That solution was neither constructible nor economical.

Even though the maximum theoretical service stress load in the GFRP-bars was only 14 % under all-loading, additional steel reinforcement were selected to meet the standards minimum reinforcement required. The steel reinforcement also provided a safeguard against collapse failure.

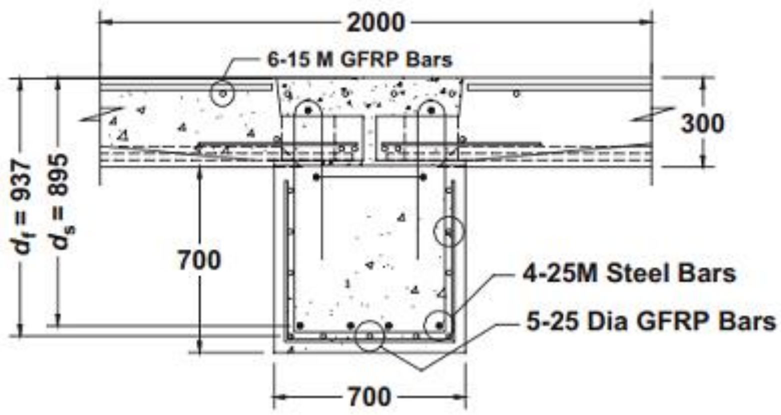

Figure 9

Hall`s Harbor Warf was the first marine structure in Canada build with GFRP-reinforced concrete. An article written by Aftab Mufti et al. [26] present a durability test done on Hall`s Harbor five years after it was built.

It has been a discussion whether the GFRP reinforcement could be used to replace steel reinforcement. Several published articles have concluded that GFRP should not be positioned in contact with concrete. However, the subjecting of the GFRP was carried out using idealized simulated conditions. Having a high-pH fluid environment often involves having higher temperatures. The laboratory test could be considered to be unrealistically harsh on the GFRP-bars. A field expedition was executed to check the durability of the GFRP reinforced concrete. Five structures were selected for this study, among them was the five year old Hall`s Harbor Wharf. Concrete cores containing GFRP were removed from the structures and then tested.

There was no degradation of the GFRP in the structure in any of the five tested structures. The test showed up that actual engineering structures were not in agreement with the results obtained in the simulated laboratory test. The results from these tests were the basis for the new version of the Canadian Highway Bridge Design which allows for GFRP construction to be used as a primary reinforcement building system.

Sunshine Skyway Bridge

Figure 10

The Sunshine Skyway Bridge (seen in figure 10 [27]) is one of the most the most recognized structures in the U.S. When it was built in 1987, it was the world's longest bridges having an overall length of 8.9 km [28]. The Sunshine Skyway Bridge is the most famous of four bridges that span over Tampa bay.

The Sunshine Skyway Bridge consists of two levels, the high-level and the low level trestle spans. A total of 650 girders support the northbound trestle spans and another 650 girders support the southbound span. Type IV girders were used for a majority of the 30 m long trestle spans. An article [28] from "Concrete Repair Bulletin" present how the bridge was repaired successfully by applying CFRP-sheets on the surface of the concrete girders. Shear cracking was observed during a routine investigation of the trestle span girders. In additional cracks were observed in numerous pier caps. In some cases the cracks were very large and exhibited visible signs of water penetration damage.

After an examination of the bridge, a comparison between original and final plans was carried out. The examination concluded that there were significant deviations between the two plans. The beams were originally designed resisting maximum service loads and not having any structural failure. But the appearance of opening and closing cracks created a significant durability problem. The bridge needed to be repaired to prevent further fatigue and additional reinforcement needed to be added to strengthen the existing bridge structure.

A full scale load test was also performed on precast IV girders. Some of the girders with CFRP strengthening were check to see how the shear strength was affected. The shear strength of the beam increased when with the application of CFRP. The test concluded that CFRP upgrade would be the best solution for repairing the shear deficiency.

Before applying the CFRP-sheet to the girders of the bridge, the surface of the girders needed rehabilitation. The cracks with width exceeding 0.3 mm were epoxy injected. All places were some of the concrete had peeled of, were patched with a cementitious repair mortar. All uneven surfaces were filled in with in with a leveling mortar an all the bug holes and smaller cavities were repaired using an epoxy-paste. In addition a protective sealer was applied to protect the concrete against further moisture and chloride intrusions.

Figure 11

Once the surface was repaired, a 610 mm wide strip of bidirectional carbon-fabric was placed vertically down the girder, around the bottom and up the other side to create a U-wrap similar to stirrup (seen in figure 11 [28]). Next a strip was wrapped around the bottom bulb of the girder to strengthen the flange. In the end another strip were placed longitudinally along the top flange adjacent to the underside of the bridge deck. The fabric orientation was chosen to achieve the supplemental loading in multiple directions without having to install additional plies. It could also be mentioned that the fabric went through many investigations/tests to secure the quality of the fabric such as the tensile strength, modulus of elasticity and strain on the laminates. Control tests were also conducted on the epoxy carbon fiber system throughout the project.

The working process met some challenges in the repair of the bridge, such as tides and waves making it tough to have a level working surface for the workers. However, the project was successfully completed one month ahead of schedule. The repairs to the underside of the bridge were accomplished without any disruption of traffic.

The Beddington Trail Bridge, a bridge with structurally integrated sensors

In a paper [29] written by Aftab A. Mufti and Kenneth W. Neale some findings were presented about the durability and the life cycle cost and engineering when using FRP and the SHM (structural health monitoring) process. In some cases fiber optic sensors (FOSs) have been placed on building structures to monitor the structural integrity of these structures.

The Beddington Trail Bridge in Calgary, Alberta opened in 1993. It was the first bridge in Canada that outfitted the FRP tendons with a system of structurally integrated optic sensors for remote monitoring. The bridge was built before the appearance of guidelines so an extensive amount of researchers and planning was required. The investigators involved in the project confirmed a need for a concerted effort and new network that could spearhead transferring this new technology to the construction industry. This project lead to the creation in 1995 of ISIS Canada, a federally funded network of Centers of Excellence that encompasses academic and industrial partners across Canada focusing on FRP, SHM, R&D and their implantation in practice.

They also employed the use of Fiber optic Bragg Grating (FBG) sensors to monitor the strain and temperature behavior of the bridge during construction and later for service maintenance purposes. Six of the girders in the bridge had been pre-tensioned with different types FRP tendons. The fiber optic sensors were attached to the surface of the tendon after the pre-tensioning had been done.

In 1999 the bridge were tested statically and dynamically to check the bridge's structural integrity and the durability of the fiber optic sensors. After six years all the sensors all the FOSs were functioning fine. The same test was performed in November 2004 offering the same results. All the sensors still worked and that the CFRP-bars used to reinforce the girders performed the same way that they were designed to do in 1993.

The Floodway Bridge over the Red River, Winnipeg

Figure 12

The [29] paper also present some information about North Perimeter Highway Red River Bridge, located in Winnipeg Manitoba. The 10-span bridge is 347 m long bridge which is a part of the Trans-Canada Highway system. It should also be noted that this was one of the first FRP-reinforced bridges in Canada that was constructed without receiving some form of grant money. The GFRP-rebar was chosen based solely on life cycle costs [30].

The bridge consists of steel girders spaced at 1.8 m and a composite, cast-in place, steel reinforced concrete deck. The concrete slab thickness is 200 mm and is reinforced with FRP mats in both the top and the bottom [29] . CFRP reinforcement was used as main reinforcement in negative moment regions for both the vehicular and pedestrian cantilevers. Transverse confinement of the deck was provided by steel straps, which had been welded to the top flanges of the steel plate girders. A picture from the construction process is showed in figure 12 [6].

An integrated structural health monitoring system was designed and installed in the steel-free bridge deck to provide data on stressed in the GFRP reinforcement and transverse steel straps.

The stresses in the steel plate girders and the CFRP reinforcement in the negative moment regions were also measured. The system was compromised of a combination of various types of sensors. The monitoring system generated a lot of information which must be stored for a period of time. An automatic system that would incorporate this information into the SHM system was needed. The readings form these systems are able to warn the design engineer if anything is wrong. The system will also reduce the time and cost which is required to review the entire load history of the deck span.

Sub-structure element confined with FRP

An article [31] written by Rajan Sen and Gray Mullins presented a study on sub-structure that require emergency repairs. Some recommendations were also highlighted on which strategies have been reported to be the most effective.

The application of FRP on structures under water is still a challenge because traditionally methods used to reinforce concrete members have been developed for dry conditions only. Modifications for wet conditions are need here. FRP is also a barrier element which can trap moisture that already exists inside the structure. Evaporation of this water by heat generated during curing may trigger localized debonding

Gandy Bridge spans over Tampa bay, likely the Sunshine Skyway Bridge does. The bridge was originally built in 1956 and was scheduled for demolition in 1997. However, it was decided it should be rehabilitate instead of demolished. The bridge is 4.2 km long and it is supported by 254 piers and 22 columns. 77 % of the pillars have needed to be repaired due to the harsh environment.

Eight piles were selected to be repaired in their study. The different piles are presented in table 4. Six of the eight piles were instrumented, which meant rebar probes were installed along the pile at two locations to provide a measure of the corrosion currents due to waves action. It should also be mentioned that the piles reinforced with glass needed a greater number of sheets to compensate for its lower strength.

Pile nr.	***Wrap system***	***Material***	***Instrumentation***
1	Non		Yes
2	Non		Yes
3	Wrapped using a pre-preg system developed by Air logistics ***(system A)***	*Carbon, 1+2 layers (one layers of unidirectional fibers, two layers of bidirectional fiber)*	*Yes*
4	Wrapped using a pre-preg system developed by Air logistics ***(system A)***	*Carbon, 1+2 layers (one layers of unidirectional fibers, two layers of bidirectional fiber)*	*No*

5	Wrapped using a pre-preg system developed by Air logistics ***(system A)***	*Glass, 2 + 4 layers (two layers of unidirectional fibers, four layers of bidirectional fiber)*	*No*
6	Wrapped using a pre-preg system developed by Air logistics ***(system A)***	*Glass, 2 + 4 layers (two layers of unidirectional fibers, four layers of bidirectional fiber)*	*Yes*
7	Wrapped with a wet layup system developed by Fyfe ***(system B)***	Glass, 2+ 4 layers	
6	Wrapped with a wet layup system developed by Fyfe ***(system B)***	Glass, 2+ 4 layers	

Table 4

System A (pre-preg system): The Aqua-wrap Repair system uses a unique water-activated urethane resin and a custom woven FRP fabric that can be wrapped around the pile. The FRP needs to be pre-impregnated with the resin and sent to the site in hermetically sealed foil pouches. The pouches are opened just before the application.

In case of carbon, one longitudinal layer was placed vertically followed by two transverse layers that were spirally wrapped around the pile without overlap. In the case of glass, two longitudinal layers and four transverse layers were needed to provide the same amount of strength.

A 25 cm wide glass fiber veil with a 5 cm overlap was used to consolidate the wrap. This was covered by a plastic film to keep wrap in place as it cured. On average it took less than one hour to wrap a pile. The piles were allowed to be cured for one day, before the piles were painted using a primer that provided protection against UV radiation.

System B (wet layup system): Tyfo SEH-51A is a custom weave, uni-directional glass fabric that is normally used with Tyfo-S epoxy. However, the SW-1 epoxy was mixed on-site and the FRP impregnated just before the application.

Impregnation on site offered greater flexibility since wrap lengths could be adjusted. However, it also created greater logistic problems since the impregnation had to be done on-site. This required careful planning and system redundancies. However, the wrapping procedure was identical to that of system A.

The role of the FRP during the service process was passive; it was designed to be unstressed except for the additional load that was applied to the structure after the columns had been retrofitted. By applying the composite jacket the lost axial, bending and shear capacity due to corrosion was restored. The low strain capacity of the FRP makes the maximum permissible strain the critical parameter in the design. As piles corrode, the lateral expansion of the steel can be as much as 600 % of the original steel. A lower maximum allowed strain limit of the FRP was chosen to accommodate this increase. The limit was set to 0.1%. For the piles reinforced in this structure the ACI guidelines specify what the strain limit should be for both “contact critical and “bond critical” applications of FRP. “Contact critical” applications can be described as FRP in intimate contact with the structure with no specific adhesion requirement. “Bond critical” applications can be described as an application requiring a minimum adhesion since the load transfer is by bond.

An interaction diagram of the FRP reinforced column was developed by using compatibility analyses. In the diagram (figure 13 [31]), the reduced capacity was estimated to be 20 % of the original strength.

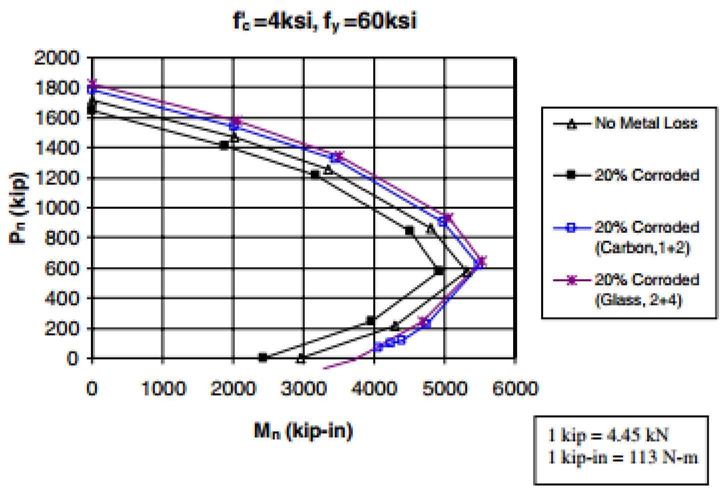

Figure 13

The edges of the piles were chamfered to a curve with a minimum of 19 mm radius. Quick drying hydraulic cement was used to fill all voids to provide a smooth surface. In the end all surfaces were pressure washed.

Two years after the application of the FRP-sheets, the piles were tested to evaluate the status of the piles. The tests were carried out in accordance with established standards. The surface was scored using a 4.4 cm diamond cure drill. A pull-out test was then conducted. The bond between the FRP and the concrete was found to be poor. Most of the wet layup testes showed epoxy failure where the sheet separated from the concrete at its interface. All the testes on the pre-preg system had inter-layer failure. This indicated that the bond between the FRP-sheets was poorer than the one of the interface to the concrete. The test also showed that system B performed better in wet regions, while system A performed better in dry regions. Similar differences have not been founded in the laboratory tested specimens. Therefore, should this problem attributed to the field-technology of pile wrapping. Further research to prevent such inconsistency needs to be conducted.

The estimated costs of wrapping the piles were established for both systems. The costs do not include that for mobilizing or installing the system. The price for glass reinforcement was $ 670/m per square pile and for carbon reinforcement the price was $ 885/m per square pile.

However, the reinforcing systems used in this test is only suited for emergency repairs. They did not perform well in the long run, but within a short-time aspect it will provide a good solution to increase the strength of a pile. It should also be mentioned that if the section needs to be re-formed then Styrofoam or wood insertions with a curved profile should be placed in the corners of the member

Prefabricated FRP decks

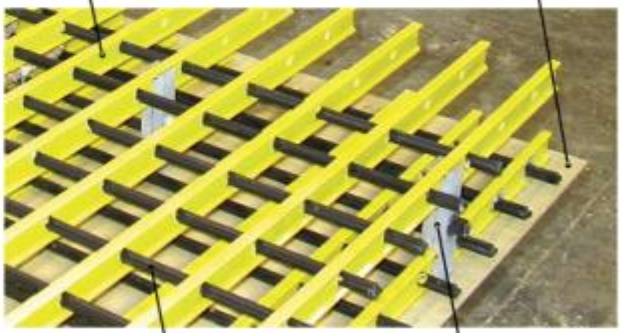

Figure 14

An article [32] written by Fabio Matta et.al. presents a FRP reinforcement deck solution. The system consist of panels which are 23.2 ft. (7.07 m) wide, 8 ft. (2.44 m) long and have a weight of 900 Ib. (408.23 kg) per panel. The components of the system are:

- I-bars (38 mm) running continuously in the direction perpendicular to traffic.

- Tree-part cross rods running through pre-drilled holes spaced at 100 mm on-center in the I-bar's web in the direction that runs parallel to traffic.

- Two part vertical connectors that space the grating layers 100 mm apart.

The whole system ties together forming a 3 dimensional wafer structures that is fixed to the (SIP) form (shown in figure 14 [32]). I-bars are the main carrying elements which carry the load and conduct it down to the perpendicular beams that support the deck. The cross-rods provide shrinkage and temperature reinforcement and control the core concrete to ensure load transfer into the I-bars.

This system provides for strong long-lasting-bridges that can be built in a shorter time frame, thereby reducing the construction cost. Also the time traffic is disrupted is cut and the time required for periodic inspections and maintaining is reduced. So we can see that the advantages of this prefabricated system are:

- Easier and faster construction, as much as 70% reduction in construction time from the reinforcement installation to deck cast finishing.

- Higher productivity, the rate of concrete replacement can be increased by 50 % with this type of deck replacement compared to that of traditional concrete decks with steel reinforcement.

- Improved work conditions which also increases project safety. The panels are easy to handle and can be lifted with a single pick of a crane. The panels are also deigned to allow for workers to walk over the top mat, resulting in a safer work environment.

- Enhanced durability.

The FRP SIP panels can easily be adopted to ensure full continuity of the reinforcement. The width can be adapted to the required bridge width. An acceptable degree of continuity in the secondary reinforcement can be ensured by panel-to-panel overlap. The end panels can easily be adjusted on-site to accommodate the other panels. The panels can easily be cut in the desired dimensions on-site with a hand-saw .Glass FRP with 1/8 in. (3.2 mm) thicknesses are inserted between the bottom layer of cross-rods to cover the butt joints and prevent leaking of the concrete while it is being poured. Each panel can be anchored to the steel girders by a stainless steel threaded bolts and FRP washer.

Bridge 1482301, Green Country, Missouri

The 73 year old bridge was designated for replacement due to severe deterioration of the concrete deck and steel girders. The new bridge was designed by Great River Engineering and had exactly the same dimensions as the old one. It is a total length of 144 ft. (43.9 m) and it consist of four symmetric spans, the exterior once with a length of 37 ft. (11.3 m) and the interior once of 35 ft. (10.7 m). The deck is 7 in. (177.8 mm) thick and is supported by W24 X 84 (in.) rolled steel girders spaced at 6 ft. (1.8 m). The bridge has a clear roadway of 24 ft. (7.3 m)

The bridge “re-decking” operations took only five days to complete, instead of two to three weeks which are required for similar steel reinforced decks. The bridge “re-decking” was executed as follows:

- Day 1, all the 18 decks were set in place an anchored to the steel girders by six workers in a total of six hours.
- Day 2, the prefabricated rail post-cages were inserted to the cut-outs pockets in the SIP panels.
- Day 3, the deck was cast and finished in a total of two and a half hour.
- Day 4, the prefabricated top longitudinal rail-cages were mounted and the post-open railing formed. Since the temperature was below the minimum required, by the County, to precede the casting operation was delayed by one day.
- Day 5, Casting of the rails was completed.

The FRP reinforced deck system had an estimated cost of \$ 44.90 /ft.^{2} (\$ 483.3/m^2), which included the \$ 26/ft.^{2} (\$ 279.9/m^2) of the prototype SIP panels delivered on site. This is more expansive than putting the FRP framework together on-site, but the solution was cost competitive because of the rapid construction time. The indirect costs due to traffic disruption were significantly reduced by using SIP panels on the project. With the projects successful completion it has shown to be a good example of prefabricated FRP-panels providing a beneficial solution for new bridge constructions. It should also be mentioned that the replacement bridge were funded with a \$ 400,000 grant from the U.S. Department of Transportation [33]

Self-compacting concrete in structures

During the last decade, SCC has been used instead of conventional concrete on a great scale. Even with all its advantageous, there still remains a lot of skepticism about its use. It was first developed in Japan in the mid-1980s. Since then much research has been done on SCC and it is often used to solve complex design construction problems. It can, for instance, increase the speed of construction and improve the final finished touch of the concrete surface. In fact, it can give a smooth surface with no need for any further finishing operation after it is casted. As David Rich pointed out in a research paper [34] ; *"Concrete should not be viewed as just a material or a part of the construction process, but classed as a method instead, so the true potential can be realized. Early consideration will allow the engineer to design the structure different than with conventional concrete. This will result in optimized deign and significantly savings on-site. Self-compacting concrete has the potential to create details and complex elements not possible with conventional concrete. To do so, it needs to be perceived as an important tool that can enhance the entire construction process".*

In the following part of this chapter some buildings were SCC are applied will be presented.

Alturki Business Park

The research paper presented by Redwan A Hameed [35] reported a case study done on The Alturki Business Park (seen in figure 15 [36]) project in Al Khobar city in the eastern Province of Saudi Arabia. The structure was solely built using SCC construction and consuming more than 20,000 m^3. The tower itself has 16-floor above ground and two floors underground providing 34,000 m^2 of leasable space.

Figure 15

The design of the structure is based on a 9 m structural grid with variable cantilevers forming a cylindrical building 38 m in diameter. The slabs was designed to be post-tensioned with drop panels at column locations.

Alturki Business Park is a full-scale example of a concrete structure that was completely constructed using only SCC. It was also the first structure in the Arabian Gulf Area to be constructed using this method. Alturki Business Park made the construction industry aware of the numerous advantages that SCC construction provides. Listed below are some of the reasons why SCC was chosen as the primary building material in this construction:

- Technical advantages. Some of the elements were highly congested with steel and utilization of SCC would prevent segregation and honeycombing. Moreover, the SCC would ensure a uniform consolidation. Placing and finishing was also easier, the concrete could be poured into a 7 m high wall continuously in a very short time without any difficulty. The surface finish of the SCC was excellent and there was no need for surface repair after casting.

- Provide awareness to the construction industry about the advantages you could possibly get with adapting SCC for use in future projects.

The concrete used in this structure was required to perform excellent in many ways. The compressive strength of all vertical elements needed to be greater than 60 MPa, while all other elements could be built with a compressive strength of 45 MPa. The maximum concrete temperature at the point of discharge was 32^{o}C. Great concrete workability was also required. The workability of the concrete was also required High demands for workability of the concrete was also required (shown in table 4).

Test	Unit	Min	Max	Description of the test
Slump flow *(Flowability, viscosity)*	Mm	650	750	Fill a cone with concrete, the SCC than spreads out like a pancake batter. The slump flow is then measured as the diameter of the pancake [37].
T50 *(viscosity)*	Sec	2	5	Measure the amount of time it takes for concrete in the slump flow test to reach a diameter of 50 cm [37].
J-ring *(passing ability)*	mm	0	50	The J-Ring is a cage of rebar that is set up around the slump cone. The test then measures the passing ability of the SCC through the rebar cage
L-box *(passing ability)*	h_1/h_2	0.8	1.0	Concrete is placed inside the vertical portion of an L-shaped testing apparatus. A grill is placed in the horizontal portion to simulate reinforcement. When the concrete has flowed to a resting position, the heights of the two different portions gets measured two check the ratio of the two heights [38]
V-funnel (viscosity)	sec	6	12	Consists of a V-shaped apparatus with an opening in the bottom. The time taken to empty the funnel is regarded then gets measured. [38]

Table 4

The concrete mixture was designed in a laboratory using different mixtures and adjusted until the concrete producer (Saudi Readymix Concrete Company) was satisfied with the mixture. The main adjustments were: the increase of paste content, reduction of the w/b ratio, and optimization of the mix grading, as well as the optimization of the HRWR dosage and the variation of the dosage of the cementitious materials.

Local ordinary Portland cement (Type 1) conforming to ASTM C-150 was the chosen cement for the mixture. They also added ultra-fine fly ash with a mean particle size of 5 micron and surface area of 13.000 cm^2/g as a supplementary cementitious material and filler. The dosage was initially 10% by the weight of the cement. In a later stage, silica-fume conforming to C-1240 was used instead of ultra-fine fly ash. The initial dosage of silica fume was 7 % of the cement weight. A coarse aggregate of crushed limestone of 20 mm, 10 mm and 5 mm was used. The fine aggregate was local fine dune sand. Due to SCC`S nature and sensitivity, the SCC required a stricter control system than that of conventional concrete.

The pumping process of high strength self-compacting concrete in high-rise buildings is a challenge. SCC requires more care and attention compared to that of conventional concrete. SCC is very sensitive and may exhibit segregation and decrease of slump flow if pump pressure changes. Laboratory tests were carried out to check if the concrete mixture was in fact pump-able. Pressure sensors were also installed on the pump to monitor the concrete.

Different challenges were experienced during pouring operations of the concrete:

- Lack of awareness and understanding of SCC among most parties involved in the project.

- Resistance to change. However, this was expected and was a result of the previous point.

- Inconsistency of raw materials. SCC was affected due to change of aggregate quality. This resulted in that each concrete batch had to be adjusted with different admixture dosages. Different qualities of the cement were also experienced.

- Loss of flow. This was confronted several times and sometimes it was necessary to use a vibrator to compact the concrete. One of the reasons for flow loss was the pumping itself. Pumping affect the viscosity and yield stress due to structural breakdown and an increase in air content. The air content was measured at 1.6 % before pumping and at 2.5 % after the pumping. Another reason for flow loss was caused by the delayed arrival of trucks on-site. High temperatures and fast drying of the concrete also caused loss of flow.

- Bleeding water was in some cases noticed on the top of the concrete after pouring. In those mixtures the mix was immediately adjusted.

- Occasionally, blowholes were noticed on the surface. To minimize this, the viscosity and flow of the mix was modified.

The use of SCC in this building project had a lot of advantages in spite of all the challenges. The project schedule was accelerated and the manpower needed was greatly reduced. The working conditions were also significantly improved because there was no need for a poker vibrator thereby reducing the noise pollution. All in the entire project was a success. The use of SCC construction resulted in a uniform concrete structural consolidation and a smooth concrete surface on the building.

Ritto Bridge, Japan

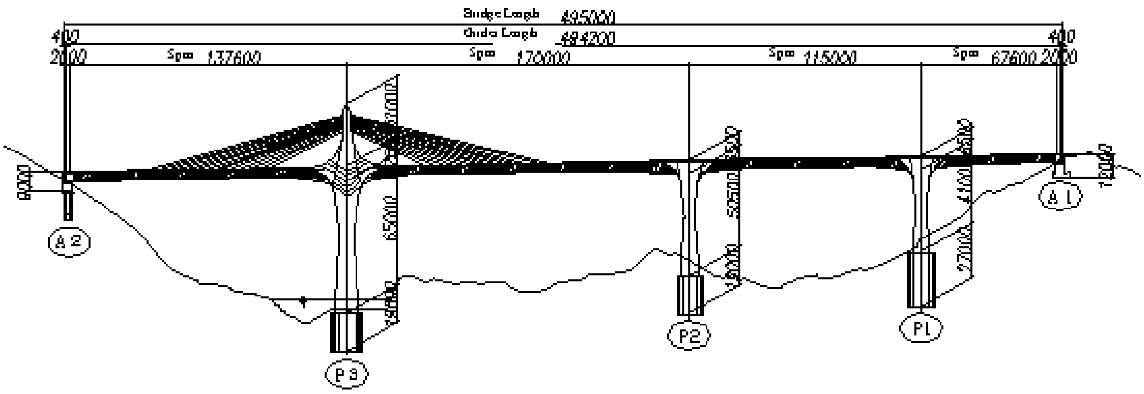

Figure 16

The paper presented by Masahiro Ouchi et.al. [39] discuss some projects were SCC has been chosen instead of conventional concrete. Among those projects, is the Ritto Bridge (shown in figure 16 [39]) presented. The bridge is a prestressed concrete extra-dosed bridge with corrugated steel webs on the New Meishin Expressway in Japan. The highest pier of the bridge is 65 m. Because of possible dangers from earthquakes the use of high strength concrete with a compressive strength of 50 MPa was used. The pier was constructed using concrete with yield strength of 685 MPa.   The arrangement of steel was densely placed in the pier's construction, so SCC was chosen for its ease of workability. The requirements for the SCC are shown in table 5 [39]. The concrete was proportioned based on in-house mixes, plant trial mixes and mock-up tests. The final mix proportions are given in table 5 [40].

TESTING ITEMS	UNIT	SPEC.
FRESH CONCRETE		
SLUMP FLOW	(mm)	600 or 650
FLOW TIME UNTIL 500MM.	(sec.)	3 to 15
U TYPE FILLING CAPCITY	(mm)	Min.300
V TYPE FUNNEL FLOW TIME	(sec.)	8 to 15
AIR CONTEND	(%)	4.5
CHLORIDE ION CONTENT	(kg/m^3)	Max 0.3
HARDENED CONCRETE		
COMPRESSIVE STRENGTH	MPa	50

Table 5

Design Compressive Strength (MPa)	Water Cement Ratio (%)	Maximum Aggregate size	Slump Flow (mm)	Air content (%)	Unit weight (kg/m³)						
					Cement	Water	Fine Aggregate	Coarse Aggregate		HRWR	Pigment
								20 mm	13 mm		
50	33.0	20	600-650	4.5	470	155	868	505	336	6.11	14.1
Test reults										note	
Testing Time form Mixing Complete	Slump Flow (mm)	Flow Time 500mm (sec.)	Flow Time Stop (sec.)	U Type Filling (mm)	V Type Funnel (sec.)	Air content (%)	Concrete Temp (°C)	Compressive Strength (MPa)		Target slump flow was revised when push up height of pump rose.	
								7 days	28 days		
5	630	6.1	34.0	338	11.8	4.3	19.0	41.0	74.0		

Table 6, Mix proportion (In house trial mix)

In the final mixture it can be seen that the coarse aggregate is limited by a diameter of 20 mm. Limiting the size of the coarse aggregate will decrease the internal stress and result in better flow of the concrete [18]. We can also see that the mixture has a relatively high content of cement; this is common in HSC and SCC. The water w/c ratio is also quite low compared to conventional concrete mixtures. To obtain high flowability of the mixture while having a low w/c ratio, the mixture added HRWR (High range Water reducer). In the end, over 12.000 m^3 of concrete was successfully placed in the piers without any vibrating work needing to be done [40].

The article [40] written by Y. Nakajima et.al. introduces a discussion about the costs of S.Q.C. (super quality concrete) and high strength steel compared to the costs of conventional concrete and steel. The unit costs of high strength steel are slightly more than two times that of conventional steel. The unit cost of S.Q.C. is 1.5 to 2.5 times that cost of conventional concrete. The S.Q.C. Association has set design standards for the use of S.Q.C and established cost estimating under various conditions. The result of their investigation was that S.Q.C required lower construction costs because smaller volumes of concrete are required. Table 7 [40] show a cost comparison of a bridge construction.

Table 7, Total Cost of Bridge Construction

	Conventional Structure	**SQC Structure**
Construction Cost Comparison	1	0.97
	Superstructure	
	Conventional Structure	SQC Structure
Weight Comparison	1.00	0.90
Construction Cost Comparison	1.00	0.99
	Bridge Pier	
	Conventional Structure	SQC Structure
Weight Comparison	1.00	0.78
Construction Cost Comparison	1.00	1.03
	Foundation Pile	
	Conventional Structure	SQC Structure
Concrete Volume Comparison	1.00	0.63
Construction Cost Comparison		0.81

	Box Culvert	
	Conventional Structure	SQC Structure
Concrete Volume Comparison	1.00	0.65
Construction Cost Comparison	1.00	0.81

The S.Q.C. Association has also conducted some test to estimate costs of maintenance of a reinforced concrete pier located at the seashore. The test showed that S.Q.C. only needs maintenance two times in a life cycle of hundred years; on the other hand conventional concrete needed maintenance 4 times during the same period. In the test is was found that more than 40 % of the life cycle costs could be reduced by adapting S.Q.C. in the construction of the piers.

Self-compacted concrete, provide an excellent surface finish to the structure

Ando`s pavilion in Piccadilly Gardens, Manchester is a 130 m long gently curved concrete wall [41]. On the concave part of the structure is a covered space providing coffee shops and shelter. The structure is an example of how SSCs provide an excellent surface finish after casting (shown in figure 17 [42]). The wall itself is made from Ando`s trademark pour-in-place concrete. The wall consists of 2x1 m casted in a plywood formwork [41]. The pattern from the plywood can easily see be appreciated on the concretes surface.

Figure 17

Another example of a structure where SCC was applied, is the National Museum of the 21st Century Arts in Rome. The contractor chose SSC in order to meet the strict requirements deeded for the surface finish [43]. The concrete was cast along the entire lengths of the walls to avoid construction joints. The longest members were up to 70 m long and the highest were up to 9 m tall. The concrete was mixed on-site, and to avoid segregation the concrete was limited to not be poured out from heights higher than 15 cm. Application of powdered limestone and epoxy-resin provided perfect a perfectly smooth surface. It can also be mentioned that the building won the award for “World Building of the year” during the World Architectural Festival 2010 [44].

Can save you time and effort

“The self-compacting method: concrete that can save you time and effort” is the name of an article [45] written by David Rich et.al. The article provides us information about a study done on a typical residential concrete slab casted with SCC. The goal for the article was to find out how much time you can save by casting a typical residential slab with SCC instead of conventional concrete.

The study explored that using SCC instead of conventional concrete can reduce both time and costs. However, they say in their study that in order to utilize the full potential of SCC it is important to accept that SCC is a method of working, what means more than a single construction material. This is the same conclusion that was found in the article [34] written by David Rich.

Findings of this investigation showed that considering rates per square meter, a residential concrete slab could be constructed up 70 % faster by adopting SCC. This significant outcome was due to the simpler construction offered by SCC. The need for much of the leveling process was removed. Only a minimal leveling was required. The need for operatives was also cut down to two persons, which is about half the manpower needed for conventional concrete casing. Furthermore, conventional concrete is subject to the unpredictability of curing that can delay the opportunity to power float. Observed examples in their study showed that such variability with power floating conventional concrete could delay the work from two hours to as much as two days after a pour. Such a delay would obviously contribute to increase the building costs.

Construction with SCC is a method that does not permit any time delays, once started it must be finished immediately. This will reduce the projects risk of time over-run because it will be easy to estimate the time required to caste a concrete slab. In other words, casting with SCC is not only time saving, but be cost favorable as well.

A guide for design and construction [46] written by Peter J M Bartos offers some information about the construction of Akashi-Kaikyo bridge. The construction of the bridge is one of the most remarkable projects built using self-compacting concrete. The bridge required approximately 500.000 m^3 of SCC for the massive anchorages blocks that encase the cable endings and for the 60 m-deep foundation beneath. The concrete required in construction of the foundations of the 300 m high main towers are excluded from this number. Self-compacting “non-dispersive” underwater-concrete was also used in the construction of these foundations.

Amounts of up to 19.000 m^3 of fresh SCC were placed in the foundations per day. Such a high volume of placement would not be possible with conventional concrete. It was said that by using SCC, the construction time was shorten by 3-4 months, saving about 20 % of construction work. The concrete was pumped and distributed through a system of pipelines. The placement was controlled by video cameras and a system of sensors. Fresh concrete was allowed to free-fall up to 3 m when leaving the discharge pipes. Because of the large volumes of concrete, the concrete required use of special “low-heat” cement and pre-cooling of aggregate, which had a maximum size of 40 mm.

Improved work conditions of the labors

Constructing with conventional concrete could cause injury to the workers in the long run due to the vibration requirements of the concrete compacting. A webpage [47] from the Canadian Centre for Occupational Health and Safety give a little insight in how vibration affects our body.

The vibrated induced health conditions progress slowly. In the begging it starts as pain, as further exposure it develops into an injury or in some cases a cause's disease. Some operators of hand-held vibrating tools can develop vibration-induced white finger. Vibration can cause changes in tendons, muscles, bones and joints and can affect the nervous system. Collectively, these effects are known as Hand-Arm Vibration syndrome.

Hand-Arm vibration syndrome exposure affects the blood flow, and causes loss of touch sensations. It is not only the arms that suffer illness due to vibration. Cases of whole-body vibration can cause fatigue, insomnia, headache and "shakiness" shortly after and during exposure. It can also increase heart rate, oxygen uptake respiratory rate and could produce changes in blood and urine. East European researchers have noted that the exposure to whole-body vibration can produce an overall ill feeling which they call "vibration sickness.

In Great Britain, 1040 new cases of Vibration White Finger were reported during the year 2010/11 [48]. In addition, there was in the same year reported 475 cases of Carpel Tunnel Syndrome (CTS).

Not all of these cases can be contributed to the vibration of fresh concert, but continues exposed to vibrating tools will affect your health. Therefore, we can see the benefits of using a system of concrete construction like that of SCC that does not require vibration. In conclusion, having a quieter work place is better for all including the workers, the people who live in the vicinity of the construction site and for the environment as a whole.

Future of self-compacting concrete in Norway

A rapport [49] written by the concrete NORCEM AS provides some information about the use of self-compacting concrete in the Norwegian construction industry. Of all pre-mixed concrete used in Norway in 2007, only about 2-3 % of the total concrete market was comprised of SCC. Inquiries showed that the reason for this low percentage was mainly due to the lack of knowledge about the advantages of using the SCC product. Other reasons included its higher cost, its sensitive installation requirements, unclear responsibility rules and lack of general guidelines.

To increase the volume of SCC applied in structures, Norway, decided NORCEM AS in corporation with Norwegian Concrete Association two construction projects were SCC will be used instead conventional concrete. The goal of the two projects was to increase the awareness about the advantage of implementing SCC in Norwegian construction. NORCEM AS was assigned the task of exploring the possibility of developing a new more environment friendly SCC mix.

Existing cement plants in Norway need modification to meet the increasing demand of the SCC production. New silos must be built to deposit the new contents needed in the SCC mixture. But as SCC demands increase its higher initial cost will be reduced.

What responsibility the SCC manufacturer and the SCC contractor have over this product still remains unclear in Norway. Typically, in traditionally caste concrete the contractor is responsible for the quality and proper vibration of the concrete to ensuring its correct strength. But in the case of SCC, the concrete manufacturer will produce the concrete with the compacting properties required contained with-in the mixture. In response to this dilemma the industry has adapted new guidelines. The NB29 is a set of new specifications clarifying the responsibilities between the contractor and the concrete manufacturer. The contractor will be responsible for the amount and the mix makeup of the concrete order, while the concrete manufacturer will be responsible for correctly producing the specified mixture. Even with these new guidelines in- place good communication between the two is critical.

NORCEM AS has set out a goal to increasing the use of SCC to 50% of Norway's total concrete market shear within 5 years.

High strength concrete

During the last four decades HSC concrete has been used in many structures. Through these projects, much information has been gained on how this material can be used in new ways. Today you can find HSC in many superstructures. It has been applied in marine structures, skyscrapers and highway projects. Their cost effectiveness has been demonstrated through research and by the FHWA giving grants and support for these new HPC highway and bridge construction projects [50].

In begging this section of my project I will state what HPC is by referring to ACI; *"high-performance concrete –concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices."* High strength concrete fits this definition. Through the many articles I read I found that the terms "high-strength concrete" and "high-performance concrete" refer to the same thing. Therefore. In what follows the references are consistent to the original paper that has been reviewed.

A guideline [50] written by the FHA discusses the costs of HPC compared to conventional concrete. HPC typically cost 20 % more than that of conventional concrete per cubic yard. However, in some instances, HPC has cost the same as conventional concrete because the fly ash and slag being used cost the same or less than that of the cement that is being replaced. It is known that HPC can increase the short- and long-term durability of the project, extending its service life while reducing its life-cycle costs. By applying HPC to the Oakland Bay Bridge, the construction engineer calculated the life of the bridge would be extended by 150 years. But the successful application of HPC depends greatly on designers working closely with local fabricators and contractors to arrive at a cost-effective solution. The use of trial mixes in both the laboratory and onsite setting allows for concrete checks to be performed assuring that the concrete meets the specified criteria. This also prevents any costly concrete mistakes from being made, saving time, money and manpower.

Burj Khalifa

Figure 18

Burj Khalifa (shown in figure 18 [51]) is currently the world`s tallest building, stretching more than 828 m up in the sky. An article [52] written by Joe Nasvik offer some information about the construction process of Burj Khalifa. Concrete was used as the primary building material due to its very good characteristics. The mass and rigidity of concrete provides twice the dampening effect compared to that of steel, thereby, reducing the wind load on the tall super-structure. Concrete is also fire resistant and is a good sound insulator.

Building the world`s tallest building was a great challenge. The ground soil had high chloride and sulfate content. The caissons were built in high performance dense concrete to resist the high sulfate soils. The superstructure is anchored to the soil by a total of 194 caissons, where each caisson is 5 (1.54m) feet in diameter and have a depth of approximately 150 feet (45.8 m).

The onset of creep and shrinkage occurring at different rates over time could be critical for a building of this scale. However, to evade this problem in the Burji khalifa tower the builder used the same concrete mix for all the vertical members on a given floor level. This strategy maintained the same surface to-volume ratios for columns and core walls.

The architecture and engineering company Skidmore, Owings & Merrill (SOM) who were in charge of the projecting of the building, believe the elastic modulus had influence into the compressive strength. Therefore, they required concrete with a high modulus of elasticity, about 6300 ksi (about 43400 MPa), to be reached, after 90 days. The concrete mix used contained Portland cement, fly ash, and silica fume. The mixture also had also very fine aggregate fraction and contained up to 650 pounds/cubic (385.6 kg/m^3) yard of cement content. Superplasticizer was also added to maintain the flowability needed with w/c ratios below 0.32. The concrete used in this superstructure preformed excellent in many ways. The concrete mixed used in the columns and core walls had compressive strengths of 15000 psi (103.4 MPa), the specified requirements had only been for 11600 psi (80 MPa). Significant amounts of water in the form of ice had to be added to the mixture in order to keep the concrete temperature between 75^{o}F (23.9^{o}C) and 90^{o}F (32.2^{o}C).

In another article written by W. F. Barker et al. [53] we are given some more information about the mixture used in Burj Khalifa tower. The mixture contained 13 % fly ash, 10 % silica fume and the maximum aggregate size was about 20 mm. The concrete mix was virtually self-compacting with an average slump flow of approximately 600 mm. Above level 127, the structural requirement to compressive strength reduced to 60 MPa, the maximum allowed aggregate size was also reduced to 10mm.

Freedom tower

Figure 19

Freedom tower (seen in figure 19 [54]) is currently under construction at Ground Zero. Reviewed information for this work is published at the webpage named PR Newswire [55].

For constructing the Freedom Tower, Concrete was the chosen material for both the tower and the To construct the Freedom Tower concrete was the chosen material for both the tower and the 186 foot (56.7 m) inner safety core supporting the superstructure. The concrete brand chosen to raise this superstructure is the brand iCrete offered by a company named iCrete which is based in Beverly Hills.

The primary supplier for the concrete is Quadrozzi Concrete. The iCrete contains supplementary materials such as fly ash allowing the building team to come up with a high compression high performance mixture that is needed in this towers construction. The iCrete optimizes the space voids between the aggregates, reducing the amount of paste required to bond the aggregates together resulting in reduced shrinkage and creep. By reducing the amount of cement needed harmful carbon dioxide emissions will be reduced by about 40%, compared to other concretes with similar properties.

Tony Arnold, president of iCrete said this bout the concrete being used in freedom Towers: *"To get this kind of strength in concrete you usually have to increase the amount of cement. We reduced the amount of cement."* [56].

iCrete is currently the strongest concrete ever used in New York City, it has a compressive strength of 14000 PSI (96.5 MPa). It will take some 240 000 cubic yards (about 183 000m^3) of iCrete to build the tower and 186 foot inner safety core supporting the super-structure.

A patent disclosure [57] written by the inventor Per Just Anderson on product, presents some information about the iCrete. The iCrete is produced by using an improved method that allows for the design and manufacture of concrete's composition to maximize its final compressive strength and slump. The patent is based on this optimization of the concrete manufacturing process. Stated here is a brief description of its disclosure:

- The disclosure is directed to a method for designing a concrete with high workability and optimized gradation. It is a system that comprises memory for storing data related to a concrete mix design and a processor configured to: (1) access the data related to a concrete mix design, (2) calculate a water to cement ratio in order to achieve the compressive strength, (3) calculate an amount of water to be added to the concrete mix design having target on the compressive strength and the slump amount, (4) provide the calculated amount of water for display.

This method is more effective than the one provided in standardized tables, because the presumed characteristics of the raw materials utilized by the concrete manufacture rarely reflect the true characteristics of the raw materials.

Bridge constructions

The guideline [50] written by the FHWA discloses some information about bridges located in the U.S. where HPC has been applied.

In constructing Georgia's SR 920 over I-75 the structural engineer faced a challenging design process; he chose to use 127 ft. (38.7 m) long AASHTO Type IV beams. By choosing this type of girder the overall depth would be minimized. The solution lay in casting the girder using HPC instead of conventional concrete. The lower permeability of HPC was also expected to result in a more durable structure with a longer service life and minimal maintenance.

Before the mixing trials began, some requirement demands of the concrete, prior to the girder pour, needed to be met:
-There should be the addition of class F fly ash.
-It should contain silica fume volume ranging from about 5-10%.
-The chloride permeability had to be less than 3000 coulombs 56 days after casting.

HPC was also used in the bridge decking. The same requirements where demanded of this mix except it needed a stricter set of parameters in case of oxygen permeability occurring. The HPC's permeability needed to reach a number less than 2000 coulombs 56 days after casting. In addition, the deck required it be continually moist cured for seven days. This was achieved by utilizing a burlap-polyethylene cover and soaker hoses.

The final mixtures are given in the table 8:

MEMBER:	GIRDER	DECK
CEMENT (kg/m^3)	479.2	365.0
SILICA FUME (kg/m^3)	44.4	15.4
FLY ASH (kg/m^3)	49.5	61.5

Table 8

The mix result of the concrete poured in the girders had an oxygen permeability of 500 coulombs at 56 days after casting. However, for the deck the result was 3963 coulombs at 56 days after casting. This variation can be explained because the final mix design was done by the contractor in the field.

A year after the construction a field investigation was done on the structure to check the time-dependent mechanical properties and structural responses of the girders. They conducted a large number of tests to check the compressive strength, modulus of elasticity and to see if any creep and shrinkage had occurred. The test revealed an inaccuracy of creep and shrinkage conflicting with the trial test results obtained from the HPC modeling predictions. The AASHTO LRFD Specification and the PCI Design Handbook used to calculate the time-dependent pre-stressed loss and deflection of HPC beams had largely overestimated the values. The new findings resulted in change and modification to these existing specifications allowing for a better performing HPC girder to be developed.

Ultra-High Performance Concrete used in construction of infrastructure

The FHWA has also written an article [58] about Ultra High Performance Concrete (UHPC) and its application in structures.

The mechanical properties of UHPC include compressive strength greater than 150 MPa and sustained post-cracking strength greater than 5 MPa. Today UHPC is considered for use in a wide variety of infrastructure applications. The enhanced durability property that UHPC provides has also resulted in discussion about using it in thin overlays to protect concrete members.

However, only a handful of state transportation departments have deployed UHPC components within their infrastructure. It has so far been applied to pre-stressed concrete girder simple-span bridges, deck panels and field-cast connections between bridge components. The first UHPC highway constructed in the U.S. was in Iowa where it was used it in Little Cedar Creek Bridge. You can also find it has been applied to two bridges in Virginia where UHPC was used in place of conventional concrete within the I-girder shaped members. The reason for replacement was to engage the tensile properties of the members which would allow elimination of mild steel reinforcement shear stirrups.

Some technical information about Little Cedar Creek Bridge, Iowa is presented by Lafarge [59]. The bridge is 18.2 m long and 10 m wide. The bridge consist of five type B precast, prestressed UPHC girders, spaced at 2.23 m. The 200 mm deep waffle slab was designed to be integrated with the girders. The deck cross-section is composed of two precast Ductal ® UHPC waffle deck panels which are connected at the centerline of the bridge. Each panel is 2.4 m long and 4.94 m wide. The system is shown in figure 20 [59].

Figure 20

The deck panel system was chosen because it was considered as a perfect solution in order to construct new bridges, or in order of rehabilitate older bridges. Due to the material`s superior properties, the advantageous is numerous and typically include improved bridge deck performance and improved speed of construction. Other indirect benefits may include improved durability, lower maintenance, reduced user inconvenience and extended usage of life.

Nevertheless, by applying the solution into construction, they received some grant money to accomplish the construction of this demonstration bridge. The project was a success and in 2012 the project won two prestigious industry awards:

- Portland Cement Association (PCA) Bridge Design award of Excellence.

- Precast/Prestressed Concrete (PCI) Design Award.

Mixing and Casting of UHPC

An article [58] written by FHWA presents information on how to mix and cast UHPC. Casting and mixing UHPC is quite similar to casting and mixing conventional concrete. You can use nearly any conventional type concrete mixer to mix UHPC. However, UPHC requires more increased energy input resulting in a longer mixing time. The energy input, in combination with the reduced or eliminated coarse aggregate and low water content, will result in a high risk for overheating. To avoid this unfortunate result, traditional procedures need to be modified. This modification can be addressed through the use of a high-energy mixer or by lowering the temperatures of the constituents and/or partly replacing the water with ice. Taking these steps will allow UHPC to be mixed in conventional pan and drum mixers.

The pouring of UHPC can be delayed while additional admixes are added. UHPC frequently requires many hours before it begins to set, but the mix must not be allowed to self-desiccate during the extended delay time. The long-term mechanical and durability properties of UHPC can be affected by how it was handled during the casting procedure.

Casting the fiber orientated concretes requires special consideration in terms of the placement operation. Internal vibration of fiber reinforced UHPC is not recommended. The casting procedure can also affect the fibers and how they are dispersed. This should be considered when the casting sequence for a mixture is developed.

Outer tank for LNG storage

An article [60] written by T. Nishizaki et.al. offers some information about the construction about an outer tank located in Japan which is used for LNG storage. The tank was constructed with SCC to meet the liquid-tightness and durability requirements necessary as well as to reduce the costs and construction time.

The concrete was poured into pre-stressed walls which were designed to form a circle with a diameter of 82.4m. The wall is 38.4m high and has a thickness of about 0.8 m. Approximately 12.000 m^3 of concrete was used in constructing the walls.

However, this was not the first outer tank for LNG storage constructed using pre-stressed concrete. Osaka Gas had constructed several tanks earlier using this method with concrete that had a compressive strength of 40 MPa. The difference was, that the outer tanks for LNG storage had been built using a concrete with a higher compressive strength of 60 MPa allowing the new tank to increase its capacity from 140.000 kl to 180.000 kl

To increase the compressive strength of the concrete, the new concrete mixture had about 1.4 times more cement content than the old ones had. But this addition would result in faster hydration causing more heat to be expelled during hardening. To prevent thermal cracking of the new concrete a low heat Portland cement was used in the concrete mixture. This kept the cement mix temperature from rising. The mixture also added a CSA base expansive admixture to increase the resistance against cracking due to shrinkage.

The construction of the walls was divided into 10 lifts with height of 4.4m. One lift required approximately 1000m^3 of SCC. The concrete was poured continuously in the circumferential direction of the 267m length. Each concrete placing took about 5 hours. The concrete was produced and supplied by five ready mixed concrete production plants.

The concrete production plants were self-controlled and were responsible for delivering the concrete with the required concrete specifications. However, an additional quality system was established to inspect all delivered concrete at the construction site before placement. This was said to have played a major role towards the wider use of SCC in construction, which helped to change SCC from being viewed as a “special concrete” to an “ordinary concrete” in the future.

It should be mentioned that a number of basic experimental trials using the actual equipment as well as several concrete tests were carried out before the project got under way. This effort led to the successful completion of the tank, reducing both the construction time and overall costs.

My findings:

Table 9 and 10 which follows below give a short general summarization of what found in the reviewed research papers. The tables present my findings in an easy to understand manner that explains the advantages and challenges of using high-strength concrete, self-compacting concrete, FRP-bars and FRP-sheets on concrete.

Material:	**The material should be considered in projects were:**	**Advantageous:**	**Challenges:**
High strength concrete	- The dimensional size of a member needs to be reduced such as in high - rise structures or where structural space is limited.	- It allows us to construct structures with ever greater dimensions and structural complexity. - Easy access to fly ash and slag brings the cost of HSC down to about the same price as conventional concrete. -Due to its denser pour structure, higher strength, durability and finish can be achieved as compared to that of conventional concrete applications	-Can be more expansive than conventional concrete, typically about 20 %. - Increased risk for overheating during mixing. However, if this is addressed before mixing, this problem is quite solvable. - Today's Eurocode used to calculate concrete member's capacity is not recommended for used with concrete which has a compressive strength greater than C90/105. $(NS - EN\ 1992 - 1 - 1{:}\,2004)\ [3.1.2\ (2)]$
Self-compacting concrete	- The concrete members are highly congested with steel reinforcement (decreased risk of segregation and honeycombing). - In harsh environments (SCCs have better long-term durability). - Need for an accelerated project schedule. - Need of excellent surface finish. - In areas with strict requirements to sound noise	- High durability of the final product. - Easy application of the concrete. - Less injuries to the workers. - Can cut the construction costs of a structure. -Does not exhibit the same delay in power floating as conventional concrete may. - Faster construction process.	- More sensitive to inconsistency of the constituents than conventional concrete is. - More sensitive to changes in pump pressure than conventional concrete. - Higher costs, generally about 1.5 to 2.5 times greater than conventional concrete. - A more extensive formwork is required because pouring of SCC exerts greater pressure on the formwork than pouring of conventional concrete. [61] - Lack of knowledge about the material. - Workers resistance to replace convention concrete with SCC.

Table 9

Material:	Materials to consider for use in different projects environments are:	Pros	Cons
GFRP-bars	- When a structure will reside in a harsh environment. GFRP-bars will perform better than steel when exposed to salts and acid environments. (Hall`s Harbor Wharf and the Floodway Bridge) - When rapid construction repair is required. Prefabricated polymer panels are light-weighted and easy to handle. High indirect costs can be cut to a minimum due to a shortened traffic disruption time. (Prefabricated FRP-panels and Bridge 1482301)	- Does not corrode - Has an assumed longer service life [29] . Manny articles assume that if a steel reinforced deck can have a service life of 50 years, adequate GFRP-bars reinforced deck can have a service life of 75 years	- Generally has a higher material costs compared to steel. A glass fiber reinforced deck usually cost 2-5 % more to construct than that of a similarly reinforced steel deck construction -The technology is quite new therefore no field observations have confirmed its longer assumed service life.
GFRP-sheets	- When quick repair to the structure is needed. (“The Old” Gandy Bridge) - When there is a need to prevent corrosion of the steel reinforced concrete members. GFRP-sheets can provide a good solution in this situation preventing alkaline levels from dropping in the concrete member. - When the interruption of traffic causes high indirect costs. (“The Old” Gandy Bridge)	- The same advantages as written in the tab below for CFRP-sheets. - It is cheaper than CFRP-sheets.	- Has the same disadvantages as written in the tab below for CFRP-sheets. - GFRP sheets have lower elastic modulus and tensile strength compared to CFRP-sheets so a greater number of sheets are required. .
CFRP-sheet	- The same use as written in the same tab above for GFRP-sheets. However, carbon fiber has greater strength than glass fiber making the construction process less time consuming. Fewer members need installing so the man-power required can be reduced	- Can bring back lost capacity of a concrete member. Completely wrapping of a concrete column can provide better flexural and compressive strength. - Very easy to apply - Quick repair time - Could be applied on old concrete members thereby increasing the member’s service life (sunshine skyway bridge)	- The failure mechanisms are quite complex. - Even though standards exist for calculating the strength the CFRP-sheet contributes to a structure, it is not very well known or understood. - The epoxy mixture used in the interconnection of concrete and the sheet can become a weak link

			(“The Old” Gandy Bridge

Table 10

Conclusion

This research paper hopes it has presented an insightful general overview for the application of high-strength concrete, self-compacting concrete and for fiber-reinforced polymer sheets and bars in building structures. The basic properties of high- strength concrete and self-compacting concrete materials have been discussed in order to give a better understanding of how we can best utilize them in new construction projects. Whereas, the primary use for the application of fiber-reinforced polymer sheets and bars have been presented for its ability to reinforce older structures. Thereby, making them safer and extending their life-cycle.

By going through these building processes an increased knowledge and awareness has been gained on how the materials work together and what happens when mistakes are made.

All the materials reviewed here seem to have a great future in both the new and old construction industry:

- High strength concrete will allow us to build taller structures and at the same time decrease the size and number of columns and beans required to support these new and innovative structures. Since fewer materials will need to be used in the building process, the environmental footprint will also be reduced. So, we can clearly see the advantages these new concrete systems have over the old conventional concrete model.

- Self-compacting concrete will increase the durability of new constructions by providing a smooth surface finish to the casted concrete. With no need to finish or vibrate the concrete, construction time and cost will be reduce. Injuries due to vibration will also be significantly reduced and the construction environment as a whole will also be far better off without the loud vibration noise that the concrete vibration pokers make.

- FRP bars can be used as a replacement for conventional steel reinforcement. Because of its non-corrosive behavior, we can increase the service life of a structure by utilizing FRP-bars instead of conventional steel. FRP can also be easily applied above a concrete surface. Thereby, providing a fast solution to increase a concrete member’s lost strength.

- FRP sheets can also provide a fast solution to increase a concrete member’s strength. The application is easy and takes little time.

During these last years, all the materials discussed here have been used on an ever increasing scale. However, there still remain a lot of skeptics who need to be convinced of the advantages gained by using these new construction materials over conventional concrete and steel. Better guidelines would also make it easier for these new technologies to be applied in the field.

As knowledge increases about these new materials and their many uses, it will become more commonplace to see them in new building structures. Within a few years, all these materials could perhaps be regarded as just another “conventional structural building material”.

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