2.0 CHAPTER 2 - CONCRETE

2.1 DEFINITION

Concrete is a construction material, consisting of cement paste, an inert filler called aggregate, water, and admixtures. When produced, this material is in a plastic state and will take on a more or less permanent shape as imparted to it by forms or extrusion machines.

The three main constituents, Portland cement, aggregate, and water, must pass certain qualitative criteria, or tests, before they may be used in bridge construction. Water, for instance, must be potable.

2.2 AGGREGATE

Aggregates are classified as normal or lightweight. Normal aggregates are made of materials mined from gravel deposits with high silicon content. This gravel must be sound which accounts for its high density and may be processed through crushers and screens to specific maximum particle sizes and grain size distribution. The portion passing a 5 mm sieve is called sand. Very fine particle sizes (i.e. passing a 75 micron sieve and classified as silt and clay) must be excluded from or at least restricted in the manufacture of concrete due to their high water demand and negative affect on strength. Normal weight aggregate shall contain no more than 3% clay lumps and friable particles, 5% passing the 75 micron sieve and 0.5% coal and lignite, as per ASTM-C33. Lightweight aggregates are made synthetically of pumice, slag, expanded clay or slate particles. If the concrete is manufactured with both the sand and the coarse aggregate lightweight, it is called lightweight concrete. If the concrete is manufactured with only the coarse aggregate lightweight and the sand normal weight, it is called semi-lightweight concrete.

Normal weight concrete has a density of approximately 2400 Kg/m3, Semi-lightweight concrete is approximately 2000 Kg/m3, and Lightweight concrete is approximately 1700 Kg/m3. Semi-lightweight and lightweight concrete has been used for precast/prestressed concrete girders to reduce the dead load for transporting and erection. With heavier moving and lifting equipment now available, the lightweight concretes are not as commonly used in precast/prestressed girders as in the past. Practically, all cast-in-place concrete used in bridge construction is normal weight concrete.

2.3 CEMENT

The Canadian Standards Association distinguishes 5 types of Portland cement, only 3 of which are in common use in Alberta. Type GU is a normal Portland cement, and the most widely available. Type HE is of a finer grind than Type GU, and is referred to as high-early because of its ability to develop compressive strength faster than Type GU. It is used in the precast concrete industry where fast turn-around times on forms and early release strengths are important. However, most bridge specifications leave the choice of Type GU or Type HE cement up to the manufacturer, who must balance the operational advantages of Type HE cement with its higher price and the installation of extra storage bins.

Either Type GU or HE may be used where the concrete does not come into contact with soil. If this is the case, the soluble sulphate content of the soil is determined by test, and upon findings of certain concentrations, a sulphate-resistant concrete is specified, utilizing Type HS cement. This cement type is usually somewhat darker than Type GU or HE, but chemists find it difficult, if not impossible, to distinguish between these types with any confidence once the paste has hardened. It is therefore of some importance to ascertain the use of Type HS cement at the time of manufacture.

Some specifications use ASTM terminology, which classifies Type GU as Type 1, Type HE as Type 3, and Type HS as Type 5.

The following table is reproduced from CSA A23.1 for information regarding the presence of soluble sulphate such as calcium, sodium, and magnesium salts. There are other implications for minimum design compressive strengths, e.g. 27 MPa for severe sulphate attacks, and proper drainage around the structure, which hopefully will keep sulphate bearing water migration out of the concrete.

Polential Degree of Sulphate Attack	Total Sulphate In Soil Sample (%)	Water Soluble Sulphate in Soil Sample (%)	Sulphate in Ground Water Sample (mg/L)	Type of Cement to be Used	Max. Water Cement Ratio
Negligible	0.00– 0.10	---------------	0.00-150	GU,MS,HE,LH,HS	---------
Mild	0.00-0.20	---------------	150-1000	MS, HS	0.50
Considerable	------------	0.20-0.50	1000-2000	HS	0.50
Severe	-------------	Over 0.50	Over 2000	HS	0.45

Table 2.1 Types of Cement and Water/Cement Ratio Requirements for Concrete In Contact With Soils and Ground Waters Containing Various Sulphate Concentrations

2.4 ADMIXTURES

2.4.1 AIR ENTRAINING

A widely used admixture in concrete bridge structures is air entraining. Air entraining incorporates very small air bubbles in the concrete matrix that have the ability to accept the volume increase of absorbed freezing water without bursting the concrete. The prescribed air content of fresh concrete, usually in the range of 5.5%, plus or minus 0.5%, can be ascertained with a pressure-type air meter. Once hardened, the inspector may order a linear traverse test of concrete cores, which is a microscopic determination of air bubble content and spacing. This would be called for if the concrete surface shows scaling and freeze-thaw damage is suspected as its cause. The engineering report that should accompany the linear traverse test results will offer a professional opinion as to the adequacy of the air entraining agent that was incorporated. The spacing factor should be 230 microns or less.

Since air entraining is sometimes obliterated by excessive surface finishing efforts, tests should be carried out near the surface of the core, and then again somewhere in the middle of the core. In this manner the inspector may provide feedback with respect to the efficacy of concrete finishing standards of the Department. Clearly a massive gathering of test evidence showing loss of air entrainment near the surface of finished concrete, as opposed to formed concrete surfaces and interior concrete segments would call for a review of concrete finishing practices.

2.4.2 WATER REDUCERS

Another commonly used admixture is a water reducing agent. Water reducers have the effect of reducing the mixing water demand of a concrete mix for a given index of workability, which is usually expressed in millimetre of slump. The pozzolanic admixtures are widely incorporated in departmental specifications, whereas the use of super plasticizers, which are usually associated with only short-term slump increases, must be compatible with air entraining agents to produce a mix with correct air content, bubble size and spacing.

2.4.3 SILICA FUME

Silica fume is an extremely small particle sized by-product of thermo-electric power plants. Some of the particles are smaller than the particles of cigarette smoke. The addition of silica fume to a concrete mix has been shown to increase the compression strength and improve the impermeability of the concrete. In the mid 1980's, the Department started to use silica fume as an admixture for concrete used for deck overlays. Although silica fume improves the strength and impermeability of concrete, it can make the concrete more susceptible to shrinkage cracks and extra effort is required in curing the concrete. Presently, the Department uses 7.5% to 9.5% silica fume by mass of concrete in all concrete used for decks, curbs, medians, roof slabs, approach slabs and deck overlay concrete.

2.4.4 FLY ASH

Fly ash is also a by-product of thermo-electric power plants but it has larger particle sizes than silica fume. Fly ash has cementitious qualities and has long been used as a substitute for up to 20% of the Portland cement in certain concrete mixes particularly concrete used in mass concrete structures. Besides the economy of reducing the Portland cement requirements, fly ash was also found to improve the pump ability and workability, and improve the impermeability of hardened concrete. Also fly ash reduces the amount of heat of hydration generated and reduces the high temperatures that can occur in mass pours. However, concrete with fly ash has been shown to have some durability problems with de-icing salts and compatibility problems with some common air entraining agents. The 28 day strength gain for concrete with fly ash is generally less than concrete without fly ash.

Therefore, until recently fly ash was not permitted in concrete used by the Department in bridges.

Recent developments with high performance concretes have shown that fly ash used with silica fume with compatible air entraining agents and proper mix design can overcome the previous problems with the use of fly ash and produce a very high quality concrete. The Department presently has a specification for a Modified Silica Fume Concrete which allows for the sum of silica fume and fly ash to be equal to 25% by mass of Portland cement. The silica fume content must be 7.5% to 9.5% by mass of the Portland cement. However, a minimum Portland cement content of 350 kg/m3 must be used.

2.4.5 STEEL AND PROPYLENE FIBRE

Concrete is weak in tension and cracks easily when subjected to shrinkage, temperature or tensile stresses. Approximately twenty years ago manufacturers started placing steel and polypropylene fibres in concrete mixes to reduce cracking. The steel fibres are approximately 50 mm long and come in a number of different shapes. Polypropylene fibres are generally shorter than steel fibres. The Department started using steel fibres in concrete overlays in the mid 1980's. The present specifications for fibre reinforced concrete calls for 60 kg/m3 of steel fibres. Polypropylene fibre has not commonly been used by the Department to date.

2.4.6 SELF COMPACTING CONCRETE

Self compacting concrete (SCC) is a fairly recent development with the first prototype being developed in Japan in 1988. By using a number of admixtures such as high-range water-reducers and viscosity modifiers and adjusting the aggregate content, a concrete mix is produced which flows easily and completely fills spaces between reinforcement and forms by virtue of its own weight with little or no vibration required. SCC mixes are 'thixotropic' meaning that these mixes become fluid when stirred or shaken and then they return to a semi-solid state at rest.

The Department has used self-compacting concrete on a trial basis for a few applications but has not yet adopted it for common use.

2.5 FACTORS AFFECTING STRENGTH

Concrete strength depends on many factors, and the inspector is sometimes called upon to identify particular items that may have occasioned an observed lack of predicted strength development.

2.5.1 WATER

Although water is a necessary ingredient in the manufacture of concrete, it is also directly and indirectly responsible for most strength loss. The concrete industry has embraced

Abram's concept of water-cement ratios, and often the concrete quality is defined by this variable rather than a compressive strength index.

Depending on the exposure condition and durability requirements, many concretes are specified with a water cement ratio of 0.35 to 0.5, the lower number being indicative of a better concrete, other factors being equal. Only about 0.2 of this factor is needed for hydration, the rest of the water being added to improve mixing, placement, and workability of the concrete. This excess water will eventually dry up, leaving behind capillaries and voids that detract from the compressive and other strength parameters of the concrete matrix. As well, these voids are fairly large and do not contribute to the element of freeze-thaw protection as do smaller, entrained voids. The capillaries provide routes for salt and water to penetrate the concrete.

The inspector can see these voids and formulate, on the basis of some experience, an opinion as to the possibly excessive water content of the concrete as mixed. Shrinkage cracks are another indication of an excess of water used in the mix.

2.5.2 AGGREGATE SOUNDNESS

By picking through a detached concrete chunk, the inspector may notice the shape of the coarse aggregate particles. Angular rock faces, as produced by a crusher, are more desirable than polished, rounded gravel pieces coming from a river deposit.

Sometimes these rounded particles may be further prevented from forming a good bond with the matrix by certain natural coatings that act as bond breakers.

Other problems obvious to the inspector may be thin and elongated pieces, iron-clay stones and other unsound intrusions like clay lumps, cherts, etc. Clay lumps may be distinguished from lightweight aggregates by the soundness and uniform distribution of the latter.

2.5.3 SAND

Concrete sand is subject to certain gradation limits that are difficult to ascertain in hardened concrete. However, one of the more common occurrences that detract from proper strength development is the ratio between sand and coarse aggregate. A matrix that obviously contains much more than 50% sand, as opposed to rock, would have a higher water and cement demand than one that is properly proportioned. Given the producer's preoccupation with considerations of economy, it is often the case that the higher demand for more water in oversanded mixes is more likely met than the demand for more cement to cover the increased surface area of this aggregate combination. A wetter concrete also tends to shrink more while drying to a constant moisture content condition. In the extreme, this may cause cracking of the concrete member.

The inspector may also encounter pockets of unbound sand that are running freely from a failure plane of hardened concrete. This may be indicative of improper mixing and should be noted in the inspector's report.

2.6 DEVELOPMENT OF STRENGTH

Most specifications of concrete call for a demonstration of compressive strength in lab-cured concrete samples at the age of 28 days. The selected age is an arbitrary cut-off point agreed upon by convention in the industry and lends itself to the specification of contractual obligations between concrete supplier and consumer. It capitalizes on the fact that in general, cements have gained most of their potential strength at that time, but ignores the capabilities of 'late bloomers', like cements augmented by fly ash, or concretes that were accidentally overdosed with a water reducing agent.

The mentioned strength specification also ignores what may have happened to the concrete after the sample was withdrawn. Placement and curing conditions may be quite distinct from those provided to lab samples. For this reason, it is important to be able to identify problems in hardened concrete that can be attributed to improper compaction, segregation due to a high drop, loss of paste due to forms that were not quite tight, lack of surface curing, frost or wind and heat damage and the like. Many, but not all of these problems, may be discovered, and their repetition prevented, by monitoring the results of field cure cylinder samples. Other improper placement, finishing and curing practices can only be identified through careful field observations, either at the time of placement or afterwards by inspection of the affected concrete structure.

2.6.1 HYDRATION

Mention was made of temperature extremes, and their deleterious effect on strength development. Mass pours may overheat due to the exothermic reaction of cement hydration, even during moderate temperatures, causing cracking and sometimes dark discolorations on formed surfaces. Such situations call for special cooling efforts and temperature monitoring during and after the pour. Similarly, surface cracking may occur in winter construction when heating is removed and the concrete surface cools and shrinks much faster than the interior. This is known as thermal shock and it is advisable to leave the hoarding in place after heating is turned off so as to allow concrete to cool gradually.

Ambient temperatures best tolerated by man are also ideal for concrete placement. Freezing causes disruptive growth of ice crystals, whereas hot temperatures and wind tend to rob the concrete of water necessary for hydration. In addition, fast temperature changes cause significant volume changes that in turn cause internal stresses, which may be more than the barely hardened matrix can absorb without cracking.

2.6.2 CURING

Proper curing methods are designed to counteract these damaging effects of the environment and to guarantee short and long term strength development.

For sake of a more or less complete account of curing procedures, we mention here the heat-curing methods used by most precasters. These are often referred to as steam curing, although in many cases the heat is transferred to the freshly cast concrete member by circulating hot water heaters using pipes equipped with sheet metal radiators. This heat is carefully incremented and applied after the member is wrapped up so as to decrease water loss due to drying. Comparisons of lab cured and heat cured test cylinders indicate early strength gain of the latter, which is however surpassed at the 28 day stage by the former. Clearly, the lab test method overlooks these effects due to curing, and the field inspector should be constantly on the lookout for indications of understrength concrete, even if the lab-cured test samples indicate adequate strength gain.

2.6.3 FINISH AND DURABILITY

One of the most persistent complaints about concrete concerns surface is appearance and finish. Several classes of finish are specified by the Department. Class 1 is ordinary or formed surface finish, Class 2 rubbed finish, Class 3 bonded concrete surface finish, Class 4 floated sub-deck finish and Class 5 floated finish with surface texture. The use of steel trowels for finishing fresh concrete is not acceptable by the Department. Even the most elementary of these finishes (Class 1), requires that all fins and irregular projections be removed and cavities produced by form ties and all other holes, honeycomb spots, broken corners or edges and other defects be thoroughly chipped out, cleaned, moistened for 30 minutes, filled with mortar, and cured.

Despite all of these precautions, the inspector will encounter instances of rough concrete surfaces that can give rise to further deterioration by trapping water and possibly chloride ions. Often the underlying cause is a concrete that satisfies the actual structural demands of strength but fails to live up to expectations of durability as a result of poor aggregates, improper air entrainment, inadequate water-cement ratios, poor placement and finishing practices and/or inadequate curing. It is therefore not correct to conclude that the compressive strength requirement of a member has been satisfied because it has successfully opposed all compressive forces imposed on the member. Durability is also designed in part in terms of water-cement ratio, which translates into compressive strength that may easily surpass that required for structural purposes, including all applicable safety factors, by several increments.

2.7 PROPERTIES OF HARDENED CONCRETE

2.7.1 COMPRESSIVE STRENGTH, FLEXURAL STRENGTH

As first discovered by Abrams in 1918, the water-cement ratio of concrete determines its compressive strength. This type of strength is the concrete property that is most often relied upon, although there are others that are of importance as well. Some handbooks derive the tensile strength and shear strength of concrete from the compressive strength, e.g. 10% and 12% respectively. Other manuals equate tensile and flexural strength values in concrete.

In practical terms, these facts reveal that concrete by itself can be relied upon to pass on substantial compressive forces or loads to a substrate, but requires steel reinforcement to withstand flexural moments and shear forces typically encountered in suspended beams and structural slabs. Steel is however very susceptible to corrosion, due to ingress of salts or acids, and must be protected from these by all available means.

To appreciate this problem, the inspector needs an understanding of the ability of hardened concrete to absorb water and water-borne ions that can attack reinforcing steel. Sometimes a false sense of security is developed by a specification that requires a minimum of 25 or 50 mm of concrete cover over all structural steel contained. Even though this requirement has been met, the inspector should consider all available means that would reduce the availability and ingress of water, such as positive drainage, deck drains, sealing and the like.

2.7.2 SHRINKAGE

As concrete changes its water content during absorption and drying, it also changes its volume slightly. There is expansion during absorption of water up to 150 millionths per unit length, and corresponding contraction during drying. Similarly, temperature changes cause expansion of a unit length of hardened concrete at the rate of approximately 10 millionths per degree C increase. If a 100 m concrete span undergoes a total temperature variation of 60 degrees Celsius, this may amount to 60 mm of length change. Fortunately, high temperatures are associated with low moisture contents, and vice versa. As a result, the length changes due to thermal expansion and drying contraction tend to cancel each other out, i.e. concrete drying to constant weight at 50% relative humidity will shrink by 60 mm per 100 m. Furthermore, if the concrete is not constrained at either end, the length changes do not have any negative effects unless a bearing 'freezes' and does not allow for relief of stresses caused by length changes, in which case these stresses are retained in the concrete structure, particularly in the anchor points, that may result in cracking of the concrete.

Another implication of length changes due to temperature changes affects the behavior of reinforcing steel in concrete. Standard reinforcing steel has roughly speaking the same thermal coefficient of expansion as concrete, and therefore is not in danger of relaxation inside the concrete. As for prestressed steel cables, the expected temperature variations are allowed for in the design of the amount of prestress force applied to the cable during manufacture.

Since dry concrete is made up of aggregate and paste, it should be noted that aggregate acts as a restraining force on volume changes due to drying shrinkage and temperature expansion. Thus larger maximum aggregate sizes help combat negative effects of shrinkage. However, water in the mix increases drying shrinkage, and water reducers therefore help combat drying shrinkage cracking through their ability to curb water demand. Air content has no appreciable effect on drying skrinkage.

Dirty aggregates cause excessive shrinkage, and light weight aggregates may also increase the amount of drying shrinkage by up to 100% when compared to normal weight aggregate. The use of accelerators has also been shown to increase drying shrinkage and associated cracking.

As for thermal contraction and expansion, again the two constituents of paste or gel and aggregate provide different coefficients of expansion to the total for the mixture. Quartzites and other hard aggregates are rated at 13 millionths per degree Celsius, but limestone only

5.6 millionths per degree Celsius. Cement paste, which makes up 15% to 20% of the total mix, may range from 9 to 22 millionths per degree Celsius, a higher water cement ratio being associated with a lower coefficient of thermal expansion. This is so because a good portion of thermal expansion and contraction is due to migration of water from capillaries to or from gel pores. (STP 169B, ASTM)

These factors have two implications for the inspector. On the one hand, concrete which is not free to change its shape under the influence of these forces may show signs of distress such as cracking or disintegration near anchor points. Secondly, a concrete area that is to be patched or otherwise repaired must be replaced or covered with patching or repair materials that have at least comparable thermal coefficients of expansion.

2.7.3 CREEP

In addition to the effects of shrinkage there are long term changes under load, referred to as creep. Factors affecting creep are the water-cement ratio of the concrete, its paste content, the age of the concrete, physical properties of the aggregate, the size and shape of the member, amount of reinforcing steel in the member, relative humidity, temperature, and carbon dioxide content of the air, as well as curing conditions. Further, several factors acting in combination to cause shrinkage in drying concrete may have a larger effect than the sum of each component would account for.

2.8 CRACKS AND DETERIORATION IN CONCRETE

Cracks in concrete may have many causes and different manifestations. If necessary, crack width may be measured with a crack comparator. Cracks of less than 0.1 mm in width are called hairline, equal to or greater than 0.1 to less than 0.3 mm are narrow, equal to or greater than 0.3 to less than 1.0 mm are medium, and those equal to or greater than 1.0 mm are called wide.

Narrow cracks are permitted to occur in certain concrete designs and codes, cf. ASTM C76, which describes a crack feeler gauge of 0.2 mm in thickness that is used to define certain permissible loads on concrete culverts. Similarly, CSA S6, allows flexural cracks of up to 0.35 mm (0.013") in the design of exterior concrete. However, no vertical cracks are permitted in prestress beams, and cracks may close in other reinforced concrete after passage of the live load that opened them.

2.8.1 SHRINKAGE CRACKS

Beginning with the initial set of the cement paste after pour, fresh concrete undergoes shrinkage which may result in stresses that cannot be absorbed by it, hence the development of shrinkage cracks. Cracks are further promoted by rapid evaporation of surface moisture and other cracks may be caused by stresses induced by rapid temperature variations of hardened concrete. Most of these cracks are straight across a slab but may also be oriented at a right angle to the wind direction in the case of drying shrinkage cracks.

2.8.2 CRAZING CRACKS

Rapid evaporation of bleed water may result in crazing, which is a network of shallow surface cracks in steel troweled concrete surfaces. If these cracks do not penetrate the concrete more than 1 mm they are usually not considered serious. A more serious case of cracking with similar patterns is called map cracking, which may be associated with alkali-silica and alkali-carbonate reactions. These reactions, which result from a reactivity of certain silicious and other aggregates to cement paste, cause swelling and gradual destruction of the bond between aggregate and paste.

Settlement cracks do not follow any predictable pattern and are associated with partial failure of form work or dropping of supports due to settlement in foundation soil. These are other possible reasons for random cracking.

2.8.3 D-CRACKS

Designers attempt to combat cracking problems by specifying low water-content concrete mixes and adequate curing. They also provide for expansion joints in the concrete surface. Special crack patterns, so-called "D" cracking, form along these joints if they are inadequate.

2.8.4 FLEXURAL CRACKS

As the concrete is loaded by dead and live loads, it may be flexed in certain areas beyond its flexural strength limits as anticipated by the designer. If there is insufficient reinforcement to pick up these flexural loads, it may crack there as well. Such cracks are considered to be working cracks and should be identified as such in deck zones that are marked by the transition from negative to positive moments. This is a feature of continuous beams or slabs in the vicinity of non-end supports where the top surface is in tension rather than in compression as would be the case for most of their length.

The inspector should mark such lateral crack locations with lath extending over the side of the bridge and then investigate under the bridge to determine if lateral crack locations there coincide with the ones on top. With some structures, there may be much stronger indications of cracking and efflorescence. These cracks may require repairs paid at a rate per linear metre, and an estimate of the total length of cracks should be entered in the report.

2.8.5 DIAGONAL CRACKS

Diagonal cracks in the vertical faces of pre-stressed beams near their supports are indicative of overloads and should be treated as critical.

Often cracks are neglected as a result of certain design assumptions with respect to concrete zones in tension. In these areas the stresses are presumed to be carried by steel reinforcing, but the open path of brine into the steel that they provide makes it only a matter of time until some corrective action becomes mandatory. The inspector should faithfully record all cracking and its propagation by marking the ends of cracks and giving some indication of depth and width in order to establish a valid case history of all deleterious influences that impinge on a structural concrete member. If any cracking is observed in a pre-stressed (or post-tensioned) member, it should be regarded as very suspicious since the effect of pre-stressed or post-tensioned cables would normally place most of the concrete in compression.

Cracks that allow ingress of solutions of deicing chemicals to cause corrosion of rebar may be responsible for spall damage, which is the loss of entire chunks of concrete ranging up to 50 mm and more in depth, usually exposing the corroded rebar.

2.8.6 CRACKING IN PRESTRESSED GIRDERS

Prestressed girders are designed to place concrete around the tendons in compression, which rules out vertical cracks in the lower region. Nevertheless, vertical cracks may be encountered in that region, revealing irregularities in the applied prestress force. Both over-stressing and under-stressing may result in vertical cracks.

In some cases insufficient prestress force combined with excessive shrinkage and creep may render the prestress mechanism inoperative and allow cracks to open in the lower zone that contains the tendons.

On the other hand, overstressing may overpower the compressive strength of the concrete surrounding the tendons and result in crushing of this region and bending of the beam, creating vertical cracks in the upper region which does not contain prestressed cables.

There have been occurrences of horizontal cracks paralleling the tendons in prestress girders which appeared a few years after construction. As a rule these cracks are not very deep and do not exceed 0.2 mm in width. Once they have opened, they tend to increase in length due to freeze-thaw action. Their appearance has been explained firstly as a result of excessive compression stresses generated by prestressing, and secondly as a result of excessive shrinkage and temperature deformations imparted by heat curing. Rapid temperature changes can result in wide temperature differences between internal and external concrete regions which set up stresses due to differential heat expansion that crack the concrete. If the horizontal cracks produce rust staining, they may be due to corroding prestress cables, which is spalling the concrete cover; such a situation is serious and calls for quick corrective action.

Cracks in the girder ends may be the result of insufficient shear reinforcement or a malfunction of the bearings.

Prestressed concrete bridge members may be damaged by high load strikes. Damage has been classified as minor if only concrete portions are affected, with all reinforcing steel being untouched; cracks in the damaged area are less than 0.1 mm in width only. Moderate damage is also said to affect concrete only, exposing reinforcing bars and prestress strands; cracks may exceed 0.1 mm, but there shall be no severed strands. If strands are severed or deformed, the damage is classified as severe. This condition also is associated with loss of concrete across the whole bottom flange of concrete girders, with some loss of concrete in the web offset to the location of damage in the bottom flange.

Cracks that continue from the bottom flange into the web indicate that the force impacting on the prestressing strand has exceeded its yield strength, and the damage is critical. A lateral offset along the bottom flange is also critical, as is vertical misalignment in excess of the allowable. Permanent deformation of stirrups as indicated by longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface damage are also critical. Critical damage may require immediate installation of temporary supports at time of inspection, or closing of the bridge.

2.8.7 SCALING

Scaling is a surface damage in concrete which lifts off patches of concrete, usually due to freeze-thaw action. This action starts with the loss of surface mortar in patches and progresses deeper with subsequent freeze-thaw cycles. Usually, it is due to the lack of sufficient air entraining at the time of manufacture. Scaling may also be the result of poor finishing or curing practices or construction in rain; such scaling would normally not affect concrete below the top surface. Other possible causes are cold placement, deicing salts, high slump, lack of curing, and lack of cold weather protection.

Scaling may occur under asphalt wearing courses, and should be checked for by exposing a test area of the concrete surface. A chain drag may also indicate hidden spalls and scaling.

2.8.8 SPALLING

Concrete damage that results in the loss of surface chunks is called spalling. Usually this is due to corrosion cracking, which, as the name implies, is the result of corrosion of the reinforcing steel near the surface. The corroded bar gains in diameter due to the products of corrosion taking on more volume than the original steel occupied. This generates expansive forces that burst the concrete. Chloride ion, which is chiefly responsible for this cracking, does not require a crack opening to reach the embedded steel, as it is able to penetrate uncracked concrete to start with. However, the first signs of cracking promote spalls and escalate its damaging effects. Some sources suggest that spalling is also caused by trapped bleed water and early finishing of a slab.

2.8.9 POP-OUT

Where only individual aggregate pieces break out of their surrounding matrix, the result is called a pop-out. Usually this is the result of frost-susceptible aggregate, though sometimes the mere absorption of water can cause pop-outs. Examples of aggregates susceptible to pop-outs are ironstone, coal, and sandstone.

2.8.10 PUNCH-OUTS

Punch-outs are small holes in decks that are made up of pre-cast or pre-stress girders. These girders are built to specifications that put a premium on thin, low-weight sections, hence the use of lightweight aggregates and the likelihood of traffic wear creating the occasional perforation. Cast-in-place decks are usually too thick to permit this type of perforation, although punch-outs are possible here as well after prolonged periods of neglected spalling.

2.8.11 SCOURING / ABRASION

If the bridge crosses a river, its piers may be subjected to erosion due to collisions with ice masses and debris. Most concrete piers are protected by steel plate nosing against this hazard. Another hazard under water is cavitation, which is the pitting of concrete by water borne bubbles that were created by a disturbance upstream. The collapse of these bubbles is accompanied by considerable implosive forces which are capable of causing a honeycombed appearance in the concrete surface. Removal of all stream obstructions near the piers is indicated by such scouring activity.

2.8.12 CARBONATION

Carbonation is the lowering of the pH of the concrete. To explain this process, it is necessary to understand that the various products of cement hydration are highly alkaline with a pH value from 12 to 13. Carbon dioxide, which is present in the air, but may be produced in higher concentrations by open flame heaters in winter concreting, reacts with the hydrated cement compounds. This process is accompanied by a lowering of the alkalinity of the product, say to a pH of 9.5, which may be tested by the inspector through use of indicator solution. The lowering of the pH value means loss of protection for reinforcing steel, which results in corrosion and spalling in concrete. Usually, the process is restricted to a few millimetres of surface concrete exposed to the environment, and the approximate depth can be determined as follows:

A vertical cut or gash is prepared in the affected area and sprinkled with a solution made up of phenolphthalein (1 gram diluted in 50 cc alcohol and diluted to 100 cc with distilled water). The test causes a purple discoloration in the unaffected substrate and leaves the carbonated surface more or less unchanged. (The discoloration may be unsightly and some consideration should be given to this fact when selecting a test site. In some cases it may be advisable to test a section of concrete after removal from the slab).

Carbonation may reach a depth of 15 mm or more after 50 years and can result in considerable loss of section in concrete and rebar.

2.8.13 LEACHING AND EFFLORESCENCE

Efflorescence is a white powdery substance often found on cracks on the underside of concrete slabs. If the build-up is a gel still loaded with moisture, it is called an exudation. It is the result of leaching which dissolves calcium hydroxide and other products of cement hydration from the paste and transports these usually to the lower surface where they react with carbon dioxide in the air. As the water responsible for the leaching action evaporates near the concrete surface, it leaves behind the white stain and powdery salt build-up called efflorescence. Most salts are sulfates or carbonates of sodium, potassium and calcium, such as calcium carbonate. The action is especially prevalent in the presence of soft rain water that is free of salt ions when it enters the concrete.

2.8.14 DAMAGE TO REINFORCEMENT

The possibility of overload and high load damage to concrete structures has been widely publicized. Such accidents may be accompanied by breaks of tendons that require very specialized repair procedures. The presence and effects of misaligned strand conduits are less well known, though they may have the same consequence. Usually these situations are occasioned by crowded rebar conditions and the fact that manufacturers like to fasten these conduits to rebar cages, i.e. after most of the crowded conditions have occurred in the beam form.

Strand conduits are utilized if deflection of strand cables cannot be accomplished during the prestressing phase, or would result in undesirable geometrics of strand patterns. In these cases a conduit is laid along the desired track and the strand is inserted and post tensioned, i.e. stressed after the concrete has attained sufficient strength.

There remains little evidence of any misalignment after the concrete has been poured, and usually a pre-pour inspection is called for to snag this type of problem. In the field, there may be some irregularity in the amount of camber obtained, or other problems that may be due to poor alignment of conduits.

2.8.15 CONCRETE VOIDS OR HONEYCOMB

Also, there is the problem of voids in concrete due to rebar crowding. Such a problem occurs if the rebar cage is so dense that it acts like a fine sieve that gets clogged by large aggregate particles, preventing passage of plastic concrete into all intended spaces. Such voids may sometimes be detected by hammer sounding or by more sophisticated ultrasound measuring methods. Concrete voids visible from the outside are called honeycombs.

2.8.16 CHEMICAL SPILLS

In daily service, concrete may be exposed to chemical spills and other such accidents. The Portland Cement Association has published a list of chemicals and substances that may cause rapid disintegration. In general, all acids are harmful to concrete, but the following have been singled out by the PCA as especially destructive: Hydrochloric acid 10%, Hydrofluoric acid 10%, Nitric acid 2%, Stearic, Sulfuric acid 10%, and Sulfurous acid. Salts and Alkalis in solution under this heading include Calcium (sulfite solution), and Aluminum chloride. There are many other substances causing disintegration, and a decision on how to remove the spill from a bridge deck should take into account environmental considerations for the stream and other factors. Clearly, such incidents call for rapid communication of pertinent details to the Engineer in charge of the region.

2.8.17 FATIGUE IN CONCRETE

Elastic deformation of concrete under load may turn into inelastic deformation after many loading cycles. Hardened concrete does not completely recover from the effect of repeated stresses that are only one half of those that would cause failure on a first try. In fact, experiments have shown that a member may fail after a million or so cycles of approximately one half of the normal failure load.

Fatigue also affects the reinforcing steel in structural concrete. However, as long as concrete has not cracked there is little probability of fatigue failure in the steel, even though the working load may have been exceeded. If prestressed concrete cracks, there are high stress concentrations in the steel in the vicinity of these cracks. Under repeated loading, the bond between wires and concrete may be lost, or the wire may break.

2.9 FIELD TESTING OF CONCRETE

2.9.1 CHAIN DRAG

Chain drag is one of the most productive and extremely simple methods designed to detect delaminations and other voids in concrete. It consists of a T-shaped bar with or without wheels that trail a number of short lengths of chain with links of 1 to 2 inches from its T-bar. The long piece is pulled by the operator who listens to the sound generated by the chains as they are dragged across the pavement. Delaminations and other discontinuities return a different frequency than normal concrete and the operator marks all such areas with a spray paint can if such marking is permissible. Marked areas may be further defined by hammer soundings that determine the precise extent of the distressed area.

This method can lead to operator fatigue and may thus miss out on the identification of some suspect areas. As well, it requires a quiet environment, as traffic noise and other excessive industrial noise can mask the auditory return. It does not produce a permanent test record and requires some grid pattern marking on large slabs to avoid needless overlapping or unintended misses. Chain drag is not useful for decks which have asphalt overlays since it detects the debonding between the concrete/asphalt interface only and may thereby mask delamination in concrete.

2.9.2 REBOUND HAMMER

The Schmidt Impact Hammer is a convenient instrument for the determination of field compressive strength of hardened concrete. Its accuracy is often contested, but in conjunction with dispersion values given for each strength range, it can be relied upon as a first line investigative tool with well defined limits of confidence. Results are affected by the following:

1. The surface finish of the concrete.
2. The moisture content of the tested concrete.
3. Temperature.
4. Carbonation in the concrete surface.
5. The rigidity of the tested member.
6. The attitude of the test instrument (vertical, horizontal, etc.)

The manual accompanying this device is self explanatory. ASTM C 805 refers to its product as the 'Rebound Number of Hardened Concrete', thereby sidestepping the issue as to whether this number correlates with compressive strength as claimed by the manufacturer; in fact, 5.3.2. states, 'This method is not intended as an alternative for strength determination of concrete', although in practice there is no other use for it.

The ASTM method calls for 10 shots per test area, and the deletion of a maximum of two readings if they depart by more than 7 units from the average. Tests exceeding this variability are discarded.

Since the ASTM method does not purport to yield compressive strength values, it is not interested in the use of the calibration anvil which is available to calibrate a hammer. Type N hammers should read at least 78 on the anvil, the upper limit being 82 (which is seldom encountered). The surface condition of the test area and the attitude of the hammer should be reported along with the location and strength value.

The hammer is packaged in a round cylinder, which should not be allowed to roll off the seat of a car, since a drop of the instrument causes warping of its delicate mechanism.

If the results of impact hammer readings give rise to a conflict between observed and expected concrete strength results, deficiencies must be confirmed by core testing before they can be taken up with outside agencies such as contractors or other jurisdictions.

2.9.3 CORING

Coring involves the use of a diamond-studded, water cooled core barrel that is rotated by an electric drill, cutting into the concrete, and isolating a cylindrical core sample for removal. The sample is then tested in compression much like formed concrete test cylinder samples by an independent concrete test laboratory.

Contractors doing this work have several core diameters available, the most popular being 75 mm and 100 mm. The core diameter should be chosen based on the following criteria (CSA-A23.2-14C):

1. The diameter should not be less than three times the maximum aggregate size.
2. The diameter should not be more than the depth of the concrete member.
3. The applicable test specification in the CSA prefers a diameter of 100 mm.

If the concrete is fairly fresh, it should have a minimum age equivalent to 7 warm curing days before coring is attempted.

In preparation for coring, the inspector should refer to drawings of the concrete member and mark 3 coring locations in close proximity to each other for each concrete area suspected of containing under-strength concrete. Locations should be selected so as to avoid cuts through reinforcing bars and other miscellaneous steel. If a core is accidentally cut through a piece of reinforcing steel, it should be replaced by another one taken from a different location. All cores must be marked so as to trace them back to their origin, and handled very carefully. Under no circumstances should a prestress strand be put at risk by coring. Locations of these prestress cables can be found on the drawings. This consideration applies also in cases where contractors are permitted to drill cores in bridge concrete at their own expense. Arrangements must be made also to backfill or patch holes with grout of a compressive strength that matches or exceeds the design strength of the concrete in question.

Cores are tested after 24 hrs. of conditioning in lab air if they are taken out of concrete in dry service, or after 40-48 hrs. of immersion in water if they are taken out of concrete in wet service, i.e. concrete that is exposed to the elements or wet soil. Wet testing can give results 30% lower than dry testing. The class of service must be given to the concrete test laboratory, along with details regarding core diameter and core length, which is normally twice the core diameter. Other items of interest to the contractor carrying out coring are:

1. Availability of water and electric power on site.
2. Height of scaffolding if needed.
3. Patching instructions.
4. Time and place to meet the inspector prior to coring.

All details of this nature, including a statement as to the party paying for the core tests, must be transmitted in writing.

The laboratory shall condition the cores as indicated, trim them with a diamond saw as necessary, and test the samples in compression after capping them on both ends with a sulphur compound. Where the length-diameter ratio falls short of 2, a correction factor is applied to the reported compressive strength.

2.9.4 PULL-OUT TEST

A slightly less expensive test is the pull-out test, which is even cheaper if preparations for it can be scheduled before the concrete pour. In this configuration it is known as the Lok test, which measures the force necessary to extract a specially shaped cast-in-place bolt. If the bolt has to be inserted after the concrete surface has hardened, it is necessary to grind out a hole with a router and the test is then known as the Capo test. Results have been correlated to compressive strengths by the manufacturer. More accurate correlations may be established by comparison of Lok test results to companion cylinder test results in a pilot project.

2.9.5 WINDSOR PROBE

The Windsor probe utilizes the same mechanism in reverse by shooting a projectile into the concrete surface with a pyrotechnical charge (gun & cartridge), and measuring its depth of penetration. This is correlated to compressive strength.

This test depends on the hardness of the aggregate as expressed by Moh's hardness number. Local aggregates may vary from 4 to 7, and a number of 6 has been used with acceptable results.

2.9.6 ULTRASONIC PULSE VELOCITY

This test method is particularly suited to determine the extent of fire damage, and may also be used for the measurement of depth of cracks. This test method consists of measuring the time of travel of an ultrasonic pulse of 20 to 250 kHz issued at a frequency of 50 to 150 pulses per second. Equipment of 50 or more kHz is not recommended for use on concrete less than 100 mm thick, and 20 kHz should be limited to use on sections exceeding 300 mm in thickness. Higher frequencies, though more sensitive to voids, are subject to higher attenuation. The results are shown on a digital readout in milliseconds and reflect the time sound travels from one probe to the other. The probes are held against the concrete surface, and the contact area is filled in with grease or some other medium to close any acoustic gaps. This procedure makes for a somewhat messy test environment. The precise distance between the two probes must be known in order to make inferences about the soundness of the concrete under study.

2.9.7 CHLORIDE ION CONTENT

Chloride ion content testing is a method of determining the likelihood of corrosion of the reinforcing steel in concrete. Quantities of chloride ions of 0.03% by weight of concrete are considered sufficient to initiate corrosion of rebar.

Until fairly recently, testing for chloride ion in concrete had to be done in a laboratory with samples taken in the field. However, during the US Federal Highway Administration's Strategic Highway Research Program (SHRP) methods for field testing for chloride content were developed. These are commonly referred to as Rapid Chloride Tests (RCT). The field testing gives chloride content readings in the field and allows for adjusting the number and locations of the chloride testing while in the field. However, samples are generally also taken in the field for laboratory testing to confirm field results.

For both the Rapid Chloride and Laboratory Tests, proper sampling of the concrete in the field is very important to the accuracy of the results. More details on the chloride testing methods used by Alberta can be found in Chapter 4 of the Level 2 BIM Inspection Manual.

2.9.8 CSE TESTING

Copper Sulfate Electrode (CSE) test is a repeatable, non-destructive field test that measures the electrical potential between the steel reinforcement of the bridge and a reference electrode. ASTM C876 describes this method of estimating the electric half cell potential of reinforcing steel for the purpose of determining its corrosion activity. It consists of a copper-copper sulfate half cell, which is a dielectric container holding a copper sulfate solution surrounding a copper rod. The copper sulfate is brought into electric contact with the wetted concrete surface by way of a sponge. The other terminal coming from the copper rod is connected by a lead to the rebar-network of the concrete and cannot function if the rebar is epoxy coated, discontinuous, or if there is an impervious insulating membrane like asphalt on the concrete. The copper-copper sulfate cell provides a standard reference in the measurement of the potential of corrosion half cells that exist between the rebar and the surrounding concrete. A high impedance voltmeter cut into this circuit yields indications as to the corrosion activity in the rebar. Readings are taken to the nearest 0.01 V, and are given in negative values by convention. Readings of numerically less than -.20 V CSE indicate a greater than 90% probability that no corrosion is present. Readings of -0.35 V CSE and those numerically above this value indicate a greater than 90% probability that corrosion is present.

Since 1977, the Department has used CSE testing to help evaluate the condition of concrete bridge decks. The Department has made some modifications to the ASTM-C876 test method. More details on the Alberta CSE testing method can be found in Chapter 3 of the Level 2 BIM Inspection Manual.

2.9.9 THERMOGRAPHY

Infrared thermography has been used to detect delaminations in bridge decks and other concrete structures. This method exploits the thermal gradient in concrete resulting from heating and cooling cycles. It is a result of differential lags in the transfer of heat through the concrete and other media such as air in areas of discontinuity. Thus, during periods of heating, the surface temperature of delaminations is higher than that of surrounding concrete, whereas at night, during cooling, the surface of delaminations is cooler than its surroundings. The temperature differences are picked up by infrared sensitive photographic film, which is exposed from a platform raised 4 to 6 m above the deck. In this application the photographs can cover a full lane width at a pass. However, despite some very promising results, the method is considered not quite as good as chain dragging.

2.9.10 RADAR

The development of low power, high frequency pulsed radar equipment in the 1960's has made possible the detection of small flaws in bridge decks no matter whether they are covered by an asphalt wearing course or not. The pulses are approximately one nanosecond in duration and are transmitted by a monostatic antenna (combining transmission and receiving tasks). The transmitter receiver processes the signal returned from the concrete into the antenna through an oscilloscope which allows interpretation of the resulting returns in terms of discontinuities on the basis of differing dielectrics.

In addition to the discovery of voids and other discontinuities of the deck, claims are made that the radar can measure the thickness of the asphalt overlay. However, the interpretation of the massive raw data produced by this method requires considerable experience in reading radar signatures and relating them to physical distress in bridge decks. On the other hand, this method is temperature independent, yielding good results in summer or winter, its only limitation being a very wet deck surface, which causes attenuation of the signal. Attenuation means the signal is powering out before a reading is produced.

The method is currently being used in the investigation of the Department's roadways for inventory purposes and pavement profiling. However, it has given somewhat inconclusive results when used to detect damage in bridge decks.

2.9.11 OTHER TESTING METHODS

There are other field testing methods for concrete such as In Situ Testing by Microwave, Radiography and Electrical Resist of Membrane Pavements. These are very specialized test methods and not commonly used in Alberta.

2.10 CONCRETE GIRDERS

At the end of this section are lists of most precast and prestressed girder types that will be encountered by a bridge inspector in Alberta. At the beginning of this section are some information and general observations on the concrete girders that are common to all these types.

2.10.1 PRECAST GIRDERS

For the sake of convenience concrete girders have been classified as precast and prestressed, although most of the latter are also precast. Precast girders by our definition are concrete girders that are manufactured at a central plant using only ‘mild steel reinforcing’ and then are transported by truck to the bridge site. Concrete members that are cast at the site in their final position are called ‘cast-in-place’, abbreviated ‘CIP’. .

2.10.2 PRESTRESSED GIRDERS

'Prestressed' denotes a special kind of reinforcing method that involves the use of high tensile steel strands that are tensioned to produce compression forces in the girder concrete. These girders can be ‘pre-tensioned’ in which the steel strands are pulled/tensioned in a stressing bed before the girders are cast in a form and the strands are cut after the concrete has hardened transferring compression forces to the girder or they can be ‘post-tensioned’ in which ducts are provided in the girders at the time of casting and steel strands are placed and tensioned against the ends of the girders after the concrete has hardened. The ducts in these girders are generally grouted after the stands are tensioned.

2.10.3 PROTECTION OF REINFORCEMENT

All prestressed and precast girders depend on the embedded steel reinforcement to pick up tensile stresses. Reinforcing bars are usually protected from corrosion by a certain amount of clear cover of concrete, ranging from 25 mm to 50 mm or more. The concrete tends to provide an alkaline environment of up to pH=13, which prevents rusting.

In the case of prestressed girders, the design requirements of this method have the steel strands exiting the end of the member at some time during manufacture, and only later are the cables cut off, ground back, and the ends grouted in to protect them from the elements. If this grouting is not done properly water may get access to the ends of the strands and moisture may travel down the strands to critical locations along the girders. Unfortunately, in most cases the ends of prestressed girders are not easy to inspect, since they are normally obstructed by other girder ends or by the abutment. Inspectors should check the ends of these girders as much as possible and look for signs of leakage and deterioration that might indicate this type of problem.

Concrete girders with normal mild steel reinforcing are designed with less reinforcing steel in tension than is required to develop the full compression strength of concrete. This is commonly referred to as an ‘under reinforced beam’. This is done so that in failure mode the reinforcing steel will yield and the girder will sag developing very large detectable cracks in the tensile zone which will give lots of warning of failure.

However, prestressed girders do not behave this way and steel strands that are subjected to damage by corrosion or high load hits can fail suddenly and, if multiple strands break, there can be a sudden collapse of the girder. Inspectors need to inspect these girders very closely for any corrosion straining, spalling or any other evidence of problems with the steel strands.

One methods of detecting the failure of stressing strands is by Acoustic Emission Monitoring which can record the elastic energy of the breaking strand. This method has been used on steel structures for many years but is not commonly used on prestressed concrete girders.

2.10.4 HIGHWAYS LOADINGS

The allowable live load on highways has increased significantly over the past fifty years. New concrete girders have been designed for these higher live loads but many older girders were only designed for lower live loads. In Alberta these older concrete girders have been evaluated for live load capacity. Some of these girders have been strengthened and/or improved analysis techniques have shown these girders have additional load sharing capacity which can carry these increased loads. Where girders have not been strengthened or have not been found to have extra capacity, the bridge has been posted for lower then present legal live loads.

2.10.5 LOAD SHARING

Load sharing offers a possible increase in bridge loadings due to the fact that some of the applicable codes did not allow the benefits of load sharing to be taken into account when designing the strength of older precast concrete girders. In other words, such a girder was designed to take the full impact and load from a set of left or right truck tires without passing some of this load on to neighboring girders through connections such as rods, bolted flanges, welded connections, or grouted candy cane rebar connections. Later designs called for field-cast diaphragms with normal reinforcing bars and lateral post-tensioning connections. Even though the design sometimes did not allow for the benefits of load sharing, they are exploited at every opportunity, either at time of manufacture or by retrofit joining methods.

2.10.6 EARLY CONCRETE GIRDERS

Precast girders in Alberta are designated by a single or double letter indicating the girder type, and by a two or three letter abbreviation utilized by the computerized bridge information system (BIS). 'A'-girders, which are described below as a channel with end blocks, resemble an inverted, shallow bath tub, as do G, E, and H girders (however, the term bath tub girder is reserved for a larger, more recent girder type). Type 'A' girders were used until 1952. In those days designers intended for the most part to produce a simple bridge building method that could be carried out in winter, without the use of cast in place concrete, welding, or other temperature-sensitive installation devices. The idea was to bridge a pair of abutments and possibly a number of timber pile bents with a number of precast girders that could be hoisted into place with a small boom crane attached to a transport truck. At that period of time, calculations required for load sharing were very complicated and the state of the art for shear design did not provide for proper stirrups which were either absent or lacked hooks that wrapped around the main reinforcement. Main steel was spot-welded to the stirrups, a method that was later ruled out due to possible loss of section in the bars. These early bridge girders performed quite well on gravel roads, but exhibited considerable distress once these roads were paved and exposed to deicing chemicals, beginning in 1960.

The year 1952 marked a change-over to H20-S16 loadings, which was accomplished with a change from 'A' girders to 'G' girders. 'G' girders in turn gave way to HC girders in 1961. Recently, 'G' girders have been the subject of considerable attention when it was suspected that they may have insufficient shear strength at the end blocks due in part to improper or insufficient stirrups. They resemble 'A' stringers in utilizing the inverted bath tub shape, and were produced like the 'A' stringers in lengths up to 8.5 m (28'). Experimental evidence has since shown that this girder type will safely carry 16 tonne axle loadings on 8.5 m spans, and 20 tonne axles on 6.1 m (20') spans.

More information and details on some of these early concrete girders can be found on the Department's Standard Drawing S-534.

2.10.7 CURB UNITS

Early curb girders typically have a cross-section which will accommodate the impact of run-away vehicles, guard rails and utility lines. On the other hand, these girders receive very little normal traffic load. These curb units are subject to lateral displacement by grader strikes and other traffic impacts, and connections to adjoining girders have been a major concern ever since the early days of precast girder installations.

In 1958 the 8.5 m curb 'G's were made of light weight concrete weighing 1681 kg/cu.m in order to allow for installation by the usual truck-mounted boom crane, rather than by special heavy duty cranes. However, in 1961 the use of lightweight concrete in precast girders was restricted to secondary and local roads when it was discovered that this material was especially prone to break up under heavy traffic and salt action. However, a few years later, semi-lightweight concrete in which only the coarse aggregate is lightweight was adopted for use on primary highways.

2.10.8 GIRDER CONNECTIONS

There have been a variety of bolted connections tried on early precast girders, some of which were abandoned when it was discovered for instance that holes provided in the girders could not be lined up during erection. The first girder fully designed to be connected to its mates was the 'HC', 'C' symbolizing the connection. In 1974, the 'HC' was in turn replaced by the VS, which was 510 mm deep, and also departed from the inverted bath tub shape to become a closed box containing hollow voids and prestressed cables. The VS girder was replaced in 1979 by its metric counterpart, the SM. In 1990 the SM girders were replaced with SC girders which remains the major girder type for spans up to 12 m. Connections are made by channel pockets.

Other longer prestressed girders were connected with grout keys and reinforcing steel candy cane shape bars. These include the Type ‘M’ box girders with spans up to 15.2 m, the Type ‘RD’ and ‘RM’ box girders with spans up to 25.4 m, and the F(Fernich) series girders (Type ‘FC’, ‘VF’ and ‘FM’) with spans up to 38 m.

2.10.9 PROBLEMS WITH CONNECTIONS

The pocket type of connection used in 'SC' and its predecessors proved to be somewhat less than satisfactory in that the bolts connecting the pockets worked loose and then allowed differential movements of adjoining girders. The pockets are relatively small, less than a square foot in size, and somewhat infrequent along the length of the girder. Re-tightening is only possible after the wearing course and grouting material is removed from the pockets. Pockets have also been welded together in order to enhance the connection.

The grout key/candy cane connections have also had problems. In some respects the grout key connection (candy cane) resembles a zipper. In girders exceeding a length of 30 m this candy cane connection is prone to fail, usually beginning with the 2nd or 3rd girder connection from the curb, due to differential live load deflections. This type of failure is assisted by the spacing of the candy canes. Installation requirements preclude close proximity of opposing canes as this arrangement would result in candy cane collisions during the approach of two adjoining girders during erection. For this reason the candy canes are spaced so that they will fall into the middle between two adjoining canes when the girders are joined. The slack in shear resistance was then supposed to be made up by inserting a long rod into the circle described by opposing hooks before grouting the small blocked-out space at the top outside edges of the girders containing the rows of candy canes. Since this grout key was stressed in flexure rather than in tension, it often let go. Impending failures of this kind are noticeable by raveling and other delaminations in the grout key.

2.10.10 CONCRETE OVERLAYS

Since 1977 approximately 250 bridges have undergone major deck repairs followed by a concrete overlay. The purpose was to replace the deteriorated concrete and then to provide some waterproofing of the bridge deck to prevent or lessen future problems. The concrete overlays were generally in conjunction with new waterproof deck joints. The types of concrete overlays used have evolved over the years.

High Density Overlays
The first concrete overlays were referred to as high density overlays. These were a special concrete mix design with very low water/cement ratio and zero slump. It had to be put down with a special finishing machine. Over time it was shown that these overlays developed cracks and were more porous than originally assumed.

Silica Fume Overlays
With the development of silica fume as an admixture that improved the impermeability of concrete, silica fume overlays were developed for use on bridge decks. Although these overlays proved to be less permeable than the high density overlays, the overlays still develop cracks.

Silica Fume Fibre Reinforced Overlays
To help prevent and control the cracking, short steel fibres were then introduced to the concrete mixture. This seemed to improve the cracking problem particularly when combined with better and longer curing techniques.

High Performance Fibre Reinforced Concrete Overlays
Recent developments with high performance concrete have shown a fly ash used with silica fume can improve the quality of the concrete overlay. The Department has now developed a specification for a high performance fibre reinforced concrete overlay using silica fume and fly ash. This specification also requires fog misting of the concrete overlay while it is being placed and seven days of moist curing.

2.10.11 PRECAST CONCRETE STRENGTH

The design strength of the first precast units was 27.6 MPa (4000 psi) and increased to 34.5 MPa (5000 psi) in the late 50's for type 'E' girders, which were extended to span lengths of up to 12.8 m as compared to the 8.5 m of maximum length of 'A' and 'G' girders. When HS-25 design loading was introduced in 1973, the required release strength increased to 27.6 MPa and also the 28 day strength increased to about 41.4 MPa (6000 psi). Currently for NU girders, the required release and 28 day strengths are 40 MPa and 55 to 70 MPa respectively.

2.10.12 TABLE OF PRECAST GIRDERS

In the following tabulation of girder types the symbol ">" denotes sequential developments or metrication of a given girder model, e.g. HH > HC > VH means that the VH was developed from the HC, which itself was a further development of the HH girder model.

2.10.12.1 Type A. Abbreviation: PA

Description:	Channel with end blocks. Slanted inside walls.
Concrete Specification:	4000 psi, Air: 3 - 6%, Normal Weight
Connection:	Some bolted. 'Peterson fix' involved retrofit hollow connection bolt capable of filling void space between girder legs with grout.
Dimensions:	L = 16', 20', 28' W = 3' D = 16"
Years of Production:	1950 to 1952
Design Loading:	H 15-S12
Problems:	Design at one point had called for a long rod connecting girders laterally across the whole roadway. Installers had troubles lining up the holes for the rod and omitted it frequently. The design was for a relatively light loading and required revision with the advent of heavier trucks.
Service:	Inspection is to refer to girder mark and if possible, to indicate condition of connector bolts near mid span. Observe load sharing between girders near mid span under live loads, and look for flexural cracks along girder legs at main steel level, normally 100 mm from the bottom of leg. Look for shear cracks close to the supports, and check for spalled girder legs.

2.10.12.2 Type G. Abbreviation: PG

Description:	Inverted open box channel with end blocks.
Concrete Specification:	5000 psi, Normal and lightweight concrete; Air: 3 - 6 %.
Connection:	None except some curb stringers were connected to prevent grader push out. This type was referred to as 'MC'.

Dimensions:	L = 20', 28' W = 3' D = 16"
Years of Production:	1953 to 1960
Design Loading:	H 20-S16
Problems:	Insufficient number of stirrups. Presumed insufficient shear strength in end diaphragms. Lack of load sharing. Gravel entered between girders wedging them out of plumb. Stringer legs spalling at bottom.
Service:	Tests carried out in 1986 showed girders were safe - 28' girders good for 16 tonne axles, 20' good for 20 tonne axles. There have been no failures in service to date. Inspection should be similar to 'A' girders. Check for diagonal cracks indicative of torsional stresses in the endblocks. These can only be inspected routinely from the inside. Check stringer legs for vertical cracks, changing to diagonal cracks toward the ends. A program of load sharing is underway by connecting girders with a 100 mm reinforced concrete overlay. Some stringers have been repaired with fibre-reinforced shotcrete. Check for spalled girder legs.

2.10.12.3 Type E. Abbreviation: PE

Description:	Channel. Outside of legs slightly slanted.
Concrete Specification:	5000 psi; Air: 4.5% + or - 1%.
Connection:	Candy cane grout key.
Dimensions:	L = 20' to 42' W = 3' D = 24"
Years of Production:	1952 to 1965
Design Loading:	H 20-S16, 3/5 wheel load per stringer.
Problems:	Candy cane connection is coming apart in places. Notice that the load design is predicated on the ability to share one fifth of the H20-S16 loading on either side.
Service:	This was a strengthened version of the Type 'G' girder, with the addition of a grouted connection. Inspect same as Type 'G', and sound out grout keys with a hammer.

2.10.12.4 Type H. Abbreviation: HH > HC > VH

Description:	Channel with end blocks.
Concrete Specification:	Normal weight and light weight concrete. HC 4000 psi., VH 4500 psi. Minimum Air: 5%
Connection:	HH had no connection. HC & VH had channel connectors, which are bolted pockets.
Dimensions:	HC L = 20', 28', 33', 38' W = 3'

	D = 20"
	VH L = 20', 28', 33', 38'
	W = 3'
	D = 22"
Years of Production:	HC = 1961 to 1974
	VH = 1973 to 1974
Design Loading:	HC = HS 20-44
	VH = HS 25
Problems:	There are problems with the connector pockets. The connecting bolts proved inadequate. Salt collected in the pockets causing deterioration of steel and adjoining concrete.. Girder legs are spalling at the bottom.
Service:	Some girder legs have been repaired with shotcrete. The 'V' in 'VH' stands for 5 and denotes the HS-25 loading, which was achieved by lengthening the legs 2". Channel connectors were welded together later to improve the connection between girders. Some of these girders have been repaired with a 50-65 mm concrete overlay. Prior to the overlay U connectors were installed and grouted in between each girder at 1 m intervals. In a few cases drill holes for the connectors were misaligned so that U connectors exit on the side of the girder leg. Record such occurrences.

2.10.13 TABLE OF PRESTRESSED GIRDERS

2.10.13.1 Type O. Abbreviation: PO > OM

Description:	This is a non-adjacent I-girder requiring field casting of a deck between girders. The bottom flange of the I-section is thickened to a bulb. The I is fused to form a rectangular end block to resist shear.
Concrete Specification:	42 MPa. Semi-lightweight concrete.
Connection:	By cast-in-place slab.
Dimensions:	PO L = 28' to 148' (dimensions are re-calculated for each bridge)
	W = 2' approx.
	D = 4' approx.
	OM L = 31 m, 40 m
	W = 850 mm
	D = 1800 mm
Years of Production:	PO = 1955 to 1965
	OM = 1980 to 1981
Design Loading:	PO = H 20-S16
	OM = MS 230
Problems:	Deterioration of the grout patches over the post-tensioning anchorages at the ends of the girder.
Service:	OM is the metric version of PO.

2.10.13.2 Type F (FENRICH). Abbreviation: FC > VF > LF > FM

Description:	Channel with picture frame sides. Legs have thick bottom ends. End blocks present.
Concrete Specification:	FC = 5000 psi VF = 5500 psi FM = 35 MPa (Curb-38 MPa) Concrete = Normal weight and semi-lightweight; Minimum Air: 5%
Connection:	Candy cane grout key. Legs of interior units were bolted together.
Dimensions:	VF L = 52' W = 5'-4" D = 4'-6" (varies) FM L = 22 m to 38 m W = 1625 mm D = 1170 mm (varies)
Years of Production:	FC = 1961 to 1975 VF = 1973 to 1977 LF = 1976 to 1979 FM = 1978 to 1983
Design Loading:	FC = HS 20-44 VF = HS 25-44 LF = HS 25-44 FM = MS 230
Problems:	Zipper-like failures in grout keys, due to corrosion of candy cane bars, sometimes at age 5 years or less. There was also a suggestion that this girder type needs more dead load to improve the ratio between live load and dead load.
Service:	After the grout key failure of some of these longer units, a number of repair procedures for improving the lateral connection of the girders were developed. These include lateral post-tensioning through the girder legs, underslung diaphragms, lateral post-tensioning through the deck or combinations of these.

2.10.13.3 Type M. Abbreviation: PM > VM

Description:	Closed box with tubular voids and slightly slanted sides.
Concrete Specification:	5000 psi. Normal weight concrete. Minimum Air: 5%
Connection:	Candy cane grout key.
Dimensions:	L = 50' W = 3' D = 24"
Years of Production:	PM = 1963 to 1973 VM = 1974
Design Loading:	PM = HS 20-44; VM = HS 25

Problems:	Failure of some candy cane connections. In skewed applications the girders sometimes refused to sit flat on abutments during erection.
Service:	This girder worked well. Inspect as before.

2.10.13.4 Type VS. Abbreviation: VS > SM >SC

Description:	Closed rectangular box with tubular voids.
Concrete Specification:	VS = 5000 psi, SM = 35 MPa; Semi-lightweight.
Connection:	Channel connections (bolted pockets).
Dimensions:	VS L = 20', 25', 30', 35' W = 4' D = 1'-8" SM L = 6 m, 8 m, 10 m, 11 m W = 1206 mm D = 510 mm SC L = 6 m, 8 m, 10 m, 12 m, 14 m W = 1206 mm D = 510 mm
Years of Production:	VS = 1974 to 1979 SM = 1978 to 1989 SC = 1990 to present (2001)
Design Loading:	VS = HS 25-44; SM = MS 23 & 225; SC = CS750
Problems:	Bolts in connector pockets proved to be inadequate. Some pockets were therefore welded together. Voids tend to float up during pour.
Service:	The SM is the metric version of the VS. Some applications were modified by adding a connecting slab to the top which is tied to the SM girders by rebar dowels to form a composite deck (SMC). In 1990, SM girders were modified to SC girders to accommodate the new CS 750 design loading. SC girders with projecting rebar to form a composite deck are designated as SCC girders. Inspect as before.

2.10.13.5 Type RD. Abbreviation: RD > RM

Description:	Closed rectangular box with rectangular voids.
Concrete Specification:	5000 psi; Semi-lightweight concrete
Connection:	Candy cane grout key.
Dimensions:	RD L = 30' to 80' W = 4' D = 2'-6" RM L = 12 m to 28 m W = 1206 mm (varies) D = 610 mm to 915 mm
Years of Production:	RD = 1974 to 1979 RM = 1980 to 1981
Design Loading:	RD = HS 25-44

	RM = MS 23 (0.7 wheel line per girder)
Problems:	Sometimes cracks developed in endblock during stressing. Also in skewed applications girder would not sit flat on abutments during installation.
Service:	Load sharing is assumed at 15% on either side of each girder. This requires checking of the candy cane grout key. Also check the end block for stressing cracks. Otherwise inspect as before.

2.10.13.6 Type Deck Bulb T. Abbreviation: DBT > DBC

Description:	T-girder with thickened bulb-shaped leg.
Concrete Specification:	1800 Series uses semi-lightweight in the bulb and webs. Remaining concrete is normal weight 35 MPa and 38 MPa.
Connection:	Post-tensioning laterally through top flange of T. Field-cast end and intermediate diaphragms utilizing reinforcing steel tying to Richmond inserts in curb members.
Dimensions:	DBT L = 10 m to 42 m W = 1.2 m to 2.0 m D = 0.8 m to 1.8 m DBC L = 18 m to 42 m W = 1.2 m to 2.0 m D = 0.8 m to 1.8 m
Years of Production:	DBT = 1981 to 1989 DBC = 1990 to 2007
Design Loading:	DBT = MS 300; DBC = CS 750
Problems:	Cracks along longitudinal grout keys have been observed in some bridges and is a potential problem.
Service:	The improved connecting detail reflects current state of the art. Connections require specific inspection, especially at ends of lateral tendons.

2.10.13.7 Type NU. Abbreviation: NU

Description:	Spaced apart. I-shaped girders require field casting of concrete deck. The bottom flange is 975 mm to 1000 mm wide to accommodate a large number of strands and provide added stability.
Concrete Specification:	55 MPa to 70 MPa; Normal weight concrete
Dimensions:	L = up to 65 m W = 1225 mm (prestressed), 1250 mm (post-tensioned) D = 1200 mm to 2800 mm
Years of Production:	2000 to Present (2007)
Design Loading:	CS750 & CL800