The material presented in this paper is based on more than 30 years of research, observations and experience concerning causes, control, and consequences of cracking in concrete structures. This extensive background was helpful in the preparation of this paper which deals with questions of concrete cracking.

The presence of cracking does not necessarily indicate deficiency in strength or serviceability of concrete structures. While currently available design code provisions lead to reasonable control of cracking, additional control can be achieved by understanding the basic causes and mechanisms of cracking.

**CAUSES OF CRACKING**

Concrete can crack due to a number of causes. Some of the most significant causes are discussed in detail.

**Tensile Strength of Concrete**

The tensile strength of concrete is a widely scattering quantity. Cracking occurs when tensile stresses exceed the tensile strength of concrete. Therefore, to control concrete cracking, the tensile strength of concrete is of primary im-
portance. Laboratory test data conducted by H. Rüsch were analyzed statistically. As presented in Ref. 1, this analysis furnished the following relationships for the mean direct tensile strength, $f_{tm}$, related to the 28-day compressive cylinder strength $f_{cm}$ of concrete:

\[ f_{tm} = 2.1 \left( f_{cm} \right)^{2/3} \text{(psi)} \]

\[ f_{tm} = 0.34 \left( f_{cm} \right)^{2/3} \text{(N/mm²)} \]

The statistical analysis indicated that the coefficient in this equation can be modified to 1.4 (0.22) and 2.7 (0.45) to obtain the 5 and the 95 percentiles, respectively, of the tensile strength, $f_t$. The tensile strength of concrete is slightly higher in flexure. However, it is recommended that values for direct tension be used in practice. Concrete cracks when the tensile strain, $\varepsilon_{tt}$, exceeds 0.010 to 0.012 percent. This limiting tensile strain is essentially independent of concrete strength. The 5 percentile of the tensile strength, $f_{t5}$, should be used in design to locate areas in the structure that are likely to crack by comparing calculated stresses with the expected concrete strength. The 95 percentile, $f_{t95}$, should be used to obtain conservative values for restraint forces that might occur before the concrete cracks. These restraint forces are used to calculate the amount of reinforcement needed for crack width control.

Causes of Cracking During Concrete Hardening

Concrete cracking can develop during the first days after placing and before any loads are applied to the structure. Stresses develop due to differential temperatures within the concrete. Cracking occurs when these stresses exceed the developing tensile strength, $f_t$, of the concrete as indicated in Figs. 1 and 2. Differential temperatures are mainly due to the heat of hydration of cement during concrete hardening. This effect is usually neglected except in massive structures as indicated in Ref. 2. However, depending on cement content and type of cement, the temperature within concrete members with dimensions of 12 to 36 in. (30 to 91 cm) can increase approximately 36°F to 108°F (20°C to 60°C) during the first 2 days after casting.

If concrete members are allowed to cool quickly, tensile stresses may reach values higher than the developing tensile strength of the concrete. Even if this process results only in microcracking, the effective tensile strength of the hardened concrete is reduced. However, very often wide cracks appear even when reinforcement is provided. In addition, the reinforcement may not be fully effective since bond strength is also developing and is yet too low. It is necessary to minimize such early cracks by keeping temperature differentials within the concrete as low as possible. This can be done by one or more of the following measures:

1. **Choice of cement** — A cement with low initial heat of hydration should be selected. Table 1 shows that there is a significant variation in heat develop-
compression

T_{air}

T_{\text{concrete}}

\Delta T

\alpha T E_c

G

tension

Internal stresses in equilibrium

Fig. 1. Temperature distribution due to heat of hydration and internal stresses caused by outside cooling in a free standing concrete block.

ment among different types of cements. The cement content of concrete should be kept as low as possible by good grading of the aggregates. Heat development can also be reduced by adding fly ash or using slag furnace cement.

2. Curing — Evaporation of water must be prevented by using curing compounds or by covering the concrete with a membrane. Rapid evaporation can lead to plastic shrinkage cracking.

3. Curing by thermal insulation — Rapid cooling of the surface must be prevented. The degree of thermal insulation depends not only on the climate, but also on the thickness of the concrete member and on the type of cement used. Spraying cold water on warm young concrete, as it was done years ago, is not recommended.

4. Precooling — This is a necessity for large massive concrete structures such as dams. For more usual structures, in which shortening after cooling can take place without creating significant restraint forces, precooling is expensive and unnecessary. In this case, thermal insulation is preferable and it also has the benefit of accelerating concrete strength development. An exception may be made in very hot climates since precooling can keep concrete workable for a longer period of time.

Often shrinkage is considered as a cause of early cracking. However, this is not true under normal climatic conditions. Shrinkage needs time to produce a
Table 1: Heat of hydration of various types of cements.*

<table>
<thead>
<tr>
<th>Type of cement</th>
<th>Heat of hydration (Btu/lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 day</td>
</tr>
<tr>
<td>I</td>
<td>92</td>
</tr>
<tr>
<td>II</td>
<td>76</td>
</tr>
<tr>
<td>III</td>
<td>139</td>
</tr>
<tr>
<td>IV</td>
<td>50</td>
</tr>
<tr>
<td>V</td>
<td>58</td>
</tr>
</tbody>
</table>

*Data obtained from *Concrete Manual*, U.S. Bureau of Reclamation, 1975, pp. 45-46.
†Federal Specifications SS-C-192G, including Interim Amendment 2, classified the five types according to usage as follows: Type I for use in general concrete construction when Types II, III, IV, and V are not required; Type II for use in construction exposed to moderate sulfate attack; Type III for use when high early strength is required; Type IV for use when low heat of hydration is required; and Type V for use when high sulfate resistance is required.
Note: 1.0 Btu/lb = 2.32 J/g.

shortening as high as the tensile rupture strain. Only in very hot and dry air shrinkage can cause early cracks in young concrete, if measures against evaporation are not applied.

Causes of Cracking After Concrete Hardening

Tensile stresses due to dead and live loads cause cracking. Normal rein-
Deformed Shape considering Upper Face of Beam Warmer Than Bottom Face and assuming beam freed from interior supports

Fig. 3. Forces in a concrete beam due to a temperature rise $\Delta T$ at the upper face of the beam and external restraint provided by interior supports.

Forcement or prestressing should be designed to provide required strength and keep crack widths within permissible limits. Tensile stresses due to service loads can be controlled by prestressing. The degree of prestressing can be chosen based on structural or economic considerations. Normally, partial prestressing leads to better serviceability than full prestressing.

Cracks can also be initiated by tensile stresses due to restrained deformations from temperature variations or from shrinkage and creep of concrete. Imposed deformations such as differential settlement between foundations can also cause cracks.

There are two types of restraint which cause stress in concrete members, namely, internal restraint as shown in Fig. 1, and external restraint in indeterminate structures, as shown in Fig. 3. Restrainted deformations caused cracking in concrete bridges and it was primarily due to temperature differences produced by heating under the sun and cooling during the night. Extreme temperatures that occur at 20 to 50-year intervals must be considered. As indicated in Refs. 3, 4, 5 and 6, temperatures in bridge structures were measured in several countries. Recently, the U.S. Transportation Research Board published in Ref. 7 temperature data for bridge design.

Temperature differentials should be considered along with recommended mean temperatures, $T_m$, used for calculating maximum and minimum changes in the lengths of structural members. In Central Europe values for $T_m$ are specified for concrete bridges as varying from +68°F to –22°F (+20°C to –30°C).

The temperature distribution over a beam cross section can be subdivided into three parts as shown in Fig. 4. The constant part, $\Delta T_1$, causes axial forces if overall length changes are restrained. The linear part, $\Delta T_2$, causes restraint forces, $M_{\Delta T}$ and $V_{\Delta T}$, in indeterminate structures as shown in Fig. 3 for a three span continuous beam. The nonlinear part, $\Delta T_3$, causes stresses, which are in
Table 2. Recommended cross section temperature differentials for bridge design in Europe.

<table>
<thead>
<tr>
<th>Type of cross section and exposure</th>
<th>Box girder</th>
<th>T-beams</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maritime</td>
<td>Continental</td>
</tr>
<tr>
<td>Top of cross section warmer than bottom (°F)</td>
<td>18</td>
<td>27</td>
</tr>
<tr>
<td>Bottom of cross section warmer than top (°F)</td>
<td>9</td>
<td>14.4</td>
</tr>
</tbody>
</table>

Note: 1.0°F = (9/5)°C.

equilibrium over the cross section and produce no action forces. These stresses, which also exist in statically determinate structures, can be calculated by imposing equilibrium conditions and considering that:

\[ f_{CT} = \Delta T \alpha_T E_C \]

where \( \alpha_T \) is equal to \( 6 \times 10^{-6}/°F \) \((10^{-5}/°C)\), the coefficient of thermal expansion for concrete. Cooling causes tensile stresses in areas near extremities of the section.

For bridges in Europe, the \( \Delta T \) values given in Table 2 are recommended. In addition to temperature, restrained concrete creep and shrinkage can cause stresses. Shrinkage often leads to cracks between connected members of significantly different sizes. Stress due to restrained creep and shrinkage can be cal-

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**Fig. 4.** Division of temperature diagram into its constant, linear and nonlinear parts.
Fig. 5. Transverse cracks in thin bottom slab of box girder due to differential temperature, creep and shrinkage despite prestressing.

culated in the same way as stresses due to temperature.

Transverse cracks due to temperature, creep and shrinkage effects are frequently found in the relatively thin bottom slabs of box girders despite the fact that calculations show considerable longitudinal compressive stresses due to prestressing. Compressive stresses tend to shift towards the thick webs which undergo less creep and shrinkage strains as illustrated in Fig. 5.

Box sections are indeterminate structures. Therefore, restraint moments are developed when the section is heated on one side by the sun. This leads to vertical cracking in bridge piers and tower shafts as shown in Fig. 6. Ref. 8 shows examples of temperature cracks in prestressed concrete structures.

**Determination of Areas Likely to Crack**

Cracking occurs whenever the principal stresses due to service loads or due to restraint forces or due to a combination of service loads and restraints exceed the tensile strength of concrete. These stresses can be calculated using the linear theory of elasticity, considering the structure initially uncracked. In these calculations, $f_{ct}$ should be taken as the tensile strength of the concrete. In the tension side of a beam, cracking will occur in areas where bending moments due to service loads and restraint cause stresses in the extreme tensile fiber above $f_{ct}$. As bending increases, the depth of cracking can be calculated by considering a maximum concrete tensile strain of 0.015 percent as shown in Fig. 7.

Calculation of possible maximum bending moments due to restraint should be based on $f_{ct}$. As shown in Fig. 8, consideration of such moments increases the areas in which cracking may be expected to occur.

Bending moments due to restraint define only the location and quantity of reinforcement or prestressing necessary to limit the crack width for serviceability purposes. As proven long ago by Priestley, and illustrated in Fig. 9, these moments do not decrease the ultimate strength of the structure because they are reduced and finally disappear due to cracking and plastic deformation as service loads are increased until the
limit state is reached. However, the structure must be checked for possible brittle failure of the compression zone if a relatively high degree of prestressing is used, especially for continuous T-beams. Therefore, to satisfy strength requirements, bending moments due to restraint should not be added to moments due to service loads in sizing of main reinforcement. It must, however, be observed that restraint due to prestressing does not decrease on the way up to limit state.

Restraint forces decrease beginning with the first crack since the stiffness of the structure is progressively reduced.
with each crack that occurs. Steel stresses due to restraint are highest when the first crack occurs and decrease with each further crack. This tends to reduce crack widths. Fig. 9 shows the effect on moment due to reduction of restraint.

**Evaluation of Cracks**

As indicated in Fig. 10, crack widths are greater at the surface and decrease towards the reinforcement. Long years of research reported in Refs. 9 and 10, and experience indicate that crack

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**Fig. 9. Illustration of reduction of restraint forces as the limit state is approached.**

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Ag. 10. Crack width at the surface is used as a measure of the effect of cracking on concrete members.

Table 3. Allowable crack widths.

<table>
<thead>
<tr>
<th>Ambient condition of exposure†</th>
<th>( w_{90} )† (in.)</th>
<th>Maximum ( w ) permitted (in.)</th>
<th>Crack appearance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mild</td>
<td>0.012</td>
<td>0.020</td>
<td>Easily visible</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.008</td>
<td>0.016</td>
<td>Difficult to see with the naked eye</td>
</tr>
<tr>
<td>Severe</td>
<td>0.004</td>
<td>0.0012</td>
<td></td>
</tr>
</tbody>
</table>

* \( w_{90} \) denotes the 90 percentile of the crack width, \( w \).
† Defined as indicated in the CEB-FIP Model Code:

**Mild exposure**
- The interiors of buildings for normal habitation or offices.
- Conditions where a high level of relative humidity is reached for a short period only in any one year (for example 60 percent relative humidity for less than 3 months per year).

**Moderate exposure**
- The interior of buildings where the humidity is high and where there is a risk for the temporary presence of corrosive vapors.
- Running water.
- Inclement weather in rural or urban atmospheric conditions, without heavy condensation of aggressive gases.
- Ordinary soils.

**Severe exposure**
- Liquids containing slight amounts of acids, saline or strongly oxygenated waters.
- Corrosive gases or particularly corrosive soils.
- Corrosive industrial or maritime atmospheric conditions.

Note: 1 in. = 2.54 cm.