Durability of Precast Prestressed Concrete Structures
This research program was instituted to determine the performance of precast prestressed concrete structures regarding durability and to identify key issues effecting their performance.

To accomplish this CEG set about surveying the industry to secure performance of a wide range of structures in terms of age and geographic location. In addition site inspections of existing structures were made, a bibliography with summaries was prepared, key issues were identified and addressed, and in conclusion a summary of good practices was developed.

The survey consisted of a detailed form evaluating various performance characteristics and the results of physical site inspections were included. The responders to the survey were asked to evaluate various factors of performance in the structure on a scale of 1-10, with 10 being the best or no attempt was made to influence the results and in fact efforts were made to secure data on older structures and those not necessarily performing the best. In this regard, the survey was successful in securing reports on 25% of the total number of structures over 15 years old and 63% of the structures located in road salt application areas and 69% located in freeze/thaw regions. The number of supported structural levels ranged from 1 to 12, and 60% of the structures are in the 3 to 6 level range. The length of structures ranged from 178 feet as the shortest to 1,200 feet as the longest.

Contained within this report is a spreadsheet summary of the results of the survey. The total number of structures reported on was 54 plus eight additional structures were physically inspected. The geographic distribution included 17 states plus the District of Columbia.

While we were only able to secure whether the structures were regularly maintained in only 13 cases, the ratings clearly indicate that those structures where regular maintenance is conducted are the best performing. As an example structures numbered 3 through 8 indicate that maintenance was being performed and all indicate the concrete products to be in good shape however, structures 3 and 5 indicate rather severe leaking which can be directly related to poor drainage systems and poor maintenance of joint sealants.

A bibliography of the Durability of Precast/Prestressed Concrete can be a very vast compendium. CEG chose to focus on ten topics deemed of key interest. For each topic, a collection of significant articles is presented as is a summary review of the contents of the articles. Designers, owners and specifiers should find this section useful.
continued...

The key issues for study were made apparent by the survey as well as practical experience gained in design and construction. Each issue is addressed with recommendations gained from the survey inspections and analysis.

The summary of good practice completes the study with recommendations based upon the total research program.

In summary, the overall performance of precast prestressed concrete parking structures can be rated as very good with the general performance rated as 84% out of 100%.

The primary areas of study center on poorer performance of field placed toppings, excessive leaking, poor drainage system design and precast spandrels showed the highest incidence of cracking of any precast products.
The first phase of the Research Program was to survey all PCI producer members across the country. The intent of the survey was to determine details, designs, materials, construction practices and maintenance practices which have been used and how effective they have been in regards to durability. We were hoping to expose common deterioration problems and practices which performed well.

Specific information requested in the survey included:

1. Age
2. Location
3. Types of products
4. Lateral load resistance system
5. Water cement ratio and admixtures used
6. Material coatings
7. Performance characteristics
   a. Degree of cracking
   b. Degree of leaking
   c. Surface deterioration
   d. Exposed plates
8. Maintenance

A cover letter included with the survey requested responses for structures which were performing poorly, well performing structures, as well as old and relatively new structures. We felt this mix would provide us with a good basis of durability practices and deterioration problems throughout the industry.

A copy of the cover letter and survey follows.
August 10, 1993

Mr.

RE: PCI Research Program - "Durability and Corrosion Protection of Precast Prestressed Concrete Structures"

Dear

The Consulting Engineers Group has been given the responsibility to conduct this PCI Research Program and respond to this important industry need. Part of the program calls for a survey of membership to determine sources of the problem as well as successful applications. You will therefore find enclosed, a relatively simple three page questionnaire. To make the program successful we really need your help in responding to this major issue. If each member can provide 2 - 4 reports, we should have a representative base. Note to properly identify the problems we need reports both on well performing structures and those not performing as well. Proper completion of the form may take a site visit and we need the reports back no later than September 3, 1993.

Responses to this questionnaire shall also be used to identify those structures which will be personally in depth inspected. Again please keep in mind the importance of this program and the value of your experiences.

Thank you for your time and effort.

Sincerely,

The CONSULTING ENGINEERS GROUP, Inc.

Thomas J. D'Arcy, P.E.
President, CEG-Texas

TJD/cls
PCI RESEARCH PROGRAM -

DURABILITY AND CORROSION PROTECTION OF PRECAST
PRESTRESSED CONCRETE STRUCTURES

- Quantify results where applicable on 1-10 scale for computer analysis with 10 - Excellent and 1 - bad with 5 being average

- Structure Descriptions
  - Structure Name__________________________
  - Type of Structure__________________________
  - Plan dimensions ______' x ______'
  - Number of levels: grade + ______
  - Location: City/State ________________________
  - Age ______ years
  - Topped/Pretopped □ Topped □ Pretopped
  - Type of products
    DT______(width______ft) + Beams______ +
    L Spandrels______ + Pocketed Spandrels ______ +
    Columns______ + Walls______
  - Lateral load system
    Shear walls______ Frames______ Shafts______ Combination
  - Lateral load system performance (Rate 1-10) (#5 = minimal cracking)
    ______
  - Tees warped? Y□ N□
  - Surface sealer? Y□ N□ Type________________
  - Salt application region? Y□ N□
  - Salt spray location? Y□ N□
  - Freeze/Thaw region? Y□ N□

Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
Observed Conditions

- Cracking (by product - Rate 1-10) (with 8 being less than 5% of products cracked)
  - Double tee flange
  - Double tee stem
  - Beams
  - Spandrels
  - Walls

- Surface deterioration (Rate 1-10)
  - Topping
  - Precast products

- Leaking (Rate 1-10)
  - Condition exposed plates

- Drainage system performance
  - Condition Joint Sealants
  - Concrete

- Bearing pad performance
  - Type of bearing pad

- Condition connections (evaluate performance - more than 5% cracked or damaged equal ≤ #3)
  - D.T. flange to flange
  - Spandrel/Column
  - Column/Beam

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Durability & Corrosion
Member Questionnaire

- **Observed Conditions**
  - Cracking (by product - Rate 1-10) (with 8 being less than 5% of products cracked)
    - Double tee flange 1 5 10
    - Double tee stem
    - Beams
    - Spandrels
    - Walls
  - Surface deterioration (Rate 1-10) 1 5 10
    - Topping
    - Precast products
  - Leaking (Rate 1-10)
  - Condition exposed plates
  - Drainage system performance (7 = 1 or 2 areas of small puddles)
  - Bearing pad performance 1 5 10
  - Type of bearing pad
  - Condition connections (evaluate performance - more than 5% cracked or damaged equal ≤ #3)
    - D.T. flange to flange 1 5 10
    - Spandrel/Column
    - Column/Beam
  - Condition Joint Sealants
  - Concrete
    - Average strength attained
    - Admixtures employed
      - Silica fume
      - DCI
      - Air
      - Super
    - W. C.
      - .40 or less
      - .41 or more
      - really low: number

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Durability & Corrosion
Member Questionnaire

- Embeds - Connection Material - rebar
  - Protection employed
    - None
    - Galvanized
    - Epoxy coated
    - ZRC
    - S. S.
  - What is Protected
    - None
    - Everything
    - Flange Connectors
    - Bearing Plates
    - Loose connection material
  - Rebar - Protection
    - None
    - All
    - Some
    - Epoxy
    - Galvanized
    - Mesh coated
  - Maintenance
    - None
    - Some
    - Rate 1-10
  - Are designs and drawings available
    - Y
    - N

- Additional Information:
  
  
  
  
  
  
  
  
- Responder:
  - Name
  - Company
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PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
SURVEY SUMMARY

♦ SURVEY SUMMARY

This statistical analysis summary is based upon 54 responses.

♦ GENERAL

Size
105’ to 400’ wide
178’ to 1200’ long
Age
2 years to 24 years old average 8¼ years
Number of levels
1 to 12 levels
Location - Spread over 12 states
Environment
Salt Applied Region:
Salt Spray Region:
Freeze/Thaw:
Surface Sealer:
Warped Double Tees:
35 Structures 65%
5 Structures 9%
37 Structures 69%
12 Structures 22%
22 Structures 41%

♦ CRACKING

Degree D.T. Flange Cracking:
Degree Stem Cracking:
Degree Beam Cracking:
Degree Spandrel Cracking:
Degree Wall Cracking:
Surface Deterioration
Topping:
Precast Products as a whole
7.78 Performance Average
8.31 Performance Average
8.45 Performance Average
6.98 Performance Average
8.25 Performance Average
6.64 Performance Average
8.61 Performance Average

♦ DRAINAGE

Degree of Leaking:
Condition Exposed Plates:
Drainage Performance:
Condition Bearing Pad:
Type - Masticord:
Type - Neoprene
6.87 Performance Average
7.60 Performance Average
7.02 Performance Average
7.96 Performance Average
17 Structures 31%
17 Structures 31%
continued...

♦ PRECAST ELEMENTS

| Pretopped      | 17 Structures | 31% |
| Topped         | 36 Structures | 67% |
| Deck Members   | 52 Double Tee | 96% |
| Single Tee     | 1 Structure   |     |
| Quad Tee       | 1 Structure   |     |
| Tee Width      | 8 ft - 17     |     |
|                | 9 ft - 12     |     |
|                | 10 ft - 23    |     |
|                | Misc - 1      |     |
| Spandrels      | Pocketed Spandrels | 22 Structures | 41%* |
|                | L Spandrels   | 34 Structures | 63%* |

| Lateral Load System | 22 Structures | 34 Structures |
| Shear Walls         | 44            |              |
| Moment Frames       | 10            |              |
| Shafts              | 9             |              |
| Combination         | 7             |              |
| Performance System  | 7.63 Rating   |              |

* Note some structures were reported as employing both.

♦ CONDITION OF CONNECTION

| Double Tee Flange:  | 8.07 Performance Average |
| Spandrel/Column:    | 7.92 Performance Average |
| Column/Beam:        | 8.21 Performance Average |
| Condition Joint Sealant: | 6.78 Performance Average |

♦ CONCRETE

| Average strength attained: | 5878 psi | Ranging from 5000 psi to 7000 psi |
| Silica fume employed:      | One      |                                |
| DCI employed:              | Two      |                                |
| Air Entrained:             | 30 Structures | 56% |
| Super employed:            | 37 Structures | 69% |
| W/C ratio .40 or less:     | 31 Structures | 57% |
| W/C ratio .41 or more:     | 18 Structures | 33% |
| W/C really low:            | 2 Structures | 4%   |
| No Response                | 3 Structures | 6%   |
continued...

**PROTECTION EMPLOYED TO EMBEDS**

None: (shop coat paint only)  
19 Structures  35%

Galvanized:  
21 Structures  39%

Zinc Rich Paint:  
19 Structures  35%

Epoxy coated:  
4 Structures  7%

Stainless Steel:  
4 Structures  7%

What is Protected:

None:  
16 Structures  30%

Everything:  
17 Structures  31%

Flange Connections:  
8 Structures  15%

Bearing plates:  
20 Structures  37%

Loose Material:  
20 Structures  37%

Rebar None:  
42 Structures  78%

Mesh Epoxy Coated:  
7 Structures  13%

**MAINTENANCE SUMMARY**

None:  
13 Structures  24%

Some:  
30 Structures  56%

Performance Notary:  
5.76 Performance Average

**ANALYSIS**

The survey performed consisted of mailing forms to precast producers requesting information on precast structures. We received 54 responses which we deemed applicable to this study. A summary of all the responses is included herein. It should be noted that in some particular performance areas a few favorable or unfavorable reports can slant the summaries.

The structures ranged between 2 and 24 years of age spread over 12 states. The dimensions of the structures were very representative with a minimum width of 105' and a maximum length of 1200' and levels 1 to 12. The environmental conditions for durability study were well represented with 65% of the structures in a salt applied region, 69% in a freeze-thaw region. Another consideration within the industry is the degree warping of double tees, 41% of the responses had warped double tees.
The performance of the precast products was excellent with an average of 8.61 crack performance using a 1 to 10 scale. Spandrels had not performed as well as the other products, the main reason is due to hairline cracks near the support of pocketed spandrels and diagonal torsion cracks in "L" beam spandrels. Near a support the shear V is maximum while \( \phi V_c \) is reduced because of the pocket, this typically leads to the reported cracks at 45 degrees starting from the edge of the pocket where end reinforcing may be insufficient. In addition, the highest incident of cracking in spandrels is in the older structures when "L" beams and the accompanying high torsion stresses are most prevalent.

Drainage remains a serious durability concern with an average 7.02 performance rating which in turn induces leaking in the structure with a performance of 6.87.

The lateral load system in precast structures has relatively performed well, rating 7.63, with the majority of the structures employing a shear wall system.

In general connections have achieved a high rating except for joint sealant which seem to have a low yet acceptable performance.

The concrete strength of the precast structures surveyed was 5878 psi in average ranging from 5000 psi to 7000 psi. The high concrete strength achieved in precast plants is primarily due to the strict quality control. The precast industry seem to avoid the use of silica fume, this due to the low bleeding water associated with silica fume concrete which makes finishing a difficult task. DCI's and other corrosion inhibitor products were not utilized often by precasters. Air entraining agents as well as superplasticers were the primary choice of admixtures in precast concrete.

The primary protections employed to embeds in precast product are galvanization and Zinc rich painting. Epoxy coating and stainless steel were rarely utilized in the structures surveyed.

Because of the low maintenance of the structures, the performance rating of the maintenance effort of the surveyed structures was average, 5.76.
## STATISTICAL ANALYSIS SUMMARY

<table>
<thead>
<tr>
<th>Age</th>
<th>Lateral Load System</th>
<th>Cracking</th>
<th>Surface Deterior</th>
<th>Water Performance</th>
<th>Connections</th>
<th>Condit. Jnt/c</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DT Flange</td>
<td>DT Stem</td>
<td>Beams</td>
<td>Spndrs</td>
<td>Walls</td>
<td>Topping</td>
</tr>
<tr>
<td>0 to 5</td>
<td>6.60</td>
<td>7.80</td>
<td>8.60</td>
<td>8.70</td>
<td>7.89</td>
<td>8.50</td>
</tr>
<tr>
<td>5 to 10</td>
<td>8.35</td>
<td>8.05</td>
<td>8.86</td>
<td>8.66</td>
<td>7.81</td>
<td>7.81</td>
</tr>
<tr>
<td>10 to 24</td>
<td>6.67</td>
<td>6.93</td>
<td>7.07</td>
<td>7.38</td>
<td>6.38</td>
<td>8.67</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Performance</th>
<th>Structures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Walls</td>
<td>7.69</td>
</tr>
<tr>
<td>Frames</td>
<td>6.56</td>
</tr>
<tr>
<td>Shafts</td>
<td>8.14</td>
</tr>
<tr>
<td>Combination</td>
<td>8.75</td>
</tr>
</tbody>
</table>
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CRACKING PERFORMANCE BY AGE OF STRUCTURE

- 0 to 5
- 5 to 10
- 10 to 24

RATING

DT Flange  DT Stem  Beams  Spndrls  Walls
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DURABILITY CODE REQUIREMENTS

1. ACI CODE
American Concrete Institute building code and commentary ACI 318/318R, are the most widely used codes in the US. Chapter 4 of the 1995 code states the durability requirements.

1.1 Water-Cementitious ratio
ACI recognizes the contribution of pozzolans in general and fly ash in particular, slag, and silica fume to the cementitious properties of cement. In calculating the water-cementitious ratio the weight of these materials is added to the total weight of cementitious materials. The following are the general requirements for special exposure conditions:

- Concrete intended to have low permeability when exposed to water: 0.50
- Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals: 0.45
- Concrete exposed to chlorides intended to protect reinforcement: 0.40

1.2 Freeze and thawing
ACI requires that concrete exposed to freeze-thaw be air entrained. Air entrainment requirements depend on the exposure conditions, the size of the aggregate and the concrete strength. The required air content ranges between 7.5% for a 3/8" aggregate to 4.5% for a 3% aggregate in severe exposure conditions and 6% for a 3/8" aggregate to 3.5% for a 3" aggregate in a moderate exposure condition. The air content specified in ACI is total air content, entrapped air, generally 2%, should be included when using the admixture.

1.3 Cementitious content
Provisions for a maximum percentage by weight of cementitious materials content is included in ACI 318 for concretes exposed to deicing chemicals:

- Fly ash: 25%
- Slag: 50%
- Silica fume: 10%
- Fly ash, slag, and silica fume: 50%
- Fly ash and silica fume: 35%

1.4 Chloride ion content
The chloride ion content in concrete was limited for reinforcement protection. The following is the maximum allowable chloride ion content in concrete depending on the type of construction.
## PCI RESEARCH PROGRAM
### DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
#### CODE REQUIREMENTS

<table>
<thead>
<tr>
<th>Type of Concrete</th>
<th>Concrete Cover Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressed concrete</td>
<td>0.06</td>
</tr>
<tr>
<td>Reinforced concrete exposed to chloride ions</td>
<td>0.15</td>
</tr>
<tr>
<td>Reinforced concrete that is dry or protected from moisture</td>
<td>1.00</td>
</tr>
<tr>
<td>All other reinforced concrete structures</td>
<td>0.30</td>
</tr>
</tbody>
</table>

### 1.5 Concrete cover
Concrete cover requirements are divided into three sections in the ACI building code, cast in place, precast, and prestressed. The main durability concern addressed by the requirement are earth and weather. The requirements take into account the size of the reinforcement. The following are the general requirements outlined in the ACI building code section 7.7.

#### 1.5.1 Cast in place concrete
- **Concrete exposed permanently to earth**
  - #6 to #18 bars: 3"
  - #5 and smaller: 2"
- **Concrete exposed to earth or weather**
  - Slabs: #14 and #18: 1.5"
  - #11 and smaller: 0.75"
  - Beam, Columns: 1.5"
- **Concrete not exposed to weather**
  - Slabs: #6 and larger: 0.75"
  - Beam, Columns and plates: #5 and smaller: 0.5"

#### 1.5.2 Precast Concrete
- **Concrete exposed to earth or weather**
  - Walls: #14 and #18: 1.5"
  - #11 and smaller: 0.75"
  - Other: #14 and #18: 2"
  - #6 to #11: 1.5"
  - #5 and smaller: 1.25"
- **Concrete not exposed to weather**
  - Slabs: #14 and #18: 1.25"
  - #11 and smaller: 0.625"
  - Beam, Columns: Main: db or 0.625" to 1.5"
  - Shells: #6 and larger: 0.625"
  - and plates: #5 and smaller: 0.375"

#### 1.5.3 Prestressed concrete
- **Concrete exposed permanently to earth**
  - Walls: 3"
- **Concrete exposed to earth or weather**
  - Walls: 1"
  - Other: 1.5"

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PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
CODE REQUIREMENTS

Concrete not exposed to weather

<table>
<thead>
<tr>
<th>Slabs, walls, joist</th>
<th>Beam, Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75&quot;</td>
<td>1.5&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shells and plates</th>
</tr>
</thead>
<tbody>
<tr>
<td>#5 and smaller</td>
</tr>
<tr>
<td>0.375&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>db less than 0.75&quot;</td>
</tr>
</tbody>
</table>

2. Canadian Codes CAN/CSA

In addition to the general Canadian codes, Parking Structures S413-94 will be included here because of the additional requirements pertaining to the reinforcement protection.

2.1 Exposure to Freeze and Thawing

There are strength, water-cementitious ratio, and air content requirements pertaining to freeze and thawing.

<table>
<thead>
<tr>
<th>Strength (MPa)</th>
<th>W/C</th>
<th>Air Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete in saturated condition</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Concrete in unsaturated condition</td>
<td>25</td>
<td>0.55</td>
</tr>
</tbody>
</table>

* Depending on the size of the aggregate, see air content requirements

2.2 Exposure to deicing chemicals or sea water

<table>
<thead>
<tr>
<th>Strength (MPa)</th>
<th>W/C</th>
<th>Air Category*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Protection of reinforcement Critical</td>
<td>35</td>
<td>0.4</td>
</tr>
<tr>
<td>Non critical with freeze thaw exposure</td>
<td>32</td>
<td>0.45</td>
</tr>
<tr>
<td>Saturated condition</td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td>Dry condition</td>
<td>25</td>
<td>0.55</td>
</tr>
</tbody>
</table>

2.3 Water soluble chlorides

| Prestressed concrete | 0.06% |
| Concrete in moist or chloride environment | 0.15% |
| Dry condition | 1.00% |

2.4 Air content

<table>
<thead>
<tr>
<th>Nominal Size of coarse aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>10mm</td>
</tr>
<tr>
<td>7 to 10</td>
</tr>
<tr>
<td>5 to 8</td>
</tr>
</tbody>
</table>

2.5 Parking Structures requirements

In addition to the above requirements the Parking Structures S413-94 states additional requirements which regulate the use of combined protection systems. The accepted protection
systems are membranes, epoxy coated rebars (EC), corrosion inhibitors (CI), sealers, and low permeability concrete (LP). The following are the allowed combinations.

Non prestressed normal exposure
- Membrane
- Membrane + top bars EC*
- Membrane + CI

Non prestressed severe exposure
- Membrane
- CI + top bars EC
- LP concrete + top bars EC
- CI + LP concrete
- Sealer + CI
- Sealer + LP concrete
- Sealer + top bar EC

Prestressed

3. European Codes, CEN
The European codes are based on averages of 12 member nations of the CEN (Comite Europeen de Normalisation). A summary of upper and lower limits of the code requirements of the members is given in the summary table.

ANALYSIS

All the codes recognize that the requirements should be directed towards limiting corrosion and freeze thaw damages. The Canadian and US codes appear to be based on the same philosophy given the similarities in the requirements. The European codes are based on individual requirements of 12 nations, members of the CEN.

Concrete Cover
All codes recognize the need for a minimum cover to protect steel reinforcement. The conditions are based on moisture and chlorides. The European codes have a lesser requirements than the North American codes. The Canadian have the most conservative cover requirement with a high value of 4.5 inches for aggressive environment. The ACI requirements appear to be an average when compared to its European and Canadian counterpart.

Cement Content
The European have a minimum cement content of about 400 pcy. A minimum strength requirement is also included. The North American codes have no minimum cement content, which seem to be more logical if a minimum strength is specified. The European concern is the extensive use of mineral admixtures or other cementitious materials other than cement.
Water-Cement Ratio
Maximum water-cement ratio is required by all the codes investigated. Water-cement ratio has been found by several studies as an adequate tool to control strength and permeability. Limiting the water-cement ratio will result in an acceptable strength and permeability. The North American codes seem to agree on these limitations whereas the European codes permit higher ratios. The reason can be attributed to economical considerations. European nations tend to specify higher water-cement ratio due to the high cost of cement.

Concrete Strength
All the codes agree on a required minimum strength. An average value of 4000 psi in moderate environment and 5000 psi in aggressive environment seem to be the general norm.

Chloride Content
The European codes as a group have no maximum chloride content requirements. Some individual codes have such requirements. The North American codes agree on 0.15 and 1% of weight of cement for humid and dry environment respectively. A very low value of 0.06% is recommended for precast members.

Air Content
The North American codes limit the air content in concrete to address the problem of the low strength and permeability associated with a high air content.
<table>
<thead>
<tr>
<th>MINIMUM COVER (IN.)</th>
<th>(5 - 20 mm)</th>
<th>(20 - 30 mm)</th>
<th>(25 - 50 mm)</th>
<th>(50 - 115 mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Humid (0.20 - 0.29)</td>
<td>0.79 - 1.18</td>
<td>0.98 - 1.97</td>
<td>2.36 - 4.53</td>
<td></td>
</tr>
<tr>
<td>Wet (0.50 - 0.95)</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
<td></td>
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<th>MINIMUM CEMENT CONTENT (PCY)</th>
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<tbody>
<tr>
<td>ACI</td>
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<tr>
<td>CEN</td>
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<thead>
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<th>MINIMUM WATER/CEMENT RATIO</th>
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<th>MINIMUM CONCRETE STRENGTH (PSI)</th>
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<tr>
<th>MAXIMUM CHLORIDE CONTENT (% CEMENT WEIGHT)</th>
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<td>CEN</td>
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<table>
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<th>AIR CONTENT (%)</th>
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Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
The site inspections were conducted on a total of eight structures fairly well spaced in different geographical locations in the U. S. Two of the structures were located in the salt spray region CCII within 100 yards of the ocean, two were located in Zone III severe exposure and two in Zone I least exposure and two in less severe coastal Zone CCI. They were all at least 7 years old and two were 17 and 20+ years old respectively. The investigation purposely tried to seek out structures that had been identified as having problems in order to observe some of the causes of poor performance.

The 17 year old pretopped double tee structure in upper midwest indicated a general fair to good performance except at the double tee flange to flange location where spalling at the connection lead to leaking. It should be noted that this structure was probably one of the first ten pretopped structures in the U. S. The cause of the cracking is difficult to determine after 17 years but it may have been precipitated by overheating of the concrete during the welding process of the flange connection. This problem was not recognized in the early days of the introduction of the pretopped system. The need to allow the flange plates to expand as it is heated during the welding process is now well recognized and details to minimize this problem have been employed.

Another structure identified for inspection was the 20 plus year old structure Number IV. The precast components generally are performing well with the exception of the spandrels which are cracked at the end connection primarily because the members were welded at both ends. This type of connection has since then been typically replaced by bolted connection between the spandrels and columns which allow the structure to relieve the seasonal thermal strains (see volume change section) thin ¼” bearing pads have also deteriorated over time.

Two pretopped parking decks located in the coastal exposure Zones CCI and CCII and seven and eight years old are both performing very well with minimal cracking and leaking. Both structures employed shear walls and stair shafts for lateral loads as well as bolted spandrel to column connections and the lateral load systems and connections are performing very well. A slight difference in performance is noted with the two mile inland structure nearly a straight 10 in performance while the structure 100 yards from the Gulf of Mexico shows a slightly lesser performance with both structures approximately the same age. This emphasizes the need to provide a somewhat higher level of protection for structures in the salt spray zone within ½ mile of the coast.
ZONE I represents the mildest conditions where freezing is rare and salt is not used.

ZONE II represents areas where freezing occurs and deicing salts are not or rarely used.

ZONE III represents the areas where freezing and deicing salts are common.

COASTAL CHLORIDE ZONE I (Zone CC-I) represents areas with Zone I and within 5 miles of the Atlantic Ocean, Gulf of Mexico and Pacific Ocean.

COASTAL CHLORIDE ZONE II (Zone CC-II) is areas within Zones I and II and within one half mile of the salt water bodies described in Zone C-I.
CASE STUDY I

LOCATION: SOUTHERN COASTAL CITY
less than 100 m from gulf, Exposure Zone CCII

AGE: 7 years

NUMBER OF LEVELS: 3

FRAMING SYSTEM: 24" Pretopped double tees, Precast beams, spandrels, walls and Columns.

OBSERVED CONDITIONS:

- Double tees, beams, walls, spandrels, and columns in good condition.
- Minimal cracking in all products, non critical crack in spandrels with pocket near support.
- Minimal water ponding due to drainage.
- Non protected, exposed rebars used in ramp walls as railing are corroding.
- Bearing pads compressed to failure and sliding out of ledge, due to double tee movement or improper installation.
- All protected connections performed well, however signs of chloride attacks are showing where the patching was not applied properly.
- Lateral load system- shear walls plus shaft walls- performing well.
- Exposed embeds were ZRC coated and are performing well.

SUMMARY
Standard precast construction which is performing well except for a few isolated conditions. Minimum 3/8 inch thick bearing pads installed properly would correct the bearing problem and better surface preparation for patching would alleviate cracked patches.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
## Field Survey Report

### General Description
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)
<table>
<thead>
<tr>
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</tbody>
</table>

- Tees Warped? Yes No
- Surface Sealer? Yes No
- Salt application Region? Yes No
- Salt spray location? Yes No
- Freeze/Thaw region? Yes No

### Observed Conditions
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)
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</table>

- Double tee flange
- Double tee stem
- Beams
- Spandrels
- Walls

- Surface deterioration (Rate 1-10)
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<thead>
<tr>
<th>1</th>
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- Topping
- Precast products

- Leaking (Rate 1-10)
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</table>

- Exposed Plates
- Drainage system
  (7 = 1 to 2 areas of small puddles)
- Bearing pads

- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)
<table>
<thead>
<tr>
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</table>

- DT flange to flange
- Spandrel/Column
- Column/Beam
- Condition of Jr. sealant

Type: Neoprene
CASE STUDY II

LOCATION: SOUTHERN CITY
Exposure Zone CC I

AGE: 8 years

NUMBER OF LEVELS: 4

FRAMING SYSTEM: 24" Pretopped double tees, Precast beams, spandrels, walls and Columns.

OBSERVED CONDITIONS:
- Double tees, beams, walls, spandrels, and columns in good condition.
- Minimal cracking in all products.
- Few leaks through flanges and spandrels.
- Bearing pads performed well.
- All protected connections- ZRC coating- performed well.
- Lateral load system- shear walls- are performing well.

SUMMARY
Standard precast construction which is performing extremely well. Garage is very clean which indicates good maintenance. Coated plates performing well.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
## Field Survey Report

### General Description
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)
  
<table>
<thead>
<tr>
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</tbody>
</table>

- Tees Warped? Y  N
- Surface Sealer? Y  N
- Salt application Region? Y  N
- Salt spray location? Y  N
- Freeze/Thaw region? Y  N

### Observed Conditions
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)
  
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</table>

- Double tee flange
- Double tee stem
- Beams
- Spandrels
- Walls
- Surface deterioration (Rate 1-10)
  
<table>
<thead>
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<th></th>
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</tbody>
</table>

- Topping
- Precast products
- Leaking (Rate 1-10)
  
<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
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</table>

- Exposed Plates
- Drainage system
  
  (7 = 1 to 2 areas of small puddles)
- Bearing pads
- Type MASTICORD

- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)
  
<table>
<thead>
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</tbody>
</table>

- DT flange to flange
- Spandrel/Column
- Column/Beam
- Condition of Jt. sealant

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CASE STUDY III

LOCATION: SOUTHWESTERN CITY
2 miles inland from Gulf of Mexico. Exposure Zone CCI

AGE: 8 years

NUMBER OF LEVELS: 2

FRAMING SYSTEM: 20" and 24" Double tees with 2" C.I.P Topping, Precast beams, spandrels, walls and Columns.

OBSERVED CONDITIONS:
- Double tees, topping, beams, walls, and columns in good condition.
- Minimal cracking in all products.
- Expansion joint failure.
- Exterior spandrels bearing on hidden corbels - Recessed pocket patches spalled and Steel haunches corroded
- 1/4" double tee bearing pads compressed to failure
- Spandrel bearing pads deteriorated at expansion joint.
- Minimal chloride reading in topping.

SUMMARY
Garage was constructed with unique hidden pocketed corbel for spandrels. Grout patches for pockets did not allow for spandrel expansion and failed. The 1/4" bearing pads which looked like low grade neoprene failed. Thicker fiber reinforced pads would have performed better. There is evidence that the garage undergoes large movements in excess of 2".
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
FIELD SURVEY REPORT

General Description
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)

<table>
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</tbody>
</table>
- Tees Warped? | Y | No |
- Surface Sealer? | Y | No |
- Salt application Region? | Y | No |
- Salt spray location? | Y | No |
- Freeze/Thaw region? | Y | No |

Observed Conditions
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)

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- Surface deterioration (Rate 1-10)

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- Leaking (Rate 1-10)

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<td>Bearing pads</td>
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- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)

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<tr>
<td>Condition of Jt. sealant</td>
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CASE STUDY IV

LOCATION: UPPER MIDWEST CITY
Exposure Zone III

AGE: 17 years

NUMBER OF LEVELS: 2

FRAMING SYSTEM: Exterior load bearing double tee walls. Pretopped double tee floor system with precast IT beams and columns.

OBSERVED CONDITIONS:
- Joint sealants and expansion joints recently and frequently replaced.
- Concrete in double tees, beams and columns performing well. CIP pour strips recently restored.
- Cracking and deterioration problems with hollow core ramp.
- Spalls at floor flange connections creating leaks.

SUMMARY
Owner performing frequent maintenance on joint sealants and CIP pour strips. Precast driving surfaces, beams, columns and walls generally in good shape. Frequent problems with small hollow core ramps leaking and cracking.
# PCI RESEARCH PROGRAM
## DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
### CEG SURVEY OF STRUCTURES

## FIELD SURVEY REPORT

### General Description
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)

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- Tees Warped? **Y** No
- Surface Sealer? **Y** No
- Salt application Region? **Y** No
- Salt spray location? **Y** No
- Freeze/Thaw region? **Y** No

### Observed Conditions
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)

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- Double tee flange
- Double tee stem
- Beams
- Spandrels
- Walls

- Surface deterioration (Rate 1-10)

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- Topping
- Precast products

- Leaking (Rate 1-10)
- Exposed Plates
- Drainage system (7 = 1 to 2 areas of small puddles)
- Bearing pads

- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)

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- DT flange to flange
- Spandrel/Column
- Column/Beam
- Condition of Jt. sealant

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CASE STUDY V

LOCATION: MIDWEST CITY
Inland. Exposure Zone III

AGE: 20+ years

NUMBER OF LEVELS: 4

FRAMING SYSTEM: 20" and 24" Double tees with 2" C.I.P Topping, Precast beams, spandrels, walls and Columns.

OBSERVED CONDITIONS:
- Double tees, topping, beams, walls, and columns in good condition.
- Minimal cracking in all products.
- Expansion joint repaired and in good condition.
- Exterior spandrels bearing on corbels with top welded at both ends - Recessed pocket patches spalled and connection corroded
- 1/4" double tee bearing pads compressed to failure
- Spandrel bearing pads deteriorated at expansion joint.
- One IT beam cracked, strands exposed.
- Haunch cracking due to non symmetrical load. Repaired once then cracked again.
- Pretopped double tee performed better than CIP topping, specially at the roof.
- Exposed shim stacks corroded.
- Leaks at pockets and corbels.

SUMMARY:
The inspection of this parking structure was recommended as one that has had several durability problems. The original inspection has reported severe leaking, haunch failure, corroded connections, and spalled concrete through out the structure. When the structure was inspected for the purpose of this research study, repairs have already been made. Even though the repairs attempted to correct most of the problems, some problems remained not addressed. The cracked haunch was due to an apparent eccentric loading from a spandrel, not considered in the original design, which resulted in cracks. The application of the joint sealers to correct leaks was not effective. The cracked IT beam was due to torsion. The stirrup spacing at the support was approximately 12" which is not adequate for torsion given the long spans. The spandrels were welded at both ends to the columns with little relief from movement which caused cracks at the connection area. Regular maintenance of this structure could have prevented several problems. Parking structures in general should have an adequate maintenance program which will enhance its appearance and eliminate the cost of repairs.
**FIELD SURVEY REPORT**

**General Description**
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)
  - 1 2 3 4 5 6 7 8 9 10
  - 0 0 0 0 0 0 0 0 0 0
- Tees Warped? Y N
- Surface Sealer? Y N Type ______________________
- Salt application Region? Y N
- Salt spray location? Y N
- Freeze/Thaw region? Y N

**Observed Conditions**
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)
  - 1 2 3 4 5 6 7 8 9 10
  - Double tee flange 0 0 0 0 0 0 0 0 0
  - Double tee stem 0 0 0 0 0 0 0 0 0
  - Beams 0 0 0 0 0 0 0 0 0
  - Spandrels 0 0 0 0 0 0 0 0 0
  - Walls 0 0 0 0 0 0 0 0 0
- Surface deterioration (Rate 1-10)
  - 1 2 3 4 5 6 7 8 9 10
  - Topping 0 0 0 0 0 0 0 0 0
  - Precast products 0 0 0 0 0 0 0 0 0
- Leaking (Rate 1-10)
  - 0 0 0 0 0 0 0 0 0
  - Exposed Plates 0 0 0 0 0 0 0 0 0
  - Drainage system 0 0 0 0 0 0 0 0 0
  - (7 = 1 to 2 areas of small puddles)
- Bearing pads 0 0 0 0 0 0 0 0 0
  - Type NEOPRENE
- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)
  - 1 2 3 4 5 6 7 8 9 10
  - DT flange to flange 0 0 0 0 0 0 0 0 0
  - Spandrel/Column 0 0 0 0 0 0 0 0 0
  - Column/Beam 0 0 0 0 0 0 0 0 0
  - Condition of Jt. sealant 0 0 0 0 0 0 0 0 0

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CASE STUDY VI

LOCATION: UPPER MIDWEST CITY
Exposure Zone III, Salt Region

AGE: 23 years

NUMBER OF LEVELS: 2

FRAMING SYSTEM: Pretopped double tees, load bearing L-spandrels

OBSERVED CONDITIONS:
- Driving surface in good condition.
- Pour strip at end of tees renovated and coated with epoxy.
- L-spandrels rolled 1"-1 1/2" at top. Minor torsional cracks but no deterioration.
- Majority of tees contain longitudinal cracks on bottom of flange.
- Bearing pads in good condition.
- Plastic lifting inserts in top of spandrels well preserved with no patching.
- 20% of flange joints leaking.

SUMMARY
Older structure in road salt region which is performing well. Owner has restored CIP pour strips and joint sealants but precast has not required any restoration. L-spandrels have minor torsion cracks and tees have typical flange to stem longitudinal cracks. No evidence that cracking is detrimental to performance. Joint leaking occurring at flange connections which seem to be spalled.

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### General Description

- **Lateral load system performance (Rate 1-10) (#5 = minimal cracks)**
  
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- **Tees Warped?**  
  Y | No

- **Surface Sealer?**  
  Y | No

- **Salt application Region?**  
  Y | No

- **Salt spray location?**  
  Y | No

- **Freeze/Thaw region?**  
  Y | No

### Observed Conditions

- **Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)**
  
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- **Double tee flange**  
  0 | 0 | ● | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

- **Double tee stem**  
  0 | 0 | ● | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

- **Beams**  
  0 | 0 | 0 | 0 | 0 | 0 | 0 | ● | 0 | 0 |

- **Spandrels**  
  0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 | 0 |

- **Walls**  
  0 | 0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 |

- **Surface deterioration (Rate 1-10)**
  
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- **Topping**  
  0 | 0 | 0 | 0 | 0 | 0 | 0 | ● | 0 | 0 |

- **Precast products**  
  0 | 0 | 0 | 0 | 0 | 0 | 0 | ● | 0 | 0 |

- **Leaking (Rate 1-10)**
  
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- **Exposed Plates**  
  0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 | 0 |

- **Drainage system**  
  0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 | 0 |

(7 = 1 to 2 areas of small puddles)

- **Bearing pads**  
  0 | 0 | 0 | 0 | 0 | 0 | 0 | □ | 0 | 0 |

Type  

- **Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)**
  
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- **DT flange to flange**  
  0 | 0 | ● | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

- **Spandrel/Column**  
  0 | 0 | 0 | ● | 0 | 0 | 0 | 0 | 0 | 0 |

- **Column/Beam**  
  0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 | 0 |

- **Condition of Jt. sealant**  
  0 | 0 | 0 | 0 | 0 | ● | 0 | 0 | 0 | 0 |
CASE STUDY VII

LOCATION: UPPER MIDWEST CITY
Exposure Zone III, Salt Region

AGE: 22 years

NUMBER OF LEVELS: 6

FRAMING SYSTEM: Pretopped double tees, load bearing spandrels, IT beam, columns, and elevator walls.

OBSERVED CONDITIONS:
- CIP pour strip over IT beam cracked and spalled at second level. Isolated spots at upper levels also occurring.
- Driving surface generally in good condition. Some scaling at drains and adjacent to non load bearing spandrels. More scaling on roof.
- No drains at bottom of ramps. Water ponding.
- Significant spalling at tee flange weld connections. Leaks and stains on bottom of flange.
- Column corbel plates rusting.
- Significant cracking in non load bearing spandrels.

SUMMARY
Older structure in road salt region with little evidence of proper maintenance. CIP pour strips showing more wear than precast surfaces. Non stressed non load bearing spandrels contain many cracks (Photo 6) and improper sealing material creating unsightly stains (Photo 3). Several top flange cracks, visible (Photo 20), but no evidence of corrosion or debonding. Could not determine flange connector material but severe corrosion was occurring. Ponding was occurring in many locations due to improper drain placement. Scaling, mostly in driving lanes and roof, may be due to snow removal equipment.
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# PCI RESEARCH PROGRAM
## DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
### CEG SURVEY OF STRUCTURES

### FIELD SURVEY REPORT

**General Description**
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)

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- Tees Warped? Y  No
- Surface Sealer? Y  No
- Salt application Region? Y  No
- Salt spray location? Y  No
- Freeze/Thaw region? Y  No

**Observed Conditions**
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)

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- Double tee flange
- Double tee stem
- Beams
- Spandrels
- Walls
- Surface deterioration (Rate 1-10)

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- Topping
- Precast products
- Leaking (Rate 1-10)
- Exposed Plates
- Drainage system
(7 = 1 to 2 areas of small puddles)
- Bearing pads
  Type ____________
- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)

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- DT flange to flange
- Spandrel/Column
- Column/Beam
- Condition of Jt. sealant

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PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
CEG SURVEY OF STRUCTURES

CASE STUDY VIII

LOCATION: NORTHEAST CITY
Salt Region, Exposure Zone III

AGE: 25 years

NUMBER OF LEVELS: 6

FRAMING SYSTEM: 32" single tees, CIP Topping, beams, columns, spandrels, elevators and stair walls.

OBSERVED CONDITIONS:
- First level membrane worn.
- Driving surface generally in good condition.
- Joints in floor sealed only at roof level. Flange to flange connection rusting on all levels.
- Cracking in column corbels at several locations.
- Galvanized bearing plates in good condition except at first level where minor rusting is occurring.
- Bearing pads in good condition.

SUMMARY
Older structure in road salt region which is in good shape. Poor detailing with no joints or sealing in CIP topping has led to flange connection deterioration but all other plates and bearing pads are in very good shape. First level membrane was not properly maintained. Column cracking is mostly in protection around embedded steel hardware.

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FIELD SURVEY REPORT

General Description
- Lateral load system performance (Rate 1-10) (#5 = minimal cracks)

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- Tees Warped? Yes No
- Surface Sealer? Yes No
- Salt application Region? Yes No
- Salt spray location? Yes No
- Freeze/Thaw region? Yes No

Observed Conditions
- Cracking (by product - Rate 1-10) (with 8 being less than 5% of product cracked)

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- Double tee flange
- Double tee stem
- Beams
- Spandrels
- Walls

- Surface deterioration (Rate 1-10)

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- Topping
- Precast products

- Leaking (Rate 1-10)

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- Exposed Plates
- Drainage system

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- Bearing pads

- Condition of connections (evaluate performance - more than 5% cracked or damaged equal < #3)

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- DT flange to flange
- Spandrel/Column
- Column/Beam
- Condition of Jt. sealant
Photo #6 - Failure of joint sealant

Photo #7 - Spalling at flange connection embeds
Photo #8 - Spalling at spandrel to column connection

Photo #9 - Cracking of non-load bearing spandrels
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #10 - Spalling of C.I.P. pour strip

Photo #11 - Cracking from rebar corrosion
PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #12 - Staining from rusted embeds

Photo #13 - Corrosion of exposed plates
Photo #14 - Deterioration of beam ends due to end strand exposure
Photo #15 - Expansion joint deterioration
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST Prestressed Concrete Structures
PHOTOS

Photo #16 - Bearing pad deterioration

Photo #17 - Deterioration of spandrel pocketed haunch
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #18 - Cracking at lifting devices in top of spandrel

Photo #19 - Corrosion of embedded hardware - Column to pocketed spandrel connection
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #20 - Staining from rusting embedded plates

Photo #21 - Deterioration of surface membrane over pour strip
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #22 - Column spalling from spandrel torsion and lack of neoprene spacer

Photo #23 - Pretopped double tee flange cracking
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Photo #4 - Deterioration from insufficient cover

Photo #5 - Rusting of embedded plates
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
Based upon the survey and site inspections, the following special topics were identified and studied. The topics selected for study were those deemed important in the successful design of a parking structure.

The warped double tee analysis also involved a finite element study as well as on actual load test conducted by a PCI member with CEG staff present. In addition, the results of two other load tests were analyzed. Based upon this investigation we have based our findings.

The special topics studied are:

- Effects of Warping Tees
- Effects of Volume Change
- Flange Cracking
- Performance of Lateral Load Systems
- Protection of Embeds
- Drainage Consideration
- Concrete Add Mixtures

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Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
DOUBLE TEE WARP ANALYSIS

Double tees are warped when drainage plans detail different slopes for the bearings at each end of a double tee floor member. It has been observed that large warps can produce cracking in the double tee flange.

In order to determine the degree that a double tee could be warped without cracking a finite element analysis was run.

The finite element analysis of a warped double tee revealed that the maximum tensile stresses occur in the top of the flange near the unwarped stem, as shown in figure 1.

The stress distribution across the flange is the classical bending stress distribution. The top surface of the flange however exhibits a stress distribution similar to a concentrated load stress distribution.

![Fig. 1. Stress Pattern in Warped Double Tee](image)

In order to study the effect of warping on the maximum tension stress, a model was developed to vary the degree of warp. The model was tested using a very fine 1" grid mesh, employing the SAP90 finite element program. The model consisted of three supported stems and the fourth stem was supported on a spring. The spring with a constant K, will allow only a P/K displacement. The results of this analysis is given in figure 2. The stress shown is the maximum tension stress at the top of the flange. Once the maximum stress is reached it occurs initially only over a length of 3 inches. The use of figure 2 to calculate the allowable warp for a given concrete should be done with caution. ACI recommends using fr = 0.1 to 0.15 fc (ACI 318 R10.2.5) or determine the
modulus of rupture from tests. In situations where non critical cracks are allowed a higher value for $f_r$ can be used and a larger warps can be employed.

![Stress vs Warping Chart](chart.png)

**Fig 2. Maximum Elastic Tension Stresses in Warped Double Tee Flange**

Figure 2 shows the maximum stresses in the flange with respect to stem to stem warping. This chart can be used to predict the stresses for a given warp or determine the maximum warp for given stress. For an actual service compressive strength of 6500 psi, $f_r = 0.15 f_c$ (high quality concrete) will result in a modulus of rupture of 975 psi. Given a stress of 975 psi, a non cracked flange would produce a warping between stems of approximately 0.45" for a 40'-0" double tee and approximately 0.50" for 60'-0" double tee.

The model was further used to predict the crack propagation. The same double tee was used to model a crack in the top flange. A crack of 1 inch in depth and 12 inches in length was modeled by releasing joints in the finite element model. The stress pattern in the double tee with a crack is given in figure 3. The analysis of the cracked double tee revealed that the crack will propagate along the flange rather than through the flange. This supports the belief that most through flange cracking is created by handling rather than warping. The stresses across the flange have actually been relieved to some extent and the stresses at the crack tip exhibit a standard crack tip stress pattern. The stresses at the crack tip range between 2 to 3 times the average stress. These longitudinal cracks can be contained by the flange mesh reinforcement, typical in a double tee.
Fig. 3. Stress Pattern in Cracked Warped Double Tee

Finite Element Analysis Summary
The stresses caused by a moderate warp are generally not critical. The cracks resulting from a service warp are generally longitudinal and do not penetrate through the flange. Observation of various projects during the construction phase indicate that typical flange cracking is created by inappropriate curing, handling, hauling, and erection. The torsion of a double tee created by inappropriate handling, transporting, and erecting can generate stresses much higher than the ones caused by service warp. This study concludes that double tees can safely be warped to the amount of 3/4" between stems and 1.5" for a full tee width.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
In addition to the finite element study, a full scale double tee warping test was conducted on two 10' wide by 28" deep pre-topped double tees with 4" flange reinforced with mesh and provisions for a 1'-11" pour strip. One double tee was 47' long and one was 60'-6" long. A cross section of the tees is shown in Figure T1 and the test setup is shown in Photo T2.

Warping was produced by first setting the double tee on level shim stacks and then removing shims from under one leg to lower it in relation to the other three legs. The warp given is the elevation difference between the stems not the total warp of the width of the tee.

As the warp was increased, cracking occurred at the juncture of the stem and flange bottom for the leg which was lowered, and in the top flange bottom at the leg adjacent to the one that was lowered. No cracking was observed at the opposite end of the tee which remained level.

Figure T3 shows the crack pattern in the 60'-6" long tee and Figure T4 shows the crack pattern in the 47'-0" tee. Cracking in the wash portion of the top flange was not visible due to the rough finish.

The following general observations were noted during the test:

1. The 60' double tee withstood a warp of ³⁄₄" between stems before cracking which the 47' tee had a very minor crack start at ⁵⁄₈" warp.

2. Cracking in the top flange progressed diagonally across the flange while bottom cracking stayed in the flange to stem junction.

3. Maximum crack widths increased as warping increased.

4. Maximum crack width for 1⁵⁄₈" warp between legs was .010" in the 60' tee.

5. Cracking in the top flange extended approximately 6'-0" from the end of the tee for 1⁵⁄₈" warp between legs.

6. Crack widths were less at the bottom of the flange than at the top.
continued...

♦ SUMMARY

The load test substantiated the finite element analysis. Both studies confirmed that observable cracking occurred when the overall warp exceeded 1 1/2". A 1 1/6" warp between double tee stems in a 28" x 10-0" pre-topped double tee produces cracking in the top and bottom of the flange. However, the crack widths are not excessive and are not through and are typical of cracks created by stripping, handling and hauling. For most exposure conditions, these types of cracks can be sealed and perform well service. To minimize such cracking, a warp level of 1 1/4" or less is desirable.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Fig T1: Typical Double Tee Used in the DT Warp Test
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DOUBLE TEE WARPING TEST
PHOTOS

Photo T2: D.T. Warp test set up
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

1 5/8" WARP  NO CRACKS
2 3/4" WARP  0.002 WIDTH
3 1 1/8" WARP  0.005 @ PT1 and 0.002 @ PT3
4 1 5/8" WARP  0.01 @ PT1 and 0.005 @ PT3

Fig T3 : Crack Pattern for 60'-6" Long Tee
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Fig T4: Crack Pattern for 47'-0" Long Tee.

1 5/8" WARP  
2 3/4" WARP  
3 1 1/8" WARP
Poor drainage was found to be one of the principle culprits in the deterioration of parking structures. Poor drainage does not remove water from the surface and creates ponds. Such ponds can allow concentration of road salt to attack the concrete surface and allow the chloride ions to penetrate into the concrete. Ponding can also focus water at sealant joints where any slight imperfection can allow water to pass through the joint and attack connection hardware or improperly protected member ends. (See Photos 6, 24, 25). Ponding can also be a safety hazard particularly in regions of freezing weather.

The results of poor drainage were apparent in the survey and structural inspection. The average rating of the drainage system in the survey was 6.8 and 20% of the responses were 5 or less. Where poor drainage was present, the performance was typically decreased with excessive leakage noted. (See Photo 28)

Good drainage is found to be a matter of the following considerations:

1. Primary span floor slope
2. Cross slope
3. Drain location
4. Drain size
5. Joint sealing

**Primary span floor slope:** While the tendency might be to provide the more slope the better, excessive slopes can lead to warping problems or create special headroom considerations. Therefore, a minimum slope that adequately drains the water off the surface is desirable.

Several slopes were observed that appeared to provide adequate drainage, they were typically in the 1½% to 2% range for the primarily long span slope and ½% to 1% slopes employed on the beam lines to create crickets to direct the water to the drains. (See Photo 23). When the beams cross slopes were increased to 1½% excessive warping and a large degree of flange cracking in pretopped systems were observed. All drainage was sloped to the interior on all structures inspected. Therefore, a minimum longitudinal slope (along the tee span) of 1½% is recommended and a minimum cross slope of ½% is also recommended.
Drains located as close to the column grid lines as possible minimize local ponding and drains that are at least 11" as a minimum dimension perform the best.

Trench drains were observed to be typically employed at the bottom of the roof level ramp. They were not typically continuous but spaced out so as not to interrupt the floor framing system.

Another key to keeping water from penetrating into the structure and remaining on the surface was the proper application of sealants. The structure that performed the best were sealed at all circumstances where precast members come together. Whether the floor system was a pretopped system or a topped system when all flange to flange joints, all tee to beam and spandrel joints, all tee and beam to wall panel joints and joints around all columns were properly sealed, the deck had very few if any leaks. The typical sealant employed was urethane. Properly installed urethane appears to have an effective life of seven to ten years with touch up maintenance when and where required. In some cases in relatively dry climates, in exposure Zone I, only the top level is sealed at the joints and the lower level just employed tooled joints in C.I.P. toppings. If the connection materials are properly protected, this also appeared to work with minimum deterioration in structures inspected over 12 years old.

It was difficult to determine the effectiveness of sealers such as silane. But when the application could be identified no significant deterioration of the riding surface was observed.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
PHOTOS

Photo #24 - Scaling of pretopped double tee flange and ponding created by a curb

Photo #25 - Deterioration of beam end and column corbel caused by leaking of the surface joint
FLANGE CRACKING

Cracking in double tee flanges can be caused by several actions. The most common is shrinkage strains which are created during the production and curing cycle which build up in the flange region between the stems. During the curing process, the stem concrete is locked in place by the steel forms thereby restraining shrinkage strains in the flange between the stems. Such strains occasionally produce a crack in the flange adjacent to the inside corner of the juncture between the stem and the flange. These cracks can be minimized by proper curing techniques which retain moisture in the product.

Cracking can also be created during the stripping procedure. Such cracking is typically caused by elevating one end or one stem more than the others creating pressure on the flange from the steel form as the member pries its way out of the form. Another observed case of flange cracking is by excessive wracking of the tee as it is transported to the jobsite. The typical means of supporting these 60' +/- double tee members is either by pole trailer or an adjustable length flat bed or float trailer. These trailers are extremely flexible and have little or no transverse stiffness. Thus the tees can be warped severely getting to the jobsite on uneven road surface or crossing over curbs. Improper way of chaining double tees to a trailer can also crack the cantilever flanges. If pressure is applied to the outside edge of the flange by chains anchored to the trailer body twisting and warping of the trailer body can create cracks in the top of the outside flange or spalls at the chain location. This is avoided by elevating the dunnage so that the tie down load is transferred directly to the stem and not to the edge of the flange. It should be noted that seldom is any of this cracking (typically in the .002 to .005 of an inch in width) of structural consequence.

The summary of the inspections shows that flange cracking unless severe, seldom leads poor performance of the double tee. The survey spreadsheet shows a general deterioration of the flange cracks from an excellent rating of 8.6 for new structures to a rating of 5.5 for structures over 20 years old when maintenance is not performed. It was apparent that if serious cracks are not attended to, that deterioration can be anticipated.

Small cracks (< .004") are typically not through cracks and seldom effect the performance of the member. Therefore, nothing is typically done to such flange cracks if a field placed topping is to be applied. However, for pretopped tee systems in extreme exposure zones, such cracks can typically be sealed by painting the crack surface with silane. Low viscosity material such as silane is hyrophobic and penetrates into the crack effectively closing it down.
Continued...

Cracks .005 and larger are typically sealed by a surface application of a clear low viscosity epoxy. If the cracks are determined to be a working crack, then they are best sealed with a sealant such as urethane. Cracks .0010 and larger should be evaluated by an experienced engineer for proper repair.

At times, small horizontal cracks (<.005") at the ends of the double tee near the top of the stem where it joints the flange can occur. Such cracks are seldom structural and are usually caused by a concentration of prestress force in the bottom of stem for undraped strand pattern. If the deck is properly sealed, nothing need be done to these cracks.

Flange cracking is most troublesome in pretopped double tee parking structures. In topped systems, the cracked flanges are covered with a field placed topping which bonds to the surface essentially sealing the crack. For topped systems, unless the cracks are through the flange or deemed to be structural by the engineer, typically nothing is done. In the pretopped system however, cracking if large enough (> .005"), or not sealed, can allow water and water borne chlorides to penetrate and attack the reinforcing or leak through the flange.

Therefore, for pretopped tees, these cracks should be sealed as described above.

♦ SPANDREL CRACKING

This was reported to be the most prevalent product exhibiting cracking. Such cracking is typically observed to be diagonal torsion cracking particularly in "L" beam spandrels or cracking radiating from the bearing or connections. Diagonal cracking was observed to be much less when pocketed spandrels were employed since torsion is essentially eliminated. The ends of pocketed spandrels should receive sufficient stirrup reinforcing from the member end to the first pocket and "L" beams properly reinforced for torsion typically exhibit only fine cracks. To minimize end and connection cracking connections should be employed that allow the spandrel to relieve temperature strains. Bolted connections have proven to perform quite well. Fiber reinforced bearing pads at least 1/4" thick were also observed to perform better than plain bearing pads.
CONCRETE ADMIXTURES

Certain admixtures are fairly universally employed by precast concrete producers which have been proven to greatly enhance the durability of precast concrete products. These two are air entraining agents typically employed in all freeze/thaw regions and super plasticizers which reduce the amount of water necessary to produce a workable concrete mix.

Recent PCI sponsored research on chloride penetration into concrete generally substantiates that the most effective measure one can take to reduce the ability of water borne chlorides to enter concrete is to reduce the water cement ratio to a range of .35 to .40. Concrete with this water cement ratio have proven to be nearly imperious and the additions of other admixtures such as silica fume provide little substantial benefit.

The survey only included one project which employed silica fume. The performance rating of this three year old structure was rated at about average with a rating score of seven.

Two structures were reported as employing the corrosion inhibition DCI. One had a rating of seven and the other achieved a rating of 7.4. Because none of these questions in the survey do or could relate directly to the condition of the reinforcing, it’s difficult to determine the effectiveness of the corrosion inhibitor in this survey but no significant influence is indicated.

The importance of low water cement ratios is not unnoticed by the industry since 57% of the respondents reported they had employed water cement ratios of .40 or less in their products. Most of those who didn’t employ water cement ratios .40 or less were structures 10 years or older when the importance of low water cement ratios was not as well known. Therefore, as the industry became increasingly aware of the significant improved performance of products with low water cement ratios they embraced its employment.
PROTECTION OF EMBEDS

In a typical precast structure many steel embeds are left exposed to the elements. The types of exposed embeds employed are typically precast member bearing plates and connection plates between adjacent precast members in welded connections. These embeds are typically located below the deck surface and therefore they are somewhat protected from direct exposure to rain water, surface ponding or road salts.

Since they are located at member ends however, they are frequently located where leaks at the joints in the system can allow direct contact with leaking water, which may contain salt. To support this opinion, the worst rated embed condition (a 2 rating) also reported the worst leaking (also a 2 rating). Rusting of such exposed connections can also occur when exposed to moist atmosphere where no leaking occurs.

Regarding the protection of such embeds employed on typical structures, the results of the survey indicates that 31% had no protection, 25% were galvanized, 27% were zinc paint coated, 17% had a combination of protection (including galvanizing and zinc paint and galvanizing and stainless steel). The latter combination indicates different protection for different applications such as galvanized bearing plates and stainless steel double tee flange connections. Stainless steel was reported in only 10% of the applications and epoxy paint was employed by painting embeds in 10% of the cases.

Regarding the overall performance of exposed plates in the surveyed structures, the average totaled a good rating of 7.7. Those that achieved the highest ratings of 9 or 10 were either zinc painted, galvanized or epoxy painted with the majority galvanized. Typically the unprotected steel embeds were employed as one would expect for the older structures, and in the least severe exposure areas such as Albuquerque, Phoenix and San Antonio. It is also interesting to note, that in only one case did the non protected embed structure show a rating below 8. This can be attributed to the non severe exposure geographical location of these structures. This supports the exposure rating map (given in Site Inspection Section), where structures with the non-protected plates were employed all fell in the least severe exposure Zone I. It is generally observed that these non-protected plates at least received a simple shop coat of rust preventative paint.

Regarding epoxy coating of rebar, only 6% of the structures reported any use of epoxy coated rebar. The performance of the part of the structure where epoxy coated reinforcing was employed was mixed, with ratings ranging from 4 to 10. So, the effectiveness of epoxy coating rebars cannot be verified.
continued...

**SUMMARY**

It appears as though for the least exposure regions, no special protection of embed is necessary with only a shop paint rust preventative coat is sufficient. In regions where road salt is employed, the embeds protected with galvanizing performed well as did those protected with zinc paint and the one epoxy paint application. The projects which were inspected and exposed to road salt or atmospheric salt it appeared as though the zinc coating needs to be renewed in 7-8 years. This need to be reapplied, may also be caused by poor pre-coating preparation of the metal surface, because in cases where older ZRC coatings were performing well it was found that the precaster had properly cleaned and prepared the steel surface prior to coating. The results of a recent research project on cast in embeds with different means of protection which were exposed to salt solution indicate a slightly improved performance of epoxy coated assemblies over hot dip galvanized coating.*

Those structures with leaking problems also showed poor embed performance which also seemed to relate to poor drainage. Therefore, removing the water from the surface and sealing the surface joints also enhanced performance of the embeds.

*PSA Test Report #6 November, 1994*
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Photo #28 - Deterioration of embed plates

Photo #29 - Cracking in 22 year old pretopped double tee flange
The predominate lateral load system in both the structures reported in the survey as well as the nine structures inspected was a shear wall system. Of the 54 structures reported upon in the survey, 89% employed shear walls, 10% employed moment frames and 1% employed shafts alone. Thirteen percent of the structures also reported a combination system either employing shear walls plus moment frames or shear walls plus shafts.

The nine structures that were physically inspected all relied upon shear walls as their lateral load resisting system.

The performance of the shear wall lateral load systems as an average is quite good with an average rating of 8.2 with 50% in the 9 to 10 scale. The combined shear wall and shaft systems also perform very well with an average rating of 9.2. This would tend to belie the attitude that stair and elevator shafts need to be isolated from the structure. Apparently if appropriate connections are employed, such shafts can be included in the lateral load system. The nine structures that employed moment frames either alone or with other methods performed not as well with an average rating of 6.6.

The nine inspected structures, all of which employed precast shear walls or precast shear walls and precast shafts, were found to be performing well. It is interesting to note that all of these structures except one employed bolted connections between the spandrel and the columns and it was apparent that those connections appeared to relieve thermal strains. The structure that employed a flex-angle connection between the column and the spandrel exhibited distress at the connection. (See Inspection Report). Seven of the structures contained expansion joints which were performing good to fair. Since all of the expansion joints rated in the survey appeared to perform only fair at best, it's difficult to relate expansion joint performance with the type of lateral load system. The predominant connection type between the shear walls was a welded connection in a recessed pocket, which appeared to be performing well when properly protected.

Not surprisingly when the lateral load system was reported to be performing poorly, a rating (less than 5) typically excessive cracking was also noted in the double tees, beams and spandrels. This emphasizes the need for the structure to be allowed to breathe with each joint between the framing members able to yield to relieve the build-up of volume change strains and yet transfer appropriate loads.
Volume change in pretensioned concrete structures is caused by temperature change, creep, and shrinkage strains. These strains are accompanied by movements of concrete elements in the structure. In prestressed precast elements these movements are resisted and transferred by connections between members. To insure adequate performance of a structure, volume changes need to be determined in advance, the anticipated resulting movements predicted, and finally the resulting strains accommodated. Accommodating these strains in the design of a parking structure can be thought of allowing the structure to breathe. Each joint and connection can be looked upon as an opportunity to relieve the structure of the effect of these strains. Creep elastic shortening and shrinkage are direct strains, while environmental strains are cyclic. Figure 1 illustrates the different volume change strains. Creep strains can only develop in presence of a sustained compressive stress. Temperature strains depend on the magnitude of the temperature change, the temperature at erection and the length of the structure. In prestressed precast elements, the majority of the creep and shrinkage strains have already taken place at erection time.

The strains accompanying the volume change are seldom included in the calculations of connections but must be considered in the detailing of such connections. It was observed in the inspections that the effect of excessive strains on a poorly designed structure are mainly appearance related, but structural damages can occur and cracking can lead to deterioration (Photo 19). Volume change strains in a restrained conditions result in cracks. The designer engineer should allow for these movements by designing ductile connections. Welding both ends of a pretensioned member was typically observed to create cracks in the member ends (Photo #13) and should be avoided. Expansion joints have been used successfully in controlling temperature strains in long distances, however, their performance has been spotty. Bolted connections performed well to prevent restrained conditions. Figure 2 illustrates a typical ductile bolted connection. The void in the sleeve around the bolt permits lateral movement without impairing performance. In not one case where such connections were inspected was cracking at the connection observed. Figure 3 illustrates 2 typical layouts of a parking deck. The deck shown in (3a) might restrain the lateral movements of the deck because of the stiffness of the corner shear walls, this can be overcome by ductile connections. Deck shown in (3b) better accommodates the lateral movement of the deck because of the centers of stiffness are at the center of the structure. The requirement for expansion (contraction) joints is dependent upon many factors but the primary one being the length of the structure, the temperature gradient at the site and the ability of the connection to relieve strain. With appropriate consideration of these elements it was observed that structures performed well without expansion joints in the range of 300' +/- long. Such structures typically employed the type of connections described above and variable lateral load systems.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Fig 1: Concrete Strains due to Volume Change

Fig 2: Typical Ductile Bolted Connection
Fig 3: Typical Layouts of Parking Decks
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
The bibliographical summaries presented herein are the result of a partial literature search. All of the conclusions reached herein are either direct quotes or summary of the conclusions of the original authors.
Documented cases of durability from investigations and surveys of precast concrete have generally shown good performance of existing structures.

Quality concrete with low water/cement ratios has had a major impact on past performance. Some of the first bridge members produced over 35 years ago had 28 day concrete compressive strengths over 6,000 psi and water cement ratios less than .45.

The industry has also progressed to incorporate state of the art technology into its products. This has enabled the majority of precast structures to avoid common problems encountered in older cast-in-place structures such as improper air entrainment or corrosion from the use of calcium chloride admixtures.

Even though many structures have performed well, durability problems have been encountered. Some problems are common with cast-in-place reinforced concrete while others are unique to the precast prestressed industry. General observed durability problems include:

1. Failed joint sealants
2. Failed expansion joints
3. Corrosion of exposed plates
4. Corrosion of reinforcement
5. Corrosion of strand exposed at member ends
6. Failure of patches
7. Ponding of water
8. Failure of sliding bearing pads and bearing pads
9. Deterioration from volume change strains, pour detailing and improper construction techniques.
10. Carbonation
11. Alkali-silica reaction

Case studies show structures need to react to their exposed environment. Areas which use road salts can have high concentrations of chlorides in bridge decks and parking structures. Entrances to parking structures, ramps and drain locations are especially susceptible to high chloride levels. Very severe corrosive environments include splash zones of marine structures, cooling towers, coastal structures in warm climates and special chemical manufacturing processes.
continued....

Many problems encountered in older structures were associated with cast-in-place toppings, post tensioning details or pour detailing. Solutions to many of these early problems have been incorporated into common practice and should not reoccur.

The precast/prestressed concrete industry has successfully produced durable structures during the past 35 years with quality concrete being a major factor. Recent trends of pretopped double tees, water reducing admixtures and improved detailing will provide additional durability benefits. There is little evidence which would suggest the need for special materials such as silica fume, epoxy coated rebar or corrosion inhibitors except in severe corrosive environments.

Joint sealants, expansion joints and protection of connection hardware are the major durability problem areas common to most structures.
A. EPOXY-COATED REINFORCEMENT


A.3. "CRSI Comments, Concrete Reinforcing Steel Institute response to K. Clear’s article". Concrete International, May, 1992, Volume 14, Number 5 pg. 59-64.


continued...


A.11. "Rebar Protection Alternatives, Manufacturers' Point of View". George Reedy, Sika; Rob Lambe, Fosroc; Daniel Ruckstuhl, Master Builders; Howard Golden, ELGARD Corporation; Concrete Repair Bulletin, September/October 1993, pg. 10-13, 28.


continued...


♦ REVIEW

The review of the bibliography relating to epoxy-coating reinforcement (ECR) revealed that the industry's concerns can be summarized as follows:

- Bond of ECR bars and strands to concrete
- Special care required by ECR
- Performance at high temperatures
- Failure of some ECR structures in Florida Key

The use of ECR in corrosive environment is recommended by several highway departments. The recommended specifications and requirements to shipping, handling and placing of ECR in concrete seem to be inadequate. Further investigations by manufacturers and CRSI are being done to improve the quality of ECR. The effectiveness of epoxy coating to protect steel from corrosion in some cases has been well documented, however, it can only be achieved by adhering to strict shipping, handling and placing procedures. The main problems with ECR are associated with the inflicted defects caused at the jobsite and improperly applied coating. In the production plant typically the quality of epoxy coating on bars or strands is controlled to insure no or minimal defects. All the researchers dealing with the performance of ECR in concrete recommend the following steps be taken to insure adequate embedded ECR bars or strands.

- Coating thickness should be between 5 to 7 mils.
- Holidays (pinholes) should be limited to less than 2 per linear foot.
- In prestress and critical reinforcing, where bonding is a priority, the use of grit impregnated epoxy coating is essential.
- While heat curing, temperature near the ECR bars or strands should be limited to 150°F or less.
- Precautions need to be taken to prevent scraping and damaging the coating during transport, storage, and placement.
continued...

- Repair of damaged coated areas, cut or bend bars should be performed on jobsite.
- ECR strands should be kept on reels and ends coated after each cut.
- Flame cutting should be minimized, instead saw cutting is recommended.
- Specials provisions for anchors, chucks and wedges must be made when using ECR strands due to the added diameter from the coating.
- Where fire ratings are necessary the use of epoxy coated reinforcement is not appropriate due to the loss of adhesion of reinforcement to concrete.
- Vibration and consolidation of concrete during placement should be performed with non metal vibrating heads to reduce the risk of damaged coating.

It was noticed from past experiences that cracks in concrete elements reinforced with ECR are larger than cracks of similar concrete elements reinforced with conventional reinforcement.

The use of epoxy coated reinforcement in concrete can not guaranty a good protection against severe aggressive environment. Other means of protection systems need to be used in conjunction with ECR in order to reduce the risk of durability problems. The recent extensive study by the TxDot (Texas Department of Transportation) and the University of Texas on ECR concluded, "Epoxy-Coated reinforcement in highly corrosive environment was susceptible to chloride-induced corrosion governed by the level of coating damage and extent of debonding of the coating from the reinforcement, degree of concrete consolidation around bars, concrete cracking and macrocel formation".

The study also confirmed the general belief that damaged ECR are susceptible to pitting corrosion. Furthermore, the definition of damaged coating was extended to include hairline cracks in the coating.

The conclusion reached from the literature review and past experiences is that epoxy coated bars must be free from any defects, the coating properly applied, and special care taken in concrete placement. Unless this can be achieved, the use of epoxy coated reinforcement in concrete structures should be limited.
PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
BIBLIOGRAPHY

B. CONCRETE SEALERS


continued...


REVIEW

The review of the literature available on sealers revealed that there is a confusion on what are the most effective sealers to combat a specific durability problem. There are several marketing claims from the manufacturers with little published research. The interest of the concrete industry in sealers however, initiated research to study the mechanism and the performance of sealers in concrete. Sealers in concrete come in different types and chemical composition, choosing the right sealer for specific use can be achieved based upon past experience or tests. A few articles have been published studying a specific kind of sealers which will be reported herein. Concrete sealers are chemical products and some can have a negative side effect on concrete due to their chemical composition. ACI committee 515 report summarizes the effect of most chemical compounds on concrete.

Concrete sealers can be separated in two major groups:

- Penetrating sealers
- Surface sealers, very thin layer of coating applied on the surface.

Sealers are intended to protect concrete from penetration of water, and primarily the penetration of water soluble chloride ions. Their main purpose is to serve as a barrier to concrete against damaging environments.

Evaluating sealers by the design engineer is critical because of the lack of specific research pertaining to a given sealer. The method proposed by WJE is perhaps costly but can reveal to be cost effective saving in unnecessary repairs and recoating.

The application of the sealing agent is as critical as choosing the right one. The application of sealers is specified based on coverage and percent soluble solids. There are also added costs involved with sealers which need to be considered.

The selected studies on the performance of concrete sealers from the available literature are summarized herein.

1. Penetrating Sealers

Laboratory investigation of various sealers revealed that while most sealers reduced the penetration of water to approximately one third, some sealers had little effect or actually...
increased the water penetration when compared to unsealed specimens. After choosing a sealer, its application on the concrete is the second most critical consideration. Most researchers agree on application on dry surface and moderate ambient environment. Surface preparation of the concrete and a thorough application of the coating can produce a cost effective barrier. The ambient environment dictates the general requirements on a given sealer. In addition to a careful installation of sealers additional precaution need to be taken to prevent future failure of the sealing product. The difference between indoor and outdoor use can demand different type of sealers. Outdoor sealers are exposed to the environment constantly. Sealers used outdoors need to be breathable, resistant to wind driven rain, UV stable, Flexible, and Alkali and Fungi resistant. Typical penetrating sealers are silane(alkyltrialkoxyssilane) and siloxane(oligometric alkyltrialkoxyssiloxane). Typical application rates are 125 sq. ft. per gallon and a minimum soluble solids level of 40%, applied on air dried surfaces. Typical penetrating sealers do not affect the appearance or traction of a treated surface.

2. Surface Sealers

Coating is generally applied to concrete floors and exposed concrete surfaces. There are at least ten different types of chemical resistant surface sealers. Each type contains several different products. In selecting the proper coating the advantages and disadvantages of the coating need to be weighed against cost and need. In any coating situation the main concern should be the moisture resistance and the performance of the sealer with regard to retention and breathing. The ultimate goal is for a sealer to protect concrete from excessive wetting and harmful solutions yet permit breathing and evaporation of excess moisture at the interface. Since large surfaces are highly subject to volume movements, flexibility of the coating is essential to resist the structural and environmental changes. Surface sealers are wearable due to abrasion or weathering. Damaged coating due to abrasion reduces its effectiveness and requires recoating. The application of surface coating produces a darker surface which tends to reduce the lighting of a structure by decreasing light reflection. Coated surfaces are more slippery than surfaces treated with penetrating sealers. In slippery surfaces an abrasive grit in the top coat is highly recommended. Epoxies and Urethanes are typical surface sealer materials.
BIBLIOGRAPHY

C. CONCRETE MIXES AND ADMIXTURES


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◆ REVIEW

Achieving a durable concrete structure starts with concrete which constitutes most of its volume. Selecting high quality ingredients and a well designed proportioning of concrete ingredients enhances all its properties\(^{27}\). The main practice of concrete mixtures involves a balance between strength and workability. The ultimate goal of a concrete mix design is to reach a desired strength and workability optimum with a minimum water content. Figure C.1 illustrates this concept, where strength is measured as the failure under axial compression and workability measured in terms of fresh concrete slump\(^{27}\).

![Fig C.1. Strength-Workability Relationship](image-url)
Chemical admixtures were introduced to enhance fresh concrete properties such as water demand, air content and cement hydration rate. Chemical admixtures used to reduce the water demand of a concrete mixture are labeled water reducers. High range water reducers (HRWR), commonly called superplasticizers, have been used successfully. The chemical admixtures used to influence the hydration of cement are retarders and accelerators which can slow or speed the hydration process respectively. Another admixture that is widely used is air entrainment agent which improves the hardened concrete void system. Inhibitors have also been introduced as fresh concrete admixtures to enhance the corrosion protection of steel imbedded in concrete. The immediate effect of these admixtures are well known since it can be measured however their long term effect on hardened concrete needs further investigations. The wide use of chemical admixtures justified the increased interest of researchers on the durability of concrete containing these admixtures. The scope of this research program limits its investigation to the effect of high range water reducers (HRWR) and air entrainment on the durability of concrete.

Mineral admixtures were introduced as a partial replacement in concrete mixtures. Mineral admixtures can partially replace cement in amounts ranging between 5% and 30%, depending on the admixture. The most widely used mineral admixtures are fly ash, silica fume, and blast-furnace slag. The last two decades have seen an increase in the use of fly ash and silica fume because of their availability. The wide use of mineral admixtures prompted several studies to investigate their effect on concrete properties.

FLY ASH

Fly ash is relatively inexpensive and if used properly can enhance several concrete properties. ASTM standards classify fly ash in different categories (C, F, N, etc.) the most commonly used fly ashes are ASTM Class C and ASTM Class F. All fly ashes act upon concrete through the pozzolanic activities which consume hydration product. ASTM Class C fly ashes were found to have cementitious properties in addition to pozzolanic characteristics. Fly ashes are by-products of burning plants, near by concrete manufacturers usually are their main consumers. Fly ash was found to improve the long term strength gain, abrasion resistance, permeability and workability. Recent tests on macrocell specimens have shown that fly ash reduces the risk of rebar corrosion. Some problems associated with fly ash are low early strength, susceptibility to carbonation and slightly higher shrinkage.
continued...

♦ **SILICA FUME**

Silica fume is a mineral admixtures used in high strength concrete\(^{(a,b)}\). Silica fume in general, is more expensive than cement which increases the cost of concrete. Silica fume can be used as an added admixture or as partial replacement of cement. The amount of replacement is still debated, however, most researchers agree on a range between 6 and 10% replacement\(^{(a,b,c)}\). Silica fume improves strength and permeability. There are some problems associated with the use of silica fume such as low flow, large shrinkage strains, and a very low bleeding which makes surface finishing a delicate operation\(^{(a)}\). Another problem is the finished color which is darker than conventional concrete. The concern is mostly in parking garages where the light reflection is reduced. The high strength and low permeability accompanying the use of silica fume in concrete, in some cases where curing, finishing and shrinkage problems have been overcame, has resulted in a higher protection against chloride penetration to steel reinforcement\(^{(a,b)}\).

♦ **HIGH RANGE WATER REDUCERS (SUPERPLASTICIZERS)**

Superplasticizers are highly recommended in concrete. HRWR improve strength, workability and reduce permeability through the reduction of the water demand. Lower water-cement ratio will inevitably result in high quality concrete. Superplasticizers were found to have little direct effect on concrete except through the reduction of the water cement ratio. The dosage used in concrete mixtures are recommended by the manufacturer however trial batches are recommended. A concrete mixture with high superplasticizers content will increase the flow of the mix resulting in high bleeding and segregation\(^{(a,b)}\).

♦ **AIR ENTRAINMENT**

Air entrainment agents are chemical admixtures used in concrete to enhance its air void system. Hardened concrete containing air entrainment admixture has a uniform porous texture. The uniform porous structure of the mortar matrix was found to impede the movement of fluids within the matrix by impairing the capillary system in the matrix. Air entrainment agents were found to reduce permeability and improve the freeze thaw properties of concrete\(^{(a)}\). The ACI code recommend limiting air entrained content of concrete to 6% in moderate environment and 7.5% in severe environment.
D. CASE STUDIES


D.2. "Study Shows Maintenance Key to Garage Success". ASCENT - Spring 1993, Precast/Prestressed Concrete Institute, 175 West Jackson Blvd., Chicago, IL 60604-9773, pg. 16-21.


D.13. "Renovation to Paulina Street Parking Structure". Construction Technology Laboratories, Inc. for American Concrete Institute 1993 Fall Convention, November 7-12, 1993, Minneapolis, Minnesota, pg. 1-36.


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Documented cases of durability from investigations and surveys of precast concrete have generally shown good performance of existing precast structures.

Quality concrete with low water/cement ratios has had a major impact on past performance. Some of the first bridge members produced over 35 years ago had 28 day concrete compressive strengths over 6,000 psi.

The industry has also progressed to incorporate state of the art technology into its products. This has enabled many precast structures to avoid common problems encountered in older cast-in-place structures such as improper air entrainment or corrosion from the use of calcium chloride admixtures.
Even though many structures have performed well, durability problems have been encountered. Some problems are common with cast-in-place reinforced concrete while others are unique to the precast prestressed industry. General durability problems include:

1. Failed joint sealants
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9. Deterioration from volume change strains, pour detailing and improper construction techniques
10. Carbonation
11. Alkali-silica reaction

Case studies show structures need to react to their exposed environment. Areas which use road salts and seawater exposure can have high concentrations of chlorides in bridge decks and parking structures. Entrances to parking structures, ramps and drain locations are especially susceptible to high chloride levels. Chloride Readings of 2 to 3 times the threshold limit have been found near topping steel within 10 to 15 years of service. Very severe corrosive environments include splash zones of marine structures, cooling towers, coastal structures in warm climates and special chemical manufacturing processes.

Many problems encountered in older structures are associated with cast-in-place toppings, post tensioning details or pour detailing. Solutions to many of these early problems have been incorporated into common practice and should not reoccur.

**SUMMARY**

The precast/prestressed concrete industry has successfully produced durable structures during the past 35 years with quality concrete being a major factor. Recent trends of pretopped double tees, water reducing admixtures and improved detailing will provide additional durability benefits. Durable structures have been built without special materials such as silica fume, epoxy coated rebar.
or corrosion inhibitors but the use of these materials in increasing and beneficial results are provided in severe corrosive environments.

Joint sealants, expansion joints and protection of connection hardware are the major durability problem areas common to most precast structures. Connection detailing, increased rebar cover and crack control will also need improvement if service life is to be extended.
E. CURING


E.9. "Accelerated Curing of Concrete at Atmospheric Pressure-State of the Art". ACI Committee 517. American Concrete Institute, ACI 517.2R-87 Rev. 92, Detroit MI.
continued...

E.10. "Standard Practice for Curing Concrete". ACI Committee 308. American Concrete Institute, ACI 308-92, Detroit MI.


♦ REVIEW

According to ACI Committee 308 curing concrete is defined as ".. the maintaining of satisfactory moisture content and temperature during its early stages ..". In order for concrete to achieve acceptable durability and strength it has to be properly cured.

Concrete temperature and moisture are the most important factors in proper curing. Therefore, providing adequate ambient temperature, and preventing evaporation are in essence critical to curing practice.

Improper curing practice can have devastating effects on fresh and hardened concrete. It has been demonstrated that for temperatures less than 50°F little early strength develops, less than 40°F no early strength develops, and at 32°F or less no strength develops at all. Higher temperature curing usually ensures adequate curing; however, while high temperature (<150°F) achieve high early strength, it may reduce later strength.
The moisture content of concrete is another factor to consider when curing concrete. Poor prevention of evaporation can result in cracking, dusting, scaling, crazing, and low strength development.

There are several methods used in the precast prestressed industry to cure concrete. Accelerated curing is the most widely used method because of the advantages of an early strength development. Since most casting is done in controlled environment, a better quality cured concrete can be achieved. Accelerated curing is accomplished by raising temperatures and preventing evaporation. Steam curing is ideal, since it offers simultaneously high temperatures and moisture. There are certain precautions that need to be taken in steam curing practice. First a minimum of 3 hours of presteaming is essential. Steaming temperatures must range between 150°F and 180°F, and finally, there should be an adequate cooling period.

The reduction in strength due to a lack of moisture can be as high as 60% when dry cured specimens are compared to moist cured specimens. The prevention of water evaporation is accomplished by several methods such as moist burlaps, curing blankets, plastic sheets, and curing compounds. Any method used should eliminate the evaporation of water but most importantly avoid any wetting and drying cycle. Any drying during the curing period will result in shrinkage cracks. Burlaps need to be washed before use to avoid chemical contamination. They should also be heavy to absorb enough water to avoid repetitious wetting. During placement moist burlaps should be overlapping each other to avoid exposed concrete surfaces. Plastic sheets have been used successively in trapping concrete moisture during curing. Curing blankets offer an added advantage of insulation, however, precautions need to be taken to avoid the absorption of the water from the concrete surface. Some curing compounds are not recommended for concrete surfaces destined to be topped because the bonding of the cured surface to the topping may be partially impaired. All these water retainers are recommended to be applied immediately after finish. A short delay in placing the curing covers should not affect curing in concrete so long as bleeding water is available. If the concrete has a very low bleeding, no delay in placing curing covers should be permitted because the dried concrete surface will have severe shrinkage cracks as in the case of high performance silica fume concrete.

The precast industry favors the use of atmospheric heat curing by applying heat to the forms either electric or hot liquid. Heat curing should be applied after reasonable presetting time. Performing penetration test to determine the setting time will improve the curing. The raise in curing
temperature should be done at moderate rate, ranging between 20°F and 40°F per hour. The curing temperature should be maintained as required, followed by a cooling period.

Curing can affect considerably the strength and durability of concrete because of its effect on the concrete microstructure. Special precautions need to be taken when curing large concrete surfaces such as slabs and overlays because of the large evaporation surface. The deterioration of concrete surfaces due to improper curing can result in permeability of the concrete at the surface due to shrinkage cracks.

An adequate curing of concrete was found to enhance the durability properties of concrete. The curing temperature for precast prestressed concrete members should be limited to 150°F.
F. FREEZE-THAW


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♦ REVIEW

There are two mechanisms associated with freeze-thaw, hydraulic pressure and ice accretion. The mechanism of hydraulic pressure is the most damaging one. Upon freezing, water in the capillary pores within the cement paste tends to expand. The expansion requires a volume larger than the existing voids in the concrete. The pressure of expansion drives off the excess water creating a hydraulic pressure. If this pressure exceeds the tensile strength of the cement paste, cracks occur. These cracks allow more water to penetrate, once freezing occurs again the damaging cycle repeats until failure. The failure due to freeze and thaw depends greatly on the strength and permeability of the cement paste as well as the void system in the texture. The existence of a void system is not enough to avoid damages from freeze thaw damages. The uniform distribution of the voids as well as the distance between voids critical. Air entrainment agents have been used successfully in both regular and high strength concrete. There is a balance of strength versus air content that needs to
be achieved. Indeed, high air entrained concretes tend to have lower strength than comparable non-air entrained concretes. The use of air entrainment in areas of freeze-thaw risks is highly recommended. Comparative tests performed on air entrained and non entrained concrete clearly show the advantages of using the admixture.

It has been reported that high volume condensed silica fume, 20% or higher, increased the susceptibility of concrete in freeze and thaw damages. Low percentages of silica fume content is recommended. ACI durability requirements limit the air content in concrete to 6% in moderate environment and 7.5% in severe environment.

The hydraulic pressure mechanism assumes a high freezing rate. Where freezing rates are relatively slow, pressure may build up due to the accumulation of ice in the capillary void system. Slow freezing rates condition induce ice accretions which alone have no immediate effects however they have a cumulative effects which can be damaging after a number of winters. The hydraulic pressure mechanism of freeze-thaw has rarely been reported in North America.
G. JOINTS AND MEMBRANES


Membranes are composed of several layers acting uniformly. The first layer, primer, insures a strong bond of the membrane to the concrete surface. The effectiveness of membrane has been proven in corrosive and high traffic environments. When using toppings on high traffic surfaces the main characteristics of the topping to be considered are permeability, adhesion, elongation, strength, flexibility, and abrasion resistance. The difference in physical and mechanical properties of concrete and membrane material needs to be considered in design. In corrosion prevention, membranes have been used successfully in protecting the concrete from penetration of chlorides to the reinforcement by preventing the migration of water soluble chlorides.

A study on several precast parking structures by PCI has revealed that membranes are typically used as traffic membranes rather than buried membranes. Membranes were found to be effective when used as

- a secondary protection over occupied space,
- strip membrane over joints or gutter lines
- water proofing membrane
- surface repair method of badly scaled areas.

Epoxy traffic toppings were found to be not as effective as urethane polymers based systems because of the reflective cracking over joints present with epoxy based systems.

The typical application of membranes consists of placement of two to three layers. A layer of 30-40 mils in thickness is first applied followed by a second layer of typically 20 mils in thickness and finally a traffic layer 10-20 mils in thickness which also contains a grit for traffic traction. Such traffic bearing traffic membranes have successfully been employed over occupied spaces in parking structures.
Joint Sealing

Joint sealing requires the appropriate material and a thorough installation. The most often repairs performed on structures, especially parking garages, involve joint sealers. There are several joint sealers available however they can be grouped in six categories, mastic, thermoplastics hot-applied, thermoplastics cold-applied, thermosetting chemically cured, thermosetting solvent release, and compression seal. The choice of the appropriate one depends on the environment and the intended use. There are specific methods and specifications outlined by ACI committee 504 "Guide to Sealing Joints in Concrete Structures" which can improve the quality and performance of sealed joints. The typical material employed in parking structure joints is urethane. Such sealers should be UV resistant.

A study on the performance of joint sealers has revealed that the life expectancy of a joint sealer depends greatly on the type of sealer used and the sun exposure. Urethane based sealants were found to be the most widely used. Some urethane based sealers however exhibited hardening and weathering deterioration sooner than others. The leaks through joints can be traced back to concrete edge failure rather than joint material failure. These failures are caused by misuse or extraordinary wear. Deck-joints were found to perform very well under traffic, even in width of 2 inches or more.
H. CORROSION


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| REVIEW |

Corrosion of reinforcement in precast structures can cause structural failure and architectural deterioration. The structural failure is associated with loss of reinforcement and the architectural failure is of aesthetic nature.
The corrosion of steel in concrete is caused by either chloride ions or carbonation. The carbonation of concrete is only a few millimeters a year in an ordinary environment which reduces its effects on corrosion. The chlorides, on the other hand, are present in different forms. Their origin in concrete can be traced to the following sources:

- Seawater
- Acid rain
- Salts used in deicing snow and ice on structures
- CaCl\textsubscript{2} based admixtures
- Coarse and fine aggregates from marine origin
- Water contaminated by seawater.

The most typical form of chloride induced corrosion is from deicing salts. The chloride ions migrate through the concrete in solution. The migration can either be physical, through the cracks and the void system, or by capillary penetration.

While the chloride ions do not react with the steel, in sufficient quantity, they cause the average pH in the vicinity of the reinforcing bar to drop below 11\textsuperscript{16,26}. A study on the performance of joint sealers has revealed that the life expectancy of a joint sealer depends greatly on the type of sealer used and the sun exposure\textsuperscript{10}. At a pH level under 11 the passive film provided by the alkalis in the concrete is destroyed leaving the steel surface unprotected \textsuperscript{26}. The moisture and oxygen start the corrosion process. The by-product of the corrosion reaction, rust, is larger in volume which creates microcracks in the concrete. Figure 1 illustrates the failure mechanism\textsuperscript{16,26}.

![Diagram of corrosion mechanism]

\textbf{Fig 1. General failure mechanism of corrosion of rebars in concrete.}
continued...

The solution to corrosion of reinforcement in concrete rely mainly on protecting the embedded steel. This can be divided into three major categories

1. Concrete enhancement: make concrete impermeable to harmful chemicals
2. Steel protection: enhance the protection of steel
3. Chloride elimination: eliminate the source of the harmful chemicals.

♦ Concrete Enhancement

- Low permeability concrete, usually achieved by using chemical and mineral admixtures or very low water-cement ratio. Water cement ratios less than 0.35 have shown to produce best results.
- Concrete surface sealers, chemical products used to partially penetrate the concrete and prevent moisture or soluble chemicals from penetrating to concrete. Tests indicate that silane and siloxane to be effective.
- Membranes, thick epoxy or urethane based layers which are impermeable.
- Sulfur or polymer concrete which require no mixing water. Limited to very special applications.
- Increase the concrete cover. A minimum of 1.5 inches in salt exposure regions is recommended.

♦ Steel Protection

- Epoxy coated rebars, a layer of epoxy or polymer based product is applied to the steel surface which act as a barrier. See section A for epoxy reinforcing requirements.
- Use of stainless or hot dip galvanized steel.
- Inhibitors, chemical products which enhance the protection system by acting on the surrounding environment or the embedded steel.
- Cathodic protection, a metal bar is embedded in concrete to reduce the electrochemical reaction causing corrosion. Cathodic protection is not recommended where prestressed steel is employed.
Chloride Elimination

- Eliminate deicing salts, other alternatives are being pursued to replace the use of deicing salts.
- Wash aggregates which originate from a salty environment.
- Eliminate the use of CaCl2 based admixtures.
I. PROTECTION SYSTEMS


1.11. "The Design of Products to be Hot Dip Galvanized After Fabrication". American Hot Dip Galvanizers Association. MA-3 5M 11/84, (Revised 10/83), pg. 4-5.


1.13 "ZRC Keeps the Roof on Water Treatment Plant". Rust Notes, ZRC Product Company, fall 1994.


♦ REVIEW

There several protection systems that can be used to combat corrosion. This review deals with protection systems applied to the steel. The common methods used to protect steel from corrosion are:

- Galvanizing
- Stainless Steel
- Coatings
- Cathodic protection
In the precast industry these systems are typically applied to the exposed embedded plates and connections. While cathodic protection has been used extensively in bridge construction, its use in building structures is still limited and is not recommended for prestressed products. The other protection systems are specified in high corrosive environment or exposed connections. Stainless steel has been proven to be corrosion resistant metal in the metal industry, its use in the precast industry is still limited due to the initial cost. The immediate cost of stainless steel in precast structures is higher than any other system however the overall cost over the life span of the structure is very competitive.

Regardless of the protection system, the weakness of the application typically resides in the effect of welding. During welding operations, the coating is either impaired or completely removed. In some cases embrittlement is accompanied with extensive welding. After welding, the welded area should be recoated with the appropriate product.

The use of hot dip galvanized connections have been successful, however in corrosive environment recoating of the welded areas is necessary to prevent corrosion. ASTM A143-74 outlines a safe method to safeguard against embrittlement. The American Hot Dip Galvanizers Association has published design recommendations on the use of hot dip galvanized products. There are also several design and specification guidelines on the use of stainless steel in concrete.

In moderate corrosive environment, where moisture is the main concern, epoxy coating, ZRC, or Painting is recommended due to the relatively low cost.
J. ALKALI-SILICA REACTION


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Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
Alkali-Silica reaction (ASR) is a concrete durability problem caused by a deteriorating reaction between alkali present in the cementitious material gel and silica present in a reactive aggregate. Other compounds in cement, such as carbonate and silicate, react with aggregates. These reactions are known as Alkali-Aggregate reaction (AA). The reaction between alkali in the cement gel and the silica in the aggregate, in presence of moisture, is expansive. The expansion creates cracks in the mortar which invites outside intruders which in turn can cause corrosion damages to the reinforcing steel or freeze-thaw damages. There are three conditions necessary for ASR to occur:

- Presence in sufficient amounts of reactive silica in the aggregates
- Sufficient available Alkali in the cementitious gel
- High moisture content (80% relative humidity)

Once these factors occur simultaneously the reaction proceeds in two major steps:

- Alkali reacts with Silica to form a gel reaction product (Reaction)
- Gel reaction product in presence of moisture expands (Expansion)

The amount of moisture necessary to activate the expansion has long been debated, a relative humidity of 80% has been adopted by most researchers.

Alkali-Silica reaction potential problems can be minimized at the materials selection level. Once ASR in concrete is detected, the cost of repair and prevention can be very high. Preventing ASR in concrete consists of acting upon one of the conditions necessary for the evolution of ASR. The concrete mixture should contain at least one of the following:

- Low Silica content Aggregates
- Low Alkali content cement
- Mineral admixtures (fly ash, silica fume, slag, etc.)

Furthermore, a reduction in moisture in concrete by means of very low permeability or weather proofing compounds (sealers, membranes, etc.) can eliminate the moisture necessary to activate the expansion.
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Once ASR is detected in concrete, steps need to be taken to prevent further intrusion of moisture in the concrete by using sealers or crack injection. Repairs on damaged concrete should be performed with concrete containing very low alkali since the aggregate are reactive.

Standard Laboratory tests can be utilized to detect ASR. ASTM recommends several of such tests such as C289-87, C277-90, C9, and C441-89. In the field, a petrography examination of the aggregate is widely used.

Delayed ettringite formation in concrete is often mistaken as ASR because of the similarity of their effect on concrete. If the ettringite formation is delayed until concrete has hardened, the expansion in presence of moisture creates cracks in hardened concrete that look similar to ASR cracks.
SUMMARY OF CHARACTERISTICS OF GOOD PERFORMANCE
TO MINIMIZE SURFACE DETERIORATION

Virtually all studies on corrosion recognize the importance of low water cement ratios in producing durable low permeability concrete. The majority of precast products in the survey were reported as having concrete with water cement ratios .40 or less. Based upon this, the most durable structures would be ones that minimize field placed concretes with high water cement ratios. Thus the pretopped double tee system would produce the most durable concrete at the riding surface as well as throughout the structure. If field placed topping is employed, then a water cement ratio of no more than .45 is recommended.

In severe exposure regions, the application of a penetrating sealer such as silane has proven beneficial. Siloxanes have also been employed although with a somewhat lessor degree of success because of a lessor penetration than silane. Note, in severe exposure regions silanes should be applied to all levels since road salt exposure typically has more impact on the entrance and lower levels.

WARPING OF DOUBLE TEES

Because of varying slopes for drainage, double tees are typically twisted or warped, with one end or pair of supports at a different angle than the other end or pair of supports. As the difference between the two angles increases, the tensile capacity of the concrete is exceeded and the tee cracks, typically at the top of the flange. The crack is located at the inside surface of the flange between the two stems.

To determine the maximum angle or vertical difference between two supports a finite element analysis was run for a typical 60’ long double tee. Assuming an average actual service compressive stress of 6,500 psi and modules of rupture of 975 psi, a 5/8” value of vertical offset was reached.

To evaluate crack propagation a 1” deep crack was evaluated. It was determined that such cracks will propagate along a flange rather than through a flange. These cracks can be contained by typical flange mesh reinforcement.

To further substantiate the amount of differential support displacement that would produce cracking, a simulated warp test was conducted in a prestressed plant.

The results of these tests substantially support the fact that warping of double tees should be kept to a level of 3/8” between 5'-0” spaced stems or 1 1/8” total width of a 10’ double tee.
Because of varying span length and stiffness of double tees, this limit may vary. Also to be considered is whether the double tees are topped or pretopped. For tees that are topped, flange cracking created by warping is typically not a consideration since the topping provides the deck surface and is not effected by flange cracking. Pretopped systems however are more sensitive to flange cracking since they can lead to leaking or possible corrosion of flange reinforcing.

Therefore, for pretopped double tee systems, a warp limitation of 1½" per 10'-0" tee width is recommended. However, no evidence was found to suggest cracks created from warping produces significant detrimental effects. Structures over 20 years old were observed to have top flange cracks with no visual evidence of rebar corrosion or delamination which often occurs in C.I.P. toppings.

**TO MINIMIZE LEAKING**

Because of the exposed nature of parking structures, it is virtually impossible to promise a completely leak proof structure regardless of the construction method or material. Extremely water tight structures constructed with precast concrete are achievable, however, controlling leaking is important not only for safety and nuisance values but primarily since leaking leads to deterioration of connections and sensitive bearing ends of precast members. Some of the key considerations are as follows:

**PROVIDE GOOD DRAINAGE** - Getting the water quickly off the structures surface is one of the keys to minimizing leaking. A positive slope should exist throughout the structure with no flat spots or areas to trap water. Typically, structures are sloped to the interior for drainage a minimum slope of 1½ % is recommended. Crickets are typically created on beam lines with minimum slopes of ½ %.

Drains should be located as close to column grid lines as possible and the area immediate to the drain additionally sloped to minimize local ponding. Floor drains should be large enough with at least dimension of 11". Trench drains are typically employed at the bottom of the roof level ramp. They are not recommended to be placed continuous across a span but spaced out so as not to interrupt the floor framing system.

**SEAL ALL JOINTS** - The joints between precast members are the primary source of leaking in a structure. With the possible exception of dry climates in exposure Zone I, all joints between precast deck members should be sealed. Because water collects on the deck surfaces whether topped or pretopped systems are employed, all double tee flange to flange connection and all double tee flange to beam spandrel, column or wall joints should be sealed. In addition, joints where beams abut columns and all around columns and walls should be sealed.
It is important that these joints be properly prepared prior to sealing. In topped systems, the topping should be tooled (not sawn) to a depth of $\frac{1}{2}\text{"} \pm$ and all scale or latence removed by grinding prior to installation of the sealant. Likewise, flange joints in pretopped systems should be ground or stoned to remove any sharp edges or loose material prior to sealing. The sealant installer should be experienced in sealing parking decks or leaking can occur through improperly installed sealants. Note properly installed urethane sealants were observed to maintain their water tightness from seven to ten years with periodical replacement of cracked or torn sealant sections. Rarely is a full replacement of all joints performed at one time.

When pretopped systems are employed, the precast producer must pay particular attention to member size tolerances or wide joints can occur which are difficult to seal properly or maintain water tight.

If through cracks in the deck surface exist, then these should be sealed. Such cracks are seldom observed to be structural and can be sealed with a urethene sealant. Silane penetrating sealers have proven effective in sealing fine cracks ($<.005\text{"}$) by painting over or drizzling into the cracks.
Significant corrosion and damage can occur when steel embeds are exposed or if reinforcing steel is exposed to moisture. The severity of the corrosion has been shown to be dependent upon the location of the structure regarding the exposure zones, although some corrosion has been observed even in exposure Zone I for unprotected embeds.

**PROTECTION OF EMBEDS** - The protection of bearing plates is particularly important since they are located at the ends of members where leaks frequently occur.

The most frequently employed means of protection of exposed plates is galvanizing or zinc rich coatings. Galvanizing was observed to perform somewhat better than ZRC coatings however galvanizing has the negatives of frequent 1-2 week production delays while the embeds are sent to and are returned from the galvanizer and some experiences with strain-age embrittlement and hydrogen embrittlement of cold worked anchor bars. Therefore, if galvanized is specified, time must be provided for in the schedule for the galvanizing process and the recommendations of ASTM A143-74 must be adhered to.

Another difficulty with employing galvanizing is where welding is required. The galvanizing must be ground off to achieve proper welding and noxious fumes are emitted. Thus, a non-galvanized surface is produced in a most critical location. Such connections are later typically touched up with surface coats of cold galvanizing material in the weld location.

The success of ZRC coatings depends a great deal upon proper preparation of the surfaces to be coated. The manufacturers recommendations must be followed regarding cleaning and preparing the steel surfaces for the coating to achieve good long term performance. Manufacturers tests on cold galvanizing materials show as good or better performance than hot dip galvanizing.

A minimum protection of exposed embeds even in Exposure Zone I is recommended to be a shop coating of rust preventative paint and a field touch up where welding has occurred.

Another effective means of protecting exposed embeds is an epoxy coating of the exposed portion of the assembly. This can be achieved by assembling the embed with its anchors and then epoxy coating the exposed plate with a high quality epoxy paint and not coating the anchors so as to not effect the bond of the anchors.
Some manufacturers avoid the use of bearing plates altogether by carefully detailing the end reinforcing to be properly anchored and developed. Some risk is possible since the reinforcing cannot be observed that it has been properly located after coating. Producers have successfully overcome this by detailing the end reinforcing so that it either becomes an integral part of the main member reinforcing or it is carefully tied and chained in the correct position.

Stainless steel has also been successfully employed typically at the least protected flange to flange connection. Special welding rods and techniques are required where stainless steel is employed in welded connections. Care must also be take to allow the cast-in portion of a stainless steel connection to expand during the high heat required to weld stainless steel.

One of the observed primary causes of deterioration of cast in steel embeds is through cracking of the concrete surrounding the connections. Therefore, the system of connections in an exposed structure such as a parking deck must allow the members to breath or relieve internal strains (primarily temperature) rather than crack the concrete. Bolted connections, or welded connections which allow some rotation or flexing have proven to be successful in allowing a structure to breath.
CONCRETE ADMIXTURES

Many materials are available to enhance the performance of concrete members and the contained reinforcing against corrosion.

Unfortunately, both observations and survey reporting were inconclusive as to the benefit of special admixtures over the application of sound concrete mixes with low water cement ratios and adequate cover over reinforcing.

The most beneficial admixture have proven to be the super plasticizers, to reduce water demand, an entraining agent in freeze thaw regions, and fly ash in some mixes as an aid to produce higher strengths and perhaps reduce rebar corrosion.

As mentioned above, the use of Silica fume has been inconclusive and high shrinkage and finishing problems are a concern. Corrosion inhibitors have not been employed in precast members long enough to properly evaluate.
PCI RESEARCH PROGRAM
DURABILITY OF PRECAST PRESTRESSED CONCRETE STRUCTURES
SUMMARY OF CHARACTERISTICS OF GOOD PERFORMANCE

PROTECTION OF REINFORCEMENT

Two of the key elements in protecting reinforcement are the use of low water-cement ratio, or relatively impermeable concrete, and sufficient cover over the reinforcing. When these two conditions exist, little corrosion was observed or reported.

A maximum water-cement ratio of .40 is recommended for precast concrete products. Research had shown that concrete made with these proportions resists the penetration of chlorides extremely well. Precast manufacturers have also proven that they can produce even more unpermeable concretes with water cement ratios and blended aggregates to create w/c ratios less than .30, thereby creating extremely durable concrete. In severe corrosion environments w/c ratios of .35 are beneficial. Prior to specifying w/c ratios less than .40, the designers should inquire with local precast concrete suppliers to determine their capabilities with local materials.

Insuring proper cover is a frequently overlooked element in the durability protection program. The most frequently observed culprit was insufficient cover over mild steel stirrups for beams, columns and wall panels.

Beam stirrups are frequently detailed or bent to produce covers less than one inch. Then if a leak in the slab occurs, corrosive water runs down the beam face or soffits and small covers (sometimes less than ½) are compromised and the mild steel starts to rust eventually cracking or spalling off the small cover and eventually creating the possibility of corroding the main prestressing steel. In the case of columns when insufficient cover over column ties is present, then the same action is created by leaking water running down the column face.

It is therefore recommended, that a minimum cover of 1 ¼” be specified and detailed for beams, columns and walls where exposure to corrosive environment exists. Care should be taken as the stirrups and ties are detailed to see that minimum covers can be achieved. In addition, stirrups and untensioned reinforcing should be carefully tied and chaired so that an absolute minimum cover of one inch is achieved considering production tolerances.

Another region where insufficient cover had proven to be critical is protection for the cut off ends of prestressing strands particularly for beams. It has been observed that deck leaks frequently occur at the ends of beams where chloride laden water can run down the ends of beams and if the ends of the cut off strands are not properly protected serious corrosion can occur at this sensitive end bearing region.
Therefore, in corrosion Zones II & III, it is recommended that strands be cut back at least 1½" from the end of the beam and the pop outs be patched with a suitable patching material and then the entire end of the beam be sacked or covered over to produce a tight reasonably smooth surface.

The use of epoxy coated reinforcement was not observed to produce significantly better performance perhaps because of difficulties in achieving the ideal epoxy coating. The reduction of performance when exposed to heat or fire is also a concern regarding the application of epoxy coating. Epoxy coating of mesh has been employed where the protected loss of bond under high heat is not significant, but it has been difficult to observe improved performance over similar projects which employed low water cement ratio concrete and good cover.
VOLUME CHANGE STRAINS AND LATERAL LOAD SYSTEMS

These two aspects of the design of precast prestressed concrete parking structures are closely linked. The lateral load system and its connections to the remainder of the structure must take into account volume change strains or damage to connections or structural components can occur.

VOLUME CHANGE STRAINS

The primary consideration for volume change strains based upon long term observations are seasonal temperature range as the primary consideration, creep a much lesser consideration and shrinkage seldom observed. Creep, and shrinkage to a much lesser degree, typically only became apparent problems when both ends of the member are rigidly fixed, so they can’t rotate, such as when the bottoms of each end of a prestressed member are welded. Observations indicate that virtually all shrinkage and except for some minimal long term creep effect is accounted for during the curing, storage period and time lag until final connections are made. Once all diaphragm connections are complete tieing the entire structure together, it is typical that three to six months has transpired since WHEN the members were initially cast.

Thus once the structure including the lateral load systems is connected together the primary concern regarding volume change strains are temperature variations, mostly notably contractions during cold weather. The concept of allowing the structure to "breathe" by employing strain relieving connections such as bolted, ductile or flex welded connections appears to perform well on structures that have been observed as well as reported ones where it was identified.

LATERAL LOAD SYSTEMS

By far and away the most popular and constantly good performing lateral system is the system employing shear walls. Where solid shear walls are objectionable because of personal safety, walls with large openings have been employed successfully. These open shear walls typically take the shape of inverted "C" or "E". Connections between the walls and the foundation are achieved by welding, mild steel reinforcing spliced with proprietary devices or sleeved, or if uplift loads are large, post-tensioning.
Connections between the diaphragm and the shear walls are typically accomplished by welding (again with one way flex or ductile connections) or with dowels connected by threaded inserts in the walls into pour strips or topping over the adjacent double tees.

Stair and elevator shafts have also been successfully engaged as part of the lateral load system where the concept of "breathable" connections have been employed.

* SUMMARY *

It should be noted that all structures surveyed or inspected were in serviceable condition even when over 25 years old and in severe corrosion exposure and subjected to poor maintenance. In our effort to identify problems we purposely sought out structures with differences therefore the extent of problems were magnified however, the typical good performance of precast parking structures of all kinds in all ranges of exposure was apparent.
There are a few actions that can be taken to improve the durability of a precast structure. Plan ahead by considering the environment and conditions of service of the structure. The quality of concrete has been determined as a major factor in durability, therefore specifying a good mix proportioning and proper testing is highly recommended. Proper detailing can fervent and enhance durability of precast prestressed structures. And finally, a maintenance program needs to be established to maintain the structure and increase its service life.

♦ PLANNING PHASE

A. Determine Structures environmental exposure
   1. Temperature and humidity
      a. Freeze thaw
      b. Potential volume change strains
   2. Chloride exposure potential
      a. Road salts
      b. Sea water
      c. Coastal Exposure
   3. Chemical exposure potential
      a. Sulfates

B. Consider design service life

C. Use appropriate loadings, and live load reductions, to limit excessive prestress, to control cambers and minimize high top surface tensile stresses.

♦ MIX DESIGN

A. Test Aggregates and water
   1. Chlorides
   2. Alkali reactivity
   3. Impurities

B. Cement Types
   1. Use appropriate cement type for service exposure

C. Mix proportions
   1. Low W/C ratios - ≤ .35
   2. Well graded sound aggregates
   3. Admixtures
      a. Air entrainment
      b. Limit calcium chlorides, thiocynats and chloride ions
      c. Use water reducers
PROPER DETAILING
A. Sufficient reinforcing cover for expected environmental exposure
B. Minimize and protect exposed connections and plates
C. Provide details which relieve volume change strains
D. Provide adequate drainage
   1. Slopes $\geq 1\frac{1}{2}$
   2. Proper drain size and location
   3. Limit warping of members
E. Provide proper joint sealants and seal all joints
F. Protect strand ends and lifting devices.

CONSTRUCTION
A. Proper curing
B. Proper vibration
C. Proper finishing techniques
D. Seal cracks or surfaces which are vulnerable to chlorides
E. Restore protective coatings damaged by welding, handling and construction
F. Consider penetrating sealer in severe environments

MAINTENANCE
A. Develop maintenance program
B. Educate owner on requirements and proper techniques