Deauville Beach Resort - Structural Condition Assessment
Anesta Consulting File Number: 21-04005
Report Issued December 15, 2021

Structural Condition Assessment

Subject:

Building: Deauville Beach Resort

Location: 6701 Collins Avenue, Miami Beach, Florida 33141

Prepared For (Client):

Mr. Jose M. Chanfrau, Esq.
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Summary of Conclusions & Recommendations

Based on our observations, experience, analysis, and review of the documents referenced herein, and within a reasonable degree of engineering certainty relative to our scope of work, this report discussed the following conclusions and recommendations:

A. Permit History:

a. The Deauville has undergone concrete repairs in 1997, 2000, 2002, 2012, 2013, and 2014, the building had been pressure cleaned, painted, and caulked in 1996 and 1998, and the flat roofs had been re-roofed in 1993, 2000, and 2008.

B. Historical Aerials:

a. At the time of our inspections, the flat roof above the southern ballroom Low Roof was approximately 9 years old, the flat roof above the Lobby Low Roof was approximately 7 years old, and the Main and Upper Roof flat roofs were approximately 5-8 years old.

b. The corrosion repairs cited within the Permit History in 2012-2014 continued through 2017.

C. Classification of Damage per FBCEB:

a. The condition of the Deauville qualified as substantial structural damage, and its ability to comply with the provisions of the FBC 2020 must be considered within our assessment.

D. General Conditions:

a. Our assessment of the general condition of the Deauville indicated that the concrete system throughout the building suffered from widespread corrosion damage as well as widespread discontinuity of load path throughout the reinforced concrete members and their connections due to construction defects and material deterioration.

E. Testing:

a. The GPR resulted in identification of the following deficiencies within the columns: Closely spaced reinforcement (appeared as “solid” readings), Widespread voids within the concrete (appeared as “fuzzy” readings), Discontinuous stirrups

b. The main result of the compression tests indicated that the strength of the concrete throughout the columns was inconsistent (nonuniform) and the results of the compression tests could not be relied upon within structural design analysis since structural theory depends on a consistent (uniform) compressive strength throughout the member.

c. The water-soluble chloride intrusion into the Ground Level columns and beams exceeded the threshold set forth by ACI 318-14 within the reinforcement layer of the tested columns.

d. The high chloride content in conjunction with the construction defects indicated that the concrete will need to be replaced in order to repair the concrete system. Extensive chloride testing through the depth of all concrete members would be required in order to reasonably replace and thus repair the concrete system of the Deauville.

F. Design Loads:

a. The Deauville handrails and its adjacent structural components would require an increase of applied load by a factor of 2.5.

b. The analysis of the Deauville for current wind speeds would generate an approximate 32.7% increase in wind pressures as compared to its original design wind speed, at a minimum.

G. Structural Integrity

a. Due to the extent of the construction defects, corrosion, and deterioration discussed within this report, the Deauville was not able to be analyzed by strength evaluation or load test as described within ACI 318-14, and as such cannot be returned to service.

b. The nature of the construction defects within the reinforced concrete system makes it infeasible to analyze and therefore repair the structure in order to withstand its original or current design load requirements.

c. The recommended 5-year cycle of corrosion repairs, the chloride ion content measured in select columns, and the magnitude of deterioration of steel and concrete observed during our inspections indicates that the building as a whole is in distress and has exceeded its service life.

H. Potential Collapse Locations

a. Due to the presence of transfer slabs and the lack of isolation joints, areas of potential localized collapse are likely to cause progressive collapse to the remainder of the adjacent continuous structure either north or south of the isolation joint

I. Remaining Service Life

a. The Deauville has exceeded its service life and cannot return to service without extensive, widespread replacement of the reinforced concrete and a complete design analysis to meet current code requirements.

J. Recommendations

a. The entirety of the interior non-structural elements of the Deauville would need to be removed, and the entirety of the structure would need to be inspected relative to the visible and hidden reinforced concrete conditions. Such an inspection, and its resultant repairs, would require a tremendous expenditure of time and costs, would be intrusive, and may cause sudden local and/or progressive collapse. The hidden nature of the construction defects, and the observed conditions during our scope of work, also presents a high risk of uncertainty during and following the repair and rehabilitation

b. It is our opinion that the only rehabilitation approach which could potentially extend the service life of the Deauville is to essentially rebuild the reinforced concrete structural system in a controlled and segmented manner. As such, we do not recommend rehabilitation or repair of the Deauville.

c. Based on our assessment as discussed herein, we recommend that the Deauville be demolished in a controlled fashion and in conjunction with additional guidance from a licensed Florida Professional Engineer with experience in the demolition and partial demolition of structures. It is our recommendation that the Deauville be demolished as soon as possible, and completed prior to the start of the 2022 Hurricane Season.

K. Conclusions

a. The Deauville has exceeded its service life and cannot return to service.

b. The Deauville cannot be repaired or rehabilitated without extensive testing and replacement of each structural element of the reinforced concrete system and the institution of a 5-year maintenance cycle. Such a repair and maintenance protocol is infeasible and not maintainable and therefore the Deauville cannot be repaired or rehabilitated.

c. The demolition of the Deauville should be completed as soon as possible and prior to the start of the 2022 Hurricane Season.

Introduction

Scope of Work

Jose M. Chanfrau, IV, P.A. (Chanfrau) retained Anesta Consulting, Inc., (Anesta) to determine if the structural condition of the Deauville Beach Resort (Deauville) was able to be returned to service in its current state, and if not, if the structure could be repaired in order to return to service. As of the issuance of this report, the Deauville has been vacant since 2017 and will remain vacant until the Conclusions and Recommendations within this report have been completed, as determined by a Licensed Florida Professional Engineer. As such, our inspection, analysis, conclusions, and recommendations focused on the overall structural condition, repairability, and general feasibility of the Deauville to return to its originally intended use. We were not retained to perform destructive testing services or to inspect or assess the condition of the structures surrounding the Deauville.

The result of destructive testing by others has been included within this report. Our office performed site inspections of the exposed structural elements of the Deauville relative to our scope of work on August 27, September 24, September 28, September 29, October 8, October 22, and November 3, 2021. All inspections were conducted by Ms. Heather Anesta, PE. Our report represents conditions observed during our scope of work. It was not within our scope of work to perform an Economic Feasibility analysis, or to prepare signed and sealed design, repair, or demolition plans.

Description of Structure

The main entrance of the Deauville faced west and was located on a plot of land between Collins Avenue and the Atlantic Ocean. The Deauville is comprised of a 3-story multi-use floor plan comprised of the lobby, utility rooms, office space, ballrooms, amenities, restaurants/dining halls, and banquet rooms. The lobby, ballrooms, banquet kitchen, and old ice rink featured two-story clear stories. The ground level was located at street level, which was below grade along the building's front entrance. The majority of the north and east walls along the ground floor did not feature windows or openings, and for that reason coupled with its "below-ground" appearance along the front entrance, the ground level is referred to in some areas as a basement. However, it should be noted that the ground level was at/near street grade level at all locations with exception to the Boiler Room, Pool Equipment Room, and Utility Rooms, located along the north and east face of buildings, respectively, which stepped down below grade approximately 2'-3'.

In general, the 2nd and 3rd levels located to the south of the main hotel were clearstory from the first elevated slab to the low roof of the 3-story portion of the structure. The office space at the northwest corner of the structure was comprised of three elevated slabs, with no apparent clearstories. The hotel portion of the structure protruded vertically from the 3-story structure for 15 stories plus an upper 1-3 story penthouse level above the main high roof. The hotel did not label a 13th floor, and as such the hotel has been described as 16 stories, although it primarily has 15 stories. For clarity purposes, we refer to each area of the structure in the following manner:

Ground Level	Entire structure's ground level, including the portion of the structure below the hotel.
Lobby Level	The 1st elevated level (2nd floor) throughout the entire structure, including the portion of the structure below the hotel, and the ballrooms/kitchen.
Low Roof	The 3-story roof portion of the structure. Note that the low roof is technically located at the fourth elevated slab of the hotel portion of the structure.
3rd Floor	The 2nd elevated level within the hotel and northwest office portions of the building and the south end of the 3-story portion of the building. Note that the areas between the hotel and south end of the building are clearstory and as such do not feature a 3rd floor between the Lobby Level and Low Roof.
Hotel Portion	4th – 16th Floors (as named by Hotel; no #13), 3rd through 14th elevated levels. Not including High Roof, Stair/Elevator Towers, or Penthouses.
Main Roof	Main Roof of the Hotel, 15th elevated slab. Not including Stair/Elevator Towers or Penthouses.
High Roofs	Roof levels above the Main Roof (There are 3 High Roof Levels, 16 – 18th elevated slabs).

Image 1: *View of the Deauville Beach Resort facing east. Image taken from Google titled, "Deauville Beach Resort Miami Florida USA" by felixtm. Photo taken sometime after 2017 based on reviews of Google Earth historical aerials.*

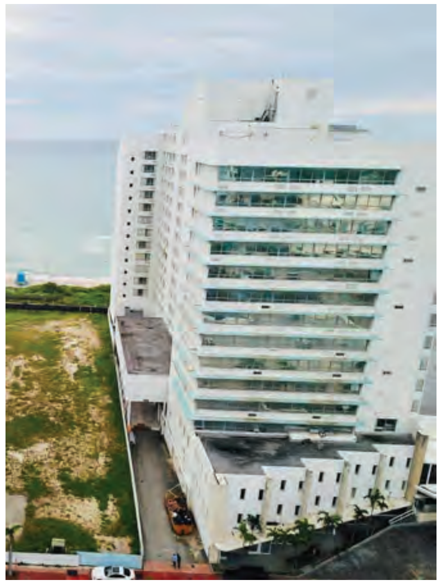


Image 2: *North End of the Deauville Beach Resort, facing East. Image taken from Bisnow Article, credited to APEX. Photo taken sometime between 2016 - 2019.*

Image 3: *Northeast corner of the Deauville Beach Resort. Image taken from quehoteles.com, Author and Date unknown.*

Image 4: *Southeast corner of the Deauville Beach Resort. Image taken from quehoteles.com, Author and Date unknown, but apparently after 1995 and before 2007 based on reviews of Google Earth historical aerials.*

Image 5: *View of the Deauville Beach Resort available from floridamemory.com, circa 1965, Identifier PR13733. Author Unknown.*

Image 6: *View of the Deauville Beach Resort available from floridamemory.com, circa 1970, Identifier WE230. Author Unknown.*

Image 7: *View of the Deauville Beach Resort available from floridamemory.com, Date unknown but apparently after 1970, Identifier PC13010. Author Unknown.*

Relevant Property & Historical Data

According to the Miami Dade County Property Appraiser Website, the Deauville was constructed in 1957 and the current Owner, Deauville Associates LLC, purchased the property in 2004. Prior sales information was not available via the Property Appraiser site.

We performed a permit search of the subject property utilizing BuildFax Property History. The search returned results between April 1, 1992, through August 1, 2021. The full Buildfax Report is available upon request. Table 1 below summarizes the permit history relative to the building structure within the Buildfax Report. We have highlighted items within Table 1 to identify permits of similar description.

Table 1: Summary of Buildfax Property History relative to the Structural Frame

Ref	Status Date	Description	Permit Status	Job Cost
1	July 31, 1992	Remove & Repair Window Tube Support	Expired	$6,000.00
2	March 9, 1993	Re-Roof Remove Down to Deck	Expired	$5,000.00
3	August 6, 1993	Rplc 1 Dr/2 Windws/1 Pctr Wndw/1 Opening	Expired	$3,500.00
4	July 12, 1996	Repair Bar Joists A/P Engineers Drawings	Expired	$20,000.00
5	July 17, 1996	Exterior Pressure Clean, Seal and Paint	Closed	$42,000.00
6	March 11, 1997	Repair T/Spalled reinforced Concrete Wall Section Building	Final	$14,000.00
7	April 9, 1998	Interior & Exterior Renovations Rooms/Common Areas	Final	$5,000,000.00
8	April 30, 1998	Pressure Clean/Caulking/Exterior Paint	Closed	$333,315.00
9	July 9, 1998	Installation of Glass Doors/Existing Opening	Final	$4,400.00
10	October 1, 1998	New Storefront Doors	Final	$12,000.00
11	March 4, 1999	Ceiling/Drywall/Laminate Walls/Demo/Comm	Final	$180,000.00
12	August 10, 1999	Partial Structural Demolition	Final	$10,500.00
13	April 18, 2000	Recovering Modified Roof to Tropical Asphalt Cements, Adhesive and Coatings	Expired	$130,000.00
14	September 27, 2000	Emergency Repair/Concrete Over Electrical Pipes	Final	$2,270.00
15	September 10, 2002	Repair 36"x3" piece of stucco on façade and paint	Final	$500.00
16	September 27, 2002	Concrete Repair in rear of building	Closed	$6,000.00
17	September 6, 2005	Interior Remodel/Partial Demo, Partitionals, Plum Fixtures, & Elec Fixtures	Closed	$151,200.00
18	January 31, 2008	Repair Flat Roof	Final	$17,000.00
19	January 16, 2009	Unit #1131, Remove damaged windows from unit and replace with impact windows (1 opening w/ 3 windows)	Final	$5,380.00
20	March 19, 2009	Interior Remodel/Partial Demo, Partitions, Plumb Fixtures & Elec Fixtures	Final	$151,200.00
21	April 19, 2012	Re-Roof flat roof 4,603 SF	Expired	$30,600.00
22	July 26, 2012	Concrete repairs, remove all loose concrete, repair all area as per details on SH. S-2, protect all pedestrians area, leave areas clean & in good conditions. All repaired existing walls & surfaces.	Void	$0.00
23	August 12, 2012	Stop Work Order Issued…Spalling Concrete appears in some areas (service area)…Structural Engineer to evaluate the structure …need to obtain proper permits and inspections	Fines	$0.00
24	October 4, 2012	Notice of Violation Issued. Water Intrusion from the roof affected Penthouses & Units 1601, 1603, 1604, 1605, 1607 and below…Need to submit an engineer report…evaluation the extent of the damages…	Dismissed	$0.00
25	December 11, 2013	Concrete repairs, remove all loose concrete, Repair all areas details on SH. S-2, protect all pedestrians area, leave areas clean and in good condition, all repaired existing walls and surfaces.	Final	$5,000.00
26	November 7, 2014	Concrete Restoration	Approved	$5,000.00
27	February 9, 2015	Alteration (w/o Phased)	Issued	$125,000.00
28	April 21, 2015	New platform and infill of louvers per plans	Applied	$20,000.00
29	March 28, 2016	Pedestrial Scaffolding Placement	Finaled (Completed April 27, 2017)	$0.00
30	March 10, 2020	Debris netting east façade of Deauville Tower	Applied	$19,500.00

Based on the available permit history relative to the building conditions, we noted that the building had been undergone concrete repairs in 1997, 2000, 2002, 2012, 2013, and 2014, that the building had been pressure cleaned, painted, and caulked in 1996 and 1998, and that the flat roofs had been re-roofed in 1993, 2000, and 2008. The permit history also indicates that the Deauville had noticeable corrosion damage in need of repair as early as 1997, 40 years after its initial construction. The corrosion repairs continued into 2002, and then occurred again approximately 10 years later in 2012. The cycle of corrosion repairs indicated that the corrosion process was inherent within the concrete system, as discussed further within this report. Note that the Buildfax Report was only able to search Property Records back to 1992. It is possible that additional corrosion repairs were performed prior to 1992.

We reviewed the historical aerials available via Google Earth and the Property Appraiser's Pictometry site. Aerials available via Google Earth dated back to 1985, however the aerials were blurry before 1995 and intermittently between 1995 – 2021. The most recent Google Earth aerial was dated June 2021. The aerials available via Pictometry dated back to 2006. The most recent Pictometry aerial was dated April 2021. The screenshots of the Google Earth and Pictometry aerials are available upon request. Table 2 summarizes our observations relative to the building structure within the Google Earth and/or Pictometry aerials.

Table 2: Summary of Observations within Historical Aerials, relative to exterior modifications.

Ref	Aerial Date 1	Aerial Date 2	Conditions Observed between Aerial Dates 1 & 2
1	Archive dated 1970	Archive dated sometime after 1970 (before 1992)	• Addition of 1-story room between the east and west stair towers on the Upper Roof level • The addition of this room is apparent between Images 5-6 compared to Images 1 and 7, within this report. • The permit history indicates that the addition occurred sometime before 1992.
2	Jan 1995	Dec 1999	• Addition of stairs from the 2nd floor (1st elevated) Lobby deck to the Pool Deck (Ground Level)
3	Dec 2005	Feb 2008	• Reroof of Upper Roofs • Reroof of east and west end of South Ballroom Low Roof • Exterior paint in progress • Removal of east Pool Deck pergolas
4	Jan 2009	Dec 2009	• Reroof of NW quadrant of South Ballroom Low Roof • Reroof of east mid-section of South Ballroom Low Roof
5	Dec 2009	Dec 2010	• Stucco repair at Upper West corner of Ballroom South Wall • Reroof in progress of entire South Ballroom Low Roof
6	Dec 2010	Jan 2012	• Reroof of entire South Ballroom Low Roof completed • Reroof of Lobby Low Roof in progress
7	Jan 2012	Mar 2013	• Reroof of Lobby Low Roof in progress • Reroof of Main Roof in progress
8	Mar 2013	Jan 2014	• Reroof of Lobby Low Roof completed
9	Jan 2014	Jan 2016	• Reroof of Upper Roofs completed
10	Feb 2015	Sept 2017	• Concrete repair along building North Face
11	Dec 2018	Dec 2020	• Building to south of Deauville demolished
12	Jan 2020	Jan 2021	• Installation of debris netting along Hotel East Face • Installation & Removal of apparent covered public boardwalk along East Hotel wall

Based on the available historical aerials, we understood that at the time of our inspections, the flat roof above the southern ballroom Low Roof was approximately 9 years old, the flat roof above the Lobby Low Roof was approximately 7 years old, and the Main and Upper Roof flat roofs were approximately 5-8 years old. We also observed that the corrosion repairs cited within the Permit History in 2012 continued through 2017. The January 2021 aerial view from Google Earth has been provided within this report as Image 8.

Image 8: *January 2021 Google Earth Aerial of the Deauville.*

Deauville Plan Layouts

Upon execution of our scope of work, we requested any and all Plans, Details, and Relevant Building Information from our Client. As a result of our request, we received a PDF file of the available Record Set of Deauville Building Plans on August 24, 2021. The PDF file contained 344 pages, which appeared to be scans of microfilm. The scans were of poor quality, with little to no legibility of text and without legible sheet names and numbers. Additionally, the microfilm appeared to have been damaged, by staining and/or heat, prior to the scans. As such, the plans could not be utilized to garner design load information or structural frame details.

Our office undertook an extensive effort to garner structural layout information from the 334 page PDF file. The result of this effort is shown in Images 10-13 within this Report. Images 10-13 represent mosaics of the best quality layouts from several different sheets, in order to portray the Ground Level, Lobby Level, General Hotel Levels, and Upper Roof Level. Note that we were not able to locate and/or decipher any plans for the foundation system or 3rd Floor level other than to confirm that the hotel was on apparent piles and that the 3rd floor level acted as a transfer slab below the Hotel Level. We utilized the plan layouts to orient ourselves during our inspections, as well as to depict the conditions observed in the field within this Report.

Within the PDF file, we were able to locate partial structural plan layouts for the 1st elevated level south of the Lobby, the 4th elevated level of the Hotel, and the 2nd elevated level (Low Roof) above the north end of the Lobby. We were also able to locate an apparent structural plan for the Hotel's two elevator cores. The quality of the apparent structural plans was poor, and we were not able to utilize the plans to garner design or construction information other than to orient the main vertical and lateral systems. In this regard, we were able to utilize the partial structural plans to further confirm our understanding of the building's structural frame systems based on our field inspections.

We compiled the most relevant plan sheets described above into one PDF set of 73 pages. This compilation of sheets has been included within Appendix F of this report. For reference between the Plan Sheets and the Lobby Level of the Deauville, we have provided Image 9. Note that the North orientation of Image 9 differs from 10-13.

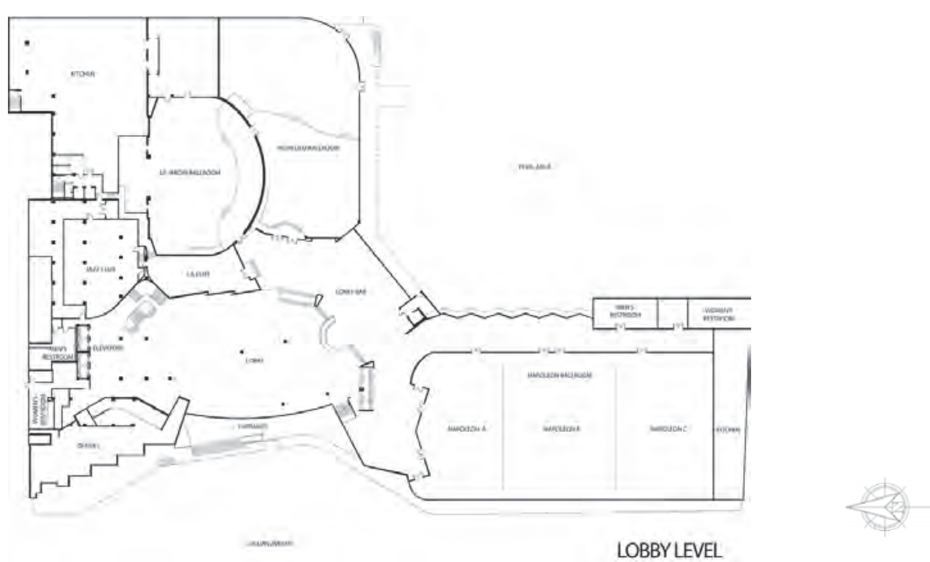


Image 9: *Lobby Level of Deauville, available from mobilemaplets.com. Author unknown.*

Image 10: *Ground Level of Deauville from Northernmost to Southernmost Walls*

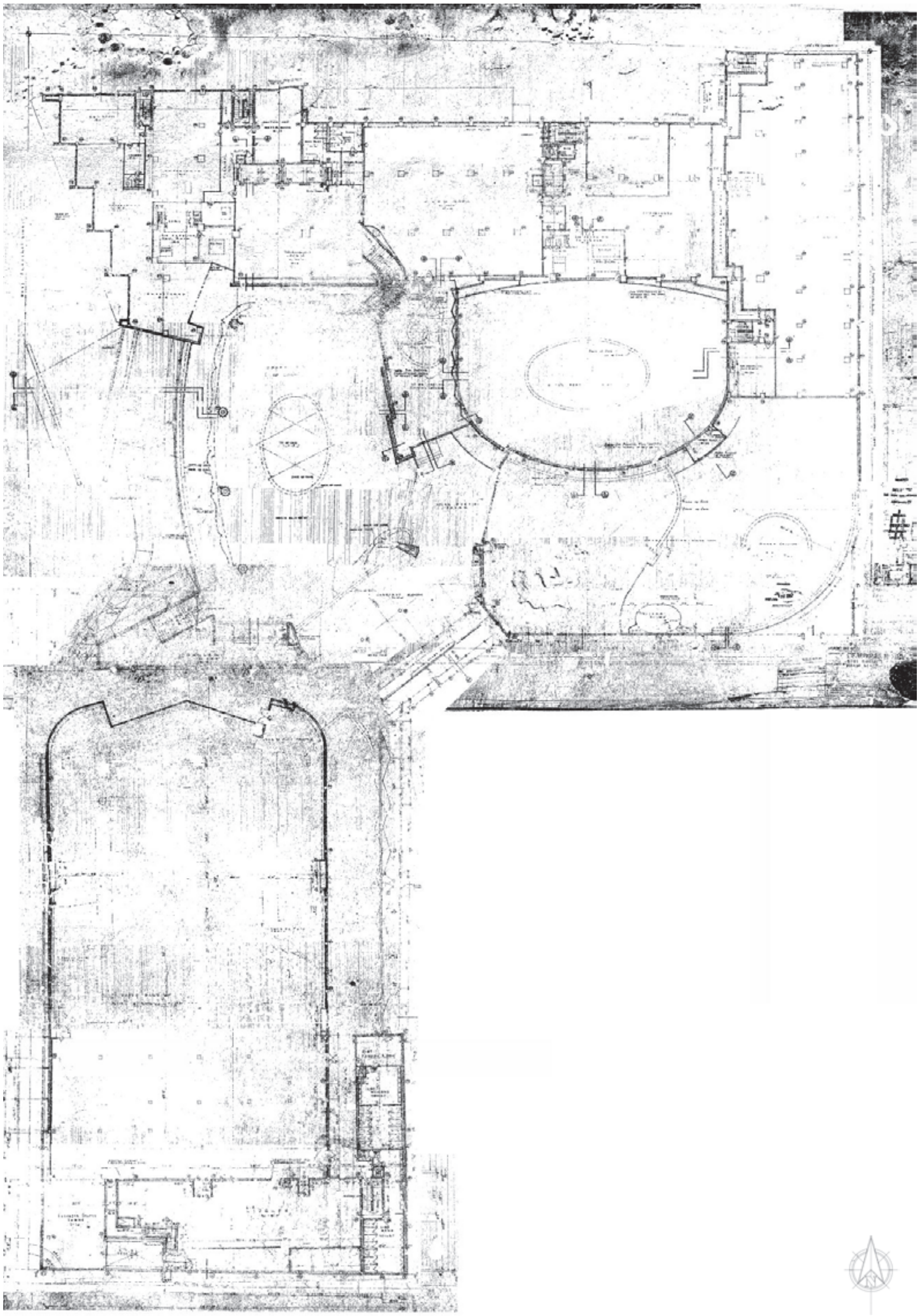


Image 11: *Lobby Level of Deauville from Northernmost to Southernmost Walls*

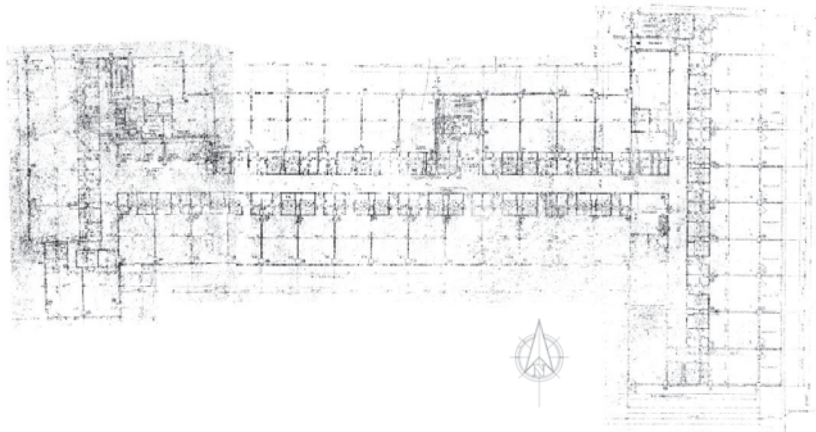


Image 12: *Hotel Levels of Deauville (above Low Roof)*

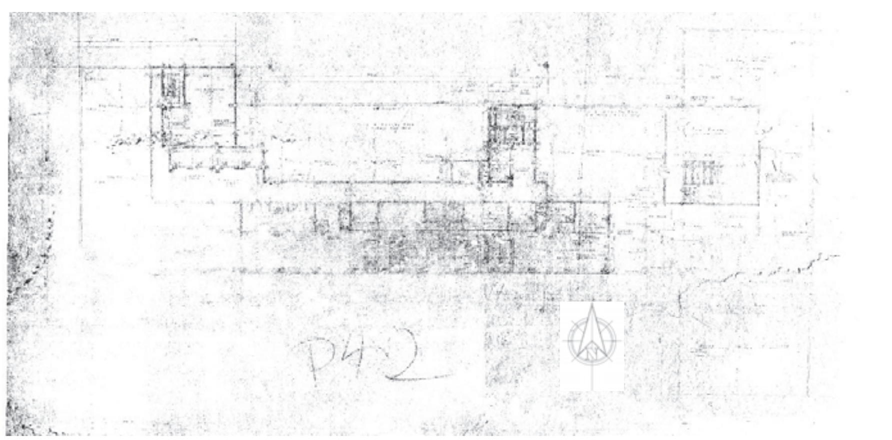


Image 13: *Upper Roof Level of Deauville (above Main Roof)*

Images 12-13 exhibit the mosaic floor layouts of the Hotel portion of the Deauville, above the Low Roof. The location of the Hotel and the general floor layout is depicted within Image 14.

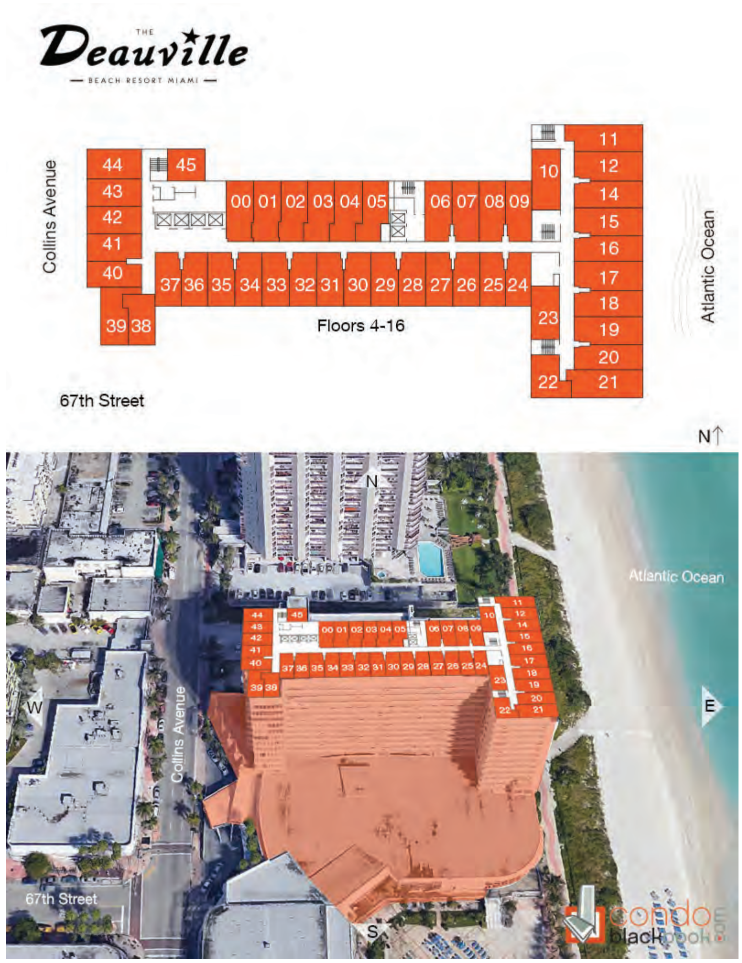


Image 14: *View of the Deauville Beach Resort Hotel and general Unit Layout available from condoblackbook.com, Date and Author Unknown. The accentuations within this image were not added by Anesta. Note that the unit numbers as shown are not accurate for each floor.*

Applicable Codes and Standards

The 1953-1954 Revisions to the 1945 Southern Standard Building Code (SSBC) were adopted in 1957 and as such were referred to during our scope of work as the most likely governing code during the design of the Deauville. The current Florida Building Code took effect on December 31, 2020, and is dated 2020, 7th Edition (FBC 2020). The SSBC and FBC reference additional Standards for additional load and material information.

The following Codes and Standards were referenced during our assessment:

- ACI/BRE/ICRI Concrete Repair Manual - 4th Edition, 2013 (CRM 2013)
- ACI 201.2R-16 Guide to Durable Concrete
- ACI 222R-19 Guide to Protection of Reinforcing Steel in Concrete Against Corrosion
- ACI 318-14 Building Code Requirements for Structural Concrete
- ACI 364.1R-19 Guide for Assessment of Concrete Structures Before Rehabilitation
- ACI 364.10T-14 Rehabilitation of Structure with Reinforcement Section Loss
- ACI 365.1R-00, Report on Service-Life Prediction
- ACI 546R-14 Guide to Concrete Repair
- ACI 562-19, Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures
- AISC Steel Construction Manual 2017 (AISC 2017)
- ASCE 7-16 Minimum Design Loads for Buildings and Other Structures with Supplement No. 1
- ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
- Florida Building Code 2020, 7th Edition, Building (FBC 2020)
- Florida Building Code 2020, 7th Edition, Existing Building (FBCEB 2020)
- 1953-54 Revisions to the Southern Standard Building Code (SSBC)

Service Life

ACI 365.1R-00 (ACI 365) is a Report on Service-Life Prediction of new and existing concrete structures and forms the basis of our Conclusions and Recommendations. The ACI 365 includes important factors controlling the service life of concrete, as well as methodologies for evaluating the condition of existing concrete structures. ACI 365 defines key physical properties and techniques for predicting the service life of concrete. The relationship between economics and the service life of structures is also discussed. We utilized the ACI 365 within our Inspection Methodology and Analysis.

The below information has been paraphrased from ACI 365 in order to most effectively convey its contents relative to our scope of work. The information paraphrased below was utilized in order to complete our assessment.

> *Service-life concepts for buildings and structures date back to when early builders found that certain materials and designs lasted longer than others. Throughout history, service-life predictions of structures, equipment, and other components were generally qualitative and empirical. The understanding of the mechanisms and kinetics of many degradation processes of concrete has formed a basis for making quantitative predictions of the service life of structures and components made of concrete. In addition to actual or potential structural collapse, many other factors can govern the service life of a concrete structure. For example, excessive operating costs can lead to a structure's replacement.*
>
> *"Durability" is the capability of maintaining the serviceability of a product, component, assembly, or construction over a specified time. Serviceability is viewed as the capacity of the above to perform the function(s) for which they are designed and constructed.*

"Service life" (of building component or material) is the period of time after installation (or in the case of concrete, placement) during which all the properties exceed the minimum acceptable values when routinely maintained.

Three types of service life have been defined:

1. *Technical service life is the time in service until a defined unacceptable state is reached, such as spalling of concrete, safety level below acceptable, or failure of elements.*
2. *Functional service life is the time in service until the structure no longer fulfills the functional requirements or becomes obsolete due to change in functional requirements, such as the needs for increased clearance, higher axle and wheel loads, or road widening.*
3. *Economic service life is the time in service until replacement of the structure (or part of it) is economically more advantageous than keeping it in service.*

To predict the service life of existing concrete structures, information is required on the present condition of concrete, rates of degradation, past and future loading, and definition of the end-of-life. Based on remaining life predictions, economic decisions can be made on whether or not a structure should be repaired, rehabilitated, or replaced.

To predict the service life of concrete structures or elements, end-of-life should be defined. For example, end-of-life can be defined as:

- *Structural safety is unacceptable due to material degradation or exceeding the design load-carrying capacity;*
- *Severe material degradation, such as corrosion of steel reinforcement initiated when diffusing chloride ions attain the threshold corrosion concentration at the reinforcement depth;*
- *Maintenance requirements exceed available resource limits;*
- *Aesthetics become unacceptable; or*
- *Functional capacity of the structure is no longer sufficient for a demand, such as a football stadium with a deficient seating capacity.*

Environmental Considerations:

Service life depends on structural design and detailing, mixture proportioning, concrete production and placement, construction methods, and maintenance. Changes in use, loading, and environment are also important. The process of chemical and physical deterioration of concrete with time or reduction in durability is generally dependent on the presence and transport of deleterious substances through concrete, and the magnitude, frequency, and effect of applied loads.The rate, extent, and effect of fluid transport are largely dependent on the concrete pore structure (size and distribution), presence of cracks, and microclimate at the concrete surface. Concrete damage due to overload is not considered in this document [ACI 364.1] but can lead to loss of durability because the resulting cracks can provide direct pathways for entry of deleterious chemicals (for example, exposure of steel reinforcement to chlorides).

Design and Structural Loading Considerations:

Many of the parameters important to service life are established by ACI 318. Minimum design loads and load combinations are prescribed by legally adopted building codes (for example, ACI 318). ACI 318 makes no specific life-span requirements. Other codes, such as Eurocode, are based on a design life of 50 years, but not all environmental exposures are considered. ACI 318 addresses serviceability through strength requirements and limitations on service load conditions. In 1963, an appendix was added to ACI 318 permitting strength design. Then in 1971, strength design was moved into the body of ACI 318, and allowable stress design was placed into the appendix. The use of strength design provided more safety and it was possibly more cost-effective to have designs with a known, uniform factor of safety against collapse, rather than designs with a uniform, known factor of safety against exceeding an allowable stress.

Interaction of structural load and environmental effects:

Actions to eliminate or minimize any adverse effects resulting from environmental factors and designing structural components to withstand the loads anticipated while in service do not necessarily provide a means to predict the service life of a structure under actual field conditions. The load-carrying capacity of a structure is directly related to the integrity of the main constituents during its service life. Therefore, a quantitative measure of the changes in the concrete integrity with time provide a means to estimate the service life of a structure. Quantifying the influence of environmental effects on the ability of the structure to resist the applied loads and to determine the rate of degradation as a result is a complex issue. The application of laboratory results to an actual structure to predict its response under a particular external influence requires engineering interpretation. As noted previously, the deleterious effects of environmentally related processes on the service life of concrete are controlled by two major factors: the presence of moisture and the transport mechanism controlling movement of moisture or aggressive agents (gas or liquid) within the concrete.

Construction-related considerations:

The ways and means of construction are the contractor's responsibility. Most often, the construction methods employed meet both the intent and the details of the plans and specifications. In some instances, however, the intent of the plans and specifications are not met, either through misunderstanding, error, neglect, or intentional misrepresentation. Service-life impairment can result during any of the four stages of construction: material procurement and qualification, initial fabrication, finishing and curing, and sequential construction.

Steel reinforcement placement tolerances are given in ACI 318. Deviations from ACI 318 can result in service-life complications such as those listed as follows (relative to non-prestressed concrete):

Condition	***Potential service-life impact***
Reinforcement out of specification	*Cracking due to inability to support design loads.*
Deficient cover	*Accelerated corrosion potential, possible bond failure, reduced fire resistance.*
Excessive cover	*Potential reduction in capacity, increased deflection, increased crack width at surface, decreased corrosion risk.*
Insufficient bar spacing	*Inability to properly place concrete, leading to reduced bond, voids, increased deflection and cracking, increased corrosion risk.*

Proper placement of concrete, including consolidation and screeding, is important to the service life of concrete structures. Lack of proper consolidation leads to such things as low strength, increased permeability, loss of bond, and loss of shear or flexural capacity. These in turn diminish service life by accelerating the response to corrosive environments, increasing deflections, or contributing to premature failures.

Performance of a structure is measured by the physical condition and functioning of component structural materials. Tests are conducted on reinforced concrete to assess performance of the structure. The questions faced in predicting service life are: establishing how much data should be accumulated, the desired accuracy of the predictions, available budgets for the predictive effort, as well as subsequent levels of inspection, maintenance, and repair.

Methods for predicting the remaining service lives of concrete structures usually involve the following general procedures: determining the condition of the concrete, identifying the cause(s) of any concrete degradation, determining the condition constituting the end-of-service life of the concrete, and making some type of time extrapolation from the present state of the concrete to the end-of- service life state to establish the remaining service life.

There could be any of a number of reasons for considering replacement of a structure, including:

- *The inability of the existing structure to continue to perform its intended duties without extensive repair or modifications;*
- *The inability of the existing structure to meet current or predicted future requirements due to changes in demand; and*
- *The appearance on the market of challengers that can perform the duties of the structure more economically.*

ACI 365.1 Section 2.2 describes the primary chemical and physical degradation processes that can adversely impact the durability of reinforced concrete structures, and can be summarized as follows:

1. Chemical Attack: Alteration of concrete through chemical reaction, which generally occurs on the exposed surface region of the concrete unless the chemicals are able to affect the cross-section through surface cracks.
2. Physical Attack: The degradation of concrete due to environmental influences such as surface wear and cracking.

Of the Chemical and Physical Attack types described within ACI 365, the Chemical Attack within Section 2.2.1.6, Steel reinforcement corrosion, was applicable to the conditions observed at the Deauville. A more in-depth discussion of concrete corrosion is contained within the Discussion Section of this Report.

ACI 562-19, Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures, defines damage as a decrease in the capacity of an existing member or structure resulting from events, such as loads and displacements, or as a result of deterioration of the structure. Deterioration is defined as (1) physical manifestation of failure of a material (for example, cracking, delamination, flaking, pitting, scaling, spalling, and staining) caused by environmental or internal autogenous influences on rock and hardened concrete as well as other materials; (2) decomposition of material during either testing or exposure to service. Design service life (of a building, component, or material) is defined as the period of time after installation or repair during which the performance satisfies the specified requirements if routinely maintained but without being subjected to an overload or extreme event. Active corrosion may create distress and deterioration beyond the limits of the repair area. The design service life should consider the existing conditions and potential distress in repairs areas and areas adjacent to the repair.

Chloride penetration can cause corrosion that can lead to cracking and spalling. The depth of a spall reduces the effective area of concrete section. Degradation of the concrete affects the concrete compressive strength. Concrete cover protects reinforcement in concrete construction from corrosion until the concrete cover becomes contaminated, cracks or is compromised. The protection provided by the concrete cover is important in determining the service life of the structure. The minimum cover is typically required by the design-basis code. The effects of concrete cover on reinforcement corrosion, chloride contamination, and carbonation should be considered when evaluating the maintenance requirements and design service life of alternative methods for corrosion protection. Concrete cover also provides fire protection. Fire protection requirements can be met by techniques such as increasing cover, spray-on fire protection or intumescent coatings.

Additionally, as summarized in ACI 562-19, Section R8.1, some examples of end-of-service life where durability parameters are not met include:

- Unacceptable reduction in structural performance
- Unacceptable frequency of maintenance cycles and associated activities

- Exceeding maximum crack width or crack frequency from corrosion, shear, torsion, flexure
- Exceeding maximum permissible chloride level at the interface of the steel in the repair area, or in adjacent areas
- Depth of carbonation leading to corrosion of reinforcement
- Unacceptable reinforcement section loss due to corrosion
- Exceeding maximum concrete deterioration level, mass loss or unacceptable surface conditions due to deterioration mechanisms, such as corrosion, freeze-thaw, chemical attack, abrasion, sulfate attack, alkali-silica reaction (ACI 221.1R, ACI 364.11T), or delayed ettringite formation
- Loss of watertightness

As a result of our understanding of ACI 365.1 and ACI 562, and with Structural and Human Safety in mind, we considered the following relevant conditions within our assessment, as they relate to the remaining service life of the Deauville:

a. Past and Future Loading
b. Concrete Condition
c. Reinforcement Condition
d. Failure of Elements
e. Chloride Ion Content at Reinforcement Depth
f. Feasibility of Repair

Florida Building Code, Existing Building 2020 (7th Edition)

The provisions of the Florida Building Code, Existing Building apply to the repair, alteration, change of occupancy, addition to and relocation of existing buildings. The intent of FBCEB is to provide flexibility to permit the use of alternative approaches to achieve compliance with minimum requirements to safeguard the public health, safety and welfare insofar as they are affected by the repair, alteration, change of occupancy, addition and relocation of existing buildings.

The following definitions from Section 202 of the FBCEB relate to our assessment of the Deauville:

1. DANGEROUS. Any building, structure or portion thereof that meets any of the conditions described below shall be deemed dangerous:
    a. The building or structure has collapsed, has partially collapsed, has moved off its foundation, or lacks the necessary support of the ground.
    b. There exists a significant risk of collapse, detachment or dislodgement of any portion, member, appurtenance or ornamentation of the building or structure under service loads.
2. EXISTING BUILDING. A building erected prior to the date of adoption of the appropriate code, or one for which a legal building permit has been issued.
3. REHABILITATION. Any work, as described by the categories of work defined herein, undertaken in an existing building.
4. REPAIR. The reconstruction or renewal of any part of an existing building for the purpose of its maintenance or to correct damage.
5. SUBSTANTIAL STRUCTURAL DAMAGE. A condition where one or both of the following apply:
    a. The vertical elements of the lateral force-resisting system have suffered damage such that the lateral load carrying capacity of any story in any horizontal direction has been reduced by more than 33 percent from its predamage condition.

   b. The capacity of any vertical component carrying gravity load, or any group of such components, that supports more than 30 percent of the total area of the structure's floor(s) and roof(s) has been reduced more than 20 percent from its predamage condition and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75 percent of that required by the Florida Building Code, Building for new buildings of similar structure, purpose and location.

6. UNSAFE. Buildings, structures or equipment that are unsanitary, or that are deficient due to inadequate means of egress facilities, inadequate light and ventilation, or that constitute a fire hazard, or in which the structure or individual structural members meet the definition of "Dangerous," or that are otherwise dangerous to human life or the public welfare, or that involve illegal or improper occupancy or inadequate maintenance shall be deemed unsafe. A vacant structure that is not secured against entry shall be deemed unsafe.

FBCEB Section 406.2.2 states that a building that has sustained substantial structural damage to the vertical elements of its lateral force-resisting system shall be evaluated in accordance with Section 406.2.2.1, and either repaired in accordance with Section 406.2.2.2 or repaired and rehabilitated in accordance with Section 406.2.2.3, depending on the results of the evaluation. Section 406.2.2.1 states that the building shall be evaluated by a registered design professional, and the evaluation findings shall be submitted to the code official. The evaluation shall establish whether the damaged building, if repaired to its pre-damage state, would comply with the provisions of the Florida Building Code, Building for load combinations that include wind or earthquake effects, except that the seismic forces shall be the reduced level seismic forces.

As discussed herein, the result of our inspections determined that the capacity of a group of vertical component carrying gravity load, that supports more than 30 percent of the total area of the structure's floor(s) and roof(s) has been reduced more than 20 percent from its pre-damage condition and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75 percent of that required by the Florida Building Code, Building for new buildings of similar structure, purpose and location. As such, we determined that the condition of the Deauville qualified as substantial structural damage, and its ability to comply with the provisions of the FBC 2020 must be considered within our assessment.

Structural Collapse Mechanisms

Tension and/or Compression forces can cause rapid collapse of a concrete frame system during lateral or vertical load conditions. When tension is concentrated at the edge of a concrete frame or shear wall during lateral loads, it can produce very rapid loss of stability of the building. When the reinforcing steel within the frame columns or walls is inadequately proportioned or poorly embedded, the building can fail in tension, resulting in rapid collapse of the wall or frame by overturning.

Additionally, loss of strength, rigidity, or continuity within the joints in a concrete moment frame, whether by deterioration or poor construction, can cause a rapid degradation of the structure during lateral load conditions, which can result in partial or complete pancaking during beam/column failure. Local column failure can occur during vertical or lateral loading due to column instability from poor construction, horizontal offset, or insufficient capacity. Local column failure can lead to loss of stability and/or progressive collapse within a structure. In either case, the failure can occur suddenly while experiencing vertical or lateral loads. In this regard, our assessment included the evaluation of the Deauville's structure wholistically, and with consideration of frame (column & beam) condition, joint condition, transfer slab locations, and isolation joint locations. Such considerations at the Deauville are discussed further herein.

Corrosion Repair

Based on our understanding of ACI 546R-14 Guide to Concrete Repair, ACI 364.10T-14 Rehabilitation of Structure with Reinforcement Section Loss, and ACI 562-19, Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures, we understand that the most frequent cause of damage to reinforcing steel is corrosion. A licensed design professional should be consulted for any corrosion repairs. Other possible causes of damage are construction defect, fire, chemical attack, and accidental cutting. This section of our report paraphrases the above-mentioned standards as well as our understanding and experience with concrete corrosion repair, relative to our scope of work, and was considered within our assessment.

The quality of concrete repairs is largely dependent upon the workmanship during construction. Inspection is necessary to verify repairs and rehabilitation work are completed in accordance with construction documents. Typical repair construction is different from new construction in scope, and new construction testing requirements may not be sufficient for repair construction. Construction documents should specify inspection requirements for concrete repair and rehabilitation construction during the various work stages. The licensed design professional should recommend that the Owner retain a licensed design professional, a qualified inspector, a qualified individual, or some combination thereof for the necessary inspections

After deteriorated and damaged concrete is removed, it is necessary to expose the reinforcing steel, evaluate its condition, and prepare the reinforcement for repair if required. Proper inspection and preparation of the reinforcement helps to assure satisfactory long-term performance of the repair solution. If additional or replacement reinforcement is required, a new reinforcing bar may be lap spliced to the existing bar(s). Lap length is determined in accordance with ACI 318. Additional concrete removal may be necessary to properly splice the new steel reinforcing bar. Mechanical or welded splices that follow code provisions could also be used.

Removal of deteriorated concrete and reinforcement often uncovers unanticipated conditions that should be examined. Visual inspection and verification of existing conditions may require review of project specific conditions before continuing the construction process and thus require pauses in the construction processes so as not to conceal components of the work before completing necessary inspections and verifications. If unanticipated conditions are identified by the repair inspector, the licensed design professional should be informed. The licensed design professional should examine these conditions and determine what measures are to be implemented before placement of new repair materials. The construction documents should specify the locations where inspection is necessary before concealment and provide for possible changes in these locations due to unforeseen conditions. In some projects, all locations will not need to be inspected and representative locations will provide suitable inspection.

Situations exist where corroding reinforcement that cannot be adequately cleaned or repaired will remain in the repaired structure. The effects of uncleaned reinforcement on the long-term durability of the repaired structure should be considered in these situations. Supplemental corrosion mitigation strategies may be needed in these situations. The corrosion of embedded metals adjacent to the repair may be accelerated due to differing electrical potential between electrically continuous reinforcement in the repair area and external to the repair area. This form of corrosion is commonly referred to as the “anodic ring” or “halo effect”. The rate of anodizing corrosion depends upon the chloride content, internal relative humidity, and temperature. The anodic ring effect, which may be induced by certain repairs, can be addressed by incorporating appropriate corrosion mitigation strategies such as cathodic protection or corrosion inhibitors.

It is not uncommon that a concrete repair involves replacing only deteriorated concrete at spalls or delaminations. This approach often leaves chloride-contaminated concrete surrounding the repair area, creating a highly conducive environment for continued corrosion of the reinforcement. Such repairs may actually promote corrosion of the reinforcing steel in the surrounding concrete and contribute to the anodic ring or halo effect. Such effects are exhibited by a cycle of necessary corrosion repairs, typically alongside of previous repair areas.

A properly prepared substrate is achieved by removing existing deteriorated, damaged, or contaminated concrete. The exposed sound concrete is then roughened and cleaned to allow for adequate bond of a repair material. In addition to replacing the unsound concrete and deteriorated reinforcement, the forces acting on the interface between cementitious repair materials and existing substrate can include tension, shear, or a combination of tension and shear depending on repair geometry and the applied loads. The tensile and shear demand at an interface between a cementitious repair material and the substrate from applied loads and from volume changes that occur as a result of shrinkage or thermal movement can be calculated using principles of structural mechanics, but these calculations can be complex. Where the required nominal interface shear stress is lower than 80 psi, and where good surface preparation, placement, repair materials, and curing techniques are employed, satisfactory composite behavior will likely be achieved without interface reinforcement.

Inspections

Assessment Methodology

Our assessment methodology was based on our experience, knowledge, and guidance from ACI 318-14 Building Code Requirements for Structural Concrete, ACI 364.1 R-19 Guide for Assessment of Concrete Structures Before Rehabilitation, ACI 562-19 Code Requirements for Assessment Repair and Rehabilitation of Existing Concrete Structures, and ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings. We utilized our engineering judgement throughout the inspection and assessment process of our scope of work, with consideration of structural stability as well as integrity, ethics, and human safety. Our assessment was of a qualitative nature.

Our assessment was based on the following objectives:

1. Identify the remaining service life of the structure by means of visual observation of the overall structural condition of the exterior walls, ground and lobby level structure, roof level structure, and main lateral and vertical systems. Utilize destructive testing and/or calculations as necessary to further validate our observations.
2. Identify the continuous structural systems within the Deauville (location of isolation joints).
3. Identify mechanisms for potential progressive collapse of the Deauville as a result of isolated, local, failure.
4. Assess the condition of structural elements throughout the Deauville and identify areas of reduced strength.
5. Identify locations of reduction of strength relative to construction defects, design defects, material deterioration.
6. Identify the feasibility to repair or rehabilitate the structure

We approached our assessment in a progressive manner in order to meet our objectives and preserve the overall integrity of the structure. In this regard, we requested that the Owner remove interior non-structural gypsum board and drop-ceilings throughout the Ground, Lobby, and Roof levels, and we prescribed destructive tests (performed by Others) during the latter part of our assessment as discussed in the Testing section of this report.

Observations

During our inspections, we observed the following conditions.

Isolation Joints

The design of a structure is based on the individual member strength as well as the strength of the wholistic structural system. The extent of each structural system is identified by the exterior perimeter of the structure and interior boundaries by means of isolation joints. There are many reasons to include an isolation joint for design and construction purposes. In essence, an isolation joint creates a break in the continuous structure, thereby creating independent structures which form the aesthetic of one continuous structure. The location of an isolation joint must be established by the structural engineer and is an integral part of the structural design of a building. In regard to our scope of work, the main purpose of the identifying the location of isolation joints is to understand the original design intent of the structural systems, as well as to identify the potential collapse mechanisms of the structures.

We identified one isolation joint within the Deauville during our scope of work, located as shown in Image 15. There were no isolation joints present between the Hotel portion and Lobby/east ballroom portions of the Deauville. The structure below the Low Roof was continuous below the 4th elevated slab of the Hotel portion of the Deauville, as shown in Image 16.

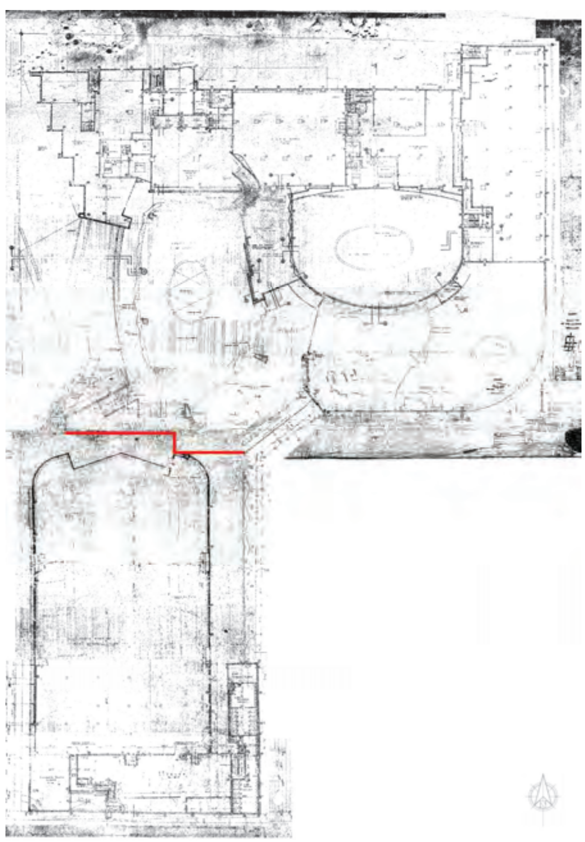


Image 15: *Location of Isolation Joint below Low Roof Level (red line). Note that there was no Isolation Joint along the Hotel interface with the Lobby Level.*

Transfer Slabs

A transfer slab is a structural system which transfers vertical and/or lateral load from one location to another by means of the slab system (slab and/or beams), rather than directly from the column above to the column below. The presence of transfer slabs is an important consideration in the condition assessment of a structure because a local failure within a transfer slab will, by nature, cause progressive failure of the structure alongside and above the local failure area.

The most substantial transfer slab we identified within the Deauville was located at the 4th Elevated Level below Hotel Portion of Structure, and was continuous with Low Roof Structure (Image 16 and 17). Essentially, the columns along the hallways in the Hotel portion of the structure were transferred by cantilever beams within the transfer slab to the column locations in the Ground, Lobby, and 3rd Floor levels of the structure below (Image 18). The transferred columns in this area were part of both the vertical and lateral systems of the Deauville.

We also identified additional local transfer slab areas, such as the southeast corner of the radius ballroom at the Lobby Level, above the Pool Room, and isolated transfer beams between the Lobby and Ground levels, between the Hotel and Isolation joint.

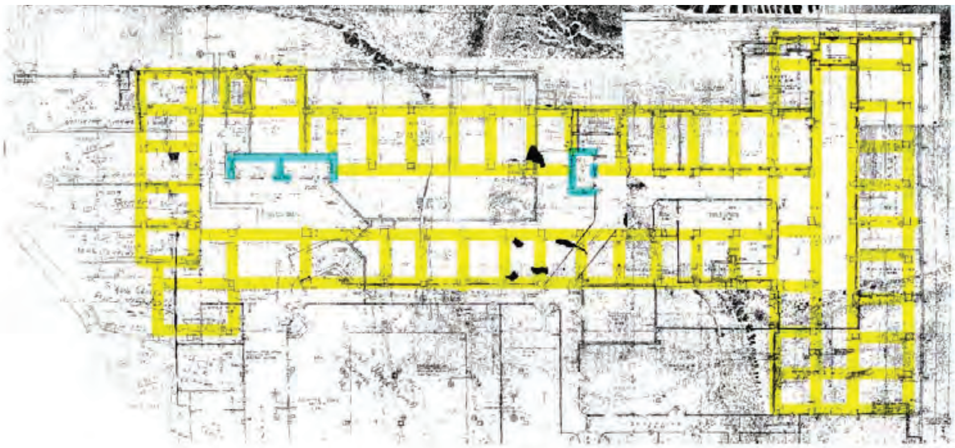
Apparent Lateral Systems

The lateral system of the Deauville appeared to primarily consist of reinforced concrete frames located along its perimeter walls. We observed few, if any, shear walls while on site or within the record set of plans. There was one apparent shear wall which ran north-south within the center of the main elevator shaft (nearest the lobby). The remainder of the system whether within the Hotel portion or throughout the Lobby and Ballroom areas were frame systems along the perimeter walls. The frames within the Hotel portion featured smaller cross-sections and larger on-center spacings on the main roof level than on the ground level, as depicted in Image 18. For example, the columns on the ground level ranged from 28”-48” square with frames along every bay, while the main roof level columns ranged from 14”-20” square with frames along every other bay.

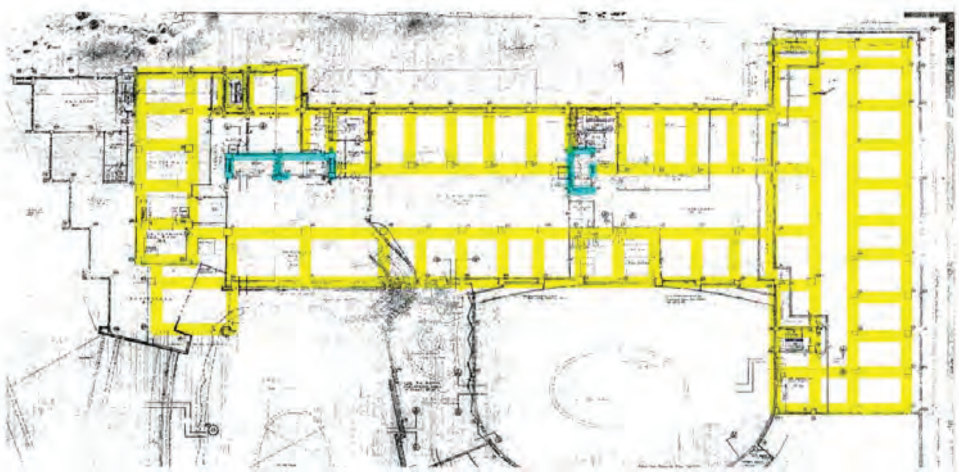
Such a frame system is referred to as a rigid frame or moment-resistant frame. Rigid frame systems resist lateral load and consist of beams and columns with rigid connections in order to keep the frame from deflecting into a parallelogram under applied lateral loads. In particular, buildings over 60’ in height, such as the Hotel Portion of the Deauville, will generate significant tension and compression forces in the columns and high moments within their rigid frames. High tensions and moments can be detrimental, since severe cracking can result in catastrophic failures when tension or bending forces are induced within the member. Construction defects and material deterioration can have catastrophic impacts on the strength of a ridge frame. Additionally, lateral systems rely on the continuity and strength of diaphragms at each floor/roof level to transmit the forces from the exterior walls into the frames.

Image 16: *Graphic Depiction of Continuous Structure with Hotel Portion (blue shaded area) based on Isolation Joint Location.*

Image 17: *Location of the Transfer Slab at 4th Elevated Level below Hotel Portion of Structure, Continuous with Low Roof Structure.*



Ground Level



Lobby Level

Hotel Levels

Image 18: *Apparent Lateral System Layout of Deauville within Hotel Portion of Structure*

General Conditions

Upon initial walk-through of the Deauville Lobby and Ground Levels on August 27, 2021, the majority of the columns, walls, and ceilings (slabs and beams) were clad with gypsum board, architectural features, drop-ceiling, and/or non-structural elements which obstructed our view of the their condition. During the initial walk-through, there were no obvious signs of distress of the structure with exception to the Ground Level within and surrounding the Boiler Room, at the north end of the Hotel Portion of the structure. In this location, the columns, slabs, and beams were exposed and we observed severe cracking and corrosion at the base of the columns; the corrosion appeared to be an isolated condition to this location. Following the initial walk-through, we requested that the Owner remove the non-structural cladding from the Ground and Lobby Levels nearest the elevator shafts and within the office space. We noted that several office and common areas contained furniture and stored items, which further obstructed our view of the structure's condition. We requested that the items positioned alongside columns and walls throughout the Ground and Lobby Levels be removed or relocated. We understood following our initial walk-through that there appeared to be limited areas of reinforced concrete corrosion.

During our site visits on September 24, 28, 29, and October 8, 2021, based on our experience with corrosion damage and observations during the initial walk-through, we anticipated to locate additional corrosion primarily along the perimeter of the structure, and to locate sound structural elements within the interior of the structure. Our expectation was to identify and quantify the amount of corrosion in order to determine the most suitable repair method. In accordance with our assessment approach, we also inspected interior areas for purposes of establishing the baseline condition of the structure and to gain an understanding of the structural layout of the Deauville.

Upon completion of our September 24 – October 8, 2021, inspections, we identified widespread severe corrosion throughout the Ground Level columns, both along the perimeter and within the interior of the structure. The corrosion damage did not feature an apparent pattern in relation to exposure to the exterior environment, and in most cases, severely corroded columns were located along the interior of the structure and adjacent to columns with no apparent corrosion. Further, the interior and perimeter conditions of the Deauville featured widespread honeycomb visible along the exposed faces and corners of columns, beams, joists, and slabs. We also identified several areas of horizontal and vertical joints within columns, and accumulations of cement paste which created a plaster-like consistency of the concrete.

During our site inspections on October 22 and November 3, 2021, we further confirmed the widespread nature and degree of reinforced concrete corrosion, deterioration, and construction defects upon inspection of the 3rd floor, 15th Floor, 16th Floor, and Penthouse levels of the Hotel Portion of the building. We accompanied Wingerter Laboratories, Inc., and ScanTekGPR, LLC, as they performed GPR and Windsor Probe tests on November 3, 2021, as discussed further within the Testing Section of this report. The GPR and Windsor Probe efforts exposed that the "control" columns, selected for testing based on their apparent "good" condition, featured closely spaced reinforcement, missing or discontinuous stirrups, and/or extensive concrete voids below the surface. We also identified numerous areas of deteriorated concrete and prior corrosion repairs in the form of patched concrete without proper bond to the original concrete surface, which had otherwise not been apparent by visual inspection alone.

During the testing efforts, discussed further in the Testing Section of this report, it became apparent that the honeycomb was not limited to the face or edges of the structural elements, and were also located within the structural elements. The internal honeycomb created concealed voids within columns and were not identifiable through visul or audible inspection. The widespread honeycomb appeared to have been caused by offset rebar cages, closely spaced rebar, and inadequate mixing/vibration of the concrete. These concealed conditions indicated widespread strength reduction caused by construction defects which originated from poor concrete and reinforcement placement.

The concealed and widespread nature of the defects prevented us from performing sound structural design calculations since we could not assume consistency of parameters throughout the length and cross-sections columns and beams. The observed concrete deterioration as well as the high chloride content of the concrete system indicated that the concrete could not be relied upon to be sound without extensive testing of each element. Even with testing of each element, the inconsistency of each element's construction would cause different parameters along the length of the members. As a result, our assessment of the general condition of the Deauville indicated that the concrete system throughout the building suffered from widespread severe corrosion damage as well as widespread discontinuity of load path throughout the reinforced concrete members and their connections.

The typical conditions noted throughout our inspection are discussed further herein. We have included the above general summary of conditions in order to summarize the progression of our assessment resultant from our methodology and objectives. While we began our assessment with a general expectation to address isolated corrosion damage and repair approaches, the actual result of our inspections produced multiple types and degrees of damage throughout the structure which caused further adjustment to our testing and analysis as described herein.

Typical Conditions Noted During Inspection

During our inspections, we observed the following typical conditions. Photographs representative of the below noted conditions have been included within this section of our report. Additional observations are included within the Prior Repairs portion of this report. See Appendix C for approximate strength reduction of corroded rebar.

1. Reinforcement Condition
    a. Different bar types within the same group of bars
    b. Main columns utilized wire ties rather than #3 rebar stirrups
    c. Confinement steel was not adequately provided
    d. Closely spaced steel, either placed directly next to or within 1" of one another
    e. Offset rebar cages with less than 1" clear cover and more than 3" clear cover across the same section.
2. Concrete Condition
    a. Concrete deterioration by hand
    b. Concrete deterioration by light hammer strike
    c. Cracked concrete along interface between hotel and lobby portion of the building (no isolation joint)
    d. Cracked slab system along hotel perimeter walls observed above ground level, lobby level, 3rd floor, and roof/penthouse levels.
    e. Cracked slab (diaphragm) system adjacent to the concrete frame system at upper floor levels.
    f. Deflection of floor slab at roof level (within Penthouse) at west end of Hotel
3. Structural Steel
    a. Honeycomb along embed into reinforced concrete structure
    b. Minor to Moderate corrosion of steel
4. Corrosion
    a. Corrosion of concrete columns, beams, slabs, and joists were located throughout the building
    b. Typical reinforcement cross-sectional loss approximately 17-46% within corrosion area, and in

some cases 50-100% cross-sectional loss.

c. Stirrup deterioration 100%
d. Unfinished and open corrosion repair areas were present along the hotel's exterior wall system.

5. Construction Defects (Original Construction)
    a. Inadequate mixture of aggregate and paste during concrete placement
    b. Closely spaced reinforcement
    c. Honeycombed concrete
    d. Insufficient lap splice of reinforcement
    e. Offset reinforcement (inadequate clear cover)
    f. Vertical construction joints within columns
    g. Sonotube column paper was left on the columns within the beam-column joints
6. Construction/Design Implications
    a. Congested joints between columns and beams within Hotel, Lobby, and Ballrooms

Representative Photographs

Photographs representative of the typical conditions noted above have been included within this section of our report. Additional photographs from our inspection are available upon request.

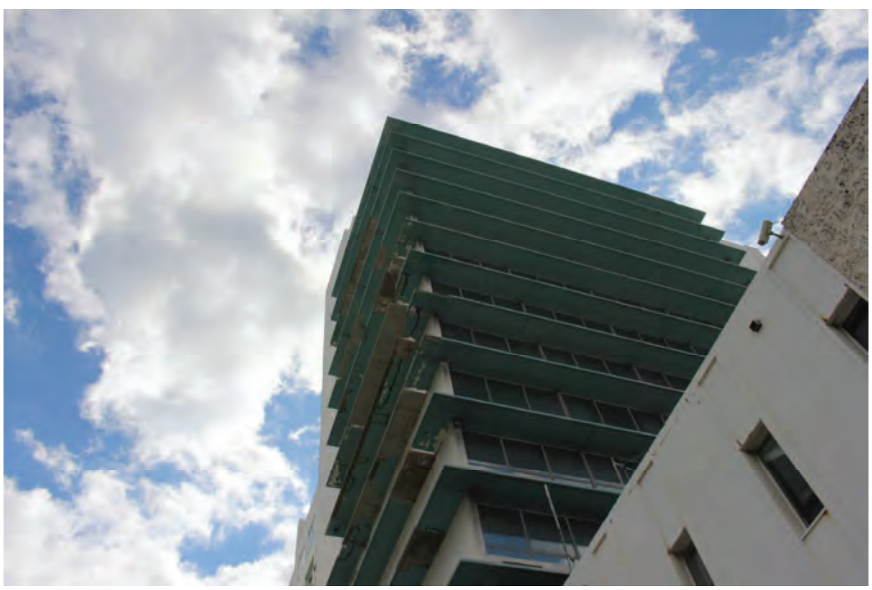


Image 19: View of the exterior condition of the northwest corner of the hotel. Note the unfinished corrosion repairs to the north eyebrows.

Image 20: View of the exterior condition of the west wall of the Hotel.

Image 21: View of the apparent beam corrosion along the Hotel windows on the north end of the building.

Image 22: View of the apparent beam corrosion and prior repairs along the Hotel windows on the north end of the building.

Image 23: View of the apparent beam corrosion and prior repairs on north face of hotel.

Image 24: Condition of the north end of the east wing of the Hotel. Note that the north wall of the Hotel's east wing was cantilevered off of the main structural frame at the Lobby Level.

Image 25: Unfinished and open repair areas of structural columns along the Hotel's north face.


Image 26: Unfinished and open repair area for structural column at east end of the Hotel's north face.


Image 27: View of the northeast corner of the Hotel.

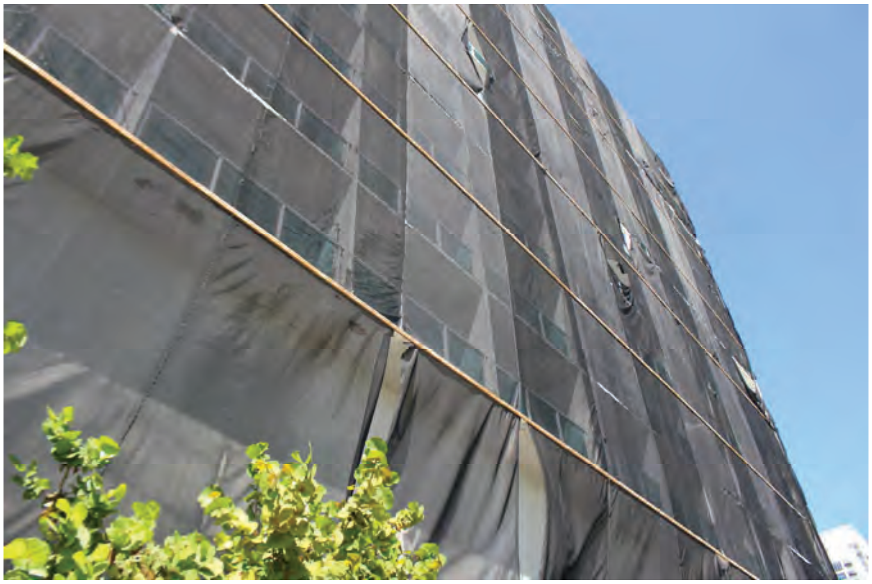


Image 28: Typical Hotel east face balcony and column corrosion.

Image 29: Typical Hotel east face balcony and column corrosion.

Image 30: View of the Hotel, Lobby, and East Ballroom portion of the buildings, taken from the south end of the exterior of the property, facing north.

Image 31: View of the position of the Lobby Level east ballroom above the Ground Level pool equipment rooms.

Image 32: View of the condition of a corroded and cracked beam (shear and flexural cracks), located below the exterior radius wall of the east ballroom. (Beam Label 6)

Image 33: View of the condition of the support wall below the east ballroom.

Image 34: View of the loss of steel cross-section within support wall below east ballroom.

Image 35: View of the condition of the east support wall below east ballroom, from within the pool equipment room.

Image 36: View of Lobby area, taken from front entry facing northeast toward the Hotel Portion of building.

Image 37: View of Lobby area, taken from front entry facing east toward the east Ballroom at Lobby Level of building.

Image 38: View of Lobby area, taken from front entry facing southeast toward the pool.

Image 39: View of the east ballroom, facing east.

Image 40: View of the stage located along the south face of the hotel portion of the building, facing northwest.

Image 41: View of South Ballroom and Staircases, taken from Lobby facing southwest.

Image 42: View of isolation joint between Lobby and South Ballroom.

Image 43: View of isolation joint between Lobby and South Ballroom.

Image 44: View of the isolation joint between the north and south portions of the structure, shown in Image 15.

Image 45: View of the South Ballroom, facing southwest.

Image 46: Typical condition of concrete frame system at roof level of hotel. Note the size of the beams and columns.

Image 47: Typical propagation of cracks off concrete frame system into adjacent slab (diaphragm) system. Photo shown below main roof level.

Image 48: Deflection of floor slab at roof level (within Penthouse) at west end of Hotel.

Image 49: Typical steel truss and concrete plank system above the South Ballroom.

Image 50: Typical steel truss and concrete plank roof system above the Lobby and Ballrooms (excluding south ballroom).

Image 51: Typical sonotube column within Lobby. Note that the column's sonotube paper was encasing the column.

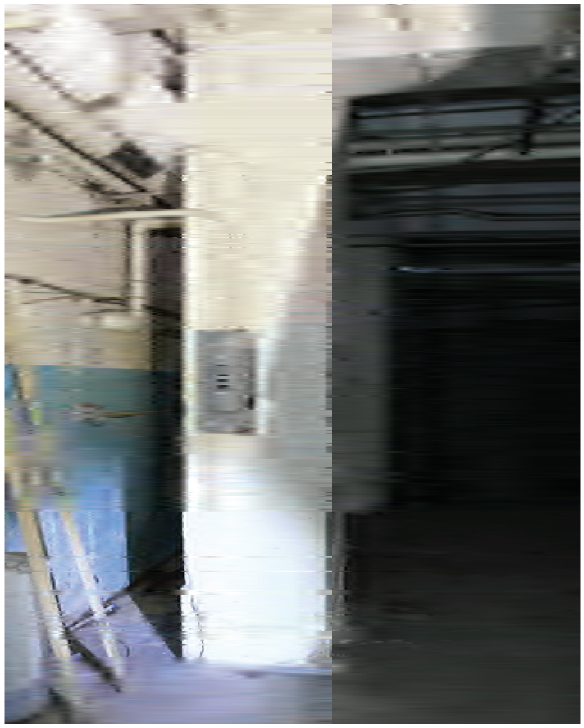


Image 52: Typical condition of Ground Level column, below Hotel, within the interior of the building. Note corrosion as well as honeycombed concrete.

Image 53: Typical condition of slab system from Ground Level, below Hotel. Note corrosion as well as honeycombed concrete.

Image 54: Typical condition of slab system from Ground Level, below Hotel. Note corrosion as well as honeycombed concrete.

Image 55: Typical condition of corroded slab rebar apparent above Ground, Lobby, and Upper Levels.

Image 56: Typical condition of corroded plank slab system above the steel truss framed roofs above the Lobby and Ballrooms

Image 57: Typical condition of corroded plank slab concealed by fireproofing/insulation.

Image 58: Typical condition of 3rd Floor Level (transfer slab). Note spalled, cracked, and deteriorated concrete.

Image 59: Typical condition of Ground Level column within and along Boiler Room, below Hotel.

Image 60: Typical condition of joist corrosion.

Image 61: Beam corrosion along southeast end of Lobby, above curtain wall. Note that this beam supported the adjacent Lobby roof structure.

Image 62: Close up view of concrete beam corrosion in previous image.

Image 63: Typical concealed corrosion damage with visible distress within Lobby along perimeter walls.

Image 64: Typical concealed corrosion damage with visible distress within Lobby along perimeter walls.

Image 65: Typical crack condition along top of South Ballroom floor slab, along the supporting Ground Level office walls, which was an indication of slab deflection between supports.

Image 66: *Typical condition of honeycombed concrete, apparent during visual inspection.*

Image 67: Typical condition of column reinforcement with less than 1” of clear cover on Ground Level below South Ballroom, within the interior of the building.

Image 68: Corrosion of interior Ground Level columns, located at south end of building.

Image 69: Corrosion of interior Ground Level columns, located at south end of building.

Image 70: Corrosion of perimeter Ground Level columns, located along west face of south end of building.

Image 71: Typical exterior corrosion damage along south portion of building.

Image 72: Use of caliper to measure thickness of flaked reinforcement steel due to corrosion. This particular photograph exemplifies a typical condition of flakes ranging between 3/16 – 5/16" from #11 bars in Frame Columns.

Image 73: Typical condition of Ground Level Hotel frame column with previous patch repairs.

Image 74: Typical condition of Ground Level Hotel frame column closely spaced reinforcement as well as inadequate concrete mix. Note the loose aggregate and little to no cement paste. (Column Label 10)

Image 75: Typical condition of Ground Level Hotel frame column with substantial honeycomb. Note corrosion cracks.

Image 76: Condition of Ground Level Hotel frame column which featured flaked paint. While sounding the column with a hammer, we encountered powder-like concrete at an apparent horizontal joint. (Column Label 12)

Image 77: Condition of Ground Level Hotel frame column which featured flaked paint and powder-like concrete. While sounding the column with a hammer, a concrete spall detached and revealed severe corrosion, closely spaced rebar, and offset rebar cage (inadequate clear cover). (Column Label 12)

Image 78: Condition of the east end of the main lobby elevator shaft, including a Ground Level hotel frame column. Note the corrosion. (Column label 14)

Image 79: Typical condition of the joints within the main lobby elevator shaft's rigid frame system. Note the honeycomb.

Image 80: Typical condition of congested beam-column frame joint. Note honeycombed concrete and poor horizontal joint within column.

Image 81: Typical condition of congested beam-column Ground Level Hotel frame joint. Note honeycombed concrete. Note the size of the columns and beams (beam spans left to right above column in image)

Image 82: Typical condition of congested Lobby Level Hotel frame beam-column joint. Note honeycombed concrete. Note the size of the columns and beams (beam spans upper left to bottom right above column in image)

Image 83: Typical condition of sonotube paper within the beam-column joint. Note honeycombed concrete.

Image 84: Typical condition of offset rebar within column cross-section, affecting clear cover and depth of reinforcement. Note advanced corrosion particular to this column, Label 1.

Image 85: Typical condition of sporadic and widespread nature of honeycombed concrete. Note exposed reinforcement.

Image 86: Typical honeycomb at steel truss connection to reinforced concrete structural frame. Primarily located above Lobby and Ballrooms. Note corrosion of exposed reinforcement.

Image 87: Typical horizontal joint within frame column in conjunction with poor mix and concrete quality. Note that the lower portion of the column was a powder-like consistency, able to be removed in large pieces by hand, and the upper portion of the column was a grainy consistency which was able to be scraped away by hand. (Label 3)

Image 88: Typical vertical joint in column.

Image 89: Typical condition of slab system cracks which propagated through the joists near hotel perimeter walls in upper floors.

Image 90: Structural crack which transmits through walls, beams, and slabs along the interface between the hotel portion and lobby portion of the building, viewed within the Ground Level. Note that there was no isolation joint between the hotel and lobby portions of the building.

Image 91: Close up view of structural crack in previous image. Note the continuation of the crack through several elements. Note the honeycomb in the slab.

Image 92: Typical condition of main roof and parapets.

Image 93: Typical condition of low roof and parapets.

Image 94: Typical condition of low roof and parapets.

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Evaluation of Observed Conditions

As discussed within this report, our overall assessment of the Deauville represents our observations of individual elements as well as our analysis of the building as a whole. In order to most effectively convey our evaluation of observed conditions, we created Image 95 and utilized the following Condition Rating System:

A. Good
  - Concrete: Inspector was unable to visually observe any surface cracks, discoloration, abrasions, spalls, or deterioration.
  - Steel: Inspector was unable to visually observe any surface corrosion, discoloration, or deterioration.

B. Fair
  - Concrete: Inspector was unable to visually observe any surface cracks greater than 0.2 mm or spalls deeper than 0.25 inches, and was not able to visually locate any discoloration, abrasions, or deterioration or exposed steel.
  - Steel: Inspector was able to visually observe a small amount of surface corrosion and/or discoloration, and was not able to observe any deterioration.

C. Poor
  - Concrete: Inspector was unable to visually observe any surface cracks greater than 0.5 mm or spalls deeper than 0.75 inches, and was not able to visually locate any discoloration, abrasions, or deterioration that appeared to weaken the structural members. Exposed steel may be present, but it does not exhibit any flaking or signs of loss of cross-sectional area. Honeycombed concrete of depth less than 1", with no exposed reinforcement was present along face of member.
  - Steel: Inspector was able to visually observe a small amount of surface corrosion and/or discoloration, and was not able to visually locate any discoloration, abrasions, or deterioration that appeared to weaken the structural members. The surface corrosion and/or discoloration does not exhibit any flaking or signs of loss of cross-sectional area.

D. Severe
  - Concrete: Inspector was able to visually observe surface cracks greater than 0.5 mm and/or spalls deeper than 0.75 inches, and was able to visually locate discoloration, abrasions, and/or deterioration that appeared to weaken the structural members. Exposed steel may be present, and it exhibits flaking and/or signs of loss of cross-sectional area greater than 20%. Honeycombed concrete present along face and within member, with depths greater than 1" and/or exposed steel.
  - Steel: Inspector was able to visually observe corrosion and flaking which appeared to weaken the structural members, including loss of cross-sectional area.

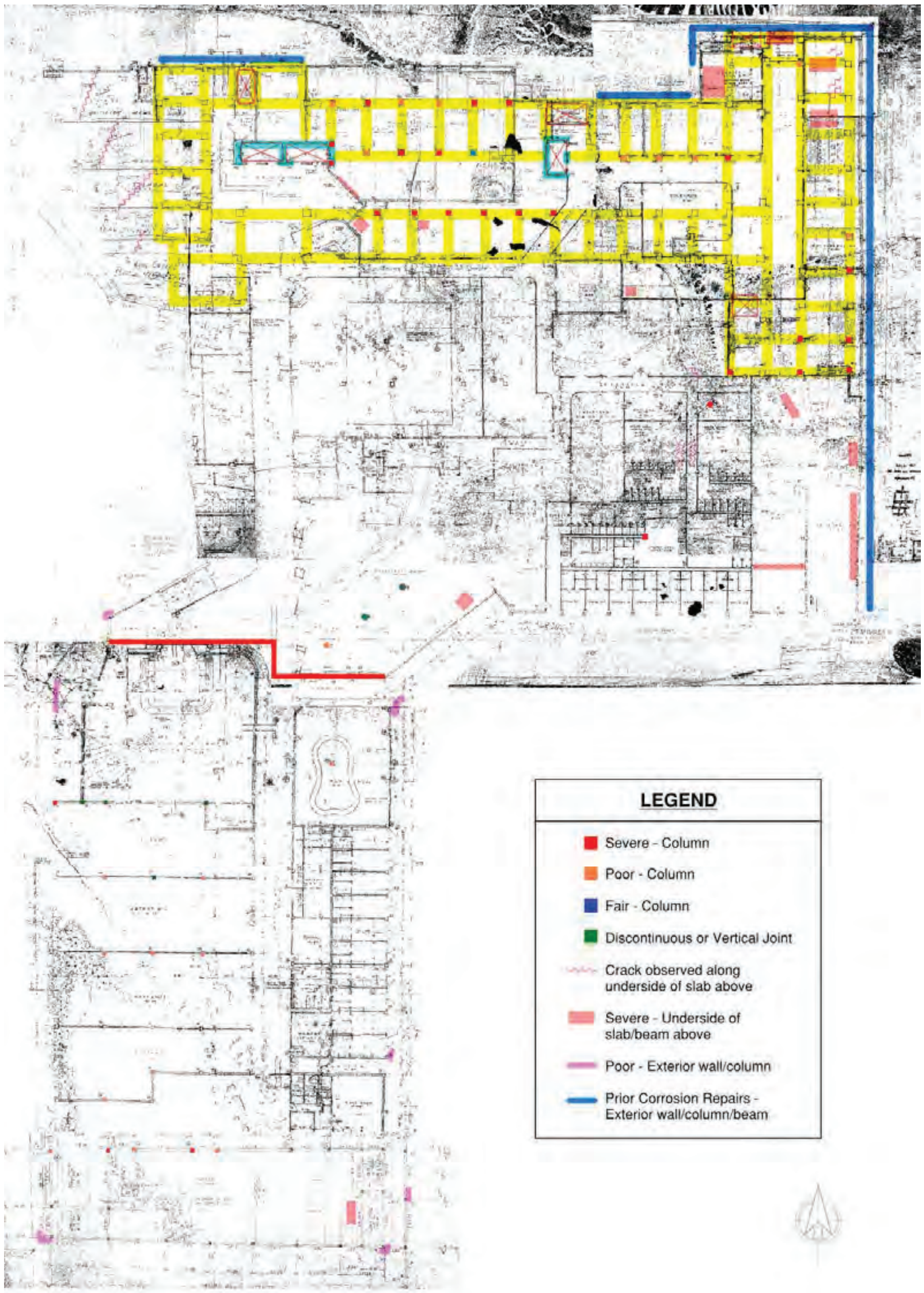

***Image 95:** Depiction of the Conditions Observed during our Inspections of the Ground Level Columns, 1st Elevated Level and Exterior Walls/Columns/Beams*

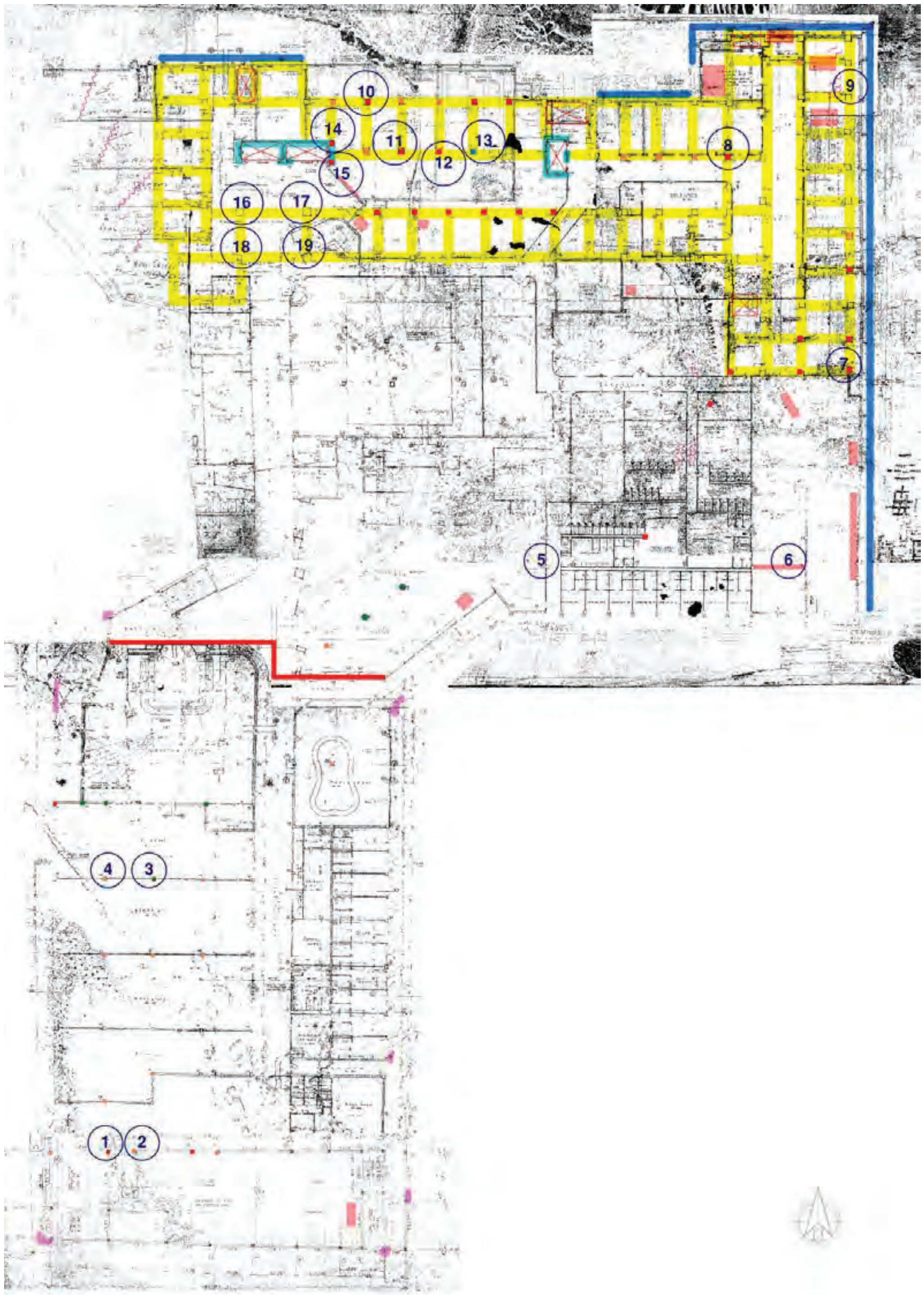

Image 96: *Test Location Labels – Ground Level*

Testing

Test Methodology

In order to obtain a general understanding of the concrete compressive strength and the chloride content of the reinforced concrete system, we prescribed the tests described herein. Per ACI 318-14, the number of tests required depends on the uniformity of the material within the structure and should be determined by the licensed design professional responsible for the evaluation. It should be noted that our Client forwarded a request for us to obtain core samples for compression and chloride measurements from each unique structural system's foundations, walls, floors, columns, beams, and roofs. However, due to the conditions we observed on site, it is our professional opinion that conducting such tests would cause additional damage to the Deauville which could cause sudden local and/or progressive failure of the structure. As such, we approached the Testing in a phased manner, and initially prescribed only those tests which would provide us with the information we required in order to complete our scope of work.

We accompanied Wingerter Laboratories, Inc., and ScanTekGPR, LLC, as they performed GPR and Windsor Probe tests on November 3, 2021, as discussed further within the Inspections Section of this report. The GPR and Windsor Probe efforts further indicated that the "control" columns, selected for testing based on their apparent "good" condition, featured closely spaced reinforcement, missing or discontinuous stirrups, and/or extensive concrete voids below the surface. Additional details of the Testing results are discussed herein.

The test locations and their associated labels are shown within Image 96. We identified 14 columns and 1 beam within our first round of prescribed compression and chloride tests. We identified 8 additional columns for GPR testing only, below the Hotel near the main elevator lobby on the Ground and Lobby Levels. The conditions observed and test results resultant from the first round of testing were sufficient for us to complete our assessment. As such, and to prevent unnecessary damage to and/or weakening of the building, we did not prescribe additional tests.

GPR Testing & Results

GPR was performed by ScanTekGPR on November 3, 2021, in order to locate the reinforcement within the columns. If a column did not present reinforcement spaced more than 3" on center, we did not core drill the column in order to preserve its integrity. The GPR of the Ground Level Hotel columns indicated that the majority of the steel was located along the north and south faces of the column, which further verified that the main lateral system was comprised of rigid frames, and that the frame columns were design and required to rests tension and compression forces in the north-south and east-west directions.

The GPR resulted in identification of the following deficiencies within the columns:

- Closely spaced reinforcement (appeared as "solid" readings)
- Widespread voids within the concrete (appeared as "fuzzy" readings)
- Discontinuous stirrups

Concrete Compressive Strength Testing & Results

In order to test the columns for their respective compressive strength, we prescribed compressive tests utilizing core drilling, a Windsor Probe, and a Schmidt Hammer in accordance with ASTM C-42-84a, ASTM C803 & C670, and ASTM C805, respectively. In an effort to calibrate the results of the tests, we prescribed multiple locations on select columns, and performed all three test methods on select columns. ScanTekGPR and Wingerter Laboratories performed the concrete compressive strength tests. The results of the compressive strength tests are included within Appendix D, and are summarized within this section of our report.

Based on our review of the results of the compressive strength tests, and within a reasonable degree of engineering certainty, we determined the following:

- Adequately mixed reinforced concrete within the columns tested presented a compressive strength from core tests between 3,000-4,000 psi
- The Windsor Probe returned values well above the Core samples, which indicated that the results were not accurate enough to use within design calculations. The Schmidt Hammer returned results higher than the Windsor Probe, and as such its results were also not able to be utilized within design calculations.
- While not effective for determining compressive strength, the Windsor Probe was effective in identifying the extent of voids and/or accumulations of paste within the concrete columns which were not apparent upon visual inspection. In such cases, the probes would blow out of the concrete (see Images 99 – 101).
- The columns with multiple test locations featured varied compressive strength results, which indicated that the measured or average compressive strengths could not be extrapolated in order to perform design calculations.
- The columns with “blow out” results indicated that the concrete was not placed adequately, and the compressive strength measured by other means could not be utilized in a design calculation.
- Overall, the main result of the compression tests indicated that the strength of the concrete throughout the columns was inconsistent and the results of the compression tests could not be relied upon within structural design analysis since structural theory depends on a consistent compressive strength throughout the member.
- Column Label 7 is of particular concern, as it is the southeast frame column of the Hotel portion of the building, and was not able to be core tested due to the column presenting as “plaster” during core-drill operations. We have included Images 97 and 98 of Column 7, the core location, and the condition of the frame beams and slab system above.

	Compressive Strength (psi)		
Ref Number	Core	Windsor	Schmidt
1		9300	
2		Blow Out	
3A		5100	8000
3B		7550	8500
3C		Blow Out	6800
4	3950	7200	
5	4073	5800	
6			7300
7	3630		
8A	4300	7200	
8B		Blow Out	
8C		Blow Out	
9	Untested ("Plaster")		
10A		Blow Out	6800
10B			
11	4290	Blow Out	7300
12		Blow Out	7400
13	4170	8075	6800
14		Blow Out	8500
15		Blow Out	8000
16			6800
17			not tested (water)
18			7400
19			7000

Image 97: *Condition of Column Label 7, facing south. See Image 96 for location of Column 7 in plan view.*

Image 98: Condition of the frame beams and slab system above Column Label 7, facing southeast. See Image 96 for location of Column 7 in plan view.

Image 99: Typical condition of the 3 probes installed for the Windsor Probe Test. (Column Label 5)

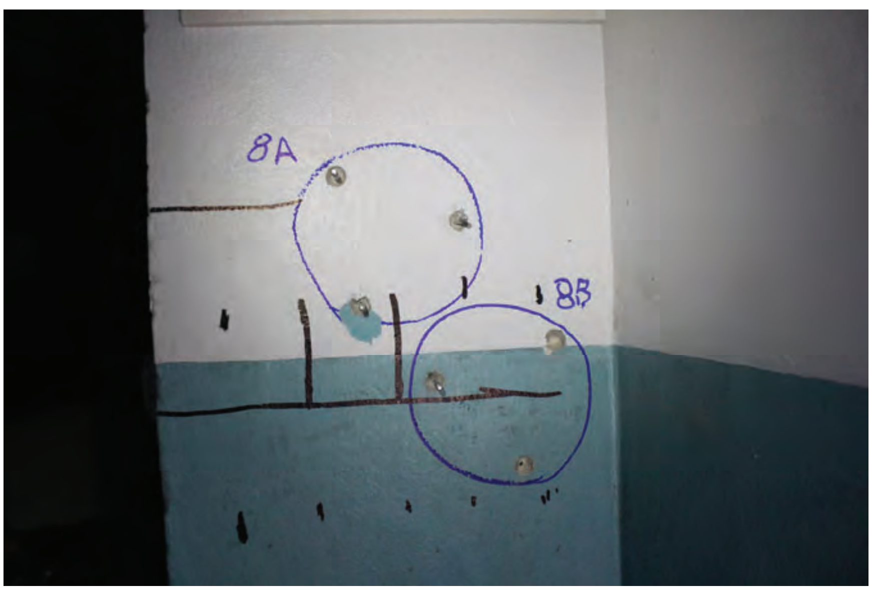

Image 100: Condition of the completed Windsor Probe installation for location 8A, and the “blow out” condition of attempt to test at location 8B.

Image 101: Typical condition of a “blow out” location of a Windsor Probe which exposed interior voids within the concrete columns.

Water-Soluble Chloride Ion Content Testing & Results

In order to test the columns for their respective chloride content, we prescribed water-soluble chloride tests in accordance with ASTM C1218 within the reinforcement layer of the members and within the column, at the inner end of the core sample. We also tested one beam (Label 6) since that particular beam exhibited noticeable corrosion, flexural/shear cracks, and supported the east ballroom radius wall. ScanTekGPR and Wingerter Laboratories performed the concrete compressive strength tests. The results of the water-soluble chloride tests are included within Appendix E, and are summarized within this section of our report.

Based on our review of the results of the water-soluble chloride tests, and within a reasonable degree of engineering certainty, we determined the following:

- The water-soluble chloride intrusion into the Ground Level columns and beams exceeded the threshold set forth by ACI 318-14 within the reinforcement layer of the tested columns, at a minimum.
- The high chloride content in conjunction with the construction defects indicated that the concrete will need to be replaced in order to repair the concrete system.
- The high chloride content within perimeter and interior columns and beams in conjunction with the prior corrosion repairs and maintenance cycle indicated that the concrete deterioration is a widespread condition throughout the concrete system.
- Columns which were tested in multiple locations produced varied chloride content per test location.
- Extensive chloride testing through the depth of all concrete members would be required in order to reasonably replace and thus repair the concrete system of the Deauville.

	Chloride Content				
Ref Number	WS Chloride Content (ppm)	WS Chloride in Cement (%)	Depth of Sample	Exceed 0.15?	Exceed 0.30?
1	996	0.47	0"-3"	Y	Y
2	606	0.29	0"-3"	Y	N
3A	153	0.073	0"-3"	N	N
3B	364	0.17	0"-3"	Y	N
3C	130	0.062	0"-3"	N	N
4	34	0.023	7"-9"	N	N
5	57	0.038	3"-5"	N	N
6	2936	1.4	0"-3"	Y	Y
7	51	0.034	5"-6"	N	N
8A	40	0.027	6"-8"	N	N
8B					
8C					
9					
10A	1137	0.54	0"-3"	Y	Y
10B	3160	1.5	0"-3"	Y	Y
11	38	0.025	5"-6"	N	N
12	386	0.56	0"-3"	Y	Y
13	104	0.069	5"-7"	N	N
14	2472	1.18	0"-3"	Y	Y
15	1169	0.56	0"-3"	Y	Y

Discussion

Design Loads

The design of any structure requires that there be a continuous load path from the origin of the applied load, through the structural members and connections, through to the foundation, and into the soil or bedrock. In instances where a continuous load path cannot be achieved, failure will occur at the weak link along the load path, whether by inadequate design, overstress, or discontinuity of elements. When evaluating the health of a structure, it is imperative that the structure have viable load paths for vertical and lateral loads in order to remain in service. Deterioration or discontinuity of elements and connections can contribute to structural failure during service and design loads due to its disruption of the load path through the structural system.

The design loads of structures are specified by the Building Code and its Referenced Standards. We referenced the SSBC for the design loads utilized during the time period of construction of the Deauville for the purposes of this report. SSBC Chapter 12 specified the minimum design loads as shown in Table 3. SSBC Chapter 16 specified the concrete material, mix, and design parameters as shown in Table 4. SSBC Chapter 15 specified the steel materials and allowable stress parameters as shown in Table 5. SSBC Chapter 13 specified the foundation and soil parameters as shown in Table 6.

In the event that the Deauville would undergo repairs, the extent of the repairs would require that the building be analyzed for current code requirements. FBC 2020 Chapter 16 specifies the minimum design loads as shown in Table 3 (further referenced within ASCE 7-16). FBC 2020 Chapter 19 specifies the concrete material, mix, and design parameters as shown in Table 4 (further referenced within ACI 318-14). FBC 2020 Chapter 22 specifies the steel materials and allowable stress parameters as shown in Table 5 (further referenced within AISC-2017). FBC 2020 Chapter 18 specified the foundation and soil parameters as shown in Table 6. We noted that based solely on code required load and strength conditions, that the Deauville handrails and its adjacent structural components would require an increase by a factor of 2.5.

All structures, including their components and cladding, are required by Code to be designed to withstand a Basic Wind Speed. The Basic Wind Speed is generally based on geographic location and building use, and is listed within the Florida Building Code and its referenced Standard, ASCE 7. Wind design of buildings includes safety factors and adjustments which provide a design strength that exceeds the Basic Wind Speed. It is our understanding that the design Basic Wind Speed for the Miami Dade County area was approximately 110-120 mph (equivalent fastest-mile at 33 ft above ground for Exposure Category C, 50-Year Mean Recurrence Interval) within the Southern Standard and Standard Building Codes, and generally prior to the adoption of the 2001 Florida Building Code. The 2001 Florida Building Code was adopted in or near 2002, at which time, the design Basic Wind Speed for the Miami Dade County area was updated to 146 mph (nominal 3-second gust at 33 ft above ground for Exposure Category C based on 50 to 100-Year Peak Gusts). The current Florida Building Code (2020, 7th Edition) utilizes Strength (Ultimate) Level wind speeds, and lists a design Basic Wind Speed for a Risk Category II Building in Miami Dade County as 175 mph (3-second gust at 33 ft above ground for Exposure Category C, 7% probability of exceedance in 50 years). The 175 mph basic wind speed within FBC 2020 is an Ultimate wind speed. Prior to its conversion to Ultimate Wind Speeds, the FBC required a Based Wind Speed of 146 mph at Service Level, which is a more direct comparison with the wind speeds from SSBC. As such, we noted that the analysis of the Deauville for current wind speeds would generate an approximate 32.7% increase in wind pressures as compared to its original design wind speed, at a minimum.

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Table 3: Design Load Comparison		
Design Load Type	**SSBC 45 with 53-54 Rev**	**FBC 2020 (7th Edition)**
Assembly Places – Movable Seats Live Load	100 psf	100 psf
Corridors, Public, Live Load	100 psf	100 psf
Dance Halls, Live Load	120 psf	100 psf
Hotel Guest Rooms, Live Load	40 psf	40 psf
Offices, Live Load	50 psf	50 psf
Roof (Flat), Live Load	20 psf	20 psf
Railings, Special Load	20 plf horizontal at top of railing	50 plf horizontal at top of railing
100 Year Recurrence of Fastest Mile Wind Speed (Service Level)	110 mph	146 mph (see narrative)

Table 4: Concrete Parameter Comparison		
Concrete Parameter	**SSBC 45 with 53-54 Rev**	**FBC 2020 (7th Edition)**
Footings (1302.4)	2000 psi at 28 days	2,500 psi at 28 days
Minimum Compressive Strength	Not specified	2,500 psi (C1 Exposure Class) 5,000 psi (C2 Exposure Class)
Steel Reinforcement Designation	A-15-50T (Commonly 40,000-50,000 psi Yield Stress)	ASTM 615 Grade 60 (60,000 psi Yield Stress)
Minimum Rebar Spacing	Min(1”,1.33*aggregate, rebar diameter)	Horizontal: Min(1”, 1.33*aggregate, rebar diameter) Vertical: Min(1.5”, 1.33*aggregate, 1.5*rebar diameter)
Minimum Concrete Cover – Exposed to Weather or in contact with Ground	#6 or greater: 2” #5 or less: 1.5”	#6 or greater: 2” #5 or less: 1.5”
Minimum Concrete Cover – Not exposed to Weather or in contact with Ground	Not Specified	Primary reinforcement in beams and columns: 1.5”

Table 5: Steel Parameter Comparison		
Steel Parameter	**SSBC 45 with 53-54 Rev**	**FBC 2020 (7th Edition)**
Structural Steel Designation	A7-50-T	ASTM A36
Allowable Unit Stress – Tension	20,000 psi	0.6Fy = 21,600 psi

Table 6: Foundation Parameter Comparison		
Foundation Parameter	**SSBC 45 with 53-54 Rev**	**FBC 2020 (7th Edition)**
Presumptive Bearing Capacity of Wet Sand	4,000 psf	Not specified
Presumptive Bearing Capacity of Sand and clay	4,000 psf	1,500 psf
Presumptive Bearing Capacity of Fine and dry sand	4,000 psf	2,000 psf
Presumptive Bearing Capacity of Coarse sand	4,000 psf	3,000 psf
Max Allowable load on cast-in-place concrete piles without steel shells	25 tons	Not specified

Reinforced Concrete

Reinforced Concrete is a heterogeneous material comprised of concrete (high compressive strength) and steel reinforcement (high tensile strength). Together, the reinforced concrete accommodates the compressive and tensile loads as a single element such as a slab, beam, or column. Concrete is comprised of a mixture of cement, water, fine aggregate, coarse aggregate, air, and often other admixtures. The concrete mixture must be mixed evenly in order to provide a homogenous section, and is then cured to facilitate the acceleration of the chemical hydration reaction of the cement-water mix, resulting in hardened concrete. The tensile strength of concrete is approximately one-tenth of its compressive strength. Consequently, tensile and shear reinforcement within the tensile regions of concrete sections must be provided in order to resist such forces. When the various ingredients of reinforced concrete are properly proportioned, the finished product becomes strong, durable, and adaptable for use as a structural system.

Cast-In-Place Reinforced Concrete is constructed on site with use of formwork, and as such, the overall quality and strength of the reinforced concrete system is highly dependent on the quality of construction. For example, the design theory of reinforced concrete requires that at a minimum, the concrete be well-mixed without voids and that the reinforcement be placed at the appropriate locations and distances from eachother and the element faces in order to provide adequate strength, serviceability, and durability. All reinforced concrete members are designed for specific cross-sectional areas of concrete, reinforcement steel, and reinforcement placement. When one or all of these factors are affected either by poor construction or deterioration of materials, the strength of the concrete system will decrease. In this regard, this report places high importance on the concrete quality, reinforcement cross-sectional area, and reinforcement placement.

The distance between the exterior of the outermost reinforcement and the face of the concrete element is called the “clear cover” or “concrete cover”. ACI 318 specifies the minimum amount of concrete cover based on durability requirements, as discussed herein. When the provided concrete cover is less than the minimum specified by code, the concrete and reinforcement is vulnerable to deterioration and chemical attack as discussed herein. Furthermore, when the reinforcement is not placed as specified by the structural design, the strength of the concrete system is adversely affected. In summary, the strength of a reinforced concrete element requires that the concrete element be designed and constructed as specified by and within the tolerances of the applicable codes and standards. In instances of widespread construction issues such as discontinuous concrete placement or lack of adequate bond between the concrete and reinforcing steel, the only remedy to restore the design strength of the element is removal and replacement of the steel and/or concrete.

ACI 318-14 provides minimum spacing limits of reinforcement in order to permit concrete to flow readily into spaces between bars and between bars and forms without honeycombs, and to ensure against concentration of bars on a line that may cause shear or shrinkage cracking. The size limitations on aggregates are provided to facilitate placement of concrete around the reinforcement without honeycombing due to blockage by closely-spaced reinforcement. Per ACI 201.2R-16, good workmanship is vital for securing uniform concrete with low penetrability. For lowslump concrete, segregation and honeycombing can be avoided by good consolidation. Meeting the requirements of the specifications pertaining to durability are essential. Adequate spacing should be provided to allow for proper placing of the concrete cover so that honeycombing and poor compaction are avoided and good bond between concrete and steel are obtained.

The structural consequences of a 10% or less cross-section area loss are usually minor for nonprestressed concrete components because of redundancies in design. This 10% threshold is due to the nature of design vs available reinforcement sizes. Steel reinforcement used in construction is typically larger than required by structural considerations, and often times extra steel is attributed to varying practical design requirements such as bar layout and spacing. 5 to 10% more steel area is typically provided than is required by analysis. In an extreme case, a building may have a 20% surplus of reinforcement. In this regard, and in correlation with loss of steel area due to corrosion, we provided values of representative approximate strength reduction of reinforcement due to corrosion in Appendix C. It is our understanding that a 20% or greater loss of cross-sectional area will occur in reinforcement of #5 size and smaller if it sheds a thickness of 1/32" of its outer area. Accordingly, to remain near a 20% loss of cross-sectional area, #6 bars and greater cannot shed more than 1/16" of its outer area.

The continuous load path through a concrete member requires that the stresses be able to flow through the concrete and steel as intended by the reinforced concrete design. Lack of bond between the steel and concrete and/or voids within the concrete cross-section, called honeycombs, create discontinuities within concrete members which adversely affect the strength of the system and can contribute to failure during service and/or design load conditions. Additionally, due to the depth of the neutral axis, the tolerances of reinforcement placement must be upheld in order for members to perform at their design strength. The below table outlines our understanding of reinforced concrete conditions which can decrease the strength or effectiveness of a concrete system.

Condition	Affect on Structural System	Method of Locating Condition
Honeycomb	Void spaces reduce the concrete cross-sectional area for the entirety of the void area, thus decreasing the compressive, flexural, and shear strength at the location of the void, which effects the overall strength of the member. Honeycombs are typically located along the corners of formwork, and in such condition can be located and patched. However, when improper mixing and/or steel placement prevents the formation of a homogenous mixture throughout the section, they can be more prevalent within the member and may not be visible. When honeycombs are identified, they must be assessed by a Licensed Florida Professional Engineer. This condition can cause failure of concrete members below service and design load conditions.	Typically, honeycombs are located and eliminated by means of progressive inspection and repair during construction. The identification of widespread honeycomb conditions along the edges and within concrete members following completion of construction is indicative of widespread poor construction and inspection practices. To remedy a condition of widespread and hidden honeycombs, the entirety of the concrete structure must be inspected visually, audibly, and with penetrating radar in order to conclusively address and remedy the condition.
Poor mixture of concrete (cement, aggregate, etc)	The design compressive strength requires adequate mixture of the concrete. When the aggregate and paste are not distributed homogenously, the collection of paste and/or aggregate leads to reduced compressive strength and/or excessive cracking. This condition can cause failure of concrete members below service and design load conditions.	Typically, improper mixture of concrete is located and eliminated by means of progressive inspection and repair during construction. The identification of widespread improper mixture conditions along the edges and within concrete members following completion of construction is indicative of widespread poor construction and inspection practices. To remedy a condition of widespread and hidden improper mixture, the entirety of the concrete structure must be inspected visually, audibly, and with penetrating radar in order to conclusively address and remedy the condition.

Condition	Affect on Structural System	Method of Locating Condition
Lack of bond between reinforcement and concrete	In instances where the reinforcement does not properly bond with the concrete, the stresses within the reinforced concrete section cannot flow through the concrete and steel reinforcement as designed. This condition prevents the tensile force from reaching the reinforcement, and causes failure of the concrete member. It should be noted that concrete patch repairs also require proper bond strength between the new and existing concrete mixture.	This condition typically develops due to foreign substances on the surface of the reinforcement, honeycombs, and/or poor mixture of the concrete, and can be identified during progressive inspection during construction. This condition can also develop following the completion of construction, due to corrosion and cross-sectional loss of the reinforcement, as the outer layers of steel flake away from the inner section of the rebar. The latter condition is most commonly identified due to excessive cracking or spalling of the reinforced concrete. To remedy widespread and hidden debonded conditions, the affected areas must be inspected visually, audibly, and with penetrating radar in order to conclusively address and remedy the condition. Such conditions caused by construction issues are likely widespread throughout the structure and are not easily identifable. Conditions caused by corrosion are typically located along the base, roof, and perimeter of structures.
Loss of reinforcement cross-sectional area	The loss of cross-sectional area of reinforcement has a direct correlation with loss of strength of the reinforced concrete member. For instance, if a #5 rebar is utilized, and it experiences corrosion loss of a 1/16” thick flake of its outer layer, that rebar effectively becomes the equivalent of a #4 rebar **and** loses its bond with the concrete. In this regard, with each 1/16” increment of flake thickness, the rebar loses a bar size. This condition prevents the tensile force from reaching the reinforcement, and causes failure of the concrete member due to inadequate capacity as the effective area of rebar decreases.	This condition typically develops due to improper construction of the concrete section and/or exposure to the environment over the life of the structure. The improper construction conditions can be identified and addressed during progressive inspection during construction. Following completion of construction, and due to corrosion, this condition is most commonly identified due to excessive cracking or spalling of the reinforced concrete. To remedy widespread and hidden corrosion, the affected areas must be inspected visually, audibly, and with penetrating radar in order to conclusively address and remedy the condition. Such conditions caused by construction issues are likely widespread throughout the structure. Conditions caused by corrosion are typically located along the base, roof, and perimeter of structures and become more prevalent as the building ages.
Improper placement of steel reinforcement	The improper placement of steel reinforcement affects the strength of the member by moving the intended location of the tension, compression, and/or confinement steel, as well as having the potential to prevent proper distribution of the concrete. This condition causes failure of the concrete member due to inadequate capacity, and can lead to an increase in corrosion when minimum concrete cover is not achieved.	Typically, the improper placement of reinforcement is identified and remedied during progressive inspection during construction. The identification of widespread improper steel placement following completion of construction is indicative of widespread poor construction and inspection practices. To remedy widespread improper steel placement conditions, the entirety of the concrete structure must be inspected with penetrating radar in order to conclusively address and remedy the condition.
Lack of reinforcement lap or development length	Inadequate lap or development length of steel reinforcement affects the strength of the member by preventing the development and continuity of the tension, compression, and/or confinement steel. This condition causes failure of the concrete member due to inadequate capacity at the lap or embedment locations.	Typically, inadequate lap and development length is identified and remedied during progressive inspection during construction. The identification of widespread improper reinforcement continuity following completion of construction or prior repairs is indicative of widespread poor construction and inspection practices. It is near impossible to identify the location of inadequate reinforcement continuity conditions without chipping and removing the concrete to expose the reinforcement.

Corrosion of Reinforced Concrete

Based on our understanding of ACI 222R-19 Guide to Protection of Reinforcing Steel in Concrete Against Corrosion, ACI 201.2R-16 Guide to Durable Concrete, and ACI 365.1R-00 Report on Service-Life Prediction, we understand that corrosion of conventional steel reinforcement in concrete is an electrochemical process that forms either local pitting or general surface corrosion. Corrosion in reinforced concrete structures can result in significant damage. Corrosion-induced damage in reinforced concrete structures has related costs not only for the corrosion repair itself, but also for maintaining such structures in a serviceable condition. In extreme cases, corrosion-induced damage has led to structural failures in the form of partial or total collapse.

Selecting the most technically viable and cost-effective remedial measure for deteriorated structural concrete in a corrosive environment is a formidable task. The alternatives span the extremes of inaction to complete replacement of the structure. Some type of corrosion prevention or rehabilitation measure is deemed appropriate is generally acceptable in the early to mid-life of the structure. However, as discussed herein, corrosion repair is often cyclic, and a structure with deep-rooted corrosive conditions eventually require replacement of segments or the whole of the structure.

Concrete protects against corrosion of embedded steel because of the highly alkaline environment provided by the pore fluid of the portland cement paste. The adequacy of the protection depends on the depth of concrete cover, the quality of the concrete, the details of the construction, the degree of exposure to chlorides from concrete component materials and from the environment, and the service environment.

The process of corrosion of steel in concrete is divided into several phases:

1) Initiation: the normal protective passive layer on the steel breaks down
2) Corrosion growth (propagation): the (active) corrosion process is established and corrosion progresses
3) Damage: corrosion is sufficiently severe that cracking, spalling, or both, occur and eventually the structural element may not perform its intended function.

The high alkalinity, with a pH greater than 12.5, of concrete protects embedded steel reinforcement in concrete from corrosion. When oxygen is present, the high pH of the pore solution causes an ultra-thin corrosion film to form on the steel surface, termed a “passive film”. The composition of this film depends upon the metallurgy of the metal and is understood to be a combination of hydroxides and oxides. This film is in equilibrium with the environment, slows corrosion reactions, and, thus, the steel is protected against active corrosion and is said to be “passivated”.

Depending on the penetrability of concrete cover over the steel and the alkalinity of the concrete pore solution, the passive film is maintained. If the passive film breaks down, termed “depassivation,” corrosion rate accelerates and the propagation phase begins. The film can break down locally so that localized corrosion results. If breakdown occurs over larger areas, more uniform general corrosion takes place.

Good workmanship is vital for securing uniform concrete with low penetrability. For low slump concrete, segregation and honeycombing can be avoided by good consolidation. Meeting the requirements of the specifications pertaining to durability are essential. Two factors are important to consider in detailing of the reinforcement:

1. Adequate spacing should be provided to allow for proper placing of the concrete cover so that honeycombing and poor compaction are avoided and good bond between concrete and steel are obtained.

2. Corrosion is relatively more severe for small bars than for large bars. Corrosion of a No. 3 (10 mdm) bar totaling 0.04 in. (1 mm) of corrosion means nearly 40 percent loss of cross section, whereas for a No. 8 (25 mm) bar, it will mean 15 percent loss of cross section. Note, however, that large bars could cause larger cracks than smaller bars because smaller bars can give better crack distribution.

Corrosion can occur even in instances of good workmanship due to passive film breakdown. The primary causes of film breakdown include:

a) Chemical, physical, or mechanical degradation of the concrete cover
b) Chloride penetration to the reinforcement
c) Carbonation of the concrete to reinforcement depth
d) Change of polarization of the reinforcing steel such as in dissimilar metal corrosion or stray current leakage.

The most common cause of initiation of corrosion of steel reinforcement in concrete is the presence of chlorides. Chlorides are a major contributing factor in the corrosion of steel in concrete. Chloride content results are reported in percent chloride by mass of concrete, parts per million (ppm) chloride, percent chloride by mass of cement, or pounds of chloride per cubic yard (kilograms per cubic meter) of concrete. Chloride content above a certain concentration known as the chloride threshold will cause local breakdown of the passive layer, leading to corrosion. Cracks permit much faster chloride infiltration rate than diffusion processes, and can establish chloride concentration cells that accelerate corrosion. Maximum permissible chloride-ion contents, as well as minimum concrete cover requirements, are provided in codes and guides.

The Florida Building Code references the ACI 318 as the Standard for Reinforced Concrete. ACI 318-14 Table 19.3.2.1 specifies the Maximum water-soluble chloride ion content in concrete, percent by weight of cement (chloride ion limit). The concrete within the Deauville structure is considered as exposed to moisture along the building perimeter and ground level. Considering the Deauville's close proximity to the Atlantic Ocean, we determined that the concrete is exposed to salt, brackish water, seawater, or spray from these sources (ACI 318-14 T19.3.1.1 Exposure Class C2). As such, the chloride ion limit for the Deauville perimeter and ground level concrete was determined to be 0.15, per ACI 318-14 T19.3.2.1. We noted that the chloride ion limit for concrete that is not exposed to an external source of chlorides (Exposure Class C1) is 0.30.

Prior Repairs

During our inspections, we observed apparent recent corrosion repairs in the form of unpainted concrete patches along the exterior perimeter walls, columns, beams and slabs, and unfinished corrosion repairs in the form of chipped and sawcut concrete eyebrows, balconies, and exterior walls. We understood from Mr. Chanfrau that the most recent corrosion repairs took place between 2015 – 2017, which coincides with our observations during our review of the historical aerials and permit history. Mr. Chanfrau provided us with 120 photographs dated between 2015-2017. We were not provided with assessment reports, structural plans, or repair plans as of the date of issuance of this report. We were not provided with the name of the contractor or structural engineer associated with the prior repair work. We were not provided with a reason for why the concrete repairs were halted prior to their completion.

We reviewed the 120 photographs and noted the following conditions. It is our opinion that, based on the conditions observed within the photographs, the 2015-2017 concrete repairs did not feature adequate lap splices, development length, and/or replacement of unsound concrete or corroded reinforcement. As such, we do not consider the prior repairs to be sufficient in restoring the structure to its predamaged condition, and may have effectively reduced the strength of the system within and along the repair areas. The location of the prior repairs appear to have been located along the area denoted with blue highlight in Image 95.

Typical Conditions Noted During Review of Prior Repair Photos

During our review of the prior repair photographs, we observed the following typical conditions. Photographs representative of the below noted conditions have been included within this section of our report. Additional observations are included within the Observations portion of this report. See Appendix C for approximate strength reduction of corroded rebar.

7. Reinforcement Condition
    a. Different bar types within the same group of bars
    b. Main columns utilized wire ties rather than #3 rebar stirrups
    c. Confinement steel was not adequately provided
    d. Smooth, undeformed, rebar
    e. Discontinuous rebar at joints
8. Corrosion
    a. Reinforcement cross-sectional loss approximately 17-46% within corrosion area
    b. Stirrup deterioration 100%
9. Construction Defects (Original Construction)
    a. Inadequate mixture of aggregate and paste during concrete placement
    b. Closely spaced reinforcement
    c. Honeycombed concrete
    d. Insufficient lap splice of reinforcement
    e. Offset reinforcement (inadequate clear cover)
    f. Vertical construction joints within columns
10. Nonconformance of repairs to Code & Standards
    a. New rebar placed outside of stirrups
    b. Existing stirrups cut and not replaced
    c. Existing longitudinal and/or transverse rebars cut and not replaced

d. Dowel embedment of flexural and/or shear rebar (discontinuous bars)
e. Removal of reinforcement without like kind replacement
f. Lap splice of flexural beams placed at mid-span

Image 102: *Hotel Column*

Image 103: *Hotel Column*

Image 104: *Hotel Column*

Image 105: *Hotel Column*

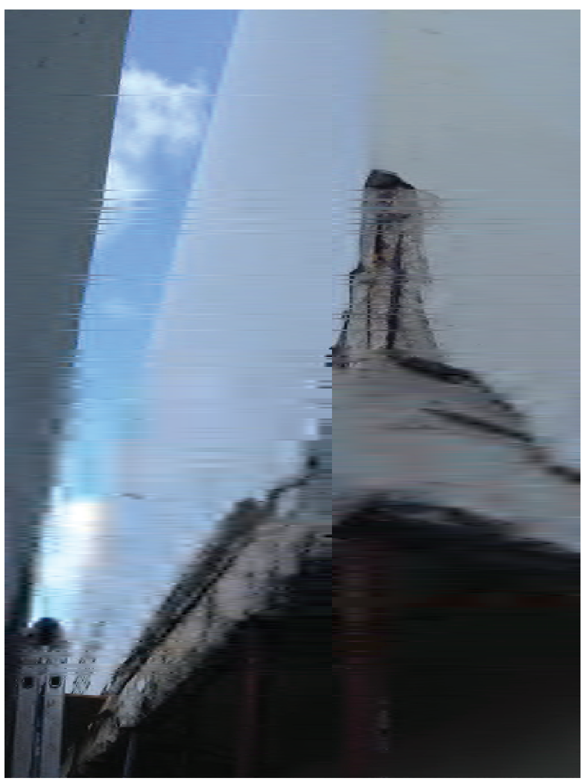

Image 106: *Hotel Columns, beams, and exterior wall*

Image 107: *Hotel North Wall Columns, beams, and exterior wall*

Image 108: *Hotel North Wall Beams*

Image 109: *Hotel Column*

Image 110: *Hotel Column*

Image 111: *Hotel Column*

Image 112: *Hotel Beam*

***Image 113**: Hotel Column & Frame Beam Connection*

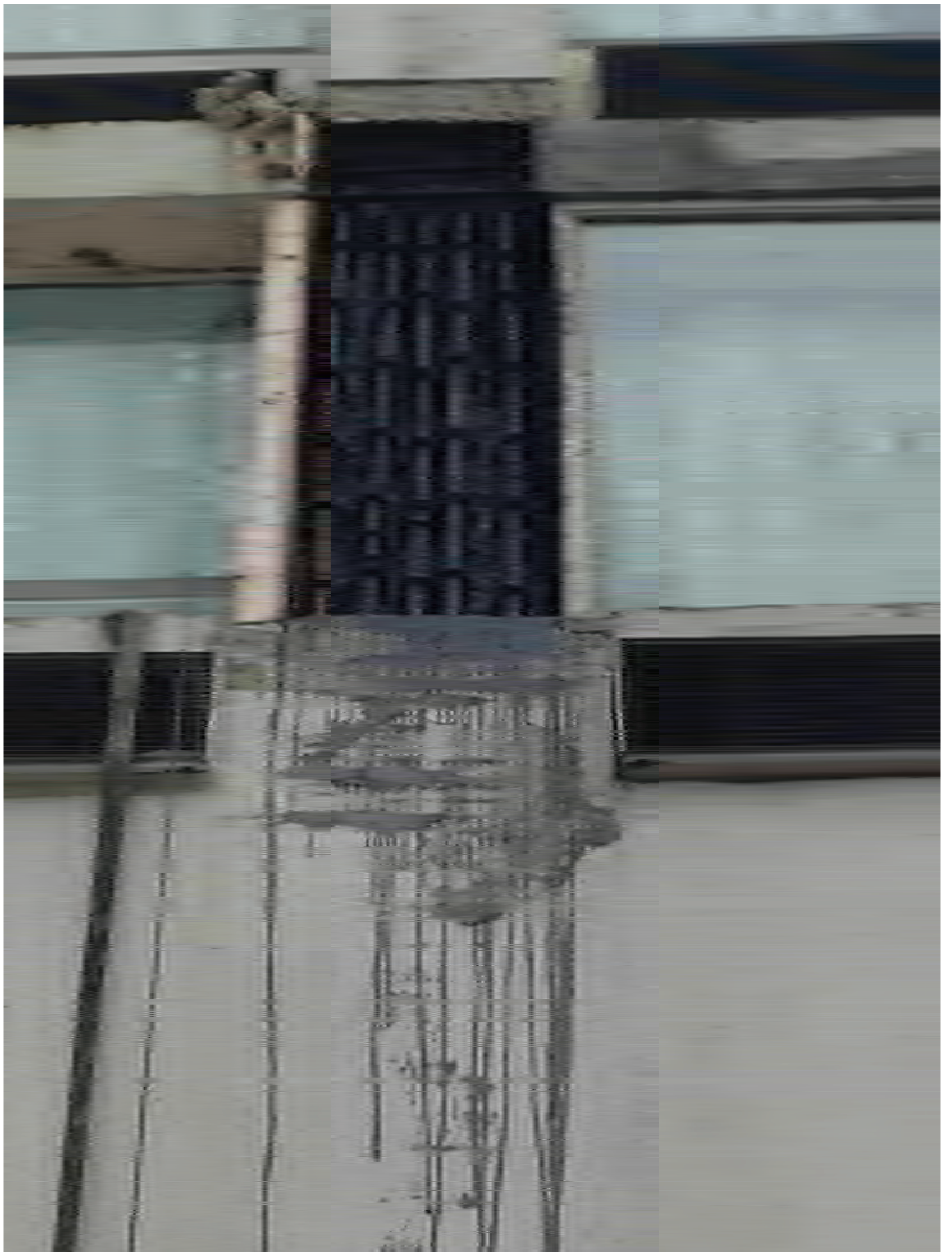

***Image 114**: Hotel Column & Frame Beam Connection*

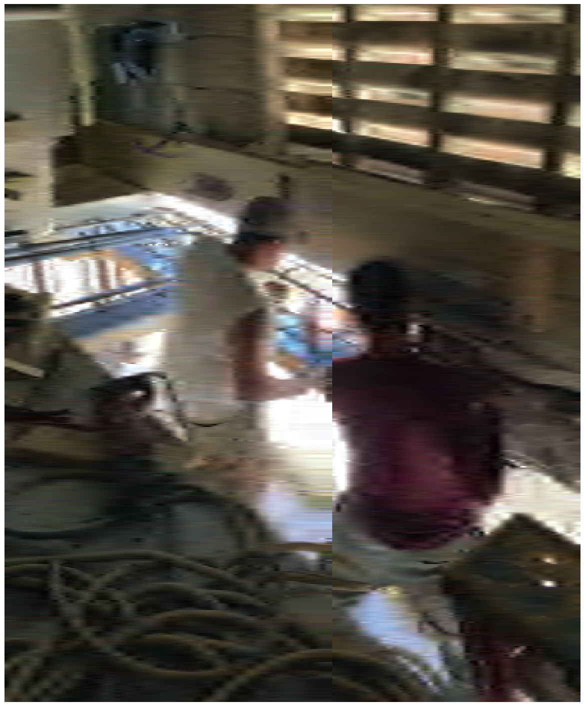

Image 115: *Hotel North Wall Frame Beams*

Image 116: *Hotel Column*

***Image 117**: Hotel Column & Frame Beam Connection*

***Image 118**: Hotel Column & Frame Beam Connection*

***Image 119**: Hotel Frame Beam*

***Image 120**: Hotel Column and Exterior Wall*

Image 121*: Hotel Column and Exterior Wall*

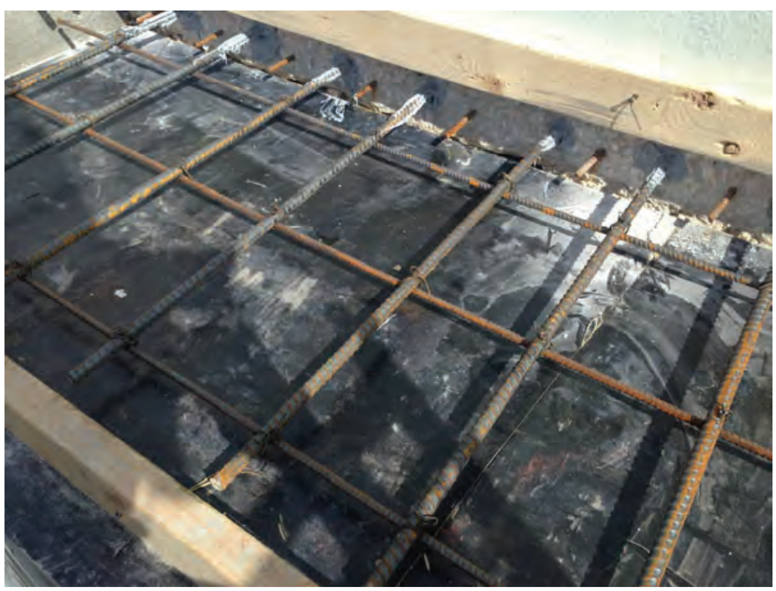

Image 122*: Hotel Eyebrow/Slab*

Image 123*: Hotel North Wall Slab at Frame Beams*

Image 124*: Hotel Columns and Frame Beams*

Image 125: *Hotel North Wall Column*

Structural Systems

To paraphrase ACI 318-14, overall structural integrity relies not only on the design of individual members, but also on the design of the structure as an entire system. A structural system consists of structural members, joints, and connections, each performing a specific role or function. A structural member may belong to one or more structural systems, serving different roles in each system and having to meet all the detailing requirements of the structural systems of which they are a part. Joints and connections are locations common to intersecting members or are items used to connect one member to another, but the distinction between members, joints, and connections can depend on how the structure is idealized.

Structural integrity of the entire system requires redundancy and ductility through detailing of reinforcement and connections so that, in the event of damage to a major supporting element or an abnormal loading, the resulting damage will be localized and the structure will have a higher probability of maintaining overall stability. Therefore, reinforcement and connections shall be detailed to tie the structure together effectively and to improve overall structural integrity.

Within a structural system, floor and roof slabs play a dual role by simultaneously supporting gravity loads and transmitting lateral forces in their own plane as a diaphragm. Diaphragms, such as floor or roof slabs, shall be designed to resist simultaneously both out-of-plane gravity loads and in-plane lateral forces. All structural systems must have a complete load path.

Structural Analysis

To paraphrase ACI 318-14, the role of analysis is to estimate the internal forces and deformations of the structural system and to establish compliance with the strength, serviceability, and stability requirements of the Code. The Code requires that the analytical procedure used meets the fundamental principles of equilibrium and compatibility of deformations.

The basic requirement for strength design may be expressed as follows:

$$\text{design strength} \geq \text{required strength}$$

$$\phi Sn \geq U$$

In the strength design procedure, the level of safety is provided by a combination of factors applied to the loads and strength reduction factors ϕ applied to the nominal strengths. The strength of a member or cross section, calculated using standard assumptions and strength equations, along with nominal values of material strengths and dimensions, is referred to as nominal strength and is generally designated Sn.

Design strength or usable strength of a member or cross section is the nominal strength reduced by the applicable strength reduction factor ϕ. The purpose of the strength reduction factor is to account for the probability of understrength due to variations of in-place material strengths and dimensions, the effect of simplifying assumptions in the design equations, the degree of ductility, potential failure mode of the member, the required reliability, and significance of failure and existence of alternative load paths for the member in the structure.

This Code, or the general building code, prescribes design load combinations, also known as factored load combinations, which define the way different types of loads are multiplied (factored) by individual load factors and then combined to obtain a factored load U. The individual load factors and additive combination reflect the variability in magnitude of the individual load effect, the probability of simultaneous occurrence of various load effects, and the assumptions and approximations made in the structural analysis when determining required design strengths.

The strength-design method was primarily adopted in the 1960s, and as such it is more than likely that the Deauville was designed using service-design method rather than strength-design method. Service-design methods do not particularly account for probability of understrength as with the strength-design method.

Providing more strength than required by structural analysis does not necessarily lead to a safer structure because doing so may change the potential failure mode. For example, increasing longitudinal reinforcement area beyond that required for moment strength as derived from analysis without increasing transverse reinforcement could increase the probability of a shear failure occurring prior to a flexural failure.

To paraphrase ACI 562-19, member deterioration and damage may result in distribution of internal forces different than the distribution of forces of the original structural design. In order to keep a structure in service, the state of the structure should be accurately modeled to determine the distribution of forces. A primary purpose of construction observation of rehabilitation work is to verify that the exposed existing construction is as assumed in the design and that the work detailed in the contract documents will fulfill the design intent. If the existing construction differs from the design assumptions, requiring modification of the design, changes should be documented and the work modified as necessary.

Structural assessments are required when damage, deterioration, structural deficiencies or behavior are observed during the preliminary assessment that are unexpected or inconsistent with available construction documents. Results of the condition assessment should also be reviewed to identify if potentially dangerous conditions are present. Potentially dangerous structural conditions include any instability, the potential for collapse of overhead components or pieces (falling hazards), or a significant risk of collapse exists under service load conditions.

Analysis

Based on our understanding of the FBC and ACI Standards referenced within this report, the goal of our assessment was to examine available information about the structure and to make a determination of its adequacy to withstand in-place environmental conditions and design loads. Structural performance cannot be considered as acceptable if past and present performance has indicated structural distress beyond expected levels. Our review and analysis of in-place conditions documented the loss of strength due to deterioration. We discuss the extent of damage and potentially dangerous structural conditions herein.

The affected structural members are not only members with obvious signs of distress but also contiguous members and connections in the structural system. Our assessment of the Deauville considered the effects of material deterioration, loss of steel area due to corrosion or other causes, and missing or misplaced reinforcement, as well as construction defects in the form of poorly mixed and placed concrete throughout the structure. Our assessment of the structure is further detailed within our Analysis.

Structural Integrity

ACI 318-14 Chapter 27 provides the building code requirements for Strength Evaluation of Existing Structures. The provisions of Chapter 27 may be used to evaluate whether a structure or a portion of a structure satisfies the safety requirements of the Code. The code requires that if there is doubt that a part or all of a structure meets the safety requirements, and the structure is to remain in service, a strength evaluation shall be carried out. The strength evaluation must include either an analytical evaluation of strength based on the existing member dimensions, layout, and material properties, or a load test is carried out on each individual structural system.

Our scope of work was to determine if the Deauville can remain in service, and as such, a strength evaluation is required by Code due to our observations that the structural materials were deficient in quality, there was evidence indicating faulty construction, and that the structure did not appear to satisfy the requirements of the Code.

The strength evaluation of a structural system requires the following information, at a minimum:

- Member layout in order to determine location of all critical sections
- Dimensions of members shall be established at critical sections
- Locations and sizes of reinforcement
- An estimated equivalent fc′ shall be based on analysis of results of cylinder tests
- Placement of concrete and reinforcement per Code requirements, to ensure a heterogenous section and continuous load path through all elements
- Sufficient lap and development length of reinforcement

The member dimensions and layout of the Deauville were not typical along the floor plans or throughout each level of the structure. The reinforcement size and distribution were not consistent in like members as observed throughout the ground level of the structure. The observed construction defects such as placement of the concrete and reinforcement did not meet code requirements for concrete cover, steel spacing, lap length, confinement, or heterogenous concrete placement. As such, the information we gathered on site in order to perform a strength evaluation of the structure instead proffered the conclusion that the as-built condition of the system could not be analyzed in order to determine its strength, since the elements inherent to reinforced concrete design were not met.

A load test of a structural system is required in order for a structure to remain in service if a strength evaluation cannot be conducted. A load test of each structural system must be carried out in order for the design professional to evaluate its strength and serviceability. Load tests shall be conducted in a manner that provides for safety of life and the structure during the test. Load tests must occur within each unique type of structural system, and load the critical members. A load test is comprised of loading the structure with a calculated load based on code requirements, and then measuring the resultant deflection and stresses. A load test is not intended to cause distress of the system and is to be halted if distress is observed during load application.

In order for a load test to be considered as acceptable, the portion of the structure tested shall show no spalling or crushing of concrete, or other evidence of failure. If the structure shows no evidence of failure, recovery of deflection after removal of the test load is used to determine whether the strength of the structure is satisfactory. Localized casting imperfections in concrete members is expected and is accommodated within strength design procedures. However, the widespread casting imperfections and widespread mixing and steel placement defects observed throughout the Deauville were not localized and would not be considered as localized or within acceptable construction tolerances. The atypical floor plans and changes in floor layouts and structural systems would further necessitate several load tests per floor, which would not be feasible without removing all non-structural elements from within the building in order to expose all structural members in order to determine the test locations and perform the tests themselves. Further, the widespread and hidden nature of the construction defects at the Deauville could cause sudden failure or progressive collapse during a load test. As such, it was not responsible nor feasible for load tests to be carried out at the Deauville.

In the event that a load test can occur on a deteriorating structure, acceptance provided by the load test is, by necessity, limited in terms of future service life. When a deteriorating structure passes a load test, a periodic inspection program that involves physical tests and periodic inspections must be implemented in order to monitor and quantify the remaining service life of the structure. The length of the specified time period between inspections should be based on consideration of the nature of the deterioration, the environmental and load effects, the service history of the structure; and the scope of the periodic inspection program. At the end of a specified time period, further strength evaluation is required if the structure is to remain in service.

The construction defects within the reinforced concrete at the Deauville have been present since its construction, and as such, we could consider that they have undergone a load test for the loads the building has experienced to date. However, due to corrosion of the reinforcement steel and deterioration of the concrete, the strength of the structural system is decreasing by the order of time. As a structural system begins to fail, the structure will experience patterns of distress in the form of cracks, deflection, and/or deterioration. The frequency and magnitude of such distress must be evaluated wholistically throughout the structure and over time, in order to determine the remaining service life.

The service history of the structure in regard to corrosion repair and halo effects, since 1992, has been necessary on a 10-year cycle until 2012, when corrosion repairs began to occur on a 5-year cycle. A prescribed 5-year corrosion repair cycle of the main structural members throughout the structure would not be maintainable or feasible if the Deauville is returned to service.

Although we were unable to calculate the design strength or perform a load test of the Deauville lateral or vertical (gravity) system in a quantitative manner due to the conditions encountered on site, we were able to perform a general analysis of the reduction in strength of the vertical elements (columns) of the gravity system in a qualitative manner by reviewing the loss of steel cross-sectional area, the presence of widespread voids within the concrete, and the ineffective placement/bond of reinforcement steel. Such analysis resulted in a reasonable assumption of 20-58% loss of steel cross-sectional area coupled with 10-30% loss of concrete cross-sectional area due to widespread voids and improper concrete placement, relative to the Severe and Poor-rated columns inspected below the Hotel portion of the building. Such conditions in conjunction with the understanding that the basic wind speed to be applied to the Deauville increased 21-32% between its original design wind load and the current required design wind load, were indicative that the capacity of the group of columns carrying gravity load, which supported more than 30 percent of the total area of the structure's floor(s) and roof(s) had been reduced more than 20% from its predamage condition and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75% of that required by FBC 2020. As such, we considered the damage at the Deauville to be classified as substantial structural damage.

Due to the extent of the construction defects, corrosion, and deterioration discussed within this report, the Deauville was not able to be analyzed by strength evaluation or load test as described within ACI 318-14, and as such cannot be returned to service. The nature of the construction defects within the reinforced concrete system make it infeasible to analyze and therefore repair the structure in order to withstand its original or current design load requirements. The 5-year cycle of corrosion repairs, the chloride ion content measured in select columns, and the magnitude of deterioration of steel and concrete observed during our inspections indicates that the building as a whole is in distress and has exceeded its service life.

Potential Collapse Locations

Based on our experience, knowledge, and understanding of the building condition as described herein, we have indicated the locations where potential local collapse is likely to occur if the building is returned to service in its present condition within Image 126. Due to the presence of transfer slabs and the lack of isolation joints, we have also indicated that the local collapse areas are likely to cause progressive collapse to the remainder of the adjacent continuous structure either north or south of the isolation joint.

Note that since a transfer slab is located above the 3rd Floor of the Hotel, the failure of the ground level columns (purple shaded areas) would inherently cause progressive collapse of the Hotel Portion of the building regardless of if the Lobby Level columns failed in a progressive manner. The local failure of the corroded beams and columns in the southeast corner pool equipment rooms would cause progressive collapse of the radius ballroom above, and thereby the potential progressive failure of the adjacent 3rd floor transfer slab below the Hotel. In a similar manner, since the South Ballroom roof is supported by the building's east and west frames, the failure of the ground level columns (yellow shaded areas) would inherently overload the east and west frames, causing the progressive collapse of the South Ballroom roof structure.

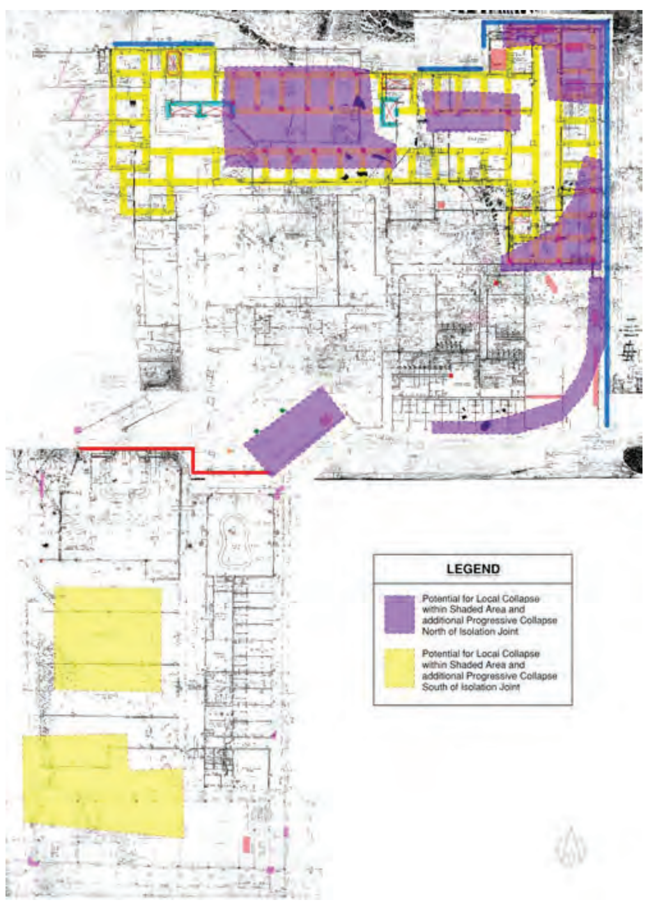

Image 126: *Potential local collapse and resultant potential progressive collapse locations within the continuous adjacent structure based on our assessment*

Remaining Service Life

The Deauville was constructed in 1957 and is approximately 64 years old at the time of issuance of this report. Reinforced concrete structures in South Florida and along the Coast generally have a service life of 30-50 years due to quality of original construction as well as the ambient corrosive environment. The Deauville's 50th year occurred in 2007. Prior to 2007, it was apparent from our review of the Permit History that the Deauville was on an approximate 10-year maintenance cycle for corrosion repairs, which is consistent with our experience of 50-year-old concrete structures along the Coast in South Florida. Following its 50th year, the Deauville's cycle for concrete corrosion repair shortened to every 5 years, which indicated that the corrosion process continued to accelerate and was therefore not able to be eliminated by means of repairs. The corrosion damage observed throughout the Deauville, coupled with the widespread construction defects within the reinforced concrete members and increased applied wind pressures, indicated that it is not a viable candidate for further extension of its service life.

As presented within this report, the conditions observed and documented during our inspections and assessment of the Deauville meet the following end-of-life criteria as defined by ACI 365 and ACI 562:

- Structural safety is unacceptable due to material degradation or exceeding the design load-carrying capacity
- Severe material degradation, such as corrosion of steel reinforcement initiated when diffusing chloride ions attain the threshold corrosion concentration at the reinforcement depth
  - Exceeding maximum permissible chloride level at the interface of the steel in the repair area, or in adjacent areas
  - Unacceptable reinforcement section loss due to corrosion
- Maintenance requirements exceed available resource limits
- Unacceptable frequency of maintenance cycles and associated activities

It should be noted that per ACI 365 and ACI 562, the presence of only one of the above criteria is sufficient to indicate that a structure has met or exceeded its service life. Based on our experience, observations, and within a reasonable degree of engineering certainty, we concluded that the Deauville has exceeded its service life and cannot return to service without extensive, widespread replacement of the reinforced concrete and a complete design analysis to meet current code requirements.

Recommendations

The deterioration and construction defects noted throughout the Deauville were not limited to areas with visible signs of distress. During our inspection, we inspected areas of visible distress as well as areas with no signs of distress, meant to act as control conditions. Following the removal of the interior gypsum board, drop ceiling, and other obstructions as well as testing efforts, we observed concrete deterioration and/or construction defects either along the face of the column or discovered the defects within the columns while attempting to measure the concrete compressive strength. Multiple tests on a single column produced inconsistent and varied results due to the poor construction and material quality of the reinforced concrete. As such, and considerate of the progressive collapse mechanisms inherent to the Deauville structural system, the entirety of the interior non-structural elements of the Deauville would need to be removed, and the entirety of the structure would need to be inspected relative to the visible and hidden reinforced concrete conditions. Such an inspection, and its resultant repairs, would require a tremendous expenditure of time and costs, would be intrusive, and may cause sudden local and/or progressive collapse. The hidden nature of the construction defects, and the observed conditions during our scope of work, also presents a high risk of uncertainty during and following the repair and rehabilitation.

Repairs and Rehabilitation

ACI 364.1R-19, Guide for Assessment of Concrete Structures before Rehabilitation, indicates that the evaluation of rehabilitation approaches should consider the following criteria:

a) Probability of success
b) Achievable service life
c) Initial costs and future maintenance costs
d) Relative risks and uncertainties
e) Disruption to operations

Each rehabilitation approach will have associated future maintenance costs. For example, lower initial costs may have considerably higher long-term maintenance costs. In any case, ACI 562 recommends that the licensed design professional establish the expected service life of repairs and advise owners of future maintenance needs of the rehabilitated structure.

Recommended rehabilitation approaches will be dependent on not only the cause of the observed distress, but also the extent of distress. Distress that is more widespread or more severe and affecting more portions of the structure may require more invasive rehabilitation approaches. For example, if chloride-contaminated concrete has contributed to widespread corrosion of reinforcement, a more significant and invasive repair approach may be necessary.

The premise of all rehabilitation and repair approaches is that there is a remaining service life of the overall structure, and that the rehabilitation and repair can allow the overall structure to reach or exceed the remaining service life. While rehabilitation and repairs may extend a structure's service life, a structure cannot be repaired in perpetuity in order to have an infinite service life. This is especially applicable when widespread construction defects and material deterioration are present. In such a case, the increased frequency and magnitude of repairs are indications that a structure is beyond its service life and that repairs are no longer effective.

Within our scope of work, we concluded that the Deauville has exceeded its service life and cannot return to service without extensive, widespread replacement of the reinforced concrete and a complete design analysis to meet current code requirements. It is our opinion that the only rehabilitation approach which could potentially extend the service life of the Deauville is to essentially rebuild the reinforced concrete structural system in a controlled and segmented manner. Such a process would take an extended amount of time and would require extensive shoring and an extremely high cost There is a low probability of success in this approach, however, and there is a high relative risk of sudden local or progressive collapse during the rehabilitation process. Adding to the complication is the unknown nature and design of the Deauville's foundation system. As such, we do not recommend rehabilitation or repair of the Deauville.

Demolition

Based on our assessment as discussed herein, we recommend that the Deauville be demolished in a controlled fashion and in conjunction with additional guidance from a licensed Florida Professional Engineer with experience in the demolition and partial demolition of structures. The Deauville must remain out of service and should undergo demolition as soon as possible and prior to the next design wind event, which is mostly likely to occur during the 2022 Hurricane Season. Per the National Oceanic and Atmospheric Administration (NOAA), the 2022 Hurricane Season is expected to officially begin on June 1, 2022. It is our recommendation that the Deauville be demolished as soon as possible, and completed prior to the start of the 2022 Hurricane Season.

Based on our experience, the Deauville's demolition procedure must consider the following items, at a minimum.

- The nature of the Deauville's transfer slabs and lack of isolation joints requires that the Lobby and East Ballroom/Stage structure north of the isolation joint be considered to brace the Hotel Portion of the Deauville. In this regard, the portion of the structure below the Low Roof and north of the isolation joint should not be removed without a plan in place to immediately initiate the demolition of the Hotel Portion of the structure. The poor and severe condition of the columns below the Hotel Portion of the structure may cause the building to react in an unexpected manner during demolition.
- The structure below the Low Roof south of the isolation joint can be considered as a separate structure from the Hotel Portion of the building, and can be demolished in a controlled manner such that the demolition activities take place after the demolition of the north portion of the building, or that the demolition of the south structure does not damage or otherwise negatively impact the condition of the north portion of the building.
- Due to the condition of the Deauville, it is further recommended that the demolition of the Deauville be coordinated with the temporary closure of Collins Avenue, public sidewalk, and the public boardwalk which border the Deauville to the west and east, respectively.
- The condition assessment of the buildings and structures surrounding the Deauville were not included within our scope of work, and as such their conditions should be further coordinated with the development of the Deauville's demolition procedure.

Conclusions

Based on our observations, experience, analysis, and review of the documents referenced herein, and within a reasonable degree of engineering certainty relative to our scope of work, we concluded the following:

1. The Deauville has exceeded its service life and cannot return to service.

2. The Deauville cannot be repaired or rehabilitated without extensive testing and replacement of each structural element of the reinforced concrete system and the institution of a 5-year maintenance cycle. Such a repair and maintenance protocol is infeasible and not maintainable and therefore the Deauville cannot be repaired or rehabilitated.

3. The demolition of the Deauville should be completed as soon as possible and prior to the start of the 2022 Hurricane Season.

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Deauville Beach Resort - Structural Condition Assessment
Anesta Consulting File Number: 21-04005
Report Issued December 15, 2021

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This item has been digitally signed and sealed by Heather R Anesta**,** P.E., on the date adjacent to the seal. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Sincerely,

Digitally signed by
Heather Anesta
Date: 2021.12.15
11:57:53-05'00'

Heather Anesta, PE, SE, MS, LEED AP, StS2
Florida PE License No. 74733

President, On Behalf of Anesta Consulting, Inc. [Registry # 31160]
1151 W Magnolia Cir, Delray Beach, Florida 33445
heather@anestaconsulting.com
(561) 702-2569

Attachments: References, Curriculum Vitae of Heather R. Anesta, Photographs

Appendices

Appendix A: References
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Appendix A: References

This report was prepared based on our professional experience, knowledge, and the review of the following reference materials, relevant to our scope of work. Note that the author may not have been aware of every possibly relevant document.

1. Site photographs and data collected by Ms. Heather R. Anesta, P.E., during our site visits on August 27, September 24, September 28, September 29, October 8, October 22, and November 3, 2021.
2. Miami Dade County Property Appraiser, Historical Aerial Information, and Deauville-associated images available online.
3. Buildfax Property History Report Number 20210917143931649751-0BOBZI-489492863.
4. Florida Building Code Building & Existing Building, 2020, 7th Edition, and its referenced standards.
5. Southern Standard Building Codes dated 1953-54, 1965, & 1973, Standard Building Codes dated 1976, 1982, 1985, 1988, 1991, 1994, & 1997, and Florida Building Code dated 2001.
6. Merriam-Webster Dictionary
7. Proprietary or protected information relative to our engineering experience.
8. Documents, PDFs, and Photographs as listed throughout this report.
9. Reviewed the following Codes, Standards, and Publications:
    a. ACI/BRE/ICRI Concrete Repair Manual - 4th Edition, 2013 (CRM 2013)
    b. ACI 201.2R-16 Guide to Durable Concrete
    c. ACI 222R-19 Guide to Protection of Reinforcing Steel in Concrete Against Corrosion
    d. ACI 318-14 Building Code Requirements for Structural Concrete
    e. ACI 364.1R-19 Guide for Assessment of Concrete Structures Before Rehabilitation
    f. ACI 364.10T-14 Rehabilitation of Structure with Reinforcement Section Loss
    g. ACI 365.1R-00, Report on Service-Life Prediction
    h. ACI 546R-14 Guide to Concrete Repair
    i. ACI 562-19, Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures
    j. AISC Steel Construction Manual 2017 (AISC 2017)
    k. ASCE 7-16 Minimum Design Loads for Buildings and Other Structures with Supplement No. 1
    l. ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
    m. Federal Emergency Management Agency (FEMA), U.S. Department of Homeland Security (2000). Coastal Construction Manual – 3rd Edition.
    n. Florida Building Code 2020, 7th Edition, Building (FBC 2020)
    o. Florida Building Code 2020, 7th Edition, Existing Building (FBCEB 2020)
    p. Forensic Structural Engineering Handbook (2010) by Robert T. Ratay, Ph.D., P.E.
    q. Reinforced Concrete, A Fundamental Approach, 6th Edition, by Mr. Edward G. Nawy, Pearson Prentice Hall
    r. Reinforced Concrete Design of Tall Buildings, 2010, by Dr. Bungale S. Taranath, CRC Press
    s. 1953-54 Revisions to the Southern Standard Building Code (SSBC)

Appendix B: Curriculum Vitae (CV)

The following CV is current as of the issuance of this report. An updated CV, if available, can be provided in the future upon request.

Deauville Beach Resort - Structural Condition Assessment
Anesta Consulting File Number: 21-04005
Report Issued December 15, 2021

Heather Anesta PE, SE, MS, LEED AP, StS2

Structural Engineer, Project Manager, Logistics & Planning
Anesta Consulting, Inc

Ms. Anesta's background is rooted in structural and civil design, construction inspections, and multi-discipline engineering consultation and project management. She has personally designed numerous structures throughout South Florida, including single-family residences, multi-story hotels, commercial retail and warehouses, emergency operation centers, hurricane shelters, recreation centers, and water/wastewater treatment facilities. She also has extensive experience in the assessment of structural integrity, building failure, property damage assessment, and seawall and retaining wall design.

Experienced in the management, design, investigation, demolition, and inspection of new construction and existing buildings for multi-discipline and structural projects in the Private and Public Sectors, she utilizes her advanced engineering knowledge, problem-solving approach, and project management skills to serve as a Forensic Expert for property damage, construction claims, commercial liability, and product liability. Fluent in the lifecycle of buildings, retaining and seawalls, civil structures, and marine structures, she is able to contribute to, optimize, and lead projects in a clear and honest manner, with the ultimate goal of providing quality work that Clients can rely on.

She has served as an Expert in forensic engineering investigations and depositions for both Plaintiffs and Defendants, involving structural failure, property losses, and construction defect matters. She acts as a Consultant to homeowners, attorneys, insurance companies, adjustment firms, and other forensic engineering firms. She is a FEMA Subject Matter Expert and FEMA Adjunct Instructor in the field of Structural Engineering, and has assisted as a Structural First Responder in Hurricane Florence (2018), Hurricane Dorian (2019), the Surfside Building Collapse (2021), and Hurricane Ida (2021). She has performed structural evaluations of damaged structures following Hurricanes Matthew (2016), Irma (2017), Florence (2018), Michael (2018), Dorian (2019), and Sally (2020).

Heather is an Advanced Structural Specialist and Structural Collapse Specialist on FEMA Urban Search and Rescue Florida Task Force 1. She is a member of the ASCE 7-22 Wind Load Subcommittee, the NCEES Structural Engineers Exam Development Committee (SE Licensing Exam Development), and serves as an Advisor for the upcoming SEI Manual of Practice (MOP) titled, Guideline for Structural Condition Assessment of Existing Buildings (to replace ASCE 11-99). She is LEED Accredited, received the Young Professional Award in 2012 from the National Council of Structural Engineers, and is the Founder and Past-President of the National Structural Engineers Association's Young Members Group.

EDUCATION

Master of Science in Civil-Structural Engineering, Florida Atlantic University, Boca Raton, Florida, 2010
Bachelor of Science in Civil-Structural Engineering, Florida State University, Tallahassee, Florida, 2007

REGISTRATIONS

Professional Engineer #74733, State of Florida
Structural Engineer #018.0120343, State of Vermont
StS2, Advanced Structures Specialist, FEMA Urban Search & Rescue
SCS, Structural Collapse Specialist, FEMA Urban Search & Rescue
LEED Accredited Professional, U.S. Green Building Council

EMPLOYMENT HISTORY

Anesta Consulting, Inc., President & CEO	2015 – Present
Rimkus Consulting Group, Senior Consultant	2015 – 2021
Stantec Consulting, Inc., Associate, Project Engineer	2012 – 2015
C3TS, P.A., Project Manager, Project Engineer	2009 – 2012
Walsh Engineering, Inc., Structural Designer	2006 – 2009
McDaniel Engineering, Undergraduate Internship	2005 – 2006

This CV Exemplifies Forensic Experience.
A full list of Design and Project Management Experience is available upon request.

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Heather Anesta PE, SE, MS, LEED AP, SI52
Structural Engineer, Project Manager, Logistics & Planning
Anesta Consulting, Inc

Forensic Investigations & Expert Opinions

Various Property & Construction Inspections & Reviews for Cause, Origin, and Duration of Damage

- Building Envelope
- Catastrophic Events
- Collapse
- Construction Claims
- Construction Defects
- Construction Materials
- Flooring
- Marine
- Moisture Exposure & Duration
- Peer Reviews
- Pool Damage
- Roof Damage
- Storm-Related Damage
- Structural Integrity Evaluations
- Vehicle Impact
- Water Intrusion
- Water Leaks

Deposition Experience

Ms. Anesta has provided deposition testimony in Florida for plaintiffs and defendants.

- *Roof Damage Cause & Origin*
- *Water Damage Cause & Origin*
- *Seawall Damage and Replacement*
- *Kitchen Cabinet Detachment Cause & Origin*
- *Floor Tile Delamination Cause & Origin*

Additional Design and Management Experience

Demolition Consultation

Project Management & Logistics Planning

Structural Analysis & Design

- Marine
  - *Boat Ramps, Boat Lifts*
  - *Seawalls*
  - *Stationary & Floating Docks*
  - *Water Parks, Aquatic Parks*
- Buildings
  - *Single-Family, Multi-Family, Hotels, Apartments*
  - *One to Six Stories*
  - *Concrete, CMU, Timber, Steel, Precast Concrete, Post-Tension Concrete*
  - *Feasibility Studies*
  - *Shallow and Deep Foundations*
  - *New Construction*
  - *Renovations & Repairs*
- Civil Structures
  - *Water Treatment Plants*
  - *Wastewater Treatment Plants*
  - *Lift Stations*
  - *Pump Stations*
  - *Fire Stations*
  - *Fleet Maintenance Buildings*

Structural Condition Assessments

- Marine
  - *Boat Ramps, Boat Lifts*
  - *Corrosion – Steel and Concrete*
  - *Seawalls*
  - *Stationary & Floating Docks*
  - *Water Parks, Aquatic Parks*
- Buildings
  - *Single-Family, Multi-Family, Hotels, Apartments*
  - *Concrete, CMU, Timber, Steel, Precast Concrete, Post-Tension Concrete*
  - *Shallow and Deep Foundations*
  - *Building Code Analysis*
  - *Concrete Crack Assessments*
  - *Repair Protocols*
  - *Retaining Walls*
  - *Structural and Civil Design Peer Reviews*
- Civil Structures
  - *Water Treatment Plants*
  - *Wastewater Treatment Plants*
  - *Lift Stations*
  - *Pump Stations*
  - *Fire Stations*
  - *Fleet Maintenance Buildings*

This CV Exemplifies Forensic Experience.
A full list of Design and Project Management Experience is available upon request.

Heather Anesta PE, SE, MS, LEED AP, StS2

Structural Engineer, Project Manager, Logistics & Planning
Anesta Consulting, Inc

Leadership Positions

HUD Residential Resilience Guidelines – Wind Task Group Leader & Advisory Group Member	2020 – Present
Soccer Association of Boca Raton – Adult Program Director	2020 – Present
National Council of Structural Engineers Associations – SEER Committee Southeast Leader	2017 – Present
Florida Structural Engineers Association – South Florida Chapter Treasurer	2015 – 2017
ACE Mentorship – Structural Engineering Mentor & Coordinator	2013 – 2017
Florida Structural Engineers Association – State Membership Committee Chair	2013 – 2016
Stantec Structural Discipline Coordination Leader – Quality Management	2014 – 2015
National Council of Structural Engineers Associations – Joint BEC & YMG Committee Project Leader	2013 – 2014
Stantec Leadership Development Program	2013
National Council of Structural Engineers Associations – Young Member Group Founder & Chair	2012 – 2014
C3TS Partnership Training	2011 – 2012
Florida Structural Engineers Association – Palm Beaches Young Member Group Founder & Chair	2011

Team & Committee Positions

SEI Manual of Practice, Guideline for Structural Condition Assessment of Existing Buildings – Advisor	2021 – Present
ASCE 7-28 Wind Research Advisory Panel – Advisor	2021 – Present
HUD Residential Resilience Guideline – Technical Advisor	2019 – Present
FEMA Mitigation Assessment Team – Technical Consultant	2018 – Present
National Council of Structural Engineers Associations – SEER Committee Member	2017 – Present
FEMA Florida Task Force 1 – Structural Specialist	2015 – Present
National Council of Examiners for Engineering and Surveying – Structural Exam Development Member	2015 – Present
ASCE 7-22 Wind Load Subcommittee – Associate Member	2017 – 2021
National Council of Structural Engineers Associations – Wind Advisory Committee Member	2015 – 2020
Florida Structural Engineers Association – South Florida Chapter Board of Directors	2013 – 2017
National Council of Structural Engineers Associations – Continuing Education Committee Member	2014 – 2016
National Council of Structural Engineers Associations – Young Member Group Committee Member	2012 – 2016
FES-FICE – Structural Committee Member	2013 – 2014

Awards & Certifications

Incident Command System 300 & 400 Certification, Management of Advanced, Complex, and Expanding Incidents	2021
FEMA Adjunct Instructor & Subject Matter Expert (Structural)	2019
SCS – Structural Collapse Specialist (Rescue Specialist) (FEMA Urban Search & Rescue)	2018
PADI Certified Diver	2018
StS2 – Advanced Structures Specialist (FEMA Urban Search & Rescue)	2017
Level I "Authorized Person" Rope Access Training	2016
StS1 – Structures Specialist 1 (FEMA Urban Search & Rescue)	2015
Stantec Emerging Leader	2013
National Council of Structural Engineers Associations – Young Member Award	2012
Certified Masonry Inspector; Masonry Institute of America	2012

Lectures & Instructor Engagements

Main Wind Force Design of Low-Rise Buildings, ASCE Learning, Virtual Workshop	2021
Wind Design of Non-Rectangular Low-Rise Buildings, ASCE Learning, Webinar & Virtual Workshop	2021
Structural Collapse Specialist Course (SCS4.0), FEMA Adjunct Instructor, FEMA FL-TF1 Training Site, Kendall, FL	2019
Lessons Learned as a Rescue Engineer during Hurricane Florence, MDFR OEM, Miami, FL	2019

Publications

SEI Manual of Practice, Guideline for Structural Condition Assessment of Existing Buildings [ASCE 11]	In Progress
HUD Contract to Create Residential Resilience Guidelines for Builders & Developers	In Progress
FEMA Mitigation Assessment Team: Hurricane Michael Recovery Advisory 1, Technical Consultant	2019
FEMA Mitigation Assessment Team: Hurricane Michael Recovery Advisory 2, Technical Consultant	2019
FEMA Mitigation Assessment Team: Hurricane Michael Assessment Report, Technical Consultant	2019

This CV Exemplifies Forensic Experience.
A full list of Design and Project Management Experience is available upon request.

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Appendix C: Approximate Strength Reduction of Corroded Rebar

Bar Size	Nominal Diameter (in)	Nominal area (in²)	Flake Thickness (in)	Flake Thickness (in)	Reduced Diameter (in)	Reduced Area (in²)	Equivalent Bar	Strength Reduction
3	0.375	0.110	1/16	0.0625	0.250	0.05		55%
3	0.375	0.110	1/8	0.125	0.125	0.01		89%
3	0.375	0.110	3/16	0.1875				
3	0.375	0.110	1/4	0.25				
3	0.375	0.110	5/16	0.3125				
3	0.375	0.110	3/8	0.375				
3	0.375	0.110	7/16	0.4375				
3	0.375	0.110	1/2	0.5				
3	0.375	0.110	9/16	0.5625				
3	0.375	0.110	5/8	0.625				
3	0.375	0.110	11/16	0.6875				

Bar Size	Nominal Diameter (in)	Nominal area (in²)	Flake Thickness (in)	Flake Thickness (in)	Reduced Diameter (in)	Reduced Area (in²)	Equivalent Bar	Strength Reduction
5	0.625	0.310	1/16	0.0625	0.500	0.20	4	37%
5	0.625	0.310	1/8	0.125	0.375	0.11	3	64%
5	0.625	0.310	3/16	0.1875	0.250	0.05		84%
5	0.625	0.310	1/4	0.25	0.125	0.01		96%
5	0.625	0.310	5/16	0.3125				
5	0.625	0.310	3/8	0.375				
5	0.625	0.310	7/16	0.4375				
5	0.625	0.310	1/2	0.5				
5	0.625	0.310	9/16	0.5625				
5	0.625	0.310	5/8	0.625				
5	0.625	0.310	11/16	0.6875				

Bar Size	Nominal Diameter (in)	Nominal area (in²)	Flake Thickness (in)	Flake Thickness (in)	Reduced Diameter (in)	Reduced Area (in²)	Equivalent Bar	Strength Reduction
7	0.875	0.600	1/16	0.0625	0.750	0.44	6	26%
7	0.875	0.600	1/8	0.125	0.625	0.31	5	49%
7	0.875	0.600	3/16	0.1875	0.500	0.20	4	67%
7	0.875	0.600	1/4	0.25	0.375	0.11	3	82%
7	0.875	0.600	5/16	0.3125	0.250	0.05		92%
7	0.875	0.600	3/8	0.375	0.125	0.01		98%
7	0.875	0.600	7/16	0.4375				
7	0.875	0.600	1/2	0.5				
7	0.875	0.600	9/16	0.5625				
7	0.875	0.600	5/8	0.625				
7	0.875	0.600	11/16	0.6875				

Bar Size	Nominal Diameter (in)	Nominal area (in²)	Flake Thickness (in)	Flake Thickness (in)	Reduced Diameter (in)	Reduced Area (in²)	Equivalent Bar	Strength Reduction
9	1.128	1.000	1/16	0.0625	1.003	0.79	8	21%
9	1.128	1.000	1/8	0.125	0.878	0.61	7	39%
9	1.128	1.000	3/16	0.1875	0.753	0.45	6	55%
9	1.128	1.000	1/4	0.25	0.628	0.31	5	69%
9	1.128	1.000	5/16	0.3125	0.503	0.20	4	80%
9	1.128	1.000	3/8	0.375	0.378	0.11	3	89%
9	1.128	1.000	7/16	0.4375	0.253	0.05		95%
9	1.128	1.000	1/2	0.5	0.128	0.01		99%
9	1.128	1.000	9/16	0.5625	0.003	0.00		100%
9	1.128	1.000	5/8	0.625				
9	1.128	1.000	11/16	0.6875				

Bar Size	Nominal Diameter (in)	Nominal area (in²)	Flake Thickness (in)	Flake Thickness (in)	Reduced Diameter (in)	Reduced Area (in²)	Equivalent Bar	Strength Reduction
11	1.410	1.560	1/16	0.0625	1.285	1.30	10	17%
11	1.410	1.560	1/8	0.125	1.160	1.06	9	32%
11	1.410	1.560	3/16	0.1875	1.035	0.84	8	46%
11	1.410	1.560	1/4	0.25	0.910	0.65	7	58%
11	1.410	1.560	5/16	0.3125	0.785	0.48	6	69%
11	1.410	1.560	3/8	0.375	0.660	0.34	5	78%
11	1.410	1.560	7/16	0.4375	0.535	0.22	4	86%
11	1.410	1.560	1/2	0.5	0.410	0.13	3	92%
11	1.410	1.560	9/16	0.5625	0.285	0.06		96%
11	1.410	1.560	5/8	0.625	0.160	0.02		99%
11	1.410	1.560	11/16	0.6875	0.035	0.00		100%

Appendix D: Test Results – Compression Tests

Appendix E: Test Results – Chloride Tests

Deauville Beach Resort - Structural Condition Assessment
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Appendix F: Relevant Record Set Sheets