ly affected in the 4-scale metric. This might either be due to lesser discriminant capability of our features for partial damages or ambiguity in the definition of these classes which may have affected the manual ground truth preparation.

CONCLUSION

This paper presented an automatic system for the assessment of damage from high resolution imagery. Towards that end, we have proposed and tested techniques for image registration, building extraction, change detection and damage classification; each of which require little or

Table 2. A confusion matrix for 3-scale metric with RandomForest, number of trees = 7 (left) and A confusion matrix for 4-scale metric with AdaBoost, number of iterations = 3 (right)

	Actual		
Predicted	A	B	C
A	35	3	3
B	2	13	2
C	2	3	12

	Actual			
Predicted	A	B	C	D
A	32	5	2	1
B	4	3	2	1
C	0	0	4	0
D	3	0	3	15

no manual supervision. While the accuracy in previous approaches was limited by the number of control points in manual registration, our application of SURF-based feature detection was found to produce near perfect registration in 14 of the 15 image pairs at 50cm-2m resolution. We proposed a novel segmentation-based building detection algorithm. Our algorithm was able to accurately extract the boundary contours of buildings in a reasonable amount of time and gave a TPR(True Positive Rate) in the range of 85%-90%. We proposed change detection measures that reflect the kind of damages that occur after a windstorm. We used a combination of edge-based and color based measures to classify damage into qualitative states. The final results in classification were promising; 80% accuracy for a 3-scale damage metric and 72% accuracy for a fine grained 4-scale damage metric. While the proposed system attempts at automating every aspect of damage assessment, there are several areas which require manual supervision. Further, there are areas which require improvement in terms of computational time and robustness. All these provide new challenges that must be addressed in future work.

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Challenges Encountered Retrofitting an Existing Concrete Building Classified as an Essential Facility

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ABSTRACT

This paper summarizes the challenges encountered during design and construction of a voluntary hurricane retrofit of an occupied 1960's era four-story concrete frame structure located in the United States.

Recent hurricane-related disasters provoked the owner to review their essential building inventory to determine if they could withstand large-scale hurricane events. The owner has developed their own internal wind design criterion for the site that is significantly higher than the minimum code required wind design. A detailed review of the original design standards and construction documents determined that the essential building would not meet the level of performance required of a "hurricane shelter" that could remain operational during and after a significant hurricane event. A structural retrofit was required to bring the building's exterior envelope and lateral force-resisting system up to the owner's current wind design standards for essential facilities.

A conceptual design study was performed to better understand the requirements of the building retrofit. Three schemes were investigated for strengthening the lateral force-resisting system, which included interior and exterior strengthening options. Given the three retrofit concepts, the owner decided to proceed with the exterior concrete shear wall system to minimize interruption to the building occupants and operations that could not be relocated during construction.

Once the concept was chosen and the decision was made to proceed with design and construction, the owner began setting aggressive goals for the overall project timeline. The design team was faced with many challenges including time constraints, high wind design parameters, specification of various architectural components, consideration of construction techniques, and scope creep. This paper discusses this unique project case study and examines some of the project challenges in detail.

INTRODUCTION

Recent wind-related disasters, such as Hurricane Katrina and Rita along the Gulf Coast, have provoked many building owners to be concerned about the safety of their structures during high wind events. The owner and users of the building discussed in this paper were concerned about this structure because of its function as a facility that is essential to their world-wide operations. The owner had previously targeted the building as essential to world-wide business operations and performed some upgrades in 1990 to convert the structure into a "hurricane shelter". No upgrades to the lateral system were performed at that time.

The available original building construction documents did not indicate the building code or the design wind speed values used for the design of the building, so the owner had no way of determining their vulnerability to a wind event without performing a structural analysis of the building. Initially, the structural review was performed by the owner's internal engineering group. The owner's initial review found deficiencies in the structure, so ABS Consulting was engaged to perform a detailed review of the building's lateral capacity, provide recommendations for upgrading the structure, and if required, design of a retrofit.

STRUCTURAL CONFIGURATION

The building is a 1960's era four-story reinforced concrete frame structure with a rooftop mechanical penthouse. The floor and roof diaphragms are composed of a one-way reinforced concrete joist system. The reinforced concrete joists are supported on reinforced concrete beams that span to reinforced concrete columns. The foundation consists of reinforced concrete pile caps and timber piles. The original perimeter wall was constructed of unreinforced concrete masonry units (CMU) backup with a brick veneer. The brick veneer is supported on steel shelf angles attached to the concrete spandrel beams at each floor level.

In 1990, the building was refaced with a precast concrete panel system that was intended to and believed to "hurricane proof" the building. The two-story tall precast panels were installed outside of the original building facade so that the existing brick and CMU wall remained. The precast panels were supported on new grade beams that were installed to span between existing pile caps. No upgrades to the main lateral force-resisting system of the structure were performed in conjunction with the precast panel installation.

DETERMINATION OF THE DESIGN WIND LOADS

Because the building was acquired from another owner, some of the original 1960's era construction documents were either not available for review or not legible due to improper storage. The majority of the structural data, such as member sizes, reinforcing bar quantities, and building sections, were available on the drawings. However, key information regarding the original design loads and building code used for the design was not contained in the available documentation. Therefore, a review of the structural capacity for current wind loads was required.

Like many large industrial companies with world-wide facilities, this building owner has developed their own minimum design criteria and specifications. This ensures that all of their facilities are built and maintained to the same minimum guideline no matter what country or jurisdiction the building resides.

For this project, the owner's minimum design criteria was based on ASCE 7, but the criteria required use of a higher importance factor and directionality factor than is required even for essential facilities designed per ASCE 7. Therefore, the wind load used for the evaluation of the existing structure and the design of the upgrade, were determined according to ASCE 7-05 with the owner's specific criteria of increased factors. The resulting velocity pressure for the evaluation of the structure was approximately 75% higher than the velocity pressure for a standard occupancy (Occupancy Category II) building in the same location calculated using ASCE 7-05.

EVALUATION OF THE EXISTING BUILDING

As noted above, the structural system is composed of reinforced concrete frames, which also function as lateral force-resisting elements. There are no original concrete shear walls present in the building. The exterior precast concrete panels cannot be utilized as shear walls due to the minimal connections to the structure and their inability to transfer the in-plane wind loads to and from the diaphragms.

The existing structure was evaluated to determine if the structure could resist the required design wind loads. A three-dimensional computer model was created in the program ETABS utilizing the material properties and the member sizes noted on the available structural drawings. All existing primary frame elements (beams and columns) were included in the model. The gravity and wind loads were input into the model and the ASCE 7-05 load combinations were utilized to determine the factored demands on the existing concrete frames.

The capacity of the existing concrete frames was determined using ACI 318-05. The results of the evaluation indicated that the existing concrete frames, specifically the beams, did not have enough reserve capacity to resist the evaluation wind loads. This was due to multiple factors. The structure was built during a time period when the requirements for shear reinforcement in beams were more relaxed than today's standards. According to ACI 318-63, shear reinforcement was required when Vu is greater than ΦVc, whereas ACI 318-05 requires shear reinforcement when Vu is greater than 0.5ΦVc. Therefore, when the beam shear capacity is calculated using ACI 318-05, the shear strength may limit the load carrying capacity of the beams. Additionally, changes in occupancy have resulted in increased live loads as compared to the original design. Based on the results of the structural evaluation, the structure was recommended for retrofit in order to resist the design wind loads required by the owner.

In addition to the evaluation of the main lateral force-resisting system, the exterior precast wall panels were analyzed to determine if they met the design criteria. As noted previously, the precast panels were installed to harden the building against a hurricane. The panels were specified to be contractor designed, which is typical for a precast system. Upon review of the original precast panel design calculations, it became apparent that the precast panels did not include the increased factors for wind design according to the owner's current design criteria.

The existing 7" thick precast panels are two stories tall and are connected to the structure at the top and bottom of the panels (i.e. every other floor diaphragm). Therefore, the existing wall panels were required to resist out-of-plane wind load over a span of two stories. The panels were typically reinforced with a layer of bars in the vertical span direction at each face and a layer of welded wire mesh at each face.

The precast wall panels were evaluated for the components and cladding wind load calculated using ASCE 7-05 in addition to the owner's increased factors for essential facilities. The panels were determined incapable of resisting the wind load criteria. The weak link found in the evaluation of the precast panels was related to the span length of the panels and the connections of adjacent panels. Under the increased wind load requirements of the building, the panels cannot span the full two stories.

CONCEPTUAL RETROFIT

A conceptual design study was performed to better understand the requirements of the building retrofit. Because the existing concrete frames had little reserve capacity, the addition of a new, stiffer lateral-force resisting system was desired in order to minimize the contribution of the existing concrete moment frames. Concrete shear walls and steel braced frames were considered as retrofit options. New structural elements were designed to comply with ASCE 7-05 including the owner specific wind criteria and ACI 318-05.

Three schemes were investigated for strengthening the lateral force-resisting system: 1) a combination interior braced frame and concrete shear wall system, 2) an exterior concrete shear wall system, and 3) an exterior steel buttress system. A three dimensional computer model was created for each option, which included the existing concrete frames and the new lateral force-resisting elements. Lateral force-resisting elements (braces and/or shear walls) were systematically added to the computer models until a very small percentage of the wind load was contributing to the concrete frame demands.

Each retrofit option had advantages and disadvantages that were considered by the project team. Architecturally, Option 1 was the preferred retrofit because the new lateral elements were hidden from exterior view on the inside of the building. However, Option 1 had the largest impact on the occupants and functions contained in the building, resulting in costly shutdowns and relocations during construction.

Option 2 altered the exterior of the building and eliminated some windows, but allowed for minimal impact to the building occupants. Option 3 significantly altered the exterior of the building, had increased foundation costs, and proved difficult to connect.

Given the three retrofit concepts, the owner decided to proceed with the exterior concrete shear wall system, Option 2, to minimize interruption to the building occupants and operations.

It is noted that all three options also included retrofit strengthening of the two-story precast panel elements to reduce demands on the existing out-of-plane tie-back connections and to reduce the effective span of the panels to a single story.

RETROFIT DESIGN & CONSTRUCTION

The final retrofit scheme involved the installation of new concrete shear walls in both primary directions of the building. The shear walls are significantly stiffer than the existing concrete frames, so they resist a large portion of the applied wind load, thus reducing the lateral force demands in the existing concrete frames.

The new shear walls are located along the perimeter of the building so that they could be formed and poured from outside of the occupied spaces in order to minimize impact to building operations. The existing precast concrete panels and the original brick veneer were removed at the locations of the new shear walls. This permitted the new concrete shear walls to be directly adjacent to the existing perimeter spandrel beams which allowed the walls to be easily doweled into the existing diaphragms for in-plane and out-of-plane shear transfer (see Figure 1). It also allowed the new concrete shear walls to be supported on the existing grade beams that previously supported the precast panels without increasing the dead load on the foundation. In fact, the dead load was slightly reduced due to the removal of the precast panel and brick, and replacement with a similar thickness concrete wall.

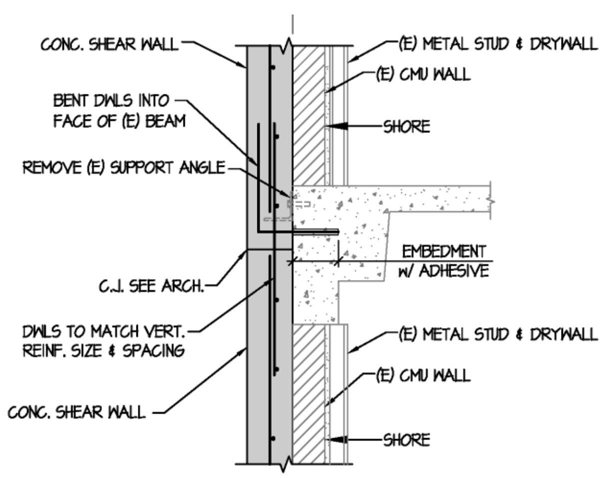

Figure 1. New Concrete Shear Wall Attachment to Existing Concrete Spandrel Beam

At the locations of the new concrete shear walls, the existing pile foundations were retrofitted with micropiles and enlarged reinforced concrete pile caps to resist the overturning induced on the foundations from the wind load. The pilecaps were enlarged and thickened to ensure that the existing reinforcing steel could provide the proper flexural resistance. The new micropiles and the existing timber piles were designed to share the design loads. Micropiles were chosen due to their ability to be installed in areas with low headroom. Additionally, the building contains equipment that is sensitive to vibration. Although some amount of vibration is anticipated on any construction site, micropiles can typically be installed at locations sensitive to vibrations.

In addition to the retrofit of the main lateral force-resisting system, the exterior precast wall panels required mitigation. In order for the existing panels to meet the same design requirements as the main lateral force-resisting system, the exterior precast wall panels were modified to reduce their span by providing connections at the intermediate floor levels. This effectively reduced the out-of-plane span of the panels allowing them to resist the higher design wind loads. The new intermediate connections were installed from the outside of the building in order to minimize the impact on the building occupants and operations. The connections were composed of a threaded rod that was doweled through the existing precast panel, air gap, and brick veneer, and anchored to the concrete spandrel beam beyond with adhesive. A steel plate was installed on the outside face of the panel to allow tension resulting from outward pressure to be transferred into the threaded rod. The plate was anchored to the precast panel with adhesive anchors to allow the transfer of compressive loads resulting from inward pressure (see Figure 2).

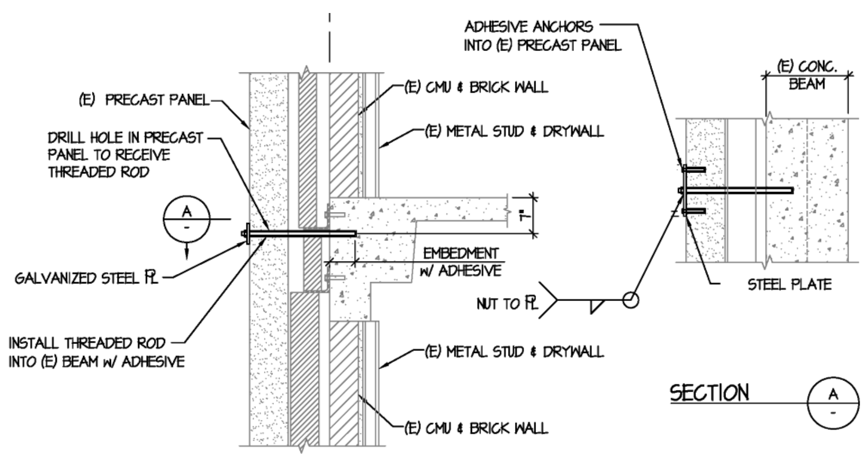

Figure 2. Precast Panel Connection to Existing Concrete Spandrel Beam

Looking back at the design of the retrofit, the concept is simple and straight forward. However, the design team was faced with multiple challenges during the design and construction of the retrofit. Some of these challenges such as time constraint, specification of architectural components, construction techniques, and scope creep are discussed below.

TIME CONSTRAINT

Once the retrofit concept was chosen and the decision was made to proceed with design and construction, the owner began setting aggressive goals for the overall project timeline. The owner was clear from the onset of the project that they wanted final construction completed prior to the onset of the upcoming hurricane season, less than seven months away. This schedule was very aggressive considering the complete scope of the renovation project was not finalized and the design team was not under contract. To assist in speeding up the project, the owner proactively selected a contractor that they had worked with on other projects and were comfortable with to participate in the project from the beginning of design throughout construction rather than putting the project out to bid.

In order to achieve the owner's goal, the design team worked without a formal contract in place for several weeks. Additionally, the design was divided into phases in order to allow the contractor to begin working as soon as possible. The first phase was issued for construction five weeks after formal authorization to proceed and the last phase was issued 16 weeks after notice to proceed.

Although a compressed schedule may seem an innocent request, it can be a significant risk undertaken by the design team. Compressed schedules eliminate the ability for the design team to fully review and vet various design solutions. It is important that quality control measures be in place. When time is tight, there is a temptation to skip the internal reviews and other QA/QC measures even though these reviews are most needed for such high pressure, short schedule projects. Eliminating important internal milestone reviews can lead to overly conservative designs or costly mistakes due to over simplification in order to finalize design. Additional design team staffing and resources were utilized on this project to maintain high quality and still meet the owner's time constraints.

SPECIFICATION AND REVIEW OF ARCHITECTURAL COMPONENTS

The most common failures observed following a hurricane for non-residential, engineered structures is the loss of exterior architectural systems (walls and roofs), damage caused by inadequately anchored exterior equipment mounted on or in close proximity to buildings, and building envelope damage caused by wind-borne debris. For non-essential buildings these architectural components (cladding systems, windows, roofing materials, rooftop equipment, etc) are not typically specified by the structural engineer. However, for an essential structure, the building envelope is expected to be maintained during a design load event and the performance is expected to be equivalent to the structure.

In order to maintain operations during and after a severe wind event, architectural elements such as the roofing system, the windows, the rooftop hatch and the louvers were required to meet the components and cladding requirements of the building code. Additionally, the windows were required to be wind and impact resistant in accordance with ASTM E1886 and E1996. These architectural items were specified using a performance based specification approach. The various suppliers and contractors were required to submit test data and certified calculations showing that the systems were capable of meeting the owner's specified design criteria. A significant amount of additional time and effort was spent by the design team during the submittal review process due to the inexperience of the contractors and architectural building component manufacturers dealing with the higher wind design requirements of the project than was expected.

CONSTRUCTION TECHNIQUES

One of the most interesting and challenging parts of the project related to the occupancy of the structure during construction. This facility houses functions that are essential to the everyday operations of the company. Not only did the building need to remain occupied during and following a strong wind event, it needed to remain occupied during construction.

As noted previously, exterior reinforced concrete shear walls were the chosen retrofit method. This option was chosen in order to minimize the impact to the building occupants and operations during the structural retrofit. As noted previously, only a portion of the existing facade was removed in order to allow for direct attachment of the new shear walls to the building structural system. The CMU layer remained in place during and after construction. This allowed the building to remain enclosed during construction without construction of temporary enclosures or waterproofing or even relocating individuals and/or equipment located directly behind the new shear wall locations. The outside face of the CMU layer aligned with the outside face of the concrete spandrel beams and spanned vertically from the floor to the underside of the concrete spandrel beams. The contractor utilized the CMU layer as the interior form for the new concrete shear walls and monitored movement of the wall until the wet concrete was set up to assure that the unreinforced CMU "form" maintained its integrity. This allowed for minimal disruption to the building occupants.

The placement of micropiles on the interior of the building at the first floor also required specialized construction techniques due to low overhead clearance and the need to keep exhaust fumes outside of the operating facility. A small-sized electric excavator was located to assist in the demolition and excavation for the new pile caps. This unit was able to fit through existing double doors and eliminated the concern of exhaust fumes. The micropile subcontractor also was able to utilize a small drilling rig that could place the piles in the low overhead conditions.

SCOPE CREEP

The fast pace of the project led to a constantly evolving project. The scope of work and fee development was based on preliminary discussions with the owner and architect. This project was unique because the structural requirements were primary to the architectural features, which played a secondary role. The architect had been