THIS BRINGS US TO THE NEXT CONCERN, THE SHEAR CAPACITY OF THE SIDEWALLS. BECAUSE OF THE OPENINGS CREATED BY THE OVERHEAD DOORS AND BECAUSE THE BEAMS BETWEEN COLUMNS SEPERATE THE WALLS HORIZONTALLY INTO UNCONNECTED SECTIONS, THE SHEAR CAPACITY OF THE SIDEWALLS IS PROVIDED PRIMARILY BY THE WALLS OF THE OFFICE AREA.

HOWEVER, BECAUSE OF THE 4 INCH GAP BETWEEN THE 8 INCH BLOCK ABOVE THE SECOND FLOOR AND THE SIDEWALL COLUMNS, (AS DETAILED ON SHEET A5, DETAIL 3), THE WIRE ANCHORS WILL BE SUBJECT TO EXCESSIVE BENDING STRESS OR THEY WILL BE PULLED OUT OF THE WALL UNDER DESIGN WIND.

THEREFORE, A STRONGER ANCHORAGE MUST BE USED AT ~~[illegible]~~ OR BETWEEN COLUMNS 5, 6 & 7. THE BEST OPTION WOULD BE TO USE 12 INCH BLOCK AT THE TOP OF THE WALL IN ORDER TO ALLOW THE EAVE STRUT TO BE BOLTED INTO THIS COURSE; SEE DETAILS. (IN ANY EVENT, SEE PAGE 13 CONCERNING RECOMMENDATIONS REGARDING THE 8 INCH BLOCK.) ANOTHER OPTION WOULD BE TO USE 12 INCH BLOCK

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IN FRONT OF THE COLUMNS TO CUT DOWN ON THE EXPOSED LENGTH OF ANCHOR AND TO INCREASE EMBEDMENT LENGTH. A POTENTIAL PROBLEM WITH THIS IS THAT THE ANCHORS WILL BE CLOSE TO WHAT IS ESSENTIALLY A FREE EDGE, i.e. THE CONTROL JOINTS. ANCHORS SHOULD BE USED AT EVERY COURSE FOR THE TOP 6 FEET WITH THIS OPTION.

BAR ~~[illegible]~~ ANCHORS WOULD ELIMINATE THE BENDING PROBLEM, BUT THE WALL ANCHORAGE MIGHT STILL BE A PROBLEM BECAUSE OF THE MOMENT IF 8 INCH BLOCK REMAINS.

NOTE THAT THE PIERS BETWEEN OVERHEAD DOORS WILL EXPERIENCE SOME SHEAR AS THE JAMBS, (WHICH ARE CONNECTED TO THE BEAMS), BEAR AGAINST OR ATTEMPT TO PULL FROM THE PIERS. THE VOID BETWEEN THE PIERS AND THE BACK FACE OF THE JAMBS SHOULD BE SOLID GROUTED.

ANOTHER AREA OF CONCERN, (NOT RELATED TO FRAME DEFLECTION), IS THE BLOCK ABOVE THE 2ND FLOOR OF THE OFFICE AREA. THE 8 INCH BLOCK, AS DETAILED, MAY NOT BE ADEQUATE TO RESIST THE DESIGN WIND LOAD. AT THE VERY LEAST, THERE IS AN INCONSISTENCY IN USING 8 INCH BLOCK HERE WHILE USING 12 INCH BLOCK FOR THE SAME SPAN IN OTHER BAYS OF THE BUILDING.

AT PRESENT, THE 8 INCH BLOCK WALL IS SUPPORTED LATERALLY BY THE SIDEWALL COLUMNS AND THE 2ND FLOOR. HOWEVER, THE INFLUENCE OF THE FLOOR SUPPORT DIMINISHES SUCH THAT THE BLOCK ABOVE THE WINDOWS IS ESSENTIALLY SUPPORTED ONLY BY THE COLUMNS, SPANNING 20 FEET BETWEEN THEM.

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ALTHOUGH OLDER, EMPIRICAL DESIGN STANDARDS MAY ALLOW A LENGTH TO THICKNESS RATIO OF 36, MORE RECENT STANDARDS RECOMMEND THAT WALL THICKNESS BE BASED ON ENGINEERED DESIGN, TAKING INTO ACCOUNT MATERIAL PROPERTIES AND LOAD MAGNITUDES. AS AN EXAMPLE, RECOMMENDATIONS PUBLISHED BY THE NATIONAL CONCRETE MASONRY ASSOCIATION SUGGEST THAT AN 8 INCH, HOLLOW, UNREINFORCED WALL SHOULD SPAN NO MORE THAN 10.6 FEET HORIZONTALLY WHEN CONSTRUCTED WITH TYPE N MORTAR AND SUBJECTED TO A WIND LOAD OF 20 PSF. WHILE THIS IS CONSERVATIVE, (ALONG WITH THE WIND LOAD), IT IS SIGNIFICANTLY LESS THAN 20 FEET. THE JOINT REINFORCEMENT CALLED FOR AT 16 INCH SPACING WILL ONLY INCREASE THE WALL CAPACITY MARGINALLY, ESPECIALLY SINCE TYPE N MORTAR IS SPECIFIED.

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IF 12 INCH BLOCK CAN NOT BE USED, THEN RECOMMENDATIONS ARE AS FOLLOWS FOR THE 8 INCH WALL:

1. USE TYPE S MORTAR

2. USE JOINT REINFORCING AT EVERY COURSE.

3. BOND BEAMS ABOVE AND BELOW THE WINDOWS SINCE THESE COURSES WILL RECEIVE ADDITIONAL LOAD DISTRIBUTED TO THEM FROM THE WINDOW SECTION.

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SUMMARY

THE ORIGINAL BUILDING DESIGN DID NOT CONSIDER THE MOVEMENTS OF THE STEEL FRAMES AS THEY AFFECTED THE MASONRY WALLS OR THE LOADS TO BE TRANSMITTED BETWEEN THE TWO. A STRUCTURAL EVALUATION OF THE ENTIRE BUILDING DESIGN, NOT JUST SECONDARY COMPONENTS OF THE BUILDING, SHOULD HAVE BEEN REQUESTED IN THE EARLY DESIGN STAGE.

WHILE THE RECOMMENDATIONS IN THIS REPORT SEEM TO RADICALLY ALTER THE INTENDED DESIGN, i.e. MASONRY AS CURTAIN WALL, IN FACT THE MASONRY ~~[illegible]~~ WILL RECEIVE SUBSTANTIAL LOAD WITH THE ORIGINAL DESIGN. HOWEVER, THE MASONRY COULD EXPERIENCE CRACKING AND FAILURE UNLESS CORRECTIVE MEASURES ARE TAKEN AS DETAILED IN THIS REPORT.

THIS BUILDING COULD HAVE BEEN DESIGNED WITHOUT THE STEEL FRAMES AND WITH ONLY A SMALL INCREASE IN MASONRY COSTS. WITH THE PRESENT CONSTRUCTION, THE FRAMES WILL BE UNDERUTILIZED TO A LARGE DEGREE WHILE THE MASONRY WILL SUPPORT MUCH OF BOTH LATERAL AND VERTICAL LOAD.

ANCHOR NOTE

TO ELIMINATE LOAD TRANSMISSION FROM THE STEEL FRAME TO THE MASONRY WALL, PROVISION WOULD BE REQUIRED TO ALLOW FRAME MOVEMENT RELATIVE TO THE WALL. HOWEVER, IT WOULD BE DIFFICULT TO DESIGN A CONNECTION CAPABLE OF ALLOWING THE AMOUNT OF MOVEMENT REPORTED TO US BY AMERICAN BUILDINGS; SEE LETTER LISTING PORTAL FRAME DEFLECTIONS. ALSO, THE ENDWALL COLUMNS WOULD EXERT FORCES ON THE ENDWALLS, AS THEY FOLLOWED THE END FRAMES, WHICH WOULD OVERSTRESS THESE WALLS. NOTE THAT "DIAPHRAGM ACTION" IS NOT AVAILABLE TO REDUCE THESE DEFLECTIONS SINCE THERE ARE ONLY THE TWO SUPPORTS.

THE WIRE ANCHORS AT ALL COLUMNS OF THE BUILDING SHOULD BE SIZED TO FIT THE THICKNESS OF COLUMN FLANGE IN ORDER TO ELIMINATE MOVEMENT TRANSVERSE TO THE PLANE OF THE WALL. THE HOOK GAP SHOULD BE ORDERED TO THE SAME THICKNESS AS THE FLANGE TO OBTAIN A FRICTION FIT. THE LENGTH

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OF ANCHOR EMBEDDED IN THE WALL SHOULD BE A MINIMUM OF 6 INCHES. LENGTH OF HOOK SHOULD BE KEPT TO A MINIMUM IN ORDER TO KEEP THE EDGE DISTANCE, OF THE EMBEDDED PORTION FROM THE CONTROL JOINT, A MAXIMUM; SEE DETAIL.

AA WIRE SAYS THAT THE WIRE ANCHORS CAN EASILY BE ORDERED TO THE DIMENSIONS REQUIRED. THE CONTRACTOR HAS ALREADY BEEN NOTIFIED.

BLOCK CORES ABOVE AND BELOW ANCHORS MUST BE GROUTED SOLID IN ORDER TO PROVIDE ADEQUATE EMBEDMENT FOR THE ANCHORS.

ANCHOR BOLT NOTE

SINCE THE ROOF DECKING WILL BE INSTALLED AFTER THE PURLINS ARE CONNECTED TO THE WALL, SOME OF THE DEAD LOAD WILL ALSO BE SUPPORTED BY THE WALL. THIS MAY ~~[illegible]~~ TEND TO INCREASE ANCHOR BOLT UPLIFT, ALTHOUGH NOT SIGNIFICANTLY.

NOTE 1: THE ORIGINAL AMERICAN BUILDINGS, (ABC), FRAMES WERE DESIGNED TO BE MUCH MORE FLEXIBLE THAN THEY ARE NOW. AFTER WE RECOMMENDED THAT THEY SHOULD NOT BE ALLOWED TO USE THE SIDEWALLS TO BRACE THEIR COLUMNS, (AGAINST LATERAL BUCKLING), THEY REDESIGNED THEIR COLUMNS; SEE MEMO TO YOU OF 10-23-85. THE INTENT OF OUR RECOMMENDATION WAS TO ELIMINATE BRACING LOAD AGAINST THE PIERS BETWEEN DOORS, SOME OF WHICH ARE NARROW, PRIMARILY BECAUSE WE WERE AWARE OF YOUR DESIRE TO ELIMINATE LOAD, (OTHER THAN DIRECT WIND LOAD), FROM THE "CURTAIN" WALL. WE DID NOT TAKE A BROAD VIEW OF THE BUILDING AT THAT TIME BECAUSE WE HAD NOT BEEN ASKED TO EVALUATE THE OVERALL DESIGN SCHEME WHICH YOU HAD COMPLETED LONG BEFORE.

AS A RESULT OF ABC'S REDESIGN, THE FRAMES WERE MUCH STIFFER. THUS, THE RIGID FRAME LATERAL DEFLECTIONS REPORTED TO US, (SEE LETTER OF 3-17), ARE MUCH LESS THAN THEY WERE WITH THE ORIGINAL ABC DESIGN. HOWEVER, THEY DID NOT SPECIFICALLY INTEND TO LIMIT THE FRAME DEFLECTIONS; THIS WAS ONLY INCIDENTAL TO THEIR REDESIGN. NOWHERE IN OUR SPECIFICATIONS OR PLANS IS ANY DEFLECTION CRITERIA GIVEN. THOSE RESPONSIBLE FOR MASONRY WALL DESIGN SHOULD CONSIDER SPECIFYING LATERAL DEFLECTION CRITERIA IN THE FUTURE FOR BUILDINGS OF THIS TYPE.

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NOTE 2: ACCORDING TO THE RECENTLY, (1986), UPDATED MANUAL PUBLISHED BY THE METAL BUILDING MANUFACTURERS ASSOC., A STEEL ROOF DOES ACT AS A DIAPHRAGM, DISTRIBUTING LOAD TO STIFFER FRAMES AND WALLS. A CHART SHOWS THAT LATERAL DEFLECTIONS CAN BE SIGNIFICANTLY LESS THAN THOSE OBTAINED ASSUMING ONLY THE FRAMES RESIST THE LOAD.

NOTE 3: SUCH A CONNECTION MIGHT INVOLVE CONNECTING THE UPPER FRAME RAFTER TO THE TOP OF THE WALL WITH STEEL SHAPES BOLTED TO BOTH THE RAFTER AND WALL. THIS CONNECTION WOULD UTILIZE SLOTTED HOLES TO ALLOW VERTICAL DEFLECTION OF THE RAFTER WITHOUT TRANSMITTING AN ECCENTRIC VERTICAL LOAD TO THE WALL WHICH WOULD CAUSE EXCESSIVE TENSILE STRESSES IN THE WALL. THE STRENGTH PARALLEL TO THE WALL WOULD BE MUCH MORE SECURE; BETTER THAN THE ANCHORED CONNECTION TO THE STUB COLUMNS WHICH MIGHT HAVE TO BE BRACED AS WELL. THE TOP OF THE WALL WOULD ALSO BE PROVIDED NECESSARY LATERAL SUPPORT THAT IS NOT ~~[illegible]~~ ADEQUATELY PROVIDED BY THE STUB COLUMNS.

AT THE ENDWALLS, THE ENDWALL COLUMNS WOULD MOST LIKELY REQUIRE BRACING AGAINST TWIST.