# Problems of seismic design of the cladding panels of precast buildings

A. Colombo

*Assobeton, Milan, Italy*

G. Toniolo

*Politecnico di Milano, Italy*

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**ABSTRACT:** Following the lesson learned from L'Aquila earthquake 2009, the paper presents a systematic definition of the design criteria for the connections of cladding panels of precast buildings, examining the behaviour of the whole system made of structure and panels, and proposes, for the typology of one-storey buildings with industrial destination, some possible alternative solutions that ensure the stability of all the construction elements.

To quantify the design parameters involved in the quoted solutions, some calculations of a typical precast building under seismic conditions are elaborated. The analysis is repeated to compare the different connection systems of the panels to the structure, that is the isostatic solution that allows the structural frame to move independently from the walls, and the collaborating solutions that integrate the wall panel to the resisting structure.

These information are intended to give the base data for a new generation of connection devices compatible with the wall stability under seismic action.

## 1 INTRODUCTION

Recordings made during L'Aquila earthquake of 2009 show in different zones horizontal peak ground accelerations between 0.35g and 0.45g (0.60 near fault) that correspond to the highest seismicity class of the Italian territory following the national code for structural design (Menegotto 2009). This strong earthquake concerned a territory with several hundreds of precast constructions, mostly industrial one storey buildings and in a minor number commercial buildings of two or three storeys.

Figure 1: Collapse of cladding panels in Ocre

Figure 2: Detail of fastening failure

From this event an important lesson could be learned. Figure 1 shows an emblematic picture of the earthquake effects: a building just finished of which the earthquake left the structure (columns, beams, roof elements) substantially intact, while a wall of vertical panels collapsed entirely. The fastenings of the panels failed as shown in Figure 2. And this can be generalised for all the territory of L'Aquila, where only two collapses of single elements on finished precast concrete structures

occurred, while about the 15% of cladding panels, both vertical and horizontal (Fig. 3), fell down because of fastening failure.

In particular, for what concerns Figure 1 the collapsed wall is placed in the direction NW-SE of the strongest acceleration component. The fastenings, made of channel bars, that is C shaped steel profiles embedded in the concrete frame of the panels, have been forced in the tangent transverse direction for which they are not designed. The anchorage head of the fastener plates worked as a lever pulling out the lips of the channel bars (see Fig. 2). In the orthogonal walls the force, normal to the connection plane and related to the local mass of the panels and not to the whole mass of the roof, found a sufficient resistance.

Cast-in channel bars are the most common fastening devices used in wall panels. Anyway failures didn't regard only them, but also other types of fastenings (Fig. 3). It has been not a question of a product inadequacy, but of an inadequate design of the connection. In fact the design followed the specific rules of the code in force, quantifying the forces with a local analysis on the basis of the mass of the single panel considered as a non structural element. Actually the panels received much higher forces and in the direction not considered in design. This was because their fixed connection to the structure made them integral part of the resisting structural system, taking it to a much higher stiffness with a lower ductility. The design approach itself, as proposed by the codes, shall be improved in its principles and in the following clauses a systematic new approach with new adequate specific solutions is presented to give a useful contribution to the seismic design of the connection systems of cladding panels of precast concrete structures.

Figure 3: Detachment of a horizontal panel in Monticchio

Figure 4: distorted steel bracket and anchor plate

The experience of L'Aquila earthquake for what concerns precast concrete constructions has no direct relation with the experience of Canterbury earthquakes. The former one refers exclusively to industrial one storey and commercial low-rise buildings, the latter refers to residential multi-storey buildings with different connection technologies.

## 2 DESIGN CRITERIA

The present design practice of the precast concrete structures of concern is based on a bare frame model where the peripheral cladding panels enter only as masses without any stiffness. The panels are then connected to the structure with fixed fastenings dimensioned with a local calculation on the basis of their mass for anchorage forces orthogonal to the plane of the panels.

This approach doesn't work: L'Aquila earthquake has demonstrated it. The panels, fixed in this way to the structure, come to be integral part of the resisting system conditioning its seismic response as of a dual wall-frame system of lower energy dissipation capacity. The high stiffness of this resisting system leads to much higher forces than those calculated from the frame model. This forces are related to the global mass of the floors and are primarily directed in the plane of the walls. The unforseen intensity and direction of the forces drove many fastenings to failure, leaving the frame of columns and beams practically undamaged.

From these observations three possible solutions for the cladding system can be presented, as described in their principles in the following text. A first solution, here called “isostatic”, is based on wall panels connected to the structure with supports that allow the free development of the large displacements expected for the frame structure under seismic action. This solution allows to follow the traditional design approach of the analysis applied to a bare frame model.

A second solution, here called “integrated”, is based on wall panels connected to the structure with an iperstatic arrangement of fixed supports that make them part of the overall resisting system. This solution requires a design approach completely new with an analysis applied to a dual frame-wall model that involves unexplored participations of the roof diaphragms. Both these solutions require a specific design of new connectors: able to allow the large free displacements of the isostatic solution; able to transfer the high forces expected in the integrated solution.

## 2.1 Isostatic wall system

Different panel-to-structure connection systems can ensure an isostatic arrangement of the panels. Examples of some of these systems are presented below with reference both to vertical and horizontal panels, pointing out some problem of practical application.

Figure 5 represents a vertical panel referred to a system of orthogonal axis, where *x* is oriented horizontally in the panel plane, *y* is oriented orthogonally to that plane and *z* is oriented vertically parallel to the gravity force. The origin is placed in a corner at the base side of the panel. Four connections are foreseen at the corners of the panel, indicated respectively by *A, B, C* and *D*. These connections are intended to give only translatory supports without any rotational support. With *E* and *F* are indicated the possible joint connections with the adjacent panels. Usually the connections *A* and *B* are attached to the foundation beam, the connections *C* and *D* are attached to the top beam.

The same reference system is associated in Figure 6 to a horizontal panel, for which usually the connections *A, B, C* and *D* are attached to the columns and *E* and *F* refer to the possible joint connections with the adjacent panels, where the uncertain friction effect due to the weight of the superimposed panels may act. In Tables 1 and 2 the effect of the supports along the three directions *x, y* and *z* is indicated, respectively for the vertical and the horizontal panel, , where the symbols are:

In the plane of the vertical panel (in *x* direction) the connection system defined in Table 1 ensures horizontal displacements of the structural frame independent from the panel, that remains supported at the base on the foundation beam, providing by itself for its stability. In the orthogonal *y* direction the panel remains simply supported at its upper and lower ends, following without reactions the vibratory motion of the structure, to which it transfers the inertia force due to part of its mass.

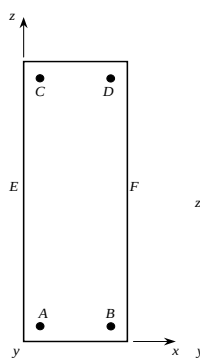


Figure 5: Scheme of vertical panel

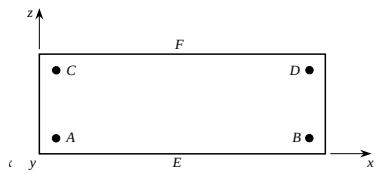


Figure 6: Scheme of horizontal panel

Choosing for the indifferent supports the simpler type of connection, that is the fixed one, the system requires two types of connectors, one with full support and one with a partial support that allows one of the three translations. And this in expectation of large displacements even up to ±15 cm.

| | A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| x | f | i | s | s | 0 | 0 |
| y | f | f | f | f | 0 | 0 |
| z | f | f | I | i | 0 | 0 |

Table 1: Supports for the vertical panels

| | A | B | C | D | E | F |
|---|---|---|---|---|---|---|
| x | f | i | s | s | s | s |
| y | f | f | f | f | 0 | 0 |
| z | f | f | I | i | 0 | 0 |

Table 2: Supports for the horizontal panels

- f = fixed
- s = sliding
- i = indifferent
- 0 = absent.

In the plane of the horizontal panel (in *x* direction) the connection system defined in Table 2 binds the displacements of the panels to those of the joints with the columns on which the inertia forces due to their mass are transferred. But, for a free motion without reactions, it is necessary that no relevant friction arises at the joint with the superimposed panels and this requires the interposition of a free space or of adequate unusual deformable sealings. Due to the friction it is therefore more difficult with the horizontal panels to make an isostatic system that doesn't react with the structure.

In *y* direction the horizontal panels follow the vibratory motion of the columns without reactions, transferring to the four joints the inertia forces due to their mass.

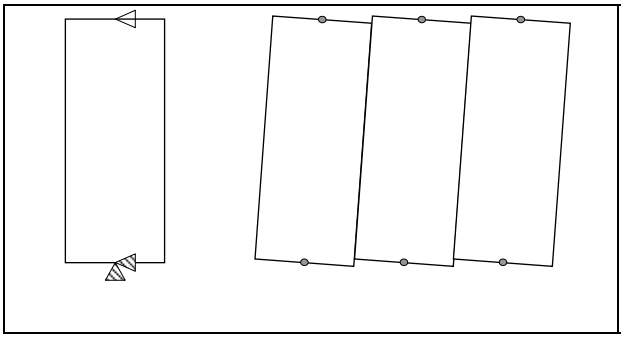

Figure 7: Isostatic pendulum system of connections

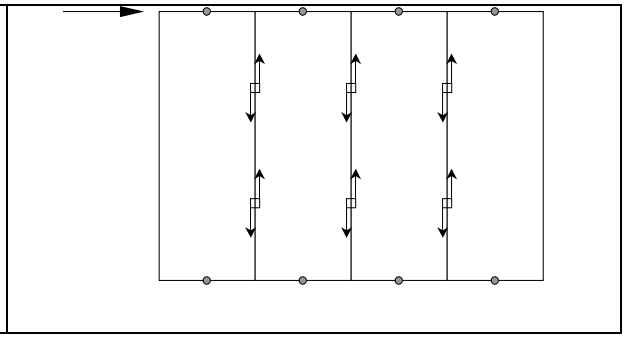

Figure 8: Iperstatic system with interconnected panels

Figure 7 shows an other possible solution with an isostatic pendulum arrangement of vertical panels, connected at the base and top with connections placed in the middle of the horizontal sides. With this arrangement the cladding system can freely follow the motion of the roof, displaying relative slidings *db/h* at the joints of adjacent panels, with *d* top horizontal displacement, *b* panel width and *h* height of the upper connection. Only fixed connections are required.

## 2.2 Integrated wall system

The same pendulum support system can be transformed into an integrated system if the *E* and *F* connections are added to prevent the relative slidings of the panels on the joint sides (Fig. 10). For these joint connections particular devices have been designed and tested to dissipate energy under violent earthquake (Iqbal et al. 2007, Shultz et al. 1994, Biondini et al. 2011).

The ordinary arrangement of the connections placed in the four corners of the panels, that make them collaborate with the structure, is based on full traslatory supports and the columns *A, B, C,* and *D* of Tables 1 and 2 become all filled with *f*. For vertical panels this arrangement is represented in Figure 9, where the distortion effects on the top beam can be noticed. A similar solution can be used for horizontal panels, with four connections as described in Figure 10 leading to distortion effects on the columns.

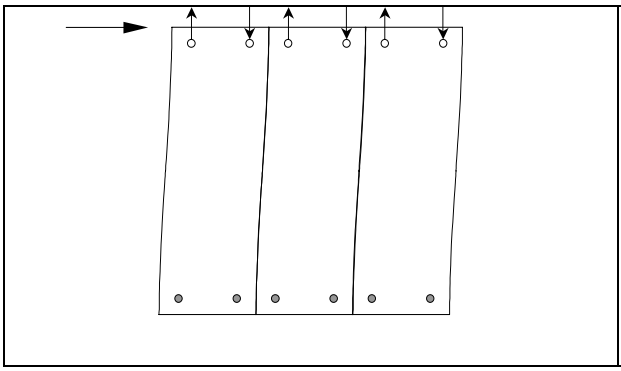

Figure 9: Iperstatic connections for vertical panels

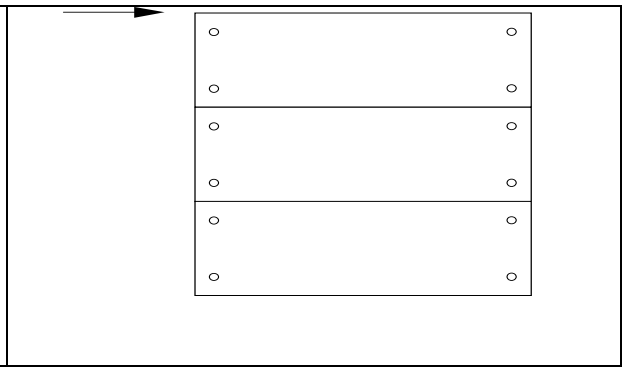

Figure 10: Iperstatic connections for horizontal panels

The technological solution of fully fixed connections is simplified, but the integrated participation of the panels, with their stiffness, to the vibratory motion of the structural system leads to very high forces on the connections. With respect to the present production, new products with improved capacities are needed. And not only the calculation of the connections is to be updated, but also the calculation of the roof diaphragm through which the inertia forces shall be transmitted to the resisting lateral walls.

## 3 ANALYSIS OF A TYPE BUILDING

The magnitude order of the displacements and forces which are demanded respectively by the isostatic and the integrated solutions described above is deduced here below. As a type building to be examined, the one-storey precast industrial building, 50x41 m of sides, described in Figures 11, 12 and 13 has been chosen. The columns have a square section of 60 cm of side with 5,2 m of height. Vertical wall panels of 2,5 m of width are placed along the perimeter to complete the building.

In the first case this building is assumed to be complete by itself with cladding panels on the four perimeter sides for a doubly symmetrical arrangement. In the second case it is intended to be one of the two parts of a double long building divided by a seismic joint and in this case the wall panels cover only three sides with an unsymmetrical arrangement in one direction.

The analyses are applied to a three-dimensional model that reproduces closely the geometry of the structure together with the distribution of the masses and of the elastic stiffness of the construction elements, including the in-plane stiffness of the roof elements. The arrangement of supports consists of full fixed supports for the column-to-foundation connections, hinged beam-to-column connections in the horizontal and vertical planes of the beam, fixed beam-to-column connections in the orthogonal vertical plane, spherical hinges between the beams and the two ribs of the roof elements.

In the calculation model the wall panels are represented by plate elements connected to the foundation by two spherical hinges, while at the top they are connected to the beam by two joint elements the stiffness of which is assumed each time very low or very high in order to simulate the two quoted solutions of isostatic and integrated connections. These arrangements are those described in Figures 14 and 15 respectively for the symmetrical and unsymmetrical wall arrangement.

The masses involved in the vibratory structural response are those of the seismic action combination with only permanent loads at their characteristic or nominal values (with $\gamma_G$=1,0). These loads are evaluated automatically by the computation code: indicatively there are 4,0 kN/m$^2$ distributed on the roof and 3,0 kN/m$^2$ distributed on the peripheral cladding walls. The seismic action corresponds to a reference peak ground acceleration $a_g$=0,25g, with the response spectrum of Eurocode 8 (EN 1998-1:2004) for a soil Category *B* (with S=1,2). For the behaviour factor (force reducing factor) the value q=3,5 has been assumed. Dynamic modal analyses have been performed by means of a computation code for the definition of the principal vibration modes, evaluating subsequently forces and displacements separately for longitudinal action (in *x* direction) and transversal action (in *y* direction).

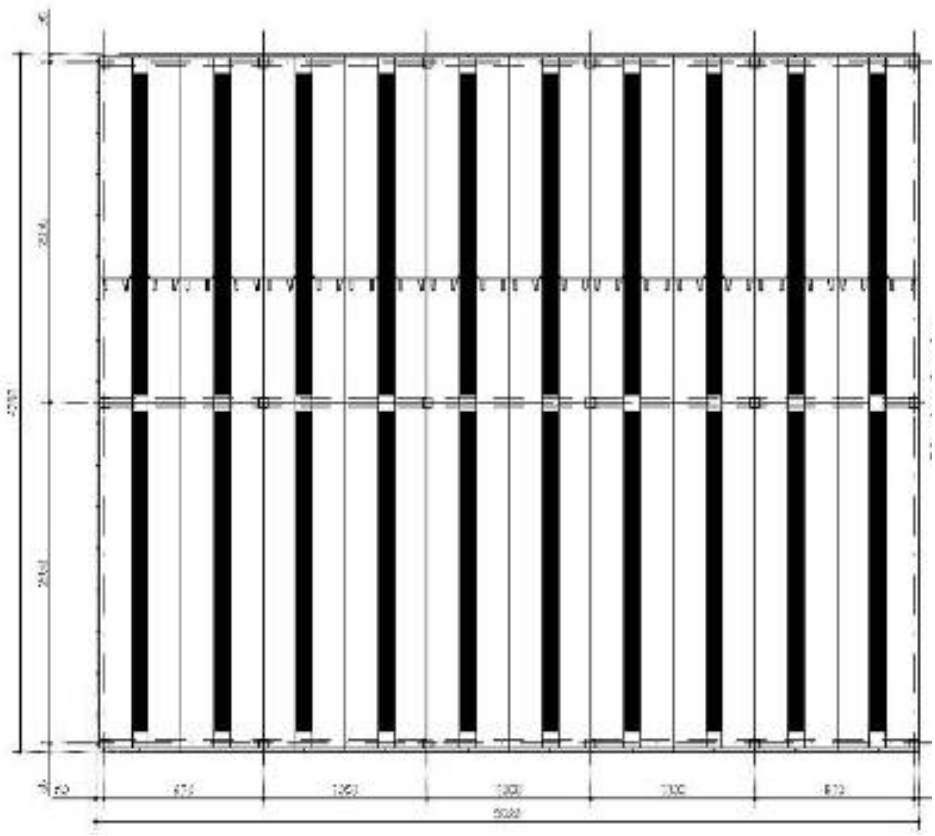

Figure 11: Plan of the type building

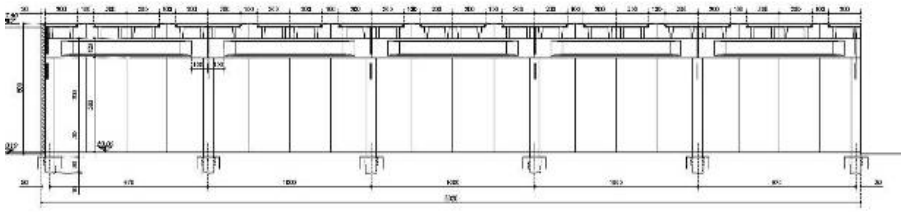

Figure 12: Longitudinal section

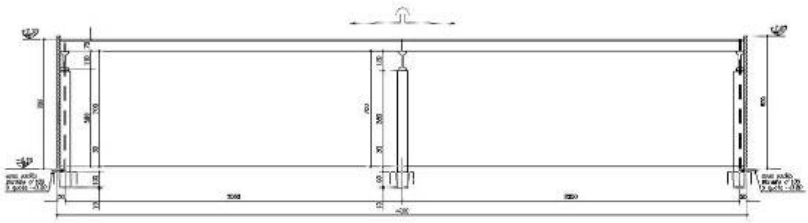

Figure 13: Transverse section

For the symmetrical wall arrangement the first two modes correspond to pure translations along *y* and *x*, with natural vibration periods for the isostatic connection system respectively of 1.03 s and 0,80 s (that are common of that type of frame structure). A third significant vibration mode corresponds to a torsional rotation with 0.74 s of natural vibration period. For the integrated connection system these periods become much lower (0.65 s, 0.42 s and 0,41 s) because of the much higher stiffness of the dual frame-wall structural system.

For the unsymmetrical wall arrangement with isostatic connection system the first three modes remain practically the same as for the symmetrical arrangement since the frame structure, disconnected from the walls, remains symmetrical. The different mass distribution modifies a little the natural vibration periods taken to 1,03 s, 0.79 s and 0.74 s. . For the integrated system the wall dissymmetry modifies

substantially the response, with a first translational-rotational mode along *y* that keeps a high natural vibration period of 0,86 s because of the absence on one side of the stiffening wall, a torsional mode that becomes the second with a low natural vibration period of 0,46 s and a symmetrical translational mode along *x* with 0,41 s of natural vibration period.

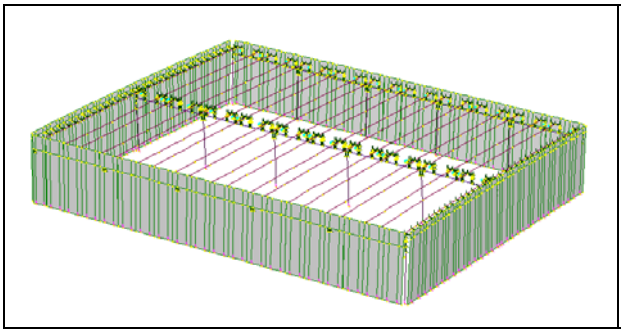

Figure 14: Symmetrical arrangement

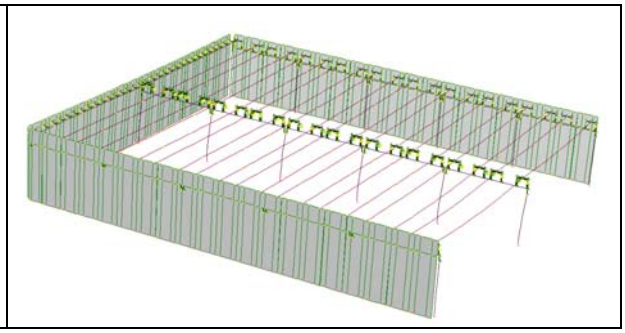

Figure 15: Unsymmetrical arrangement

For connection design purposes one shall consider the maximum values of the behaviour parameters of the response that are for the isostatic support system the relative horizontal displacements between panels and structure at their connections, for the integrated support system the horizontal components of the forces transmitted between panels and structure at their connections. The analysed structure can be assumed as representative of the typology so that the cited maximum values can be taken as indicative of the magnitude order of the parameters.

In Table 3 these parameters are given together with some other data of the overall behaviour of the structures. With *max slide* and *max force* the displacement and force in the connection quoted above are indicated. With *max drift* the maximum top displacement of the structure is indicated and with *max shear* and *max mom.* the components of the internal force at the column base are indicated.

| | Symmetrical arrangment | | | | Unsymmetrucal arrangment | | | |
|---|---|---|---|---|---|---|---|---|
| | Along *x* | | Along *y* | | Along *x* | | Along *y* | |
| | isostatic | integrated | isostatic | integrated | isostatic | integrated | isostatic | integrated |
| Max slide (mm) | 56 | - | 70 | - | 55 | - | 70 | |
| Max force (kN) | - | 104 | - | 83 | - | 100 | - | 73 |
| Max drift (mm) | 59 | 31 | 80 | 59 | 67 | 35 | 73 | 49 |
| Max shear (kN) | 62 | 31 | 47 | 38 | 62 | 31 | 47 | 44 |
| Max mom. (kNm) | 362 | 191 | 331 | 256 | 358 | 189 | 331 | 306 |

Table 3: maximum values of the behaviour parameters

It is to remember that the calculation of the displacements and forces has been performed through a modal dynamic analysis with linear elastic behaviour of the elements and with a design response spectrum reduced by a factor q=3,5 for a peak ground acceleration equal to $a_g$=1,2x0,25g=0,30g corresponding to the no-collapse limit state. The computed values of the displacements have been therefore amplified by the same factor q=3,5 to account for the elastic-plastic non linear behaviour of the structure.

As can be seen in Table 3, the maximum relative displacement between panels and structure for the isostatic support arrangement is ±7,0 cm. The maximum force transmitted between panels and structure for the integrated arrangement is ±104 kN.

For the symmetrical structure in the longitudinal direction the integrated support system practically halves the internal forces at the column base, transferring most of the action to the lateral walls because of their much higher stiffness. In the transverse direction this effect attenuates because the flexibility of the roof diaphragm on the longer span leaves the central frames less restrained. And this shows how the behaviour of the structural system is governed together by the three parts: the three-dimensional frame of beams and columns, the box assembly of the perimeter walls and the roof diaphragm.

For the unsymmetrical structure, in the transverse direction along which the asymmetry exists for the condition of integrated panels, the torsional effects delete practically any restraint on the free side where the columns don't have relevant reduction of internal forces.

## 4 COCLUSIONS

What reported above gives the first indications for the necessary updating of the design criteria of the connection system of the cladding wall panels of precast structures. Introducing a proper margin with respect to the only example treated in the text (e.g. for higher seismic actions), one can indicate in ±1,75x7,0≈±12 cm the capacity of free slide of the panel fastenings for the isostatic solution of the connection system, one can indicate ±1,75x104≈±180 kN the force transmission capacity of the panel fastenings for the integrated solution. Reduction of the required strength can be made if joined with an additional system of connections between the panels.

ACKNOWLEDGMENTS

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